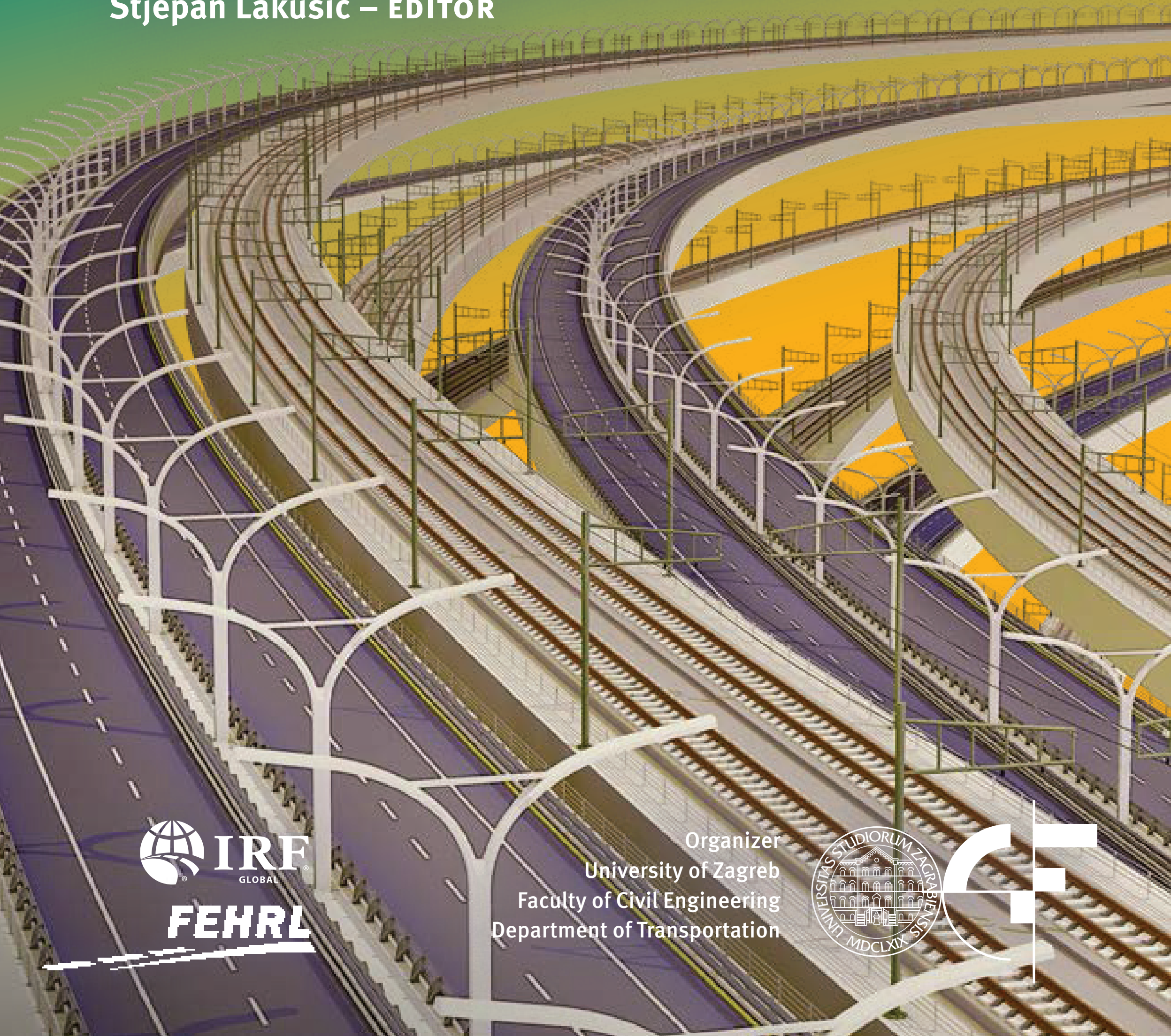
CETRA 2020*

6th International Conference on Road and Rail Infrastructure
20–21 May 2021, Zagreb, Croatia

Road and Rail Infrastructure VI

Stjepan Lakušić – EDITOR



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Road and Rail Infrastructure VI

EDITOR
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CETRA2020*

6th International Conference on Road and Rail Infrastructure
20–21 May 2021, Zagreb, Croatia

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FOREWORD

The 6th International Conference on Road and Rail Infrastructure – CETRA 2020* was organized by the University of Zagreb - Faculty of Civil Engineering, Department of Transportation Engineering. The Conference was held in Zagreb, capital of Croatia. Zagreb's history dates back to Roman times when the urban settlement of Andautonia existed at the location of the modern-day Ščitarjevo. In 1945, Zagreb was declared the capital of Croatia and today it is the cultural, scientific, economic, political and administrative centre of the Republic of Croatia, and a home to the Croatian Parliament, Government and President. It is located on the southern foothills of Medvednica Mountain and spreads along the banks of the Sava River. Culturally, it is a European city well worth visiting, with its numerous historical monuments, parks and medieval architecture. Everything is accessible by foot - from your hotel to the theatre, and for wandering around the old Upper Town or through the bustling streets of the more modern Lower Town, which has not lost an ounce of its charm despite the eternal march of time. The streets and monuments of Zagreb proudly testify to its hundreds of years of history.

The 1st International Conference on Road and Rail Infrastructure – CETRA 2010 was held on 17-18 May 2010 in Opatija. The 2nd International Conference on Road and Rail Infrastructure – CETRA 2012 was held on 7-9 May 2012 in Dubrovnik. The 3rd International Conference on Road and Rail Infrastructure – CETRA 2014 was held on 28-30 April 2014 in Split. The 4th International Conference on Road and Rail Infrastructure – CETRA 2016 was held on 23-25 May 2016 in Šibenik. The 5th International Conference on Road and Rail Infrastructure – CETRA 2018 was held on 17-19 May 2018 in Zadar. Great interest of participants in topics from the field of road and rail infrastructure, as expressed during previous CETRA conferences, confirms the adequacy of the Department for Transportation Engineering's decision to keep organising this international event. Positive comments given by participants in past conferences motivated the Department for Transportation Engineering of the Faculty of Civil Engineering at the University of Zagreb, to organise a new CETRA conference (CETRA 2020) on 20-21 May 2020 in Pula. However, due to the circumstances arising from the ongoing spread of COVID-19 - the continuing danger it still poses to public health and safety, together with an increase in travel restrictions - CETRA 2020 Organizing Committee has decided to further postpone the Conference. We held on for as long as we could, wishing that things would return to some semblance of normality. We were very optimistic, hoping that the situation with COVID-19 will be much better in October, trying our best to organize CETRA 2020 and to bring our professional and scientific community together one more time. However, the safety of the participants is our priority, and we decided it would be prudent to postpone the CETRA 2020 Conference to the spring of 2021. At the same time, postponing the Conference to the following year provided the members of our Committees valuable time to completely dedicate themselves to the determination of damage caused by the disastrous earthquake that hit Zagreb in March last year. Although we wished to organise the conference in 2020, even in the autumn of that year, we had to postpone the conference so as to be held in May 2021 on the same dates on which it was supposed to take place in 2020. We also partly kept the identity of the conference so that in 2021 the conference will be organized under the name of CETRA 2020*.

The CETRA conference has established itself as a venue where scientific and professional information from the field of road and rail infrastructure is exchanged. The idea on linking research organisations with economic sector has been the guiding concept for the realisation of this conference. Conferences of this kind are undoubtedly a proper place for establishing closer ties between the economy and university operators, and for facilitating communication and inspiring greater confidence, which might result in cooperation on new projects, especially those that contribute to greater competition. Lectures organized in the scope of the conference are based on interesting technical solutions and new knowledge from the field of transport infrastructure as gained on the projects already realised, projects currently at the planning stage, and those that are now being realized, in all parts of the world. In addition to presentations given by authors from the academic community, lectures are also presented by authors from engineering practice, the idea being to ensure the best possible synergy between the theory and practice. Because of great interest for the themes relating to the field of road and rail infrastructure, as shown during the past fourth conferences (CETRA 2010, CETRA 2012, CETRA 2014, CETRA 2016 and CETRA 2018), the Department for Transportation Engineering of the Faculty of Civil Engineering – Zagreb has assumed the responsibility to organise the new CETRA Conference in 2020 as well but, as already mentioned, the COVID-19 pandemic is the reason why the conference has been rescheduled for 2021 (but keeping the identity in the form of the name CETRA 2020*). However, due to the pandemic, the form in which the conference will be organised was also changed so that it will be held via an on-line platform.

This year, the 6th International Conference on Road and Rail Infrastructure – CETRA 2020* is organized with the intention of bringing together scientists and experts in the fields of road and railway engineering, so that they can present the results of their research, their findings and innovations, and analyse problems encountered in everyday engineering practice and, finally, offer solutions that will undoubtedly contribute to a more efficient planning, design, construction, and maintenance of transport infrastructure. The CETRA 2020* Conference serves as a platform for presenting a broad blend of scientific and technical papers in the fields of civil, transport, geotechnical, environmental, traffic and electrical engineering, with practical application in the road and rail infrastructure. Papers considered for publication are original papers that adequately contribute to the theory or practice of infrastructure engineering, and present either state-of-the-art work on topics related to infrastructure, or case studies in which theory is applied to solve significant infrastructure problems.

This year's CETRA Conference attracted a large number of papers and presentations from 32 countries. More than 140 papers were presented at the Conference and are contained in these proceedings **Road and Rail Infrastructure VI**. We believe that these CETRA 2020* proceedings will prove to be, just like the preceding five proceedings from the CETRA cycle, highly interesting and useful to all experts exhibiting a scientific and professional interest in road and rail infrastructure. The organizers of the Conference express their thanks to all Businesses and Institutions that provided support to this Conference. Special thanks are extended to the IRF - International Road Federation, and FEHRL – the Forum of European National Highway Research Laboratories, for their assistance and support in organizing very important conference sessions relating to innovations in roads maintenance and innovative transport infrastructure development. These operators have contributed, each in its own way, to the success of this conference. Great thanks are also extended to the following institutions that have supported the CETRA conference over the past ten years: University of Zagreb, Ministry of Sea, Transport, and Infrastructure, Ministry of Science and Education, and Croatian Academy of Engineering.

The Editor commends all authors for excellent papers contributed to these proceedings and wishes to thank members of the Organizing Committee and International Academic Scientific Committee, and numerous experts who participated in the review process. The gratitude is also extended to all participants for taking part in the CETRA 2020* Conference. The quality of the papers presented and the CETRA Conference is best demonstrated by the fact that a considerable interest is being expressed for most of these papers by researchers and industry operators from all parts of the world. This is not only due to the high visibility of the conference thanks to its presence in relevant databases, but is also a logical consequence of the quality of papers published in the scope of this conference series. Lectures that are organised at the conference are based on interesting technical solutions and latest findings in the field of transport infrastructure from the projects already realised, those that are at the design stage, or projects that are currently being realised in all parts of the world. In addition to representatives from the academic community, conference lectures are also given by industry operators, which constitutes the best possibly synergy of theoretical and practical achievements. Problems encountered in everyday engineering practice are analysed through papers presented at the conference, where practical solutions are offered in order to enable a more efficient planing, design, construction, and maintenance of transport infrastructure.

The organization of the CETRA 2020* Conference has proven to be a greater challenge compared to the organisation of the first CETRA 2010 Conference. The persistence of organisers and great perseverance of the authors who have accepted that their valuable scientific achievements and interesting professional projects are published not in 2020 but in 2021, i.e. in the year to which the conference has been rescheduled, are the proof that only by acting together we will be able to overcome challenges that inevitably occur in the society. High quality papers published in the Conference Proceedings are the result of great efforts of the authors and reviewers as they have worked in close synergy to achieve outstanding papers included in the proceedings and presented at the conference. All those who took part in the preparation of the proceedings (authors, reviewers, members of the Organizing Committee, technical editor, and the editor-in-chief) have worked hard to enable timely publication of the proceedings. We believe that the papers published in the proceedings will be interesting not only to our colleagues in the everyday engineering practice but also to students of technical faculties where disciplines from the field of road and rail infrastructure are studied.

Zagreb, May, 2021

THE EDITOR

Lakušić S.

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1

INFRASTRUCTURE AND TRAFFIC: PLANNING, (RE)CONSTRUCTION AND MANAGEMENT

FEATURES OF PUBLIC ROADS COST EVALATION

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Abstract

The evaluation of the property of road economy is a complex and important issue for the road sector. During carrying out the evaluation of the property, it is recommended to use regulations, provisions of national standards on property evaluation and regulations of the State Property Fund. Since the property of road economy can be attributed to a specialized type of property, which is the most useful in an integral property complex, then for its evaluation it is necessary to use cost approach methods. The article describes the methodical approaches and mechanisms of their implementation for evaluation of property of road economy. The basis of each of the methods is a set of evaluation procedures that will allow carrying out the evaluation. Practical approaches to property evaluation will promote a good coordination of the evaluation results and drawing up the reports and conclusions on their basis about evaluation of road economy property.

Keywords: inventory of property, cost evaluation methods, objects of evaluation, road assets, information and analytical system

1 Introduction

Reliable valuation of property of a road economy (realization of inventory of property) and conducting of its expert assessment in a proper manner is a very important issue in the road sector and is one of the mechanisms of road transport reform. Over the past few years, a review of legislation, evaluation methodology and standardization has led to a number of issues, which, when assessed and resolved, have helped to streamline, update and improve the legal and methodological framework for property evaluation. An important criterion in the work on the evaluation of property of the road economy was the discrepancy between the existing forms and models of evaluation, their adaptation and bringing in compliance with the evaluation model to the conditions that characterize this type of property and are necessary for carrying out the evaluation of the road economy objects.

Cost evaluation of the road is a process in which the determination of the cost of the road is carried out on the date of evaluation in accordance with the procedure established by regulations [1] on property evaluation. When evaluating the cost of the road, the following goals are achieved:

- establishing the availability of assets and their elements
- determination of the correspondence of their physical condition to the estimated value;
- determination of the real current value of road assets.

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The evaluation is carried out in accordance with the following procedure:
• definition of the objects and the purpose of evaluation;
• establishment of principles, bases and rules for assets evaluation;
• compilation of a list of assets and basic data for the calculation of the assets value;
• establishment of the nomenclature of quality indicators, analysis and selection of the most significant of them;
• compilation of the model of the qualitative condition of the evaluation object;
• initial evaluation of assets;
• determination of the level of the quality of the evaluation object;
• determination of the cost of a road asset depending on the purpose of evaluation;
• compilation of the cost valuation report.

Estimating the cost of property will facilitate the inventory of the road economy property. The purpose of the inventory is to obtain data on the availability and condition of property of entities under the control of the balance-holders of roads, as well as the creation of conditions for the organization of an information system for the operational recording of the availability, condition and use of the property. According to the results of the inventory, the existing passport of the road is also adjusted or the new passport is issed. The purpose of the inventory is to establish common principles for the preparation of technical documentation for real estate objects of the road economy in order to register the rights to real estate.

2 Evaluation methods

2.1 The process of roads evaluation

The objects of the road economy, which are subject to inventory, are distributed as follows:
• motor roads - a linear complex of engineering structures designed for the continuous, safe and convenient traffic which includes the objects located in the right-of-way of the roads within a single land plot and designed for transportation and ensuring the functioning of road transport infrastructure;
• artificial structures - engineering structures intended for the movement of vehicles and pedestrians through natural and other obstacles, as well as for the sustainable functioning of the motor road (bridges, overpasses, flyovers, viaducts, tunnels, surface and underground pedestrian crossings, pontoon bridges and ferry crossings, road junctions, retaining walls, galleries, catching ramps, snow protection structures, avalanche and mudflow protection structures, etc.);
• road drainage structures - structures intended for removal of surface and groundwater from the the subgrade and carriageway (side ditches, drainage ditches, hillside ditches, culverts, open and closed drainage systems, storm sewerage, etc.);
• technical means - special technical means intended for management and control of road traffic (road signs, information boards, road markings, signal bollards, traffic and pedestrian barriers of different types, traffic light equipment, etc.);
• real estate necessary for the operation of motor roads.

Schematically, the process of road cost evaluation is shown in Figure 1.

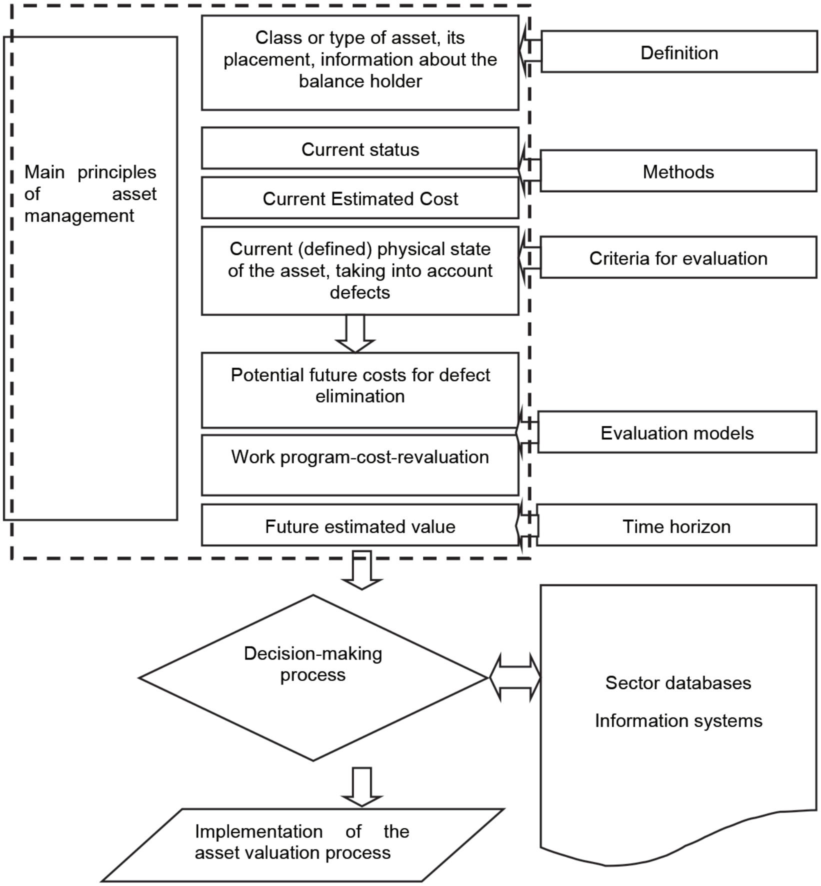

Figure 1 Flowchart of the process of evaluating the road asset

2.2 Cost approach

The effectiveness of assessing the property of a road economy depends on the methodological approaches under which it will be implemented. Building a road assessment model allows for a uniform methodological approach to the overall cost evaluation of roads, both currently and in future or projected periods. The following methods are used in accordance with [2] (Figure 2), which are used to carry out a cost evaluation of public roads:

- Asset re-evaluation / after-evaluation method;
- limit cost method;
- fixed cost method with respect to the limit state;
- method of the transferred cost (replacement).

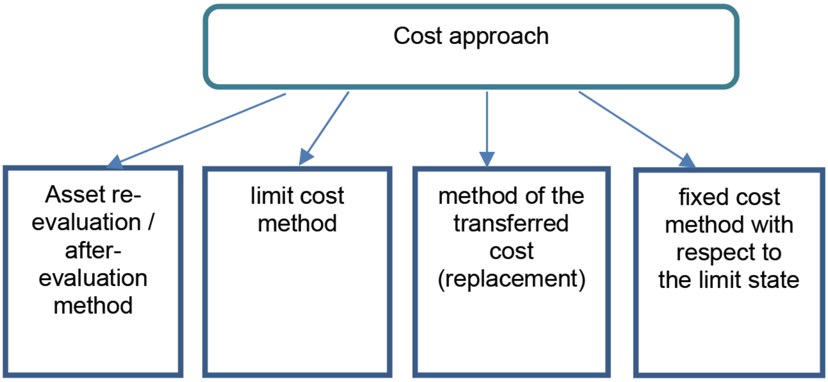

Figure 2 Methods based on cost approach

The method of re-evaluation / after-revaluation of an asset is that the asset is calculated as the product of its initial (initial) cost of construction and the ratio of the qualitative state of the asset or its element adjusted to the inflation index. Condition is the ratio of the current qualitative state of the asset or its element to a better condition. The value of assets in year t is determined according to [2] by the formula (1):

$$V_{r.t} = H \cdot C \cdot \left(\frac{K_{kt}}{K_{k\,best}} \right) \tag{1}$$

where

$V_{r.t}$ - baseline value of assets by re-valuation / after-valuation method in year t, monetary units;

HC - initial (actual) cost of construction in accordance with the consolidated estimates, monetary units;

$K_{k\,t}$ - the level of a qualitative state of the asset at time t, in the form of a coefficient or %;

$K_{k\,best}$ - the best level of quality status of an asset, recorded during its life cycle, in the form of a coefficient or %.

In accordance with [2], the estimated value of a road asset using the re-evaluation / after-evaluation method at the valuation date is obtained by adjusting the base cost for inflation by the formula (2):

$$V_{r.r.t} = V_{rt} \cdot \frac{CPI_t}{CPI_0} \tag{2}$$

where

$V_{r.r.t}$ - estimated value of the road asset by the re-evaluation / after-valuation method in year t, monetary units;

CPI_t - index of construction price in year t;

CPI_0 - index of construction price in the year when the object was built.

The limit cost method uses current and past data to determine the value of assets. According to [2], the following formula (3) is used to calculate the value of assets:

$$V_{m.c.t} = HC \cdot \left(\frac{K_{kt} - K_{k\,worst}}{K_{k\,best} - K_{k\,worst}} \right) \tag{3}$$

where

$V_{m.c.t}$ - baseline value of assets by the limit cost method in year t, monetary units;

$K_{k\ worst}$ - the worst level of a qualitative state of an asset, recorded during its life cycle, in the form of a coefficient or %.

According to [2], the estimated value of a road asset by the limit cost method on the evaluation date is obtained by adjusting the base price for inflation by the formula (4):

$$V_{r.m.t} = V_{m.c.t} \cdot \frac{CPI_t}{CPI_0} \tag{4}$$

where

$V_{r.m.t}$ - estimated value of the road asset by the limit cost method in year t, monetary units.

The fixed cost method with respect to the limit state is to bring the state of an asset or its element to a level that consistently exceeds the minimum performance threshold established for that asset. The cost of assets is a constant value over the life of assets until the quality level of the asset exceeds a certain limit value and is equal to zero; when the quality level falls below the established limit value (in this case, the issue is about the renewal of the asset or its liquidation). Each year, the calculation of the state of the asset at the time t ($K_{k\,t}$) is carried out. The value of wear (physical and / or functional) K_{wear} of the rod section in accordance with [2] is determined by the formula (5):

$$K_{wear} = 1 - K_{k\,t} \tag{5}$$

Experts of the road organization set the limit value $K_{k\ m.t.}$

When $K_{k\,t} > K_{k\ m.t,}$ the estimated value of the road asset is determined on the basis of the replacement (or reproduction) cost adjustment by the estimate as of the date of the assessment for the inflation index. In accordance with [2], the estimated cost of a road asset by the fixed cost method with respect to the limit state and according to formula (5) is determined by the formula (6):

$$V_{r.f.t} = RC_t \cdot K_{k\,t} = RC_t \cdot \left(1 - K_{wear}\right) = RC_t - RRC_{k\,t} \tag{6}$$

where

$V_{r.f.t}$ - the estimated value of the road asset by the fixed cost method in relation to the limit state in year t, the monetary units;

RC_t - cost of replacement (or reproduction) of the asset in year t, monetary units;

$RRC_{k\,t}$ - the cost of repair and restoration of the asset in year t,

$RRC_{k\,t}$ - $RC_t \cdot K_{wear}$ - monetary units.

In accordance with [2] the cost of replacement (or reproduction) of an asset is determined by the formula (7):

$$RC_t = \sum_{i=1}^{n} C_i \tag{7}$$

where

C_i - the estimated value of the i-th element of the road section, which is displayed in current prices on the actual date of assessment, using the same architectural solutions, building structures and materials, as well as the same quality of construction and installation works as the assessed object (replacement value) or using modern materials and in accordance with new standards and planning solutions (replacement value).

n - number of elements of the road section.

The final estimated cost of the road asset is determined by formula (8):

$$V_{r.a.} = V_1 + V_r \tag{8}$$

where

$V_{r.a.}$ - estimated cost of the road section, monetary units;

V_1 - cost of the road section, monetary units;

V_r - the estimated cost of a road asset determined by one of the methods shown in Figure 2, monetary units

To determine the quality of the evaluation object, the following initial data are required, which contain:

- the initial cost of construction, where the actual cost of the road construction is entered in accordance with the consolidated cost estimate;
- the quality level of the evaluation object at the time when the best condition is equal to 100 percent;
- the best condition of the asset is the condition of the evaluation object recorded after the commissioning of the construction object;
- the worst quality level of the asset (determined quality level of the construction object during which the object had the worst operational parameters);
- construction price index at the year when the object was built;
- construction price index at the year when the quality level assessment is carried out;
- the cost of replacing (or reproducing) the asset at the year when the object is evaluated;
- the limit value of the quality level of the asset which is set by experts of the road organization.

2.3 The information-analytical system

The analysis of the methods used to evaluate the property of the road economy allowed to develop an information and analytical system, the algorithms of which are convenient to use and provide automation of calculations of the valuation of roads of general use. The information-analytical system is a program with a definite structure, interface and parameters that provide an opportunity:

- the choice of the necessary method for carrying out the valuation of roads of general use;
- implementation of algorithms for conducting valuation of road assets;
- access to the database necessary for the settlement;
- centralized storage of collected data for their further use by organizations for assessing the property of the road economy;
- filling the database with highways for which it is necessary to carry out a valuation;
- access to the normative reference base of the coefficients required for the settlement;
- conducting an analysis of the results of the valuation of public roads in order to plan and make effective decisions.

The final result of a valuation is the formation of a property valuation report, which, according to [3], should contain the following information:

- a description of the object of evaluation that allows it to be identified;
- the date of the assessment and the date of completion of the report, and, if necessary, the validity period of the report and the conclusion on the value of the property in accordance with the requirements of the legislation;
- the purpose of the assessment and justification of the choice of the appropriate assessment base;
- list of normative legal acts according to which the assessment is carried out;
- a list of restrictions on the application of evaluation results;
- an outline of all assumptions within which the evaluation was conducted;
- description and analysis of the collected and used source data and other information during the evaluation;
- conclusions on the analysis of the existing use and the most effective use of the object of evaluation;
- an outline of the content of applied methodological approaches, methods and valuation procedures, as well as corresponding calculations, with which the conclusion on the value of the property is prepared;
- a written statement by the appraiser on the quality of the source data used and other information, a personal review of the object of evaluation (in the case of impossibility of personal review - relevant explanations and justification of reservations and assumptions regarding the use of evaluation results), observance of national standards for valuation of property and other regulatory acts on valuation of property during its carrying out, other statements which are important for confirmation of authenticity and objectivity of property valuation and conclusion about its value;
- conclusion about the value of the property;
- attachments with copies of all source data, as well as, if necessary, other information sources that explain and confirm the assumptions and calculations.

3 Conclusions

Carrying out a cost evaluation of public roads is important because it solves a number of important issues that are necessary for making reasonable decisions in the road property management process. These include: establishing the availability of assets, determining the conformity of their physical condition to the estimated cost, determining the real current cost of assets. Cost evaluation methods allow carrying out a cost evaluation of road property, taking into account its specific features. A practical approach to solving the issue of property cost evaluation is the basis for a comprehensive inventory and cost evaluation of public roads, which will allow further using it as one of the key mechanisms for understanding the financial condition of the road sector. According to the calculation results, it is possible to make conclusions on the property condition, which will facilitate the adoption of reasonable management decisions on the further use of road assets.

References

[1] On land evaluation: Law of Ukraine of 11.12.2003 N 1378-IV, Legislation Database of Ukraine, Verkhovna Rada of Ukraine, https://zakon.rada.gov.ua/laws/show/1378-15/ed20031211, 15.10.2018.

[2] MR D 1.2-37641918-884: 2017 Guidelines for cost evaluation of highways and facilities on them, Kyiv, 2017, information and documentation

[3] National Standard № 1, General principles of property evaluation and property rights: Resolution of the Cabinet of Ministers of Ukraine of 10.09.2003 N 1440, Legislation Database of Ukraine, Cabinet of Ministers of Ukraine, https://zakon.rada.gov.ua/laws/show/1440-2003-p, 15.10.2018.

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CROATIAN ROAD SECTOR MANAGEMENT CHALLENGES

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Abstract

The road network in the Republic of Croatia is well developed and largely responds to traffic needs. The motorway network is largely built up and no major new investment is needed in the short term. The national road network is in good standing according to national and EU standards. However, public road management companies face operational and financial challenges in terms of: (a) overinvestment in the network; (b) weak governance; (c) high operating costs; (d) large debt stock; (e) short tenor of existing loans; (f) currency risk and (g) insufficient credit strength to access the loan market for long tenors on a stand-alone basis. The Government of the Republic of Croatia has therefore decided to address these chellenges and launched a project funded by IBRD called the Modernization and Restructuring of the Road Sector (MARS) aiming to enhance operational efficiency and improve the financial sustainability of the road sector. To these ends, the Government has approved a Sector Policy Letter, which contains a set of planned reforms. To ensure the contribution of the road sector to the overall economy, in addition to the financial sustainability of the sector itself, operational improvements are needed in the following key areas: (a) management of the road infrastructure sector; (b) planning, financing and implementation of investments in the road sector; (c) corporate governance and business operations. Much of the reform has already been implemented, but some of the most important are still in the process of being implemented.

Keywords: road sector, road management companies, restructuring

1 Introduction

In general, the road network in the Republic of Croatia is well developed and mostly responds to traffic needs. The motorway network in Croatia is largely built up compared to the country's initial plans, and no significant new investment is needed in the short term. The national road network is also comprehensive and in good standing according to national and EU standards. The modal split of inland public transport indicates that 73 percent of freight and 71 percent of passengers are carried by road. When added to individual transport, more than 90 percent of travel is by road. It should be emphasized that the road infrastructure is the most valuable public good.
Companies operating the road network, Croatian Motorways (HAC), Rijeka-Zagreb Motorway (ARZ) and Croatian Roads (HC), face operational and financial challenges such as network over-investment, under-management, high operating costs, high indebtedness, short maturities of existing loans, currency risk and insufficient creditworthiness to access credit markets for long-term loans on a stand-alone basis. The Government of the Republic of Croatia

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has therefore decided to introduce reforms in the road sector, strengthen supervision and planning within the sector, increase the operational efficiency of companies and improve the financial image of the sector. In order to ensure the contribution of the road sector to the overall economy, in addition to the financial sustainability of the sector itself, operational improvements are needed in the following key areas:
a) Management of the road infrastructure sector,
b) Planning, financing and implementation of investments in the road sector,
c) Corporate governance and business operations.

The government has sought the support of the International Bank for Reconstruction and Development (IBRD) and has made excellent cooperation with the IBRD and other International Financing Institutions (IFIs) in identifying elements of the road sector restructuring. The Government also consulted the European Commission (Directorate General for Competition). As a result of this co-operation, the Modernization and Restructuring of the Road Sector (MARS) project has been defined [1], which aims to support the Croatian Government in enhancing operational efficiency and improving the financial sustainability of the road sector. The project is funded by IBRD in the form of an investment loan and guarantees. Part of the investment loan is used to restructure public road management companies, i.e. HC, HAC and ARZ. For the purpose of operationalizing the MARS project, the Government on 16 March 2017 prepared and approved a Letter of Sector Policy [2], which contains a set of reforms to put the sector on a financially sustainable path and reduce the need for state support.

2 Main elements of business and financial restructuring of the road sector

The Letter of Sector Policy [2] covers the following elements:

a) Road sector management
- The Transport Development Strategy of the Republic of Croatia for the period 2017 to 2030
- Unique system of categorization and standards of maintenance / management of public roads to ensure proper and cost effective maintenance of the property
- Strengthening the role of the Ministry of the Sea, Transport and Infrastructure as the body responsible for planning and controlling the operation of publicly traded companies in the road sector
- Reorganization of the companies in the road sector
- Debt optimization strategy of HAC, ARZ and HC.

b) Planning, financing and implementation of investments in the road sector
- Road Property Management System (RPMS) for the purpose of ensuring the accuracy of investment planning
- Approved road sector investment plan for the period 2017-2020.
- Revenue and toll collection
- Road safety.

c) Management of companies and their operations
- Applying principles of corporate governance
- Human resources management instruments
- More efficient maintenance system.

In order to achieve the above objectives, deadlines for specific activities are defined in the table annexed to the Sector Policy Letter.

3 Road sector management

The Government adopted the Transport Development Strategy of the Republic of Croatia for the period from 2017 to 2030. The strategy gives the vision, main goals, long-term investment plans and principles of business of the road sector and other modes of transport. The main guidelines of the Strategy are in line with the European Commission. The strategy was adopted following the implementation of the Strategic Environmental Assessment and public hearings during the period of June-July 2017. The strategy is also a requirement of the European Commission for accessing sectoral funding under Operational Program 2014-2020.

The government on 24 March 2017 adopted a Decision on the implementation of technical categorization of public roads in the Republic of Croatia. The purpose of the Decision is to establish, on the basis of unique criteria, clear functional road categories, independent of current administrative and geographical frameworks, to allow the application of maintenance and construction standards. Roads with a certain traffic load and functional importance within the system receive the same level of quality of maintenance no matter who manages the road.

Road maintenance standards have, over time, become too extensive and have been interpreted differently by contractors. Therefore, it was decided to develop standards that will regulate the regular and periodic maintenance of all state, county and local roads, regardless of the way of contracting maintenance works. Standards should include winter maintenance, but would not include rehabilitation works. The standards should be defined in such a way as to ensure a satisfactory level of road maintenance, similar to that applicable to other road networks in the EU. The standards will contribute to improving quality, rationalizing unit prices and promoting better competition. The standards would refer to the Technical Specifications of Maintenance Works, which have not yet been drafted. The terms of reference for the development of maintenance standards have been prepared, but the procurement process has not yet been initiated.

The Public Roads Network is being developed based on the following planning documents:

a) The Transport Development Strategy of the Republic of Croatia for the period 2017-2030 sets long-term development priorities. It deviates from its previous focus on infrastructure development and moves on to transport planning with a focus on multimodality, safe and sustainable transport services, improving regional links and completing the Croatian part of the Trans-European Corridor Network (TEN-T).
b) The medium-term objectives are set out in a four-year Maintenance Program, adopted by the Government on the basis of a proposal from the Ministry of the Sea, Transport and Infrastructure.
c) Annual construction and maintenance plans are defined by the companies operating the road network with the approval of the Ministry.

In order to meet the requirements of sustainability and future development of the road network, the Ministry will propose to the Government of the Republic of Croatia long-term strategic goals of the road sector, as well as a long-term coordinated plan of these goals, to ensure the coherence of the companies investment plans. To this end, the Ministry had to set up a separate Department and ensure that the medium-term plans of the companies operating the road network were aligned with the work of the Department. The department will also coordinate investment plans, improve control of public investment management by applying high principles of public investment management, and will control the effectiveness of project management through beneficiaries. Consequently, the Ministry, by Decree of 24 November 2017 decided to set up a Strategic Planning Department.

The efficiency of the State as the owner and contracting authority of the road sector companies can be improved by establishing a monitoring system and defining criteria for business and financial performance of companies. For this purpose it has been planned to improve the accounting and financial management instruments used by the sector companies. This includes activities to strengthen the ownership role within the Ministry, define key performance indicators (KPIs) to determine the level of business performance of the companies, and establish monitoring of companies business by the Ministry. These elements will also be referred to in the so-called. “Performance Agreements” to be signed by the Ministry with HAC and HC, the implementation of which will make publicly available and transparent reporting.
Following an analysis of the results of the Business and Financial Restructuring Study funded by the European Bank for Reconstruction and Development (EBRD), the Government decided in 2017 to carry out the merger of HAC and HAC-ONC companies and the transfer of ARZ’s operations to the HAC through delegation of responsibilities. The merger of these companies optimizes business processes, ensures economies of scale and reduces overall costs. Along with the implementation of the merger, the legal changes have defined the monopolistic position of new companies in the area of their individual motorway networks. Amendments to the law have defined, among other things, the principles of financial management to ensure that bookkeeping assets are carried on a business-to-business basis. The expected end result of these measures will be a lower operational cost of maintaining roads for a given level of service and a significant streamlining of the number of jobs. The rationalization of the number of employees and the optimization of business processes are carried out according to the developed guidelines of the said Study. The rationalization began in 2018 and should be completed by the end of 2020.
Based on a project proposal prepared by the Ministry and through a public tender, a consultant was selected to work on the restructuring of Croatian Roads. The analysis of the situation and the proposal for restructuring are expected to be submitted to the Ministry at the beginning 2020.
In 2017, the Ministry of the Sea, Transport and Infrastructure, through an international public procurement, selected and signed a contract with a financial advisor with the aim of: (i) endorsing and improving the strategic financial guidelines adopted in 2016, (ii) defining debt financing guidelines in line with strategy (iii) proposing a road sector financing strategy and structure for approval, and (iv) helping to implement optimization of the debt of the road sector companies. The objective of this strategy is to optimize the debt guaranteed by the Republic of Croatia while facilitating long-term financing of the road sector.

4 Planning, financing and implementation of investments in the road sector

The Road Property Management System (RPMS) for motorways, state and county roads will be developed through a contract that, after collecting data on property and condition of property, should be operationally implemented in the coming period. The RPMS will use all existing elements of the road asset management company. The RPMS will ensure the availability of up-to-date network status information and serve as a basis for the continuous updating of estimated periodic maintenance plans for highways and state roads. The RPMS will also be available for county-managed roads to enter property and condition information and to adopt its own future maintenance programs. The reconstruction and emergency maintenance of roads and highways will be programmed in this approach based on the actual state of the roads, not according to predefined standards.
The Ministry has proposed, and the Government has adopted, the Public Roads Construction and Maintenance Program for the period 2017-2020. Its main features are: (i) financing of periodic maintenance by each company on its own resources, without borrowing; (ii) the restric-

tion of construction in the motorway sector to ongoing projects and investments financed by EU funds; (iii) limitation of HC construction to already started projects and investments financed by EU funds, supplemented by projects selected on the basis of agreed criteria within the specified funding limit. New debt will be repaid from the state budget.
The Ministry of Finance has achieved significant success in refinancing the debts of companies created by financing the construction of the motorway network, owing to the favorable situation on the financial market, but also to the improved credit rating of the Republic of Croatia.
In addition to the 2017-2020 plan, companies, with the approval of the Ministry of the Sea, Transport and Infrastructure and the Ministry of Finance, prepared an annual indicative investment plan using justified economic and social criteria to evaluate and prioritize all proposed new investments, in accordance with the defined multicriteria framework. Future investment plans will also include the costs of emergency, investment maintenance and renovation under the RPMS by March 2020.
HC and HAC generate revenues from fuel excise taxes. HC uses its share of excise taxes for maintenance and financing of investments, while HAC uses its share of excise taxes exclusively for investments. These funds will continue to be available, taking into account the needs of these two companies, at the level set by the Roads Act.
A seasonal increase in toll prices of 10 % was introduced between 01 July and 30 September for HAC and ARZ motorways starting in 2017 onwards. Further changes, such as the reduction of the Electronic Toll Payment (ENC) discount, will be introduced gradually, in order to comply with EU standards. Additional sources of revenue for the road sector will also be analyzed.
A fully automated electronic toll collection system has been defined in accordance with the EBRD Study. Tender documentation has been prepared and a consultant selected for the new toll collection system with the aim of putting the system into operation by 2020/2021. The future automated electronic toll collection system will retain the basic distance-based toll collection principles, which are mainly applicable to the current toll collection system, but will reduce the toll collection and traffic jams, especially during the summer, and improve environmental solutions. Toll rates will be considered at least once a year.
Implementation of road safety and road safety initiatives in line with EU Directives for the Croatian part of the Trans-European Road Network (TEN-T) is an integral part of road sector plans and EU funding is already being used. The goals of the national road safety plan 2011-2020 are being realized by the action plan adopted by the Ministry. The Action Plan includes various measures for highways and state roads such as upgrading guardrails, controlling axle loads and removing black spots. Common activities on the road network include road safety audits and inspections, improvement of tunnel safety and the extension of traffic management information systems at the level of the Republic of Croatia. Audits and inspections will be carried out to cover the entire TEN-T network by the end of 2020.

5 Management of companies and their operations

Corporate governance depends primarily on a balanced division of roles and implementation of internal and external planning, control, management and decision-making mechanisms. This ensures a balanced relationship between the owner, the supervisory board and the management of the company. This results in effective governance mechanisms and addresses the problems and potential conflicts that arise between management structures.
Corporate governance plans based on the principles of the Organization for Economic Co-operation and Development (OECD) should be adopted by all companies in the road sector. The plans will be in line with the existing Decision on the adoption of the Corporate Governance Code for companies in which the Republic of Croatia holds shares or interests (Official Gazette 112/2010). Corporate governance plans include the application of contracts with com-

pany management based on operating results, as well as increased transparency through more frequent disclosure of decisions, financial data and performance and annual monitoring of their implementation. Corporate governance plans relating to boards of directors will also include defining and implementing the following elements: structure and management, supervisory committees, transparency and disclosure of information, audits and internal controls, remuneration of supervisory board members, risk management and corporate social responsibility.

Adoption of corporate plans according to the principles outlined above by road network companies has been started but not yet completed.

Human resource management is a key element in the management of the road sector. To this end, modern human resources and work management systems are being introduced, including payroll policies, and systems for controlling work, planning and job evaluation. In the context of business and financial restructuring of the road sector, job cuts and / or retraining will be done in an efficient and socially acceptable manner.

The restructuring of the companies is expected to bring about a more efficient and cost-effective maintenance of the motorway network. In other parts of the road network, public road maintenance and traffic management will be organized to optimize operating costs and the number of jobs. Further steps in operational improvement will be defined taking into account, inter alia, experience in a pilot project in Istria (performance based contract) and maintenance standards and specifications.

HC and County Roads Administration (ŽUC) continue their cooperation with the aim of developing technical road categorization and maintenance standards applicable throughout the Republic of Croatia. The agreement on the terms of the joint procurement of maintenance services defines the basic principles and conditions for the joint procurement of maintenance services in such a way that HC and ŽUC remain responsible for their individual maintenance contracts while the procedure will be carried out at the level of regional business units within an independent joint procurement committee which includes HC and ŽUC representatives. This increases competition in the market and opens opportunities for local contractors.

6 Conclusion

As a result of the cooperation of the Ministry of the Sea, Transport and Infrastructure with the International Bank for Reconstruction and Development, the Road Sector Modernization and Restructuring Project (MARS) has been initiated and is aimed at supporting the Croatian Government in enhancing operational efficiency and improving the financial sustainability of the road sector. The Ministry approved a Sector Policy Letter defining the activities of the MARS project, as well as the deadlines for their implementation. Much of these activities have already been implemented, but some significant activities are only in the initial stages. The restructuring project has so far yielded the best results in refinancing companies' debts.

References

[1] Project Appraisal Document, http://documents.worldbank.org/curated/en/467961493604154334/Croatia-Modernization-and-Restructuring-of-the-Roads-Sector-Project

[2] Pismo sektorske politike usvojeno Odlukom Vlade 16. ožujka 2017.

APPLICATION OF FIDIC GENERAL CONDITIONS IN TRANSPORT INFRASTRUCTURE PROJECTS IN CROATIA

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Abstract

Infrastructure is a broad term encompassing a wide range of facilities from roads and railway lines to wind, waste and water projects, oil and gas facilities, pipelines and processing plants. Whilst infrastructure construction contracts have key provisions in common, there is no one standard form that fits all projects. Any standard form will need to be tailored to suit the demands of the projects, the risk profile of the parties and comply with the legal jurisdiction of the contract and project location. Increasingly contractors are enhancing their expertise and working on infrastructure projects internationally. As a result, the forms of construction contracts used are becoming more standardized. Nowadays, FIDIC forms of contract are intended to be suitable for projects carried out around the world by all types of employers with the extensive support of large investors such as the World Bank or the European Union. The terms of the Conditions of Contract for Construction have been prepared by the Fédération Internationale des Ingénieurs-Conseils (FIDIC). Two most frequently used FIDIC models of construction contracts are Conditions of Contract for Construction (known as Red Book) and Conditions of Contract for Plant and Design-Build (known as Yellow Book). These general conditions are also used as contract conditions in Croatia for public procurement of transport infrastructure projects. The use of FIDIC General Conditions of Contract in the realization of transport infrastructure construction works in Croatia is presented in the paper.

Keywords: transport infrastructure projects, construction contracts, FIDIC, FIDIC General Conditions

1 Introduction

FIDIC is a French language acronym for Fédération Internationale des Ingénieurs-Conseils, which means the International Federation of Consulting Engineers. It was started in 1913 by the trio of France, Belgium and Switzerland. FIDIC is now headquartered in Switzerland and boasts of membership from over 100 different countries. [1]
Over the years, FIDIC has become famous for its secondary activity of producing standard form contracts for the construction and engineering industry. FIDIC published in 1957 its first contract, titled *The Form of Contract for Works of Civil Engineering construction*. As the title indicated, this first contract was aimed at the civil engineering sector and it soon became known for the color of its cover, and thus, *the Red Book*. It has become the tradition that FIDIC contracts are known in popular parlance by the color of their cover. [1] The first edition of FIDIC Contract Conditions for Civil Works was compiled on the basis of the general contract

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conditions of the British Institute of Civil Engineers (ICE), which were in their initial form drawn up at the end of the 19th century. Consequently, the first sets of FIDIC contracts were based on English law principles. Since 1957, future FIDIC contracts have successfully incorporated the principles of other legal systems especially the European civil law system. This development has been particularly emphasized in recent editions of 1999 as well as of 2017. However, the basic framework of English law principles has survived.
Today, more than 100 national professional associations are members of FIDIC. Only one national association in a country can be a member of FIDIC. In order for a national association to become a member of FIDIC, it must be confirmed that its members adhere to the ethical and moral principles inherent in consulting engineers, and in particular to act in accordance with the principles of [8]:

a) retaining absolute independence in the relation to contractors, manufacturers and suppliers, and that they never receive services that might interfere with their objectivity;
b) to act solely in the interest of the contracting authority that has engaged them;
c) be experts in their profession, enabling them to perform their task conscientiously.

Because of the broad support it enjoys, FIDIC contracts are the foremost contracts in international construction. FIDIC Conditions of Contract are recommended for general use in works subject to international tender, but they are also suitable for use in national tenders. The FIDIC Conditions of Contract only apply if the parties have expressly included them in their contract. This is most often done in such a way that the entire contract is based on the terms of the FIDIC Conditions of Contract. The main conditions of contract consist of two parts: General Conditions and Particular Conditions. The contracting parties may modify the General Conditions, but if they wish to do so, they should do so in the Particular Conditions with the markings on those provisions of the General Conditions that the parties wish to change. While preparing a contract, some clauses of General Conditions shall be implemented generally and some should be modified depending on the features of the work and location. General Conditions of Contracts and the Particular Conditions of Contracts are linked to each other by numbering the clauses mutually, thus the first and second part together create the Conditions of Contract. [3] The contracting parties are therefore free to change the General Conditions of FIDIC as they wish, and these contract conditions only apply if they are expressly made an integral part of their contract by the parties.
Each FIDIC book also contains Guidance for Preparation of Particular Conditions, which sought to assist the compilers of a contract in the preparation of the Particular Conditions. This assistance consists in the fact that FIDIC has already made the selection and prepared the provisions that are expected to be amended, and has already prepared the appropriate formulations for some of these changes.

2 FIDIC Rainbow Suite

Over the years FIDIC has consistently improved its General Conditions of Contract adding new forms replacing previous ones and updating important terms. In 1999, FIDIC published a completely new suite of contracts, (known as “Rainbow Suite” for a variety of their cover colors) which includes 10 different books [6]. These include:

- *The Red Book*: Conditions of Contract for Construction for Building and Engineering Works designed by the Employer (1st Ed 1999).
- *The Pink Book*: Harmonized Red Book (MDB Edition) Conditions of Contract for Construction for Building and Engineering Works designed by the Employer (3rd Ed 2010) - for use as part of the standard bidding documents by the Multilateral Development Banks only.
- *The Yellow Book*: Conditions of Contract for Plant and Design-Build - for electrical and mechanical plant, and for building works, designed by the Contractor (1st Ed 1999).

- *The Silver Book*: Conditions of Contract for EPC/Turnkey Projects (1st Ed 1999).
- *The Orange Book*: Conditions of Contract for Design - Build and Turnkey (1st Ed 1995).
- *The Gold Book*: DBO Contract - Conditions for Design, Build and Operate Projects (1st Ed 2008).
- *The Green Book*: Short form of Contract (1st Ed 1999).
- *The White Book*: Client/Consultant Model Services Agreement (4th Ed 2006)
- *The Blue-Green Book*: Dredgers Contract (1st Ed 2006)
- *Conditions of Subcontract for Construction*: Used in conjunction with the Red Book and The Pink Book (1st Ed 2009)

The criterion for distinguishing between different issues of the FIDIC Condition of Contract is based on the risk sharing between the employer and the contractor, and this division depends primarily on which of the two parties provides the design under which the works will be performed and what additional obligations the contractor assumes.
Unlike previous editions of the FIDIC contracts, which focused on the nature of the works (for example, civil engineering or electrical and mechanical works), the new 1999 generation of FIDIC contracts focuses on the relationship between the parties. The choice of form was changed, to be based on which party was doing the design and the procurement method used, with less emphasis on the type of work being undertaken [7]. The contract structure of all books, except the Green and White book, is generally the same [6]:

- General provisions (Clause 1)
- The Employer, Employer's administration or Engineer, Contractor, Nominated subcontractors, or Design (Clauses 2-5)
- Staff and labor, Plant, materials and workmanship (Clauses 6-7)
- Commencement, delays and suspension, Tests on completion, Employer's taking over, Defects Liability, Tests after completion (Clauses 8-11/12)
- Measurement and evaluation or Variations and adjustments, Contract price and Payment (Clauses 12-14)
- Termination by Employer, Suspension and Termination by Contractor (Clauses 15-16)
- Risk and responsibility (Clause 17)
- Insurance (Clause 18)
- Force majeure (Clause 19)
- Claims, disputes and arbitration (Clause 20)

Besides the Pink Book, which has to be used in all projects funded by the international financial institutions, the Red, Yellow and Silver books have the greatest application in practice.
The Red Book i.e. Conditions of Contract for Construction, are recommended for building or engineering works designed by the employer or by his representative, the engineer. The contractor constructs the works in accordance with a design provided by the employer. However, the works may include some elements of contractor-designed civil, mechanical, electrical and/or construction works. The Yellow Book, i.e. Conditions of Contract for Plant and Design-Build, are recommended for the provision of electrical and/or mechanical plant, and for the design and execution of building or engineering works. The contractor designs and provides, in accordance with the employer's requirements, plant and/or other works, which may include any combination of civil, mechanical, electrical and/or construction works. The Silver Book, i.e. Conditions of Contract for EPC/Turnkey Projects, are recommended for the provision on a turnkey basis of a process or power plant, and which may also be used where one entity takes total responsibility for the design and execution of infrastructure project, which involves little or no work underground. Under the usual arrangements for this type of contract, the contractor carries out all the engineering, procurement and construction ("EPC"), providing a fully-equipped facility, ready for operation (at the "turn of the key") [11].

All three of these books are formed in the same way so that they contain the same constituent parts. Many of the provisions of these three Contract Conditions are identical, which greatly facilitates their use, so that those who are familiar with only one of these General Conditions are familiar with many of the provisions that are in General Conditions of other two books. The employer and the contractor shall choose those general conditions which are appropriate to the extent of their assumed responsibility. However, in the most sensitive area of the contract, the division of responsibilities between the employer and the contractor, the Silver Book has made quite a radical change over the Red and Yellow Books. In this context it should be noted that in construction practice there is no one universally accepted definition of the term "turn-key". However, it is undisputed that the term encompasses "total contractor responsibility for the project", and that the term typically includes project design, construction, installation of equipment, and components within the scope defined by the contract.
Most recently, in 2017 FIDIC published the second edition of the FIDIC 1999 suite. However, only the Red, Yellow and Silver Books were published by FIDIC so far. In order to be used in Croatia these books have to be translated into Croatian language.

3 FIDIC Conditions of Contract and Croatian Law

The Conditions of Contract prepared by FIDIC form only part of the contract. It is intended and expected that the contract will be formed of many other documents. However, the Parties' rights, obligations and liabilities under the contract will also depend on the law governing the contract and other laws that might apply to the parties' performance of their obligations. From the point of view of the application of Croatian legislation, when FIDIC General Conditions of Contract are an integral part of a contract to which Croatian law applies, then the relevant provisions of the Civil Obligations Act [2] also apply to such contractual relationships. The general terms of the contract are contained in Article 295 and 296 of the said Act. According to these provisions, the general conditions are such contractual provisions that have been drawn up for a number of contracts proposed by one contracting party to the other party before or at the time of the conclusion of the contract, either by way of being invoked or contained in the contract. The general conditions, when accepted by the other party, become an integral part of the contract and supplement the particular provisions of the contract and, as a rule, bind as well as these. If there is a disagreement between the general and particular conditions, the latter apply.
The Civil Obligations Act determines that the provisions of the general conditions which are contrary to the principles of morality and conscientiousness and which in this way may cause an inequality in the rights and obligations of the contracting parties (Article 296/1) are null and void. At the same time, the Act stipulates that if a contract is concluded "according to pre-printed content or when the contract was otherwise prepared and proposed by one contracting party, ambiguous provisions will be interpreted in favor of the other party" (Article 320/1). This rule is called contra proferentem (against the proposer), and its meaning is clear - the party who prepared and proposed the text of the contract should also be held responsible for its vague provisions. The provisions or conditions of a contract prepared by one party which are not clear and which could cause harm shall be interpreted in favor of the party which did not propose or did not draw up the contract. This rule is obviously also applicable to the general conditions of a contract prepared or proposed by one party and which have become an integral part of a contract. With the FIDIC Conditions of Contract, such a situation is unlikely to occur, given that these Contract Conditions are not made up of either party and are made with the express intention of bringing a fair distribution of responsibilities into the relations between the parties.
When FIDIC General Conditions of Contract are applied in Croatia, such contract is subject to those Croatian regulations which are of a mandatory nature. In the area of civil law, the

Civil Obligations Act is applicable, which contains provisions on contracts and contractual relations. In the Act, there are provisions, some of which are mandatory and others non-mandatory i.e. dispositive. Provisions of a dispositive nature may be freely amended by the contracting parties, and such provisions shall apply only if the contracting parties have not resolved the matter differently or have not addressed it at all in their contract. Provisions that are mandatory in nature cannot be modified by the contracting parties, and if they have been modified in their contract, the courts will not recognize such changes and will apply the mandatory regulation.
However, it should be noted that the "mandatory provisions" of the Civil Obligations Act do not have any prescribed sanctions or penalties if the contracting parties envisage a different solution in their contract than the one stipulated in some mandatory provision. This means that the contracting parties may, in their contract, resolve an issue, which has been resolved in the Act through a provision having a mandatory character, in a different way in their contract. Until one of the contracting parties addresses the court or arbitration, an amended settlement of some mandatory provision of the Civil Obligations Act will be binding on relations between the parties. The contracting parties are expected to fulfill the commitments as agreed. Consequently, if one party has a claim against the other party under a contractual decision contrary to a mandatory regulation, it is entitled to that claim until the other party addresses the court claiming that the contractual solution is contrary to the mandatory regulation. If the other party does not go to court and does not dispute his obligation based on the application of the contractual agreement, he will be obliged to perform it because it is so agreed. However, if he does go to court or arbitration and raise an objection that a contractual decision is contrary to a mandatory provision of applicable law, and if the court or arbitration determines that it is indeed a mandatory provision, the court will not recognize the amended contractual solution and will apply the mandatory rule of the applicable law.
Unlike mandatory civil obligations law regulations, the situation with mandatory administrative regulations is different. Namely, mandatory regulations from various administrative regulations, most often prescribe sanctions for those who do not comply with them. In the area of construction, such regulations may, for example, be those contained in the Physical Planning Act [4] or Building Act [5] or in many other administrative regulations. These regulations, as a rule, set penalties for non-compliance. Such mandatory regulations shall not be amended by the contracting parties and, if they are not complied with, shall be subject to the prescribed legal sanctions.
A further question is whether the contracting parties can contract the application of a foreign law and thus avoid the application of Croatian law. Croatian regulations allow contracting parties to contract the application of foreign law as the applicable law for the settlement of contractual relations, in cases where a relationship has an "international character". However, if the contract is executed in Croatia and does not have "international characteristics", then even a contractual provision on the application of a foreign law will not prevent the application of Croatian mandatory regulations. The existence of an "international characteristic" is a factual issue, as determined by the court, examining the circumstances of each individual case. If the court finds that there are no "international features" in the relationship (if the contracting parties are Croatian nationals, if the contract is executed in Croatia, if the contract is signed in Croatia, etc.), it will not allow the application of foreign law. However, even when the application of a foreign law is permitted, if the works are performed in Croatia, the contracting parties must comply with those administrative mandatory regulations that are subject to penalties for non-compliance with Croatian law.
Consequently, when construction works are carried out in Croatia and the FIDIC General Conditions of Contract are applied, the contracting parties should also adjust these Contract Conditions to Croatian mandatory administrative regulations. Therefore, if a contract provides for Croatian law to be relevant then, in principle, it is advisable to align FIDIC General

Conditions of Contract with the mandatory regulations of Croatian law. However, if a dispute arises about an issue that is subject to a mandatory regulation of Croatian law, courts or arbitrations are required to apply a mandatory regulation, regardless of how the issue is regulated in the contract.

4 Use of FIDIC General Conditions of Contract in construction of transport infrastructure in Croatia

Infra means “below” so the infrastructure may be understood as the “underlying structure” of a country and its economy, the fixed installations that it needs in order to function. These include roads, bridges, dams, the water and sewer systems, railways and subways, airports, and harbors. Infrastructure is generally government-built and publicly owned [9].

The application of FIDIC General Conditions of Contract in Croatia began with the construction of road infrastructure financed by loans from international financial institutions, primarily from the World Bank and the European Bank for Reconstruction and Development. These banks insisted on the application of FIDIC General Conditions of Contract, mostly the Red Book, in motorways and state roads construction contracts. The contracts were written in English language, so there was no necessity for translation of the Red Book into Croatian.

While Croatia was preparing for EU membership, pre-accession funding was used for financing infrastructure and other projects. FIDIC General Conditions of Contract were mandatory for construction contracts. The language of contracts was English, so the original FIDIC books, primarily the Red and Yellow Books, were used. With the accession of Croatia to the EU, Croatian language became one of the official languages of the organization, thus construction contracts have been written in Croatian. This was one of the reasons for translating the FIDIC General Conditions of Contract into Croatian. Only three books, i.e. the Red (Uvjeti ugovora o građenju, Hrvatska komora inženjera građevinarstva, 2013), Yellow (Uvjeti ugovora za postrojenja i projektiranje i građenje, Hrvatska komora inženjera građevinarstva, 2013) and White Book (Model ugovora Naručitelja i Konzultanta za pružanje usluga, Hrvatska komora inženjera građevinarstva, 2013) have been translated into Croatian language and are nowadays most commonly used FIDIC General Conditions of Contract in Croatia. In this context, it should be noted that FIDIC gives licenses to member associations to publish translations of the Red, Yellow, Silver or any other book. However, FIDIC considers the official and authentic texts to be the versions in the English language and it does not accept any responsibility for the correctness, completeness or accuracy of the licensed translations.

In the meantime, several Croatian public companies, such as Croatian Motorways Ltd., have translated (with the permission of FIDIC) the FIDIC Red Book, produced their own particular conditions suited to their requirements and have been using these contract conditions as their standard model of contract for construction work financed by the state budget.

After the Croatian motorway network has been largely constructed, transport infrastructure development plans have been focused on the railway network. Recently, the EU has been financing the reconstruction and construction of the railway network in Croatia. These projects simultaneously use the Red Book for earth, concrete and track works and the Yellow Book for work on the power supply network. Since the contracts include both, the construction works and power supply works, the simultaneous use of the Red and Yellow Books produces a very complex contract structure. In addition, unlike road construction works, the employer and contractor are facing requirements for the works to be performed in relatively short period of time in which the employer ensures that the track is closed. In order to make the works as successful as possible at the time of closing the railway, it is essential for the contractor to plan in detail all the activities that need to be done, and for that it is also necessary to develop a detailed plan of the required resources into which, for critical activities, reserve resources should be included. Otherwise the closing time would be extended, which can ultimately cause considerable costs to the employer due to unforeseen traffic jams. Moreover, unlike the road infrastructure, in the railway infrastructure there are minimal possibilities of making temporary bypasses [10].

5 Conclusions

Widespread application of FIDIC General Conditions of Contract began in Croatia on road infrastructure projects funded by the international financial institutions. The contracts were then in English. This was also the case for projects funded through the European Union pre-accession funds. After Croatia's accession to the European Union, all contracts co-financed by this organization have been subject to the Croatian language, which was also one of the reasons for translating FIDIC General Conditions of Contract, namely the Red and Yellow Books into Croatian.

Once the Croatian motorway network has been largely constructed, the development focus was shifted from roads to the railway network. These projects simultaneously use the Red Book for earth, concrete and track works and the Yellow Book for work on the power supply network.

FIDIC contract conditions, by their very nature, are complex and require experience as well as diverse knowledge, technical, legal and economic, including project management experience, to ensure that contract implementation is smooth and undisputed. FIDIC always emphasizes that its contract conditions were drawn up by engineers for application by engineers. Practice has shown that, unfortunately, there are not enough engineers in Croatia capable of properly understanding and applying FIDIC contract conditions. In this regard, education and continuous training are required.

References

[1] Udom, K.: A brief introduction to FIDIC contracts, NBS, www.thenbs.com/knowledge/a-brief-introduction-to-fidic-contracts, 2014.

[2] Civil Obligations Act (Official Gazette no. 35/05, 41/08, 125/11, 78/15, 29/18)

[3] Köksal, T.: FIDIC Conditions of Contract as a Model for an International Construction Contract, International Journal of Humanities and Social Science, 1 (2011) 8

[4] Physical Planning Act (Official Gazette 153/13, 65/17, 114/18, 39/19)

[5] Building Act (Official Gazette 153/13, 20/17, 39/19, 125/19)

[6] Savage, D.: The top 10 things you need to know about FIDIC, https://www.charlesrussellspeechlys.com/en/ 30.01.2020.

[7] Baker, E., Mellors, B., Chalmers, S., Lavers, A.: FIDIC Contract Law and Practice, Routledge, London and New York, 2009

[8] FIDIC International Federation of Consulting Engineers, The Global Voice of Consulting Engineers, http://fidic.org/node/923

[9] Merriam-Webster Dictionary, https://www.merriam-webster.com/dictionary/infrastructure#h1 30.01.2020.

[10] Bralić, E.: Iskustva u primjeni FIDIC-ovih ugovora kod projekata željezničke infrastrukture, GRAĐEVINAR 71 (2019) 11, 987-993

[11] The FIDIC Contracts Guide First Edition, FIDIC 2000

THE POTENTIAL FOR EVITA PROJECT E-KPIS TO BE USED BY ROAD AUTHORITIES

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Abstract

Planning different strategies in road maintenance is one of the most important activities in road asset management. Assessment of different strategies and their comparison can be done by implementing an appropriate measure - Key Performance Indicators (KPIs). KPIs are currently used in many Road Authorities, and systematic research on the subject and development of indicators has been ongoing for many years. The Conference of European Directors of Roads (CEDR) funded project "EVITA - Environmental Performance Indicators for the Total Road Infrastructure Assets" aimed at developing and integrating new and existing environmental KPIs (e-KPIs) into the asset management process, taking into account the expectations of different stakeholders (users, operators, residents, etc.). The research focus was on environmental areas: Noise, with KPIs on day-evening-night & night noise, exposed population, population with sleep disturbance; Air, with KPIs on CO_2, NO_x, NO_2 and PM_{10} emissions; Water, with KPIs on water quality and salting of roads; and Natural resources and GHG emissions, with KPIs on resource consumption and CO_{2e} calculation. The project outputs were a set of e-KPIs produced after a comprehensive investigation of the state of the art during the project. The main benefit of this project is therefore to provide an applicable solution for the environmental assessment of different road infrastructure assets and to describe the expectations of different stakeholders in form of objective indicators. The 'User Evaluation Trial' phase of the project was used to gather feedback on the potential of e-KPIs to be used by national Road Authorities across Europe. Two Slovenian Road Authorities and one each from Denmark and Sweden were involved. The Slovenian Authorities provided input data for case studies, while all evaluated the proposed e-KPIs from their own perspective, taking into account national conditions and specificities.

Keywords: environmental performance indicators, key performance indicators, road asset management, stakeholders' expectations

1 Background

1.1 Assessment of road pavement performance

In 2004, the COST-Action 354 "Performance Indicators for Road Pavements" was initiated to define uniform European performance indicators and indices for the assessment of road pavement performance. COST-Action considered and defined several technical criteria from the perspective of users and operators. At that time Action recognised the importance of the impact that the construction and maintenance of the road network has on the environment, but at the same time not enough information was available to provide a comprehensive result [1].

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Research on (key) performance indicators was later complemented during the project EVITA "Environmental Performance Indicators for the Total Road Infrastructure Assets". The main objective of the project was to develop and integrate environmental KPIs (e-KPIs) into the asset management process, again taking into account the expectations of different stakeholders.
A priority for the project was firstly that the e-KPIs should be easy to understand and use, and secondly that they could be used to manage the full range of road infrastructure components - pavements, structures, road furniture etc. EVITA defined E-KPIs for four main categories and developed recommendations for the implementation and use of performance indicators [2].

1.2 Main groups of environmental performance indicators

The EVITA project started with a comprehensive investigation of the state of the art in collaboration with European Road Authorities and with other key stakeholders in road sector. In a second step, various e-KPIs for the environmental domains "Noise", "Air", "Water" and "Natural resources" were further developed and described in detail in one of the reports [3].

1.2.1 Noise indicators

The developed e-KPIs are based on noise mapping, using data from both sound level measurements and modelling. A three-level indicator was developed for noise impacts: The Emission indicator is based on physical measurements of noise levels, the Exposure indicator is based on noise exposure and thresholds, and the Impact indicator is based on noise exposure and "annoyance".

1.2.2 Indicators for air pollution and greenhouse gas emissions

Air pollution can be generated by traffic itself, throughout the life cycle of the infrastructure, or by construction and maintenance activities that take place at specific times. For the proposed e-KPI, calculations of NO_x, PM and CO_2 emissions (in t/km/yr) are needed. The technical parameters are total vehicle emissions per km of road.
Separate e-KPIs are proposed for PM and NO_x on the one hand, and CO_2 as a greenhouse gas on the other: an emission rate indicator for NOx and PM, based on modelled total emissions using traffic data and vehicle emission factors; and an exposure indicator for NO_2 and PM_{10} reflecting health impacts, based on an assessment of the population exposed to concentrations above EU limits.
The proposed e-KPI for CO_2 emissions from vehicles is based on the amount of CO_2 generated by road transport, using the same methodology as the emissions e-KPIs for air quality.

1.2.3 Indicators for water pollution

Water pollution from road infrastructure is mainly due to the wash-off of pollutants from the road surface and can be mitigated by protective measures related to the drainage system. Indicators can be developed depending on the quality of the drainage system and associated pollution control measures. Activities such as the spreading of salt and the sensitivity of the environment into which run-off is discharged must also be considered.
Two indicators are proposed: Water quality, based on an assessment of pollution load, environmental sensitivity and drainage system quality; and Salt, based on a comparison of salt load for the road section being assessed against the network average, weighted by local requirements and environmental sensitivity.

1.2.4 Natural resources indicators

Natural resource use in road infrastructure is mainly associated with material and energy consumption and waste generated during construction and maintenance. Care must be taken when developing indicators to avoid perverse results, e.g. encouraging unduly long transport of recycled material when new aggregate is available locally, so the indicator must take full account of life cycle impacts. Two indicators are suggested for use if the user has all the necessary data available. Material Resource Efficiency Indicator (MREI) is used to calculate the recycled content of the construction material, which is weighted to represent the relative impact on natural resources as a proportion of the total materials used. Embodied Carbon Reduction Indicator (ECRI) is used to compare the reduction in CO_2 emissions for a maintenance strategy to a nominal strategy that would have the maximum CO_2 emissions.
Greenhouse gases released into the air during maintenance or construction activities are also considered in this section.

1.2.5 Guideline for use of indicators

To guide the application of each indicator, so-called application sheets have been prepared alongside worked examples. An application sheet contains information needed to calculate the KPIs and recommendations for their practical application. The main elements (data groups) included in the application sheets are: Identification of the indicator (information on its use and application), Input data collection (parameters needed for the calculation), Calculation procedure (equations, transformation functions to determine a dimensionless index from 0 as very good to 5 as very poor), Output and use (context in which the e-KPI is used) and References. Figure 1 shows part of such an application sheet, Calculation procedure, for the environmental index for noise pollution Day-Evening-Night.

3. Calculation procedure		
Pre-calculation:		
Technical parameter:		$TP_{Noise,den} = 100 \cdot \frac{n_{den,i}}{n_{den}}$
Transformation Function:		$EPI_{Noise,den} = 0.05 \cdot TP_{Noise,den}$ with $[0 \leq EPI_{Noise,den} \leq 5]$
Description	$TP_{Noise,den}$	*Technical parameter for the percentage of people along the road section exposed to a Day-Evening-Night noise level higher than the threshold* $L_{den,threshold}$
	$EPI_{Noise,den}$	*Environmental index for Day-Evening-Night noise exposure above the threshold*
	$EPI_{Noise,den} = 0$	*All neighbouring people are exposed to a noise level below the threshold*
	$EPI_{Noise,den} = 5$	*All neighbouring people are exposed to a noise level above the threshold*

Figure 1 Part of Application sheet (extract from [2])

2 Trials for evaluation of E-KPIs

Slovenian Infrastructure Agency (SIA) and Motorway Company of the Republic of Slovenia (DARS) participated in a trial to evaluate the e-KPIs proposed by EVITA. The first Authority manages the Slovenian national road network with the exception of motorways, while the second Authority manages the motorway network separately. The aim of this experiment was to evaluate the relevance of the e-KPIs for the NRAs (strategically), to investigate the data availability to support the selected E-KPIs, to analyse the data and to comment on their usefulness. The focus of the work was on E-KPIs for noise, water quality and salt consumption.

2.1 E-KPI group: Noise

This group consists of 4 E-KPIs: $EPI_{Noise,den}$, $EPI_{Noise,night}$, $EPI_{Noise, \% HA}$ (high annoyance residents), and $EPI_{Noise,HSD}$ (high sleep disturbance residents).
While both Authorities expressed strong interest in all e-KPIs, the available data depends on the policy driver - the EU Noise Directive [4] and its requirements. EU member states are required to produce strategic noise maps for all major roads with more than six million vehicle passages per year. This means that input data were available for few road sections managed by SIA and for most sections managed by DARS. If we take this into consideration, it can be said that this group of e-KPIs was the easiest to calculate as all the required data is available. From the perspective of the results, the E-KPIs show a good situation, especially in terms of the percentage of people highly annoyed by noise and the percentage of people whose sleep is disturbed by high noise levels. Looking at the results specifically for the motorway network, they also show a very good situation. This is not surprising when one knows that at the time of writing the case studies a large part of the motorway network was completely new and that the noise problem was solved in many places by installing noise barriers.
Figure 2 shows an example of trial results - motorway network with $EPI_{Noise, night}$ for motorway sections for which noise maps were available. The noise ranges (see legend) were chosen for experimental purposes only and do not reflect Authority policy or preferences.

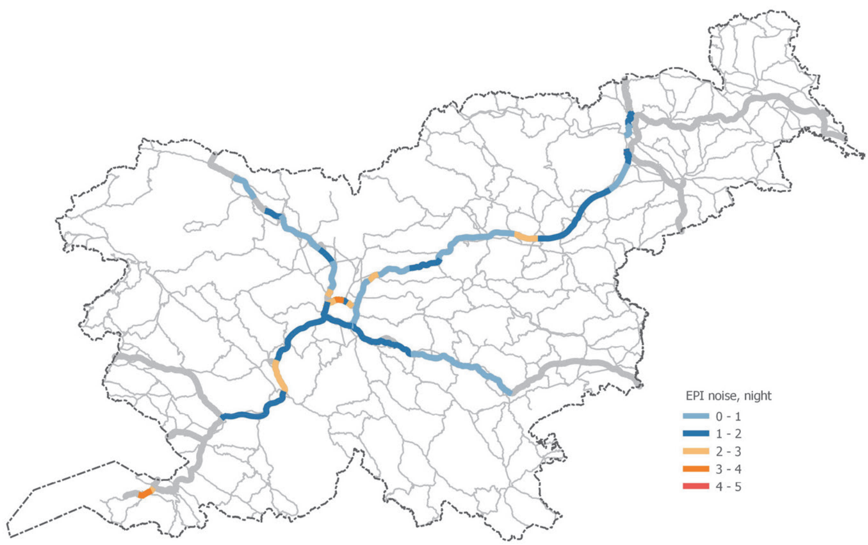

Figure 2 $EPI_{Noise, night}$ for motorway sections in Slovenia

2.2 E-KPI group: Air quality and CO_2 emissions

This group consists of 5 E-KPIs: $EPI_{Emissions,CO2}$, $EPI_{Emissions,NOx}$, $EPI_{Emissions,PM}$, $EPI_{Emissions,NO2}$, and $EPI_{Emissions,PM10}$. Both Authorities expressed low to moderate interest in all e-KPIs. In all cases, some data are available, but the estimate is that high costs would be necessary to complete the data.
Emissions data for roads (own data sets on vehicle emissions) were not available for either Authority. The estimate is that both the emissions data and the model data are associated with high costs that would require external resources. It is assumed that the current low level of interest will increase over time.

2.3 E-KPI group: Water quality

This group consists of 2 e-KPIs: EPI_{Water} and EPI_{Salt}. For the first there is some data and low interest at SIA (high for groundwater sensitive areas), while for the second there is high interest and almost all data available for calculation.
EPI_{Water} is quite complex and requires relatively detailed input data. There is generally no problem with collecting traffic data, but rather qualitative assessment data. This relates to information on the type of runoff, ability to handle risk volumes, structural condition, and operational status. As these data are not always available, the calculation was only carried out for a single case (a detention basin).
During the construction of the motorway network, special attention was paid to the drainage system, among other things. Therefore, DARS was very interested in the future collection of input data and calculation of EPI_{Water}.
The average amount of salt on individual road sections and on the entire road network is routinely recorded by both NRAs. SIA has in the past developed a system to track salt use during winter maintenance, which has resulted in little interest in implementing EVITA's proposed EPI_{Salt}. On the other hand, DARS is interested in the possibility of comparing salt consumption between regional maintenance bases. Figure 3 shows schematically motorway sections managed by specific regional maintenance bases.

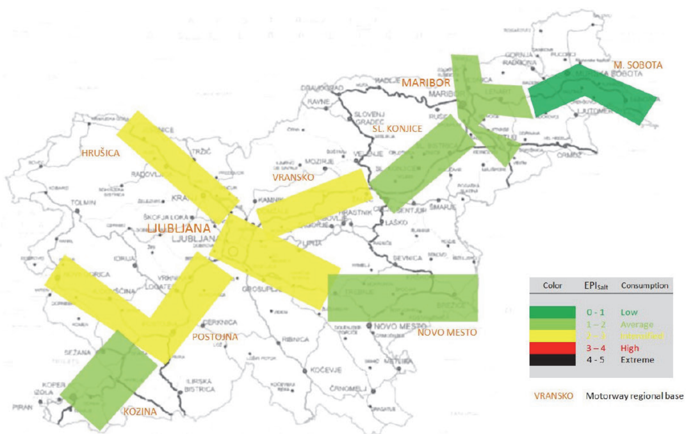

Figure 3 Scheme of calculated EPI_{Salt} on motorway network

Salt consumption ranges (see legend) were chosen for experimental purposes only and do not reflect DARS policy or preferences.

2.4 E-KPI group: Natural resources

This group consists of 3 e-KPIs: $EPI_{Resources}$, EPI_{ECR}, and CO2e. For all these e-KPIs, it was assumed that data is difficult to obtain, therefore the interest in calculating these e-KPIs is low. No study has been conducted to calculate these e-KPIs. While other e-KPIs can be used well for implementation on whole road networks, the e-KPIs for natural resources are highly dependent on local conditions. Different maintenance or construction strategies, different production practices, pavement designs, available materials, electricity mixes, and other region-specific elements all contribute to differences in calculated results and make them difficult to compare. As the focus on environmental sustainability increases, the collection of input data should improve.

3 Additional feedback from road authorities

A summary of the feedback received from the Danish and Swedish Road Authorities is presented here for each group of e-KPIs.
In Denmark, data is available for all noise e-KPIs and the Authority is very interested in all of them. However, they approach the noise issue somewhat differently and in the future they may look at the differences and the advantages and disadvantages of both systems.
In Sweden they are in favor of the proposed indicators $EPI_{Noise,den}$ and $EPI_{Noise,night}$. However, the indicators are not very suitable for Swedish conditions and therefore there is little interest in implementing them. Inventory and monitoring in Sweden is based on a Swedish measurement methodology, which is different from the proposed indicators. Another reason for the low interest is that, due to the EU Noise Directive, there are inventory data of the road network above 3 million vehicles per year, which would make it possible to calculate EPIs for these roads. However, it must be said that this only applies to a small part of the Swedish state road network. The noise experts find that the $EPI_{Noise,\ \%\ HA}$ and $EPI_{Noise,HSD}$ are generally very well designed indicators. Specifically for the purposes of pavement management practice, the indicators are not useful because pavement management in relation to noise in Sweden is based on Cost Benefit Analysis.
In Denmark, the data for calculating the indicators from the air quality and CO_2 emissions group were not readily available. However, they were keen to learn from this group of e-KPIs.
In Sweden, HBEFA was used at the time as an emission model that can be used at the network level down to the detail level when in-data is available, and SIMAIR, a Swedish model developed by SMHI (Swedish Meteorological and Hydrological Institute) to calculate concentration (dispersion model) and be able to estimate dose and impact of emissions. Using the calculation sheet to convert to the e-KPI would be straightforward once one has the in-data (emissions or exposure), but requires external resources. Therefore, there has been little interest in indicators from this group.
Almost all the data is available for calculating the indicators of the water quality group in Denmark, and they are also keen to learn from this group of e-KPIs.
According to the water specialists from the Road Authority in Sweden, the indicators seem to be well designed. Regarding the indicator for water quality and drainage systems, it is possible to acquire the in-data on drinking water resources in many places. The proposed indicators show a strong similarity to the risk assessment system developed and used by the Authority. In their system, additional consideration is given to how many people use the water resource and whether there are alternative resources. They also have data available that can be used to calculate EPI_{Salt}.
Similar to the water quality indicators, Denmark is very interested in learning from the natural resource e-KPI group as well. They kind of have data available, although the right data is hard to retrieve.
Natural resources and indicators are an area of high interest and focus in Sweden, as at the time they were developing their own climate calculator to be able to calculate CO_2 emissions from construction, maintenance and operation of road (and rail) infrastructure. One comment on this group was that all separate e-KPIs could also be a common one.

4 Conclusions

The EVITA project developed and provided a set of environmental KPIs and a practical guide for their use in pavement management practise, together with ‘Application Sheets’ with essential information that a user can pick up and use if interested in applying an indicator. During the user evaluation trial, it was found that for the calculation of some indicators, data was readily available, for some reasonably available and for others difficult to obtain.

As the project was completed some time ago, this situation may have largely improved as international environmental awareness has increased significantly over time. In adopting the proposed indicators, it is also important to keep in mind that they are either site-specific or global. This is important because a global indicator generally impact an entire organisation, while a site-specific indicator generally only impacts very specific mitigation measures at that site.

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References

[1] Litzka, J., Leben, B., La Torre, F., Weninger-Vycudil, A., de Lurdes Antunes, M., Kokot, D., Mladenovic, G., Brittain, S., Viner, H.: The Way Forward for Pavement Performance Indicators Across Europe, COST Action 354 Performance Indicators for Road Pavements Final Report, FSV – Austrian Transportation Research Association, Austria, doi: http://cost354.zag.si/index.php?id=102

[2] Practical Guide for the use of E-KPIs in pavement management practice, EVITA Deliverable D4.2, http://se-kpi.fehrl.org/?m=49&mode=download&id_file=14636, 10. 2. 2021.

[3] Jones, M., Bradbury, T., Cesbron, J., Harris, B., Kokot, D., Lester, A., Thomas, C., Viner, H., Weninger-Vycudil, A.: Report on recommended E-KPIs - EVITA, Deliverable D 3.1, http://se-kpi.fehrl.org/?m=49&mode=download&id_file=14633, 10. 2. 2021.

[4] Directive 2002/49/EC of the European Parliament and of the Council of 25 June 2002 relating to the assessment and management of environmental noise - Declaration by the Commission in the Conciliation Committee on the Directive relating to the assessment and management of environmental noise, https://eur-lex.europa.eu/legal-content/EN/TXT/?uri=celex%3A32002L0049, 10. 2. 2021.

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MILITARY ROAD INTERPOLATION INTO PUBLIC ROADS NETWORK IN CONDITIONS OF NATURAL DISASTER

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Abstract

The traffic connection of an area is important for the economic development of the country, but it is of decisive importance for defence purposes. Unlike carefully planned public roads, the concept of military road construction must include solutions for "sudden and non-standard" or sudden and unusual circumstances and events, whether military forces participate in war or peacetime (natural disasters) operations. The paper presents the military roads partition and methods of construction in specific terrain. Military engineering units have a key role in the provision of traffic communications. The example shows the construction of a road for evacuation of the population after a natural disaster by bridging the river with a launch bridge with a tank bridge carrier MT-55A with a construction time framework.

Keywords: public roads network, military road, natural disaster, military engineering units

1 Introduction

Today, military forces are not engaged solely for the purpose of state territory protection. Because of the rise of terrorism, as well as natural disasters due to global warming, the role of the military is increasing even in peacetime [1]. The role of each army is to protect and defend the sovereignty of a state, its integrity and territorial integrity. During peacetime, the military has a major role in helping the population when civilian services are not sufficient to assist in times of natural disasters. It possesses material and technical means and the machinery needed to solve such problems. In order to perform successful military and peaceful operations, it is necessary to enable relocation of human recourses and techniques by using public roads and/or construction of new military roads. In doing so, the military road network should be interpolated into the public road network and its use should be coordinated with the companies operating the public highways in accordance with national regulations. Engineering, as part of the military, participates in the planning and construction of military roads having its own structure and command and does not undertake anything without the command of superior commanders.
This paper shows the division of military roads and how they are constructed in special and difficult conditions. It indicates the capability of military engineers to overcome obstacles on the terrain in a limited time and in such circumstances. The method of organizing the work and planning of engineering works for the construction of a temporary military road by observing the general principles of occupational safety and maintenance procedures are described in the paper.

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2 Division of military roads

The military road network consists of: public roads when used by the military units, permanent and temporary military roads.

Public roads are divided [2], [3] by social, traffic and economic features into motorways, state, country and local roads, by type of traffic into motor and mixed roads, by size of motor traffic and by the task of connecting and medium-length of travelling into motorways and five-class and according to terrain configuration to those without restriction (flat terrain), with slight restriction (hilly terrain), considerable restriction (highland terrain), and great restriction (mountain terrain).

Permanent military roads are roads constructed for internal military purposes and are intended for military traffic in peacetime. During the war, they are used together with the existing public roads (for military and civilian traffic). Temporary military roads are roads of limited duration and are built on the routes of the movements of units when existing roads are insufficient or when they are more heavily damaged. These roads are being constructed to bypass sensitive traffic nodes, larger settlements, and contaminated parts of the existing roads, as well as to transport the troops to positions and areas of gathering and similar.

Just as public road regulations prescribe the design of road elements in such a way that motor vehicles can move safely and smoothly, so the design of military roads and their classification and marking depends on [4]: whether there will be wheeled or tracked vehicles, whether it is intended for single row of vehicles (usually 1 to 1.5 lane width of existing public roads) or for two rows of vehicles, depending on axle load, height and width of military vehicles and on space constraints such as curves of small radii, large longitudinal slopes, limited heights and widths (bridges, tunnels, etc.)

The maps of military roads contain the classification of roads depending on the possibility of use in adverse weather conditions and the specific conditions (deep snow, the possibility of floods, etc.) and depending on traffic load [4]. By this classification the roads are divided into: those without restrictions - possible year-round traffic in all conditions and for unlimited loads; with partial restriction - while keeping the potential traffic in almost all weather conditions and with occasional bans, but in difficult conditions it is necessary to limit the traffic load; those whose use is not possible in adverse weather conditions and the potential use would require long-term repairs.

3 Construction in special conditions

In the construction of public roads, the construction in special conditions is not considered, that is, taking into account the principles of tracing, all disadvantages are avoided. Unlike public roads, temporary military roads are usually small in length and the entire section can be built in marshes or in difficult mountainous terrain, and there is a need to build during wintertime when the land is covered with snow.

3.1 Selection of the route

The route of the new military road should be selected in such a way that it is as quickly and easily accessible as possible to the endangered area or to the specified location. The type and slope of the terrain, the length of the route, the narrowest places for overcoming the obstacle, the boundaries of private estates should be determined in order to inflict minimal damage to the land of the inhabitants. The barrier overhang should be suitable for mounting the bridge from the bridge carrier tank. Preferably, the route should be selected in such a way that the route is cleaned with as little mining work as possible and the machinery can be used. In order to optimally select the route of the road, the reconnaissance is carried out on the map and in the field.

3.2 Construction on poorly bearing ground

Coherent soils have poor bearing capacity during the rainfall period, while in the dry season, military vehicles can move outside the designated road. The most difficult are the road sections over the marshy soil, so the survey should determine the optimal conditions for crossing the marsh: the most suitable and shortest route of the road, the conditions for the passability of the marsh, the level of groundwater and surface water. A reconnaissance group is usually formed for routing.

The first variant of the road route is where the marsh is narrowest and closest, with maximum utilization of already existing roads and paths. The wetland may be more passable elsewhere, which determines the crossing point [5]. Passability means the number of vehicles that can pass on one track so that the vehicle (wheels) does not fall into the ground more than the height of the center of the vehicle relative to the ground. The passability can be estimated by starting the vehicle (test load), always having another vehicle to pull out the test or by using a probe or the sonde (thin twig) that is inserted into the marshy soil, and the peat depth is estimated according to the resistance provided by the drive. If trees grow in height over 3 m in height and over 5 cm in thickness, this is usually an indication that the marsh is passable to caterpillars. If a reed and sedge grow on wetland, the soil is moist and impassable. The required bearing capacity of the soil is min. 50 kN/m2. The caterpillars require load capacity of 70-100 kN/m2 and in the event that it has been fulfilled, the traffic can be released after marking the route and removing vegetation at the required crossing width. However, if more vehicles are expected, then it is necessary to improve the carrying capacity of the soil by logs or geotextiles. Up to 600 vehicles should be allowed to pass by improving the load carrying capacity of the soil. If a satisfactory load-bearing cannot be achieved with the described improvements, it is possible to construct an embankment or wooden bridge.

The highest level of surface water is determined by spotting the highest level of flooding by detecting traces on the surrounding land or collecting data from the population. If mixed trees and other vegetation grow on the land, it is a sign that the surface water does not linger for a long time, and if the bush and reeds grow, it is the opposite. Groundwater level is determined by digging bore holes on the routing of reconnaissance. The road level should be above the surface water level.

3.3 Building a road on mountainous terrain

The first step in building a road on mountainous terrain is to determine the route of the road on the map using the existing trails and roads to the maximum extent possible. The lengths of the sections with the maximum longitudinal slope shall not exceed 1 km. Otherwise, the length of 50-100 m with a slope of 2 % should be made after 1 km. This is followed by field reconnaissance and the final route selection. Only after that, the route of the temporary military road is marked.

In the mountainous terrain, large quantities of earthworks are required, the work in the rock material with the use of explosives, the work in confined spaces, construction of large number of facilities, all of which require a lot of care and responsibility to organize the work. To accelerate the progress of work, the method of excavating is applied whereby a wider range of work is opened. Timber (trees that grow in close proximity) is used for the construction of retaining walls on sections where there is a danger of stones falling down or snow avalanches. In the construction of temporary military roads, some solutions not otherwise used in the construction of public roads may be applied: using crossfalls independent of the road geometry (safety), filling in the river or launch bridge crossings, etc. Despite the fact that modern military vehicles can overcome water barriers of greater depths, draft must be managed (remove large stones and level the river bottom, approach the ramps with gentle slopes). If

a bridge is still being built, it is necessary to determine the vertical alignment , considering that the water levels of the mountain rivers can rise sharply and that the water flow is at high speed. Along the dangerous road sections (sharp bends of the precipice and the like), fences are placed on the outside of the pavement in the form of wooden or stone pillars or walls which are painted white on the inside for safer driving at night.
The pavement is made of stone material. In case that the road passes over the rocky soil, the road body is well straightened and the pavement may be avoided. On one-way mountain roads, every 250 m is used for lay-bys of up to 100 m in length. Platforms for shorter vehicle breaks are created in front of long and larger ascents or descents. Along the road, gravel, sand or sand reserves are being prepared to prevent the vehicle from slipping during periods of rain, snow or ice.

3.4 Building military roads in wintertime

Road construction during the winter implies conditions in which the country is covered with snow, temperatures are low and for a long time below 0°C, rivers, lakes and wetlands are frozen. Under these conditions, public roads are not built. However, such conditions do not pose a major problem for the construction of temporary military roads which usually serve traffic only for a short time. These are even easier conditions than periods of heavy rainfall [5]. The frozen soil has good bearing capacity and the movement of vehicles is impeded due to icy conditions which can be mitigated by spreading gravel or dunes. In this case, too, the scouting is required. First, it is necessary to determine the route of the road on the map and then collect information on: thickness and compactness of the snow, air temperatures, thickness, condition and bearing capacity of ice on rivers and lakes, connection of ice with shores, other information about the route (gradients of ascents, falls, etc.), and the site of construction material. After marking the route of the road and clearing the vegetation route, it is necessary to fill the holes at the location of the extracted stumps and clear the route from the snow. Usually it takes the longest time to clear the snow by using different machines and hand tools. On such a road, except for traffic signs, poles 1.2 to 1.6 m high should be installed along the road, or snow pyramids should be constructed to channel traffic and clean up in case of new precipitation. If the section of the road is without major ups and downs, the traffic can be opened immediately after clearing the snow. Passing spaces should be made on one-way roads.

3.5 Maintenance of military roads

Road maintenance work for military purposes includes: removing defects identified by reconnaissance and ongoing maintenance work during use. Maintenance work is no different than that required on public roads.

4 Engineering units

Engineering is an Army branch, equipped and trained for the combat support in all forms of combat. It also has a role in assisting the civilian population in times of natural disasters such as earthquakes, floods, etc. Engineering support is a set of engineering works on overcoming natural and artificial obstacles, eliminating the consequences of attacks, making communication links (roads, airports, heliports, ports, docks) and facilities on them for movement and manoeuvering, delivery and evacuation, landscaping with the aim of protecting the military forces and resources, building artificial and improving natural barriers to prevent the rapid penetration of enemy forces and protecting their own units, finding the required amounts of water and making water available for use, and participating in the disguise of facilities and units in fire positions, command posts, liaison centers, etc.

In accordance with its basic tasks, military engineering has task units which are equipped with the appropriate material resources. Engineering units whose task is the construction of roads, the road units, maintaining and repairing the existing roads and building temporary ones and structures on them, clearing rubble and other obstacles on roads and in populated areas. They are equipped with: dockers, graders, crushers, mixers, rollers, compressors, power drills, chainsaws, cranes, snow cleaners, trucks, launch bridges, loaders, diggers, self-loaders, tool kits, accessories etc.

5 Example of the construction of a temporary military road

A section of a public road that is out of traffic due to the damage, is interpolated by a temporary military road. Overcoming the water barrier on the temporary military road was carried out by the launch bridge, the MT-55A armoured vehicle-launched bridge.

5.1 Organization of engineering works

Depending on the available time, military forces and resources for the construction of a temporary military road should be defined. In order to make the construction faster, on the road alignment which is a linear object, the engineering unit is spread over a relatively long area and works both in groups and individually (humus removal, excavation, mining, etc.), but all are interconnected in the task. It is essential to maximally use the human and material resources of the engineering unit, depending on the work to be performed. In its content, methods of work and means for work, the organization of engineering works is very similar to the organization of work in construction, so these disciplines in scientific and professional sense are mutually developing and complementary. The stages of organization are the planning and preparation of engineering work.
Planning engineering works. The basis for the preparation of the plan of final construction works is the order of the superior commander to perform the engineering tasks and the corresponding technical documentation is made. An engineering work plan must be carried out in all cases, whether the technical solution was given by a task or afterwards. It must be achievable in accordance with the weather, terrain and combat conditions, and should allow for easy monitoring of the progress of works and making the necessary corrections. The development of the plan consists of predicting the external factors that have a positive or negative effect on performing the task, and developing a dynamic plan of activities. The following general principles apply to the planning: gradual engagement of forces and engineering and technical means in the initial phase of work and shutdown in the final phase, simultaneous execution of several work operations for the purpose of faster performing of the task, maximal use of engineering machinery with careful handling and maintenance.
Preparing engineering works. The preparatory work consists of: studying the task, the assessment of the situation, the scouting of the area of the construction works, the arrangement of the work site, preparation of materials, the organization of transport and giving the command.

5.2 The MT-55A armoured vehicle-launched bridge (AVLB) tank

One of the most important engineering machines for overcoming obstacles is the MT-55A armoured vehicle-launched bridge (AVLB) tank (Figure 1). It is modified specially fitted T-55A tank with no turret and weaponry. It is equipped with a bridge and devices that allow the bridge to be laid over an obstacle and placed back on the armored body of the tank [6]. It is tasked for the quick laying of a bridge over antitank barriers (antitank trenches, steep inclines and slopes) on a land that is otherwise passable under normal combat conditions. It is possible to lay a bridge over water barriers with muddy or soft bottoms, with steep banks without prior engineering arrangements, over deep and impassable parts of riverbeds. The bridge can be used to further strengthen the existing bridges as well as to secure weakly bearing sections of the terrain. The bridge carrier tank has mostly the same equipment found on the T-55A tank as well as modified and new assemblies that are missing from the base tank. Figure 1 shows the launch bridge installation process [7] which is also possible on terrains of various slopes.

Tactical and technical features: MT-55A is a medium tank type with a total weight of 36 t; the crew consist of two members; bridge weight 6 t, load capacity 50 t, width 3300 mm, laying time 3 min. and the time of tank placement 5 to 8 min; the length of MT-55A with folding bridge is 9880 mm and set on the flat surface 27100 mm; the length of the folded bridge is 9600 mm and unfolded is 18000 mm; the width of MT-55A without bridge is 3270 mm and with bridge 3300 mm and the height of MT-55A with folded bridge is 3350 mm.

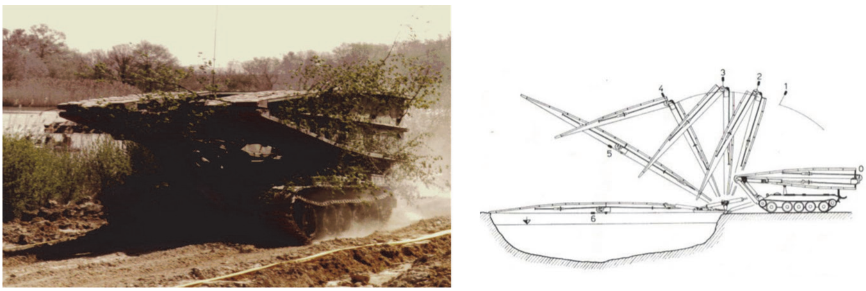

Figure 1 The MT-55A armoured vehicle-launched bridge tank and the assembling process

5.3 Calculation of time and resources required for the given task

The location of the bridge installation on coherent ground, in the example described (Figure 2), was selected at the location of the most favourable slope of the terrain and the least width of the obstacle.

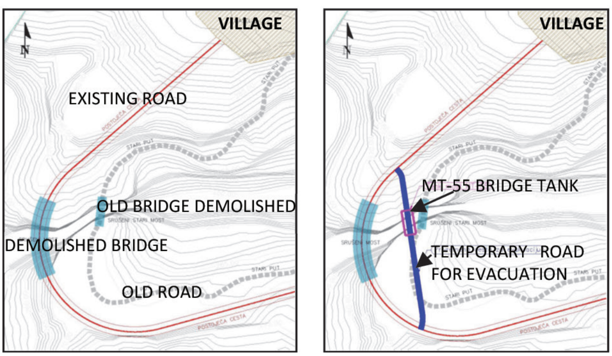

Figure 2 Current state and overview of project draft

Based on the horizontal and vertical alignment and cross sections (Figure 3), the quantities of stone material were calculated for: the road bed (46 m^3) and pavement (16 m^3). For each engineering machine available to this engineering unit, the efficiency was calculated using known empirical terms and taking into account the following influential parameters: soil category on which the works were performed, the coefficient of age of the machine and the competence of the machine operator, the speed of movement of the machine at work, etc.

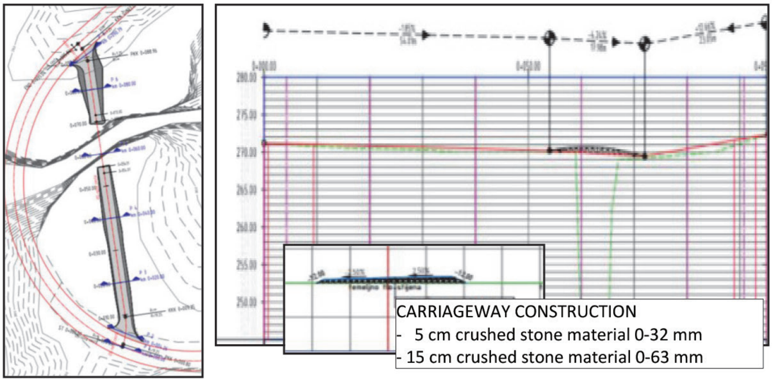

Figure 3 Horizontal and vertical alignment and normal cross sections of temporary military road

Table 1 Overview of works, resources, and time needed to complete

No.	Type of work	People/ means	Quantity	Time needed (min.)
1.	Reconnaissance	Officers	5	20
2.	Stakeout of the first part of road alignment	Soldiers	4	40
3.	Cleaning of the first part of road alignment from vegetation	Soldiers /chain saws	6	30
4.	Mining a rock	Miners	2	35
5.	Levelling by dozer (1st part)	Dozer	1	150
6.	Filling in and levelling of nivelette road (1st part)	Dumper Dozer	5 1	Simultaneously with works under no. 5
7.	Launching of the bridge	TM-55	1	15
8.	Stakeout of the second part of the road alignment	Soldiers	4	15
9.	Cleaning the second part of the road alignment from vegetation	Soldiers /chain saws	6	20
10.	Levelling by dozer (2nd part)	Dozer	1	45
11.	Filling in and levelling of nivelette road (2nd part)	Dumper Dozer	5 1	Simultaneously with works under no. 10
12.	Filling in the road alignment with rough stones	Dumper Dozer	5 1	120
13.	Filling in the road alignment with small stones	Dumper Dozer	5 1	90
14.	Rolling	Road Roller	2	30
15.	Establishing of the traffic	Soldiers	8	20

The following machines were used to perform the task: dozer, loader, excavator, road roller and road grader. Levelling of the ground with a dozer of 14 m^3/h takes approximately 2.5 hrs. It takes 30 minutes to roll the total length of the road. By using 5 dump trucks with a load capacity of 15 m^3, it is possible to transport materials from a quarry 15 km away by monitoring the dynamics of the planned works. The loader is in reserve for spreading excess material. Table 1 shows the type, resources, and dynamics of temporary military road construction done by a military engineer unit. The road was built in 630 minutes (10.5 hours).

6 Conclusion

Military roads differ from civil roads because they are mostly temporary roads and have specific uses (eg. in this case the evacuation of the population and protection of the property). The advantage of engaging the armed forces over civilian services lies in the speed of their work and the ability to currently deploy forces that are more prepared to operate in specific conditions. In its composition, military engineering units have all the necessary material resources and machinery to overcome obstacles and to access more inaccessible terrain, which leads to a faster and more efficient solution of the problem. The example of the construction of a temporary military road shows the organization of work, planning and management of the execution of the task, which is organized hierarchically and is carried out solely according to the commands of the superior ones. The engineering units are capable of finding out quick solutions which is of utmost importance in combat operations but not less important in helping the population during natural disasters. The engagement of the armed forces to remedy the effects of natural disasters shows their humane features and importance during peacetime activities. The armed forces, especially engineering units, will continue to play a major role in recovering and rescuing civilian population during natural disasters, and the best indicators for this are numerous examples from the past when the armed forces made an invaluable contribution in helping the population in exceptional circumstances.

References

[1] https://ops.fhwa.dot.gov/publications/fhwahop05029/chapter_1.htm, 29.12.2019.

[2] The Roads Act (Official Gazette 84/2011 including all amendments to the last one 110/19)

[3] Pravilnik o osnovnim uvjetima kojima javne ceste izvan naselja i njihovi elementi moraju udovoljavati sa stajališta sigurnosti prometa (NN 110/2001).

[4] http://www.cedus.it/documents/PoliziaLocale/Gestione/Tecniche_di_controllo_del_territorio/MilitaryPoliceControls_TacticsTechniquesProcedures.pdf , 29.12.2019

[5] Španović, I.: Vojni putevi, Military Publishing Company, Beograd, 1983.

[6] Fusić, M., Šimić, M.: Predavanja - Inženjerijski strojevi, Croatian Defence Academy "Dr. Franjo Tuđman, Zagreb, 2018.

[7] The MT-55A armoured vehicle-launched bridge tank, Federal Secretariat for National Defense, Technical Administration, Beograd, 1979.

COMBINING CAPITAL GRANT AND AVAILABILITY PAYMENT TO KEEP TOLL RATES AFFORDABLE

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Abstract

In several countries public budgets cannot provide all the funds needed to build priority transport and other infrastructure projects that are economically justified and environmentally and socially sound. Under certain circumstances, projects meeting such conditions can be implemented by involving private financing, through public-private partnerships (PPP), which is a means to get projects completed by leveraging scarce public resources. Priority highway PPP projects may require toll rates above the affordability level of road users, particularly when construction costs are relatively high and traffic volumes are relatively low. The provision of capital grants and/or availability payments to the concessionaire (i.e., the private partner) by the government (i.e., the public partner) would reduce the toll rate required to attract private investors for the project. Such projects, where the sources of revenue to the private partner (or concessionaire) include both the users of the facility and the government, are usually called hybrid PPPs. A key step in assuring that a proposed PPP highway project would attract private investors is to determine whether financial public support would be required, and if so, how much. To this endeavor, this paper reviews and applies a hybrid PPP financial model for highways that facilitates carrying out projects' financial viability by decision makers and practitioners. A numerical case study is used to illustrate applications of the model to conditions deemed representative of Southeastern European countries. The main outputs generated by the model include the project's internal rate of return, equity internal rate of return, annual debt service coverage ratio, and the present value of the government's cash flow. A sensitivity analysis carried out shows the impact of key input parameters on the main outputs. While the financial model discussed has been developed for roads, it can also be adapted to other forms of transport infrastructure, such as rail.

Keywords: public-private partnership, capital grant, availability payment, toll rate

1 Introduction

Public-private partnership (PPP) is a long-term contract between a private party and a government entity for providing a public asset or service, in which the private party bears significant risk and management responsibility, and remuneration is linked to performance [1]. There has been a substantial contribution of the private sector to finance roads and other forms of transport infrastructure across the world. In 2019, private investment commitments in energy, transport, information and communications technology (ICT) backbone, water, and municipal solid waste (MSW) infrastructure in low- and middle-income countries totaled US$96.7 billion across 409 projects in 62 countries [2].

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Private investment in sustainable, quality infrastructure is critical to boost economic growth and promote resilience – resilience against the current public health crisis and climate-related risks, as well as future global and national shocks [3]. Attracting more private financing to road projects in Southeastern European countries would be a means toward greater investments to keep road infrastructure in acceptable condition and carry out required expansions in a context of public budget constraints. When arrangements for private participation or, more generally, public-private partnerships (PPP) are designed well, they can lead to [4]:

1. Greater financial efficiency, by leveraging public money through the mobilization of private capital, reducing the impact of investments in infrastructure on the fiscal budget, and creating fiscal space to expand public service delivery in other sectors;
2. Better distribution of risks, by transferring design, construction, and performance risks to the private sector, which is best able to manage such risks; and
3. Better governance, by increasing the accountability of the service provider through competitive bidding, disclosure policies, and public reporting.

Government support to potential PPP road projects is justified when an economically feasible project does not offer, without such support, the financial benefits required to attract private concessionaires. The mixing of public and private funding to get projects completed is a way to leverage scarce public resources. Combining Capital Grant and Availability Payment to Keep Toll Rates Affordable. This paper analyzes the combination of capital grants (or construction subsidies) and availability payments to attract private partners to a PPP road project, keeping toll rates at an affordable level.

2 Sources of revenue to PPP road projects

In a PPP road project, the sources of revenue to the private partner (or concessionaire) may include:

- the road users, through tolling,
- the government (through, for example, availability payments, capital grants, or shadow tolls),
- both road users and government, which is usually called a hybrid concession.

When traffic levels are relatively low and/or construction costs are relatively high, it is likely that a proposed motorway or expressway will require government support (e.g., capital grants and/or availability payments) to complement toll collection in order to generate enough revenues to attract private partners to compete for such PPP road project. A national (or international) electronic road tolling collection (ETC) system would reduce toll collection costs and, consequently, facilitate the implementation of such projects.

3 Open and competitive bidding procedure

Measures to increase competition may include improved contract design (e.g., avoiding too big or too small contract size), wider advertisement of the bidding, clarification of issues raised by potential bidders, and providing enough time for bidders to prepare their bids (usually a minimum of 90 days is required).

Assuming a good degree of competition in the selection of the concessionaire, an open and competitive bidding procedure would minimize the amount of the availability payment to be paid to the concessionaire, by the government agency, during the O&M phase of the contract, when this is the key financial criterion to select the successful bidder.

Another related option, which may be available to decision-makers, would consider the toll rate as a constant (for example, based on the maximum affordable toll rate), but establish as

the financial criterion, to select the successful bidder, the sum of the capital grant and the availability payment to be paid to the concessionaire. This approach would have as a drawback the risk that some bidders may “frontload” their financial proposal, that is, exaggerate the proposed capital grant and minimize the availability payment. Such risk, however, could be minimized by specifying a maximum limit for the capital grant (for example, 60 % of the total construction cost). Case study: A numerical example of the interrelationship between capital grant, availability payments, and toll rates
The previous paragraphs described, in general terms, options to implement PPP in the road sector. To provide a quantitative assessment of potential PPP projects, in light of future investments and funding sustainability, the next paragraphs discuss a numerical example to illustrate, for a hypothetical proposed motorway, the combination of availability payments and capital grants to keep the toll rates at an affordable level. We will assume four scenarios (i.e., low, medium, high and very high cost) for the total construction cost (including design) and the related annual O&M cost to be:

a. Construction cost: €100 million; annual O&M cost: €5 million;
b. Construction cost: €200 million; annual O&M cost: €10 million; and
c. Construction cost: €300 million; annual O&M cost: €15 million.
d. Construction cost: €400 million; annual O&M cost: €20 million.

A World Bank governance study [5] showed an average cost increase in road works contracts (i.e., cost overrun) of 18 % in Southeast Europe, based on a two-country sample (namely Albania and North Macedonia). Assuming the same cost overrun would occur in a traditional road works if the proposed project would not be implemented as a PPP, it would seem fair to expect the construction cost to be inflated by 18 %. If implemented as a PPP project (where there is no provision for variation orders), the above construction costs would prevail. However, if implemented as a traditional bill-of-quantities (BOQ) contract, it is likely that the ultimate construction costs would be (i) €118 million, (ii) €236 million, (iii) €354 million, and (iv) €472 million, respectively. Consequently, implementation of the project as a PPP might save the relevant country about €72 million, in the case of the highest construction cost scenario. We will carry out the financial analysis using the originally assumed construction costs.
In the case of an actual proposed road concession, a feasibility (or pre-feasibility) study would need to be carried out and relatively precise estimates would be done for all key parameters of proposed project. In our particular case, the following project related data will be assumed for the analysis of the three hypothetical scenarios:

- Concession life: 30 years
- A range of construction cost during the 3-year investment phase of the contract: €100 million; €200 million; €300 million; and €400 million
- Annual O&M cost in subsequent years of the contract: €5 million; €10 million; €15 million; and €20 million. Such values are expressed in terms of present values and would be adjusted for inflation in subsequent years
- Road length: 40 km
- Annual average daily traffic (AADT): 8,000 vpd (80 % cars, 2 % buses, and 18 % trucks)
- Annual traffic growth: 3.0 %
- Capital structure: Debt/Equity ratio, 75/25; Assumed construction subsides: 0 %, 30 %, and 60 % of the capital investments
- Assumed availability payments: 0; €20 million/year; €40 million/year; and €60 million/year
- Nominal interest rate: 8 % per year
- Loan grace period: 3 years
- Debt maturity: 15 years (loan repayment period of 12 years)
- State discount rate (in nominal terms): 10 %

• Inflation: 4 % per year
• Tax rates: (a) Value added tax (VAT): 20 %; (b) Corporate tax: 15 %
• Amortization period: 27 years

It is also assumed that the following targets (or constraints) will have to be met for the project to be able to attract private investors (i.e., parameters deemed applicable to the country and project under consideration):
• Equity Internal Rate of Return (or Return on Equity): ROE ≥ 12 % (in real terms)
• Annual Debt Service Cover Ratio: ADSCR ≥ 1.15

A financial model will be required to analyze the above data. Financial models are analytical tools that allow the user to assess the financial robustness of a project by representing its expected financial performance, including cash flows and returns [6].
There are several toolkits, including financial models, available for the analysis and ex-ante assessment of highway PPP projects [7]. The Government of India [8] released a web-based toolkit for the improvement of the decision-making process in PPP arrangements for the delivery of infrastructure projects. The toolkit can be used for the assessment of highway projects, which is one of five sectors covered.
In 2013, the Federal Highway Administration's (FHWA) Office of Innovative Program Delivery launched a new toolkit, P3-Value, Public-Private Partnership Value-for-Money Analysis for Learning and Understanding Evaluation [9]. Although the main purpose of the toolkit is to help decision makers in the "value-for-money" analysis, it covers other important aspects of PPPs such as risk evaluation and financial feasibility.
Subsequently, the US Department of Transportation [10] published a related Guidebook on Financing of Highway Public-Private Partnership Projects. The World Bank, supported by the Public-Private Infrastructure Advisory Facility (PPIAF), developed a Toolkit for Public-Private Partnership in Roads and Highways [11] to assist policy makers in implementing procedures to promote private sector participation and financing in roads. The Toolkit includes financial models (in graphical and numerical formats) that can be used for the financial assessment of PPP toll roads.
Based on the World Bank/PPIAF Toolkit toll road graphical financial model, a model was developed to assess the financial feasibility of hybrid PPP projects, that is, projects involving both tolling and availability payments [12]. Because of its relevant features, in particular the combination of tolling, availability payments, and construction subsidies, such model was selected for using in this paper.
Using the input data discussed above, the hybrid financial model was deployed to estimate the minimum required availability payment and/or construction subsidy to attract private investors, while keeping the toll rate at an affordable level. As a first step, we will assume an availability payment and construction subsidy equal to zero and estimate the required toll rate.
Application of the model to the four assumed construction (and O&M) cost scenarios shows that the following weighted average toll rate (WATR) would be required:
a. Low construction cost scenario: WATR = €7.0/vehicle;
b. Medium construction cost scenario: WATR = €13.9/vehicle;
c. High construction cost scenario: WATR = €20.8/vehicle, and
d. Very high construction cost scenario: WATR = €27.7/vehicle.

In case there are uncertainties regarding the parameters used to derive such toll rate, the financial model can be easily rerun to carry out a sensitivity analysis [12].
Toll affordability levels are usually expressed in terms of the maximum toll rate that passenger car drivers are willing and able to pay, expressed in €/car-km. In the absence of specific

studies and surveys for the proposed toll road (e.g., motorway), such as willingness to pay study, we will assume the following range of maximum affordable unit toll rates: (i) €0.03/car-km; (ii) €0.05/car-km; and (iii) €0.07/car-km.
The relationship between the weighted average toll rate and the toll rate for cars, trucks and buses can be written as [13]:

$$WATR=(PC\ TRc+PB\ TRb+PT\ TRt)/100 \tag{1}$$

where:
WATR is the weighted average toll rate per vehicle;
PC, PB and PT are the percentages of cars, buses and trucks in the traffic flow;
TRc, TRb and TRt are the toll rates for cars, buses and trucks.

Usually, the toll rate for a commercial vehicle is equal to the toll rate for cars times the vehicle's number of axles. Based on the above and the estimated traffic composition, the following relationships between toll rates for different types of vehicles will be assumed as representative of the proposed motorway:
• Average bus toll rate = 2 x car toll rate
• Average truck toll rate = 3 x car toll rate

The average traffic flow composition on the proposed motorway, as indicated above, will be assumed as: PC, 80 %; PB, 2 %; and PT, 18 %. Accordingly, Equation (1) can be re-written as:

$$WATR=(80TRc+2\ 2\ TRc+18\ 3\ TRc)/100$$

or

$$TRc=WATR/1.38 \tag{2}$$

WATR was computed by the model for the four investment scenarios, as €7.0/vehicle, €13,9/vehicle, €20.8/vehicle, and €27.7/vehicle, respectively for the low, medium, high, and very high-construction cost scenarios. Consequently, for the proposed road concession, the required toll rate per car, for each investment scenario, using Equation (2), would be:
a. Low construction cost scenario: TRc = €5.1/car;
b. Medium construction cost scenario: TRc = €10.1/car;
c. High construction cost scenario: TRc = €15.1/car; and
d. Very high construction cost scenario: TRc = €20.1/car.

As the total length of the proposed motorway section is 40 km, as indicated before, the unit toll rates, in Euro per car-km, will be:
a. Low construction cost scenario: Unit TRc = €0.13/car-km;
b. Medium construction cost scenario: Unit TRc = €0.25/car-km;
c. High construction cost scenario: Unit TRc = €0.38/car-km; and
d. Very high construction cost scenario: Unit TRc = €0.50/car-km.

If a required unit toll rate is higher than the maximum affordable toll rate, there is a need for an availability payment and/or construction subsidy to make such project financially viable. As indicated above, we are assuming three levels of maximum affordable unit toll rates, representing, respectively, a pessimistic, most likely, and optimistic scenario:
• €0.03/car-km;
• €0.05/car-km; and
• €0.07/car-km.

When the required toll rate is higher than the maximum affordable toll rate, there is an affordability gap. For example, if the required toll rate is €0.08/car-km and the maximum affordable toll rate is €0.05/car-km, the affordability gap would be €0.08/car-km minus €0.05/car-km or €0.03/car-km. We will now discuss how affordability gaps can be bridged with a combination of availability payment and construction subsidy.
We can use the available hybrid financial model (previously discussed) to estimate how much availability payment and/or construction subsidy would be required to meet the financial constraints adopted, i.e., a minimum ROE of 12 % and a minimum ADSCR of 1.15.
Table 1 shows the required car toll rate for availability payments varying from zero to €60 million/year and construction subsidies varying from 0 to 60 % of the total construction cost. For example, for an availability payment of €20 million and a construction subsidy of 30 %, the minimum required toll rate would be €0.06/car-km for the medium construction cost scenario.

Table 1 Estimated relationship between availability payment, construction subsidy, and the required toll rate (€/car-km) for the proposed motorway project

Construction cost scenario	Construction subsidy (%)	Availability Payment (€ million/year)			
		0	20	40	60
Low (€100 million; annual O&M cost: €5 million)	0	0.127	0.002		
	30	0.091			
	60	0.065			
Medium (€200 million; annual O&M cost: €10 million)	0	0.252	0.127	0.004	
	30	0.181	0.060		
	60	0.130	0.031		
High (€300 million; annual O&M cost: €15 million)	0	0.377	0.254	0.129	0.005
	30	0.272	0.147	0.040	
	60	0.194	0.096		
Very high (€400 million; annual O&M cost: €20 million)	0	0.502	0.379	0.254	0.130
	30	0.361	0.237	0.120	0.020
	60	0.259	0.159	0.062	
Required Unit Toll Rate per Car (€/car-km)					

Once the affordable toll rate has been established for a proposed motorway project, the road agency has a choice regarding the level of construction subsidy it can offer. For example, in case the agency can count on a loan from an international finance institution (IFI), the construction subsidy could be established based on the amount of the expected loan (i.e., the road agency could use the loan to pay for the construction subsidy) and let the bidders compete on the required level of annual availability payment.
PPP is a more market-oriented project delivery structure, compared to traditional public road construction contracts, with greater private sector involvement, whose preparation usually takes longer than traditional projects. Nevertheless, the preparation time could be shortened with a dedicated and well-prepared team taking the lead role in all the required preparatory steps. This would be particularly relevant when the acute phase of the Covid-19 crisis is over, and more infrastructure, including road works, will be needed for economic recovery.

4 Conclusions

Attracting private financing to road projects, through some form of public-private partnership (PPP), would be a means toward greater investments to keep road infrastructure in acceptable condition and carry out required expansions in a context of public budget constraints. In Southeast Europe there is an average cost increase in traditional road works contracts (i.e., cost overrun) of about 18 %. Assuming such cost overrun would occur in a traditional road works contract, it seems fair to assume that the construction of a motorway section would cost, if implemented as a PPP project (where there is no provision for variation orders), about 15 % less than if implemented by the traditional bill of quantities approach.
Moreover, additional benefits may be introduced by private sector efficiency gains. It is recommended that countries select road sections most likely to be adequate for implementation as PPP and carry out a preliminary financial assessment to identify a potential PPP pipeline. Because of relatively low traffic levels on some proposed motorways and expressways, it is anticipated that a combination of government support (e.g., capital grants and/or availability payments) and toll collection would be required to generate enough revenues to attract private partners to compete for such PPP projects.
The quantitative financial assessment of a hypothetical PPP project, with a range of estimated construction costs, shows that the project could attract private investors with a combination of availability payments and construction subsidies to keep the toll rates at affordable levels.

References

[1] PPP Reference Guide, Version 3.1.World Bank, Washington D.C., USA, 2017.

[2] Private Participation in Infrastructure (PPI) Annual Report, 2019., World Bank, Washington D.C., USA, 2020.

[3] Saner, P., Gillespie, F., Kerr, L., Lev, C.: Closing the Infrastructure Gap: Mobilising Institutional Investment into Sustainable, Quality Infrastructure in Emerging Markets and Developing Economies (EMDEs), Swiss Re Institute and Global Infrastructure Facility (GIF), 2021.

[4] Mladenovic, G., Queiroz, C.: Assessing the Financial Feasibility of Availability Payment PPP Projects, Proceedings, T&DI Second Congress 2014, June 8-11, 2014, Orlando, Florida, USA, pp. 602-611

[5] Alexeeva, V., Queiroz, C., Ishihara, S.: Monitoring Road Works Contracts and Unit Costs for Enhanced Governance in Europe and Central Asia, Transport paper series No. TP-33, World Bank, Washington, D.C., USA, 2011.

[6] Benchmarking Infrastructure Development: Assessing Regulatory Quality to Prepare, Procure, and Manage PPPs and Traditional Public Investment in Infrastructure Projects. World Bank, Wahinston D.C., USA, 2020.

[7] Vajdic, N.: Stochastic model for the assessment of acceptable toll rate. PhD Dissertation, University of Belgrade, Faculty of Civil Engineering, Belgrade, Serbia, 2016.

[8] PPP Toolkit for Improving PPP Decision-Making Processes, Public Private Partnerships in India, Department of Economic Affairs, Ministry of Finance, Government of India, 2010.

[9] P3-Value: Financial Assessment Tool User Manual. Innovative Program Delivery, U.S. Department of Transportation, Federal Highway Administration (FHWA), Washington, D.C., USA, 2013.

[10] Guidebook on Financing of Highway Public-Private Partnership Projects. U.S. Department of Transportation. Washington, D.C., USA, 2016.

[11] Policy Guidelines for Managing Unsolicited Proposals in Infrastructure Projects. PPIAF (Public – Private Infrastructure Advisory Facility), Washington, D.C., USA, 2017.

[12] Mladenovic, G., Queiroz, C.: Enhancing the Financial Feasibility of PPP Projects with Hybrid Funding, International Conference on Traffic and Transport Engineering – ICTTE, 27-28 September 2018, Belgrade, Serbia, pp. 592-599

[13] Queiroz, C., Mladenovic, G.: A Review of Tools for Project Financial Assessments. AIIT 2nd International Congress on Transport Infrastructure and Systems in a changing world (TIS ROMA 2019), Rome, Italy, 23-24 September 2019.

CONCESSION AS ROAD INFRASTRUCTURE FINANCING MODEL IN BOSNIA AND HERZEGOVINA

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Abstract

The apparent slowdown in the construction of road infrastructure in Bosnia and Herzegovina for a number of years necessarily requires a review of the existing and the analysis of new models of financing the road infrastructure construction in Bosnia and Herzegovina. The existing mainly credit-based financing models, have become increasingly exhausted over the years, and the interest of other Western Balkan countries in the EU funds clearly indicate that Bosnia and Herzegovina may expect only a fraction of the total infrastructure financing to be made to it through some of the European funds. This shows the undeniable need for analyzing and applying other models of road infrastructure financing, without additional borrowing, through a concession as a model applied in far more developed, and economically and financially more stable societies than the one in Bosnia and Herzegovina. The aim of this paper is to demonstrate that the undoubtedly complex socio-political arrangements in Bosnia and Herzegovina, in terms of the competencies and legal framework, cannot justify the delays in the construction of the road infrastructure in Bosnia and Herzegovina. Moreover, it may pose a challenge to engineers to seek possible new options.

Keywords: concession, financing, road infrastructure, motorway, Corridor Vc

1 Introduction

Considering the financial requirements, building infrastructure in general, with a particular emphasis on motorways, is a challenge for every country, and developed or developing countries have been involving the private sector in the design, construction, management and maintenance of highways for more than two centuries, whereas the countries in the region, such as Croatia, have used this practice since 1995.
The main hypothesis of this paper is to prove that concession, road infrastructure financing model, is absolutely unused potential in Bosnia and Herzegovina, and the aim of the paper is to present the situation in this field.
With the exception of the Introduction, the paper consists of six Chapters, the first of which presents the state of public debt in Bosnia and Herzegovina, the second presents the strategic transport framework at the level of Bosnia and Herzegovina, and the third concerns the Motorway on the Corridor Vc as the Bosnia and Herzegovina's strategic project.
The fourth Chapter sets out the legal framework concerning concessions in Bosnia and Herzegovina, and the fifth Chapter describes concessions within the road sector in Bosnia and Herzegovina, while the Conclusion of the paper is in Chapter six.

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2 State of public debt in Bosnia and Herzegovina

According to the Information on the state of public debt in Bosnia and Herzegovina as of 30 June 2017, prepared by the B&H Ministry of Finance and Treasury, the financial indicators related to Bosnia and Herzegovina are as follows:

- the total public debt of Bosnia and Herzegovina as of 30 June 2017, amounted to KM 11,708.33 million, of which foreign debt amounted to KM 8,075.89 million or 68.98 %, while the domestic debt amounted to KM 3,632.44 million or 31.02 %;
- compared to 31 December 2016, Bosnia and Herzegovina's debt has decreased by KM 389.25 million or 3.22 %, of which foreign debt decreased by KM 471.70 million or 5.52 %, whereas domestic debt increased by KM 82.45 million or 2.32 %;

In the total public debt, the Federation of Bosnia and Herzegovina participates with 53.38 %, Republic of Srpska with 45.69 %, Brčko District of Bosnia and Herzegovina with 0.29 % and Institutions of Bosnia and Herzegovina with 0.64 %

- In terms of foreign debt, the World Bank – IDA (The International Development Association - IDA, is the part of the World Bank that helps the world's poorest countries. IDA aims to reduce poverty by providing loans (called *credits*) and grants for programs that boost economic growth, reduce inequalities and improve people's living conditions. http://ida.worldbank.org/about/what-ida) and the International Bank for Reconstruction and Development - IBRD account for 33.01 %, European Investment bank - EIB 21.84 %, International Monetary Fund 10.45 %, Paris Club 8.6 % and the European Bank for Reconstruction and Development 7.17 %, which makes 81.08 % of the total foreign debt;
- Foreign debt servicing in the first half of 2017 amounted to KM 446.54 million, of which KM 384.51 million or 86.11 % relates to principal, and KM 62.03 million or 13.89 % relates to interest.
- In the total amount of external debt servicing, the Federation of Bosnia and Herzegovina participates with 64.37 %, Republic of Srpska with 34.86 %, while the Institutions of Bosnia and Herzegovina and Brčko District participate with 0.42 % and 0.35 % respectively.

According to the international financial institutions criteria, Bosnia and Herzegovina does not fall into the category of highly indebted countries, since the participation of Bosnia and Herzegovina's public debt in Gross Domestic Product (GDP) is 36.74 % (Information on public debt of Bosnia and Herzegovina, B&H Ministry of Finance and Treasury, 30 June 2017).
However, it is evident that the problem of Bosnia and Herzegovina, as well as of the countries created by the breakup of the former Yugoslavia, is the economic growth based on domestic consumption, which in the absence of its own and competitive production leads to an increase in imports and a balance of payments deficit, financed by new borrowing.
From the foregoing, it may be concluded that further long-term borrowing has a short-term effect, which is resorted to by almost all political options in Bosnia and Herzegovina, regardless of the duration of the mandate, knowing that they will not be participants in the government's debt collection.
Projects, such as the construction of hospitals, schools, kindergartens, in their structure and purpose require not only budgetary but also financing through some form of borrowing. But in the long run, when large and financially demanding infrastructure projects are at stake, permanent borrowing is certainly not a quality solution.

3 Strategic framework for transport at the level of Bosnia and Herzegovina

At the proposal of the B&H Ministry of Communications and Transport and the approval by the Council of Ministers of Bosnia and Herzegovina, followed by the adoption by the House of Representatives of the B&H Parliamentary Assembly on 30 July 2015, the House of Representatives of the B&H Parliamentary Assembly adopted the 2015-2030 Framework Transport Policy of Bosnia and Herzegovina (Official Gazette of Bosnia and Herzegovina, 62/15).
The Framework Transport Policy of Bosnia and Herzegovina set a multi-annual framework for the transport infrastructure development in Bosnia and Herzegovina and created the preconditions for the development of the Strategy and Action Plans.
Also at the proposal of the B&H Ministry of Communications and Transport, following the adoption and integration of sectoral strategic documents of the Entities and the Brčko District of B&H, on 14 July 2016, the B&H Council of Ministers adopted the Decision on the adoption of the 2016-2030 Framework Strategy for Transport of Bosnia and Herzegovina (Official Gazette of Bosnia and Herzegovina, 71/16), , the planning document for the transport and infrastructure network in Bosnia and Herzegovina, which contains structural proposals for the development of the transport sector and programs for capacity upgrading for the purpose of alignment with the long-term objectives and strategic documents of the European Union in the transport sector.
With the adoption of the B&H Framework Transport Policy and the B&H Framework Transport Strategy, the preconditions have been created for financing the construction infrastructure projects by the European Union, which has earmarked one billion euros for in next five years for the infrastructure connectivity of the Western Balkans region under the Connectivity Agenda launched at the Western Balkans High-level Summit in Berlin in 2014.

4 Motorway on the Corridor Vc as Bosnia and Herzegovina's strategic project

Corridor Vc is one of ten Pan-European Transport Corridors on the Road and Rail Transport Networks agreed by the United Nations Economic Commission for Europe and the European Union Conference of Transport Ministers.
The Network of Corridors has been set up for the purpose of smooth operation of the international traffic on the European continent, better connectivity of the EU Member States and traffic between Europe and Asia.
The Corridors were defined in Prague in 1991 and supplemented at the Second Pan-European Transport Conference held in Crete in March 1994 and at the Third Conference held in Helsinki in 1997, the reason why these Corridors are also referred to as the *Crete Corridors* or the *Helsinki Corridors*.
These ten Corridors connect Europe from the Atlantic to the Volga and from Scandinavia to the Mediterranean Sea.
At the session held on 7 March 2006, the B&H Parliamentary Assembly declared the Corridor Vc Construction Project as the project of strategic interest for the entire territory of Bosnia and Herzegovina, and the B&H Presidency, by its acts, ordered the B&H Council of Ministers to undertake and accelerate all the necessary activities in connection with the Project, while respecting the competences of the B&H Ministry of Communications and Transport, as defined in Article 10 of the Law on Ministries and Other Bodies of Administration of Bosnia and Herzegovina (Official Gazette of Bosnia and Herzegovina, 5/03, 42/03, 26/04, 42/04, 45/06, 88/07, 35/09, 59/09, 103/09, 87/12, 6/13, 19/16).

Figure 1 Network of Pan-European Corridors (https://courrierdeuropecentrale.fr/wp-content/uploads/2013/11/)

Considering the legislative framework, it is also important to emphasize that the Law on the Highway on Corridor Vc has been in force in the Federation of Bosnia and Herzegovina since 2013. (Official Gazette of the Federation of Bosnia and Herzegovina, 8/13).
About 21 years after the Helsinki Pan-European Transport Conference, and 12 years after it was declared a project of strategic interest for Bosnia and Herzegovina, of the total 335 km of the Vc Corridor Vc route passing through Bosnia and Herzegovina, approximately 100 km of the Corridor Motorway has been constructed and, except for the Svilaj - Odžak section, all the parts are operational. With this dynamic, it will take another 20 to 25 years to complete the entire project, which is unacceptable from any point of view and it points to a number of problems, directly related to the construction planning and implementation.
The aforementioned just over 100 km of the motorway was funded through different methods.
In the series of financing models for strategic infrastructure projects, the Western Balkans Investment Framework (WBIF) and The Connecting Europe Facility (CEF) stand out in particular. A common feature of WBIF and CEF is that both instruments apply not only to Bosnia and Herzegovina, but to the countries of the region when it comes to WBIF, and to CEF, the EU Member States in addition to the countries of the region.
The movement of the countries of the region towards EU membership, as well as the intention of the European Union to undoubtedly reduce the financial space of Bosnia and Herzegovina through its projects to regionally support the stabilization and association, which imposes the need to reconsider the future and seek new models and ways of financing strategic infrastructure projects in Bosnia and Herzegovina.

5 Legal framework for concessions in Bosnia and Herzegovina

The Law on Concessions of Bosnia and Herzegovina (Official Gazette of Bosnia and Herzegovina, 32/02 i 56/04), The Law on Concessions of the Federation of Bosnia and Herzegovina (Official Gazette of Bosnia and Herzegovina, 32/02 i 56/04), The Law on Concessions of Republic of Srpska (Official Gazette of the Republic of Srpska, 59/13;) and the Law on Concessions of the Brčko District of Bosnia and Herzegovina (Official Gazette of the B&H Brčko District, 41/06, 19/07 i 02/08) set the legal framework for concessions in Bosnia and Herzegovina.
From the point of view of the position of the Corridor Vc Motorway, apart from the Brčko District of Bosnia and Herzegovina, this paper will review the activities of the Commissions for

Concessions, the first of which is the Commission for Concessions of Bosnia and Herzegovina established by the Law on Concessions of Bosnia and Herzegovina (Official Gazette of Bosnia and Herzegovina, 32/02) and the Law on Amendment to the Law on Concessions of Bosnia and Herzegovina (Official Gazette of Bosnia and Herzegovina, 56/04), as an independent regulatory body of the B&H Parliamentary Assembly in the process of awarding concessions. Analysing the officially available data, except for the Decisions on the procurement of vehicles, fuel or equipment, it is evident that in its 16 year existence, the Commission for Concessions of Bosnia and Herzegovina has made only one decision related to its purpose, namely the Decision on approval of the economic feasibility study (Official Gazette of Bosnia and Herzegovina, 26/07), by which the Grantor, the B&H Ministry of Communications and Transport of, has been granted the Economic Feasibility Study for the *Corridor Vc Motorway Project*. As for 2016, as the last year for which an official work report is made available, it is evident that no requests for regulatory approval have been submitted to the Commission during the reporting period by the state-level Ministries, nor has the Commission received any new application related to the self-initiated bid of the interested bidder/concessionaire by a bidder or by the competent Ministry for which the Commission is to issue an approval (Official Gazette of Bosnia and Herzegovina, 26/07).

How remarkable this data is also shows the fact that the World Bank and SIGMA/OECD have made a number of recommendations to the authorities of Bosnia and Herzegovina in the previous period, including the analysis of the situation of concessions and PPP investments in Bosnia and Herzegovina, including*: In order to attract domestic and foreign investment for the purpose of developing the infrastructure, the authorities in Bosnia and Herzegovina must create a modern and efficient system of awarding concessions.* (Review of financial oversight and procurement in state-owned enterprises and concessions award in B&H, 2007.).

One of the recommendations in the documents produced by SIGMA/OECD and the EU states that it is essential to establish a clear and meaningful legal framework governing the award of concessions and public-private partnerships in Bosnia and Herzegovina in such a way as to improve the development of important infrastructure and other projects in the form of model concessions and public-private partnerships.

True, open and fair competition for the award of concessions and public-private partnerships is a key element in enabling Bosnia and Herzegovina to achieve *value for money* in the use of public funds (Review of financial oversight and procurement in state-owned enterprises and concessions award in B&H, 2007.).

The SIGMA/OECD comprehensive report, which would in many countries, as was often the case, form the basis for the reform changes, was not even discussed by the B&H Council of Ministers, although it was submitted by the Commission together with its conclusions on measures for improvement the concessions and concession market in Bosnia and Herzegovina.

In its 2016 Work Report (FB&H Concessions Commission Activity Report 2016, May 2017), the Commission for Concessions of the Federation of Bosnia and Herzegovina states, among others: *In 2016 (as in the entire period of the existence of the Law as of 2002 to the present), no Ministry initiated any proposal, nor did it identify any potential concessions, despite persistent insistence, oral and written communications from the Commission for Concessions to the Government of the Federation of Bosnia and Herzegovina, and despite the conclusions reached by the Parliament of the Federation of Bosnia and Herzegovina when adopting the annual reports of the Concessions Commission Federation of Bosnia and Herzegovina*(FB&H Concessions Commission Activity Report 2016, May 2017, p 8.).

The same report also states that: ... *all previous Concession Contract were concluded on the basis of a self-initiated bids, even though the provisions of Article 28 of the Concession Law stipulate that a self-initiated bid for concession allocation may only be submitted under an urgent and exceptional procedure.*.(FB&H Concessions Commission Activity Report 2016, May 2017, p 9.).

Based on self-initiative bids, the Federation of Bosnia and Herzegovina has recorded concession contracts for: *Vranduk* and *Janjići* Hydro Power Plants on the Bosna River, a concession for the use of water to supply Travnik and future water use through regional water supply lines for the municipalities of Travnik, Novi Travnik, Vitez, Busovača and Zenica, Public Company RV *Plava voda* d.o.o. Travnik, concession for the extraction of mineral water at the site of Crni vrh, spring Kiseljak, *OAZA* Tešanj, concession for the use of water from the spring Krušcica, Babina rijeka, Strmešnjak and Klopče, Public Utility Company *VIK Zenica* d.o.o. Zenica, concession for water pumping at the Wells SB2 and SB3, *Spa Sanska Ilidža* in Sanski Most, *Spa Sanska Ilidža* d.o.o. Sanski Most, concession for the exploitation of thermal water sources from the Šumatac wells, in the village of Donji Purići, Municipality of Velika Kladuša, *CIPREX* d.o.o. Velika Kladuša, concession for the use of the water spring of Zatoča, Tarevčica and seven wells in the Municipality of Kladanj, Toplice and Sprečko polje in the Municipality of Živinice, Dobrina in the Municipality of Tuzla for public water supply to the municipalities of Tuzla, Živinice and parts of the Lukavac and Kladanj Municipalities.
Therefore, the current concessions in the Federation of Bosnia and Herzegovina, based on self-initiated offers, solely relate to the concessions for two hydropower plants and water abstraction, without a single infrastructure project.
Considering the allocation of potential concessions in road infrastructure, in the case of the Federation of Bosnia and Herzegovina, it is evident that the Government of the Federation of Bosnia and Herzegovina decided to build the Lašva-Donji Vakuf Motorway section through a concession.
However, point VI of the FB&H Government Decision of 2014 (Decision no. 2162/2014 of 4 December 2014 on determining the general interest in constructing the Lašva - Vitez - Donji Vakuf expressway), stipulates that the construction of the Lasva - Vitez - Donji Vakuf expressway section is to be started even though the project had not undergon the concession award procedure under the aforementioned Government Decision of 2010, which had not been repealed. For unknown reasons, the FB&H Ministry of Transport and Communications did not proceed with the implementation of the aforementioned decisions, and by the end of 2016, no request was submitted to the Commission for Concessions of the Federation of Bosnia and Herzegovina regarding the granting of certain consents and approvals in accordance with the Law on Concessions (FB&H Concessions Commission Activity Report 2016, May 2017, page 32).
Furthermore, by a Decision (Decision no. 1775/2014 of 10 September 2014 concerning the procedure for awarding the concession for the Corridor Vc motorway section "Karuše – Poprikuše", (Official Gazette of the FB&H, 82/14)) of 2014, the FB&H Government determined that the procedure for awarding the concession for the *Karuše-Poprikuše* section on the Corridor Vc Motorway is to be initiated.
No response has ever been received to the comments and suggestions of the B&H Commission for Concessions dated end of 2014.
Unlike in the Federation of Bosnia and Herzegovina, in 2016 a total of 515 (five hundred and fifteen) concluded contracts, annexes to contracts, requests, resolutions, certificates, various reports, data and other receiving documentation were referred to the RS Commission for Concessions by the competent Ministries, the RS Government, concessionaires, news agencies, citizen associations and a significant number of other legal and natural persons (Activity Report and Financial Report for 2016, Commission for Concessions of the Republic of Srpska, Banja Luka, 04/2017).
The decisions of the RS Commission for Concessions concerned various projects related to the construction and use of energy facilities: hydroelectric power plants, wind power plants, thermal power plants, mineral resources facilities: research and exploitation of coal, technical building stone, quartz sand, thermal mineral water, trade and tourism related objects, and those in the field of agricultural land use and water resources. Although in the field

of finance and transport and communications, the RS Commission for Concessions had no activities due to lack of receiving any related request for a one-time concession fee for use rights and concession fees for the use of natural resources or provision of services in 2016 a significant amount of KM 44,382,272.89 was paid into the budget of the Republic of Srpska (Activity Report and Financial Report for 2016, Commission for Concessions of the Republic of Srpska, Banja Luka, 04/2017, page 51).
Considering that in 2014, the total of KM 22,193,380.96 was paid as concession fees, and KM 29,289,907.27 in 2015, an almost 100 % increase was recorded in the two-year period.

6 Road concessions in Bosnia and Herzegovina

Given that a detailed review of the situation regarding concessions in the EU and the countries of the region would require serious consideration, and the focus of this paper is on concessions as a possible model for financing the construction of road infrastructure in Bosnia and Herzegovina, this chapter is omitted in this paper.
Namely, analyzing the dynamics of the construction of the Corridor Vc Motorway through Bosnia and Herzegovina, it is evident that out of the total of 335 km (The first 100 kilometres, PC Autoceste FB&H, Mostar 2015, p. 17), 102 km have been built, which include the Svilaj - Odžak section that has not yet been put into operation.

Figure 2 Motorway on the Corridor Vc through Bosnia and Herzegovina (The first 100 kilometres, PC Autoceste FB&H, Mostar 2015, p. 17)

Unlike in Bosnia and Herzegovina, in the Republic of Croatia, for example, the total of 1,313.8 km of motorways and semi-motorways are operational, which also include a particularly interesting section for Bosnia and Herzegovina A5 Beli Manastir - Osijek - Svilaj, with a total length of 89 km, of which 56 km are operational.
Analyzing the total indebtedness of Bosnia and Herzegovina, the dynamics of construction of the Motorway on the Corridor Vc through Bosnia and Herzegovina, previous indicators in relation to the financial needs of one of the strategic projects in Bosnia and Herzegovina, the main regulatory issues (Framework Transport Strategy B&H, Official Gazette of B&H, 00000, p. 000), sources of financing in the Project List (Framework Transport Strategy B&H, Official Gazette of B&H, 00000, p. 000), the author of this paper attempts a personal initiative look for a path to a solution.

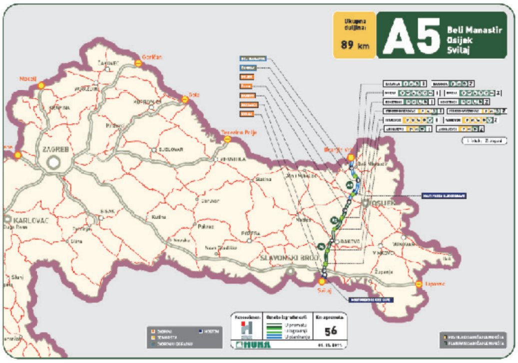

Figure 3 Motorway on the Corridor Vc through the Republic of Croatia (http://www.huka.hr/mreza-autocesta)

Article (3) of the draft decision, the author of this paper proposes a concession model *Build, Operate, Transfer*, the so-called BOT model (BOT-model (build, operate and transfer), is a contemporary legal type of building large economic and infrastructure facilities, where a contractor is entitled, on the basis of a contract, to finance and build a facility, and to operate and economically exploit such facility for a specified period of time, upon the expiry of which such facility is transferred to the investor. http://www.poslovni.hr/leksikon/bot-model-156), with the possibility of applying similar types of concessions depending on the interest of potential concessionaires.
The author finds the legal basis for proposing and adopting a decision by the B&H Council of Ministers on the subject, scope and type of concession on the Corridor Vc Motorway section Tarčin – Ovčari in Article 4 of the B&H Law on Concessions (B&H Law on Concessions, Official Gazette of B&H, 32/02 and 56/04), which states that the B&H Council of Ministers takes a decision on the type, subject and scope of the concession awarded, and is approved by the B&H Parliamentary Assembly. The competence of the B&H Council of Ministers of Bosnia is regulated in Article 1(2) of the said Law, according to which concessions may be granted to domestic and foreign legal entities:

- in sectors within the jurisdiction of Bosnia and Herzegovina under the Constitution of Bosnia and Herzegovina and the laws of Bosnia and Herzegovina;
- in case of representation of the international subjectivity of Bosnia and Herzegovina,
- when the concession well extends to the Federation of Bosnia and Herzegovina and the Republic of Srpska.

Following the previously approved initiatives, the author further proposes to take a new decision, this time on the subject, scope and type of concession on the Corridor Vc Motorway section Karuše (Medakovo) - Rudanka, but based to the so called DBOT model (*Design, build, operate transfer*). In addition to the BOT model, this model also involves designing, with the possibility of applying other similar types of concessions, depending on the interest of potential concessionaires.
After requesting and obtaining a positive opinion from the Legislative Office of the B&H Council of Ministers and the B&H Commission for Concessions, the opinion of the B&H Ministry of Finance and Treasury is sought, to which there is no reply, even after six urgencies.
Although the B&H Law on Concessions stipulates that: ″In a case of joint jurisdiction of Bosnia and Herzegovina and/or the Federation of Bosnia and Herzegovina and/or Republic of Srpska and/or Brčko District of Bosnia and Herzegovina for the award of concessions, the competent authorities agree on the conditions and form of the award of the concession″ (B&H Law on Concessions, Official Gazette of B&H, 32/02 and 56/04), except the opinion of the Legislative Office of the B&H Council of Ministers and the B&H Commission for Con-

cessions at the end of 2017, the Ministry of B&H Communications and Transport received a document from the RS Ministry of Transport and Communications titled *Protest Concerning the Proposal for Taking a Decision on the Subject, Scope and Type of Concession on the Corridor Vc Motorway Section Karuše (Medakovo) - Rudanka*, requesting the withdrawal of the proposed decision from further proceedings.
Therefore, after almost a year and a series of urgencies to the B&H Ministry of Finance and Treasury, attempts to change the current dynamics of the construction of the Motorway on the Corridor Vc by changing the financing model through concessions, the phase of attempts is still ongoing, and it is clear that, after the positive opinions by the B&H Legislative Office and the B&H Commission for Concessions, the comment in the form of the *protest* by the RS Ministry of Transport and Communications is irrelevant.

7 Conclusion

By analyzing the aforementioned, unlike in incomparably more developed and economically more stable countries that generate multimillion-dollar revenues on the basis of concessions and build infrastructure facilities that, upon construction and operation, are being returned to the state, there is an obvious resistance to concessions in Bosnia and Herzegovina as a model for financing the construction of road infrastructure in Bosnia and Herzegovina. The views expressed in this paper are the author's own and may differ from the views of colleagues in the B&H Ministry of Communications and Transport and the official positions of the B&H Ministry of Communications and Transport. It is clear that the construction of road infrastructure through credit lines is necessary for some sections that do not have sufficient PGDP and PLDP (Average Annual Daily Traffic (AADT) and Average Summer Daily Traffic (ASDT)), although in such cases the construction can be implemented through concessions, with guarantees or some kind of government involvement. In the concession road construction model, the profit for the state is not necessarily visible in the budget, since the construction of motorways without long-term borrowing or investment is a kind of benefit.
Furthermore, permanent and long-term borrowing will undoubtedly as consequence lead to a fiscal deficit in the long run. Depending on the positions of the future convocations of the B&H Council of Ministers of and the Entity Governments, the possible subsequent monetization of motorways, i.e. the conversion of public debt into cash (Croatian language portal, http://hjp.znanje.hr/index.php?show=search_by_id&id=e1lvURA %3D), after their construction through credits, may undoubtedly have only a short-term positive budgetary effects, while the long-term effects are undoubtedly negative.
In trying to analyze the background for resistance to concessions, a number of reasons are noticeable, and since some of them enter the sphere of indications, they are not the subject of this paper.
Without going deeply into indicia, it is clearly easier to completely cease or continue the road construction with micro-steps through continuous borrowing, knowing that the decision makers will not be able to repay such loans, given the time and grace period in loan repaying, when at the time of their maturity and credit collection, they will probably not even be alive.
As stated in the introduction, the undoubtedly complex socio-political organization of Bosnia and Herzegovina in terms of competencies and legal frameworks, cannot be justification for the delays in road infrastructure construction in the country, but must be a challenge for engineers and political structures to seek possible solutions.
In terms of road infrastructure construction, this paper pointed out some absolutely unexplored potential, one of which is concessions that, with the best engineering intentions, cannot be exploited without support, or at least without obstructions by those structures that in some ways take part in decision making process.

References

[1] Information on public debt of Bosnia and Herzegovina, Ministry of Finance and Treasury of Bosnia and Herzegovina, 30 June 2017.

[2] Official Gazette of Bosnia and Herzegovina, 62/15; 71/16; 32/02 and 56/04; 5/03, 42/03, 26/04, 42/04, 45/06, 88/07, 35/09, 59/09, 103/09, 87/12, 6/13, 19/16; 32/02; 56/04; 26/07

[3] B&H Law on Concessions, Official Gazette of B&H, 32/02 and 56/04

[4] Official Gazette of the Federation of Bosnia and Herzegovina, 8/13; 40/02

[5] Official Gazette of the Republic of Srpska, 59/13

[6] Official Gazette of the B&H Brčko District, 41/06, 19/07 and 02/08

[7] B&H Concessions Commission Activity Report 2016, no. 01-50-1-7-42-1/17 of 10 May 2017.

[8] Review of financial oversight and procurement in state-owned enterprises and concessions award in Bosnia and Herzegovina, 2007.

[9] Concessions and public-private partnerships in Bosnia and Herzegovina, Final Report, 2009.

[10] FB&H Concessions Commission Activity Report 2016.

[11] Decision no. 2162/2014 of 4 December 2014 on determining the general interest in constructing the Lašva - Vitez - Donji Vakuf expressway

[12] Decision no. 1775/2014 of 10 September 2014 concerning the procedure for awarding the concession for the Corridor Vc motorway section ″Karuše – Poprikuše″, Official Gazette of the FB&H, 82/14

[13] Activity Report and Financial Report for 2016, Commission for Concessions of the Republic of Srpska, Banja Luka, 04/2017

[14] The first 100 kilometres, PC Autoceste FB&H, Mostar 2015.

[15] Framework Transport Strategy B&H, Official Gazette of B&H, 71/16

[16] [International Development Association, http://ida.worldbank.org/about/what-ida

[17] Le Courrier D'Europe Centrale: https://courrierdeuropecentrale.fr/wp-content/uploads/2013/11/

[18] Hrvatska udruga koncesionara za autoceste s naplatom cestarine http://www.huka.hr/mreza-autocesta

[19] Poslovni.hr, http://www.poslovni.hr/leksikon/bot-model-156

MODEL FOR ASSESSMENT OF EXTERNAL TRANSPORT COSTS

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Abstract

The term "external effect" can be explain as a condition that occurs when production or consumption activities of an entity affect the welfare of other(s) subject(s) without having to pay compensation for that impact. The major difficulty for determining external costs is that they cannot be confirmed through the application of market laws and well-known market analysis with interactive effects of demand and supply. The transport greatly affects the quality of life of people, flora and fauna. The interest of studying transport externalities is objective of several researches and special attention is given to how reduce these negative externalities of transport in practice. This paper considers the external transport costs, their significance and their monetary values estimated in relevant EU studies. The methodology for estimating external transport costs as well as specification of a model for assessment of these costs in Republic of North Macedonia is also shown in this paper. The results obtained by this model are discussed and commented.

Keywords: external transport costs, assessment, passenger and freight transport

1 Introduction

The external costs, or negative externalities, are such type of costs when the economic activities of one social or economic entity affect the welfare of other(s) subject(s) or persons without having to pay compensation for that impact. The estimation of these costs is very difficult that they are not market costs and they cannot be determined through the application of market analyses with interactive effects of demand and supply. Transport activities cause external costs that are not directly assumed and paid by transport users but they have an effect on the immediate environment. These costs are submitted by other persons not directly involved in the transport. However, the external costs of transport need to be estimated and expressed in monetary values as well as other costs and benefits in rank to be included in the cost-benefit analyses. This paper will consider the external costs of transport, their significance and their monetary value. EU recommendations and estimations of these external transport costs are the starting point for their assessment in Republic of North Macedonia.

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2 Previous studies and relevant documents

2.1 Previous studies

The International Union of Railways (UIC) is a pioneer in the study and evaluation of external cost of transport in European countries presenting assessments of rail, road, air transport and inland waterways [1]. In 2012, UIC publishes the document Green Transport, Reducing External Costs [2]. The methodology for monetary expression of external cost of transport in this paper uses an approach to determine the value of non-market goods.
The study developed in the HEATCO project under the 6th EU Framework Program 2002-2006 for harmonization of European practices in the estimation of transport costs and project assessment, pays particular attention to estimation of the external cost of transport in the EU countries [3], [4].
Internationalization of External Transport Costs is presented in a study funded by the European Commission, developed by CE Delft and published in the 2008Handbook [5]. This study is a follow-up to the 2006 HEATCO study. The update of this study is carried out in 2011 "to obtain a state-of-art overview of the total, average and marginal external costs of transport in the EU" [6]. The previous Handbook was updated also in 2014 version taking in account new input values [7]. The last edition of the Handbook of the external costs of transport is published in version 2019 by European Commission [8].

2.2 Categories of external cost of transport

The external costs of transport frequently assessed in the previous studies and published Handbooks concern the following five core cost categories: Traffic Accidents, Air pollution, Climate change, Noise and Congestion. External costs of transport are estimated for four modes of transport and they are separated for passenger and freight transport. The four modes of transport are following:
- Railway transport for passenger and freight and for diesel and electric engine of traction.
- Road transport for passenger: cars, buses and coaches and motorbikes/mopeds; Road transport for freight: light vehicles (LDV), heavy vehicles (HDV).
- Air transport for passenger.
- Inland waterways for freight transport.

The most important impact for estimation of external costs of transport has the road transport sector, because it is responsible for the majority of external costs.

2.3 Methodology used for assessment of external transport costs in relevant studies

The estimation of external costs includes several uncertainties, but there is a wide consensus for methodological approach for their assessment. The costs of environmental transport activities cover a wide range of different impacts, including the diverse effects of emissions of a large number of pollutants that have an effect on the human health, materials, ecosystems, flora and fauna. Impacts appear at local, regional, European and global levels. The damages caused by transport activities could be prolonged in future. External costs of transport vary considerably with the characteristics of vehicles, trains, boats or aircraft. HEATCO's scientific research project uses an approach called "Impact Pathway Approach" based on damage cost. Using the concepts of welfare economics, monetary assessment follows the "willingness to pay" approach for valuation of the respective health effects and improving the quality of the environment.

The best practice estimation of congestion costs is based on speed- flow relations, value of time and demand elasticity. For example, the procedure for calculating external costs of transport from air pollution is following:

- Quantification of changes in the emission of pollutants (NOx, SO2, PM2.5/PM10) resulting from the project studied (the project being evaluated) and expressed in tonnes, using the latest national or European emission factors. The future progress of these programs should also be taken into account.
- Classification of emissions according to the amount of emission (near ground surface or high) and local environment (urban - out of urban areas).
- Impact calculation - years of life lost (YOLL) and costs per pollutant.
- Impact Report (YOLL) and Costs.

Parameters to consider for exposure to the population are: emission source size, location - urban and outdoor, location within Europe.

3 Model for assessment of external costs of transport in Republic of North Macedonia

The model for calculating external costs of transport in Republic of North Macedonia is specified according to the methodology and recommendations in previous EU studies and can be presented as follows:

1. Estimation of unit value for external costs of transport by type of externality in a given year:

$$ET_{MYi} = ET_{FUY0} \cdot PPP_{MY0} \cdot A_{GDPRZi} \tag{1}$$

ET_{MYi} – unit value of external cost of transport for a given externality in Republic of North Macedonia in year Y_i

ET_{EUY0} – unit value of external cost of transport for a given externality estimated as EU average in year Y_0

PPP_{MY0} – purchasing power parity indicator in Republic of North Macedonia in relation to the EU in year Y_0 (that indicator is 100 % for the EU)

A_{GDPRYi} – average annual growth of gross domestic product in Republic of North Macedonia between years Y_0 and Y

$$A_{GDPRYi} = (1+p)^i \tag{2}$$

p – average GDP growth rate between years Y_i and Y_0

i – number of years between Y_i and Y_0

2. Estimation of emission quantities of externalities that depend on the transport operation, type of vehicles and their engines, the location of the infrastructure, other geographical and time factors. These estimated quantities can be expressed by the following equation:

$$QE_{MYi} = \sum Q \cdot E_{MYi,v,s} \tag{3}$$

QE_{MYi} – quantities for a given externality of transport in Republic of North Macedonia in year Y_i

$QE_{MYi,v,s}$ – quantities for a given externality of transport in Republic of North Macedonia in year Y_i obtained from different transport activities, different vehicles and in specific spatial conditions.

3. Estimation of the total external cost of transport for a given externality can be expressed by the following equation:

$$CE_{MYi} = ET_{MYi} \cdot QE_{MYi} \tag{4}$$

CE_{MYi} – total external cost of transport for a given externality in Republic of North Macedonia for year Y_i

ET_{MYi} – unit value of external cost of transport for a given externality in Republic of North Macedonia in year Y_i

QE_{MYi} – quantities for a given externality of transport in Republic of North Macedonia in year Y_i.

4 Estimation of external cost of road transport

4.1 Specific data used in the model

The model presented above needs data concerning the unit values of external costs of transport expressed as average values for EU countries. These data areextracted from the RICARDO-AEA study [7] on specific vehicle types and on urban, suburban and rural environments and refer to year 2010. The unit values of external costs of transport are estimated in €ct/v.km for the following types of vehicles:
• Diesel and petrol cars in European emission standards Euro 0 to Euro 6,
• Buses in European emission standards from Euro 0 to Euro 6,
• Trucks in European emission standards from Euro 0 to Euro 6.

The unit values of external transport costs are adjusted for Republic of North Macedonia using the purchasing power indicator published by Eurostat. According to this indicator (Purchasing Power Standards) Republic of North Macedonia in 2010 was 34 % of the EU-28 average.
To estimate the unit values of external transport costs in a given year different of 2010 they should be weighted also by the average growth rate of GDP from year 2010 to the year of analysis. The assessments in this paper are made for 2015. The average GDP growth in the period 2010-2015 in Republic of North Macedonia is 2.54 %, according to the official data published by State Statistical Office (SSO).
The estimation of quantities of harmful emissions can be made using SSO data for the type of vehicle registered in the country. In addition to these data, it is also necessary to have data on average annual kilometres travelled by type of vehicle and the area of impact separated of urban, suburban or rural places. Some of these data have been estimated from their own studies, and some have been obtained by processing data from SSO.
Data of average annual kilometres travelled by cars on urban, suburban and rural roads and highways do not exist. Since such data are not available, we made assumption that cars travel an average 10000 km/year, of which 70 % are on urban roads and 30 % on suburban, rural roads and highways.
The SSO [9] Publication Transport and Other Services shows that in 2015 on average one bus travelled 79000 km. It is assumed that 70 % of them are made in urban areas, 20 % in suburban areas, and 10 % in highways and rural areas.
The same SSO publication states that trucks in total in 2015 had 860 million kilometres. Assuming that in the same year there were 33237 registered trucks and on average in 2015 one truck travels 25875 km. It is assumed that 30 % of trucks travel distances in urban areas, 20 % in suburban areas, and 50 % in rural areas and highways.

The number of registered vehicles in 2015 in the country is 451724 vehicles. The vehicle flat is very old and the average age of cars is 18.7 years, for buses 18.1 years and for trucks 15.5 years. In 2015 half of the passenger cars and buses have European emission standards with high emissions of harmful substances Euro 0, Euro 1 and Euro 2 (figure 1).
Concerning fuel consumption of motor vehicles, according to SSO data,the 53 % of passenger cars use petrol and 47 % of cars use diesel (table 1).

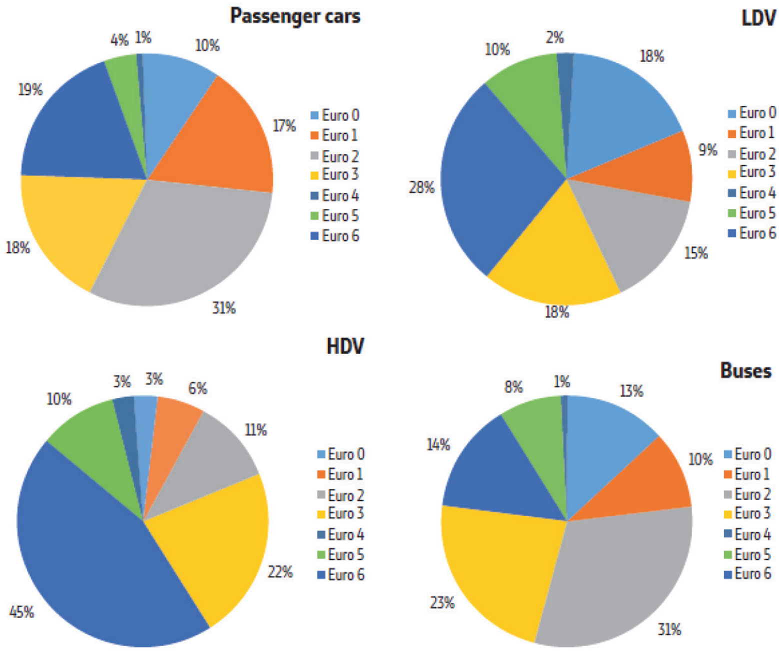

Figure 1 Share of types of vehicles in Euro 0 to Euro 6 emission standards in 2015 in Republic of North Macedonia

Table 1 Number of registred motor vehicles in road transport per type of vehicle and Euro class of motor engines in 2015 in Republic of North Macedonia

Euro Class	Number of cars petrol	Number of cars diesel	Number of trucks	Number of buses
Euro 0	20 150	18 127	997	422
Euro 1	34 256	30 817	1 994	324
Euro 2	62 466	56 195	3 656	1 005
Euro 3	36 271	32 629	7 312	746
Euro 4	38 286	34 442	14 957	454
Euro 5	8 060	7 251	3 324	259
Euro 6	2 015	1 813	997	32
TOTAL	201 504	181 274	33 237	3 242

4.2 Estimations achieved by the model

According to the above assumptions and described model, the estimations of the external transport costs of road transport from air pollution are as follows (table 2):

Table 2 Estimated external costs of road transport from air pollution in Republic of North Macedonia in 2015 in millions of euros

Euro Class	Cars		Buses			Trucks		
	Petrol	Diesel	Urban	Suburban	Rural + highway	Urban	Suburban	Rural + highway
Euro 0	2.70	5.39	3.16	0.54	0.17	1.07	0.45	0.72
Euro 1	1.17	3.45	1.57	0.28	0.09	1.54	0.62	0.99
Euro 2	1.34	5.82	3.95	0.82	0.28	2.23	1.06	1.80
Euro 3	0.45	2.66	2.57	0.51	0.16	3.66	1.71	2.83
Euro 4	0.48	1.94	0.88	0.20	0.07	4.23	2.25	3.95
Euro 5	0.10	0.22	0.44	0.10	0.02	0.65	0.30	0.38
Euro 6	0.02	0.04	0.02	0.00	0.00	0.06	0.01	0.02
Total	6.3	19.5	12.6	2.4	0.8	13.4	6.4	10.7

The estimated external costs of road transport only from air pollution are 72.2 million EUR in 2015.
The external costs of road transport from noise pollution are assessed of 48.7 million EUR in 2015.
The estimated external cost of road transport for climate change in the country is 59.1 million EUR for 2015.
Other external cost of road transport as costs of traffic accidents and traffic congestion are not assessed.

4.3 Comments of estimated results

External costs of road transport from air pollution in urban areas are predominant. Particularly high are external transport costs produced by diesel cars that are in European emission standards Euro 0, Euro 1, Euro 2 or manufactured until 2004. About 50 % of passenger cars and buses have EU standards with high emissions of harmful substances.
About 64 % of external transport costs of road transport from noise pollution are appeared in urban area from car traffic.
Estimations of external costs of transport in EU countries plus Norway and Switzerland in 2008, account about 4 % of these countries' GDP. If we apply the same percentage to Republic of North Macedonia with a GDP of 9072 million EUR in 2015, then the total external costs after this calculation is about EUR 363 million EUR.

5 Conclusion

The official data for transport collected by SSO are not appropriated for estimation of external costs of transport. The new methodology of data collection should be involved in the future to produce solid data for estimation of external transport costs.
The external costs of road transport are predominant in urban areas comparing with rural regions and highways. The very old vehicle fleet in the country and large presence of vehicles in Euro 0, Euro 1 and Euro 2 classes contribute significantly to air pollution. The transport policies have to provide state aid to citizens and transport operators for renewal of vehicles and usage of more environmental friendly cars, buses and trucks.
The external costs of rail transport are not assessed in this paper, but the UIC estimation for 2008 [2] in the 27 EU countries notes that these cost are only 2 % of total external costs of transport. Development of inter modality and favour of rail transport can also contribute to decrease external costs of transport.

References

[1] UIC: Reducing the external cost of transport: the key to environmentally sustainable mobility, Paris: UIC, 1995.

[2] UIC: Greening Transport, Reduce External Costs, 2012.

[3] HEATCO (Developing Harmonised European Approaches for Transport Costing and Project Assessment) EC-FP-6: Economic values for key impacts valued in the Stated Preference surveys, 2006.

[4] HEATCO (Developing Harmonised European Approaches for Transport Costing and Project Assessment), EC-FP-6: Deliverable 5 Proposal for Harmonised Guidelines, 2006.

[5] CE Delft: Handbook on estimation of external costs in the transport sector, Delft, February, 2008.

[6] CE Delft, INFRAS and Fraunhofer ISI: External Costs of Transport in Europe, Update Study for 2008, Delft, November 2011.

[7] Ricardo-AEA: Update of the Handbook on External Costs of Transport, TRT January 2014.

[8] European Commission: Handbook on the external costs of transport, version 2019, Publications Office of the EU, 2019.

[9] State Statistical Office of Republic of North Macedonia (SSO): Publication Transport and Other Services, 2015.

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CHALLENGES IN IMPLEMENTING CURRENT TRACK STANDARDS INTO THE EXISTING INFRASTRUCTURE

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Abstract

The rail industry in the United Kingdom is constantly investing in its infrastructure to make a better experience for passengers by providing more frequent and faster journeys. Large parts of this investment are the renewal of infrastructure that needs upgrading and introduction of new rolling stock which requires strategic and operational planning, multi-disciplinary design input and approvals from Network Rail Route Asset Managers (RAM). With the evolving standards set out by Network Rail it is becoming increasingly difficult to design track alignments without derogations to these standards. The aim of this paper is to gauge industry professionals' opinions on aspects surrounding track design such as their industry experience, their experience on working with software tools, as well as to highlight areas of design difficulty with regards to the Network Rail standards. Furthermore, this paper will explore a case study where the existing and proposed alignments have been analysed and an alternative proposed alignment has been designed in an attempt to eradicate the derogations encountered in the original design. Qualitatively collected data showed that 63 % of sites designed by the respondents contain derogations, mainly connected to the restrictions and limitations in the design standards, as well as the changes to the scope during the project life cycle. Following on from this, the results of the case study highlighted the original accepted proposed design contained six derogations which, in the second proposed alignment, were reduced to three more serious derogations which resulted in the design being rejected.

Keywords: permanent way, track design, derogation, design standards, railways

1 Introduction

The rail industry in the United Kingdom is constantly investing in its infrastructure to make a better experience for passengers by providing more frequent and faster journeys. Large parts of this investment are the renewal of infrastructure that needs upgrading and introduction of new rolling stock which requires strategic and operational planning, multi-disciplinary design input, and approvals from Network Rail (NR) Route Asset Managers (RAM). With the evolving standards set out by NR [1-4], it is becoming increasingly difficult to design track alignments without derogations to these standards and without the use of appropriate design methods. The aim of this paper is to gauge industry professionals' opinions on aspects surrounding track design such as their industry experience, their experience on working with software tools, as well as to highlight areas of design difficulty with regards to the NR standards. Furthermore, this paper will explore a case study where the existing and proposed alignments have been analysed and an alternative proposed alignment has been designed in an attempt to eradicate the derogations encountered in the original design.

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2 Materials and methods

2.1 Expert opinion survey

The quantitative element of this study was addressed by a questionnaire sent out to industry professionals of varied experience levels, with an aim of collating data in a simplistic number format by the use of factual questions and opinion questions where a Likert scale will be applied to the question. In the qualitative element of the survey, the respondents were encouraged to give their view on certain design elements via open-ended questions, as an attempt to determine additional blockers that designers regularly come up against.

2.2 Case study

The quantitative method used in this study was case study research method to fulfil the aim and objectives. The case study addressed a descriptive question, allowed the study of the phenomenon in real-world context, and provided an evaluation [5]. The case study was a detailed design of Elderslie Station in Liverpool with two new proposed alignments: the first was a fully compliant design to [3], while the second was a design that meets most standards with exceptional circumstances which have arisen from designs done in the past, and also looked to highlight issues that current standards have on the re-design of existing infrastructure.

3 Results and analysis

3.1 Expert opinion survey

Thirteen track design experts were surveyed and provided response to the questions set in the expert opinion survey. Figure 1 shows that 23 % of he respondents have over 20 years' experience in the design industry which is an indication that they may have been involved with all techniques that have been available to the present date. 30 % of the respondents sit in the 16 – 20 years' experience range, which gives an indication that they should have been exposed to the majority of available design techniques.

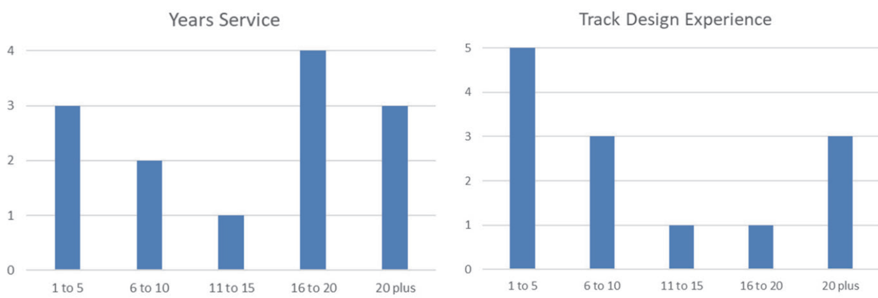
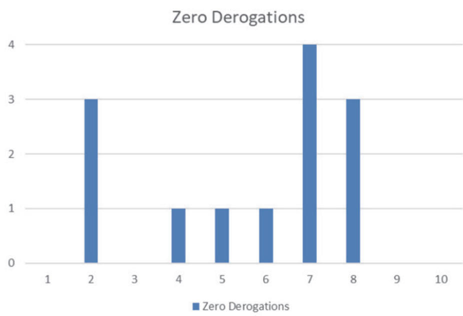


Figure 1 Experience of the respondents

Only 7 % of respondents sit in the 11 – 15 years' experience, exposing a potential skills gap in the industry. The remaining 46 % of respondents are in the 1 – 5 and 6 – 10 years' experience range and these respondents will most likely never have been involved in Hallade design [6-8]. Figure 1 also shows that knowledge gap and experience gap may be relatively high, despite the recent efforts in recognising this and increasing the number of jobs available in this area.

Based on the responses to the survey questions, it can be seen that there are mixed views on the traditional (e.g. Hallade) design methods. These methods, in the opinion of the industry, lack accuracy especially over long distances and through complex sites but is still an industry proven method of design. On the other hand, the overwhelming majority of the respondents use design software [6,7] for which they think there is room for improvement especially because there is a perceived over-reliance on the software with a lack of "First Principles" appreciation, further highlighting the knowledge gap that is developing within the rail design industry.



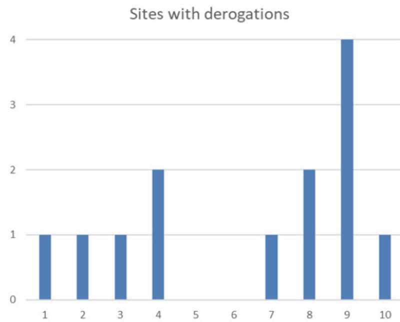


Figure 2 a) How easy is to implement NR design with zero derogations (1- very difficult; 10-very easy). b) Sites with x10 % of derogations.

Similarly, 70 % of respondents indicated that they have some experience in traditional gauging methods, and 92 % in the use of modern gauging techniques which include the use of ClearRoute 2 and the lasersweep [6]. These figures show that there is a relatively good appreciation of the importance of gauging and route clearance role in the track design.
Less experienced designers find it difficult to design to the current standards and mid-experienced designers find it slightly easier (Figure 2a). 77 % of respondents rated at 5 or higher for difficulty to implement current standards with only 23 % rating it lower than 5. This could indicate that standards have advanced so much that experienced designers who are so used to designing a certain way are finding it difficult to implement these methods to current standards. This could also indicate that perhaps there is an issue with the current standard design software tool [6,7] that more experienced designers have problems adapting to. This potentially explains the results shown in Figure 2b - out of 130 sites designed by all respondents, 63 % contain derogations which is an alarmingly high. This statistic alone shows just how difficult it is to fully comply with the current standards [1-4].

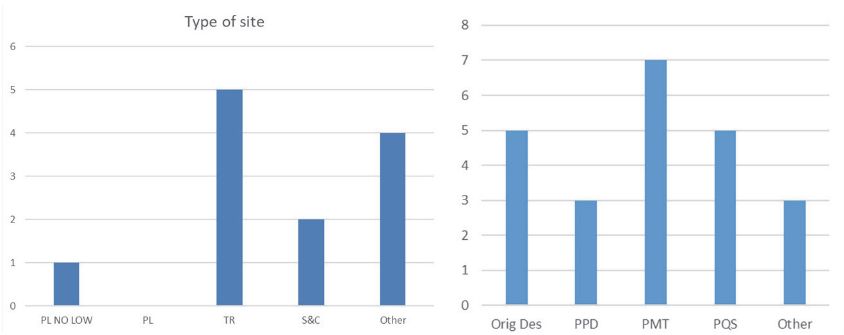


Figure 3 a) Type of site designers find most difficult to design. PLNOLOW - Plain Line Re-alignment (Track Lowers NOT Allowed), PL-Plain Line Renewal (Track Lowers Allowed), TR-Track Re-alignment through Structures (Track Lowers NOT Allowed), S&C-S&C Renewal (Track Lowers Allowed), Other. b) Factors contributing to derogation. OrigDes-Original Design that was completed to different standards; PPD-Poor previous design, PMT-Poor maintenance of track; PQS-Poor Quality survey; Other.

When asked about the type of site/project most difficult to design and, hence, most likely to contain derogations from the standards, 38 % of the respondents selected the 'track re-alignment through a structure' where there are no track lowerings allowed (Figure 3a); 15 % selected 'switch and crossover renewal (S&C)'; 8 % find difficulty in plain line re-alignments with no track lowerings allowed, while 31 % opted for the 'other' response and gave the following answers:

- "2-second-rule is almost impossible to abide by"
- "Plainline renewal where lowers not permitted, and maximum lift specified"
- "Multi-staged complex S&C junctions with Platform/Structure interfaces"
- "Those in Wessex and Anglia route where older forms of S&C are being renewed with modern form with varying geometries."

The majority of responses (including the 'other' responses) indicate that designers find most challenges on sites where there are constraints on altering the vertical alignment. This specification is generally put in contracts, however, without taking the track out, digging a hole, and then putting the track back it is impossible to lower the track. Every time there is any maintenance carried out on a length of track, the track must be lifted out of its designed position so sometimes trying to put a design on a track where the client wants it back in its original position cannot be accomplished.
Figure 3b) shows that the most recognised factor contributing to derogations was poor maintenance of the track (30 % of the respondents), followed by original design and poor quality survey with 22 % of the responses. The respondents could identify more than one answer due to the many different factors that could affect any design, and these were as follows:

- "Generally it is the scope of the job and the tight nature of the track. Construction methods also have a huge impact. If you slue a lot, you require more ballast excavation. If you lower a lot, you will also require more ballast excavation. If there is not the capacity in the wagons ordered for the job then it is not possible to implement the large slues or big lowers. This means that you have to stick to the existing footprint which is difficult whilst making things compliant. If the scope is a like for like renewal, generally moving S&C greater than 5m from its existing position will require other parts of the infrastructure to be moved. If that means a new OLE mast, or a signal move, they are expensive and generally the client (RAM) does not have the budget to implement such changes. The most common derogation that is applied to designs within S&C is network rails 'two second rule' (Clause 8.3.1 of [3])."

- “Poor Specifications at inception and at GRIP4 stage”
- “Scope changes, financial constraints (no new OLE structures for example), staging being pre-determined, limited site access (during survey/design phase), limited access for installation, not adhering to a systems engineering approach (integrated designs), unrealistic programme, unrealistic budget, interfaces with internal disciplines, interfaces with external projects”
- “Especially when the Victorian railway was built around structures, bridges etc. which now present great restrictions”
- “Note Merseyrail where existing track in poor condition and platforms are out of gauge”

Based on the responses, it would appear that poor track maintenance poses most of the challenges for designers. However, it would be unfair to tie the cause of not being able to implement current standards solely to that and, based on the other responses above, it is apparent that many factors can impact how a track design is undertaken. It is very clear that finance can play a large part in the process of failing to meet standards, and this is potentially connected to the age and history of the tracks. The United Kingdom Rail Network is largely Victorian in age, with significant amount of grand structures incorporated in it, many of which are listed and protected. Even if the funds were available to build a new bridge or station, sometimes there are still blockers that are outwith the control of the client, contractor or designer.

3.2 Case study

To illustrate the issues identified through the expert opinion survey, an attempt was made to compare two designs for Elderslie Station: one submitted to and accepted by the client (porposed desing) and an alternative one produced in an attempt to minimise the number of derogations (Tables 1 and 2). The Elderslie Station is located 3 km SW of Liverpool, oriented N-S, and comprises a cutting between two tunnels (Figure 4).

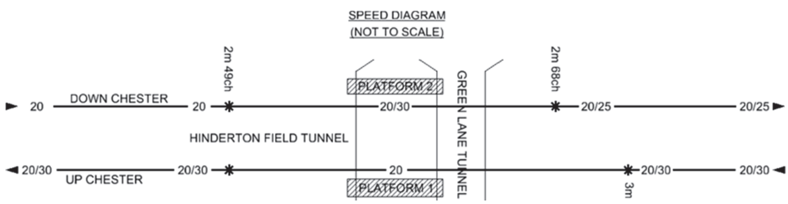


Figure 4 Schematic representation of the Elderslie station.

The design task from NR included adjustments to the proposed track alignment design to achieve ‘standard’ platform X and Y dimensions (730mm – 745mm and 890mm – 915mm) in accordance with GI/RT7016 (issue 5) & GC/RT7073 (issue 1). Additionally, clearance had to be provided for the new Merseyrail vehicle as well as any aspirational vehicles as listed in the NR Route Gauge Capacity Database. The existing gauging information showed that there are several clearances less than normal values with the smallest being 66mm between the Class 507/508 and another Class 507/508 vehicles. The proposed design was accepted with derogations on: virtual transition, lower SD Values (Table 1), short transition lengths, short vertical curves, lower six-foot values, lower clearance values (Structure & Passing, Table 2). The impact of these derogations will be relatively low, however the comfort and the riding experience of the passenger would be compromised with passengers potentially feeling like they are on a rollercoaster due to the multiple changes of direction and short element lengths.

Table 1 Comparative parameters for the Proposed and the Alternative design.

Track	Cat.	Const. Toler.. Band 2	Req'd Track Quality Stand.	Design SD Output		Expected Post Installation SD		Track Quality	
				Prop.	Alt.	Prop.	Alt.	Prop.	Alt
Up	AL35	1.800	2.700	4.312	2.121	6.122	3.921	Very Poor	Satis'y
Down	WT35	2.600	4.300	1.561	1.533	4.161	4.133	Good	Good

Table 2 Minimum clearance for the proposed (including interim) and alternative design

Stage	Deflated			Inflated		
	Ch. (m)	Min. Clear. (mm)	Vehicle	Ch. (m)	Min. Clear. (mm)	Vehicle
Interim	4520	73	Class 507, 508	4520	38	W6a 3rd Gen
Proposed	4455	71	Class 507, 508	4455	40	W6a 3rd Gen
Alter.	4455	93	Class 507, 508	4455	58	W6a 3rd Gen

In the alternative design that tried to address the number and effect of derogations, the following derogations were unavoidable: lower SD Values, lower six-foot values, and larger relative values (Figure 5). However, the cause of these derogations is not the design that has been carried out on the Down line, but the fact that no design has been carried out on the Up line or the six-foot, while the relative measurements are the measurements between the tracks.

4 Discussion and conclusions

Due to the country-specific nature of the topic of this study, the research presented in this paper identified a lack of published literature on the subject area which can be considered one of the limitations of this study. The existing literature comprises and is limited to the existing standards and in-house reports which sometimes contain commercially sensitive information. Considering the extent and the age of the UK railway network, it is clear that future investment in the industry and railway development will have to be tied to the dissemination and measurable impact of the research associated with the investment.

The number of track design experts surveyed for this study was limited to the designers within one large international, multi-disciplinary, consultant house which may be interpreted as too narrow and not representative. The authors tried to address this potential limitation with targeting a spectrum of professionals with various degree of experience who may have worked in other companies, albeit in the same field, in the past. Future studies would be focussed on surveying experts from a number of companies of varying sizes as well as experts who work (or worked) in a range of stakeholders: clients, contractors, engineers in order to obtain more representative and holistic view form the industry.

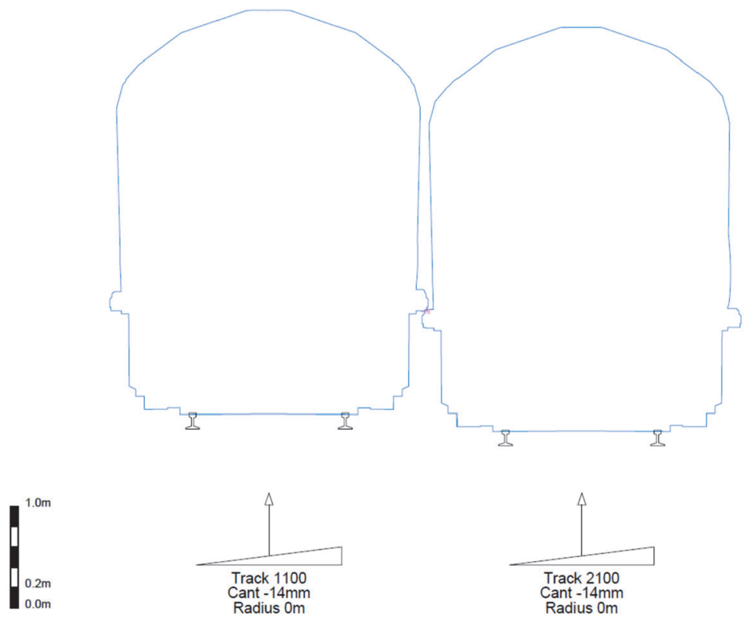


Figure 5 Alternative design for the case study with minimum number of derogations form the standards.

Another potential source of subjectivity in this study is the alternative design for the illustrative case study. This design was carried out by the first author and represent their subjective view on the topic. However, this design attempt showed that the required track design was unable to be completed without derogation on two occasions (the originally proposed design accepted by the client and the alternative design produced in attempt to eradicate the derogations and asses the effect of these derogations). This supports the expert view of design in cases when, unless a large-scale remodelling were to take place, the design will always have derogations due to the original layout which was designed to different standards. In this case, the removal of some derogations only resulted in further derogations.
This study only focused on one single case study of plain line re-alignment through structures with track lowering not allowed which, based on the responses from the questionnaire, proved to be the type of site that designers find most difficult to design. For the future, it would be worthwhile to investigate the other combinations of sites in a similar fashion, involving experts with different levels of experience and using different design methods. The responses provided to the questionnaire would suggest that there are more to derogations in certain sites than just the style and original scope would suggest. Case studies on specific sites where large changes of scope, which have caused designers to radically change designs and introduce derogations, would be welcome addition to the existing body of knowledge.
The comments made by some of the respondents appear to suggest that the software that is used as the industry standard would appear to be outdated and further analysis should be undertaken to verify this. It is the understanding of the authors that this is the only software that is available to the track design industry in the UK. The developments in the design software, however will have to be guided towards integrated design tool functionality i.e. a package that will allow all design work and requirements to be done under one tool.
The responses from the surveyed experts hinted at a potential skill gap developing in the industry, with larger percentage of experienced designers and relatively inexperienced designers who appear to be over reliant on software for design. It would be beneficial to carry out a study across the majority or all railway design houses to assess if this is common across

the industry. Such study would also include an insight into the one aspect that was missed in the present study: assessing the site experience of the designers. This experience can be extremely helpful when designing and an investigation into this area could help understand the link between designers who may be over reliant on the software and designers who understand the ground conditions and the nature and magnitude of loads coming from the new proosed railway construction or upgrade.

References

[1] Network Rail, GI/RT7073 Requirements for the position if infrastructure and for defining and maintaining clearances, Network Rail, 2015.

[2] Network Rail, NR/L2/TRK/3203, Structure Gauge Recording, 1 ed. Network Rail, 2011.

[3] Network Rail, NR/L3/TRK/2049, Track Design Handbook, Network Rail, 2016.

[4] Network Rail, Track Geometry - Inspections and minimum actions, Network Rail, 2015.

[5] Yin, R. K.: Applications of case study research, 3rd ed, Thousand Oaks, 2012.

[6] Hawkes, J.: Bentley Rail Track Workflow Guidance Notes, URS, 2014.

[7] Bentley Rail Track Fundamental, Bentley Institute, 2011.

[8] British Railway Track, 6^{th} ed, The Permanent Way Institution. n.d.

INFRASTRUCTURE PROJECTS AND BUILDING INFORMATION MODELLING IN BOSNIA AND HERZEGOVINA

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Abstract

Building Information Modelling (BIM) is a relatively new technology. The industry, especially when it comes to infrastructure projects, is just beginning to realize the potential benefits of it. Large capital projects are being done today using BIM technology and standards, while in Bosnia and Herzegovina today, we do not have a project implemented by it. BIM is still exhibiting varying states of maturity among its participants. The research was carried out in B&H to realize perceptions of BIM in the infrastructures projects from the perspective of different participants (investor, consultant, designer, supervising engineer, contractor, supplier). The following aims are to demonstrate perception about BIM, the willingness to apply it, and different varying states of maturity among its participants and the current degree of application in practice.

Keywords: BIM technology, research, construction industries, B&H, results

1 Introduction

Building Information Modelling (BIM) technology provides an integrated and comprehensive information repository and makes the sharing of visual, integrated, quality information possible for the project implementation. BIM introduction in infrastructure generates some changes in the process. The designers, contractors, companies, managers, universities, public and private research centres and others are involved in that innovation. From the conceptual to the execution phase, every step of the life cycle phase of the infrastructure project involved [1].
The Hong Kong Institute of Building Information Modelling defined BIM as “the process of generating and managing building data during its life cycle [which typically] uses three-dimensional, real-time, dynamic building modelling software to increase productivity in building design and construction.” BIM is not just a “three-dimensional drawing tool but a new tool to holistically manage information relating to construction projects from the preparatory stage to construction and operational stages” [2].
In the context of Industry BIM is a powerful methodology, which has been implemented with great success in the domain of Architecture, Engineering, and Construction. The construction industry has been working intensively on the implementation of the BIM methodology in several segments, BIM is a collaborative work concept, strongly based on technological advances in computation. BIM tools enable the development of building projects during their lifecycle, including the design, construction, maintenance, management, and demolition phases. However, it is as yet hardly ever used in the transport infrastructure sector (roads, railways, bridges, tunnels, airports, and ports), [3].

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Although BIM is significantly present in the world, especially in the construction industry, there is still resistance to the application of BIM methodology in an infrastructure project in Bosnia and Herzegovina. The article presents the results of research on the possibility of applying the BIM methodology in infrastructure projects in Bosnia and Herzegovina from the perspective of all participants in the project (investors, designers, contractor supervision, etc.).

2 Implementation of BIM – review

According to the Smart Market Report in 2013, 55 % of contractors in the US used BIM at a very high and high level, in France 39 %, Germany 37 %, Australia 33 %, Canada 29 %, the United Kingdom 28 %, Japan 27 %, Brazil 24 % and South Korea and New Zealand 23 % Recent research, the results of which are presented in the NBS International BIM Report 2016 shows that in 2016 in Denmark 81 % of construction companies applied BIM, in Canada 71 %, the United Kingdom 50 % , Japan 49 % and the Czech Republic 30 %.[4]
In 2015, the use of BIM was highest in North America, where BIM has been used for 8.5 years, and the rate of BIM use is 73 %. Behind North America are Oceania (application rate 7.7 years; application rate 65.5 %), the Middle East and Africa (application 5.9 years; application rate 60 %), Europe (application 5.3 years; application rate 55.9 %), South America (3.4 years; application-level 55.7 %) and Asia (application 4.9 years; application-level 46.4 %). Also, construction is the phase in which the level of BIM application is highest in North America, Asia and South America, and in Oceania, Europe and the Middle East and Africa it is the design phase. A study of the application of BIM in the Middle East (participating countries are Saudi Arabia, the United Arab Emirates, Kuwait, Oman, Bahrain, Qatar, Yemen, Jordan, Lebanon, Iraq, Syria, Egypt, Sudan, Libya and Algeria) shows that 20 % of organizations in the construction sector apply BIM with the most significant number of BIM projects in the United Arab Emirates and the smallest in Lebanon and Jordan. Thus, although BIM is recognized as a new way of digitized work in construction, its application is still very diverse in different markets, in companies and projects. [5]
Benefits of using BIM are recognized in all countries. The most significant ratio (93 %) of engineers who see some value in BIM use is in the USA. However, only 29 % of them see BIM's full potential. Although in other countries the percentage of BIM users who report some value in it is smaller (76-89 %), number among them who see the significant potential is almost identical (in the range 25-31 %). The reason for these numbers may be in short experience in overall BIM use and possible high initial investments in training and equipment. [4]
Among the list of 13 benefits from BIM use, most of the respondents selected these five as crucial: fewer errors, greater cost predictability, a better understanding of the project, improved schedule and optimized design. The biggest advantage of BIM is recognized during the design stage (49 %) while the least respondents (0 % in Germany and the USA, and 12 % in France) said that BIM provides the most significant value in post-construction phase. When it comes to the future of BIM, training and software and hardware development were recognized as the key aspects. [4]
The analysis of the results in Croatia shows that in 2016 BIM was used by 23.33 % of business entities, and in 2017 by 21.12 % compared to the results from 2015, when 0 to 25 % of Croatian companies used BIM, the use of BIM and on the Croatian market for three years. A comparison of the use of BIM in Croatia with the results of the latest survey on the use of BIM in the United Kingdom, according to which 69 % of businesses use BIM, shows that the use of BIM in the United Kingdom is 45 % higher than in Croatia. [5]
The results from 2016 and 2017 show that the importance of BIM has increased. Survey participants plan to apply BIM, most of them within two years and are aware of the benefits that BIM brings to business (the most significant advantage is that BIM improves the coor-

dination of participants). The results show that project participants in the Croatian market are generally not ready for the full implementation of BIM, with designers being the most prepared and contractors the least ready. This is confirmed by a detailed analysis of the application of BIM towards project participants, where designers stand out as participants who use BIM the most (25 % of survey participants use BIM), and architecture as an activity in which BIM is widespread (25.29 % apply BIM, of which 17.82 % work most of the time in a BIM environment). [5]

Although awareness of the benefits of BIM has increased over the three years, it is still shallow, as evidenced by data that 65.94 % of participants did not use BIM, that 61.15 % believe that BIM is only 3D software, that 23.81 % of them never plans to apply BIM and that they are of the opinion that the application of BIM should be legally regulated even though project participants in Croatia are not ready for its application. The misperception about the meaning of BIM and working in a BIM environment is confirmed by the analysis of BIM's understanding of activities and project participants, where most of them still believe that BIM is just a 3D tool. [5]

Last nine years NBS make a detail report about BIM use in the UK. In the survey for 2019 year, 988 professionals participated, and 98 % of them were aware of BIM and used it. BIM is still the goal for many organizations, and 96 % of them have the plan to use BIM in the next five years. Only 7 % of current users regret adopting it, while 63 % of them think they adopted BIM successfully. Most of the active users (73 %) said that BIM results in operation maintenance savings. Also, 69 % of respondents said that they need manufacturers to provide BIM objects. Besides the advantages, the main barriers to using BIM are recognized. The most significant barriers are lack of client demand and experience. Other significant barriers are cost, no time to get up to speed, small project on which BIM cannot be applied (or there are no big benefits of doing that), lack of standardized tools and protocols. At the same time, respondents thought that the Government does not sufficiently support BIM use and that private clients do not see the benefits of BIM approach. [6]

Dodge Data & Analytics conducted a survey in 2017 among 368 engineers and contractors in France, Germany, the UK and the USA. [1] The most of active BIM users are involved with tunnels engineering (86 %) followed by the bridge (79 %) and road engineers (76 %). Over 75 % of respondents use their own models, while the rest use already built models. This situation is very similar in all four countries. It was expected to grow BIM use in all of these countries by more than 60 % in the period 2015-2019. Based on this research, the USA has the most experienced BIM users (over five years of work) with 46 % of them. At the same time, Germany and France have less than 20 % of experienced BIM users. One of the critical element of BIM growth is how frequently clients are requesting BIM approach. More than one-third of investors are asking for BIM use in these countries. [4]

In 2013, "BIM France" (association of architects and engineers) later followed by the French government, public customers and professional organizations decided to support the development of BIM in France. In 2014, the Ministry of Housing and Construction declared that the use of BIM will be mandatory in public markets from 2017. There is no regulation on BIM in Sweden, but some initiatives are underway, especially among public project owners. Sweden's most significant transport project administration, the Swedish Transportation Administration, published a BIM strategy in 2013 to include BIM for all new investment projects from June 2015. [7]

3 Method

3.1 Questionnaire and statistical methods

After reviewing the literature, consulting with participants in construction projects in B&H and expressing the need to review the state of BIM in B&H, a questionnaire was prepared and distributed via e-mail. The surveys covered a large number of questions, and this paper shows the results of some of them. We sent a questionnaire (targeted) to participants in road and railway infrastructure projects for this research. Investor, designer, consultant/ supervising engineer, contractor/ supplier participated in the research conducted through surveys related to BIM in B&H construction.
The part of the questionnaire contained questions related to using of the offered technology was evaluated according to the Likert scale of assessment (1 - I did not use, 2 - I occasionally used, 3 - I mostly used, 4 - I used most of the time). The part of the questionnaire contained questions related to the BIM perception and their importance was evaluated according to the Likert scale of assessment (1 - strongly disagree to 5 - strongly agree). The application of BIM contributes to the competitive advantage of the company.
Cronbach's alpha coefficient is 0.942, which measures the level of reliability of the measuring scale. A higher Cronbach's alpha coefficient indicates higher reliability of the scale used to measure the latent variable. The set measuring scale has an excellent level of reliability.
Statistical analysis is conducted on the results of the surveys. The significant BIM perception are determined and validated using the relative importance index (RII).

$$RII = \frac{\sum w}{A \cdot N} \tag{1}$$

Σw is the sum of grades given to each factor, A is the max. given assessment for each factor and N is the total number of respondents. The value of RII is in the interval $0 \div 1$, ordinary rating scale used in research, and many researchers advocate this way of ranking. The BIM perception is more important if the higher RII.

3.2 Survey sample

The application of BIM to different activities and project participants was analysed. Construction project investors, civil, geotechnical, mechanical and electrical engineers and others took part in the survey, out of 64 questionnaires filled-in by the respondents. The percentage of respondents are 52.3 % micro, 15.7 % small, 14 % medium and 17.9 % large companies by annual income. Figure 1 shows the percentage of respondents by group: investor – 34 %, designer – 16 %, consultant/ supervising engineer – 17 %, contractor/ supplier – 33 %.

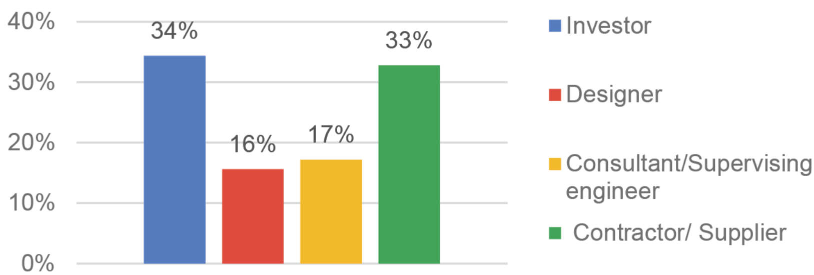

Figure 1 Participation of respondents by group

4 Results and discussion

Table 1 shows how much investors, designers, consultants/ supervising engineers, contractors/ suppliers have used 2D, 3D and BIM technology so far. Most respondents produced 2D digital drawings, 77.0 % designers 66.8 % contractors/ suppliers, 36.3 consultants/ supervising engineers and only 22.7 investors worked or used most of the time. Even 45 % of consultants/ supervising engineers and contractors/ suppliers did not use or create a 3D digital model. Over 20 % of designers and investors did not or occasionally used the 3D model. Over 70.3 % of the 64 respondents participating in that survey indicated that they did not use BIM model (table 1).
Participants' thoughts on the benefits of using BIM: the application of BIM enhances the success of the project, BIM improves the coordination of participants in the preparation of project documentation and the application of BIM contributes to the competitive advantage of the company are shown in Table 2 and Figure 3.

Table 1 The % of use of 2D, 3D and BIM technology with survey participants

		2D	3D	BIM
Investor	1	0.00	0.0	63.7
	2	36.4	27.2	22.7
	3	40.9	22.7	4.5
	4	22.7	50.1	9.1
Designer	1	1.0	12.0	60.0
	2	2.0	10.0	20.0
	3	20.0	18.0	10.0
	4	77.0	60.0	20.0
Consultant/ Supervising engineer	1	45.5	48.0	63.6
	2	9.1	19.5	18.2
	3	9.1	15.1	9.1
	4	36.3	17.4	9.1
Contractor/ Supplier	1	4.7	47.6	85.7
	2	9.5	33.3	9.5
	3	19.0	14.3	4.7
	4	66.8	4.8	0.0

The statements that BIM is currently too expensive to implement in our company by all respondents, Consultants/ Supervising engineer and Contractors/ Suppliers is the last ranked. All respondents think that BIM is not too expensive to implement in the company (RII=0.544). That the BIM is intended only for large companies 1.6 % (RII=0.422) (table 2, fig.2).
As many as 46.9 % of respondents use only BIM 3D software, and only 14.1 % of respondents were not accurate. 37.5 % disagree with the statement that BIM is intended solely for large companies. Over 85 % of respondents agree or strongly agree that investors will to insist on the application of BIM (RII=0.784). The application of BIM contributes to the competitive advantage of the company is recognise like the advantage of using BIM (table 2, fig.2).

Table 2 RII- relative importance index, the BIM perception

	Adopting of BIM enhances the success of the project	Investors will insist on the application of BIM in the future	BIM improves the coordination of participants during the preparation of project documentation	Adopting of BIM contributes to the competitive advantage of the company	BIM is 3D software	The implementation of BIM requires the assistance of a consultant.	The application of BIM requires the assistance of a consultant.	BIM is currently too expensive to implement in our company	BIM is intended only for large companies
	Rang								
All resp.	1	2	3	4	5	6	7	8	9
Investor	1	3	2	4	5	6	8	7	9
Designer	2	3	1	4	6	7	8	5	9
Consultant...	1	3	4	2	6	7	9	8	5
Contractor...	2	1	4	5	3	6	7	8	9

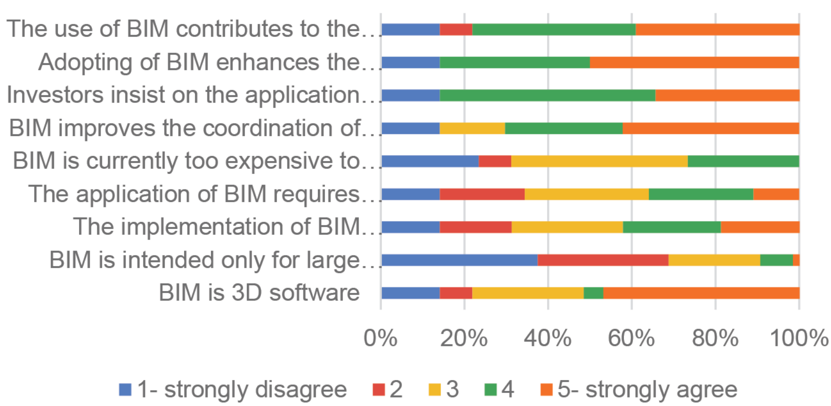

Figure 2 BIM perception by participants

5 Conclusion

BIM is now recognized as the current best practice methodology to have a go at building and infrastructure projects [8]. This research dive an exciting overview of BIM technology utilization in Bosnia and Herzegovina and challenges to widespread adoption of this technology in infrastructure projects. We assessed the perceptions of participants (investor, designer, consultant/ supervising engineer, contractor/ supplier) on the relative importance of BIM perception in the infrastructure projects using RII.

The implementation of any project full according to BIM technology is not present. The understanding of BIM as a 3D technology is very present, as this research shows. The legal norms and framework in Bosnia and Herzegovina still do not acknowledge the Building Information Modeling. The infrastructure projects participants if want can use BIM, but they do not must do in this technology. The participant's road infrastructure projects are the ones who have so far realized the advantages of this technology in construction projects. The road and rail infrastructure participants are on the first step towards understanding the BIM importance and benefit, only when they understood that, it would be possible to start drafting regulations and legal acts.

References

[1] Abbondatia, A., Biancardob, S.A., Palazzob, S., Capaldob, F.S., Viscioneb, N.: I-BIM for existing airport infrastructures, AIIT 2nd International Congress on Transport Infrastructure and Systems in a changing world (TIS ROMA 2019), Rome, Italy, 23-24.09.2019.

[2] Chan, D.W.M., Olawumi, T.O., Ho, A.L.M.: Perceived benefits of and barriers to Building Information Modelling (BIM) implementation in construction: The case of Hong Kong, Journal of Building Engineering 25 (2019) 100764, pp. 1-10, doi:10.1016/j.jobe.2019.100764

[3] Neves, J., Sampaio, Z., Vilela, M.: A Case Study of BIM Implementation in Rail Track Rehabilitation, Infrastructures, 4 (2019) 1, p. 8, doi:10.3390/infrastructures4010008

[4] The Business Value of BIM for Infrastructure 2017, https://www2.deloitte.com/content/dam/Deloitte/us/Documents/finance/us-fas-bim-infrastructure.pdf, 10.03.2020.

[5] Kolarić, S., Vukomanović, M., Bogdan, A.: Analiza primjene BIM-a u hrvatskom graditeljstvu, GRAĐEVINAR, 72 (2020) 3, pp. 205-214, doi:10.14256/JCE.2774.2019

[6] National BIM Report 2019, https://www.thenbs.com/knowledge/national-bim-report-2019, 10.03.2020.

[7] Bensalah, M., Elouadi, A., Mharzi, H.: BIM integration to railway projects - case study, ASME Joint Rail Conference, Pittsburgh, PA, USA, 18-20 April 2018.

[8] BIM for Infrastructures: from Planning and Design to the Maintenance Phase, https://www.hindawi.com/journals/jat/si/127656/, 10.03.2020.

IMPLEMENTATION OF BIM IN PUBLIC TRANSPORT INFRASTRUCTURE WORKS IN THE CZECH REPUBLIC WITH RESPECT TO THE FIDIC STANDARDS

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Abstract

The paper describes the actual status of BIM implementation in public projects in the transport sector within global powers and the Czech Republic, emphasising the specific aspects of the construction market and legislation base in the Czech Republic. The research part of the paper presents the proposed methodology of BIM implementation in road construction, and demonstrates the different levels of detail and information using a BIM model. Last but not least, a risk analysis is introduced and the risks identified are briefly introduced.

Keywords: building information modelling/management (BIM), pilot project, BIM implementation, BIM model

1 Introduction

The Czech Republic plans to implement BIM as integral part of the project preparation and implementation of all public projects in civil engineering that are in terms of public procurement law defined as “above-the-limit” investments. This covers all public construction works (under Act no. 134/2016 Coll. on public procurement) amounting to an investment higher than CZK 149,224,000 (5 855 476 €) starting from January 1, 2022. The obligation of BIM implementation in public procurement works is stipulated by Resolution no. 682 of the Government of the Czech Republic dated September 25, 2017 on the Concept of BIM implementation in the Czech Republic – part III of material file no. 918/17.
The Resolution thus has an impact on all larger construction or rehabilitation projects in the road construction sector. The content of this paper focuses on the road construction issues, particularly in relation to motorways, 1st class (trunk) roads and structures like bridges, tunnels, utility networks etc.

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2 BIM implementation around the world

The literature search of available sources [1, 4-8] suggests that BIM has been so far gradually implemented in the construction industry all over the globe. The level of implementation is illustrated by Figure 1 below.

Figure 1 BIM implementation status over the world [9].

However, the problem is the difference between BIM implementation on private enterprise and public or state enterprise levels. This is also set out in [5]. Another incomparable aspect is the different character of road construction projects and building projects where BIM has been involved for a number of years. For example, [9] and Fig. 1 demonstrate that BIM has been completely or partly implemented on government project levels in the USA and Great Britain while other countries are in the implementation stage, or BIM is in preparatory stage or the initial implementation is already in place. Finding out whether the BIM implementation concerns at least the 4-5D level (including time scheduling and cash-flow) and whether heavy infrastructure/motorway projects and road construction projects in particular form a part of the implementation, is rather difficult. The literature search has failed to detect the precise methods applied in the individual countries around the globe where the implementation has already been completed, and the solutions applied to the specific road construction issues.

This means primarily the specific aspect associated with the linear character of the project whose trajectory passes through environments which might change significantly along the course (e.g. geological conditions, location of underground utility networks, geomorphology etc.). The linear nature also requires a huge volume of data to be processed which require a high level of detail. The search has failed to obtain any available methodologies applied by foreign public contracting authorities, procedures used and ways of implementing BIM with respect to risk analysis and overall economic assessment [1, 5].

3 Implementation of BIM in the Czech Republic and the legal framework

The Czech Republic presents a specific environment of local practices and existing, applicable legislation. The biggest public contracting authority in the field of road construction is the Road and Motorway Directorate of the Czech Republic (Ředitelství silnic a dálnic, hereinafter "ŘSD" or "Directorate"). Even though the Directorate has regulations which allow public procurement according to the FIDIC Yellow Book, it so far has tendered projects solely based on the methodology according to the FIDIC Red Book due to internal ŘSD regulations and in compliance with public procurement law. This means that both the Client and the Contractor accept roughly equal risks addressed on the basis of risk and claim management preparations. Another feature is the status of the project administrator: in the case of the Directorate, the project administrator is the public client itself. The competences of the administrator include independent claim handling, too.

In practice, the client first assigns work to prepare the project designs and documents on the individual levels, from the planning study to the tender specifications. All the levels may be compiled by a single independent designers, as well as up to four different ones. The documents are compiled separately for the Study, Urban Planning Decision, Building Permit and Tender Specification levels. Only then does the Client announce a tender for contractors to compete for the public project as General Contractor or as a construction consortium.

The FIDIC Yellow Book (D&B – Design and Build) has so far tended to be avoided in the Czech road construction market. The Yellow Book model is used in the developed countries like France, the UK, the Netherlands, the U.S. and others. In the Central European region it has been partly used in Poland or Austria and recently has also been applied to the tunnelling and construction projects on motorways in Slovakia.

The Yellow Book is mostly based on the Red Book with the significant difference of having the Project Specifications prepared by the construction contractor itself (the contractor has been known right from the start) based on the Investor's requirements, and the contractor is also duly liable for the specifications. The price of the work is not measured or quantified; it is determined by a fixed fee which poses equal risks for both contracting parties. With respect to the lump-sum drawdown of funds, the claim management must be established in advance as well as the conditions for invoicing, etc.

The process normally involves selecting a contractor to prepare the Project Specifications. The great advantage is having one entity (internal or external) that prepares the project documents and designs for all or at least several design levels. The compilation of the documents may involve some optimisation of work flow, materials etc. which might ultimately have a positive effect on the final price of the project. Technical changes and optimisations in the Red Book regime are a nightmare for all parties in the construction process with respect to the administration workload associated with administrative process known as "changes during construction".

The "single contractor" concept and the possibility of changes/optimisation then encourages the application of e.g. observation methods of earthworks but, primarily, BIM. This is because a single model is made which carries all the information from the initial design through to the last part of the Construction Design Specifications. The project is managed, ideally from the very start, by a single principal designer who is aware of all the aspects of the project and the environment. He knows the client´s requirements and the contractor's limits. This eliminates the tardy process associated with the quantification/measurements of the contract where each new designer of a new design level, including the contractor itself, must study the documents and get an understanding of the continuity and logic of all operations first. This also means that number of aspects might be omitted.

As ensues from the above, the use of BIM within the Design and Build model is the most effective application. Considering that the BIM model allows even facility management and offers the option of facility management by private sector entities, we are almost reaching the level of Public-Private Partnership projects (PPPs) wherein BIM might play an interesting role. The biggest problem and, therefore, the biggest risk in BIM-designing for a public contracting authority in any specification level lower than the implementation specifications is the fact that, the designer is not allowed to use precise specification, company name or characteristic of a component that would conspicuously encourage the use thereof (legal requirement of the public procurement law).
As an example, it is impossible to specify anything other than the material, height and length for a crash barrier according to the existing regulations. Under no circumstances may the brand name, length of the crash barrier as declared according to the manufacturer's specifications, or the shape of the crash barrier be indicated. The problem of BIM, or its clients, is an enormous effort to achieve the highest precision, level of detail and reflection of reality (digital twin). However, there is no potential for that in the public sector. A crash barrier, for instance, will therefore be modelled as a rectangle – the envelope of the deformation and construction space occupied by the crash barrier. The barrier may only be modelled in detail in the Construction

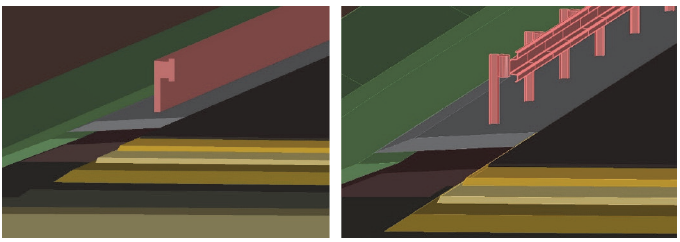

Figure 2 The difference between traffic barrier for lower levels (LOD200) and implementation specifications (LOD400)

Design Specifications stage and afterwards. The difference between the BIM model for the general level and for the Construction Design Specifications is obvious from the Figure 3. Two pilot projects were completed in 2018 within the framework of BIM implementation. The first one was the reconstruction of junction of a trunk road (No. I/32) and a regional road (No. II/125) at Exit 42 from the motorway D11 (about 40 km to the east of the capital city Prague) – conversion to roundabout (the author was a member of the design team). The second project was a part of D1 motorway modernisation – section 04, Exit 34 Ostředek – Exit 41 Šternov, Overpass no. D1-040 and local road at km 37,170.

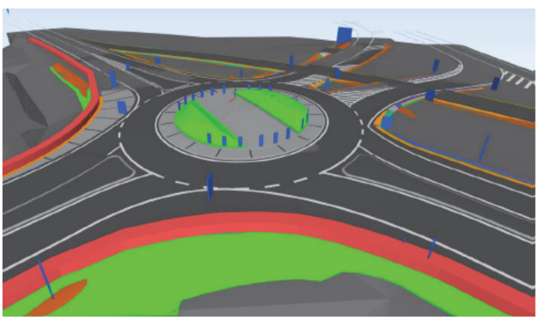

Figure 3 Sample from the Directorate's pilot project – conversion to roundabout

The pilot projects were expected to verify particularly the possibility of compilation of the Construction Design Specifications and as-built drawings. The scope – overpass and junction – defined these two as points rather than linear structures. Another problem was having the BIM model created based on 2D construction documents during the implementation stage. The possibility of clash detections and coordination was checked.
Although the projects were characterised as points, interesting findings transpired. For example, the bridge cones were more precise in BIM than in traditional documents, allowing an incorrect bill of quantities to be optimised. Clash detection demonstrated hundreds of unreal clashes within the framework of relocations of utilities networks crossing the sewer line excavations or side posts on the road edge. Contrastingly, some relevant clashes were difficult to be identified, e.g. the large traffic signs in the field of view at the junction.
During 2020-21 the Directorate envisages further pilot projects which should also include the permit-granting process and document compilation for early design stages.

4 Implementation method

The CESTI Competence Center and the student research project no. SGS OHK1-083/18, "BIM – implementation in the road construction practices in the Czech Republic" involved the generation of a BIM model for road construction, see figures 2-3, used as the basis for the examination of the required information detail level. The basic attributes (non-graphic information, see Figure 4) were designed and the Methodology for BIM process utilisation in road construction and road construction project management was proposed [10].
The BIM Implementation Methodology is proposed containing seven separate levels:
Preparation work; Target definition; Project preparation pilots; Construction execution pilot projects; Asset/facility management; Education and awareness; Implementation.
The completion of the aforementioned levels brings the methodology to the next stage. The sequence is designed to prevent leaving out any levels although the individual levels can be initiated in overlapping time windows. It should be borne in mind that completion of the seven levels in itself does not guarantee a smooth and easy start of the mandatory BIM system, and that it will be necessary to continue improving the process, optimisation and review even after the implementation.

5 Methodology and used processes

The methodology in question addresses the method of BIM implementation in the road construction industry. It is divided into the aforementioned seven levels. Each level aims to provide the completion or checking of a certain theme, contributing to the overall implementation. The individual levels always require a risk analysis, discussion of the results by a broad expert public community and the requirement for the final report to be published. A time schedule of the individual levels and the proposed attributes (non-graphical information) for non-solid pavements, road crash barriers and vertical traffic signs are attached to the methodology. For some objects the attributes are designed for noise barriers as well. The attributes are designed for the design levels from building permit documents to Construction Design Specifications and facility management model.

<u>Level 1:</u> Preparation work – This level is dedicated particularly to reviews of the work completed, and existence of supporting materials in the Czech Republic, reviews and provision of available documents from abroad and, mainly, the degree of relevance of the document content. An overall summary of the findings of the aforementioned sources also forms a part of the level.

Level 2: Target definition – The second level addresses the definition of the BIM project preparation targets, including its added value for the technical study, zoning decision documents, building permit documents, Tender and Construction Design Specifications levels. It also defines the objectives of the facility completion and management with an emphasis on the project's life cycle. The second level also entails the identification of necessary amendments to the legislative documents (laws, Government decrees and decisions, technical standards and other regulations). Last but not least, this level is dedicated to the issues of liability for defect and total demands for funding the project preparation which will undoubtedly be higher than the current market prices.

Level 3: Project preparation of pilot projects – This level handles the pilot projects for design work on the various levels of project specification documents. It first designs an independent pilot project for a road relocation, bypass as a conclusion of a complex project involving objects in classes 100-500 according to the national classification system for construction elements and objects. It appeals primarily to the quality and price of obtaining good quality source materials for BIM, especially the approximate course of utilities networks and structure diagnostics, or geological surveys. The level requires a time and economy-related examination involving a risk analysis. Projects where no difficult and lengthy property-related issues are expected should be selected as pilots. Last but not least, the level also researches the possibilities of construction project permit granting and positions of the state administration bodies concerned or other parties to the proceeding within BIM.

Level 4: Construction execution of pilot projects – This level is dedicated to Construction Design Specifications and the as-built documents where BIM has a great potential because the document is subsequently used as an input for the BIM facility management model. The level also handles BIM processes during project completion. The recommendation for this level is to complete the projects from the preceding level which have been prepared in BIM right from the start. Relevance and effectiveness shall be pointed out as well, e.g. whether BIM should include (and if so, in what level of detail) also construction objects to be handed over to other owners for whom the information is of no value (municipalities, private owners, technical infrastructure managers etc.). Lastly, there is a requirement for a time and economy-related assessment with a risk analysis.

Level 5: Asset/facility management – This level concerns facility management and structure/building life cycle. A pilot project (which will necessarily take several years and extend over to the post-implementation period) should check the essential problems – primarily the supporting documents for the BIM maintenance model, the level of detail and identification of the information needed by the facility manager as opposed to unnecessary ballast information (i.e. relevant level-of –information).
The BIM facility management model must be clear and systematic, simple and sophisticated at the same time. It must be tailored to any management or road maintenance and operation centre. The greatest potential is the implementation of planning regular and winter maintenance, as well as heavy maintenance schedules, standard and extraordinary inspections etc. The model must certainly avoid causing unnecessary delays and administrative load and it must effectively improve the overall process. Finally, an analysis of economy and risk is required in this stage as well.

Level 6: Education and awareness – This level is dedicated to the education and awareness-raising of broad specialist public, state administration bodies concerned, public contracting authorities, students as well as the public which might be interested in the reasons why more financial means should probably go into project preparation. The crucial problem is the method of teaching young students not only at universities but at secondary schools already. Their instruction should start as of now but there are far from enough experts with practical experience in the BIM processes in road construction.

List of Hazards				
Number	Identification		Hazard materialisation scenario	Proposed measures and objectives
	Classification	Hazard		
23	Target definition (level 2)	Lack of clarity in the brief and the consequent LOD	The public contracting authority will require an insanely high LOD disproportionate to the importance of the design level and construction item content (e.g. simultaneous relocation of drainage).	Reduce LOD, only do the main items to be managed by the Client in the future (or items with difficult coordination) in BIM.
24	Target definition (level 2)	Economy and time effectiveness of BIM for design document compilation in different levels (zoning permit, consturction permit, realization).	BIM is an uneconomical tool with too high time demands. The cost and efficiency is disproportionate to the importance of the work.	Use common CAD tools. Partial sections of BIM may be used (3D object creation).

Figure 4 List of risks (example)

At the same time, there are not enough experts to provide education and training for state administration staff or public contracting authorities and road management experts.
Each level requires an economic assessment and risk analysis. The risk analysis of the individual levels of BIM implementation forms a part of the main author's dissertation thesis currently in progress. The dissertation will present a catalogue of approximately 100 risks detected through the examination of the existing applicable legislation, BIM models and the proposed Methodology for BIM process utilisation in road construction and road construction project management. The risks identified will then be assessed by a selected exact method (e.g. Failure mode and effects analysis - FMEA or Universal Matrix of Risk Analysis - UMRA) and an evaluation will be prepared. Based on the degree of risk and its total weight, the most suitable measure will be recommended. The full risk analysis will be conducted latest by the end of 2021.
The figure 4 gives an example of the list of risks with the expected scenario for risk development and the proposed mitigation measures.

6 Conclusion

As a conclusion we may state that there is almost one year left till January 2022 and the majority of problems are likely of having been addressed and solved by then. Still, it is worth thinking about the ways of handing the major problems as mentioned above. Within the framework of the available information, there has been no similar project (handling a BIM model of a construction project, risk analysis and methodology) in the working groups addressing the BIM implementation in the Czech Republic. The country proceeds on the level of the Ministry of Industry and Trade with the State Fund of Traffic Infrastructure and the working groups by gradually formulating regulations and partial sections of a complex methodology (e.g. [2]) along with the pilot projects. This can be characterised as rather chaotic in comparison to methodology of phasing in proposed by this paper. Unfortunately, the fact that the progress of BIM implementation in road construction is partly a closed-door discussion issue to the specialist public [1] and there is no involvement of broader academic community which has been suggesting the completion of a due risk analysis with an economic evaluation for a long time, seems another major problem.

Acknowledgement

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References

[1] Fazekas, K., Lambert, J., Valentin, J., Pánek, P.: Vybrané pohledy k průběhu zavádění BIM do dopravní infrastruktury dle usnesení vlády č. 682 ze dne 25. září 2017, Silniční obzor. 2019, 80 185-232. ISSN 0322-71540

[2] Hořelica, Z., Mertlová, O., Vykydal, I. a kol.: Plán pro rozšíření využití digitálních metod a zavedení informačního modelování staveb, https://www.sfdi.cz/bim-informacni-modelovani-staveb/, Praha, 09/2017.

[3] Usnesení Vlády č. 682 25.9.2017

[4] Crotty, R.: The Impact of Building Information Modelling. Oxon: SPON Press, 2012. ISBN 978-0-415-60167-2.

[5] Matějka, P.: Rizika související s implementací informačního modelování budov. Disertační práce, ČVUT Praha 2017.

[6] Eastman, C., Teicholz, P., Sacks, R., Liston, K.: BIM Handbook: A Guide to Building Information Modeling for Owners, Managers, Designers, Engineers and Contractors. 2. John Wiley & Sons, Inc., 2011. ISBN 978-04-7054-137-1.

[7] Deutsch, R.: BIM and Integrated Design: Strategies for Architectural Practice. John Wiley & Sons, Inc., 2011. ISBN 978-04-7057-251-1.

[8] Race, S.: BIM Demystified. 2. London: RIBA Publishing, 2012. ISBN 978-18-5946-373-4.

[9] https://www.geospatialworld.net/article/bim-adoption-around-the-world-how-good-are-we/

[10] Fazekas, K., Lambert, J., Valentin, J., Pánek, P., Vébr L.: Metodika pro využití procesů BIM v silničním stavitelství a při řízení projektů silničních staveb, CESTI, ČVUT Praha 2019

[11] Fazekas, K., Lambert, J., Valentin, J., Pánek, P.: Metodika pro využití procesů BIM v silničním stavitelství a při řízení projektů silničních staveb, Technické listy CESTI 2019_WP1_TL1_14, ČVUT Praha 2019.

BIM IMPLEMENTATION: ROUTE 6 PRISHTINE – HANI I ELEZIT

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Abstract

Over the years, in construction industry, BIM (building information modelling) is one of the most mentioned topics with the main question: "Do we need it?" The answer should be commonly known, like with every solution that modernizes the profession and society. BIM in linear construction projects (transportation facilities and structures, sewage/water supply) and landscape design are still "one step behind" with already accepted "BIM implementation" in AEC (Architecture, Engineering and Construction) industry and MEP (Mechanical, electrical, and plumbing) industry. With more than 10 years of experience KAP4 company is one of the leading brands in BIM – Croatia. Along with our knowledge, courage, and motivation, we successfully realized multitude projects in Croatia and Europe, mostly in AEC industry but also in linear construction projects. Through our biggest project in linear construction industry "ROUTE 6 PRISHTINE – HANI I ELEZIT" (Sections 2&3), on real life project we will show advantages of BIM implementation in road structures design (bridges, overpasses, underpasses, etc.), but also in survey data, road design and earthwork optimizations, along with challenges in process (changes in design, no BIM environment, etc.) and later BIM impact on our Client and associate designers on route. With dozens of BIM road/structure models on route we will demonstrate output results with drawings/documents full of information, improved graphic outputs, document and team design collaboration, process of design, associate designers' collaboration and facilitating the client to follow the whole process and documentation. Hopefully, and answer to first question, do we need it.

Keywords: BIM, road structures design, KAP4 d.o.o., model, documentation, collaboration

1 Introduction

Among knowledge, idea, experience, and other essential engineer skills, for quality and optimal solutions, time and information's are crucial. If we add control and managing, before time and information's, in "equation", then we can fulfil most of our engineers' potentials and deliver quality product (buildings & structures as ultimate goals) in reasonable time with minimum cost. And that is BIM, the tool (technology...) that give us power to control and manage information's trough whole process of design, construction, and management of building/structure.
From early days, society/professions seek progress through optimization of processes (time & expenses following...) and innovations that enables them. BIM – "Building information modelling" was introduced more than 50 years ago [1], and only last 10-20 years is accepted by part of profession/industry [2]. AEC (Architecture, Engineering and Construction) industry is leading part in BIM implementation and commonly known as "only" construction industry where BIM is effective, which for sure cannot be true.

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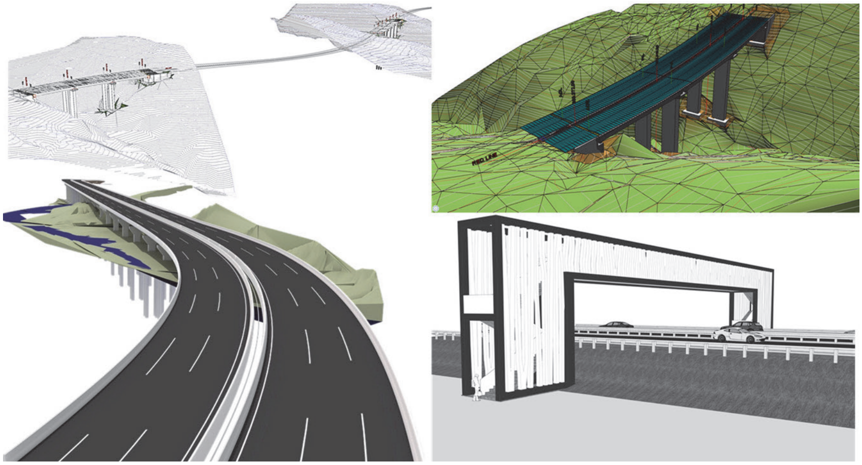

Figure 1 ROUTE 6 BIM models – KAP4

BIM or Building Information Modelling is a process for creating and managing information on a construction project across the project lifecycle. One of the key outputs of this process is the Building Information Model, the digital description of every aspect of the built asset. This model draws on information assembled collaboratively and updated at key stages of a project. Creating a digital Building Information Model enables those who interact with the building to optimize their actions, resulting in a greater whole life value for the asset. [3]
Intention of this paper is to show benefits (or at least part of them) of BIM application through all phases of the project. Management of processes and information from the early-stage design, through the main and implementation design for the benefit (as the final goal) of construction and building usage (or generally, project itself) but also for benefit to all participants in construction.

2 BIM in design

2.1 Early-stage design

In the early-stage design phase, the available information and especially the possibilities of its use (and speed of exploitation) come to the fore. With route input data, terrain topography (3D scan or classical survey data), geomechanics, etc., the engineer can think globally and locally to create the best and most optimal solution.
From the global level and consideration of road route correction, through variant solutions of structures regarding location possibilities, access roads, on-site construction, or delivery, to more detailed approach of important structure elements regarding earthworks, slope stability, etc. Each individual solution / variant can be analysed relatively quickly with defined main quantities for construction and recognizing types, scope of works and their time-cost analysis.

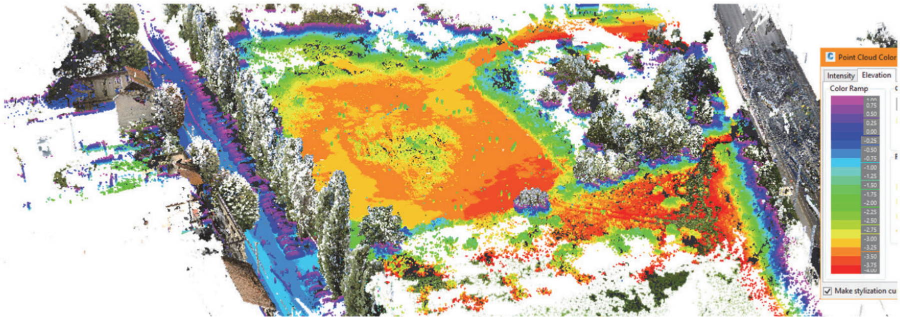

Figure 2 Point cloud terrain topography – earthwork analysis

In the early-stage design phase, analysing bridges and other structures of section C3, significant earthworks were noticed (visually and quantitatively) to perform foundations of structures but also subsequent large areas of unstable cuts that needed to be permanently protected. The problematic part of the section (approx. 3 km) was modelled as designed and quantified. Parallel with that a completely new model was made with different approach. In addition to the visually clear difference, the quantities could be compared and analysed and then presented to the client for final confirmation and change of route.
Afterwards, given the possibilities of location, access, period when which structure should be in function for access to other facilities, etc., each building is observed separately, and variant solutions were made. Solutions were harmonized with the surrounding structures to present to the contractor and client and adapted to required technology and capabilities for faster, more economical, and simpler construction on site.
According to the selected building concept, the construction is further analysed to at least identify all key problems and present them to all participants in the project, if it is not already possible to solve them immediately before the next design phase.

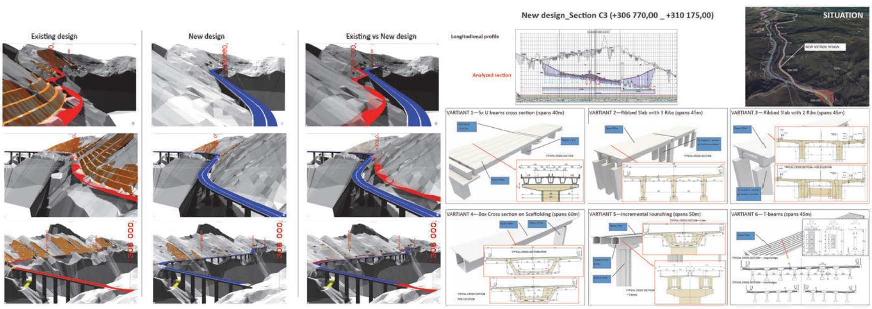

Figure 3 Route optimizations & structure variants (global & local level of optimizations)

2.2 Main and implementation design

In BIM design, the process of creating documentation and the design itself moves in reverse order. The engineer has the ability (or rather, the opportunity) to build his building virtually and to identify all the key problems he must solve. He proves his solution with calculations, harmonizes all load bearing, non-load bearing and auxiliary elements (drainage installations, lighting, equipment, fences, etc.) and then approaches the preparation of documentation and drawings for construction. Of course, the drawings are not drawn but are automatically extracted from the BIM model and equipped with needed and desired information by the designer, contractor, and client. This gives the engineer extra time to think and create, instead of wasting the same on 2D drawings, manually calculating quantities, etc.

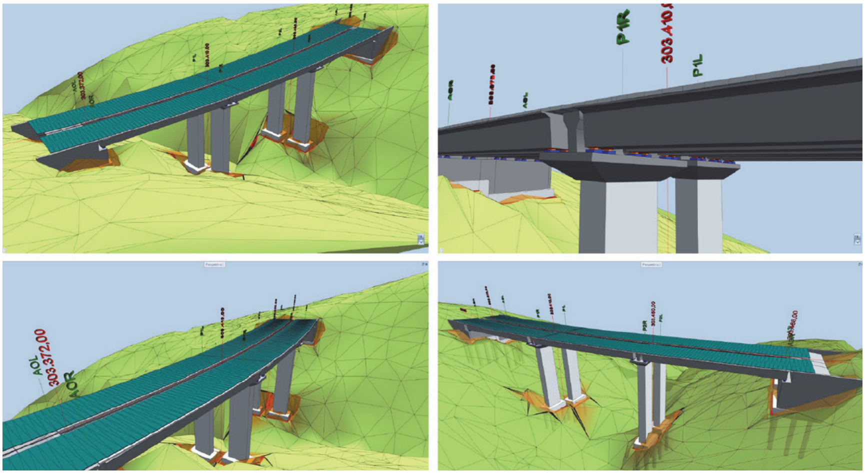

Figure 4 Bridge level of details (main design BIM model)

At this stage, each element (or segment, set of elements) contains all necessary information for the designer and basic or advanced information (available to the designer) for the Contractor (order of construction, quantities, maximum dimensions and weight for delivery and installation, etc.) and the Client (time required for construction, costs ...). All this enables preparation of 4D, 5D, n-D BIM analysis for the Contractor and the Client, as well as better preparation, planning, and organization of the construction of the building.
All documentation (general drawings, 3D detailed reinforcement and formwork plans, survey data, automatic quantity reports) is made with superior precision and almost flawless (especially without human errors like in no-BIM environment).
3D representations on the drawings give a new level of simplicity to representation of complex solutions, explanations of construction from the technical aspect but also the order of execution, level, etc. The level of information (necessary and additional) on the drawings and documentation is at a very high level and there is almost no possibility of unknown, unforeseen problems and costs at a later stage of construction.

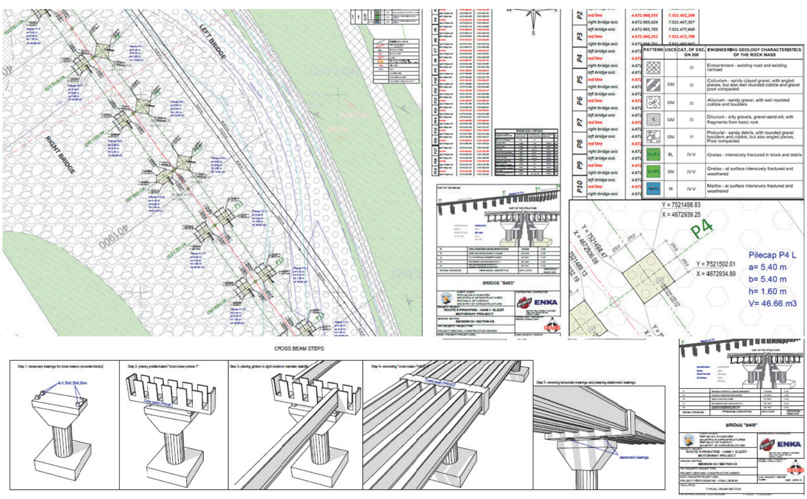

Figure 5 Detailed general drawings with necessary and additional information

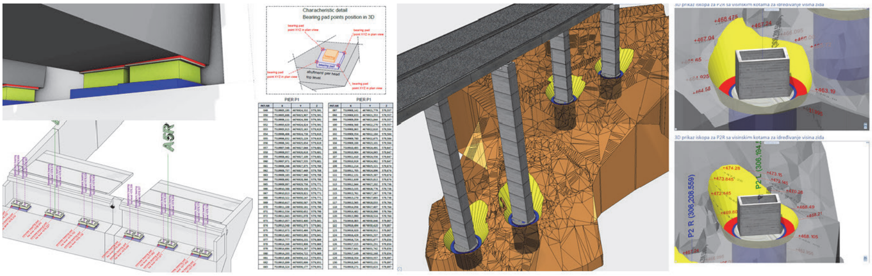

Figure 6 Automatic detailed global coordinates (X,Y,Z) & 3d position representation

As in the previous phase, the collaboration and communication with the Contractor and the Client is constant, clear, and transparent and they are familiar with all technical solutions, construction and installation sequences and quantities, with quality visually representations. For communication, it is possible (and desirable) to launch BIM platforms for direct communication on the BIM model at this stage.

3 BIM in construction and maintenance

3.1 Construction and collaboration

With well-done project preparations in the earlier stages, the Contractor is already thoroughly familiar with the project and has at his disposal all the necessary information for quality organization and planning, which is certainly one of the main challenges at this stage. It is not necessary for the contractor to use BIM technologies (like in this project) and BIM model to improve it (supplement with new information), which is certainly recommended. In the previous phases of the project, quality approach of the designer and active participation of all necessary participants in the construction, it is possible to anticipate and solve all identified and potential problems/challenges, which provides an opportunity to actively monitor the project to its final purpose.

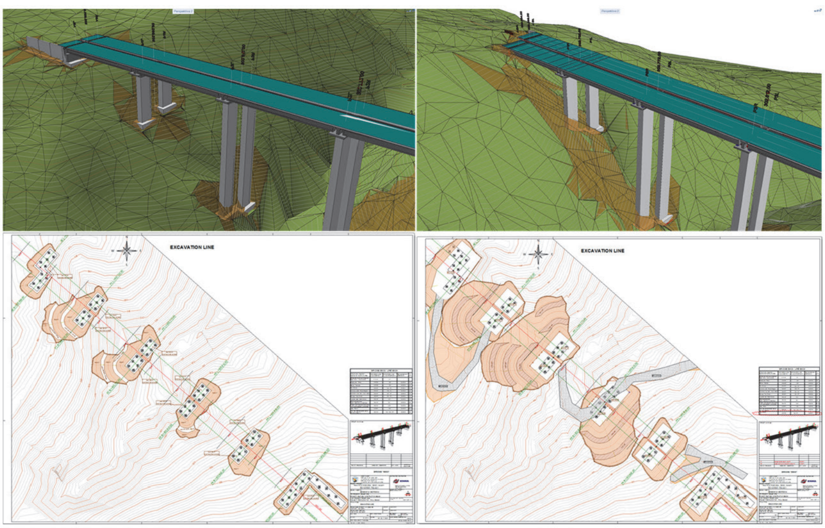

Figure 7 Change in earthworks and pier heights after construction of an access road for heavy machinery

However, what if previously known conditions change? Then, of course, there are changes in the project and project documentation. The connection between the BIM model and the created documentation is constantly active. On the example of making the approach of heavy machinery to the location of the bridge, the descending ramps disrupted the planned existing condition of the ground. By adapting the terrain model to the new survey data and correcting the excavation (3D and then automatic 2D drawing and excavation quantities) in a short time, the impact of the change on the designed solution was known.
Such changes and all other needs, requirements and adjustments are best done through platforms for communication and collaboration of construction participants. With clear visual connections to the elements and/or parts of the structure, all communication and changes remain permanently and clearly visible to the necessary participants (via control rights) and all drawings, technical sheets, etc., are directly linked to the model and available with just a few clicks (current and archived versions of documents).
Changes (especially large-scale) are very rare and mostly must be caused by changes in project settings, technology, etc. All participants are actively involved in the project through earlier stages, structure is already virtually built, and more time is available for detailed cost/time planning before construction, etc.
All this allows participants feel of control and stability over the project. Result is additional energy, good relationships, and better cooperation between associates and ultimately, time&cost benefit to the project itself.

Figure 8 Bridge on site vs virtual bridge (BIM model)

Most of the work for the designers in this phase is only actively monitoring construction site and, if necessary, preparation of additional displays and explanations of technical solutions. Designer can also record changes and enter additional information into the BIM model for later stages of building use. Also, for the Contractor or the Client, he can prepare measure proof of installed quantities and / or the required quantities of materials for ordering, installation, production, and delivery, with automatic (or custom made) reports from the BIM model.

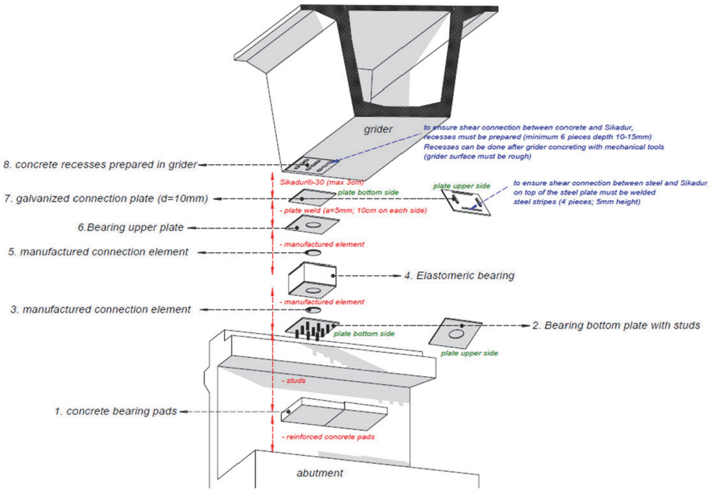

Figure 9 Supplemental 3D detailed installation guides for contractor

3.2 As built and maintenance

As Built documentation is the standard for essential buildings. If the designer actively participates in the construction (updating the BIM model during the construction and thus automatically the drawings), the preparation of such documentation does not require very much time and effort. In addition to the 2D drawings and BIM models, active scanning (3D scan and point cloud) can record the exact position (georeferenced) of all hidden / closed elements (ground installations, piles, etc.) and can be submitted to the Client. Additionally, all certificates and technical documentation and serial numbers of equipment (installation / replacement dates, etc.) can be added to the BIM model as information (and/or document). Such as-built model (with as-built 2D documentation) can be submitted to the Client (or end user) who later can continue to use it for active maintenance of the building/structure.

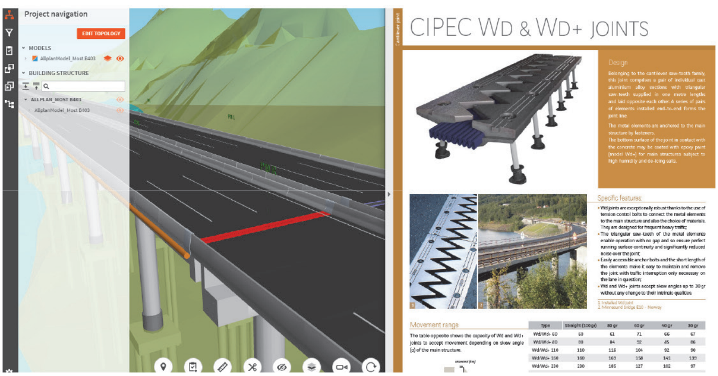

Figure 10 Technical data can be added to BIM model for further maintenance

4 Conclusion

If you could have second chance to do same thing all over again, would you make same mistakes or you would avoid them? If you had necessary information which indicates problem, would it occur on site or you would eliminate it in earlier stage? If you knew all information, exact quantities, execution order, etc. and virtually examined whole structure that you are building, would you plan and organize site in same way? Etc.

These are rhetorical questions, but they can present main BIM advantages in simple examples. Virtual construction, or design and build (and maintenance) with BIM gives you more time (in all project phases) for better planning, optimisations of design and technical solutions, but also a control of the entire project and processes. Financial, organizational, logistical, construction issues (and other) can be recognized in early phases of design and eliminated so that they never even appear as a problem, additional cost, downtime, etc. on site.

Shortly, BIM design gives designer, contractor and client, key information's in right time to control the whole project and related processes.

In profession, common mistake is belief that BIM has advantages only for designer, or at least, mostly for designers. Yes, we spare much time making automatic drawings instead 2D drawing, using automatic quantities and reports instead of calculations, etc., but all that is to invest that time on actual design and solutions to the benefit for all participants in the construction and construction/project itself. Better planning and more efficient processes prevent unexpected issues and therefore additional cost, time, etc.

Another benefit is that Client (and contractor) can be included in early-stage design with clear & attractive visual representation, quantitative information, etc., so that they can actively participate and make decisions/suggestions. On previously shown project, client, contractor (and other designers on route) completely changed their perception about 3D, BIM design, collaboration, etc. but mostly about possibilities and advantages that come with BIM. Biggest "win" was that all structures were designed almost flawless at all and that both, contractor, and client, gained security, stability and (a sense of) control over all parts and processes on the project.

References

[1] Building Information Modeling, https://en.wikipedia.org/wiki/Building_information_modeling, 05.03.2021.

[2] Kolarić, S., Vukomanović, M., Bogdan, A.: Use of BIM in Croatia, GRAĐEVINAR, 72 (2020) 3, pp. 205-214, https://doi.org/10.14256/JCE.2774.2019

[3] What is Building Information Modelling (BIM), https://www.thenbs.com/knowledge/what-is-building-information-modelling-bim, 05.03.2021.

[4] BIM models and digital archive, BIM designers personal communication, etc., KAP4 d.o.o, www.kap4.hr

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REAL-TIME MONITORING AND ANALYSES OF SENSORY DATA INTEGRATED INTO THE BIM PLATFORM

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Abstract

Bridges and tunnels, crucial elements of the railway infrastructure, are exposed to various types of deterioration processes. Their condition is a subject of monitoring, as it is important to collect as much as possible information in every life cycle phase to reliably predict their future performance. An enormous quantity of monitoring data is generated during the whole life cycle of these assets. EU funded Shift2Rail research project Assets4Rail which is focusing on measuring, monitoring, and data handling for railway assets, as data management is as important as their generation. This paper presents the major outcomes of the Assets4Rail project and its application to infrastructure projects.

Keywords: monitoring, data management, bridge, tunnel, Assets4Rail

1 Introduction

Building Information Modelling (BIM) is a model-based process of generating and managing building data during the building life cycle. The concept of BIM was first introduced by Eastman et al. [1] and explained in detail more by Van Nederveen and Tolman [2]. Real implementation and popularity of BIM started at the end of the millennium with various commercially available solutions, which first extended traditionally building design from two-dimensional drawings to 3D modelling (ArchiCAD, AutoCAD, MicroStation). BIM augments spatial dimensions with time as the fourth dimension and cost as the fifth [3]. Thus, nowadays BIM is defined as a digital representation of physical and functional characteristics of a facility and a shared knowledge resource for information about a facility forming a reliable basis for decisions during its life-cycle (6D); defined as existing from earliest conception to demolition [4]. Compared to the area of the average building, infrastructure assets can span several kilometers in one direction while only a few meters in another dimension. Therefore, 3D representation of engineering infrastructure seems less attractive and GIS asset data were transferred to asset management systems for operation and maintenance for a long time [5]. 3D virtual design, construction, and facility management of civil engineering infrastructures are implemented into modern engineering practice to enhance collaboration of all involved stakeholders, resolve conflicts and improve cost-effective performance of infrastructure. Furthermore, a survey conducted by ASCE and associates reveals the recent accelerated application of BIM for Infrastructure, i.e. Infrastructure Building Information Modelling (I-BIM). Engineering firms adopt technology from vertical buildings for infrastructure projects most quickly, but to an extent, they are waiting for demand from their clients [6]. Although spatial visualization does not attract infrastructure owners so much as vertical building operators, it is evident [6] that infrastructure owners often tend to make far more effective use of the information once

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data is collected. In general, they increasingly recognize the benefits of 3D modelling using intelligent objects.
Globally, there are two major vendors; Autodesk and Bentley, and several minor vendors providing software solutions for infrastructure design supporting BIM workflows. Each provider has its data format and object models that are not compatible. Industry Foundation Classes (IFC) is a platform-neutral, open file format specification developed by BuildingSMART [7]. As such, IFC is the most commonly used vendor-neutral format to allow BIM data exchange between different applications and disciplines in the AEC industry. However, in its current state, IFC mostly supports building information with IFC for the infrastructure still being in development stages [8]. Nevertheless, the bulk of object model data such as geometry, properties, relations, etc. can still be transferred from I-BIM via existing IFC standards.
The objective of the EU (Shift2Rail) funded research project Assets4Rail is to contribute to improving the inspection, maintenance, and upgrade methods for cost reduction and quality improvement of railway bridges and tunnels. It aims to improve information gathering and analysis for bridges and tunnels by developing a Building Information Modelling (BIM) platform to optimize inspection, maintenance, and upgrade costs. The project, which started in December 2018, will develop an integrated platform for handling data based on the BIM concept. The BIM approach enables the data layer integration for bridges and tunnels (sensors information, infrastructure geometry, traffic data, loads and fatigue detection, graphical information, etc.) within a single platform. This promising technology will facilitate and optimize the decision-making process regarding maintenance issues and will improve the monitoring of the infrastructure.

2 Assets4Rail project approach

Assets4Rail aims to develop a holistic monitoring data handling procedure based on integrity inspection of railway assets (bridges and tunnels) and processing algorithms built into the information model. It consists of four steps: (1) Monitoring; (2) Information modelling; (3) Fatigue consumption assessment, and (4) Intervention measures. These four circularly related steps offer the development of novel technologies, which will be consequently tested and validated in relevant environments on selected test sections within the project. New alternative automated and enhanced inspection methods will allow faster and more accurate inspection of tunnels and bridges, including improved repeatability and reproducibility. The second step aims to develop novel central information models for data collection and further processing. The third step aims to provide a tool for realistic fatigue capacity assessment for individual structural components and thus to allow for larger axle loads and higher speeds of trains. The fourth step aims to provide a set of novel techniques (e.g., reduction of noise and vibration intensity on structures, cleaning of long tunnel drainage pipes, etc.) which permit an increase in rail traffic, less traffic disturbance due to intervention activities, reduce future problems and prolong infrastructure service life. When the last step is performed, the whole procedure is repeated, starting with monitoring to evaluate how the measures affect the asset performance.

2.1 Infrastructure monitoring

Various monitoring technologies represent the source of data for the assessment of the infrastructure's condition. They can be listed into following groups: (1) manual inspection - visual or using some apparatus, (2) traditional sensors - strain gauges, geodetic instruments, inclinometers, etc., (3) safety and security (S&S) sensors, (4) remote sensing technologies, (5) distributed fiber-optics sensing, (5) wireless sensor networks (WSN), (6) low power micro-electromechanical system sensors (MEMS), and (7) citizens as sensors.

The first three monitoring approaches engage manual inspection or installation of S&S and traditional monitoring sensors. They are still constantly evolving and producing new instruments (models). The remaining five emerging sensor technologies have only recently gained more and more attention. According to [9], sensor and communications research has been going through dramatic innovative changes resulting also in numerous remote sensing technologies including photogrammetric image platforms (drones) and laser or radar sensing systems (scanners, ground-penetrating radar, etc.). Sensing is rapidly becoming part of everyday life not only for health and living but also for the environment and security. Effective use of existing and new smart monitoring systems with a better understanding of how people use the infrastructure services would lead to the realization of resilient adaptable infrastructure systems.

Of the above-listed emerging technologies, we highlight the optical fiber sensing technology because the standard optical fiber becomes the sensor that can be installed to cover even large infrastructure elements to assure continuous and distributed measurements of conditions around the optical fiber (e.g., temperature, strain, acoustic noise, etc.). Its' simple and quick installation and low production cost compared with point measurement sensors, make it ideal for long-term monitoring once the fiber is permanently embedded in a structure.

Wireless sensor networks (WSN) transmit sensor data using radio frequencies. This allows rapid deployment of monitoring instrumentation due to the elimination of some of the cabling. Combined with micro-electromechanical system (MEMS) sensors it is possible to significantly reduce the overall costs for large-scale monitoring purposes. WSN sensors are typically small-sized and low-powered enabling on-the-fly on-board calculations to derive acceleration, inclination, and displacement in real-time without human intervention. Thus, sensor data is not only collected but can also be processed and interpreted using custom-made algorithms that can be embedded into these sensors. This way, users can access final WSN outputs on any Internet-enabled device. MEMS are the product of the ever-increasing miniaturization trend in the design and processing capabilities of emerging sensor systems. MEMS are small integrated devices or systems that combine electrical and mechanical components varied in size from micrometers to millimeters (or even smaller for the next generation nanoelectromechanical systems), which can merge the function of computation and communication with sensing and actuation. These miniature systems can perform measurements ranging from acceleration, strain, inclination, temperature, and pressure. In combination with other sensors, MEMS are integrated into novel instrumentation systems able also to monitor surface defects, e.g., cracks [10].

Structural health monitoring (SHM) has greatly benefited from rapid sensor advances in recent years. But, no matter novel monitoring approaches the question of data management and handling with a huge amount of data remains. Data processing technologies, as well as advances stemming from sophisticated computer aided construction management tools, can help on that significantly. Successful implementation of any SHM system depends on employing the appropriate technical instrumentation and equipment. First, the monitoring data acquisition approach (i.e. choosing the appropriate sensor technology and configuration) is selected to meet the established monitoring requirements. Furthermore, data acquisition and data analysis tools and methods for structure state evaluation are also chosen. Finally, a detailed installation and monitoring operation plan is prepared. Next, monitoring database requirements are defined (if needed) and procedures for data handling and communication should be described in detail to optimize the monitoring system's long-term function avoiding possible data redundancy occurrences. The physical architecture of the SHM system can be very different, depending on various factors, e.g. investigative structure's size, data acquisition rate, level of automation, etc. In Fig.1 an example of the monitoring system architecture is shown into which BIM is integrated as the final data repository unit.

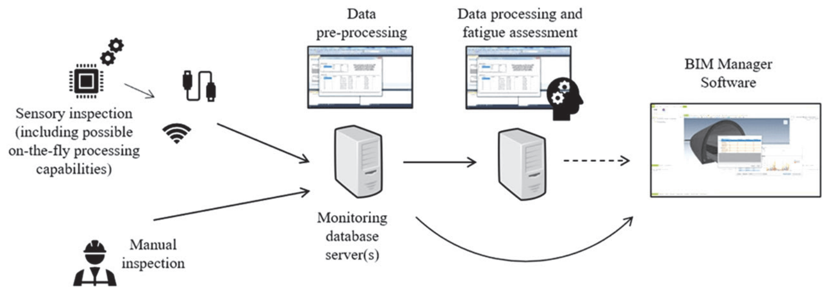

Figure 1 SHM general architecture units

This kind of approach was used for Assets4Rail project. The monitoring results are typically imported on demand via the BIM software's sensor data integration module. Hence, the oversaturation of the BIM model with the monitoring results can be prevented, especially when dealing with SHM of higher data acquisition rates. Thus, BIM environment becomes an optimal environment for visualization of monitoring data and results of analysis based on those data. BIM capabilities enable the convenient presentation of various parameters related to the structure being monitored and they enable an effective decision-making process.

2.2 Bridge and tunnel information modelling system

BIM methodology has seen a rise in adoption across different types of construction projects, and throughout the entire project lifecycle, both isolated and in an integrated manner. BIM software platforms offer a wide variety of use cases, analyses, and scenarios supported by the BIM model, from design development and review, through tendering and construction planning, to construction management. Operations and facility maintenance planning and tracking is the newest addition to BIM, and it is still being developed and improved. Consequently, sensor information within the BIM model has been scarce, especially with a focus on real-time data, and appropriate case-studies with infrastructure projects have been even more scarce. Currently, the state of the art provides a limited subset of the above-mentioned functionalities, as discussed in some recent articles regarding the combination of BIM and sensor data [11]. However, no solution offers an industry-scale, integrated BIM environment with information from the entire project lifecycle (the "single source of truth" approach, with design, construction, cost, asset, operations, and maintenance data) together with real-time sensory data and analyses. Current solutions offer either only 3D geometry and sensor data without asset information and other BIM integrated data, or offer no sensory information in the asset model.
It is a BIMs task to help in collecting, analyzing, and aggregating the huge amounts of data necessary to connect the design of assets to the context of the surrounding environment and its future performance. Employment of parametric engines to make the connections between design and reality is possible [12]. The design model needs to be connected to reality (via monitoring), so that huge amounts of data can be accessed, analyzed, and adapted over time. A possibility of artificial neural network (ANN) employment for decision-making is recognized. Building a neural network forecaster for a particular problem is a non-trivial task. ANN suffers from knowledge extraction and extrapolation uncertainty. Data contained within BIM models about influencing variables should be available based on FEM/FDM of an asset from its structural analysis. Sufficient predictions of infrastructure asset performance can thus be achieved using the feedforward ANN and to consider adaptive-network-based fuzzy inference systems (ANFIS) [13]. By using a hybrid learning procedure one can construct an input-output mapping based on both human knowledge (in the form of fuzzy if-then rules) and stipulated input-output data pairs.

Multiple iterations of very complex analysis also need very powerful computational tools, which can be recognized in today's penetrating cloud-based computing approach. Using virtually infinite power of cloud-based parallel processing it will be possible to simulate analysis of multiple factors in a shared model environment [12] and thus reliable real-time prediction performance of infrastructure assets, particularly bridges and tunnels, impacted by various load cases soon.

The platform being developed within Assets4Rail will incorporate the integrated BIM approach, with real-time and historical sensory data and related analyses. This means that the currently available integrated BIM – tunnel and bridge geometry, element properties, quantities, linked documents, drawings, and other information such as cost, scheduling, operations and maintenance plans and data which can be tracked and managed, will incorporate a new layer of information - sensory data. Specifically, it will include sensory readings, both real-time and historical, enabling this data to be displayed side-by-side with all other relevant asset and maintenance information, 3D model data and properties, already present within the BIM environment.

Additionally to the reading and visualization of sensory data, analyses of that data will be improved by new algorithms based on Bayesian networks, taking full advantage of the gathered information with the help of sensors and making use of synergies derived from the use of BIM approach. Analyzed monitoring results will be present as well, within the same BIM platform, from simple color-coding, alarms, and warnings based on sensory data thresholds, to other more sophisticated analyses. To enhance different visualizations of the information, the goal will also be to work on optimizing the usability of the user interface system, as well as in scenarios of high visual quality for dissemination and communication based on the BIM models and data generated within the framework of the project. Although the IFC standard is not yet at the wanted maturity level for infrastructure, open data exchange can still be facilitated using available schemas and custom model view definitions (MVDs). By establishing well-defined mappings and utilizing already existing element class structures such as IfcSensor and IfcSensorType, as well as time-stamped data types such as IfcTimeSeries, the combined I-BIM and sensor data can be exported in an open, vendor-neutral format, further enriching the open BIM ecosystem.

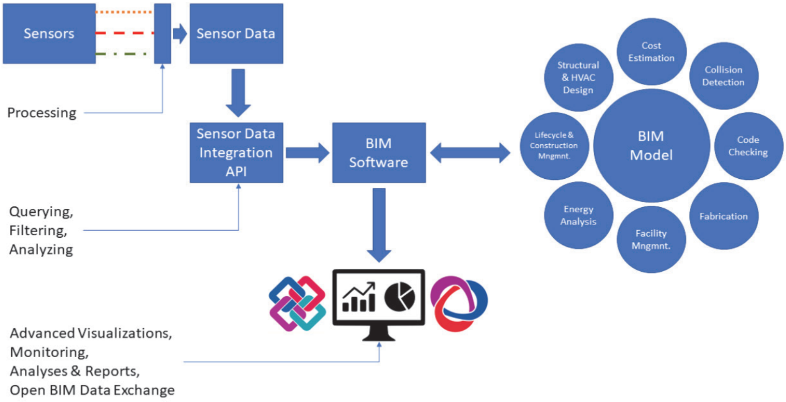

Figure 2 An overview of the Assets4Rail integrated BIM platform solution

2.3 BIM integration example

The following example was developed as a part of “Information Modelling” (WP2) in Work Stream 1 of the Assets4Rail project for demonstrational purposes. The entire solution is built on top of Bexel Manager BIM software platform using its Application Programming Interface (API) to develop a sensor data integration add-in. This extension allows the user to connect to an arbitrary data source (xml and csv files are used in the example) to import the processed sensor readings into the BIM environment. The imported values can then be linked to related assets (e.g. sensor elements or parts of the infrastructure model) by using a 1-1 mapping schema through a unique sensor identifier. On the BIM side, this identifier is defined as an attribute (property) on the BIM element itself, while on the sensor data side it is defined as a data column (Figure 3, left). This kind of mapping enables an automatic bi-directional relationship between the BIM elements and the related sensor data which allows for rich visualizations, advanced filtering, and more. For example, the add-in allows loading sensor data into BIM only for the specified timestamp, but this will be expanded upon further in development. The add-in has a dedicated User Interface (UI) which displays loaded sensor data as a bar chart based on selected BIM element or a specified time range. Once the data is loaded into BIM environment, all the benefits of the BIM software can be utilized. In this particular example, Bexel Manager’s dedicated 3D color-coded view allows the user to easily distinguish between various sensor readings in 3D space and quickly locate a section of the infrastructure which requires attention. Additional documentation can be attached to these elements to provide more details on the issue (e.g. thermal scan images) using the concept of BIM element document linking. All of the integrated data can be exported into an open BIM format using Bexel Manager’s IFC and BCF exchange capabilities to allow information flow between different BIM applications.

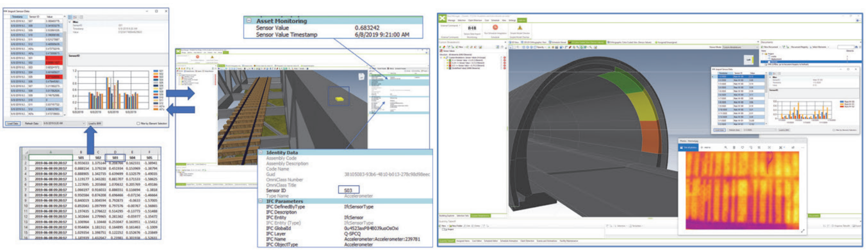

Figure 3 An example of the Assets4Rail integrated BIM platform solution on a railway bridge (left) and tunnel (right)

3 Conclusions

Traditional monitoring includes periodically prepared reports including all necessary information to assist in the planning of the future infrastructure operation. Unfortunately, a large number of data sets become uncontrollable as the number of sensors and frequency of data logging increases. Therefore, despite (or even due to) the huge amount of information, reliable prediction of future performance of infrastructure assets becomes very difficult. Big data needs to be put into context. Thus, BIM should become a central hub for all information about the infrastructure assets from its design and construction onward. At its heart is a computer-generated model that contains all graphical and tabular information about the asset since its design, construction, and operation.

This paper presents the background of the BIM approach in the field of infrastructure management and the issue of proper handling through monitoring data. Decision-making processes based on the data obtained from real-time monitoring are also included. Data generated during the whole life cycle of assets, in the presented case - railway bridges and tunnels, are important information for the prediction of an asset's future performance when combined with the proper expert system. Monitored data management supported by various API for data analysis can be provided by BIM. The use of IFC standards is highly important in these processes.

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References

[1] Eastman, C., Fisher, D., Lafue, G., Lividini, J., Stoker, D., Yessios, C.: An Outline of the Building Description System, Institute of Physical Planning, Carnegie-Mellon University, 1974.

[2] Van Nederveen, G.A., Tolman, F.P.: Modelling multiple views on buildings, Automation in Construction, 1 (1992) 3, pp. 215–24

[3] Wikipedia, http://en.wikipedia.org/wiki/Building_information_modeling, 17.10.2014.

[4] NBIMS-US. Frequently Asked Questions About the National BIM Standard-United States - National BIM Standard - United States, Nationalbimstandard.org., 17.10.2014.

[5] ESRI. GIS for Transportation Infrastructure Management, http://www.esri.com/library/ brochures/ pdfs/transportation-infrastructure.pdf, 2014.

[6] Shuster, L.A.: BIM Is Not Just for Buildings Anymore, Civil Engineering, The magazine of ASCE, http:// www.asce.org/CEMagazine/Article.aspx?id=25769809631#. VEGAeRZRfpo, 10.10.2014.

[7] BuildingSMART. International home of openBIM, http://www.buildingsmart.org/, 17.10.2014.

[8] Infrastructure Room. https://www.buildingsmart.org/standards/rooms-and-groups/infrastructure-room/, 2019.

[9] Soga, K., Schooling, J.: Infrastructure sensing, Interface Focus, 6 (2016), doi: http://dx.doi.org/10.1098/rsfs.2016.0023

[10] Ferri, M., Mancarella, F., Seshia, A., Ransley, J., Soga, K., Zalesky, J., Roncaglia, A.: Development of MEMS strain sensors for crack monitoring in aging civil infrastructures. Smart Struct. Syst, 6 (2009), pp. 225 – 238, doi:10.12989/sss.2010.6.3.225

REVIEW OF THE PROJECT OF RECONSTRUCTION OF THE EXISTING AND RECONSTRUCTION OF THE SECOND TRACK ON THE SECTION HRVATSKI LESKOVAC - KARLOVAC

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Abstract

The Hrvatski Leskovac - Karlovac section is located on the M202 Zagreb GK - Rijeka railway line, which is part of the Mediterranean corridor of the EU core network. The section is currently a single-track line, and represents a bottleneck in terms of infrastructure capacity. The project envisages the reconstruction of the existing and construction of the second track with the reconstruction of the existing stations in order to meet the conditions of interoperability, the transformation of individual stations into stops, and the reconstruction of existing stops. Some of the existing railway-road crossings will be delevelled by constructing crossroads in two levels (underpasses and overpasses), some will be eliminated with the construction of connection roads and some will be reconstructed. The project is currently in the contracting phase of works and supervisions. In the period from 2017 until today, the Ministry of Environmental Protection and Energy (MZOE) Decision was published on the Environmental Impact Study, the Location Permit was obtained, and the Feasibility Study was completed and approved by the JASPERS Mission in the Republic of Croatia.

Keywords: Hrvatski Leskovac - Karlovac rail line, track reconstruction, track construction

1 Introduction

The Zagreb Gk - Karlovac section is part of the main corridor of importance for international traffic M202 Zagreb GK - Karlovac - Rijeka, the starting point or destination of the former Vb corridor Budapest - Rijeka, and now part of the Mediterranean TEN-T corridor, or RH2 corridor. According to the Decision on the classification of railway lines, this international railway line Botovo - Zagreb - Rijeka, on the territory of the Republic of Croatia has the designation RH2 and consists of lines:
- M201 state border (DG) - Botovo - Koprivnica - Dugo Selo
- M102 Zagreb Central Station - Dugo Selo
- M202 Zagreb Central Station - Karlovac - Rijeka
- M203 Rijeka - Šapjane DG (Ilirska Bistrica)

This railway line is important in connecting central Croatia, Gorski Kotar and the northern Primorje, but also in connecting European regional integrations such as the Alps - Adriatic, Mediterranean - Danube and the Central European Initiative.
The project in question deals with a part of the traffic route M202, on the section from Hrvatski Leskovac to Karlovac (stations included) in the length of 44.02 km. This project envisages the reconstruction and modernization of the existing railway in terms of the construction of

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a new track next to the existing one and the reconstruction of the existing railway track on the section Hrvatski Leskovac - Karlovac. Also included is the reconstruction of the Hrvatski Leskovac, Jastrebarsko and Karlovac stations, the conversion of the Horvati, Zdenčina and Draganić stations into stops, and the reconstruction of the existing Mavračići, Desinec, Domagović and Lazina stops.
Reconstruction and upgrading takes place in the narrower corridor of the existing railway, following the position of the route of the existing railway.
After the implementation of the project, which also includes the reconstruction of some LCPs and modernization of all other railway infrastructure subsystems (electricity and traffic management and signaling - safety), and in full compliance with applicable regulations governing rail transport, including interoperability, the line will allow passenger traffic at a design speed of up to 160 km / h, and freight traffic up to 120 km / h.

2 Route geometry and track structures

On the existing line on this section, the maximum permissible train mass is D4 (22.5 t/o and 8 t/m). The maximum current speed per area is: Hrvatski Leskovac - Zdenčina 110 km/h (conventional trains) or 130 km/h (trains with tilting technique), Zdenčina - Jastrebarsko 140 km/h, Jastrebarsko - Draganić 80 km/h with a limit in the curve behind Jastrebarsko at 75 km/h, Draganić - Karlovac 100 km/h. The railway is electrified with the AC 25kV / 50Hz system.
The type of insurance is APB, while relay devices are installed in the stations. The traffic takes place in a block distance.
The line is envisaged as conventional, for mixed passenger and freight traffic.
New line retains the existing geometry on most of the route. The distance between the rails will remain 1.435 mm, the distance between the new track and the existing track on the open track will be 4,75 m. Applied minimum radius of the horizontal curve is 700 m and the maximum 25.000 m, while stopping way is 1.500 m.
For vertical geometry, the maximum longitudinal slope is 12 mm/m. Vertical curves have minimum radius of 10.000 m, and maximum of 40.000 m. Level of the railway after reconstruction basically follows the existing level, with deviations in some sections in the range from +2,00 m to -0,41 m, so it can be generally stated that the level of the railway after reconstruction and upgrade is slightly highrt than the existing condition. Distance between tracks at stations and at AV junctions the rail is 4,75 m, and railway will be constructed for GC profile and electrification is AC 25kV / 50Hz.
The superstructure of the track consists of: rails type UIC 60E1, prestressed reinforced concrete sleeper 260 cm long and elastic fastening accessories.
The minimum thickness of the gravel curtain is 30 cm below the concrete sleeper at the side of the lower rail, and on bridges 40 cm below the lower rail.
The substructure of the railway consists of a bed 40 cm thick, geotextile for open drainage, and geomembrane for closed drainage, embankment core, soil stabilization where necessary and protective coverings of the slope with humus 30 cm thick.
As part of the reconstruction of the railway, there is also a drainage system, canals, drainage ditches, noise protection walls, service roads.
According to the state of design solutions, the characteristic cross-sections of the designed line have a total planum width of 13.35 m and the track spacing is 4.75 m. Depending on the situation and available space, the circumferential ditch and service road are added on one or both sides.
A 0.4 m thick protective layer and a geotextile layer are located between the ballast and the embankment bed. The protective layer has a bed-like slope, of 3 % towards the ends of the planum to ensure the removal of water from the body of the track. On the parts of the route

where closed internal drainage is planned, the geotextile is replaced by a geomembrane and elements for collecting rainwater and draining to treatment plants are added.
The upgrade of the second track will take place by step excavation of the existing embankment and connection with the existing embankment in phases.
The first step envisages the cascading design of the existing embankment on the side where the construction of the new track will be performed. After that, a new embankment and all layers in the hull of the railway, including the upper structure of the new track, are built. After the traffic is moved to the new track, part of the embankment and the upper structure of the existing track will be renovated, in accordance with the design solutions and depending on the condition of the existing embankment.
The implementation of the project is in principle planned without interruption of traffic, ie at the same time as the traffic. Exceptions to this are situations of switching between the northern and southern tracks and the construction of switch areas, when complete closures of traffic for a certain period of time are possible.

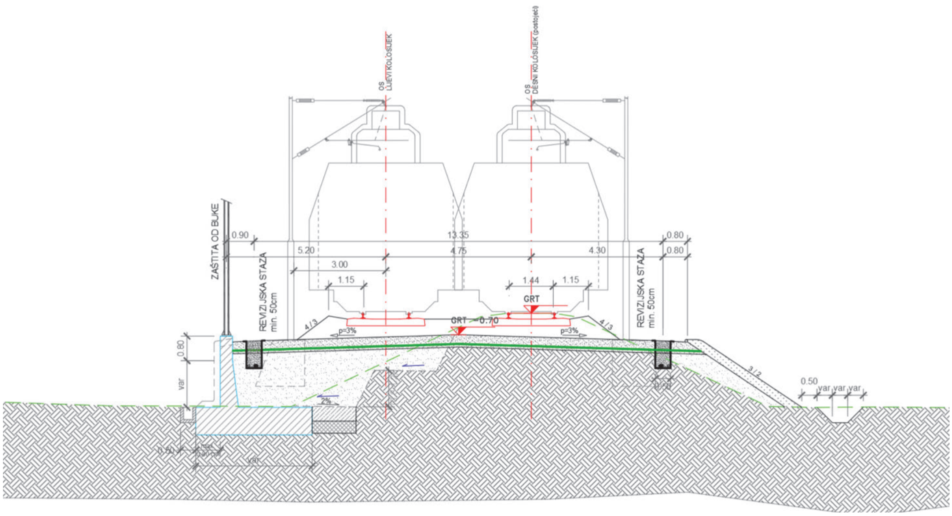

Figure 1 Cross-section of a double-track rail line

3 Stations and stops

The project envisages the reconstruction of 3 stations, the conversion of 3 stations into stops and the reconstruction of existing stops. The project envisages that existing stations Hrvatski Leskovac, Jastrebarsko and Karlovac will be reconstructed. Stations Horvati, Zdenčina and Draganić will be converted to stops, and existing stops Mavračići, Desinec, Domagović and Lazina will be reconstructed and upgraded. Stations that allow the retention of interoperable freight trains will be able to accept trains up to 750 m in length. Stations that allow the retention of interoperable passenger trains will have a platform length of 400 m, and other stations and stops with a platform length of 160 m.

3.1 Hrvatski Leskovac station

As part of this project, the reconstruction of the Hrvatski Leskovac station is planned in order to meet the conditions of interoperability. In relation to the current situation, the reconstruction of the entire station is envisaged, except for the part where the track for the necessary local industries is located. It is planned to build platforms for receiving passengers and underpasses. In order to achieve a useful track length of 750 m, the station will be extended in the direction of Zagreb, since in the direction of Karlovac it is not possible to perform an extension due to construction.
The existing building with a toilet is being reconstructed on the same site. A new station building is planned on the west side of the existing building. The new building will house the new ESSU, TK equipment, uninterruptible power supply and a new traffic office (since the station will be occupied after the reconstruction). The building will be single-storey.
The installation of a new traffic - management and signal - safety infrastructure subsystem, as well as the electricity infrastructure subsystem in accordance with the new track plan of the station is planned.

3.2 Jastrebarsko station

Reconstruction of Jastrebarsko station is also planned in order to meet the conditions of interoperability.
After the reconstruction, the number of tracks in the station will not change compared to the existing condition. In relation to the current plan, the reconstruction of the entire station is planned, except for the part where there are industrial tracks for the necessary local industries (Betongrad, Drvoproizvod), the construction of platforms for receiving passengers and underpasses is planned.
By upgrading the second track of the open track on the north side of the existing one, it is necessary to build a new 1st track, and the existing tracks 4 and 5 will be completely dismantled. The existing curve on the exit side towards Karlovac is maintained and the construction of a new deviation for 160 km/h is not planned. In order to achieve a useful track length of 750 m, the station will be extended in the direction of Zagreb
The existing station building is being reconstructed in order to accommodate the devices and arrange the waiting rooms and conversion of individual rooms. The part of the station building in which the rooms with the SS device and the traffic office are located will remain in function until the works on the new ESSU are completed, ie as long as the APB is in function. After putting the new device into operation, the premises can be rearranged for other purposes. The part of the building where the toilet is now located is being demolished to build a parking lot.
For the needs of passenger transport, the construction of two side platforms with a length of 160 m is planned. They are planned along the first and fourth track, respectively, and their connection is provided by an underpass with a staircase and elevators.
The installation of a new traffic - management and signaling - safety subsystem and electric power infrastructure subsystem is planned in the station in accordance with the new track picture of the station.

3.3 Karlovac station

The project plans a complete reconstruction of Karlovac station in order to meet the requirements of interoperability, except for the part related to the bridge over the river Kupa on the exit side towards Mrzlo Polje and the track for garaging DMV.

After the reconstruction, the function of the station will not change in relation to the existing one and the station will be functionally divided into two parts, the first part of the station (tracks 1-4) will be used for traffic on the Zagreb GK - Rijeka line, and tracks 5. - 7. for the railway Karlovac - Kamanje – State border. In relation to the existing situation, the number of tracks is reduced from 12 receiving-dispatching or shunting to 7 receiving-dispatching tracks. The existing track 9 will be completely dismantled, and the tracks 10 and 11 will be partially, ie the tracks 11 and 12 will become tracks 8 and 9, and will serve as connecting / pull-out tracks for the garage part of the station.
Due to the extension of the track in the direction of Zagreb, and the construction of a new connection for the railway Karlovac - Kamanje – State border, dismantling of three tracks 12, 13 and 19 is planned. Due to the construction of the side platform next to the station building, track 15 is also dismantled. Under the current conditions, maintenance TMDs are installed on said track, and it will be placed on track 9 after reconstruction with respect to a sufficiently useful length.
The installation of a new traffic - management and signal - safety infrastructure subsystem and the electricity infrastructure subsystem is planned in the station in accordance with the new track picture of the station.
The construction of an island platform 400 m long is planned between 4 and 5 tracks, which will result in the dismantling of the existing 5 tracks. In addition to the island platform, between the station building, 1. and 1a. the construction of a side platform with a length of 400 m is planned. The connection of the platform is planned by an underpass with elevators. An underpass is provided under the entire station to allow the unimpeded arrival of passengers and to the right side of the station. The entrance to the underpass is planned next to the existing station building. Along with the planned exit from the underpass on the west side, the construction of a new parking lot is planned.
Since the station building and the canopy next to it are registered as a protected cultural asset, the solution of the new side platform has been adapted to this and the supporting pillars of the canopy. In the station area, only the removal of the building is planned for the construction of a new connection to the L104 line, a dilapidated building next to the Kupa bridge and several smaller dilapidated buildings for the construction of a noise protection wall. In the existing station building, interventions are planned in certain rooms for the installation of new SS devices and equipment, and a common entrance area, corridors and a toilet on the ground floor are being renovated.
The construction of a new pedestrian and bicycle underpass at Karlovac station is also planned, which will connect the newly planned parking lot and the existing underpass under DC1 (V. Holjevac Street).

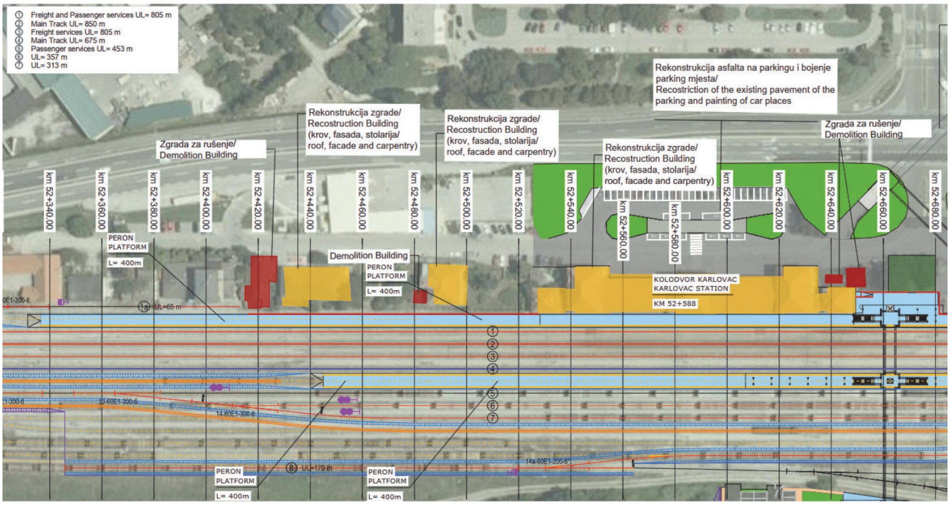

Figure 2 New track plan (red) with pedestrian and bicycle underpasses and arrangement of Karlovac station

3.4 Stops

The existing stations Horvati, Zdenčina and Draganić are being converted into stops. Existing tracks are removed completely, as well as all switches, devices and equipment.
The existing station building in Horvati is being removed, and in Zdenčina and Draganić (cultural property) they remain and are being renovated. Parking lots with accesses from the local road network are being built next to all stops.
At all stops, two side platforms 160 m long, 0.55 m high above the upper edge of the rails will be built, connected by an underpass with a staircase and elevators and equipped with canopies 100 m long.

4 Railway facilities

4.1 Railway crossings with roads

Depending on the importance and rank of the road, and the traffic load on it, the intersections of the road network with the railway are solved by denivelations or arrangement in level with the complete equipment of the LCP. Some pedestrian or crossings on uncategorized roads are abolished and reduced to adjacent locations.
The underpasses of Bedekova Street in Hrvatsko Leskovac, the underpass in Lazina and the Ribnjak underpass in Draganić are planned. Overpasses in Pavučnjak, Zdenčina, Cvetković and Domagović have been solved.
LCP Demerje, Stupnik, Desinec, Draganić and Zagrebačka (in Karlovac) remain level crossings, equipped with appropriate signalization in accordance with regulations.
LCP Orlovac is being abolished and the construction of a new road through the industrial zone in Karlovac is planned.
Underpasses are monolithic frame structures that translate the road below the track.
Overpasses are constructions combined from concrete and steel elements. The end spans are made of concrete T-beams with a monolithic slab, and the span across the track is a composite structure of steel girders coupled with a monolithic slab. The overpasses are 85-135 m long.
Pedestrian paths are run through individual underpasses and overpasses, and the corresponding part of the road on both sides of the building is arranged.

4.2 Structures in therailway substructure

It is planned to demolish the existing structures in the substructure of the railway because they do not meet the condition or geometry of the conditions for the construction of the second track and the construction of new ones. Of the larger facilities in the route are the bridge Ilovac 1 (15 m) and the viaducts Ilovac 2 (32 m) and Kupa-Kupa (100.5 m). At all three locations, the existing facility is being renovated and a new one for the second track is being added. In addition to these facilities, it is planned to install 50 smaller bridges or culverts over the existing watercourses in the hull of the railway. These constructions are prefabricated frame elements (box elements) of openings 6.0 * 4.0 m, 3.5 * 5.0 m, 2.5 * 3.0 m, 2.0 * 4.0 m which are in the profile of the watercourse. stack individually or two or three in a block, depending on the required hydraulic parameters of the watercourse.

4.3 Installations, security systems, equipment

It is planned to install ESSU in the stations Hrvatski Leskovac, Jastrebarsko and Karlovac and an electronic automatic track block (APB) on the sections Hrvatski Leskovac - Jastrebarsko and Jastrebarsko - Karlovac. The so-called “Multistation” solution will be used, in which one signal-safety station device (at Karlovac station) controls and manages several stations and their external elements.
The stations Hrvatski Leskovac and Jastrebarsko will have the possibility of local management from the local traffic office and the possibility of remote management from the station Karlovac.
Railway - road crossings are provided with a new electronic device for securing the LCP, which must have a technical dependence with electronic signal - safety devices.
The telecommunications system along the Hrvatski Leskovac - Karlovac railway will be completely renovated, and the existing SDH will remain during the construction works.
Appropriate optical cables for the transmission of all types of data are installed along the entire section. The section will be equipped with a telephone system, a radio system, ticket vending machines and all passenger information devices.
Reconstruction of two EVPs, Zdenčina and Mrzlo Polje, is planned for the supply of electricity to the section and the supply of the catenary.
The construction of two new sectioning plants, PSN Hrvatski Leskovac and PSN Draganić, is planned.
The catenary will be provided in accordance with the new solution and the new configuration of the station. The equipment for overhead lines will be designed for a maximum permitted train speed of 160 km/h.

5 Conclusion

The main projects have been prepared, the remaining building permits are being obtained, the land has been purchased and a tender for the execution of works is in the process. At the beginning of March 2019, the project was submitted to an independent EC quality control office and a positive decision was made to finance the project, with a total estimated cost of eligible costs of € 366 million. The grant contract was signed on 27.12.2019. the MZOE Decision was published on the Environmental Impact Study, the Location Permit was obtained, and the Feasibility Study was completed and approved by the JASPERS Mission in the Republic of Croatia. The main projects have been prepared, the remaining building permits are being obtained, the land has been purchased and a tender for the execution of works is in the process. At the beginning of March 2019, the project was submitted to an independent EC quality control office and a positive decision was made to finance the project, with a total estimated cost of eligible costs of € 366 million.

References

[1] Upgrade, construction of second track and construction of new double track line on sub-sections of railway line section Hrvatski Leskovac - Karlovac, Feasibility Study - Phase 1, Identification of the solution based on option analysis and decision tree, Italferr SpA in association with IRD and Technital, Roma, May 2017.

[2] Preliminary design Reconstruction of the existing and construction of the second track on the section Hrvatski Leskovac - Karlovac on the main railway for international traffic M202 Zagreb Gk - Rijeka, Italferr SpA in cooperation with IRD and Technital, Zagreb / Rome, November 2017. rev June 2018.

[3] Main design Reconstruction of the existing and construction of the second track of the railway for international traffic M202, on the section Hrvatski Leskovac - Karlovac, Italferr SpA in cooperation with IRD and Technital, Zagreb / Rome, November 2018.

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POSSIBILITIES OF RAILWAY CONNECTION BETWEEN RIJEKA AND TRIESTE WITHIN THE EUROPEAN TEN-T NETWORK

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Abstract

In the existing railway network of the Republic of Croatia there is no direct connection between Rijeka and Trieste via Istria. It is theoretically possible to achieve this indirectly through several railway lines crossing Slovenia. As a result, our largest peninsula of Istria has remained completely isolated from the rest of the country from the point of view of rail transport. Trieste-Rijeka section is not included in the Mediterranean corridor, that is, in the basic TEN-T network. Some research [1], [2], [5] and facts show that there are numerous reasons for the construction of a new modern railway line between Rijeka and Trieste, which could be of great importance in the transport, technological and economic system of the Republic of Croatia and the European Union. The new concept of railway connection between Rijeka and Trieste envisages the construction of a new high-efficiency two-lane railway across the territory of Istria and partly across the territory of the Republic of Slovenia. In fact, the construction of a new railway line is foreseen on the Rijeka - Jurdani - Divača route, while the Divača - Trieste section is foreseen for reconstruction and modernization within the investment program of the Mediterranean Corridor. According to above research, this connection can be achieved in two ways: according to the first, the connection can be achieved by upgrading and modernizing existing lines on the route Jurdani - Pivka - Divača, and the second, by constructing a new line on the route Jurdani - Lupoglav - Divača. This second method of connection has the advantage in that it directly connects the Istrian railways into a complete system of Croatian railways.

Keywords:

1 Introduction

In the existing railway network of the Republic of Croatia, there is no direct connection between Rijeka and Trieste via Istria. Theoretically, it is possible to achieve it indirectly through several railways corridors that cross the territory of Slovenia. Thus, our largest peninsula, Istria, from the point of view of railway traffic, remained completely isolated from the rest of the country.
The Primorje-Gorski Kotar County, in partnership with a number of public bodies led by the Friuli Venezia Giulia Region of Italy, was involved in the “ADB Multiplatform” project, a cross-border project with the full name “Adriatic-Danube-Black Sea multimodal platform”.

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The intention of the project [3], was to develop and promote environmentally friendly, multimodal transport solutions from ports in the program area of Southeast Europe (Black Sea, Aegean, Adriatic) to inland countries and regions along the selected pilot transnational network. This will be achieved through the development and establishment of a multimodal transport development platform that integrates different regions and stakeholders from the transport industry. As part of this project [3], a study was conducted and a study of framework options for connecting the northern Adriatic port system by high-efficiency railway [2], as a platform for undertaking appropriate activities at EU and Croatian level to include a new line in the Mediterranean Corridor, and thus in the basic TEN-T network. EU, and in the ADB multiplatform concept which outlines the efficient connection of the northern Adriatic ports with the Danube and the Black Sea.
It should be noted that the proposed network concept fits in and does not undermine the importance of the project of a new railway from Rijeka to Zagreb and further to Hungary. In this research in the context od studies [3], special emphasis was placed on the exceptional need for this project as a complete solution of the railway transport network in Croatia.

2 About the project

There is no direct connection between Rijeka and Trieste in the existing railway network. Theoretically, it is possible to achieve it through several railways that cross the territories of Croatia, Slovenia and Italy. These are the railways: Rijeka-Šapjane DG-Ilirska Bistrica-Pivka, then the railway Pivka-Divača-Sežana and finally Sežana-Trieste. Existing railways have very unfavorable technical and technological parameters:

- across Divača and Pivka railway it is 122 km long, which is almost 25 % more than the planned new railway (80 km)
- almost all lines have a maximum allowed longitudinal slope of 25 mm/m
- the maximum speed on the mentioned lines is 60/80 km/h

It follows from the above that the existing lines are not favorable for the establishment of direct railway traffic Rijeka-Trieste, so they do not run any direct train between these destinations.

2.1 New European policy for transport infrastructure TEN-T

The global concept of the core network is based on the fact that transport is crucial for the efficiency of the European economy. Without good transport links, the European economy will not be able to grow and develop. As a means of boosting growth and competitiveness, a strong European transport network is being set up under the new EU infrastructure policy, covering 27 Member States. It will be a true European network that will connect East and West and thus eliminate the current traffic fragmentation of Europe.
The new infrastructure policy is tripling the EU’s transport budget. At the same time, the focus of transport financing is shifting to a clearly defined new core network. The core network will be the mainstay of the European single market. It will remove bottlenecks, modernize infrastructure and increase the flow of cross-border traffic. As a first step in establishing this network, nine main transport corridors have been set up, connecting Member States and allowing them to pool their resources to achieve better results.
The new core TEN-T network will be supported by a comprehensive network of traffic routes that will flow into it at regional and national level. The goal is to gradually ensure, by 2050, that the vast majority of European citizens and businesses are no more than 30 minutes away from this comprehensive network.

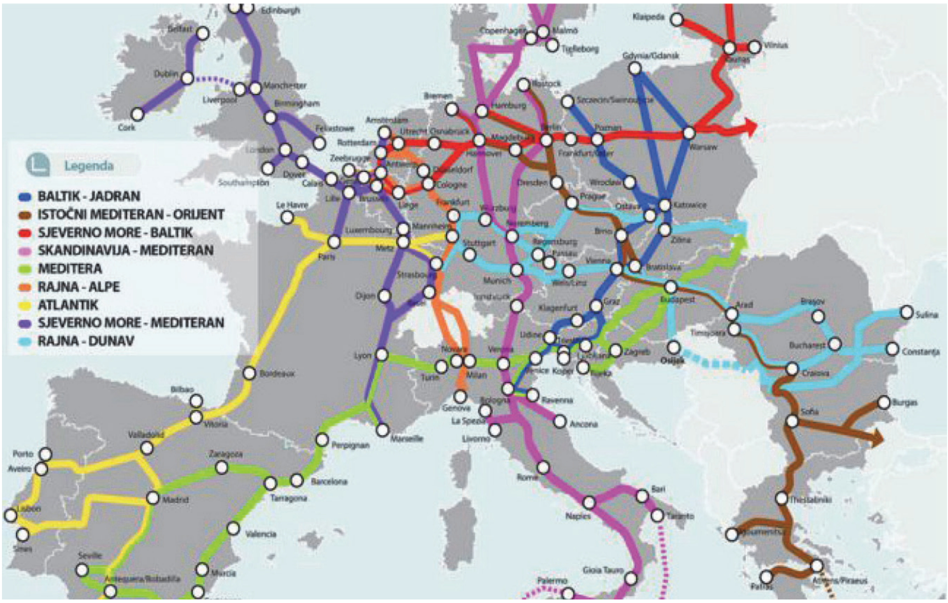

Figure 1 TEN-T core network corridors

Furthermore, by 2050, freight traffic is expected to increase by 80 % and passenger traffic by more than 50 %. Adequate trade is needed for growth, and it is not possible without turnover. European areas that are not well connected will not be able to develop as planned. The lack of links, especially on cross-border sections, is a major obstacle to the free movement of goods and passengers within and between Member States, but also towards neighboring non-EU countries. There is a large gap in the quality and availability of infrastructure between Member States, and often within them. Improvements are particularly needed on East-West connections, which can be achieved by building new transport infrastructure and / or maintaining, renovating or modernizing existing ones.

The transport infrastructure necessary to connect the various modes of transport is also incomplete. Many European freight and passenger terminals, land and sea ports, airports and city hubs are not up to the task. Due to the weak multimodal connectivity in these hubs, the possibilities of combined transport, which could solve the problem of bottlenecks and the lack of connections, are underused. Investment in transport infrastructure should contribute to the 60 % target of reducing greenhouse gas emissions in transport by 2050. Member States still have different operational rules and requirements, especially in the area of interoperability, which further increase infrastructure barriers and bottlenecks.

A major novelty of the new TEN-T guidelines is the introduction of nine corridors that make up the core network. Each corridor must cover three modes of transport, three Member States and two cross-border sections. Corridors that stretch in area and are important for Croatia are the Mediterranean and Rhine-Danube corridors. Within the new TEN-T network, the importance of the Mediterranean corridor was especially emphasized, which is extremely important for Croatia and for the project of the planned new Rijeka-Trieste railway. It connects the Iberian Peninsula with the Hungarian-Ukrainian border, follows the Mediterranean coast of Spain and France, passes the Alps where it turns to the east and north of Italy, from where it crosses the Adriatic coast of Slovenia and Croatia to Hungary. It can be seen from the above that the planned new Rijeka-Trieste railway is an integral and unavoidable part of this corridor.

2.2 Objectives of the planned project

The main goals of this project would be:
- development of a network of multimodal hubs in the southeast area with common quality and performance standards related to innovative IT and transport services
- development of accessibility and trade within Southeast Europe and the mentioned corridors
- multimode development transport as a real alternative to inland roads but also the inclusion of Adriatic / Aegean / Black Sea ports through joint activities for the development of multimodal transport
- development of the railway as a reliable solution for economic entities in the southeastern area, through the development of a railway corridor connecting the Black Sea with the mainland countries, with branches towards the main ports on the Adriatic;
- integration of rail and river transport through the strengthening of major rail and river hubs and the promotion of intermodality on the rail-inland waterway route;
- environmental protection in the area of Southeast Europe through the change of modalities of transport from roads to railways and inland navigation, the development of international agreements for the development of regulations for the internalization of external costs.

2.3 Railway development opportunities in the Republic of Croatia

The railway system in Croatia has organizationally adapted to the railway system in the EU. Now follows the harmonization of the elements of railway infrastructure in Croatia with that in the EU. In order to create a single railway area in the EU and liberalize access to railway infrastructure, the railway system in the EU countries (and candidate countries for EU accession) has been divided into infrastructure and carriers. Infrastructure has been declared a public good and the care for the maintenance and development of railway infrastructure has been taken over by the state.
As independent economic entities on the railway services market, railway undertakings equally impose themselves in the provision of railway services, and take care of their operations and development plans. Regulatory bodies established in the Member States should provide unhindered and non-discriminatory access to railway infrastructure for railway undertakings.
The EU transport system is giving increasing importance to transport branches that have a less negative impact on the environment, and contribute to the overall efficiency of the transport system and the reduction of total transport costs. Preference is given to combined and intermodal transport, with an incentive to develop water (inland waterways and maritime transport) and rail transport. Croatia has all these components of the transport system, so we can expect more generous support for their development as part of the development of the entire EU transport infrastructure.
In addition to the construction of a new high-efficiency railway Rijeka-Zagreb, one of the primary directions for the development of transport infrastructure in the Primorje-Gorski Kotar County is to consider the development of high-efficiency railway infrastructure on the Rijeka-Trieste route. Apart from being the backbone of the North Adriatic transport route, which is the shortest and most economical route to the Mediterranean and further to Asian countries, this transport route will be the starting point for further connecting the transport infrastructure on the Adriatic-Ionian transport route whose main task is transport and economic integration. countries along the corridor (Italy, Slovenia, Bosnia and Herzegovina, Montenegro, Albania, Macedonia and Greece).

In the case of the port of Rijeka, connecting to Trieste by rail would be vital for integrating regional and urban economy and competitiveness. Namely, such a connection would open the door for Croatia to the whole of Italy and further through the railway freight corridor 6 (RFC 6) to EU countries, which would greatly contribute to the overall regional development, and this cannot be achieved through the existing railway.
With this new line, Istrian lines could be connected to the Croatian railway network within the territory of the Republic of Croatia, which would mean a strong economic rise induced by a quality railway network directly for the Istrian county.
The construction of a new high-efficiency railway Rijeka-Zagreb and a new railway Rijeka-Trieste will shorten the connection between Southeast Europe with its central and western part, which will increase the exploitation of transport infrastructure in the Republic of Croatia, which will lead to job creation. , creating economic growth and increasing government revenue.
It is to be expected that this railway will greatly stimulate economic activities in the area through which it passes. According to the Transport Development Strategy of the Republic of Croatia, specific objective 5b states the need to improve accessibility in freight transport - North Adriatic (Rijeka), according to which the commitment is that the railway line between Trieste and Rijeka gained better and safer access to the Danube corridor and further east to the Black Sea, and would import the Adriatic-Ionian initiative into the system.

3 Variant solutions for the new railway from Rijeka to Trieste

Three possible variant solutions for the railway connection between Rijeka and Trieste were considered. An analysis of traffic demand was made, an analysis of the financial and economic aspects of the new railway. By multi-criteria analysis of the proposed corridors, a preferred solution was nominated.
Variant solutions were consideredthe possibility of connecting to the railway network of Italian railways, and previously by connecting to Slovenian railways, in order to analyze the possibility of connecting with the ports of Koper and Trieste.
Variant solutions were also considered the possibility of connecting to the existing and planned transport (railway) network in Croatia, Slovenia and Italy in accordance with national development plans as well as EU development plans. The railway was planned as a two-track, and in the economic evaluation the cost-effectiveness of the two-track was examined with regard to the traffic forecast, ie the phasing in the realization of the two-track.

3.1 Project solutions

The boundary elements of the open line route (floor and height elements) are determined for a conventional line with a design speed:
- V_{max} = 160 km / h.
- $V_{freight}$ = 100 km / h.

In the area of Primorje-Gorski Kotar County and the corridor from Jurdani to Pivka, two basic variants were analyzed:
- V1: new railway route with a detour of Šapjane and Ilirska Bistrica with a connection to the Ljubljana - Divača railway in Pivka,
- V1A: a variant that goes around the settlements north of Jurdani (in this context a larger tunnel appears), uses part of the existing line (with the necessary geometry corrections) and ends the same at the Pivka station. In terms of “environmental protection”, there are basically no significant differences between variant V1, but it is less investment-friendly.

In the Istrian County, 5 variants were analyzed in principle, primarily due to finding a possible connection of Istrian railways to the new route, and the connection of the new route to Slovenian railways, whether on the line Koper - Divača or directly in Divača.
Namely, the configuration of the terrain and protected areas in Istria are extremely complex. In addition, HŽI made the decision to abandon the Učka tunnel, so the connection of the Istrian railways became even more complex. In such circumstances, the only possibility was the connection between Judani and Istria by a tunnel through Ćićarija, and only so that the tunnel exits in front of the Lupoglav station. In that way, the station would remain in function and new railways (with renovation), which would enable the connection of the existing railways for Pula, Raša and Buzet to the new railway. With a minor deviation of the existing line to Pula in the station zone, direct rides from Rijeka to Pula and vice versa are provided. The route of the existing railway for Buzet and Raša remains as it is, and the connection to the new railway is provided at the Lupoglav station. The following variants of the route were considered in Istria:

- V2: new route of the railway from Jurdan through the Ćićarija tunnel to the Lupoglav station, and further a new route in the wider corridor of the existing railway (elements of the railway for 160 km/h) towards Divača,
- V2A: a new route from Jurdan through the Ćićarija tunnel, but the exit from the tunnel does not allow a connection with the Lupoglav station, but the route would remain on the plateau above the station, which would result in one tunnel less than in V2. In the continuation, the new route is in the wider corridor of the existing railway (elements of the railway for 160 km/h) towards Divača,
- V3: new route of the railway from Jurdani through the Ćićarija tunnel to the Lupoglav station, and further a new route (elements of the railway for 160 km/h) towards the connection to the railway Divača - Koper near Črni Kal. Technically extremely complex and demanding route considering the relief and population,
- V3A, V3B: subvariants of variant V3 in the part from Lupoglav to Črni Kal, primarily trying to overcome the complex relief in the wider surroundings of Buzet.

In the group of variants 3, there would be a particularly technically complex connection of the railway to the Divača - Koper railway (with a large viaduct that should be connected to the Črni kal viaduct on the Divača - Koper railway). The connection of the viaduct to the viaduct cannot be avoided, because before and after the Črni Kal viaduct there are tunnels, which makes this connection even more difficult. Very demanding technical solutions and connection conditions (on the open line, without the station) cause exceptional costs and put the group of variants 3 in a less favorable position than the other groups of variants.

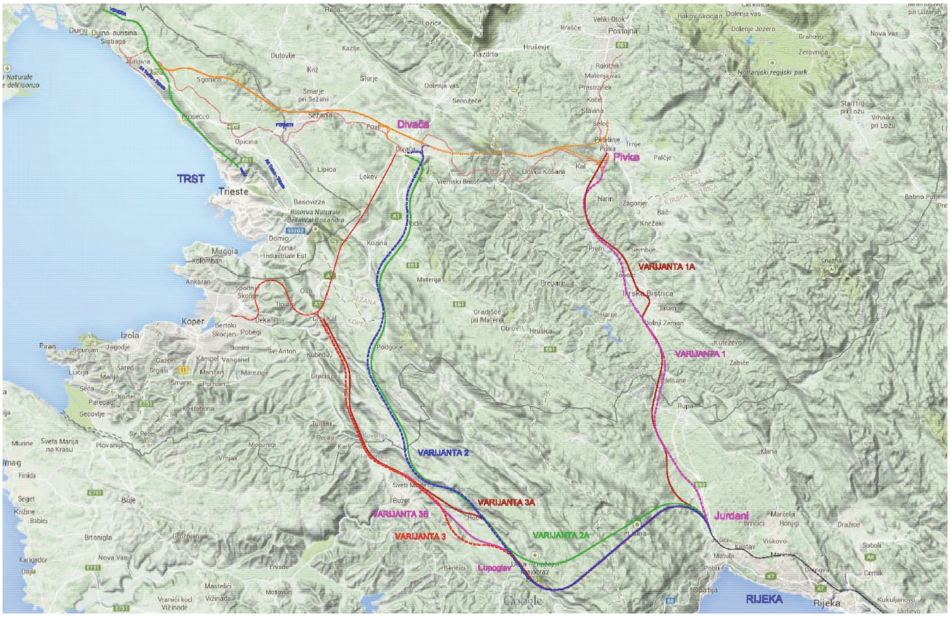

Figure 2 Spatial position of the investigated corridors with variants of the Rijeka - Trieste railway

Based on the elimination criteria and expert assessment, of the analyzed variants for further analysis, three were proposed: V1, V2 and V3.

- Variant 1: Jurdani station - Pivka - Divača station;
- Variant 2: Jurdani station - Divača station;
- Variant 3: Jurdani station - connection to the new railway Koper - Divača

In order for the variants to be analyzed and properly valorized, they always started in the same station (Jurdani station). The lengths of the sections in question, measured from the middle of the Jurdani station to the middle of the Divača station (in Slovenia), as the end points of all variants, are:

- according to Variant 1: 51,688 km
  (Rijeka - Pivka 35,857 km Pivka - Divača 15.83 km)
- according to Variant 2: 66,630 km
  (Rijeka - Lupoglav 20.00 km, Lupoglav - Divača 46.63 km)
- according to Variant 3: 66,980 km
  (Rijeka - Lupoglav - Črni kal 50.98, Črni kal - Divača 16.00 km)

From the Divača station, the reconstruction is planned, ie the construction of a new two-track railway to Trieste and to Ljubljana. In these analyzes, these corridors were taken over, and new routes from Rijeka were planned so that a connection to this planned line would be established. The combination of the selected variant from Rijeka to Divača and further with a new connection to Trieste (Aurissina) and Koper, ensures the interconnection of the ports of Rijeka, Trieste and Koper, ie their connection to the 6th freight corridor TEN-T network (via the newly planned line Trieste - Aurissina - Palmanova - Venice).

Figure 3 Newly planned corridor (blue) Divača - Sežana - Aurissima (connection to the Trieste - Venice railway) (green)

The total length of the route according to variant V1 is 35,857.19 km, of which 13 km of the route is in the territory of the Republic of Croatia and the remaining 22 km of the route is in the territory of the Republic of Slovenia. In variant V1, the total length of bridges and viaducts is 6.56 km, and tunnels 28.91 km (the share of buildings is 55.15 % of the section length). Longitudinal slopes range from 1-12 mm/m.
The total length of the route according to variant V 2 is 64,320 km, of which 35.4 km of the route is in the territory of the Republic of Croatia and the remaining 28.92 km of the route is in the territory of the Republic of Slovenia. In variant V2, the total length of bridges and viaducts is 11.55 km, and tunnels 7.74 km (the share of buildings is 53.82 % of the section length). The longest tunnel is Ćićarija, 15.57 km. Longitudinal slopes range from 3-12 mm/m.
The total length of the route according to variant V3 is 50,329 km, of which 38.5 km of the route is in the territory of the Republic of Croatia and the remaining 11.8 km of the route is in the territory of the Republic of Slovenia. In variant V3, the total length of bridges and viaducts is 16.19 km, and tunnels 20.16 km (the share of buildings is 72.13 % of the section length). The longest tunnel is Ćićarija, 15.57 km. Longitudinal slopes range from 0-10 mm/m.
Variant 3 (as well as variant 2) has the advantage of connecting Istrian railways into a complete system of Croatian railways, but it is extremely technically and technologically complex. The route from Lupoglav to the connection to the planned railway Divača - Koper is especially complex, where a number of very complex viaducts appear. The realization of the project is planned in stages through three stages:

- Stage 1: renovation and modernization of the existing railway for a speed of at least 80 km/h,
- Stage 2: construction of one track of a new two-track railway for a speed of 160 km/h,
- Stage 3: construction of the second track of the new two-track railway with the completion of all necessary works.

In the study [1], a multi-criteria evaluation of variant solutions was performed by the method of Promethee I and II and GAIA (Geometrical Analysis for Interactive Aid) according to the group of input parameters: economic, traffic, technical-technological, urban-planning and ecological-social, in principle with 4 sub-criteria in each group, on the basis of which the input matrix was created. Additional evaluation was performed using the AHP (Analytic Hierarchy Process) method.

Based on the conducted evaluation and CBA analysis, it can be concluded that the construction of a high-efficiency railway: Rijeka - Trieste is fully socio-economically justified in the case of stage 1 (variant V1) - modernization of the existing railway and stage 2 - construction of a new single-track railway. Realization of stage 3 - upgrading of the second track does not represent a socio-economically justified investment. Comparing all variants, variant V1 from the perspective of defined criteria produces the best socio-economic results.
It should be emphasized that all analyzed variants show a high level of socio-economic justification and as such are suitable for implementation. Variants V2 and V3 are provided through the area of Istria and the realization of one of these two variants contributes to the socio-economic development of the Istrian peninsula due to better access to modern railway infrastructure, especially considering the fact that the railway in the corridor will be built on the section Jurdani - Lupoglav (**Črni** kal) which does not exist today, or be modernized on the stretch Lupoglav - Divača.
All analyzes and forecasts, as well as technical solutions and variants of the high-efficiency railway presented in the research [1], represent an assessment of the framework possibilities of connecting the North Adriatic ports by railway from the current perspective of the possibility of its realization. The economic justification confirmed by the conducted analysis is expected to be higher in case the project implementation deadlines are moved to future periods; provided that no decline in railway demand is expected in the future compared to the forecast and the view that there will be no significant decline in GDP and other macroeconomic indicators in the project area.
The project "Multimodal Platform Adriatic-Danube-Black Sea", an integral part of which was the research of possible connections of northern Adriatic ports, included significant stakeholders from the Republic of Croatia. The holder of part of the activity was the County of Primorje-Gorski Kotar. With the cooperation of the Port of Rijeka Authority, the Intermodal Transport Cluster and with the support of HŽ Infrastruktura, further long-term and medium-term development potentials of this area are being integrally considered. This approach of joint development considerations of all entities in the transport chain certainly ensures the assumption of dynamic and sustainable development and thus the only possible basis for ensuring the emphasis on comparative advantages but also a credible positioning within the entire transport corridor.

References

[1] Studies of framework possibilities for connecting the system of northern Adriatic ports by high-efficiency railway, Institut IGH, dd Zagreb, December 2014.

[2] ADB Multiplatform: Study of the framework possibilities of connecting the system of northern Adriatic ports by high-efficiency railway, SEE Initiative 2012.

[3] ADB Multiplatform: ADB and Green Transport, Institute of Traffic and Transport Ljubljana 2013.

[4] New EU Transport Infrastructure Policy (Mediterranean Corridor), European Commission Brussels, 11 September 2014.

[5] Spatial and traffic integral study of Primorje-Gorski Kotar County and the city of Rijeka, IGH Institute Zagreb, March 2011

KEY FEATURES OF THE PROJECT OF RAILWAY RECONSTRUCTION AND MODERNIZATION: SECTION ŠKRLJEVO-RIJEKA-ŠAPJANE

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Abstract

The Škrljevo-Rijeka railway section is part of the main international railway corridor M202 Zagreb GK-Rijeka, a part of the Mediterranean TEN-T corridor and the RH2 corridor. M203 Rijeka-Šapjane-State border line is also an integral part of the RH2 corridor. M202 is part of the EU Core Transport Network, while M203 is part of the Comprehensive Transport Network. They were both built and put into operation in 1873 and are in need for reconstruction and modernization. This paper presents the basic project features. The modernization includes the construction of the second track and the reconstruction of the existing track on the section Škrljevo-Rijeka-Opatija / Matulji in the length of about 23 km, and the reconstruction of the existing track from Opatija / Matulj to Jurdani in the length of 6.5 km. The existing stations and stops are being reconstructed to meet the requirements of the new organization of traffic and the upgrading of the second track, and new stops are planned for the establishment of urban and suburban traffic. Reconstruction and modernization of all other infrastructure subsystems, as well as the construction of a new Operations and Management Center for traffic management in the wider area of the country, are on their way. With presented solution, the railway line will meet the ports needs in Rijeka, Sušak and Bakar basins, and at the same time the needs of the city of Rijeka in terms of organization of passenger transportation in urban and suburban area.

Keywords: documentation preparation, design requirements, project solutions

1 Introduction

Global concept of TEN-T core network, according to data and documentation published by the European Commission [1 – 7], and in Croatian strategies and regulations [8, 9] is based on the fact that railway transport is of key importance for the efficiency of the European economy. Without good transport links, the European economy will not be able to grow and develop. As a means of boosting growth and competitiveness, a strong European transport network is being set up under the new EU infrastructure policy, covering 27 Member States. The starting point of the new European infrastructure policy assumes that freight traffic will increase by 80 % by 2050, and passenger traffic by more than 50 %. One of the basic goals and policies of the European Union is to encourage the use of intermodal transport, i.e., alternative solutions that put energy-efficient modalities of transport in the foreground, and at the same time are acceptable from the transport-technological and economic aspect. Therefore, the new modalities are also acceptable for environmental protection. The basic principle of this transport system is based on combining at least two types of transport in the transport chain where most of the road freight is transported by rail or inland waterways, while the representation of road transport is significantly reduced.

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The project of railway reconstruction and modernization on section Škrljevo-Rijeka-Šapjane, presented in this paper, is part of a broader project of establishment of a high-efficiency double-track railway for mixed traffic on the Croatian part of the Mediterranean Corridor, which connects the Iberian Peninsula with the Hungarian-Ukrainian border via the ports of Rijeka, Zagreb, and Budapest. The transport, technological and economic significance of this railway section stems from the fact that the Rijeka transport hub is connected to the interior of Croatia and part of the European area through it. Therefore, special emphasis is placed on the connection of this railway section to the TEN-T basic network of the Mediterranean Corridor, i.e., to the railway transport corridor RH2 on the territory of the Republic of Croatia. Also, the position of the Rijeka junction and connecting railways in relation to the spatial and urban plan of the city of Rijeka and its surroundings is extremely important for the development of urban and suburban passenger traffic. In addition, through the railway section Škrljevo-Rijeka-Jurdani, it is possible to connect all ports and other freight terminals to railway traffic.
The entire project is eligible for EU co-financing and the documentation creation was co-financed by the European Union Connecting Europe Facility.

2 Project requirements

Railway section Škrljevo-Rijeka-Jurdani consists of two sections: section Škrljevo-Rijeka which is an integral part of the main corridor line of importance for international traffic on corridor RH2, M202 Zagreb GK-Karlovac-Rijeka, and section Rijeka-Šapjane which is an integral part of the main corridor line of importance for international traffic on corridor RH2, M203 Rijeka-Šapjane-State Border (Figure 1). Along the entire length (27,494 km), the existing railway section is single-track, electrified by a single-phase AC system 25kV, 50Hz and equipped for the maximum allowed mass of D4 trains (22.5 t/o and 8 t/m).
The project is a complex and multidisciplinary in nature, as it overlaps with nine other projects in the corridor: Bus terminal Zapadna Žabica and extension of Riva Street (City of Rijeka); reconstruction of the freight part of the Rijeka railway station (HŽI) and construction of the container terminal (Rijeka Port Authority); Port of Rijeka (Port of Rijeka); State road D-403-from the Škurinje junction to the port of Rijeka (Croatian roads); business complex Interspar Krnjevo and RIO; underpasses 3. Maj and Rukavac and Matulji junction.
The railway section is very demanding since it passes mostly through the central urban area of Rijeka and Matulji, very complex relief and geological-geotechnical areas, through the zones of sanitary protection of springs, and large development area next to the railway. In the area, there are several buildings of importance for cultural and historical heritage, which further limits the choice of possible technical solutions and affects the investment.

Figure 1 Škrljevo-Rijeka-Jurdani (Šapjane) railway section

The required performance parameters for desired future rail line categorization (P4, P5 and F2) are as follows: available profile GB (P4, F2) and GA (P5); axle load 22.5 t/a (P4, F2) and 20 t/a (P5); line speed: 120-200 km/h (P4), 100-120 km/h (F2) and 80-120 km/h (P5); useful platform length for interoperable stations: 200-400 m (P4) and 50-200 m (P5); and train length 600-1050 (F2). However, the section Škrljevo-Rijeka-Jurdani passes through the Rijeka city area and topographically specific area, the maximum train speed will be limited to 70/80 km/h, with limited maximum train length. Therefore, reconstruction of the existing and construction of the second track of the open railway is designed for speeds up to 80 km/h with the associated infrastructure. The line reconstruction and modernization will enable the performance parameters given in Table 1. In addition, the renovation and modernization of all other infrastructure subsystems (construction, electricity, traffic management and signalling safety, and reconstruction of buildings in the function of railway traffic), as well as road infrastructure is included to meet traffic safety conditions.

Table 1 Design performance parameters

Traffic	Profile	Axle load (t/a)	Train speed (km/h)	Platform length (m)
P4 –P5	GC	22.5	70-80	160-400 m
F2	GC	22.5	75-80	420

3 Project solutions main features

The modernization of the existing railway includes the construction of the second track along the existing one and the reconstruction of the existing track on the section Škrljevo-Rijeka-Opatija/Matulji in the length of about 23 km, and the reconstruction of the existing track from Opatija/Matulji to Jurdani in the length of about 6.5 km. Existing stations and stops are being reconstructed to meet the requirements of traffic organization and upgrading of the second track, and new stops are planned due to the establishment of urban and suburban traffic [10 – 12]. The reconstruction and modernization of all other infrastructure subsystems, as well as the construction of a new Operational Management Center (OUC) for traffic management in the wider area of the country are also under way.

3.1 Track route and geometry

Horizontal and vertical track geometry is designed in accordance with the standard HRN EN 13803, and applicable technical regulations and European directives. In places where the existing horizontal arches must be reconstructed, the radii are selected so that the reconstructed route deviates as little as possible from the existing route, and that an intermediate straight of the prescribed length can be placed between adjacent curves. This project does not envisage the reconstruction or renovation of the railway line on the subsection Jurdani-Šapjane -DG and the existing route is retained. Only the reconstruction of the Šapjana station and the Permani stop is planned. During the design, the route of the existing railway track was largely retained with two deviations: in the part in front of the Škrljevo station and at the exit from the Škrljevo station by about 60 and 35 m.
From Škrljevo station to Rijeka station, the second track is being added on the right side. At the exit from Rijeka station, the axis of the line is shifted to the left by about 2.85 - 4.75 m, and the second track here is upgraded on the left to km 1+200. From there to the Opatija-Matulji station the route of the existing line is retained as the left track, and the new, second track is upgraded on the right side of the line at an axial distance of 4 m, thus avoiding upgrades of high embankments, and cuttings are widened. When designing the single-track line from Opatija/Matulji station to Jurdani station, the existing line route was fully respected

and retained, except at the exit from Opatija/Matulji station where the reconstructed station tracks should be connected to the existing line. Further along the open single-track line, the deviation of the designed axis from the existing one is minimal.
The minimum radius of the curve on the open track is 270 m and it is located where the line turns from Drage to the City of Rijeka at an angle of 100°.The vertical track geometry of the new second track and the reconstructed existing railway was mainly maintained on the entire railway section Škrljevo - Rijeka. The projected (as well as the existing) longitudinal slopes from Škrljevo station to Jurdani station are mostly around 25 mm/m. From Jurdani station to Šapjane station, the existing longitudinal slopes are milder, the largest of which is 16.8 mm/m. The distance between the track axes of the open double-track railway is 4 m. In the "A-V" connections, the track distance is 4.75 m. At the intersection with the future D403 road, the tracks are spaced at 5 m and at the underpass 3. Maj at 7.6 m.

3.2 Track superstructure and substructure

The superstructure of open tracks and main tracks in stations consists of new 60E1 type rails on prestressed reinforced concrete one-piece sleepers with elastic track fastening accessories and direct fastening without base plate. Concrete sleepers are positioned at an axial distance of 60 cm in ballast bed. The ballast is at least 30 cm thick below the sleeper at the side of the lower rail, 50 cm wide from the front of the sleeper, with a stone throw. To reduce vibrations and noise from the tracks, the installation of sleepers with elastic lining on the lower side of the sleeper (USP - under sleeper pads) is planned on the entire Škrljevo-Jurdani section. The track and switches will be welded into a continuous rail strip.
The plain of the double-track protective layer on the upgraded and renewed track is 11-11.20-11.50 m wide. The plane has a double-sided transverse slope of 5 %. The protective buffer layer is 30 cm thick. The plain of the single-track line on the renewed / modernized track is 6.70 - 6.85 m wide. The transverse drop is 5 % on one side in the width of 5.85-6.00 m, while the other part of the planum is 0.85 m wide (sidewalk) provided in the counter-slope.
Railway sections on which it is necessary to add a new track in cuts (there are several cuts over 20 m high by the surrounding construction), complex technical solutions and various structures must be applied to ensure soil stability and existing buildings. On railway sections where the existing track is laid on the embankment (there are existing embankments on the railway over 20 m high built at the time of railway construction, 150 years ago), the construction of the second track will be achieved by upgrading or widening the existing embankment. The widening of the embankment will be carried out by constructing stairs on the existing embankment, and by installing embankment material with the application of modern materials and methods of slope stabilization.
The project envisages the reconstruction of the existing and the construction of a new drainage system. The drainage of the railway area outside the water protection zones and hydrogeologically sensitive areas is provided by a system of parallel ditches (channels) with direct discharge into the recipient without special wastewater treatment. A closed drainage system is planned at the places where the railway passes through the zones of the sanitary protection of springs. Water from the internal drainage system will be treated in oil and grease separators with integrated filters and, if necessary, in infiltration ditches before being discharged into the recipient. The design solutions ensure that wastewater from bridges is not discharged directly into the watercourse but is collected and redirected to a system of parallel ditches of external drainage. Wastewater from bridges in closed protection zone before discharge into the recipient will be treated in oil and grease separators. Other facilities in the area (roads and parking lots) will have solved local drainage systems in accordance with the special conditions.

3.3 Stations and stops

The stations are being reconstructed to adapt the track plan to the second track, and to achieve greater useful track lengths, to improve the functionality of the station for the purpose of meeting or overtaking trains. The existing reception buildings are being reconstructed due to the need to arrange the space for the accommodation of ESSU, ITK and TK devices within the station buildings.
Škrljevo station will remain an intermediate station on the line M202 Zagreb GK–Rijeka, a separate station for the line M602 Škrljevo–Bakar and becomes the terminal station of urban-suburban traffic of the city of Rijeka. Due to the new track plan, the reconstruction of the existing garage group of tracks is necessary. The total number of tracks in the group will decrease compared to the existing condition, but the useful length of individual tracks will increase. The garage group itself is very important for local work at the station and servicing the Kukuljanovo industrial zone.
The function of the Sušak/Pećine station will not change. This will be an intermediate station on the line M202 Zagreb GK-Rijeka, and a separate station for the line M603 Sušak/Pećine-Rijeka Brajdica. It will be used for urban-suburban and local passenger transport.
Rijeka station will remain the distribution and shunting station of the M202 and M203 line. The station will be open for the passengers in domestic and international traffic as well as all types of wagon shipments. This project included the reconstruction of the track in the passenger part of the station. It is planned accommodate long-distance trains, i.e., passenger trains longer than 160 m, at the station.
With the addition of the second track, Opatija/Matulji station will become an intermediate station for passengers in local and international transport, as well as a station for switching from double to single-track. In urban-suburban traffic, the station becomes the terminal. The reconstruction includes all existing station tracks. After the reconstruction, it will have 4 main tracks and two garages. One for garaging the city suburban train set and the other for garaging motor rail vehicles.
Jurdani station will be located on a single-track section of the modernized line M203. It will be open only for passengers and have 4 tracks.
Šapjane station will remain the border station. The project envisages a complete reconstruction of the station, considering space constraints related to the width of the station plateau on the right, as well as the slope of the railway line at the entrance, which limits the possibility of extending the station towards Jurdani. After the reconstruction, four receiving and dispatching tracks are planned at the station.
Reconstruction of existing or construction of new stops in the function of urban and suburban passenger traffic is planned on the projected section of the railway. New stops are planned: Draga, Vežica, Zagrad, Kantrida, Zamet, Martinkovac, and reconstruction of the existing ones: Krnjevo, Rukavac, Jušići and Permani. Side platforms 0.55 m high above the TOR, 160 m long, are planned at the stops.
For the connection of platforms at stations and stops, the construction of new underpasses with a clarence width of 4.8 m is planned. In the stations where it is not planned to stop long-distance trains, i.e., passenger trains longer than 160 m, two side platforms 160 m long, 3.50 m wide and 0.55 m high from the TOR at 1.65 m from the line axis are planned. Canopies 24-80 m long are planned on the platforms.

3.4 Structures and tunnels

There is a significant number of structures, and some of them are the subject of interest of the conservation department. Reconstruction of all underpasses, overpasses, viaducts, and bridges is planned because, regardless of the condition of the structure, the geometry does not meet the requirements for the passage of the second track.
On the section Rijeka - Jurdani, which is subject to reconstruction and renovation, there are 5 overpasses, 8 underpasses and 7 overpasses. The largest underpass opening is 32 m long Matulji underpass over the state road to the Učka tunnel.
On the section Škrljevo - Rijeka there are a total of 2 tunnels and one gallery. The Baudine tunnel is completely abandoned by the deviation, and a new tunnel is envisaged, while the Kalvarija tunnel and Zagrad gallery are retained as they are reconstructed according to the profile. The existing railway tunnel Kalvarija, 452 m long, was built in 1873 for a two-track railway. Zagrad Gallery has a total length of 206.42 m. The project envisages the construction of two more galleries: Sv. Ana, approximately 350 m long, and the Ciottina Gallery (a continuation of the Zagrad Gallery), approximately 90 m long.
On the Rijeka-Šapjana section, there is one railway tunnel, Rukavac, 312.40 m long. The tunnel is located between Jurdani and Opatija/Matulji station. The tunnel, elliptical in shape, was built in 1873. The tunnel will be reprofiled and equipped for the required cross-section of the single-track line.

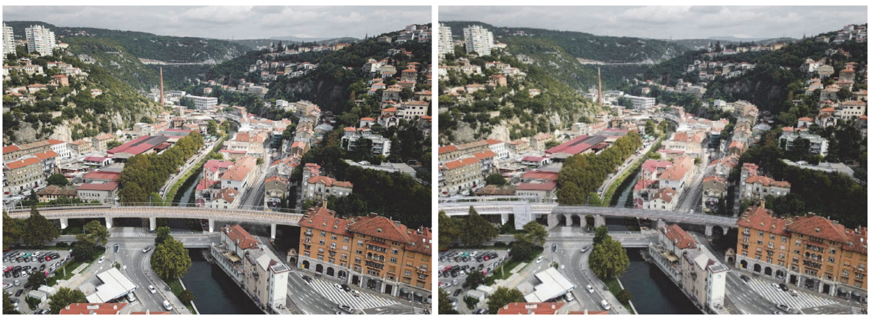

Figure 2 Rječina-Školjić viaduct solution option (new facility – left, reconstruction and upgrade - right)

4 Conclusions

Although the section of the railway Škrljevo-Rijeka-Jurdani is relatively modern, it is not able to meet all the required functions and goals. The very fact that it is a single-track line limits its capacity and transport capacity, making it a bottleneck in the transport system of European and national corridors in the area. The European Union has recognized the need to eliminate the existing so-called bottleneck around the Rijeka hub, which would enable the further development of the existing capacities of the Port of Rijeka and the creation of efficient urban and suburban railway transport. That is why in 2015, it allocated 8.5 million euros from the Instrument for Connecting Europe for the project for the preparation of project documentation, which is 85 percent of the eligible project costs of 10 million euros. The remaining part of the maximum value of the project in the amount of 1.5 million euros is co-financed by the Republic of Croatia.
The main goal of the study and technical documentation for the construction and reconstruction of this railway section was to define, in a modern and professional way, all solutions related to the preparation of construction, reconstruction, and modernization of the railway section and meet the necessary conditions for land acquisition and construction permits.

The goal of created main designs was to achieve conditions for starting the construction of sections in the manner required by international financial institutions (financial and technical profitability, proven through a feasibility study and feasibility study, conceptual and main design) considering all provisions and measures of Environmental impact studies. Therefore, the goal of the entire project was to ensure the upgrade this railway needs to accommodate a larger amount of passenger and freight traffic. As this paper shows, needed upgrade primarily refers to the construction of the second track with the reconstruction of existing one, and the construction and modernization of stations and stops for passenger traffic, and of all ancillary facilities on the line. Presented railway infrastructure upgrade will increase the quantity and quality of all modes of transport in this area.

References

[1] White Paper 2011-2020 - Plan for the Single European Transport Area

[2] Trans-European Transport Network, TNT-T Core Network Corridors, European Commission 2014.,

[3] New Policy EU Transport Infrastructure (Mediterranean Corridor), EC Brussels, 2014.,

[4] Europe 2020 - Strategic Framework of the European Union,

[5] Rail Freight Corridor 6 - Implementation Plan - EU 09.04.2013- 17.04.2013.,

[6] Rotterdam Ministerial Declaration on the Implementation of the Trans-European Transport Networks (TEN-T) adopted on 20 June 2016.,

[7] Ministerial Declaration of Rotterdam on Railway Freight Corridors for the Promotion of International Rail Freight Transport of 21 June 2016.,

[8] Transport Development Strategy of the Republic of Croatia (2014-2030), October 2014.

[9] Act on Safety and Interoperability of the Railway System, Official Journal 63/20

[10] Feasibility study of the project Construction of the second track, reconstruction and modernization of the railway section Škrljevo-Rijeka-Jurdani, Institut IGH, dd Zagreb, November 2017.

[11] Environmental impact study Construction of the second track, reconstruction and modernization of the railway section Škrljevo-Rijeka-Jurdani, IGH Institute, dd Zagreb, Granova, doo Zagreb, November 2017.

[12] Preliminary design for the construction of the second track, reconstruction and modernization of the railway section Škrljevo-Rijeka-Jurdani-Šapjane, Institut IGH, dd Zagreb, Granova doo Zagreb, October 2019.

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RECONSTRUCTION AND MODERNIZATION OF RAILWAY LINE STALAĆ – KRALJEVO – RUDNICA – OPTION ANALYSIS

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Abstract

Serbia is upgrading its Core Railway Network in line with international agreements with a view to reaching the EU standards of interoperability. It aims to revitalise and develop the railway network giving priority to Pan-European Corridor X, which is the backbone of the system, and to SEETO routes 10 and 11 (as part of Indicative Extension of TEN-T Core rail network) on which the Stalać-Kraljevo-Rudnica line is located. The overall objective of Reconstruction and modernization of the railway line Stalać-Kraljevo-Rudnica is to safeguard the functionality by aligning it with the relevant standards as specified in the TEN-T regulations and TSI requirements. The purpose of this paper is to define the options for each of the proposed parameters (Single-track or Double-track, Axle load, Design speed, Technical solutions for structures (tunnels, bridges, underpasses and overpasses), Electrification, Signalling, Telecommunications and management, Stations, Environmental protection and Social Environment) and select the desired option.

Keywords: railway line, reconstruction, modernization, option analysis, parameters

1 Introduction

Modernization, reconstruction and construction of the railway network in Serbia aims to create a modern and functional trasport network that will in part be integrated in the united Trans European network. In line with international agreements, and with the goal of reaching the standards of interoperability with the EU, Serbia is upgrading its railways, giving priority to the railways on the Pan-European Corridor X, and also routes 4, 7, 10, 11, 9A and 13 (SEETO network) [2]. The general goal of the upgrades to the national railway network is an improvement in the functionality of the high priority railway sections, with compliance with TEN-T and TSI requrements [1, 6].

As one of the priorities of the development of railway infrastructure in Serbia, the project of reconstruction and modernisation has already begun on the Stalać-Kraljevo railway on route 11 and the Kraljevo – Rudnica railway on route 10, based on European initiatives and strategies. The strategic goal of reconstructing and modernising the railroad section Stalać-Kraljevo-Rudnica is creating larger transport capacities for transport towards the port of Bar, Macedonia and Greece.

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Figure 1 Expansion of the TEN-T railways into the western Balkan with the placement of the Stalać-Kraljevo-Rudnica railway.

The improvements to the railway infrastructure in the already mentioned sections will contribute to the establishment of high quality regional passenger traffic, by which a good connection will be made between south Serbia and Belgrade, while when it comes to cargo transport, the needs of local economy will be meet, primarily the automobile industry and energetics. The task of the Previous feasibility study and the General reconstruction and modernisation project of the railway section Stalać-Kraljevo-Rudnica (WBIF WB14-SRB-TRA-01) was to define alternative solutions and ways in which the reconstruction and modernisation of railroad sections could be carried out in line with the requirements of modern transport systems, then comparing them on a functional, technical, economical, financial, spatial, ecological and social level, and making the decision on the optimal variant solution.

1.1 Analysis of the state of the existing railway sections

By using the data collected from the infrastructure management, and the data collected during railway inspection, a thorough analysis has been performed of the topographical, geological, spatial and ecological conditions, and the general state of railway infrastructure.
Both of the railway sections, Stalać - Kraljevo and Kraljevo – Rudnica are single-track and not electrified. The lenght of the section from Stalać to Kraljevo is ~72 km, while the length of the section from Kraljevo to Rudnica is ~77 km.
According to the allowed mass of vehicles, the Stalać-Kraljevo section belongs to the B2 (180 kN, 64 kN/m) category, while the Kraljevo-Rudnica section belongs to the C3 (200 kN, 72 kN/m) category.
On the Stalać – Kraljevo section, trains achieve the top speed of 40 km/h, while trains on the Kraljevo – Rudnica section achieve the top speed of 60 km/h. There are 20 stations on the railway sections (9 on the section Stalać-Kraljevo and 11 on the section Kraljevo-Rudnica).In order to rationalize its workforce and business, Joint Stock Company Serbian Railways closed some of stations and converted them into hault stations. At the moment, there are 4 stations on the section Stalac-Kraljevo and 8 stations on the section Kraljevo-Rudnica. The buildings, equipment and devices in the stations are in poor condition.The equipment is outdated and insufficient. Passenger service areas (halls, ticketing, waiting rooms and platforms) are not equipped with modern passenger information devices and ticketing service system. Administrative offices and facilities meant for railway service users are not equipped with modern equipment. Many of station buildings require different levels of intervention.

Figure 2 Official posts on the Stalać-Kraljevo-Rudnica railway

Railway signalisation and security systems mostly consist of electromechanical devices with light signals and above-ground telecommunication lines. The equipment is obsolete and insufficient. The signalisation equipment on the railway is either does not exist or does not work. The telecommunication systems are based on analogue telephony. The telecommunication platform does not support IT. The stations are not equipped with modern technology systems.
The Stalać – Kraljevo section has two (2) pairs of passenger trains, while the Kraljevo – Rudnica section has three (3) pairs of passenger trains (that go to Kosovska Mitrovica).

2 Methodology applied for the analysis and selecting options

The suggested methodology can be summed up in two phases:
- Defining of the variable solutions, based on an independent evaluation of the adopted parameters.
- Cost – benefit analysis.

The analysis of the current state of the railways, in terms of traffic indicators, infrastructure, functioning/operation, etc. provided the basis for:
- Identification of the functionality of the railway, both the existing and the planned one.
- Identification of the need for railway modernisation and reconstruction.

Accepting the recommendations of the Joint Assistence to Support Projects in European Regions (JASPERS) for defining of the variable solution options, a simplified methodology was applied, evaluating each of the parameters individually, while rejecting those parameters that have no influence on the decision on a variable solution. The process can be seen as a series of filters where:
- Every filter removes some of the variants.
- As the number of variants that need testing decreases, the level of work on each individual filter increases.
- Variants that do not show a clear advantage during testing or quick evaluation, can be discarded early.
- Hipothetically, in the case where different variants cannot be simply eliminated after a quick evaluation, each should be thoroughly evaluated.
- Sustainable variants, with more potential, that are included in the package for detailed evaluation are the ones that pass through all the filters.

The evaluation process focuses on determining whether a solution and/or its variation is strategically in line with the goals of the transport system, including the goals of the end user, strategies, plans and procedures.

2.1 Definition of variable solutions, based on an independent assessment of the adopted parameters

The key parameters used in the process of defining the alternative solutions were:
- number of tracks (single-track or double-track railway),
- axle load,
- design speed,
- solutions for infrastructural objects (tunnels and bridges),
- electrification (whether it is necessary to project electrification or not),
- safety signalling and telecommunication facilities and devices,
- stations,
- environmental protection and social environment.

The need for a single-track or double-track railway is based on an assessment of the current need for traffic, future demand for transport services, as well as throughput. In order to fully meet the transport requirement in the period of 30 years from the completion of modernization and reconstruction of railway sections (period from 2027 to 2057), it is necessary to provide:
- seventeen (17) trains in both directions according to pessimistic forecasts or twenty-nine (29) trains in both directions according to optimistic forecasts, on the section from Stalać to Kraljevo,
- twenty-seven (27) trains in both directions according to pessimistic forecasts or forty-two (42) trains in both directions according to optimistic forecasts, on the section from Kraljevo to Rudnica.

Based on the forecast and calculation of capacity on the section from Stalać to Kraljevo, it is shown that the current situation in terms of the number of official positions in operation (with increasing speed) can meet all transport requirements in the planned period.
Since the routes 10 and 11 are located on the comprehensive TEN-T network, the axle load on the railway sections is defined in accordance with Regulation 1315/2016, Article 39, which prescribes/provides for-stipulates the axle load of 22.5 t as a minimum [1].
Depending on the geometric elements of the route, the permissible speeds at the inter-station/stop distances have been defined. Depending on the designed geometric elements of the route which have a minimal impact on expropriation, while at the same time meeting the requirements of TSI and TEN-T [4] regulations, variant solutions have been defined in relation to the allowed speeds. An overview of variant solutions defined as a function of designed speed is shown in Table 1.

Table 1 Overview of variant solutions defined in terms of design speed

Variant solution	Design speed
S 1.1	Stalać-Kraljevo: Speed V = min 60 km/h (min R300) – Retaining the existing speed limit with a correction of horizontal curve radius to min 300m.
S 1.2	Stalać-Kraljevo: Speed V= min 80 km/h – Increasing the speed to min 80km/h with the correction of geometric railway elements where necessary.
S 1.3	Stalać-Kraljevo: Speed V = min100 km/h – Increasing the speed to min100 km/h with the correction of geometric elements of the railway where necessary.
S 1.4	Stalać-Kraljevo: Speed V = min120 km/h – Increasing the speed to min120 km/h with the correction of geometric elements of the railway where necessary.
S 2.1	Kraljevo-Rudnica: Speed min 60 km/h (min R300) - Retaining the existing speed limit with a correction of horizontal curve radius to min 300m.
S 2.2	Kraljevo-Rudnica: Speed V = min 80 km/h - Increasing the speed to min 80 km/h with the correction of geometric elements of the railway where necessary.
S 2.3	Kraljevo-Rudnica: S 2.3 Speed V = min 100 km/h – Increasing the speed to min100 km/h with the correction of geometric elements of the railway where necessary.

According to the required national and international regulations, the reconstruction of the tunnel should include an increase in the area of the clear profile so that the railway should allow unhindered passage of the railway vehicles that, together with the cargo they carry, have a GC loading gauge. Based on the verification of the bearing capacity of the existing bridges, a decision is made on the scope of intervention for each individual facility. Where the load-bearing requirement is not satisfied, it is necessary to reinforce the structures. If the control shows that reinforcement is not possible and that the bearing capacity is significantly endangered, it is necessary to design a new facility. Electrification of the railway sections is envisaged on the basis of strategic decisions and development plans of the Serbian Railway Infrastructure, as well as on the basis of the Spatial Plan of the Republic of Serbia [4].
The section of the railway from Požega to Kraljevo, including the station in Kraljevo, is electrified, as is the railway on Corridor X, which passes through Stalać.
Considering the fact that along the line/railroad between Požega and Stalać only the section from Stalać to Kraljevo is not electrified, the conclusion is that it is necessary to envisage electrification of this section as well in order to avoid expenses/costs and reduce/shorten the travel time of trains due to the replacement of the electric locomotives/engines with diesel locomotives.
The reason for designing the electrification of the Kraljevo - Rudnica section is the planned expansion of Route 10, from Jarinje to the border crossing with Northern Macedonia, in order to ensure interoperability and thereby allow a smooth flow of traffic.
The improvement of safety-signalling and telecommunication plants and devices is mainly based on the application of modern technological solutions that satisfy both national and ER regulations.
The technical solution for both sections should encompass a complete upgrade, based on the principles of ETCS1 and the local railway network based on IT with optic infrastructure.
The number of tracks in the stations on the sections of Stalać - Kraljevo and Kraljevo - Rudnica is sufficient for the planned scope of operation/work. The tracks in the stations are to be reconstructed in accordance with the TSI requirements as regards/in terms of useful lengths.
From the point of view of environmental protection, variant solutions that do not depart significantly from the existing conditions are considered to be more favourable. Since these are inhabited and protected areas, lager departures from the existing railway will cause extensive expropriation and demolition of facilities, and also have a greater impact on archaeological sites and ecologically significant areas.

2.2 Cost – Benefit Analysis

The Cost-Benefit Analysis (Table 2) considered options defined as a combination of alternative solutions. Estimated costs of reconstruction for Option 1 amount to a total of EUR 473.431.173, Option 2 - 473.490.973, and Option 3 - 490.310.179. The prices do not include VAT and unforeseen works. Investment costs are based on the project and unit prices in the region.

Table 2 Options obtained by combining variant solutions for the need of producing Cost-Benefit analysis

Section		Stalać-Kraljevo		Kraljevo-Rudnica
Solution variant/version	S1.2/S1.3	S1.2/S1.4	S1.3	S2.2
Option 1				
Option 2				
Option 3				

The reconstruction and modernization of the Stalać-Kraljevo-Rudnica railway is expected to be completed within a period of seven years, and the beginning is due in the first quarter of 2027.The project was analysed for the time period of 30 years, including the construction period, i.e. from 2027 to 2056. Future transport forecast requirements are based on:

- Identification of previous traffic flows on all railway and road sections included in the Referent Traffic Network. Existing traffic flows are identified for the base year of 2016.
- Development of growth factors that will be applied to existing traffic flows.
- Estimations of diverted traffic flows from road to railway after the modernization of the railway line.
- New traffic requirement – maximum capacity utilization of mines located along the related railway corridor.

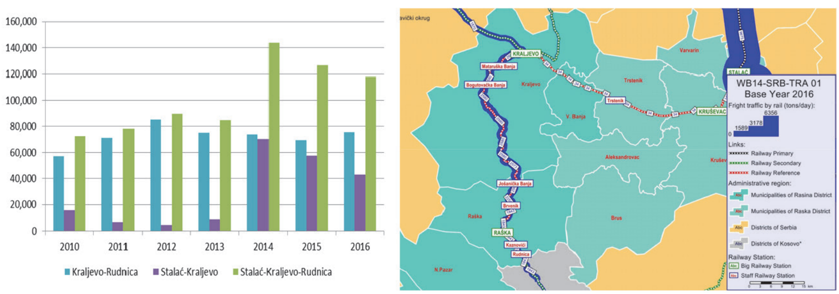

Figure 3 Annual number of passengers on the railway line in question (2010-2016) and freight on the line in 2016 (tons / day)

The CBA was conducted based on the instructions provided in the Guide to Cost-Benefit Analysis of Investment Projects. [8]
The financial return of investment costs is estimated by the indicators: financial net present value of investment FNPV (C) and financial internal rate of return FIRR (C). Table 3 shows financial analysis indicators for a discount rate of 4 %.

Table 3 Financial analysis indicators for a discount rate of 4 %

	Option 1 and 2	Option 3
FNPV (EUR)	-262.567.755 < 0	-243.954.173 < 0
FIRR (%)	-7.32 < 4	-5.89 < 4

Financial analysis indicators for a discount rate of 8 % are also negative. The financial analysis indicates that the investment defined in this way cannot be self-financed, and the revenues themselves would not be sufficient for mandatory loan repayment, so it would be necessary to provide a grant for the implementation of this investment or subvention for loan repayment by the State.
The economic analysis was done based on constant prices and adopted values of the discount rate of 5 % [8].The following elements were considered within the economic analysis: investment costs, maintenance costs, travel time savings, delay costs, accident savings, air pollution savings, noise savings, climate change / global warming savings, marginal infrastructure costs, vehicle operating costs (VOC), train operating costs (TOC) and secondary materials. Economic analysis indicators are shown in Table 4.

Table 4 Results of economic assessment

	Discount rate 5%	
	Option 1 and 2	**Option 3**
ENPV (EUR)	97.996.342 > 0	103.478.510 > 0
EIRR (%)	9.65 > 0	9.46 > 0
B/C	1.52 > 1	1.58 > 1

Economic analysis shows that the project is justified for society, ie. as a social investment, and is recommended for the further implementation.

3 Conclusion

For the railway Stalać-Kraljevo-Rudnica, possible alternative solutions for the railway have been designed, along with options for the reconstruction and modernization in keeping with the requirements for modern transportation systems and their mutual comparison was performed in terms of functional, technical, economic and financial aspects.
To define variant solutions, a simplified methodology was applied, which involved evaluating each of the parameters individually, rejecting the parameters that had no influence on the decision on the variant solution.
The combination of variant solutions was used to define the options that were considered within the Cost-Benefit analysis.
The presented results for options 1 and 2 and for option 3 are very close in terms of economic performance indicators and therefore a more detailed investment cost analysis and cost-benefit analysis should be performed in the preliminary design phase.
All the indicators that are analyzed speak about the fact that this project is sustainable and acceptable from the socio-economic point of view, but also about the fact that there is a need and justification for co-financing through EU funds or other sources. Specific details about the source of financing/funding will need to be determined at the level of preparation/ producing a feasibility study with the preliminary design.

References

[1] Regulation (EU) No 1315/2013 of the European Parliament and of the Council of 11 December 2013 on Union guidelines for the development of the trans-European transport network and repealing Decision No 661/2010/EU

[2] http://www.seetoint.org/links/seeto-participants-institutions/railways/ 12.03.2020.

[3] Generalni projekat rekonstrukcije i modernizacije železničke pruge Stalać-Kraljevo-Rudnica, CESTRA d.o.o. Beograd, 2019.

[4] Zakon o prostornom planu Republike Srbije od 2010. do 2020.godine ("Sl.glasnik RS", br. 88/2010)

[5] Directive 2008/57/EC of the European Parliament and of the council of 17 June 2008 on the interoperability of the rail system within the Community (Recast) (Text with EEA relevance)

[6] Commission Regulation (EU) No 1299/2014 of 18 November 2014 on the technical specifications for interoperability relating to the 'infrastructure' subsystem of the rail system in the European Union

[7] Prethodna studija opravdanosti za projekat rekonstrukcije i modernizacije železničke pruge Stalać-Kraljevo-Rudnica, CESTRA d.o.o. Beograd, 2019.

[8] Guide to Cost-Benefit Analysis of Investment Projects, Economic appraisal tool for Cohesion Policy 2014-2020, December 2014

REVIEW OF THE REMETINEC ROUNDABOUT RECONSTRUCTION PROJECT IN ZAGREB

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Abstract

A very complex problem of the main roundabout projects development concerning the solutions given in the location permit and preliminary design, as well as new knowledge about communal infrastructure is given. Based on the new geodetic survey, subsequent requirements of the audit, investors, and obtained special conditions, it was necessary to adapt the technical solutions of the preliminary design. Particularly demanding work was defining the protection of the construction pit. The solution foreseen by the preliminary design and location permit for the works to be carried out under the traffic could not have survived, because of the technological reasons for the construction of the underpass and the very complex situation with the installations at that location. These facts required a complete suspension of traffic at the site and finding solutions to the temporary traffic regulation around the site. This was an unplanned and very demanding moment, given that the daily traffic at the intersection was around one hundred thousand vehicles a day. The solution to this problem was found in the construction of a temporary road, which conducts traffic through and around the construction site, thus enabling the smooth technological and technical organization of works while simultaneously conducting public and individual traffic to and from the city.

Keywords: project modification, construction works, traffic organization

1 Introduction

In the fifties of the last century, the formation of the town south of the Sava began. Urbanized settlements are being built and the old villages in their surrounding are growing rapidly and urbanizing. The Sava Bridge, built in 1938, soon became insufficient for traffic jams. To relieve traffic in the western part of Novi Zagreb - the western entrance to the city in 1981, the Jadranski bridge and a roundabout intersection connecting Jadranska Avenue and Dubrovnik Avenue, Remetinečka Road with Savska Street, the so-called “Remetinec Rotor”. The roundabout (“Rotor”) with a radius of 74m was built at the level of +1 concerning the surrounding terrain. The +1 level is conditioned by the planned passage of the tram line at ground level, which was built in 1984. The roundabout is designed for a traffic of 50,000 vehicles per day, but with the construction of the Lanište settlement and the sports hall, and the Arena shopping center during the 2000s, the volume of traffic doubled. Roundabout has become one of the busiest traffic intersections in the city of Zagreb, with a traffic load of more than 100,000 vehicles per day, on which one traffic accident occurred on average per day. In addition to the high traffic load, the position of the roundabout, elevated concerning the access roads, is an additional cause of a large number of traffic accidents. The first study of the roundabout reconstruction was made in 2007, and two years later a proposal was selected with an additional level of crossing denivelation, in such a way that the traffic in the east-west direction was lowered to the level of -1 [1].

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As part of the reconstruction project (1), the geometry of the inner roundabout radius is retained, but the traffic lights are added, (2) overpasses on the northeast, northwest, southwest and southeast sides are upgraded to widen the northern and southern access ramps, (3) traffic is being deleveled in the east-west direction by building an underpass at level -1, (4) on the site of the western overpass an embankment and a green plateau are being built between the western ramps above the underpass, (5) a new overpass "East" with larger span is being built, (6) the pedestrian underpass under Remetinečka cesta is being upgraded on the west side, while the pedestrian underpass on Dubrovnik Avenue is maintained in the existing dimensions, (7) the tram tracks in the scope of intervention are reconstructed and a tram turnaround is built within the roundabout, (8) noise protection walls are being built on the south-east side, next to the Savski Gaj settlement, and (9) existing installations are being relocated to prepare for the passage of future installations (protective pipes in the road body for the future hot water pipeline). Based on the requirements from the special conditions of construction and additional requirements of public bodies, and the requirements of authorized auditors during the preparation of main designs, concerning the solutions in the preliminary design, the solution was adjusted [2].

2 Features of the main and detailed design

With the elaboration of the Preliminary Design on a more detailed geodetic survey and further elaboration, the traffic areas on Dubrovnik Avenue were additionally corrected in terms of layout and height, so that the pedestrian underpass Savski Gaj could be kept in the existing dimensions. Bus stops on Dubrovnik Avenue have been further extended. The relocation of the existing water supply pipeline along the northwest ramp is planned, due to the requirements of VIO Zagreb. The locations of the noise protection walls have been corrected concerning the existing car entrances. Additionally, fire hydrants were designed at the entrance-exit of the underpass with hydrant supply pipelines with connections to the existing city water supply network. During the construction, new problems appeared that were not included in the preliminary or main project. Due to the above, an amendment to the location permit, and the amendment to the construction permit were obtained. The elements of the modified design solutions are described below.

2.1 Construction pit protection

In the process of project development, and given the circumstances at the site, variant solutions for the protection of the construction pit of the underpass with retaining walls were made, to select the optimal final solution for the protection of the construction pit and the underpass. A wide excavation for the protection of the construction pit with a diaphragm was carried out from elevation 117 m above sea level on the access ramps in the east and west to elevation 113 m above sea level in the central part of the roundabout. The excavation depth within the diaphragm area was up to a depth of 8.5 m, and in the area of the pumping station up to 11.33 m. Considering the need to protect underpass structure during exploitation from buoyancy forces, depth of excavation, and groundwater at the height of 112 m above sea level, the protection of the construction pit was designed with two anchored reinforced concrete diaphragms (south and north) and two clay concrete diaphragms (east and west). The construction of the diaphragm and the underpass are separate, for simpler construction and better waterproofing. The construction of the diaphragm was also the formwork for the outer walls of the underpass. The diaphragm is designed to be waterproof. To accept any leachate, the diaphragm is lined with insulating-drainage tape, and circumferential drainage ribs are made by which water is channeled into the central collecting drainage pipe which is fed into the pumping station, where it is evacuated utilizing automatic pumps. Maintaining the water level below the foundation level protects the construction.

Figure 1 The diaphragm works for the protection of the construction pit (left) and peripheral drainage ribs (right)

2.2 Underpass design

Two underpasses have been designed, one for each road. In that part, the roads are separated due to the fit into the wider dividing zone on Dubrovnik Avenue, where the tram line is located. The underpass consists of three parts: closed frame constructions of the underpass itself under the embankment of the roundabout or under the ramps (black in Fig. 2), open troughs in front and behind the underpass (magenta in Fig. 2) which end below with classic retaining walls (yellow in Fig. 2) ascending towards the level of the surrounding terrain. The closed frame reinforced concrete structure has a wall thickness of 1.0 m, a wall height of 6.8 meters, and the width of the underpass is 12.5 m.

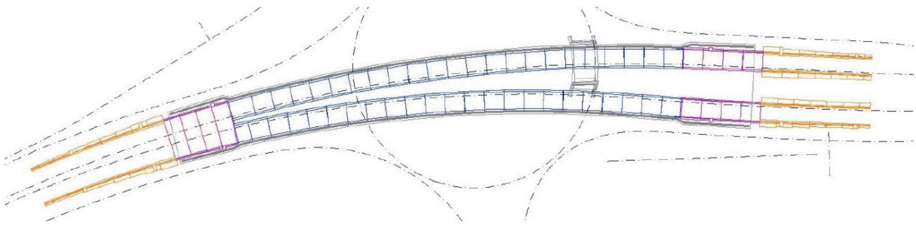

Figure 2 Underpass position with access ramps

2.3 Rainwater drainage pumping station

The pumping station is located between the north and south underpasses. It is designed as a reinforced concrete underground facility with a pumping pool and pumps (3 + 1 spare), the capacity of each pump Q = 140 l / s, and the height of the water lift H = 11.7 m. The bottom of the pumping basin is at an altitude of 102.52 m above sea level. The upper "floor" at an elevation of 107.72 m above sea level, is intended as a latching chamber for the accommodation of pressure pipelines (fittings and plumbing fittings), electrical cabinets for the accommodation of electrical installation of the pumping station, openings for assembly and disassembly of pumps, and openings for the descent into the pump swimming pool. Backup power supply energy is provided by a 250 kVA Diesel-electric generator (DEA). An external unit, soundproof - 68 dB / 7 m, with a large tank in the base of the unit -1,200 l, which provides sufficient autonomy in the event of a power outage. The main project envisages access to the pumping station from the underpass. To enable the descent to the upper floor of the pumping station without entering the underpass (requirement VIO dd at the time of construction), a "service entrance" was made on the cover plate of the pumping station to enter the pumping station from the central part of the rotor.

Figure 3 A phase of derived diaphragms and underpass (left) and derived diaphragms, underpass, and temporary road for traffic regulation (right)

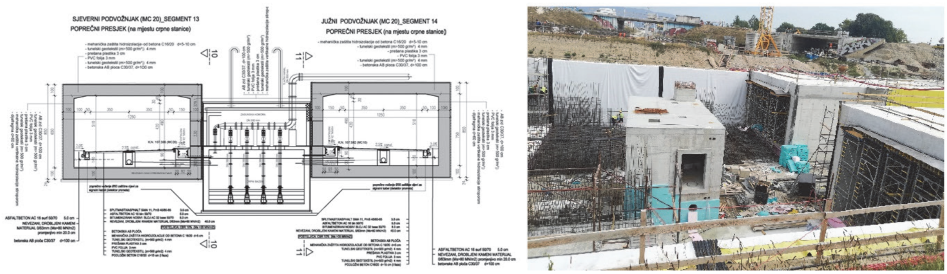

Figure 4 Position of the pumping station between the two underpasses

2.4 Overpass reconstruction

The preliminary design envisages the upgrade of the existing overpasses on the outside of the roundabout. The span assemblies of the existing constructions are made of SAN supports 70 cm high. There is no concrete pavement slab above the girder. The girders are connected into one unit by transverse prestressing. Such a solution is no longer applied today because it has proven to be poor in terms of durability. This system is difficult to upgrade, so the existing girders were removed and new 50 cm high SAN girders were installed on the overpasses, above which a 22 cm thick reinforced concrete pavement slab was constructed, and the lower structure was upgraded due to expansion.
The existing East overpass was demolished to build an underpass. The span of the new overpass East is 23.4 m (existing span 12 m) and is conditioned by the free profile of the tram line and the position of the underpass (the foundations of the new overpass are moved away from the underpass walls). Its construction posed a challenge in the dynamics of the works. Due to the size of the span, the required free profile for the tram line, and the retention of the level of the roundabout itself, the overpass could not be made of prefabricated elements. The start of work was at the same time conditioned by the completion of work on the underpasses.
At the site of the West overpass, an embankment is planned to be built over the underpass on the inner side of the roundabout, while a “green area” is planned on the western side over the underpass. The North and South overpasses were not the subjects of the Preliminary Design. The North overpass, above the tram line, has been repaired. Since there is no traffic under the South overpass, it was removed and an embankment was built in its place.

2.5 Noise protection

The locations of the walls for noise protection and the type of panels were corrected concerning the proposals from the preliminary design, and according to the Study of noise protection which was done in the phase of the main project. Noise protection was performed along the ramp Remetinečka-avenue Dubrovnik and on the part of the avenue Dubrovnik itself.

2.6 Installations

The biggest challenge in the design - and later in the planning of works and execution, was an extremely complex "installation node" in the area of the project, which had to be harmonized with all facilities and traffic areas as well as traffic equipment and signalization.
The project envisaged protection and relocation of all types of installations outside the underpass construction route with the construction of new parts of installations (sewage collector, water supply, drainage with the construction of underpass pumping station, electrical installations, TK and DTK installations, gas installations, public lighting...), as well as execution traffic equipment and accompanying installations (protective fences, traffic light cable channels, traffic information system, load-bearing structures - portal girders), and relocation of the existing and construction of a new tram cable and contact network. Before the works on the reconstruction of the intersection, it was necessary to relocate the installations in the excavation zone for the diaphragm and underpasses.
HEP-TOPLINARSTVO doo plans to lay a hot water pipeline in the project area. During the reconstruction of the intersection, preparations were made for laying pipes under Dubrovnik Avenue, through which hot water pipes will be laid/retracted later (during the construction phase of the hot water pipeline).

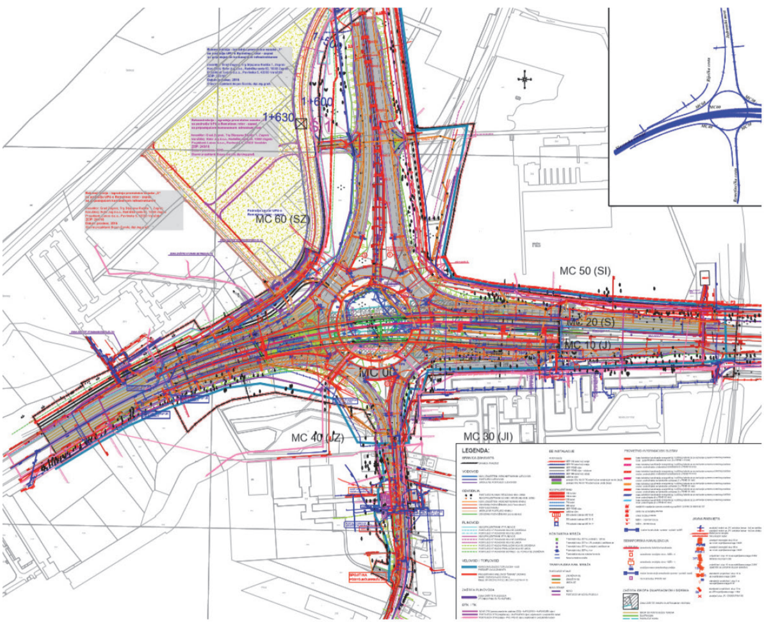

Figure 5 Illustrative view of the "installation node" in the area of works at the intersection

Figure 6 Leaking pipes for the hot water pipeline on Remetinečka cesta

2.7 Tram tracks

The tram track was repaired on the section in 2011, so the minimum length of the track was reconstructed as needed to fit into the new roundabout solutions. Within the roundabout, the project envisages a tram turnaround, with the additional possibility of a direct turn of the tram from the direction of the Adriatic Bridge towards Dubrovnik Avenue.

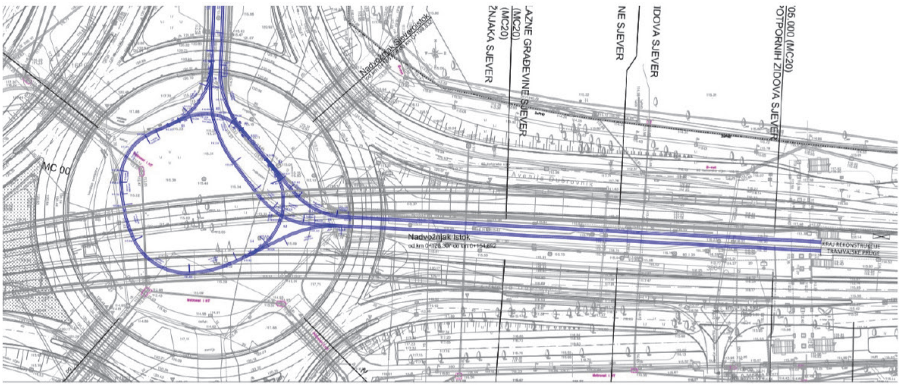

Figure 7 Presentation of a new tram line solution in the roundabout itself

3 Temporary traffic regulation

A special challenge was to find the optimal solution for the temporary regulation of traffic during the execution of works so that traffic and works take place with minimal restrictions, which would contribute to the execution of works within the defined deadlines. The preliminary design envisages the flow of traffic in the roundabout during the reconstruction works. Such a solution would significantly affect traffic flows, dynamics, and quality of work. An additional reason to think about relocating traffic outside the construction site zone was the fact that 17 lines of public city transport operate through the roundabout. Frequent changes in lines and timetables would cause additional problems in the organization of public city transport.

Since the conditions and manner of reconstruction (replacement of complete overpass supports) and the conditions for relocation of existing installations have changed, staged construction and traffic in the reconstruction zone was not possible and was replaced by a proposal to build temporary roads that completely relocate traffic outside the construction zone. In this way, the safe flow of traffic is ensured and the number of changes and necessary adjustments to the schedule of public city traffic is minimized, while at the same time the closure of the entire construction site is enabled, and thus unhindered and faster execution of works.

In the selected final solution of the temporary road, vehicles with a height of› 3.5 m (conditioned by the passage of the route under the existing HŽ overpasses on Dubrovnik Avenue) and out-of-town bus lines are excluded from traffic. Pedestrian and bicycle traffic was reduced to one traffic light crossing on Dubrovnik Avenue at the end of the project near Mate Parlova Street. With the closure of Kajzerica in the works zone, the traffic load and the possibility of congestion within the city district have been reduced. Mate Parlova Street from Dubrovačka Avenue to the school in M. Parlova Street was left as a pedestrian and bicycle corridor.

The challenge for the design was the development of a completely new road project through which 70-100 thousand vehicles will pass daily, within two months, and obtaining a building permit for that project. Traffic on the roundabout was closed gradually. First, tram traffic was suspended, to enable preparatory work on the relocation of installations, the start of work on the protection of the construction pit, and the construction of a temporary road. Only after the completion of the temporary road was the roundabout for motor vehicles closed. This organization of traffic and reconstruction works enabled the safe conduct of traffic and reconstruction works, as well as the achievement of agreed construction deadlines.

Figure 8 Presentation of the solution of temporary traffic management during the works

During the reconstruction works, occasional traffic counts were carried out at the control sections. Initially, 75,000-80,000 vehicles passed through the temporary roads daily; which was only 17–20 % less than the regular traffic before the reconstruction began. Eventually, traffic on temporary roads became higher than it was on the roundabout before work began. ZET's public city traffic took place regularly, on almost all lines as in regular condition and there were no delays caused by the works.

Figure 9 Temporary traffic management during the works

4 Concluding remarks

Since the announcement of the tender for the Contractor in November 2016, the signing of the Contract in May 2018, the preparation of the project of temporary roads, and the introduction of the Contractor in July 2018, despite many described challenges, planned deadlines - 09.01.2020. the first phase and 30.03.2020. the second phase of works - are complied with. During the project, a building permit was obtained for temporary roads and a new building permit for the entire project, which dealt with changes in the project and the construction phase. The contractual construction period of 18 months was adjusted to the selected phase of reconstruction of traffic areas in such a way that in the first phase traffic was released over the constructed roundabout so that temporary roads could be removed, which was a prerequisite for the completion of the tram, pedestrian areas and final horticultural landscaping.

Figure 10 Overview of the reconstruction of the intersection of Jadranska Avenue and Dubrovnik Avenue (visualization)

References

[1] Preliminary design of the Reconstruction of the intersection of Jadranska and Dubrovačka avenues in Zagreb, Business Association Aking d.o.o. and Faculty of Civil Engineering, University of Zagreb, Zagreb, 2010.

[2] Main and detailed projects for the reconstruction of the intersection of Jadranska and Dubrovačka avenues in Zagreb, the Association of Bidders Institut IGH d.d., and the Institute for the Improvement of Safety d.d., Zagreb, 2015-2018.

INCREASING LEVEL CROSSING SAFETY IN URBAN AREAS - CASE STUDY CITY OF ZAGREB

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Abstract

Level crossings (LC's) are one of the most dangerous points in railway traffic with frequent accidents that result in significant material damages and almost always fatalities. When level crossings are located within highly populated urban areas, they represent an even higher risk for accidents because of increased traffic volume for both the road and rail sectors. There are currently 34 level crossings in the City of Zagreb, some of which are on the roads with the highest traffic volume in the Republic of Croatia. Accident analyses on level crossings show poor traffic culture, especially pedestrians, which are intentionally disregarding traffic rules and showing poor judgment. This paper will show the existing condition and possible improvements of identified shortcomings of observed level crossings in the City of Zagreb and it will also present the existing level crossing regulations, classification, and safety on the railway network in the Republic of Croatia.

Keywords: level crossings, safety, urban areas

1 Introduction

The city of Zagreb is the capital of the Republic of Croatia and it is the largest city in the country with a population of 790.017 inhabitants [1]. As such, the Zagreb railway junction is the largest in the country and represents a central core in the railway network of the Republic of Croatia. Zagreb railway junction consists of 14 main and other international lines as shown in Figure 1.
Currently, there are 34 level crossings in the City of Zagreb within Zagreb railway junction and some of them are on the roads with the highest road traffic (vehicles as well as pedestrians) volume in the Republic of Croatia, as well as on the railway tracks with highest train movements in the country. Because of such a high traffic volume of both the road and rail sector level crossings in the City of Zagreb, level crossings are places with increased risk for traffic safety in general.

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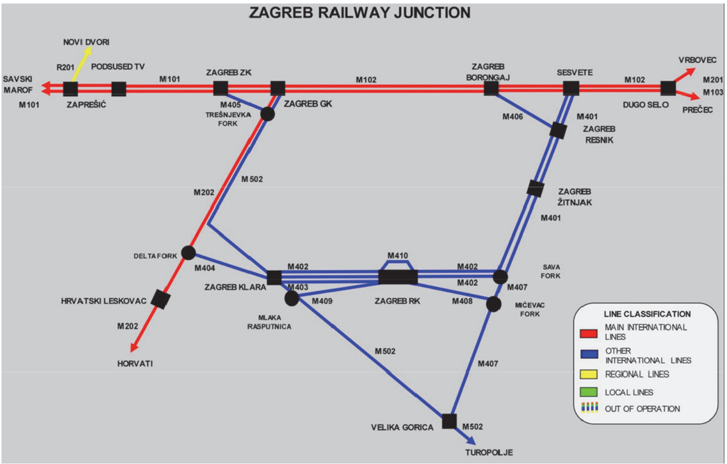

Figure 1 Zagreb Railway junction [2]

Since LC's are places where roads and/or pedestrian paths cross railway tracks, accidents at these places often result in serious damages and fatalities which diminish railway safety reputation, even though almost all of the accidents are caused by motor vehicle drivers or pedestrian violations [3]. Accidents at level crossings result in a higher mortality rate than any other types of road traffic accidents because of the disparity in mass between the train and the road user (motor vehicle, cyclist, or pedestrian). Furthermore, fatalities at level crossings accidents represent on average 30 % of all railway-related fatalities but only 1-2 % of fatalities of road traffic accidents [4]. As such, LC's represent a significant safety challenge due to complex socio-technical systems that involve interactions between many different types of road users and railway operators and infrastructure [5].

For that reason, level crossings need to be properly marked and protected with appropriate protection systems which can be divided into passive and active protection systems [6]. In passive protection systems, road users are solely responsible for observing traffic situations because traffic signs used for passive level crossings (traffic signs "Stop" and " St. Andrews Cross) do not change their state regardless of the approaching train. In contrary to passive systems, active protection changes its state to warn road users of approaching trains. This can be in the form of flashing lights and sound and/or full of half barriers [7]. Figure 2. shows the classification of protection systems in the Republic of Croatia.

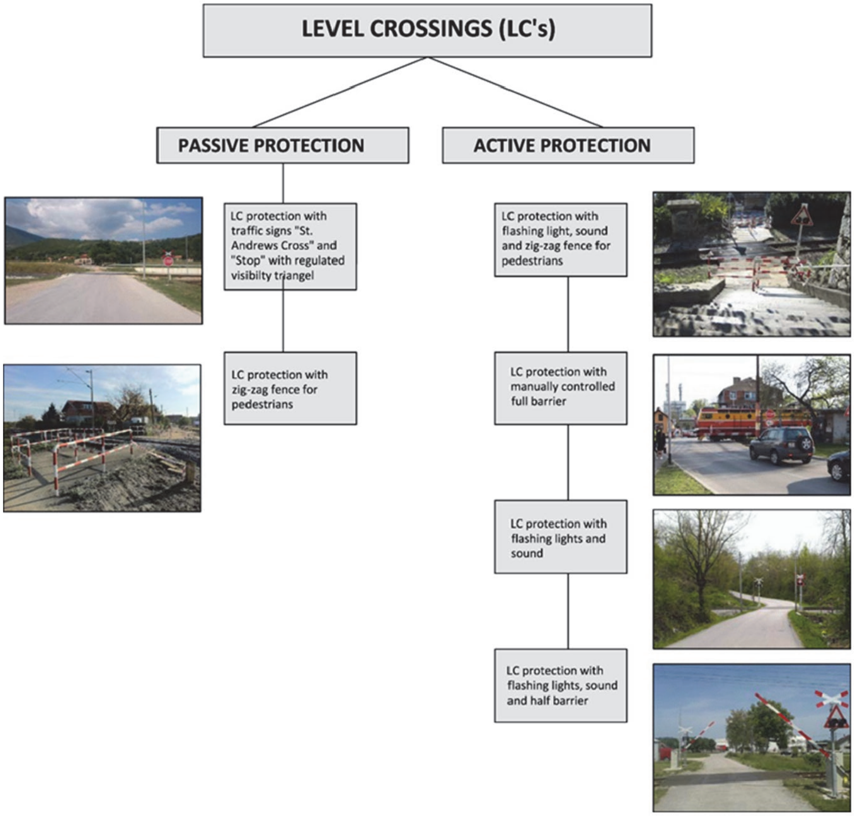

Figure 2 Classification of level crossing protection systems [7]

Despite technical protection systems, the number of accidents at LCs remains high, and studies point to user behavior as a key factor. Analysis of 256 LC accidents as part of the Safer European Level Crossing Appraisal and Technology (SELCAT) project showed that human failure caused 91 % of LC accidents in the EU; over 80 % occurred when a vehicle driver failed to respect the traffic rules [8]. A substantial proportion of LC's accidents occur when safety equipment at that crossing is functioning properly [9] further implicating risky road user behavior as the main problem. Thus, understanding what factors and situations lead road users to engage in risky behavior at LC's is key to predicting safety problems and designing effective countermeasures. In general, studies regarding level crossings safety can be divided into three categories: technical solutions, national and international safety programs, and educational campaigns [10]. To achieve maximum safety for all LC users, final safety measures should be designed equally within these three categories.

This paper aims to analyze all relevant statistical data regarding LC's safety and existing conditions within the Zagreb area and to show possible improvements of identified shortcomings of observed level crossings. The data was collected through a comprehensive search of available literature and national safety reports, as well as a field study by the authors.

2 Level crossing safety statistics for the Republic of Croatia

The total constructed length of the railway network in Croatia is 2.617 km, out of which 2.343 km are single-track and 274 km are double-track lines [2]. Out of 1.512 level crossings on the railway network in Croatia, 61,3 % have passive protection systems, and the remaining 38,7 % active systems [11]. When comparing to the EU average percentage of active protection systems (49 %) it can be noted that Croatia is behind the EU when it comes to active protection for LC's. [12].
Unfortunately, active protection systems for LC's are not a guarantee for decreased accidents and fatalities. Analyzing level crossing fatalities from 2007 to 2019 in the Republic of Croatia it can be observed that the overall number of fatalities is decreasing (Figure 3.), but what is concerning is that on average almost half of all the fatalities happened on level crossings that had active protection systems.

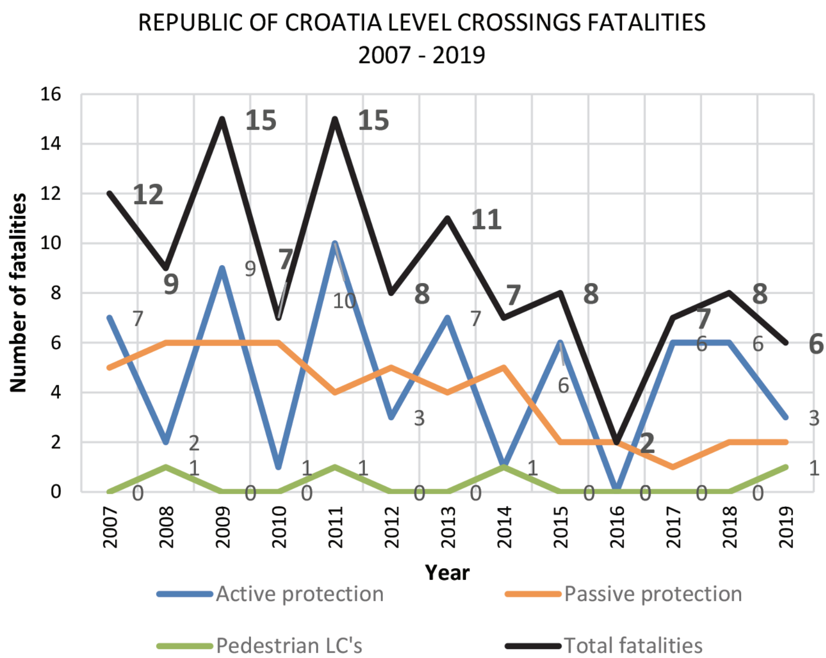

Figure 3 Level crossing fatalities in the Republic of Croatia [11][13]

Data shown in Figure 3. is a testament to a very poor traffic culture in Croatia, especially when all these active protection systems were properly working at the time of the accidents.
Another proof of poor safety culture in Croatia is the number of broken or damaged half barriers when road vehicles run into them. Since the breakage of the barriers happens while they are being lowered down or are completely in the final position, every such incident could lead to a potential accident with serious consequences because of the approaching train. Figure 4. is showing the number of broken/damaged barriers from 2007 to 2019.

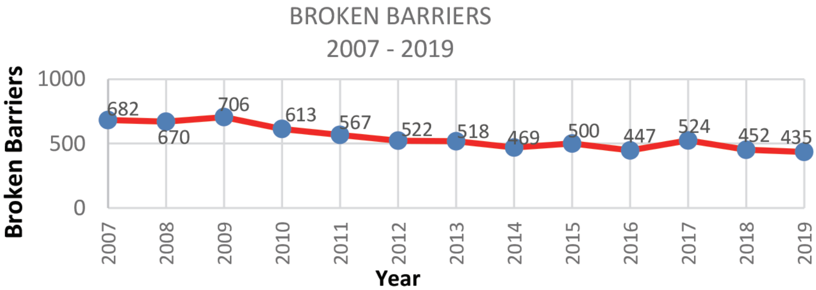

Figure 4 Broken/damaged barriers in the Republic of Croatia [11][13]

The overall number of broken or damaged barriers is continually decreasing over the observed period, but it is still significantly high, especial when there are only 454 level crossings with barriers in Croatia. The number of broken or damaged barriers only partially shows the real situation because only heavily damaged barriers are reported and there is no data about drivers who are intentionally driving around lowered barriers.

3 Level crossing in the city of Zagreb

There are currently 34 level crossings in the City of Zagreb, as shown in Figure 5.

Figure 5 Level crossings in the Zagreb area

Out of 34 level crossings, only seven of them have passive protection systems and the rest of them have active protection systems. Eight LC's have flashing light/sound systems, 17 have flashing light/sound and half-barrier and five LC's have a full barrier with the dedicated level crossing keeper [13].
In the period between 2010 and 2019, there have been a total of 34 accidents with 11 fatalities and 14 persons seriously injured on 15 level crossings, as shown in Table 1.

Table 1 Level crossing accidents in the City of Zagreb [13]

LINE	KM	LC's NAME	PROTECTION SYSTEM	ACCIDENTS (2010-2019)	FATALITIES (2010-2019)	SERIOUSLY INJURED (2010-2019)
M101 DG-Savski Marof-Zagreb Gk	426+357	R. Austrije	Full barrier + keeper	3	2	1
M101 DG-Savski Marof-Zagreb Gk	427+014	Vodovodna	Full barrier + keeper	1	0	1
M101 DG-Savski Marof-Zagreb Gk	428+853	Sokolska	Full barrier + keeper	2	0	1
M102 Zagreb Gk - Dugo Selo	430+112	Staj.Trnava	Light/sound +half-barrier	7	7	2
M102 Zagreb Gk - Dugo Selo	430+661	Osječka	Light/sound +half-barrier	1	0	1
M102 Zagreb Gk - Dugo Selo	432+393	Retkovec I	Light/sound +half-barrier	2	0	0
M102 Zagreb Gk - Dugo Selo	434+688	Sljeme	Light/sound	1	0	0
M102 Zagreb Gk - Dugo Selo	435+465	Jelkovečka	Light/sound	5	1	4
M102 Zagreb Gk - Dugo Selo	436+329	Selnica	Light/sound +half-barrier	1	0	0
M102 Zagreb Gk - Dugo Selo	440+295	Staklana	Light/sound +half-barrier	2	0	0
M202 Zagreb Gk - Rijeka	432+273	K.Mlinarić	Light/sound	3	0	3
M202 Zagreb Gk - Rijeka	436+981	Žižići	Light/sound	1	0	0
M202 Zagreb Gk - Rijeka	442+312	Horvati	Light/sound	3	0	1
M502 Zagreb Gk - Sisak - Novska	416+242	Buzin	Passive protection	1	1	0
M502 Zagreb Gk - Sisak - Novska	418+744	Utinjska	Light/sound +half-barrier	1	0	0

It can be observed that these accidents happened on 15 out of 34 level crossings that currently exist in Zagreb and only one accident/fatality on a level crossing with only passive protection system. Unfortunately, the rest of the accidents and fatalities happened on LC's with active protection systems that worked properly. Two LC's with the highest number of fatalities are LC R. Austrije and LC Trnava (Figure 6.) which are predominantly pedestrian crossings, even though they are seldom used by motor vehicles. Both LC's are located in highly urban areas on double-track railway lines with high traffic volume and very long barrier closure.
Because of such a long closure of the barriers (almost 44 % of the time within a 6-hour observed period), almost all of the pedestrians are using these LC's illegally by going under or around lowered barriers [14].

Figure 6 Level crossings R. Austrije and Trnava (Zagreb)

All the fatalities on these two LC's were pedestrians or cyclists going under or around lowered barriers. Another problem for pedestrians is a fact that these LC's are on double-track railway lines and they neglect the fact that once one train pass there is a real possibility of another train from opposite directions. Also, since most of the trains in the Zagreb urban area are low noise electric trains, it's hard to hear the approaching trains with all the rest traffic noise in urban areas.

4 Possible safety improvements on critical LC's

The best way to prevent accidents at level crossings is by building either overpasses or underpasses or completely close certain level crossings. Unfortunately, in most cases, it is just not financially or technologically justified for such large investments to be made, especially in large urban areas, such as the City of Zagreb. Since the vast majority of all illegal users on observed LC's are pedestrians and cyclists, measures for increasing safety on LC's should be concentrated on them.
Because of long LC's closure in urban areas, it is understandable for pedestrians and cyclists to be impatient, and therefore it is the main reason for illegal crossings. But, regardless of the barrier closure duration, it can be expected that pedestrians and cyclists will always try to cross illegally, especially if they can't visually see the approaching train. This is especially true in the case where railway tracks are in a straight line before the level crossing (such as LC Trnava, LC Vodovodna, LC R. Austrije direction from west, to name a few…).
The only way to prevent pedestrians and cyclists to cross illegally is to physically preclude them to enter the LC area. One way to accomplish that is to implement vertical bars underneath the barriers so that is impossible for anyone to go underneath the lowered barrier. Unfortunately, for that solution to work, it is necessary to install protection fences along the entire length of the railway track on both sides of the line. This solution will also prevent most trespassing of railway tracks outside of legal railway crossings. Another possible solution is to install warning billboards in front of each LC that will warn all users of the dangers of crossing illegally, especially on double-track lines where there is always a possibility of another train approaching from the opposite direction (all the LC's on lines M101 and M102).
Of course, even the best technical solutions will not suffice if the users don't obey them, so it is of utmost importance to have continuous educational campaigns about the dangers of level crossings. Unfortunately, the only educational campaign in the Republic of Croatia is conducted by "HŽ Infrastruktura" in form of periodical lectures in elementary schools and handing out educational safety brochures to drivers and pedestrians on selected level crossings [15]. This campaign, even though is well designed, is small in scale and doesn't cover all potential level crossing users and it should be extended to high schools and driving schools, as well as more active on available social networks which have a significant influence on the younger population. Also, it should be a part of the regular education curriculum in elementary schools so that the kids (future LC's users as drivers or pedestrians) learn from an early age about the dangers and proper use o level crossings.

5 Conclusion

Level crossings (LC's) are one of the most dangerous points in railway traffic, especially when they are located within a highly populated urban area where traffic volume is increased not only with motor vehicles but with pedestrians and cyclists. The City of Zagreb is the largest urban area in the Republic of Croatia with 34 level crossings within city limits. In the period between 2010 and 2019, there have been 34 accidents with 11 fatalities and 14 persons seriously injured on LC's in the City of Zagreb. Unfortunately, the majority of the fatalities were pedestrians and cyclists who didn't (intentionally or unintentionally) obey traffic rules when traveling over level crossings. Since most of these level crossings are located on railway lines with high traffic frequency, and thus level crossing closures are longer than average, this fact can not be justification for the illegal behavior of level crossing users. Since building overpasses or underpasses in such a highly developed urban area is not financially or technologically feasible, other measures need to be implemented to increase safety at level crossings. Since there is not one single measure that can accomplish full safety, a combination of a different set of measures is preferable. This can be accomplished with a set of technical solutions that can minimize or completely remove bad human decisions (such as vertical bars under barriers, protective fences) with a continuous educational campaign including school visits, media appearances, and warning billboards on every critical level crossing.

References

[1] Državni zavod za statistiku, Popis stanovništva RH 2011, https://www.dzs.hr/Hrv/censuses/census2011/results/htm/H01_01_03/h01_01_03_zup21.html, 2011.

[2] HŽ Infrastructure, Network Statement 2020, Zagreb, Croatia, 2019.

[3] Liang, C., Ghazel, M., Cazier, O., El-Koursi, E.M.: Developing accident prediction model for railway level crossings, Safety science, 101 (2018)

[4] Tordai, L., Olpinski, W., Schafer, W., Wegele, S.: D1 - Report about Statistics , Database Analysis and Regulations for Level Crossing, SELCAT (Safer European Level Crossing Appraisal and Technology), Paris, France, 2008.

[5] Read, G.J.M., Salmon, P.M., Lenn, M.G.: Sounding the warning bells: The need for a systems approach to understanding behaviour at rail level crossings, 2013, doi: http://dx.doi.org/10.1016/j.apergo.2013.01.007

[6] Starčević, M., Barić, D., Pilko, H.: Survey-Based impact of influencing parameters on level crossings safety, Promet - Traffic & Transportation, 28 (2016) 6, pp. 639–649

[7] Starčević, M., Broz, I.: Current Level Crossings Safety in the Republic of Croatia, ZIRP 2017 - International conference on traffic development, logistics & sustainable transport - New solutions and innovations in logistics and transportation, Opatija, Croatia, 2017, pp. 355–362

[9] Lazarevic, N., Khoudour, L., Koursi, E.M.E., Machy, C., Roberts, C., Impastato, S.: D2 - Report about Examination of actual and potential Technologies for Level Crossings, SELCAT (Safer European Level Crossing Appraisal and Technology), Paris, France, 2008.

[9] European Railway Agency, Railway safety performance in the European Union 2014., Valenciennes, 2014.

[10] Badanjak, D., Barić, D., Novačko, L.: Priority Measures of Improving Level Crossing Safety, 11th International Conference on Transport Sciences, Portorož, Slovenia, pp. 11–20, 2008.

[11] HŽ Infrastructure, Annual Safety Report 2018., Zagreb, Croatia, 2018.

[12] Starčević, M., Barić, D., Pilko, H.: Safety at Level Crossings : Comparative Analysis, 4th International Conference on Road and Rail Infrastructure - CETRA 2016, Šibenik, Croatia, May 2016, pp. 861–868

[13] HŽ Infrastructure, Internal Level Crossing Safety Statistics, Zagreb, Croatia, 2020.

[14] Barić, D., Pilko, H., Starčević, M.: Introducing experiment in pedestrian behaviour and risk perception study at urban level crossing, International Journal of Injury control and Safety promotion, 25 (2018) 1

[15] HŽ Infrastructure, Vlak je uvijek brži, https://www.hzinfra.hr/vlak-je-uvijek-brzi

NEW METHOD OF PREDICTING THE OCCURRENCE OF ROAD ACCIDENTS IN UKRAINE

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Abstract

Insufficient road safety level remains a serious problem in Ukraine. The number of people killed in road accidents in 2018 per 100 thousand of population is 9,11, while in the countries of the European Union, on average, this indicator is 5-6 deaths per 100 thousand of population. The growing number of vehicles and road users increases the likelihood of road accidents and, accordingly, the number of the suffered people increases. This regularity can be noted not only in our country, but also in the countries over the world. Method of prospective extrapolation is proposed for the prevention of road accidents which makes it possible to transfer the trends and links connected with the occurrence of road accidents in the past to the current period and for the future. To apply this method, it is necessary to have a qualitative statistical analysis of the road accident data. This provides an opportunity to foresee the occurrence of a particular type of accident, its location and the causes affecting its occurrence. By using the prospective extrapolation method, it is possible to reduce the number of road accidents and the severity of their consequences, including reducing the number of fatal accidents to a minimum.

Keywords: road safety, road accidents, road conditions

1 Introduction

Over the past decades, the world has witnessed a rapid increase in the number of vehicles and an increase in traffic which leads to an increase in traffic accidents and the severity of their consequences [1]. The UN General Assembly resolution 58/289 of 14.04.2004 «Improving the global road safety «[2] approved the concept:» It is impossible to achieve mobility at the expense of health and life of people «. According to the World Health Organization's estimates in 2030, road traffic accidents may be one of the five main causes of mortality in the world [3].
The Constitution of Ukraine recognized the life, health and human security as the highest social value in Ukraine of «[4]. In Ukraine, a number of legislative and legal acts which provide for increased responsibility of the national Ministries and Agencies, institutions and organizations for the implementation of measures to protect life and health» were adopted. The basic ones related to the road sector are the «Law of Ukraine on Road Traffic» [5]; «Presidential Decree on Additional Measures to Prevent Traffic Accidents» [6]. Based on the above list, it can be stated that human safety in Ukraine is a state policy.
According to the First International Congress on the Reform of the Road Traffic Safety Management System [7], the mortality rate on the roads of Ukraine is extremely high, and road traffic injuries are in the eighth place among the main causes of mortality and are the main cause of mortality among young people aged 15- 29 years old.

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The conducted studies on the level of accidents on Ukrainian highways show that, as compared with European countries, the state of road safety in Ukraine is extremely unsatisfactory. The number of deaths in Ukraine is 7-10 times higher than in the economically developed countries of the European Union [2 , 8].
Ways of solving the problems of reducing the accident rate on highways are proposed in the works of many Ukrainian scientists. Polishchuk V.P., Yerezov V.I., Lanovyi O.T., Kunitskaya O.M., Dzyuba O.P., in their works, analyzed and advised on reducing the accident rate and the severity of the consequences of an accident [1, 9, 10].
In the paper [11], a thorough analysis of the circumstances of the road traffic accident on public roads in Ukraine was conducted. The presented analysis showed that in many traffic accidents there was an influence of the disadvantages of road traffic conditions which, in turn, led to the driver's errors and the complications in driving. It is proved that the most frequent accidents occur in places where drivers must suddenly change the modes of movement due to the sudden complication of road conditions. Other causes were also analyzed, leading to an accident, and the measures were presented to reduce the number of accidents and the severity of their consequences [1, 11]. In the above-mentioned works it is stated that in order to achieve the reduction of the accident rate, it is expedient to develop the programs for the development of public highways on the basis of the methodology of road safety management in the region of Ukraine.
The Belarussian scientist Kapsky D.V. in his works describes the theoretical basis for predicting the accident rate and the dependence of the accident occurrence on various factors, and proposes methods for solving this problem [12]. The occurrence of accidents is considered by the author as a jump-free transition from the normal process of driving to an emergency one due to the emergence of a conflict situation when the driver makes the wrong decision and changes the parameters of driving.
The analysis of statistics of road traffic accidents in Europe convincingly proves that on the motorways allowing only motor vehicles traffic and prohibiting the traffic of all other types of vehicles, as well as pedestrians, the number of accidents per million kilometers of the vehicle mileage is 2-3 times lower than on ordinary roads open to all road users.
Overseas statistics states that only the lack of road markings can increase the number of accidents by 25 %.
It is driving skills during the critical assessment of real road conditions and the choice of safe driving techniques can help to improve road safety.
According to the World Health Organization, the World organization "Life expectancy research" ranked the countries by road traffic deaths level (Table 1).
Ukraine has the indicators of the number of deaths in road accidents by 100 thousand people worse than in Europe. Only Albania, Lithuania, Belarus and Moldova are worse than Ukraine. Thus, it is extremely important to develop methods that will influence the reduction of traffic accidents and the severity of their consequences, namely: improving the behavior and discipline of road users; improvement of vehicle technical characteristics and improvement of road conditions.

Table 1 The number of deaths in traffic accidents per 100,000 of population in Europe in 2018

Country	Killed per 100 thousand people.	Country	Killed per 100 thousand people
Sweden	2,49	Serbia	6,38
Great Britain	2,58	Bulgaria	6,40
Netherlands	2,81	Belgium	6,61
Switzerland	2,82	Luxemburg	6,71
Denmark	2,89	Greece	7,00
Norway	2,93	Macedonia	7,02
Spain	2,94	Slovakia	7,30
Iceland	3,24	Romania	7,89
Germany	3,54	Poland	7,93
Finland	3,73	Croatia	8,22
Ireland	4,01	Turkey	8,85
Italy	4,72	Latvia	8,85
Australia	4,74	Montenegro	9,01
France	4,86	Ukraine	9,11
Slovenia	5,75	Moldova	9,90
Estonia	5,94	Belarus	11,16
Czech	5,97	Lithuania	11,34
Portugal	6,11	Albania	12,32
Hungary	6,22	Russia	15,85

2 The main part

To develop methods for identifying the causes of an accident, planning and implementing effective measures to eliminate them, the authors identified objective and subjective factors that affect road safety. Objective factors include road conditions, traffic flow, weather conditions. To the subjective factors, the state of drivers and pedestrians, violation of traffic rules by them can be referred to.
Every year, a large number of traffic accidents occur in Ukraine, people are killed and injured. Only in 2018, according to the departmental data base of the accidents analysis and recording (Road Safety Management (RSM) which is created and functioning at «DerzhdorNIDI» SE, the number of traffric accident victims on the roads of state importance of Ukraine increased by 2.9 % as compared with 2017 (Table 2).
The authors of this article noted that road safety is most of all affected by the following factors:

- meteorological factors (weather conditions);
- traffic flows (human factor, vehicle's reliability);
- road conditions (road conditions).

Table 2 Accident statistics on roads of state importance for 2011-2018.

Year	Traffic accidents total	Traffic accidents with injured	Killed (persons)	Injured (persons)
2011	18687	5982	1774	8037
2012	24755	7808	2579	10506
2013	23685	7498	2152	10277
2014	17280	5958	1848	8334
2015	14499	5417	1556	7508
2016	15371	5203	1453	7382
2017	16250	5104	1378	7722
2018	16231	4686	1418	7079

Meteorological factors are characterized by the state of atomospheric phenomena. These phenomena include temperature, pressure, humidity, wind, clouds, precipitation, fog, thunderstorms, snow cover height and others. These phenomena can be long-term and short-term. Long-term atmospheric phenomena include, for example, negative temperature and snow cover in winter; the short-term atmospheric phenomena include precipitation, fog, ice. All these phenomena adversely affect the road safety. After all, when the weather conditions are changing, the situation on the highways is also dramatically changing. This affects the condition of the surface of the roadway, by reducing the coefficient of tyre grip with the pavement surface which leads to a sharp decrease in the road safety level.

The next factor that affects the road safety is traffic. Traffic includes the human factor and the reliability of vehicles. The technical condition of vehicles and their equipment must meet the requirements of standards relating to road safety and environmental protection, as well as technical regulations and other normative and technical documents.

In most countries, public opinion and official statistics often attribute the causes of traffic accidents tothe driver's mistakes. So, the World Health Organization believes that 9 out of 10 accidents are caused by the drivers and other cases to some extent depend on it [3]

Another important factor affecting the road safety is road conditions. The authors analyzed a number of major disadvantages in road conditions that affect the occurrence of certain types of road traffic accidents (Table 3).

The list of deficiencies was identified by the specialists of «DerzhdorNDI» SE by comparing the indicators of the operating condition of the road with the regulatory requirements. Identification of specific disadvantages allows predicting the probability of occurrence of certain types of accidents and estimating the probability of the absolute number of the accident level reduction provided that the road conditions are improved, that is, the elimination of these disadvantages.

The analysis of statistical data performed by the specialists of «DerzhdorNDI» SE shows that the most common types of accidents include collisions and riding on the obstacles (Figure 1).

Table 3 Disadvantages in road conditions that contribute to the occurrence of certain types of road traffic accidents

Type of traffic accident	Disadvantages in road conditions that contribute to the occurrence of this type of road accident
Collision	Nonconformance of the width of the carriageway; Nonconformance of the radius of the curve in the plan; Nonconformance of the visibility with the regulatory requirements for the roads of this category; the level of the road loading exceeds the optimal; the absence of a centre mall or the safety barrier on a centre mall depending on the category of the road; nonconformance of the intersection type and the junction to the traffic volume; the absence of speed change lanes at the approaches and ramps of road interchanges.
Roll over	Absence or nonconformance of the transverse superelevation gradient of the curve in the plan with the normative designing requirements; the radius of the curve in the plan and the expansion of the carriageway do not meet the regulatory requirements for the roads of this category; lack of safety barriers in the right places; unsatisfactory state/the absence of the hard shoulders; lack of hard pavement at the ramps.
Riding on the obstacle	Close proximity of the trees to the carriageway edge; lack of fencing of the electric lighting supports and other obstacles; Unsatisfactory state of the roadside.
Riding on the vehicle that is standing	Insufficient width of the sidewalk; insufficient width of the stopping area; nonconformance of the visibility distance with the regulatory requirements for the roads of this category; lack of parking lots near the service objects.
Riding on the pedestrian	Lack of equipped pedestrian crossings in the required places; absence or unsatisfactory condition of sidewalks and pedestrian paths in the settlements; nonconformance of the visibility distance with the regulatory requirements for the roads of this category; absence of bus stops in the right places; lack of lighting in the settlements; unsatisfactory state of the roadside.

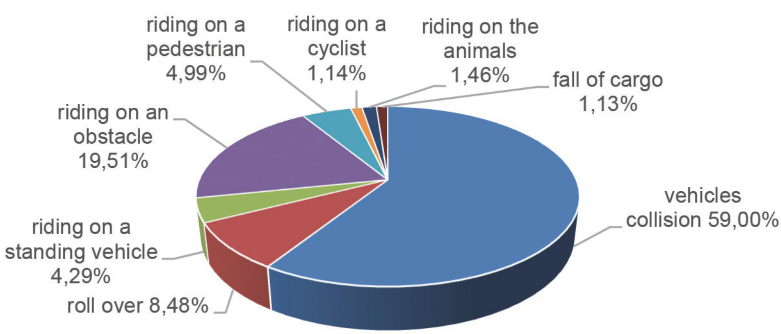


Figure 1 Distribution of traffic accidents by types of accidents on the roads of state importancefor 2018

3 Prospective extrapolation method

The authors propose an extrapolation method based on analytical indicators of dynamics series to prevent the occurrence of road accidents. The method of extrapolation is a method of statistical analysis which allows transferring the trends that are associated with the occurrence of road traffic accidents from the past to the current period and for the future. Dynamic series (the dynamic row) is called a sequence of indicators characterizing the change of the phenomenon (process, object) in time. Separate observations of a dynamic series are called the levels. By time displayed in dynamic series, they are divided into moment and interval. In the series, the dynamics of the level express the magnitude of the phenomenon to the corresponding date, for example, the number of accidents on the first day of each month or the number of accidents at the beginning, end of the year, etc. In interval series the levels express the quantity of events over a period of time, for example, the number of accidents per month, quarter, year. When constructing the dynamic series, the focus should be on the comparability of the levels of the series. This means that all the levels must be expressed in the same units of measure, calculated according to a single methodology, and include a single circle of objects.

The quality of forecasting can only be judged after the event has taken place. To assess the reliability of the method used, the so-called «ex post forecast» method is used. Its essence is as follows.

The initial data are divided into 2 parts (two periods). According to the first part, conditionally adopted for the «prehistory», an equation (model) is constructed on the basis of which the forecast for the second part (second period) is made, the results of which are then compared with the actual data. This approach is also applicable to other quantitative methods of forecasting. The disadvantage of this method is that when calculating only the extreme levels of the dynamic series are used. Intermediate values, in fact, are not involved in the calculations.

To apply the method of extrapolation, we use a qualitative statistical analysis of data on accidents. In Ukraine, a study was conducted for which certain results were obtained. Statistical data for the full four years were used for the study. The main task of this scientific study is the analysis of the number of traffic accidents for the four-year period, which occurred on the territory of the Lviv region. The criterion of distribution is the severity of consequences (Figure 2).

As can be seen from Figure 8, the number of traffic accidents increases by more than 11 % on average in each subsequent year, which can be explained by the growth of traffic volume (in general in the region by 10-15 % annually in the in the considered periods), since the riding qualities of streets and roads and road conditions in general were almost unchanged during this period.

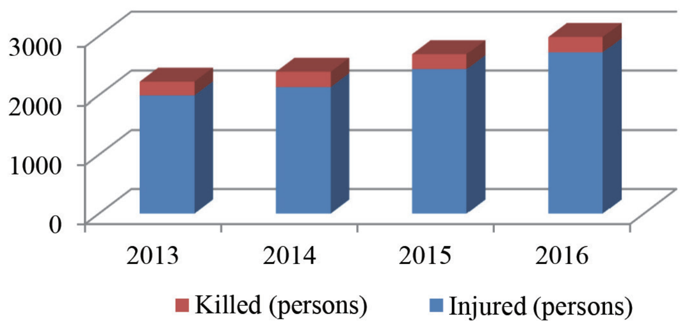
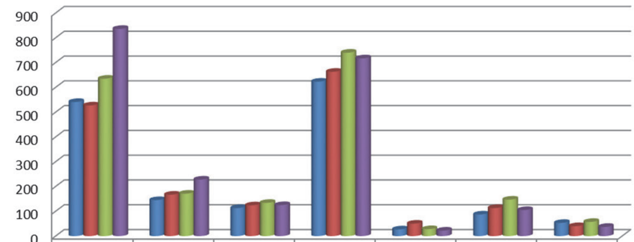


Figure 2 Number of traffic accidents in Lviv region in 2013 - 2016

It is important to note that the existence of a certain dependency between the number of traffic accidents and changes in the value of traffic volume allows making forecasts of changes in this relationship for future periods, using methods of direct extrapolation of the current state, which are usually applied for a short time period (up to 5 years and under condition that the territorial and road characteristics have not changed from the moment of the study). However, the total number of traffic accidents is not always a complete and sufficient indicator to make accurate conclusions. For this purpose the analysis of statistical data by types is also carried out: vehicles collision; rollovers; riding on a standing vehicle; riding on an obstacle; riding on a pedestrian; riding on a cyclist and the like. For considered period and territory, this distribution is shown in Figure 3.



	vehicles collision	riding on an obstace	rollover	riding on a pedestrian	riding on a standing vehicle	riding on a cyclist	others
2013	541	145	113	623	27	87	53
2014	527	167	124	663	50	113	40
2015	635	171	134	740	28	147	57
2016	836	228	125	717	22	105	37

Figure 3 Distribution of traffic accidents by the type in the Lviv region for 2013-2016.

The largest number of traffic accidents during 2013-2016 are vehicles collisions and riding on a pedestrian. Very often the causes of these types of traffic accidents are exceeding the permitted speed limit, driving while intoxicated, lack of illumination or insufficient amount of illumination [17].

4 Conclusions

The number of traffic accidents with the victims on the roads of state importance of Ukraine from 2012 to 2018 has decreased but the number of deaths, on the contrary, has increased. The most common types of road accidents are collisions, riding on a vehicle, riding on a pedestrian, riding on an obstacle and rollover. The research of these types of accidents allows making the important conclusions and issuing the recommendations necessary for preventive work with the drivers, pedestrians, cyclists and all other road users. The analysis shows that the accidents occur under well-defined, constantly repeating circumstances. Moreover, each type of accident is characterized by the same emergency situations that need to be considered in detail in order to propose the methods that will promote preventing the occurrence of an accident. That is why, in order to prevent traffic accidents, an extrapolation method is proposed which will allow transferring the trends associated with the occurrence of road traffic accidents from the past to the current period and for the future. For the application of this method, a qualitative statistical analysis of the accident data should be performed. This allows predicting the occurrence of a particular type of road accident, the place and the causes that will affect their occurrence. From documentary studies it can be argued that a detailed analysis of the causes and consequences of accident rate and single traffic accidents will allow making predictions about the dynamics of changes in their number depending on changes in road conditions, riding qualities of carriageways and application of preventive measures to improve road traffic.

References

[1] Polishchuk, V.P., Nahrebelna, L.P.: Analysis of the factors causing road accidents on public roads and proposals for their elimination, Roads and bridges, Kyiv, 16 (2016), pp. 82 - 85

[2] Resolution of the General Assembly of the United Nations 58/289 of 14.04.2004, Improving Road Safety in the World, UN Database, https://undocs.org/en/A/RES/58/289

[3] Strategy to improve the level of road safety in Ukraine for the period up to 2020, Order of the Cabinet of Ministers of Ukraine of June 14, 2017, N 481 - p, Database Legislation of Ukraine, Cabinet of Ministers of Ukraine, https://zakon.rada.gov.ua/laws/show/en/481-2017-%D1%80/print

[4] Article 3 of the Constitution of Ukraine (Reports of the Verkhovna Rada of Ukraine) dated 1996 N141, Database Legislation of Ukraine, Cabinet of Ministers of Ukraine, https://zakon.rada.gov.ua/laws/show/254%D0%BA/96-%D0%B2%D1%80

[5] About traffic, Law of Ukraine of June 30, 1993 N3353-XII revision dated 04.02.2019, N 2478-VIII, Database Legislation of Ukraine, The Verkhovna Rada of Ukraine, https://zakon.rada.gov.ua/laws/show/3353-12

[6] On urgent road safety measures, Order of the President of Ukraine dated 17.06.2008 No.556, Database Legislation of Ukraine, The Verkhovna Rada of Ukraine, https://zakon.rada.gov.ua/laws/show/556/2008, 2008.

[7] The First International Congress on the Reform of the Road Traffic Safety Management System, Kyiv, The final resolution, https://ucci.org.ua/media/filemanager/Resolution_END_20.06.17.pdf

[8] Kuzhel, V.P.: Estimation of the range of visibility of road objects in the dark time of day in the examination of road accidents with the help of fuzzy logic, Bulletin of Kharkiv National Automobile and Road University, Kharkiv, 2008, pp. 91-95

[9] Gavrilov, E.V., Dmitrichenko, M.F., Dolya, V.K., Lanovyi, O.T., Linnyk, O.E., Polischuk, V.P.: Traffic management, Manual, Kyiv, Znannia Ukrainy, 2006.

[10] Eresov, V.I., Grigorieva, O.V.: Ways of traffic management in conditions of heavy traffic, Bulletin of the NTU, Kyiv, 24 (2011) 2, pp. 153-157

[11] Polishchuk, V.P., Kutuzov, A.Ye.: On the issue of management of traffic safety on the public roads of the region, Bulletin of NTU, Kyiv, 28 (2013), pp. 380 - 385

[12] Kapsky, D.V.: Theoretical bases of prediction of accidents on conflicting objects, Roads and bridges, Kyiv, 8 (2008), pp. 83 - 87

VEHICLE MISMATCH – A CASE STUDY

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Abstract

Sport utility vehicles (SUV) gain more popularity and with more manufacturers being involved in their production their accessibility rises as well. This however creates an opportunity for collisions with smaller passenger vehicles. There is obvious mismatch in weight, stiffness and height between SUVs and other, smaller passenger vehicles. Furthermore, the average age of passenger vehicles in Czech Republic is over 15 years. Even when these older vehicles crash even with vehicle of similar weight and build, there is a significant mismatch in stiffness and safety equipment (especially airbags). These kinds of vehicle mismatches thus create risk of more serious injuries in case of crashes. The Czech In-Depth Accident Study project (CzIDAS) collects on-site crash data and injury data for further analysis of traffic accidents in order to present traffic risk factors. Analysis of vehicles' collision speed and damage is carried out and verified using simulation programme calculation, information about passengers' injuries is obtained from contracted hospital facilities. The traffic accidents presented in this case study serve to showcase the risks associated with vehicle mismatch crashes, currently happening on roads of Czech Republic.

Keywords: vehicle; crash; injury; vehicle mismatch; accident analysis

1 Introduction

Sport utility vehicles (SUVs) became more popular and with more vehicle manufacturers incorporating SUVs in their production their accessibility rises, as well as probability of their involvement in road traffic crashes. In the case of vehicles such as SUVs or MPVs, the construction is different from a conventional (compact) passenger vehicle (with a self-supporting body). The body of SUVs consists of a separate chassis, which is composed of several longitudinal elements (beams) and transverse elements (crossbars). The structure is heavier and significantly stiffer, and the distribution of forces changes during the collision. Each vehicle model and its construction have its own structural behaviours (Vangi, 2020).
While driving modern (and heavier) vehicle provides safety benefits to driver and other passengers of said vehicle, at the same time, (in case of crash with other passenger vehicle) it creates a safety issue in form of mismatched crash (i.e. weight, stiffness and safety equipment). Vehicle mismatch drastically influences course of the collision (from extent of vehicle damage to severity of sustained injuries). Important factor is magnitude of force impulse during the collision and subsequently damage to vehicle interior and so called survival space influencing severity of the road traffic crashes (Brewer and Smith, 2008).
As mentioned above, among other things, a mismatch can occur even in form of vehicle safety equipment and stiffness and seemingly matched vehicles can prove otherwise, especially when one of the vehicles is significantly older (the average age of passenger vehicles in Czech Republic is over 15 years) (CIA, 2020).

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The relative effect of stiffness and weight parameters on risk of driver's fatal injury in a head-on collision was explored in (Eyges, 2009). Authors of this study point out, that according to their research, weight of the vehicle has greater influence on collision outcome compared to vehicle weight.
Similar research concerning vehicle mismatch in collisions of passenger vehicles and light truck vehicles (including SUVs), was seen in (Acierno, 2004, Mandell, 2010; Desapriya, 2013). It was concluded that vehicle mismatch was associated with death and serious injuries in vehicle crashes. While passenger vehicles have become safer, many of the safety features have been designed for crashes with other passenger vehicles. Thus, emphasis was put on improving performance of vehicles when struck by a higher barrier and re-designing both passenger vehicles and light truck vehicles to be more compatible in frontal collisions. In (Toy, 2003) vehicle mass was found to be influencing crashworthiness of light truck vehicles relative to passenger vehicles.
Another study (Cobb, 2005) concluded increased risk of spinal injury for passenger vehicle occupants when involved in two vehicle crash with light truck vehicles. Interestingly the study also presented increased risk of spinal injury for occupants of light truck vehicles in general, however this was thought to be a result of lower safety standards for trucks. Chipman (2004) described vehicle size disparity, especially when the struck vehicle is smaller and lighter, as almost a consistent risk factor for occupant injury. Desapriya (2013) stated, that occupants in passenger vehicles that collide with vehicles on truck frames were at twice the risk for injury, because vehicles on truck frames inflict significant body damage to passenger vehicles.
Takubo (2000) also pointed that structural characteristics of larger vehicles (as SUVs) foster human errors. Though the viewpoint of the driver is high, enabling him to observe traffic situation in front of the vehicle, there is a risk that an overconfident driver may grow careless. Also, SUV's centre of gravity is high, which can magnify rolling motion. Study (Ross, 2003) supports these findings and points out that driver's behaviour (human error) and vehicle design (vehicle structural properties) can not be separated when evaluating risk factors.
It is important to note, the topic of vehicle mismatch is a safety issue discussed for more than two decades, but lately seems more pronounced (and relevant) due to aforementioned popularity of SUVs. Vehicle mismatch, however, is not issue exclusive for SUVs, but for all mismatched collision opponents. While mismatch in collision of passenger vehicle with train, tram or truck is obvious, the focus of this article are collisions of two passenger vehicles.

2 Methods and crash reconstruction

2.1 Data collection and analysis

Data used in this study were gained as part of CzIDAS (Czech In-depth Accident Study) project currently implemented by the Transport Research Centre, in Czech Republic. The methods utilised by the CzIDAS project are primarily based on German In-depth Accident Study (GIDAS), however the project uses its own certified methodology, better suited for Czech traffic environment.
The project involves detailed documentation of road traffic crashes and their consequences with aiming to present causes both crashes themselves and of the injuries sustained during these crashes. The analysis of events occurring during crashes was based on objective data gained from the scenes of traffic crashes, i.e. photo documentation of damage to vehicles, the surrounding environment and trace evidence documented by geodetic measurements (i.e. total station, GNSS). Information from medical facilities about the injuries of those involved in traffic crashes is also acquired. The events occurring during crashes are recreated with the aid of computer simulation modelling. The simulation programme Virtual CRASH (version 4.0) was used for the simulations, in which not only the dimensions of the collision partners are considered, but also their weights.

2.2 Overview of passenger vehicles crashes

There were 456 road traffic crashes in CzIDAS database involving collision of two passenger vehicles. The influence on crash severity of two main factors involved in vehicle mismatch (i.e. vehicle age and weight) is shown in following tables.

Table 1 Influence of vehicle age on crash severity

	Uninjured [%]	Minor injury [%]	Severe injury [%]	Fatal injury [%]
Up to 4 years	41	56	2	1
5 to 9 years	36	59	4	1
10 to 14 years	36	56	6	2
Over 15 years	28	63	6	3

Table 2 Influence of vehicle weight on crash severity

	Uninjured [%]	Minor injury [%]	Severe injury [%]	Fatal injury [%]
Up to 1000 kg	14	75	8	3
1000 to 1500 kg	34	59	5	2
Over 1500 kg	46	50	3	1

It is obvious in case of two passenger vehicles collision, with increasing age of the vehicle and decreasing weight the risk of severe or fatal injury increases. Safety features, standards and material used in construction of older vehicles, together with the technical condition of the vehicle and the associated gradual degradation of material (most often seen as corrosion of the vehicle body) could influence the crash consequences significantly. Especially the corrosion of main structural parts could lead to greater extent of vehicle damage possibly leading to intrusion into vehicle interior.

3 Case studies

To present the risk involved in vehicle mismatch, the following road traffic crashes of mismatched vehicles are presented together crashes of similar configuration, but of more compatible vehicles.

Table 3 Summary of incompatible (mismatched) and compatible collisions and involved vehicles

Case / compatibility		Vehicles	Vehicle type	Curb weight [kg]	Make year	Impact type	Injury severity
1	I	Kia Sportage	SUV	1576	2011	Head-on collision	Minor
		Renault Megane	Hatchback	1050	2002		Severe
	C	Škoda Fabia	Hatchback	1139	2001	Head-on collision	Minor
		Škoda Octavia	Combi	1345	2010		Minor
2	I	Ford C-Max	MPV	1450	2005	Sideswipe / Small overlap	Minor
		VW Golf IV	Hatchback	1145	2003		Fatal
	C	Ford Focus II	Combi	1351	2005	Sideswipe / Small overlap	Minor
		Škoda Octavia II	Combi	1426	2009		Minor

3.1 Case no. 1 – head on collision

3.1.1 Incompatible collision

This crash occurred on route with number of curves an elevation changes as driver of vehicle Kia, probably due to microsleep, crossed centreline into opposing lane. Afterwards, vehicle Kia hit head-on vehicle Renault (full frontal collision).

Figure 1 Incompatible collision, left: vehicle Kia; right: vehicle Renault.

Vehicle damage:

- Front side of vehicle Kia was damaged, the deformation was extended up to the front axle of the vehicle. Both driver and passenger airbags were deployed.
- There was damage to the front part of vehicle Renault as well, however the deformation was of greater extent with engine mount damage and engine itself shifting inside the engine compartment. Vehicle passenger compartment was impacted as well. Dashboard was damaged and legroom was intruded, both driver and passenger airbags were deployed.
- Injury severity:
- Driver of vehicle Kia (65 years old) sustained only minor injuries.
- Driver of vehicle Renault (45 years old) sustained severe injuries – comminuted, open fracture of right femur lower part, comminuted fracture of lower end of right tibia, fracture of left femoral neck, sternum fracture, 9th to 10th thoracic vertebra fracture, fracture of 2nd to 4th metatarsus of left leg, overall injury severity expressed by ISS was 13. A passenger in front seat (43 years old) sustained minor injuries and passenger in rear right seat (37 years old) sustained severe injuries.

3.1.2 Compatible collision

This crash occurred on long, straight stretch of the road, as driver of vehicle Skoda Fabia, from unknown reasons, crossed centreline into opposing lane and hit head-on vehicle Skoda Octavia.

Figure 2 Compatible collision, left: vehicle Skoda Fabia; right: vehicle Skoda Octavia.

Vehicle damage:

- Vehicle Skoda Fabia had damage of front right corner, the deformation was reaching right front wheel, which was shifted backwards, right front fender was destroyed, and right front rail was also damaged, the cross beam was shifted backwards. Both passenger and driver airbags were deployed.
- Front side of vehicle Skoda Octavia was damaged, the deformation was extended up to the cross bar. Both driver and passenger airbags were deployed.
- Injury severity:
- Driver of vehicle Skoda Fabia (84 years old) sustained only minor injuries.
- Driver of vehicle Skoda Octavia (37 years old) and two passengers (2 and 31 years old) sustained only minor injuries.

3.2 Case no. 2 – sideswipe collision

3.2.1 Incompatible collision

This crash occurred as driver of vehicle Ford, from unknown reasons, crossed centreline into opposing lane and hit head-on vehicle Volkswagen (small overlap / sideswipe collision).

Figure 3 Incompatible collision, left: vehicle Ford; right: vehicle Volkswagen.

Vehicle damage:

- Left and front side of vehicle Ford were damaged. The impact was pointed towards driver, which is the reason not only front side and engine compartment were damaged, but also A-pillar, left sill and left front door. Dashboard and driver seat mounting were damaged. Both driver and passenger front airbags were deployed.
- Left and front side of vehicle Volkswagen were damaged. In this case the impact was pointed towards driver as well, engine mount and left side were damaged. Leg room was intruded, and dashboard shifted inside the vehicle. Both driver and passenger front airbags were deployed.

- Injury severity:
- Driver of vehicle Ford (29 years old) sustained only minor injuries (chest bruises)
- Driver of vehicle Volkswagen (20 years old) was fatally injured. Front seat passenger sustained severe injuries.

3.2.2 Compatible collision

This crash occurred as driver of vehicle Skoda crossed centreline into opposing lane and hit head-on vehicle Ford (small overlap / sideswipe collision).

Figure 4 Compatible collision, left: vehicle Škoda; right: vehicle Ford.

Vehicle damage:
- Front side of vehicle Škoda was damaged, the deformation was of mostly the left side of the vehicle and extended up to the front axle of the vehicle. The left front rail was not deformed, however left front wheel was ripped off and there was severe deformation of left A-pillar, left sill and left front door. Both driver and passenger airbags were deployed.
- Vehicle Ford had damage was focused to left corner, the deformation reached up to left front wheel, which was shifted backwards, left front fender was destroyed, left A-pillar was damaged and left door panel was ripped off. Left front rail was not damaged Only left curtain airbag was deployed.
- Injury severity:
- Driver of vehicle Skoda (32 years old) was not injured, a front seat passenger (35 years old) sustained only minor injuries.
- Driver of vehicle Ford (48 years old), front seat passenger (44 years old) and baby (2 years old), seated in the left rear seat sustained only minor injuries.

4 Discussion and conclusion

Vehicle crash compatibility as health protection policy was discussed in number of previous studies (e.g. Jenefeldt, 2004). This complex issue, however, requires reconciliation of structural interaction, stiffness, and weight of vehicles. In case of mismatched collision, there is a risk of increased vehicle deformation and passenger compartment intrusion, leading to sever or fatal injuries (Faerber, 2001; Jenefeldt, 2004).
Just like aforementioned studies (e.g. Desapriya; 2005, Chipman, 2004), CzIDAS data suggest both vehicle weight and age affect severity of injuries documented in passenger vehicle collision. This safety issue is presented using cases of both incompatible (mismatched) and compatible passenger vehicle collisions.
The presented cases of incompability were present in form of significant weight, construction (i.e. age or materials used) and shape differences of involved vehicles, which led to severe injuries of disadvantaged vehicles' occupants. In the compatible cases, the vehicles involved were better matched in weights and the platforms on which the vehicles were built were of similar technology, making the vehicles more crash compatible.
It is obvious, vehicle mismatch poses a safety concern, which will need to be addressed by vehicle manufacturers. Euro NCAP implemented new moving barrier to moving vehicle frontal

crash test, replacing the regulation-based moderate offset-deformable barrier test, used by Euro NCAP for the last 23 years. This new crash test not only evaluates the protection of occupants inside the passenger vehicle, but also assesses how the vehicles' front-end structurers contribute to injuries in the collision partner. (EuroNCAP, 2020)

An emphasis is put on making new (larger) vehicles, such as SUVs more compatible in collisions with smaller vehicles. However, as was mentioned above, vehicle weight is only one factor, leading to vehicle mismatch, the other significant factor being vehicle age. Besides fewer safety features, old vehicles are also often associated with poor condition (most noticeable in form of corrosion) further influencing crash injury severity (mainly due to more significant deformation of vehicle). Thus, an emphasis should be placed on periodic vehicle inspection, vehicles failing requirements to pass these inspections should not be permitted to be operated in traffic to ensure safety of passengers.

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References

[1] Takubo, N., Mizuno, K.: Accident analysis of sports utility vehicles: human factors from statistical analysis and case studies, JSAE review, 21 (2000) 1, pp. 103-108

[2] Kochar, S., Rastogi, P.: Mode of death shifting from coma to syncope sports utility vehicle (SUV) accidents, Journal of Indian Academy of Forensic Medicine, 29 (2007) 3, pp. 13-17

[3] Margaritis, D., et al.: An analysis of Sport Utility Vehicles involved in road accidents, ESV, Enhanced Safety of Vehicles, Washington DC, 2005.

[4] Carollo, F., Virzi'mariotti, G., Naso, V.: HIC evaluation in teenage cyclist–SUV accident, Recent researches in mechanical and transportation systems, WSEAS international conference ICAT'15., pp. 252-259, 2015.

[5] Acierno, S., et al.: Vehicle mismatch: injury patterns and severity, Accident Analysis & Prevention, 36 (2004) 5, pp. 761-772.

[6] Mandell, S.P., Mack, C.D., Bulger, E.M.: Motor vehicle mismatch: a national perspective, Injury prevention, 16 (201) 5, pp. 309-314

[7] Cobb, J., et al.: Motor vehicle mismatch-related spinal injury, The journal of spinal cord medicine, 28 (2005) 4, pp. 314-319

[8] Desapriya, E., Chipman, M., Joshi, P., Pike, I.: The risk of injury and vehicle damage in vehicle mismatched crashes, International Journal of Injury Control and Safety Promotion, 12 (2005) 3, pp. 191–192, doi:10.1080/15660970420002657 82

[9] Chipman, M. L.: Side Impact CrashesåFactors Affecting Incidence and Severity: Review of the Literature, Traffic injury prevention, 5 (2004) 1, pp. 67-75

[10] Toy, E.L., Hammit, J.K.: Safety impacts of SUVs, vans, and pickup trucks in two-vehicle crashes, Risk Analysis, An International Journal, 23 (2003) 4, pp. 641-650

[11] Eyges, V., Padmanaban, J.: Updated Evaluation of Size and Mass Effects in Front-to-Front Crashes Involving Light Vehicles, SAE International Journal of Passenger Cars - Mechanical Systems

[12] Ross, M.T., Wenzel, T.: Are SUVs Safer Than Cars? An Analysis of Risk by Vehicle Type and Model, TRB 82nd Annual Meeting, Washington, DC, January 15, 2003.

[13] Brewer, J.C., Smith, D.L.: Preliminary evaluation methodology in front-front vehicle compatibility, SAE International Journal of Passenger Cars - Electronic and Electrical Systems

[14] Jenefeldt F., Thomson, R.: A methodology to assess frontal stiffness to improve crash compatibility, International Journal of Crashworthiness, 9 (2004) 5, pp. 475–482, doi:10.1533/ijcr.2004.0303

[15] EuroNCAP press release, Euro NCAP Presents Latest Overhaul of Its Safety Rating,https://www.euroncap.com/en/press-media/press-releases/euro-ncap-presents-latest-overhaul-of-its-safety-rating/, 2020.

[16] Car Importers Association statistics, http://portal.sda-cia.cz/?lang=en, 2020.

[17] Vangi, D.: Structural behavior of the vehicle during the impact. Vehicle Collision Dynamics, 2020, pp.1–27, doi:10.1016/b978-0-12-812750-6.00001-9

POLICY PROPOSAL TO SOLVE ROAD TRAFFIC ACCIDENTS IN PAKISTAN

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Abstract

The road traffic accidents (RTAs) have raised concern globally and become worsen with the passage of time that expedite issues of social exclusion and public health. There are approximately 1.35 million people involved annually in road crashes and 3,700 people died on daily basis. A ratio of happening an accident has found greater in developing countries due to govern of their socio-economic factors. It would contribute in long-lasting cost of pain and sufferings at micro to macro level at large. Pakistan has been experiencing the same with an annual trend of increase in RTAs. There are many demographic factors involved particular to urbanization, and willingness to pay etc. where policies had contributed a major role. A loss of 30 thousand lives on annual basis has placed Pakistan at 67th position on global ranking of having higher percentage of RTAs. This number could expect to be doubled with the functional operation of road projects associated with China Pakistan Economic Corridor (CPEC). Currently, the main challenge is to sustain the growing number of RTAs by promoting mitigation measures that aimed to move ahead on sustainable and balanced development. An adequate response to address these challenges will require best available scientific knowledge and constant re-evaluation of the developments. It will fulfil the scope of this study to identify frequent causes and propose strategies for traffic calming measures in light of those findings, and also to make ensure that it would respond to emerging needs. A comparative investigation into the literature has assisted to identify key issues for occurrence of road accident fatalities (RAFs) and severe injuries. It has highlighted and recommended those gap areas either in policy or strategy domain that need to consider in dealing with RTAs mitigation tactics (e.g., licencing system upgradation, enforcing safety laws, and etc.).

Keywords: road safety issues, safety policy proposal, Pakistan, road traffic accidents

1 Introduction

Road traffic accidents are one of the leading public health issues in all continents of the world. The rate of long-term disability, severe injuries and mortality highly depends upon the injuries caused by RTAs. According to World Health Organization (WHO), the number of deaths because of RTAs increases from 1.15 million in 2000 to 1.35 million in 2016 and the death rate has reached to 18.2 per 1,00,000 population. About 3,700 individuals lost their lives on daily basis and the majority of people become victims of long-lasting treatment. Currently RTA is marked the 8th leading cause of mortality for all ages of people and 1st leading cause of mortality for young and adult (age 5-29 years) [1]. If proper immediate actions are not taken to control RTAs then it will be the 5th leading cause of mortality for all ages of people in the year of 2030 [2]. Low and middle-income countries contribute about 85 % of

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the total road traffic injuries and fatalities. The rate of fatalities linked with RTAs is drop by 27 % in high income countries while rise by 83 % in low and middle income countries [3]. In low and middle income countries, the budget of road traffic injuries (RTIs) is predicted to be more than US$100 billion per year which is five percent of the gross national product and globally estimated to be three percent of Gross Domestic Product (GDP) [4]. WHO estimated 59 % of the total vehicles belongs to middle income countries where population is 76 % and the road traffic death is 80 % of the world [1]. Thus, determining the root causes of RTAs both at countrywide and worldwide is a key concern for policymakers and researchers [5]. Safe traveling is the major concern of national transportation system that eventually related to the nation development so the rectification of this burning issue is the need of the day [6].
In developing countries with jointed family's system like Pakistan where one or two bread earning member/s commonly play a significant role in supervising their families financially. These bread earning members of the families go outside for earning and they are mostly expose to disabilities, injuries and even death because of RTAs. RTAs disproportionality affects the lower class of Pakistani families and push them into further poverty due to the loss of their earning member/s. Therefore, RTAs nowadays a public health problem as well as economic issue in term of medical and vehicles damaged [7].Pakistan is a developing nation where roads carry a wide range of vehicles from bicycles to heavy goods vehicles without any separation. That is why, RTAs are the major cause of economic loss, disability and mortality. Younger population which play a major role in the growth of socio-economic are mostly vulnerable to RTAs among pedestrians [6]. In 2004 the total number of RAFs were 7,000 while the number of fatalities increases to 26,751 in 2009 [8]. In a recent survey, WHO predicted the loss of 30,310 lives annually. This shows the death rate due to RTA is about 20 per 100,000 population and making Pakistan rank 67th higher percentages of RTAs in the world [9].
In Pakistan, the growth rate of motor vehicle is growing at much faster rate as compared to infrastructure of roads and economy. The population of motor vehicle has jumped from 5.3 million to 11 million in the year of 2002-12 respectively [10]. The increment observed in the last decade for different types of vehicles are about 30 %, 45 %, 150 % and 110 % for buses, trucks, passenger cars and motorcycles respectively [11]. A rapid increase has been observed in the RTAs with the increase in population and motorization. The burning issue which is facing nowadays is RTAs and RTFs that cost the economy about Rs 100 billion per year [12]. A national health survey was conducted and their results showed that injuries occur as a result of RTAs are greater in number than any other source. This emerged challenge require speedy attention and remedial action by policy makers [13].

2 Key issues of road traffic crashes

2.1 Driver behaviour

Numerous researches have been carried out to find the major factors which contributes to the occurrence of RTAs. It shows that in total of five accidents, three are mostly related to driver behaviour. It is also predicted that 95 % of the total RTAs are due to driver behaviour factors [14]. A wide range of factors influences the performance of drivers including mood, distraction and fatigue etc. but distraction is recognised as utmost critical factor founded in RTAs [15]. Driver experience and age results different outcomes in interaction between roadside advertising and drivers. Several studies have founded that drivers (aged greater than 65 years) are greatly distracted by roadside advertising signs than younger drivers [16]. A wide range of research declared that the use of mobile phone during driving also influence driver behaviour. Drivers being busy in using mobile phone or in-vehicle communication delay drivers response by 15 % and are not capable to quickly respond to a traffic light turning red, brake lights of a vehicle ahead, important stop and yield sign [17]. Among these, more

emotional/sensitive discussions on mobile phones lead to higher possibilities of RTAs in a controlled environmental condition than normal discussion or no mobile phone discussion at all [18].
Young drivers are over represented in RTAs showing their higher risk of vulnerability to crashes [19]. Though, it is universally accepted fact that young drivers are at higher risk than old drivers in a simulated environment. The rate of fatal and non-fatal crashes is higher in younger drivers (age 16-20 years) and the rate is declining abruptly with increasing the age of drivers [20]. Younger drivers mostly involved in RTAs is the combination of both young age and inexperience [21]. The decrease in the rate of RTAs related with age can be better explained by changes in particular attitudes and behaviour of risk taking. Among young drivers, male drivers are mostly involved in RTAs due to violation of road and traffic laws and their aggressive nature [10]. The results of analysing risk attitudes showed that male young drivers feel comparatively immune to the hazards than older drivers and they overestimate their own competence level compared to females young drivers [22]. A study reported a total of 1296 road accidents and after conducting interviews from police officers concluded that risk taking attitude of young drivers leads to higher percentages of RTAs instead of poor driving skills [23].

2.2 Accident data management

A number of research studies have determined that there is a high level of underreporting in accident data available with police sources compared with health sectors [24]. In developing countries like Pakistan the official sources only reported 56 % of fatal and 4 % of severe RTAs [25]. A lot of inconsistencies in the injury and fatality figures exist by comparing the estimates with external organizations. For example, In the period of 2009-2010 Federal Bureau of Statistics (FBS) Pakistan stated 11,173 injuries and 5,280 deaths due to RTAs. One the other hand, WHO 2013 reported 41,494 deaths for the same period of time [26]. The same inconsistencies are further explained in Table 1, where WHO reported higher number of fatalities in 2013 than 2010 while comparing with FBS reported RTFs. The difference in RTF between FBS and WHO is 17.55 % and 49.13 % in 2010 and 2013 respectively. FBS and WHO reporting rate tend to decreased by about 10.2 % and 16.9 % respectively, despite with the increase in registered vehicles by 22 % and population density by 9.54 % for the year of 2010 to 2013.

Table 1 Road fatalities, pop density and registered vehicles from 2010 to 2013

Fatalities					
Source	**2010**	**Difference b/w WHO & FBS**	**2013**	**Difference b/w WHO & FBS**	**Variation [%]**
FBS Pakistan	4280	17.55%	3884	49.13%	-10.2
WHO (2015)	30131		25781		-16.9
Population Density (Persons/square.km)					
WHO (2015)	208		241		9.54
Registered Vehicles (Thousands)					
WHO (2015)	10443		13388		22

2.3 Vehicle factors

Vehicle factors such as defects in tyres, brake and gear etc. are other contributory factors which leads to RTAs mostly in developing countries. Those vehicles having some type of technical or mechanical defects are mostly vulnerable to road traffic crashes [26]. Defects in brakes and tyres of vehicles occurred mainly due to lack of timely maintenance. Although the design and standard of vehicles also matters a lot which needs improvement with time in developing countries [27]. Defective vehicle is also one of the primary cause of RTAs. Different studies were carried out in developing nations and reported about 8.5 to 14 % of the accidents is directly related to vehicles defects [28]. According to the European Commission, 50 % of the fatalities and injuries can be avoided by fitting crash safety system in all vehicles [29]. High income countries have standard regulations of safety for all vehicles such as airbags and seat belts etc. Conversely, lack of such standard regulations of safety in middle and low-income countries which leads to serious injuries and fatalities. Due to these reasons the rate of fatal RTAs is higher in middle and low income countries [2].

In Pakistan the manufacturing standards of vehicles set by Motor Vehicle Regulation 1969 and Motor Vehicle Ordinance 1965 but still the locally manufactured motor vehicles have poor structural standards. Most of the vehicles are lacking in safety technologies such as side impact protection, advance braking systems, crumple zones, child restraint fixtures, seat safety belts, airbags and electronic stability control. The unsafe modification and overloading of heavy vehicles are very common, which increases the probability of RTAs. In the last decade, the number of locally manufactured cars increases four to five times faster as compared to the population growth. These locally cars manufacturing companies work under CKD (Complete Knock Down kits) license given by parent companies. There are 3 major manufacture companies which produce number of cars from mid-sized 1600/1800cc sedans to 660cc/800cc hatchbacks. These local manufactured cars do not meet the same structural standards and safety as compare to their parent company/ international manufacturing company. However, if they are importing from the parent company then the standard does not meet that is exported to America, Europe and East Asia from parent company.

2.4 Motor cycle helmet

Head injury is one of the chief and predominant cause of death in RTAs [30]. Numerous studies have founded that major causes of mortality due to RTAs is multiple fractures, head trauma and bleeding which were more dominant causes in pedestrians, motorbike drivers or pillion riders and car drivers respectively [31]. According to a survey carried out in Pakistan stated that among the motorcycle accidents 10.2 % were of severe nature, 35.9 % of face injuries and 41.5 % suffered head injuries [32]. The possibility of head injury is increasing by three to four times for motorcycle riders who do not use helmet [33]. In 2013, a research study reported that there were 1130 fatalities and the involvement of motorcycle drivers and pillion riders was about 51 % [34]. It is noticeable in Pakistan that driver's pillion riders mostly do not wear helmets which makes them more vulnerable to head injuries. Among the motorcycle accidents the reported injuries indicated that the percentage of pillion riders were more than drivers [35]. The percentage of motorcycles are about 75 % of total registered vehicles. The National Highway Safety Ordinance (NHSO) and Motor Vehicle Regulation (MVR) 1969 mandate helmet wearing by all drivers and pillion riders but there is no specified technical standard for helmet. A helmet wearing survey was conducted by Centre for Communication Programs of Pakistan in September 2018 among six different cities of Pakistan (Quetta, Rawalpindi, Karachi, Peshawar, Lahore and Islamabad) for both drivers and pillion riders and their summary is shown in Table 2 [36]. Thus, it is concluded that in Pakistan 66 % (61 % +5 %) of the motorcycle drivers and 97 % (96 % + 1 %) of the pillion riders do not wear helmets.

Table 2 Percentage of driver and pillion rider wearing helmet

	Motorcycle Driver [%]	Pillion Rider [%]
Correctly wear and strap their helmet	06	0
Wear but do not strap their helmet	28	03
Carry but do not use a helmet	05	01
Do not use a helmet	61	96

2.5 Law enforcements

Many countries have enacted strict laws to modify risk taking behaviour of possible accident victims. However, the effect of these laws is not uniform in all the countries [37]. This problem is very common in middle and low-income countries and greater than 85 % of our legislation system (if it exist) is imperfect and poor implementation in many countries of the world. Numerous research studies have stated that reforms in road traffic legislation can help to avoid the occurrence of RTAs and its consequences [38]. Despite the positive benefits resulting from traffic enforcement efforts and community support, fewer resources are being allocated to traffic safety enforcement. In Pakistan political, financial, and cultural factors may affect the level of engagement in traffic safety enforcement by these agencies. For example, leaders of such agencies that are appointed or elected may feel political pressure not to enforce laws that are perceived to be unpopular amongst voters, or changes in the workforce like a reduction in staff through budget cuts or retirement may result in changes in the level of engagement with traffic safety. Traffic safety enforcement may be viewed as a lower priority than criminal enforcement [39]. There are too many traffic regulations and laws, which are enforced by different government agencies. When organizations take different approaches to the law, its interpretation varies and it becomes fragmented and incoherent. Consequently, traffic law appears to be enforced ineffectively due to legal issues lacking clarity. The problem of poor resource sharing occur when there is lack of celerity in distribution of responsibilities among agencies [40]. Another research study demonstrate the same, that mostly in Pakistan each agency makes its own plans individually and having no coordination with other agencies [41]. The performance of any law enforcement agency is gauged by number of plenty permits issued each day or number of plenty permits against a particular violation instead of reduction in violation or improvement in road discipline. Comparison in terms of quantity of plenty permits in a month of existing year is made with the permits issued in the month of preceding year. The officers are rewarded on the basis of quantity of traffic challans rather than quality of enforcement of traffic law. Resultantly, an unending and blind race of issuance of traffic challans is started among the officers to grab awards. Quality of enforcement of traffic laws may be evaluated by any unprejudiced third party for improvement [42].

3 Conclusion

The findings can be summarized as follows:
1. One of the primary factor toward safe driving is attitude, which effect the driving behaviour in Pakistan. Issues such as aggressiveness and lack of self-assessment among drivers emerged as consistent attitudinal problems.
2. Crash control can save more lives than saving injuries after the crash and the key to control crash is changing the road user's behaviour.
3. An integrated road safety data system needs to be established at the national level to ensure that road traffic fatality data will be more accurately reported to policy makers. This data will be used by all stakeholders to develop evidence-based traffic injury reporting.

4. True and effective maintenance of both public and private vehicle should be ensured and inspired by the government.
5. Having and maintaining good materials for vehicles should be monitored by strict government rules to confirm desired standard of vehicle
6. Helmet reduce the possibility of occurring severe head injury. To increase the rate of wearing helmet should be the utmost aim of policymakers for motorcyclists.
7. Strict monitoring by law enforcement agencies is mandatory and violations of traffic rules & regulations should be strictly punished on the spot to control crashes.
8. Laws should be more strengthen for all traffic offenses and training program with an evaluation mechanism should be established for traffic police officers.

References

[1] Global status report on road safety, World Health Organization, 2018.

[2] Global status report on road safety: time for action, World Health Organization, 2009.

[3] Chalya, P.L., et al.: Injury characteristics and outcome of road traffic crash victims at Bugando Medical Centre in Northwestern Tanzania, 6 (2012) 1

[4] Law, T.H.: The effects of political governance, policy measures and economic growth on the Kuznets relationship in motor vehicle crash deaths, Imperial College London, 2009.

[5] Babcock, M.W., Gayle, P.G.: State variation in the determinants of motor vehicle fatalities, 2009.

[6] Khan, A.M., Tehreem, A.J.J.: Causes of road accidents in Pakistan, 1 (2012) 1, pp. 22-29

[7] Hammad, H. M.: Environmental factors affecting the frequency of road traffic accidents: a case study of sub-urban area of Pakistan, 26 (2019) 12, pp. 11674-11685

[8] Global status report on road safety: supporting a decade of action: summary, 2013, World Health Organization, 2013.

[9] Blackburn, C.C., Lenze, J., Paul, E.: Syrian Forced Migration and Public Health in the European Union, Rowman & Littlefield, 2019.

[10] Simon, F., Corbett, C.J.E.: Road traffic offending, stress, age, and accident history among male and female drivers, 39 (1996) 5, pp. 757-780

[11] Jonah, B.A., Dawson, N.E.: Drugs, and Driving, Youth and risk: age differences in risky driving, risk perception, and risk utility, 1987.

[12] Ministry of Communications, National Road Safety Plan 2007-2012. 2007.

[13] Fatmi, Z., et al.: Incidence, patterns and severity of reported unintentional injuries in Pakistan for persons five years and older: results of the National Health Survey of Pakistan, 7 (2007) 1

[14] Petridou, E., Moustaki, M.J.E.: Human factors in the causation of road traffic crashes, 16 (2000) 9, pp. 819-826

[15] Rhodes, N., et al.: Risky driving among young male drivers: The effects of mood and passengers, 28 (2015), pp. 65-76

[16] Edquist, J., et al.: Effects of advertising billboards during simulated driving, 42 (2011) 4, pp. 619-626

[17] Bener, A., Bener, O.F.: Mobile phone use while driving and risk of road traffic injury: applying the Lorenz Curve and associated Gini Index, 13 (2007), pp. 77-82

[18] Dula, C.S., et al.: Differing types of cellular phone conversations and dangerous driving, 43 (2011) 1, pp. 187-193

[19] Starkey, N.J., Isler, R.B.: The role of executive function, personality and attitudes to risks in explaining self-reported driving behaviour in adolescent and adult male drivers, 38 (2016), pp. 127-136

[20] Ferguson, S.A., Teoh, E. R., McCartt, A.T.: Progress in teenage crash risk during the last decade, 38 (2007) 2, pp. 137-145

[21] Mayhew, D. R.: Driver education and graduated licensing in North America: Past, present, and future, 38 (2007) 2, pp. 229-235

[22] Glendon, A.I., et al.: Age and gender differences in perceived accident likelihood and driver competences, 16 (1996) 6, pp. 755-762

[23] Clarke, D.D., et al.: Voluntary risk taking and skill deficits in young driver accidents in the UK, 37 (2005) 3, pp. 523-529

[24] Periyasamy, N., et al: Under reporting of road traffic injuries in the district of Kandy, Sri Lanka, 3 (2013) 11

[25] Razzak, J.A., Luby, S.P.: Estimating deaths and injuries due to road traffic accidents in Karachi, Pakistan, through the capture-recapture method, 27 (1998) 5, pp. 866-870

[26] Kayani, A., Fleiter, J.J., King, M.J.J.: Underreporting of road crashes in Pakistan and the role of fate, 15 (2014) 1, pp. 34-39

[27] Global status report on road safety, World Health Organization, 2015.

[28] Chowdhury, A.A., Fahim, H.K.: Globalization, A Legal Approach to Prevent Road Accident in Bangladesh: Drawbacks and Recommendations, 81 (2019)

[29] Thomas, P., et al.: The new European road safety observatory–SafetyNet, 2006.

[30] Bhalla, K., et al.: Building national estimates of the burden of road traffic injuries in developing countries from all available data sources: Iran, 15 (2009) 3, pp. 150-156

[31] Heydari, S., et al.: Epidemiological characteristics of fatal traffic accidents in Fars province, Iran: a community-based survey, 127 (2013) 8, pp. 704-709

[32] Khan, U.R., et al.: Clothing-related motorcycle injuries in Pakistan: findings from a surveillance study, 22 (2015) 4, pp. 308-313

[33] Ferrando, J., et al.: Impact of a helmet law on two wheel motor vehicle crash mortality in a southern European urban area, 6 (2000) 3, pp. 184-188

[34] Jooma, R., Ali Shaikh, M.J.: Epidemiology of Karachi road traffic crash mortality in 2013, 65 (2015) 1

[35] Bhatti, J.A., et al.: Influence of an enforcement campaign on seat-belt and helmet wearing, Karachi-Hala highway, Pakistan, Annals of Advances in Automotive Medicine/Annual Scientific Conference, Association for the Advancement of Automotive Medicine, 2011.

[36] National Road Safety Strategy 2018-2030, https://www.thenews.com.pk/print/394274-first-national-road-safety-strategy-launched, 02.02.2020.

[37] Sharma, V., Verma, V.J.: The role of law in preventing accidents, Journal of Bone and Joint Diseases, 32 (2017) 2, pp. 1-4

[38] Elvik, R., et al.: The handbook of road safety, Emerald Group Publishing Limited, 2009.

[39] Otto, J., et al.: Understanding Law Enforcement Attitudes and Beliefs about Traffic Safety, Montana, Dept. of Transportation, Research Programs, 2019.

[40] Batool, Z., et al.: Road safety issues in Pakistan: a case study of Lahore, 35 (2012) 1, pp. 31-48

[41] Hisam, Z.J.E., Weekly, P: Collective care arrangements in the informal labour market: road transport workers in Pakistan, 2006.

[42] Klair, A.A.: Enforcement issues of traffic laws in Pakistan 2018, https://www.thenews.com.pk/print/329962-enforcement-issues-of-traffic-laws-in-pakistan, 02.02.2020.

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A STUDY OF ACTUAL AND POSTED SPEED ON SOME NIGERIAN ROADS

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Abstract

There is a massive problem of overspeeding on Nigerian highways. This problem is exacerbated by the absence of posted speed signs which though installed after reconstruction/rehabilitation are usually vandalised and not replaced. Where posted speed limits exist, they are largely observed in the breach as large pluralities of motorists exceed these posted speed limits. Using speed data from automatic traffic classifiers installed at locations on road sections on the federal highway network, this study is aimed at assessing the extent of the disparity between posted and actual speed with a view to identify areas of prevalent overspeeding, evaluate the extent of the problem, determine the trend of the problem, and better target enforcement activities on areas of high rates of overspeeding. The results from the study show that there is a prevalence of overspeeding in the highway sections studied with the maximum speed in all Sections in the range of 150 to 160 km/h and up to 20 percent of vehicles traveling at speeds above 100 km/h.

Keywords: posted speed limits, actual speed, 85th percentile speed, Nigeria roads, overspeeding/speeding

1 Introduction

Overspeeding, traveling above the posted speed limit, is the predominant human causative factor of road traffic crashes [1]. Overspeeding in Nigeria is so prevalent that it is considered in some quarters to have attained epidemic proportions. Overspeeding and all speed related factors accounted for about 40 to 50 % and 60 to 70 % of road traffic crashes in Nigeria in the years 2017 to 2019 [2, 3, 4]. However, extent of the problem, having not been studied, is not clearly understood

The relationship between crash severity and speed, that is, crash severity increases geometrically as speed increases is well established by the laws of physics [1, 5, 6, 7]. The impact of speed on crash occurrence is mired in controversy. This is primarily due to the fact that the precise relationship between speed and crash occurrence can be obscured by the variety of road design and operating characteristics [5, 7, 8]. While this relationship has been a topic of numerous studies that produced conflicting results [7, 9, 10], a study on the topic [8] which is considered to be the most statistically robust [7] indicates definitely that all other factors being equal, increased speed increase crash occurrence.

Generally, drivers travel at speeds they think is reasonable and safe for a given condition [6, 8]. To forestall a free-for-all where each driver chooses their own speeds and taking cognizance that some drivers will not be reasonable in their choice, speed limits are posted on highways to ensure they are safe to all drivers. Posted speed limits are typically determined using the following factors [6, 7, 10, 11]: 85th percentile speed determination, highway design,

accident history, traffic volume, road type and surface. Drivers expect posted speeds to be reasonable, unreasonable posted speeds get little or no consideration from drivers.
Given the poor condition of Nigerian highways, the near total absence of posted signs in all but newly constructed or rehabilitated highways, and the lack of speed enforcement activities (electronic speed enforcement is hardly utilized), drivers are left to their own devices regarding speed. The combination of these three factors has led to the current high rates of overspeeding that is considered to have reached epidemic proportions in some quarters. It also makes obvious the need to deploy the full gamut of safety engineering measures, particularly speed management measures to forestall the attendant effect of road traffic crashes which result in loss of lives and loss to the economy.
This paper reports on a study that generally explored the issue of actual speeds on the Federal Highways in Nigeria exceeding the posted speed limits. The study primarily aimed at assessing the extent of the disparity between posted and actual speed with a view to evaluate the extent of the problem, identify areas of prevalent overspeeding, determine the trend of the problem, and better target enforcement activities on areas of high rates of overspeeding.

2 Methodology

2.1 Data

Data for this study were collected with the aid of pneumatic tube-type automatic traffic classifiers (ATC) installed at a number of stations in each road section during 7-day traffic studies programs on the following five road sections in the Federal Highway network rehabilitated/reconstructed under the Federal Roads Development Project and the Nigeria-Cameroon Multinational Highway and Transport Facilitation Programme. The details of the road sections and the number of data collection stations are shown in Table 1.
The ATCs were installed at locations within road segments where: a) Most traffic travel at constant speed across the tubes (avoiding sites where vehicles are accelerating or decelerating due to bends, steep inclines, traffic signals, or intersections), b) Traffic cross perpendicular to the tubes (avoiding locations will stop over the tubes or turn across the tubes), and c) The posted speed limit is 100 kilometers per hour (km/h), which is the maximum posted speed on Nigerian roads. pneumatic tube-type ATCs have been used by other investigators [7, 12] to study the speed of vehicles on highways.

Table 1 Details of road sections

S/N	Highway Section	Route No.	Length [km]	No. of data stations
1	Enugu-Abakaliki	A 343	77	3
2	Abakaliki-Mbok	A 343	84	4
3	Mbok-Ikom	A 4	52	3
4	Ikom-Mfum	A 4.2	22	2
5	Akure-Ilesha	A 122	76	4

2.2 Data analysis

Binned data (1-hour intervals) was primarily used for the data analyses. Binned data have been used in speed studies by other investigators [10]. The equipment used for data collection does not provide individual raw speed data. Data from the collection stations in each highway section were combined and analysed. Actual speeds were compared to the posted limits. The statistical parameters of the actual speed data, including count, minimum, maximum, mean, median, variance, 85th percentile speed, and 95th percentile speed, were determined and used for the comparison with the posted speed limit. Also determined was the number/percentage of vehicles with speed exceeding 100 km/h. The 85th percentile speeds for the respective sections for each data year determined were used to assess the difference between the 85th percentile speed and the posted limits (100 km/h).

3 Results and discussion

The summary of the results of the analyses carried out on the actual speed data is presented in Table 2. Table 2 shows the enormity of the overspeeding problem. Generally, the 85th percentile speed for all Highway Sections investigated except the Enugu-Abakaliki Section was below the posted speed limit of 100 km/h. The 85th percentile speed is usually adopted by most jurisdictions in the determination of posted speeds [7, 12]. The adoption of the concept of the 85th percentile speed is based on the theory that the vast majority of vehicle operators have the following characteristics [12, 13]: they are reasonable and prudent, they do not want to be involved in a crash, and they desire to reach their destination in the shortest possible time. In the case where the 85th percentile speed is less than posted speed, like in the Abakaliki-Mbok, Mbok-Ikom, Ikom-Mfum, and Akure-Ilesha sections, the maximum speed is almost double the 85th percentile speed. For the Enugu-Abakaliki Section, it is about 50 % of the 85th percentile speed.

The respective difference between the minimum and maximum speed, mean speed and the 85th percentile speed, and the 85th percentile speed and the maximum speed are quite large. Large range of difference in speed, speed dispersion, is known to contribute to crashes more than speeding on its own [7, 8, 13, 14].

In terms of overspeeding, the percentage above the posted speed of 100 km/h ranged from above 1 % to about 19.7 %. The higher percentages (10-20 %) occur at the Enugu-Abakaliki Section. The Enugu-Abakaliki Section links two state capitals and has large traffic volumes. It should be noted that at the times of this study in 2014, the study highway sections were in very good condition, most of the highway sections had been reconstructed or rehabilitated within the past one to two years. The road sections being in good condition must have contributed to the high rate of overspeeding.

The mid-range percentage of vehicles traveling at speeds above 100 km/h (5-10 %) occur at the Abakaliki-Mbok, (at the early years), Mbok-Ikom, and Akure-Ilesha Sections. In the mid-range excessive speed region, the percentage of vehicles above 100 km/h decreases with increase in years. This is as a result of the deteriorating road condition particularly on the Abakaliki-Mbok Section. It would appear that drivers had chosen the appropriate speed for the road conditions as reported by other researchers that have conducted speed studies [10].

The lower excessive speed range (1-5 %) occurred mainly at the Ikom-Mfum Section. A high proportion of the length of this Section is built-up and it has quite a number of security check points which almost prevents motorists from overspeeding. The Ikom-Mfum Section leads to the Nigeria/Cameroon border at Mfum/Ekok.

Table 2 Summary of results

Year	Min [km/h]	Max [km/h]	Mean [km/h]	Median [km/h]	VAR	Count	85 % [km/h]	95 % [km/h]	>100 km/h
Enugu - Abakaliki									
2014	10.0	159.6	76.2	74.9	535.2	123574	101.5	114.1	20711 (16.76%)
2015	10.0	159.1	77.0	76.0	612.0	131056	104.0	117.0	25818 (19.70%)
2016	10.0	160	79.0	78.1	427.2	172205	100.8	113.4	27900 (16.15%)
Abakaliki - Mbok									
2014	10.1	158.6	62.9	60.8	497.6	75429	87.1	102.6	4730 (6.27%)
2015	10.0	122.1	63.5	63.0	391.9	56650	85	99.4	4040 (7.13%)
2016	10.0	155.5	67.3	64.1	512.82	69238	92.9	107.6	6707 (9.69%)
2018	10.0	144.3	57.8	55.4	400.2	66208	79.6	94	1816 (2.74%)
2019	10.0	156.4	55.0	53.6	329.8	76722	73.4	87.5	1216 (1.58%)
Mbok - Ikom									
2014	10.0	158.9	61.2	58.0	467.1	131308	83.5	102.6	7841 (5.97%)
2015	10.0	159.8	59.3	55.8	492.3	127746	82.5	102.2	7450 (5.83%)
2016	10.1	159.2	65.9	61.6	543.75	119476	92.5	108.0	11298 (9.46%)
Ikom - Mfum									
2014	10.1	157.0	58.2	57.6	296.1	49225	74.2	87.8	891 (1.81%)
2015	10.0	158.7	58.5	58.0	323.8	54760	76.0	89.3	1050 (1.92%)
2016	10.1	152.5	59.6	58.3	326.3	75052	77.4	92.2	1812 (2.41%)
2018	10.0	157.1	56.3	55.4	304.8	73823	73.1	86.8	1050 (1.42%)
2019	10.0	155.8	56.4	56.2	282.3	79030	72.7	84.6	856 (1.08%)
Akure - Ilesha									
2016	10.0	159.2	58.8	55.1	415.9	190392	78.1	100.8	10026 (5.27%)

VAR = Variance; km/h = kilometer per hour; Count = Total number of vehicles

4 Conclusions

A speed study was conducted on some five highway sections on the Nigerian Federal Highway network with the aim to identify areas of prevalent over speeding, evaluate the extent of the problem and determine the trend of the problem and better target enforcement of activities on areas of high rates at over speeding. Speed data was collected at a total of 16 stations and analyzed. Based on the analyses conducted in this study, the following conclusions can be reached:

- There is a prevalence of overspeeding (travelling above the posted speed of 100 km/h) in the five highway sections investigated.
- Maximum speeds of 150-160 km/h was registered at all five sections studied.
- The 85th percentile speed for the Enugu-Abakaliki Section was above the posted speed of 100 km/h.
- The 85th percentile speed for the other four Sections Abakaliki-Mbok, Mbok-Ikom, Ikom-Mfum, and Akure-Ilesha were generally below the posted speed of 100 km/h.
- The ranges for the percentage of vehicles travelling at speeds above 100 km/h on the various sections are as follows: Enugu-Abakaliki (16-20 %), Abakaliki-Mbok (1.5-10 %), Mbok-Ikom (5-10 %), Ikom-Mfum (1-2 %), and Akure-Ilesha (5 %).
- There is a tendency for the percentage above 100kmh to reduce with reductions in the quality of the highway with time as seen with highway section where there are data for 5 years.
- There is a need for a larger study, a network-wide study involving highways sections from all parts of the network to have a proper grip on the excessive speed problem/issue.

References

[1] Elvik, R.: Speed limits, enforcement, and health consequences, Annual Review of Public Health, 33 (2012), pp. 225-38, doi: 10.1146/annurev-publhealth-031811-124634

[2] Federal Road Safety Corps (FRSC) Annual Report 2017., https://frsc.gov.ng/wp-content/uploads/2018/09/AnnualReport2017.pdf, 05.03.2021.

[3] Federal Road Safety Corps (FRSC) Annual Report 2018., https://frsc.gov.ng/wp-content/uploads/2021/01/FRSC-2018-annual-report.pdf, 05.03.2021.

[4] Federal Road Safety Corps (FRSC) Annual Report 2019., https://frsc.gov.ng/wp-content/uploads/2020/12/2019-Annual-Report2.pdf, 05.03.2021.

[5] Fitzpatrick, K., Carlson, P., Brewer, M. A., Wooldridge, M. D., Miaou, S.P.: Design Speed, Operating Speed, and Posted Speed Practices, NCHRP REPORT 504, Washington, DC, Transportation Research Board/National Academies, 2003.

[6] Global Road Safety Partnership (GRSP): Speed Management: A Road Safety Manual for Decision-Makers and Practitioners. Geneva: GRSP, World Bank, World Health Organization (WHO), FIA Foundation. http://whqlibdoc.who.int/publications/2008/9782940395040_eng.pdf., 2008.

[7] Forbes, G. J., Gardner, T., McGee, H., Srinivasan, R.: Methods and Practices for Setting Speed Limits: An Informational Report, Report No: FHWA-SA-12-004. Washington, DC, 2012.

[8] Elvik, R. Christensen, P., Amundsen, A.: Speed and Road Accidents: An Evaluation of the Power Model. TOI Report 740/2004, The Institute of Transport Economics (TOI), Norwegian Center for Transport Research, Oslo, Norway, 2004.

[9] Choi, J., Tay, R., Kim, S.: Effects of changing highway design speed, Journal of Advanced Transportation 47 (2013), pp. 239–246, doi: 10.1002/atr.171

[10] Skszek, S.: Actual Speeds on the Roads Compared to the Posted Limits, Report No. FHWA-AZ-04-551, Phoenix, AZ: ArizonaDOT/FHWA, 2004.

[11] Krammes, R. A., Fitzpatrick, K., Blaschke, J. D., Fambro, D. B.: Speed: Understanding Design, operating, and posted speed, Report No. 1465-1 (1996). Washington, DC, FHWA/Texas, DOT., 1996.

[12] Bagdade, J., Nabors, D., McGee, H., Miller, R., Retting, R.: Speed Management: A Manual for Local Rural Road Owners, Report No.: FHWA-SA-12-027 (2012), Washington, DC

[13] Stuster, J. W., Coffman, Z., Warren, D.: Synthesis of Safety Research Related to Speed and Speed Management, Publication No. FHWA-RD-98-154. US Department of Transportation, Federal Highway Administration, Turner-Fairbank Highway Research Center, 1988.

[14] Jiang, Z., Jadaan, K., Ouyang, Y.: Speed Harmonization: Design Speed Vs. Operating Speed, Report No.: FHWA-ICT-16-019 (2016), Springfield, IL: Illinois DOT/FHWA

MULTICRITERIA EVALUATION OF DANGEROUS SECTIONS FROM THE OCCURRENCE OF WILDLIFE ON STATE ROADS OF LIKA-SENJ COUNTY USING THE AHP METHOD

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Abstract

A trend of increasing traffic accidents involving vehicles has been observed, which requires proposed measures to prevent the occurrence of wildlife on risky sections of roads. Before implementing the relevant measures, it is necessary to rank the risk sections on the roads from the occurrence of wildlife. Roads pass through the natural habitats of wildlife, so with each kilometer of newly built roads and with each newly registered vehicle, the probability of a vehicle colliding with wildlife increases. The problem of increasing the number of encounters with game is expressed both at the global level and in the territory of the Republic of Croatia. Traffic accidents related to vehicle crashes into wildlife are a major problem of road safety on the road network. The paper presents an analytical hierarchical process of AHP methods, multi-criteria evaluation on the example of ranking dangerous sections from the occurrence of large game on state roads of Lika-Senj County. The AHP method is one of the most well-known methods of multicriteria analysis, which consists of goal, criteria and alternatives. The ranking of dangerous sections from the occurrence of large game on the state roads of Lika-Senj County by the AHP method includes qualitative and quantitative criteria. The AHP method of multi-criteria evaluation of dangerous stocks from the appearance of large game was presented with the software tool Expert Choice. The obtained results rank the dangerous sections from the occurrence of large game and define the priorities of their rehabilitation from the competent authorities.

Keywords: vehicle collision with game, AHP method, Lika-Senj County, traffic safety

1 Introduction

By monitoring and analyzing data on traffic accidents involving large game on the roads, from the point of view of traffic safety, it is possible to determine dangerous sections from the occurrence of game on the roads. The aim of this paper is to show that the quality of decision-making on the ranking of dangerous sections from the occurrence of game can be improved by multi-criteria decision-making, using the AHP method, on the example of a vehicle collision with large game on the state roads of Lika-Senj County. The paper deals with traffic accidents involving large animals on state roads of Lika-Senj County in the period 2012-2016, obtained from the Ministry of the Interior, Lika-Senj Police Administration [1].
On sections that are dangerous for road safety due to the possibility of wildlife on them, it is possible to increase traffic safety by applying certain measures of regular road maintenance. So far, apart from the installation of traffic signs (game on the road), no measures have been

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taken to prevent the occurrence of game on the roads, nor have potential dangerous places from the occurrence of game on the roads been ranked. The paper ranks dangerous sections from the occurrence of wildlife on the state roads of Lika-Senj County, so that they can be rehabilitated through measures of regular or extraordinary road maintenance, all in order to increase road safety. The following criteria were applied: number of large game encounters in the period 2012-2016, section length, number of large game encounters per 100 kilometers and number of large game encounters per year, and they significantly influenced the ranking of variants.

2 Background AHP methods

Multicriteria decision making is a set of methods that allow the simultaneous use of several different criteria in order to select the optimal variant from a set of variants with respect to a given function of the goal [2].Due to the complexity of the transport system, the approach of evaluating the solution of traffic problems by applying several criteria is important. The application of several criteria is used in the evaluation of projects by multi-criteria decision-making methods. One of the most commonly used methods for evaluating projects in transport is the Analytical Hierarchical Process (AHP) method. The Analytical Hierarchical Process was founded by Thomas Saaty in the 1970s with the aim of solving complex decision-making problems, when there are a large number of decision-makers as well as criteria. It is one of the best known, most proven and most frequently used methods of decision making, ie methods for multicriteria analysis. Its main advantage is manifested in the ability to adapt the decision maker in terms of the number of attributes, or criteria and variants that are decided at the same time, and which can be described both quantitatively and qualitatively. Therefore, the AHP method allows for flexibility in the decision-making process and helps decision-makers to set priorities, and to make the best decision taking into account both the qualitative and quantitative aspects of the decision.

The application of AHP is significant in large investment projects that require significant capital investment, and have great social significance (eg investment projects in transport infrastructure), but is also important in the evaluation of other solutions to transport problems. Based on the research conducted so far, it can be concluded that the application of AHP in solving problems in the field of transport is extremely large. The analysis of relevant databases has shown that the AHP method is applied in scientific papers, scientific projects, diploma theses and doctoral dissertations [3]. The AHP method includes expert opinion and multi-criteria evaluations. Its popularity stems from the fact that it is very close to the way an individual solves complex problems, breaking them down into simpler components and that into goal, criteria and variants. These components are combined into a model in which the goal is at the highest level, the criteria are at the first lower level, their sub-criteria are at the second lower level, and variants (possibilities) are at the lowest level [2]. The AHP method converts estimates from the Saaty scale into numerical values that can be processed and compared over a whole range of problems. The stated priority weights are calculated for each criterion in the hierarchy, allowing a comparison of different and often immeasurable elements in a rational and consistent manner. This possibility distinguishes AHP from other decision-making techniques [4].

In the final stage of the process, priority weights are calculated for each variant. These numbers represent variants or their relative ability to achieve the goal, so that they allow direct observation of different modes of action. Instead of prescribing the right decision, the AHP method helps decision makers find the answer that best suits the goal and their understanding of the problem [4]. The method consists of four parts: structuring the problem, collecting data, estimating the relative weights and determining the solution to the problem. The method is intended for solving decision-making problems in which a larger number of decision-makers participate, and a larger number of criteria and sub-criteria appear. The AHP method has its advantage in solving complicated problems in that it simplifies these problems to less complex situations. The method allows when considering problems to easily find the relationships between criteria and variants, in order to find the influence of one criterion in relation to another. After the problem structuring process, the decision maker assigns "ratings" to each individual pair of attributes at each hierarchical level. The most common rating scale is the so-called. Saaty scale of importance or evaluation (Table 1)

Table 1 Saaty evaluation scale, [5]

Intensity importance	Definition
1	Equally important
3	Moderately more important
5	Strictly important
7	Very strict, proven importance
9	Extreme importance
2,4,6,8	Among values
1.1-1.9.	Decimal values

The comparison of qualitatively expressed criteria, the scale of grades, is performed according to the description of the relationship of criteria from the so-called. Saaty scales (Table 1). Assigned grades are recorded in a matrix. This is how the so-called comparison matrix.As the method proved to be successful in solving multicriteria evaluations, the software tool "Expert Choice" [6] was developed for its application, which gave a significant impetus to the development and application of decision support systems and expert systems for solving multicriteria decision making. This tool is completely suitable for the application of the AHP method.

3 Application of AHP method for ranking dangerous sections since the occurrence of large game on state roads of Lika-Senj county

The paper investigates vehicle collisions with large game on the state roads of Lika-Senj County during the period from 2012 to 2016. The basic precondition for conducting the research is data that contain sufficient information on the basis of which dangerous sections from the occurrence of wildlife on the roads could be defined. Information on traffic accidents involving vehicles on game was taken over from the Ministry of the Interior, Lika-Senj Police Department [1]. In order to make a choice of variants, it is necessary to determine the criteria, based on which the solution will be chosen. The criteria involved can be evaluated, ie their data are available. The criteria are:

1. <u>Number of collisions</u>. The analysis of the number of vehicle collisions with game on the state roads of Lika-Senj County was conducted according to the data of the Ministry of the Interior, Lika-Senj Police Administration for the period 2012-2016. years [1]. The following data were used for each traffic accident: date of the accident, time of the accident, type, number and section of the road, consequences of the accident and species of wild animal.
2. <u>Length of section (km).</u> The lengths of the state road sections of Lika-Senj County were obtained from Hrvatske ceste d.o.o., Zadar business unit, Gospić technical branch [7].
3. <u>Number of collisions per 100 km.</u> The number of crashes per 100 km was obtained by dividing the number of crashes by the length of the section and by the number 5 (the observation period is 5 years) and multiplied by 100.

4. Number of collisions per 100 km per year. The number of raids per 100 km per year was obtained by dividing the number of raids per 100 km by the number 5 (the observation period is 5 years).

The choice of criteria is very important for the correct implementation of the AHP method, but it is also very important to determine the mutual values of the relationships of the selected criteria. By assigning weights to each criterion, the criteria were compared, and weights were added to all variants, ie dangerous sections, in relation to the number of collisions, section length in kilometers, number of collisions per 100 kilometers and number of collisions per 100 kilometers per year, ie in relation to each criterion. Table 2 and Figure 1 show the most dangerous sections of state roads in Lika-Senj County from vehicle collisions with large game in the studied period, and set the criteria for the implementation of the AHP method.

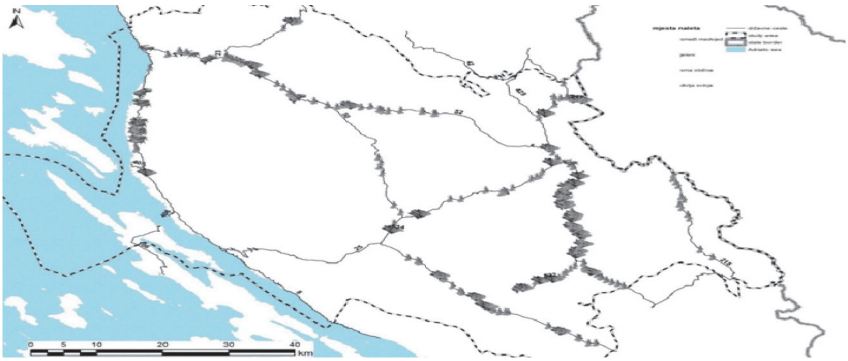

Figure 1 Location of the place of collision of vehicles with large game during the research period (2012-2016) on the state roads of Lika-Senj County

Before using the software tool Expert Choice [6], appropriate matrices were compared comparing the criteria with each one (subtracting the larger from the smaller) in relation to the given goal. Based on the comparison of criteria, we obtained that the lowest value is 0 and the highest value is 90. In the Saaty evaluation scale, the number 1 is always equal to 0. The other numbers on the Saaty scale are obtained by increasing each subsequent number in the scale by an equal number. We replaced the values obtained by comparing dangerous places in pairs with the ratings of the Saaty scale and we obtained a matrix of a certain criterion, and we do the same procedure for all criteria. We enter the results from the criteria matrices in the Expert Choice software tool.

Table 2 Number of collisions of large game vehicles on state roads of Lika-Senj County by sections during the period 2012-2016. and crash density [1] [6]

Dangerous stock N.	Road	Stock	Name	Number of flashes	Lenght of stock [km]	Number of flashes 100 km	Number of flashes 100 km per year
1	DC-1	12	Grabovac (DC42) – Vrelo Koreničko (DC52)	39	23,00	33,91	6,78
2	DC-1	13	Vrelo Koreničko (DC52) – Mutilić: čvorište Udbina (DC522)	91	33,17	54,87	10,97
3	DC-1	14	Mutilić: čvorište Udbina (DC522) – Gračac (DC27)	26	11,58	44,91	8,98
4	DC-8	7	Senj (DC23) – Stinica (DC405/LC59148)	32	36,67	17,45	3,49
5	DC-8	8	Jablanac (DC405) – Prizna (DC406)	2	13,00	3,08	0,62
6	DC-23	3	Jezerane (ŽC5191) – Žuta Lokva (DC50)	9	19,19	9,38	1,88
7	DC-23	4	Žuta Lokva (DC50) –Senj (DC8)	8	22,26	7,19	1,44
8	DC-25	1	Korenica (DC1) – Lički Osik (DC50)	9	36,57	4,92	0,98
9	DC-25	2	Lički Osik (DC50) – Karlobag (DC8)	7	47,26	2,96	0,59
10	DC-50	1	Žuta Lokva (DC23) – Špilnik (DC52)	27	21,47	25,15	5,03
11	DC-50	2	Špilnik (DC52) – Lički Osik (DC25)	8	34,37	4,66	0,93
12	DC-50	3	Gospić (DC25) – Lovinac (ŽC5165)	27	31,99	16,88	3,38
13	DC-50	4	Lovinac (ŽC5165) – Gračac (DC27)	12	15,19	15,80	3,16
14	DC-52	1	Špilnik (DC50) - Korenica (DC1)	31	41,11	15,08	3,02
15	DC-217	1	Ličko Petrovo Selo (DC1) – Novo Selo Koreničko: GP Ličko Petrovo Selo (granica RH/BIH)	14	2,97	94,28	18,86
16	DC-218	1	Nebljusi: GP Užljebić (Granica RH/BIH) –Dobroselo (Ž5203)	14	30,08	9,31	1,86
17	DC-218	2	Dobroselo (ŽC5203) – Bruvno (DC1)	1	8,70	2,30	0,46
18	DC-429	1	Selište Drežničko (DC42) – Prijeboj (DC1)	12	14,1	17,02	3,4
19	DC-522	1	Mutilić (DC1) – Gornja Ploča: čvor Gornja Ploča (A1)	21	13,19	31,84	6,37
20	DC-534	1	Gospić (DC25) –Lički Osik: čvorište Gospić (A1)	1	2,45	8,16	1,63

4 Results of multicriterion evaluation by AHP method

The Expert Choice software tool is suitable for the application of the AHP method as a method of multi-criteria evaluation, and allows direct entry of criterion values. The program models the hierarchical structure of the problem, and allows users to use their expertise. By entering values from the criteria matrices into the Expert Choice program, we obtain the following data below.
Of the 4 criteria (number of collisions, section length in kilometers, number of collisions per 100 kilometers and number of collisions per 100 kilometers per year), based on surveys and expert assessment, the most influential criterion, ie the criterion with the highest weight value was the section length in kilometers (Figure 2). it is followed by the criterion by weight criterion number of collisions per 100 kilometers per year, then the criterion number of collisions 2012-2016 and the last criterion by weight value is the criterion number of collisions per 100 kilometers. Figure 3 shows the ranking of the offered variants (dangerous shares) with a percentage.

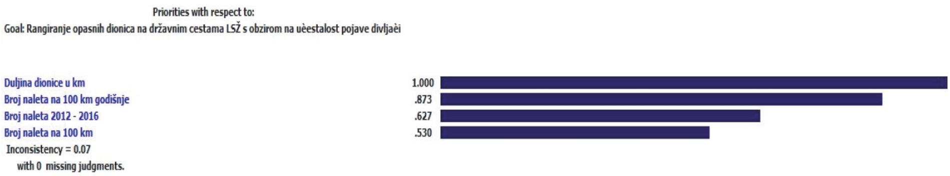

Figure 2 Weight values of the criteria

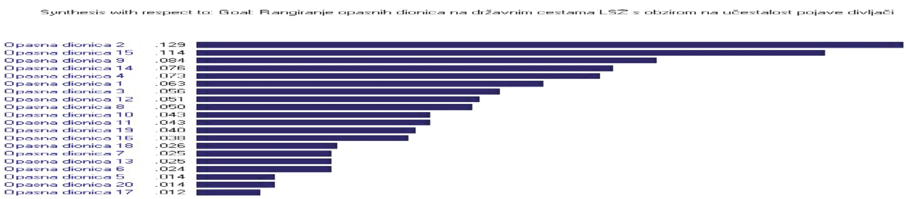

Figure 3 Ranking of dangerous sections from the smallest to the most dangerous section from the occurrence of game on the state roads of Lika-Senj County

It can be seen that the most dangerous section is section 2, followed by section 15, and measures to rehabilitate road sections from wildlife should be the first to be applied to them. Sensitivity analysis allows the determination of “critical” variables or model parameters, and its main goal is to assess the acceptability of the project if the values of critical project parameters are changed. Therefore, sensitivity analysis determines the reliability of the defined model, and it gives the possibility to make decisions to test different sets of alternative solutions. The Expert Choice software tool enables this analysis using three charts, namely the dynamics chart, the performance chart, and the gradient chart. Figures 4, 5, and 6 show the three graphs obtained with the Expert Choice software tool.

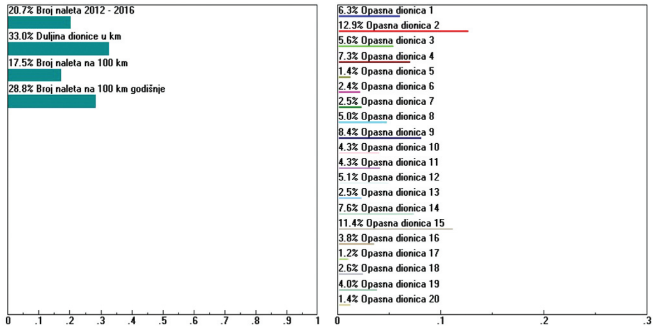

Figure 4 Dynamic chart

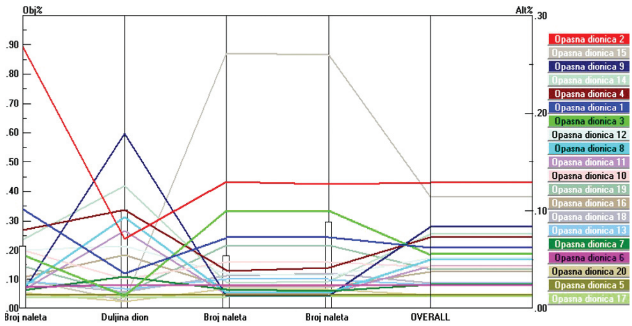

Figure 5 Performance chart

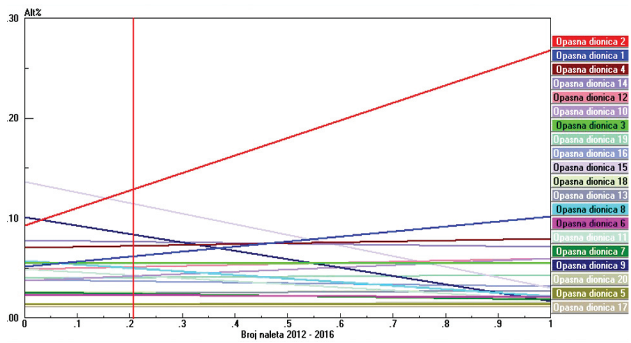

Figure 6 Gradient chart

Figure 4 shows the share of significance of individual criteria in the system of offered variants. Figure 5 shows the impact of individual criteria on the overall order of variants, while Figure 6 shows how changes in the weights of individual criteria affect the overall order of variants, and the graph provides an opportunity to analyze the impact of each individual criterion on the final solution.

5 Conclusion

Multicriteria evaluation is an extremely complex process, with the possibility of diverse applications in different spheres of activity. This paper presents a model for multi-criteria ranking of the most safety-critical sections on the state roads of Lika-Senj County since the appearance of large game on them. The AHP method was used for the valuation and ranking of dangerous stocks. Since the method proved to be successful in multicriteria decision-making, the Expert Choice program was developed for its application, which was also used in the paper. The most important result obtained during the research relates to the fact that thanks to a well-designed model of multi-criteria evaluation, it was possible, using the AHP method, to make an exact decision on the rank of risky road sections from the occurrence of wildlife. Multi-criteria evaluation by the AHP method gives us a good insight into the ranking of dangerous sections from the occurrence of large game on the state roads of Lika-Senj County. In this paper, the exact choice of the priority of the rehabilitation of dangerous sections from the occurrence of large game is obtained, which can signal the priorities of the rehabilitation of dangerous sections to the road infrastructure managers. In the end, it can be concluded that the multi-criteria evaluation by the AHP method proved to be a very good tool in the ranking of dangerous sections from the occurrence of large game on the state roads of Lika-Senj County.

References

[1] Republika Hrvatska, Ministarstvo unutarnjih poslova, Policijska uprava Ličko-senjska, broj: 511-04-03/04-4591/2-2017. M.B., Prometne nesreće- nalet na divlje životinje i sudar sa domaćim životinjama na cesti, 05.07.2017.

[2] Klanac, J., Perkov, J., Krajnović, A.: Primjena AHP i PROMETHEE metode na problem diverzifikacije, Oeconomica Jadertina 2/2013, Zadar, 2013.

[3] Barić, D., Starčević, M.: Implementation of Analytic Hierarchy Process in Solving Transport Problems, International Journal of the Analytic Hierarchy Process, 7 (2015), 2, pp. 295-312 doi:10.13033/ijahp.v7i2.251, 2015.

[4] Forman, E., Selly, M.A.: Decision By Objectives (How to convince others that you are right)

[5] Saaty, T. L.: The Analytic hierarchy process, New York, McGraw-Hill, 1980.

[6] Expert Chioce software, http://www.expertchoice.com

[7] Hrvatske ceste d.o.o., poslovna jedinica Zadar, tehnička ispostava Gospić

RESEARCH ON RELATIONSHIP BETWEEN COGNITIVE IMPAIRMENT AND DRIVING BEHAVIOR OF STARTING/STOPPING FOR ELDERLY DRIVER

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Abstract

Recently in Japan, traffic accidents caused by elderly drivers have attracted public attention. Elderly drivers are judged whether they can continue driving based on the cognitive test results at the renewal of their driver's license. However, it has not been to lead to the reduction of accidents. Therefore, it is important to understand the relationship between cognitive impairment and driving behavior for elderly drivers. In this study, we analysis on the relationship between cognitive impairment and driving behavior for elderly drivers. The driving ability is evaluated using the data of driving behavior during starting/stopping. Regression models that explain the relations between cognitive impairment of an elderly driver and driving ability were estimated.

Keywords: elderly driver, driving ability, cognitive impairment

1 Introduction

Recently in Japan, traffic accidents caused by elderly drivers have become a serious social problem. Traffic accidents caused by elderly drivers are believed by age-related in cognitive impairment. It has a great effect on driving ability and leads to the occurrence of driving accidents. Elderly drivers are obliged to undergo a cognitive test at the time of renewal of his/her driving license. Thus, they are judged whether they can continue driving based on the results of cognitive tests.

However, even if elderly drivers pass the cognitive test, a traffic accident will occur due to cognitive impairment. This is because the relationship between the degree of cognitive impairment and the occurrence of traffic accidents for elderly drivers has not yet been clarified. Therefore, performing a cognitive test has not been obtained great effect.

There are many previous studies that focus on the relationship between driving ability and cognitive impairment for elderly drivers from an engineering point of view. For example, Myers et al [1] focused on the relationship between road driving tests and cognitive tests by elderly drivers. As a result, it was clarified that the results of the cognitive test and the driving ability are proportional. Wadley et al [2] and Griffith et al [3] conducted driving experiments by an elderly driver of MCI. They focused on the driving behavior when going straight. These previous studies have shown the possibility of discriminating MCI using lane deviance, steering stability, and vehicle speed changes. In addition, Frittelli et al [4] showed that elderly drivers of MCI have less time to follow vehicles in front. Beratis et al [5] focused on the driving behavior when changing lanes. It was shown that when the cognitive impairment of elderly drivers declines, driving operations of lane change tend to be delayed.

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We have reviewed previous studies on driving ability and cognitive impairment for elderly drivers. However, it became clear that quantitative analysis has not yet been performed. The reason is that it is necessary to set clear criteria when evaluating driving ability. Thus, in this study, we analyze the relationships between driving ability and the cognitive impairment of elderly drivers. Regression models that explain the relations between cognitive impairment of an elderly driver and driving ability were estimated.

2 Data

2.1 Outline of observation experiment

Table 1 shows the outline of the driving experiment. In this study, 56 drivers aged 65 and over living in Kanazawa City, Ishikawa Prefecture were the participants. The driving behavior of the elderly driver was observed for 2 weeks.
The flow of the driving experiments is shown. First, we obtained confirmation of consent to participate in the experiment to start it. Next, we conducted a hearing survey of the participants. In this hearing survey, the participants' age, gender, visual acuity, and the presence or absence of eye diseases were investigated. Next, a cognitive test was performed. Finally, an observation device was installed in the participant's vehicle to observe the driving behavior.
Table 2 shows the attribution of participants. There is almost no gender bias and it became clear that most of them were in their 70s. In addition, most of the subjects had visual acuity of 14/20 or higher, but it was revealed that they had eye diseases.

Table 1 Outline of experiment.

Target person	Citizens living in Kanazawa city who are aged 65 and over and who drive daily.
Number of persons tested	56 persons
Observation period	2 weeks
Procedure of the experiment	1. Obtain the consent of the participant. 2. Perform a hearing survey to the participant 3. Perform the cognitive test 4. Perform the observation survey for driving behavior.
Items of the hearing survey	Sex, Age, Eyesight, Presence or absence of eye diseases

Table 2 Attribution of participants(N=56)

Sex	Male: 41.07 %, Female: 58.92 %
Age	60s: 26.79 %, 70s: 66.07 %, 80s: 7.14 %
Eyesight	Under 6/20: 1.79 %, Between 6/20 ≈ 14/20: 12.50 %, Over 14/20: 85.71 %
Presence of eye diseases	Cataract: 50.00 %, Glaucoma: 16.07 %, Other eye diseases: 23.21 %

2.2 Outline of cognitive test

In this study, participants were tested for MMSE, Pareidolia, TMT-A, and TMT-B. MMSE is composed of 11 questions that covered orientation, registration, attention, calculation, recall, language, and visuospatial perception. scores range of MMSE from 0 to 30; scores less than 24 are indicative of cognitive impairment. It was clarified that when the MMSE score is 24 points or less, it has a great influence on driving ability.
Pareidolia is a test to find the phenomenon (hallucination) in which the shape of stains and clouds on the wall looks like a human face or an animal. From the results of this test, it is

possible to detect whether or not it is Lewy body dementias. In the case of driving, it is related to the oversight of road signs.
TMT has been widely used as a test of executive function and visual perceptual and visual–motor tracking. TMT is given in two parts. Trail Making Test Part A (TMT-A) requires participants to connect a series of consecutively numbered circles and involves visual scanning, number recognition, numeric sequencing, and motor speed. Trail Making Test Part B (TMT-B) requires participants to connect a series of numbered and lettered circles, alternating between the two sequences. The results of TMT showed the possibility of making a judgment on safe/dangerous driving.
Figure 1 shows the results of the cognitive test. Figure 1 (a) shows the results of the MMSE. If the MMSE result is 23 points or less, there is a risk of dementia. However, there was not it in the participants of this study. Figure 1 (b) shows the results of Pareidolia. This score does not have a reference value. Therefore, it can be said that the lower the number of correct answers, the more Lewy body dementias possibilities. However, it became clear that most of the participants answered all the questions correctly. Figures 1 (c)-(d) show the results of TMT-A and TMT-B. It can be said that the cognitive impairment of TMT-A and TMT-B declines when the calculation time is long.

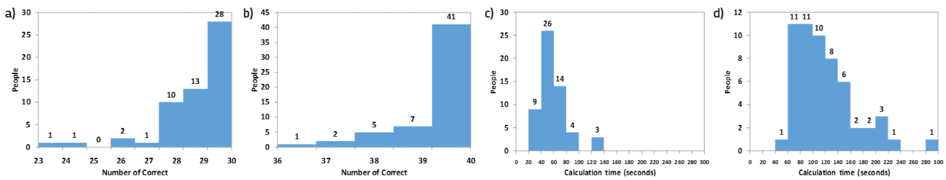

Figure 1 Result of cognitive test (N=56)

2.3 Observation device and observation method

Table 3 shows the outline of the observation device. The driving behavior was observed using the Qstarz GT BL-1000 GT. This observation device measures the vehicle speed, acceleration of the three-axis, azimuth, and location information every 0.1s. The acceleration for X-axis is the left-right acceleration of the vehicle. The right side when viewed from the driver is positive. The acceleration for Y-axis is the acceleration of the front and rear of the vehicle. The front is positive when viewed from the driver. The flow of observing the driving behavior is described. When starting operation, the driving behavior is observed by turning on the power of the observation device. When the operation is finished, the observation of the driving behavior is stopped by turning off the power of the observation device.
Figure 3 shows the driving situation of the participants during the survey period. Figure 3 (a) shows the number of driving days per week. It was revealed that all the participants were driving over three days a week. Figure 3 (b) shows the average driving minutes per day. It was found that most participants had less than an hour of driving minutes per day.

Table 3 Overview of observation device

Using device	Acceleration censor (Q-starz International Co,Ltd. (QstarzGT BL-1000GT)
Observation item	Vehicle speed/km/h/3-axis acceleration/G/Azimuth/deg/Location information /latitude/longitude
Observation interval	These data are measured at 10 Hz (0.1 seconds).
Observation procedure of driving behavior	1. Turn on the switch of the observation device before starting the driving. 2. Turn off the switch of the observation device after the driving is completed.

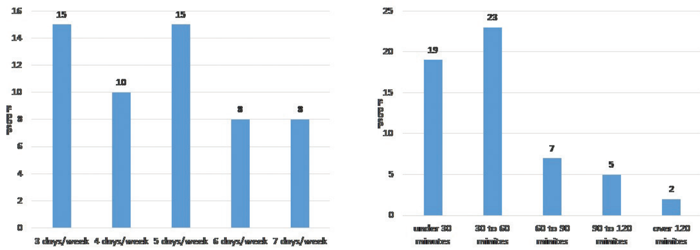

Figure 2 Driving situation of participants (N=56)

3 Production of analysis data

3.1 Extraction of driving behavior of starting/stopping for driver

In this section, the driving behavior of starting and stopping is extracted from the observation device. In this study, the driving behavior of starting and stopping is extracted from the speed of the vehicle. For the starting driving behavior, the driving behavior of the vehicle with a speed of 0 km/h to 20 km/h is extracted. On the other hand, the driving behavior of the stop extracts the driving behavior of the vehicle at a speed of 20 km/h to 0 km/h.

3.2 Index of driving ability evaluation

This section describes the evaluation of driving ability using the driving behavior of starting/stopping. In this study, we considered the driving behavior of starting and stopping due to cognitive impairment from two perspective.
One focuses on the incidence of sudden braking. A high rate of sudden braking means that a traffic accident is likely to occur. In this study, drivers with a high incidence of sudden braking were considered to correlate with cognitive impairment. Therefore, the driving behavior in which the acceleration in the Y-axis direction exceeds the absolute value of 0.25 G is counted as sudden braking. The sudden braking occurrence rate is obtained by dividing the number of starting/stopping driving behaviors in which sudden braking has occurred by the number of starting/stopping driving behaviors performed on that day.
Moreover, we focus on the value of acceleration in the Y-axis direction observed in the driving behavior of starting/stopping. A large value of acceleration in the Y-axis direction means that the effect of an accident is large. In this study, drivers with large acceleration values were considered to correlate with cognitive impairment. Thus, we focus on the maximum and minimum values for acceleration in the Y-axis direction.
Table 4 shows an example of evaluation of driving ability using the driving behavior of starting and stopping. Calculate the average value and standard deviation during the survey period from the rate of sudden braking of starting/stopping for each driving day. Furthermore, the average value and standard deviation during the survey period are calculated from the

maximum and minimum values of acceleration in the Y-axis direction of starting and stopping for each operation day(See the red frame in the figure). A high average value means a large rate of sudden braking and a large value of acceleration in the Y-axis direction. Therefore, it means dangerous driving. On the other hand, if the standard deviation is high, the occurrence rate of sudden braking and the value of acceleration in the Y-axis direction fluctuate greatly from the day. Therefore, it means that daily driving is not stable.

Table 4 Calculation example of driving ability evaluation

Observation date	Ratio of sudden staring [%]	Ratio of sudden stopping [%]	Maximum acceleration for Y-axis when starting (G)	Minimum acceleration for Y-axis when starting (G)	Maximum acceleration for Y-axis when stopping (G)	Minimum acceleration for Y-axis when stopping (G)
4/6/2020	17.24	14.81	0.37	-0.39	0.22	-0.17
4/8/20220	9.09	12.50	0.35	-0.31	0.41	-0.40
4/9/2020	14.29	16.67	0.27	-0.27	0.26	-0.26
4/10/2020	15.38	6.45	0.28	-0.46	0.41	-0.30
4/11/2020	0.00	6.25	0.13	-0.23	0.23	-0.16
4/12/2020	14.29	6.67	0.13	-0.25	0.32	-0.15
Average	11.71	10.56	0.25	-0.32	0.31	-0.24
Standard deviation	6.35	4.69	0.10	0.09	0.08	0.09

4 Model estimation

4.1 Analysis method

In this study, we build a model that estimates cognitive impairment from the driving ability of elderly drivers. Therefore, we will clarify the effect of information on starting/stopping driving behavior and driver on the cognitive test. It is important to know what variables reproduce the results of the cognitive impairment test. Therefore, we use multiple regression analysis, which can grasp significant variables, as the analysis method.
The variables to be set in the model are described. The result of the cognitive test is set in the objective variable. The driving ability evaluation obtained from the starting/stopping driving behavior and the information about the driver are used as explanatory variables.

4.2 Estimation result

Table 5 shows the estimation results of multiple regression analysis. The explanatory variable was selected by the stepwise increase/decrease method with the rejection area set to 0.2. Significant explanatory variables for each cognitive impairment test are described. In the estimation result of MMSE, the parameter of the positive sign is a factor that enhances cognitive impairment. As a result, it was revealed that the driver who has many driving days per week, has a lot of driving time per day, has a high average rate of sudden braking when stopping, and has a high average maximum acceleration for Y-axis has a good. From this, it became revealed that the above variables explain the degree of impairment of intelligence function that can be measured by the MMSE.
In the estimation result of Pareidolia, the parameter of the positive sign is a factor that enhances cognitive impairment. As a result, It was revealed that the driver who has glaucoma, has a large standard deviation of the ratio of sudden braking when starting, and has a large standard deviation of minimum acceleration for Y-axis has a good. From this, it was revealed

that the degree of Lewy body dementias that can be measured by Pareidolia is explained by the above variables.
In the estimation result of TMT-A, the parameter of the negative sign is a factor that enhances cognitive impairment. As a result, It was revealed that the driver who is older, has glaucoma, has another eye disease, has a large standard deviation of minimum acceleration for Y-axis when stopping has a long calculation time for TMT-A. In the estimation result of TMT-B, the parameter of the negative sign is a factor that enhances cognitive impairment. It was revealed that the driver who is older, has a large standard deviation of minimum acceleration for Y-axis when starting has a long calculation time for TMT-B. From this, it was clarified that the possibility that an elderly driver who can measure with TMT-A and TMT-B judges safe and dangerous driving is explained by the above variables.
Moreover, focus on the adjusted coefficient of determination. In conclusion, the TMT-B model proved to be relatively good. Therefore, it was revealed that the driving ability of starting/stopping behavior can be measured by TMT-B. Moreover, the possibility that an elderly driver can judge safe/dangerous driving can be high.

Table 5 Estimation result

Object variable	**MMSE**		**Pareidolia**		**TMT-A**		**TMT-B**	
Explanatory variable	Estimate	t value	Estimate	t value	Estimate	t value	Estimate	t value
Age					8.16	3.02	23.90	4.85
Cataract			-0.49	4.04				
Glaucoma			0.23	1.93	3.60	1.35		
Other eye diseases			-0.15	1.31	5.68	2.16		
Driving day/week	0.45	2.25						
Driving time/day	0.29	1.51						
Average ratio of sudden starting			-0.58	2.45				
Standard devitation ratio of sudden starting			0.48	2.33				
Average ratio of sudden stopping	0.38	1.88	-0.23	1.71				
Standard devitation ratio of sudden stopping					-4.10	1.56	-8.27	1.65
Avarage of maximum accelation for Y-axis when starting	0.29	1.52						
Standard devitation of minimum accelation for Y-axis when starting			0.24	2.12			14.88	2.74
Avarage of maximum accelation for Y-axis when stopping							-19.59	3.54
Avarage of minimum accelation for Y-axis when stopping			-0.67	2.35				
Standard devitation of minimum accelation for Y-axis when stopping			-0.75	3.02	4.48	1.67		
Intercept	28.98	157.60	39.52	370.27	58.87	23.25	119.72	25.04
Ajusted R-squared	0.14		0.27		0.27		0.42	
Multiple correlation coefficient	0.20		0.39		0.33		0.46	
Number of samples	56		56		56		56	
AIC	201.45		144.73		496.04		566.35	

5 Conclusions

In this study, we examined the relationship between cognitive impairment and driving behavior. The cognitive impairment of elderly drivers was tested by the MMSE, Pareidolia, TMT-A, TMT-B. Driving behavior data was collected using an accelerometer through observation experiments. We focused on starting/stopping driving behavior. To evaluate the driving ability, the rate of sudden braking at the start and stop of each day was calculated. In addition, the maximum / minimum acceleration of the Y-axis of the starting/stopping behavior for each day was also calculated. The average and standard deviation within the observation period were calculated from the evaluation of these driving abilities.
In this study, we constructed a model that estimates cognitive impairment from the created evaluation of driving ability. As a result, it was confirmed that there is a weak but causal relationship between the driving behavior of elderly drivers and cognitive impairment.
In conclusion, it was clarified that the visual processing ability and attention distribution of cognitive impairment are related to the starting/stopping behavior of elderly drivers.

References

[1] Myers, R.S., Ball, K., Kalina, T.D., Roth, D.L., Goode, K.T.: Relation of useful field of view and other screen-ing tests to on-road driving performance, Perceptual and Motor Skills, 91 (2000) 1, pp. 279-290.

[2] Wadley, G.V., Okonkwo, O., Crowe, M., Vance, E.D., Elgin, M.J., Ball, K.K., Owsley, C.: Mild Cognitive Impair-ment and Everyday Function: An Investigation of Driving Performance, Journal of Geriatric Psychiatry and Neurolo-gy, 22 (2009) 2, pp. 87-94.

[3] Griffith, H.R., Okonkwo, O.C., Stewart, C.C., Stoeckel, L.E., Hollander, J.A., Elgin J.M., Harrell, L.E., Brockington, J.C., Clark, D.G., Ball, K.K., Owsley, C., Marson, D.C., Wadley, V.G.: Lower Hippocampal Volume Predicts Decrements in Lane Control among Drivers with Amnestic MCI, Journal of Geriatric Psychiatry and Neurology, 26 (2013) 4, pp. 259-266

[4] Frittelli, C., Borghetti, D., Iudice, G., Bonanni, E., Tognoni, M.G., Pasquali, L., Ludice, A.: Effects of Alzheimer's disease and mild cognitive impairment on driving ability: a controlled clinical study by simulated driving test, International Journal of Geriatric Psychiatry, 24 (2009) 3, pp. 232-238.

[5] Beratis, I.N., Pavlou, D., Papadimitriou, E., Andronas, N., Kontaxopoulou, D., Fragkiadaki, S., Yannis, G., Pa-pageorgiou, S.G.: Mild Cognitive Impairment and driving: Does in-vehicle distraction affect driving performance?, Accident Analysis & Prevention, 103 (2017), pp. 148-155.

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TRANSPORT STRUCTURES AND SUBSTRUCTURES: MODELLING, DESIGN AND MONITORING

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STEEL PILES DRIVING PROCEDURE AND RESULT ANALYSIS OF EXTRADOSED BRIDGE MAINLAND - PELJEŠAC

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Abstract

In order to meet the high requirements of marine environmental protection and Eurocodes, based on the actual construction conditions of deep water on site in Croatia, the extra-long steel pile foundation was adopted to Pelješac Bridge. At the meantime the corresponding extra-large scale pile driving barge had to be used to carried out during the construction. The pile bearing capacity was analyzed and checked by the actual measured PDA (Pile Driving Analyzer) data. The test results showed that the toe bearing capacity of driven piles had linear relationship with blow counts, and the penetration (displacement/blow) before the stoppage was inversely proportional to toe bearing capacity. In addition, the traditional empirical formula of long-term pile bearing capacity of driven piles was only suitable for the piles, which were shorter than 100m. The stoppage criteria of extra-long pile should concentrate on penetration firstly, while the pile design elevation was subsidiary factor. Therefore, the analysis of pile driving procedure and results could be considered as significant actual engineering reference for the coming works.

Keywords: driven piles, blow counts, penetration, long-term bearing capacity

1 Project Introduction

The extradosed cable-stayed Bridge Mainland - Pelješac is financed by EU fund and constructed by the international contractor China Road and Bridge Corporation (CRBC). This project was the largest infrastructure project under construction in Croatia at present. The bridge has the whole layout of 84 +108 +108 +189,5 + 5 x 285 +189,5 + 108 + 108 + 84 = 2404 m, and the total length of the bridge including abutments was 2440 m. There was total 10 piers located in the sea. Therefore, the corresponding 150 pieces of steel pipe piles were constructed in the deep sea.

2 Classification of pile foundation

Pile works was separated into two main types according to different categories of pile toe characteristics. One type was steel driven pile with stiffening pile shoes, which were embedded into hard rock and the upper 40m were filled with reinforced concrete, however, the other type was driven piles with concrete sockets instead of stiffening pile shoes stand on the hard rock, the pile inside and socket was drilled by drilling machine and filled with reinforced concrete. The longitudinal stiffeners were welded inside for improving the stability of steel tubular piles, and also enhancing the connection between steel tubular piles and concrete. The 2-meter-long pile shoes were made of steel S460NH with the wall thickness of 60 mm. Additionally 8 pieces of stiffeners were set inside of pile shoes of the first type, so steel driven piles had larger compression areas than concrete-socketed piles, and the maximum allowable stress during the driving must be different. See Figure 1.

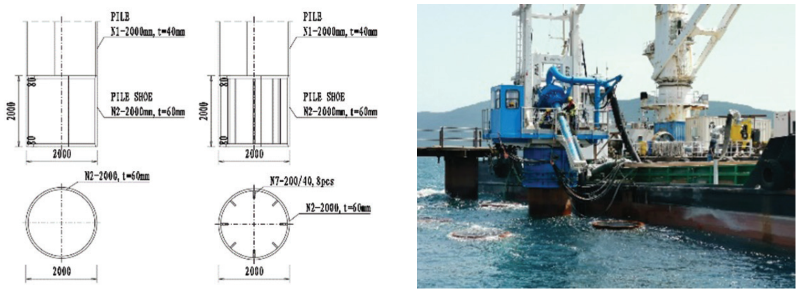

Figure 1 Pile classification and RCD drilling machine

3 Steel pile production and transport

The design lengths of piles ranged from 36 m to 130,6 m. The proportion of design length over 100m exceeds 70 %. If the extra-long steel pile was divided into two large segments for production, after that they were spliced in the air and welded on bridge site, it would lead to high risks of safety and quality. Firstly, the welders must enter inside from the 60-meter-high pile top. Secondly, in order to decrease the influence of wind and wave, the adopted platform and barges must be anchored on the sea as stable as in the workshop, in this case the straightness and concentricity of piles couldn't be ensured seriously according to EN 10219-2: 2006 Table 2 [1]. Thirdly, the manual welding quality on site couldn't meet the requirements of EXC4 B+, referring to Table 17 in EN 1090-2: 2008 [2]. As contractor's final determination the extra-long steel piles were entirely manufactured instead of connection on site. All of the extra-long steel piles were manufactured in China workshops, and then the cargo vessel took them to Croatia in six batches respectively. Since the contractor started the pile production until the completion of pile driving on site, the pile works period had been achieved and controlled efficiently within only 5 months.

Figure 2 Steel tubular piles storage at port and uploading to cargo vessel

4 Driving procedure and stoppage criteria

The mean water depth was 27,0 m on bridge site, the thickness of soil under seabed is between 30 m and 100 m, it was composed of soft clay, hard clay and gravel. The hard clay layer was started from 60 m deep and its negative skin friction was up to 300 kPa. The hard rock layer located below the hard clay. The strength of hard rock was about 80 MPa. After the com-

prehensive consideration, the driving barge was equipped with the hydraulic hammer finally, type IHC S800. The kinetic energy on hammer output was $E_{k,\ max}$ = 800 KJ, due to energy loss of system the real transmitted energy into the pile could be up to 85 or 90 % of the output $E_{k,}$. The maximum pile driving length by driving barge could be up to 133 m.
Dynamic monitoring was performed with equipment which corresponded to the standard ASTMD4 945-08 ‹Standard Test Method for High Strain Dynamic Testing of Deep Foundation›, for characteristic strikes at the end of the pile driving, CAP / WAP analysis was performed for each pile. Steel piles were driven in the accordance with the following procedures strictly,

a) Piles were lifted and positioned correctly by pile driving barge.
b) Piles sunk until stillness by self-weight of pile and hammer.
c) Started the hammer with 10 % of $E_{k,\ max}$ and recorded the blow counts.
d) Increasing 10 % of output energy E_k gradually, based on the actual penetration. The penetration was controlled around of 20 cm / 25 blows.
e) Stoppage for the installation of the Pile Driving Analysis (PDA) sensors, the last 20 m of the pile driving was monitored and the dynamic response was recorded.
f) Keeping on pile driving until final stoppage according to stoppage criteria. Meanwhile pile body and toe stresses were controlled under the maximum of the allowable stress.

Figure 3 Steel pile driving by large-scale driving barge

Before the final stoppage of hammer, the following pre-conditions should be taken into consideration comprehensively.

- Under full energy of hammer, in the range of the last 1,0 meter the average blow counts were more than 250 for each 25 cm (1 mm / blow)
- Penetration was reduced to 20 blows each 10 mm (0,5 mm / blow)
- The pile body stress was close to maximum allowable stress.
- Pile body and toe stresses by the dynamic monitoring met the requirements of PDA engineer.

In summary, the bearing capacity of the pile toe was the dominating factor of stoppage for the driven piles with stiffening pile shoes, and in the meanwhile design elevation of piles was stoppage criteria for the concrete-socketed steel driven piles.

5 Pile bearing capacity analysis

5.1 End driving results analysis (ED)

The following Figures 4 and 5 showed the relationship between the final penetration and the pile bearing capacity. The PDA equipment measured and recorded all the original driving data, which were from 45 concrete-socketed and 94 driven piles respectively. The most of two types of the piles had some points in common.

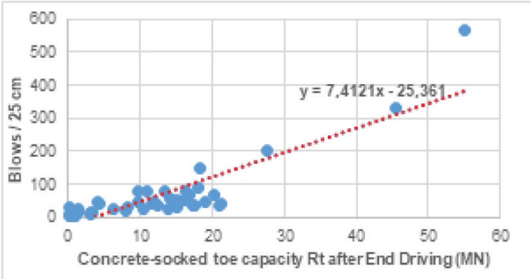

Figure 4 Concrete-socked pile curve

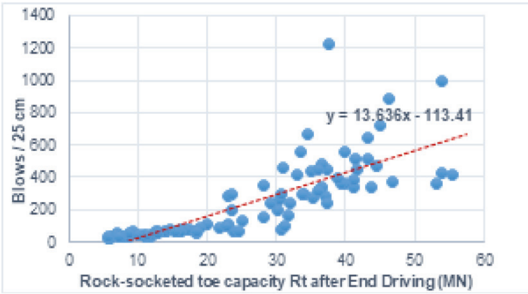

Figure 5 Driven pile curve

- Toe bearing capacity of vertical direction was directly linear proportional to blow counts of hammer.
- It shows inverse ratio of pile penetration (displacement per blow) and vertical bearing capacity.
- In comparison with concrete-socketed piles, the data of driven piles showed more discrete characteristics. And there were also some differences between the two types of piles.
- Steel driven riven piles had much higher measured toe bearing capacity than concrete-socketed piles.
- The final penetration of steel driven piles was quite smaller than concrete-socketed piles.

The reason of the mentioned differences was that the driven piles were mainly supported by hard rock directly, and the other type of piles was supported by skin friction and concrete socket together. In addition, the chart data was proved that it's consistent with the summary of stoppage criteria in previous chapter 4.

5.2 Re-Driving procedure (RD) and long-term pile bearing capacity

In order to check if the shaft resistance increased and meanwhile estimate how the growing trend for the final vertical bearing capacity will be. The 130,6-meter-long Pile No. TP7 as the chosen test pile was driven several times within one month. The RD tests were carried out after ED with a wait period as following orders:

1) Pile position was marked with total station by the surveyor on shore.
2) 5 blows with full energy $E_{k,\,ma}$.
3) Vertical settlement was recorded and reported to PDA engineer.
4) The mentioned procedure was repeated 3 times, in order to calculate the average value of skin resistance.

Based on the following data of TP7, the trend of final bearing capacity and soil setup effect could be analyzed (see Table 1):

- The skin friction $R_{s,act}$ around the pile increased much faster after initial driving.
- Within the first 2 days a rapid consolidation of the soil caused recovery by the shaft resistance.
- Although it is a long-term process of soil setup and friction increasing, over 90 % of the process were completed within 3 weeks after the initial driving.
- The activated part of toe bearing capacity is decreased step by step, due to the increasing effect of soil consolidation.

The measured maximum skin friction $R_{s,act}$ was in the range of 50 MN. However, it was only partially activated. The maximum activated friction was dominated by transmitted effective energy from hammer, so the more energy of hammer could be transmitted, the more skin friction could be activated.

Table 1 Test pile TP7 origal data of pile driving

Time	Days	Skin friction $R_{s,act}$ [MN]	Toe bearing capacity $R_{t,act}$ [MN]	Pile bearing capacity $R_{u,act}$ [MN]	Bearing capacity at superposition R_{u_sup} [MN]	Set [mm]
ED	0,001	6,6	**20,8**	27,4	27,4	5
1 hour	0,042	28,5	11,7	40,2	49,3	1,5
1 day	1,0	40,6	8,6	49,2	61,4	1
2 days	2,0	48,9	4,18	53,1	69,7	‹1
4 days	4,0	51,0	2,5	53,6	71,8	‹1
19 days	19,0	**53,5**	0,5	54,0	**74,3**	‹1

In principle, when transmitted energy was not limited by driving equipment, the shaft friction and toe resistance could be activated fully. Therefore, the bearing capacity at this moment was regarded as superposition R_{u_sup}. It could be calculated approximately by the following formula (1)

$$R_{u\_sup} \geq R_{s,act} = R_{s,act} + R_{t,act} \tag{1}$$

The maximum activated toe bearing capacity $R_{t,act}$ = 20,8 MN was confirmed after initial driving, the activated skin friction $R_{t,act}$ was measured individually. As a result, the total pile bearing capacity after re-driving $R_{u,RD}$ was confirmed and meanwhile the maximum bearing capacity $R_{u,max}$ was more than $R_{u,19}$= 74,3 MN. But it was quite closer to bearing capacity at the superposition.

Up to now, Denver & Skov (1988) long term formula was best approximation of long-term pile capacity. The total bearing capacity followed a linear increase versus the log of the time elapsed after initial driving. As follows [3]

$$\frac{R_u}{R_0} = 1 + A\log\left(\frac{T}{T_o}\right) \tag{2}$$

Remark: R_u and R_0 are the whole pile bearing capacity corresponding to time T and T_0 respectively. T_0 is the reference time at start of log-linear capacity increase. *A* is dimensionless setup factor.

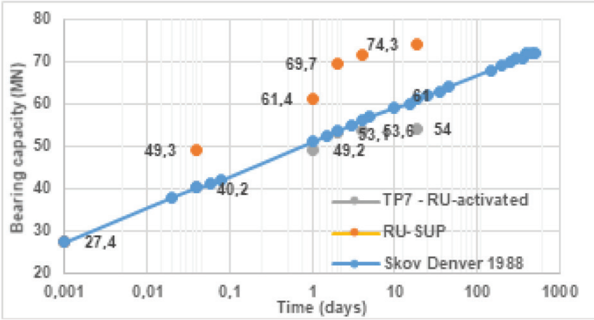

Figure 6 Log-linear long-term pile bearing capacity

Put the data of re-driving at the 0,001 and 0,042 day into above formula and get A was 0,288. As a result, the calculated bearing capacity of 19th day was 61,0 MN. But the measured $R_{u,19}$ was 54,0 MN. Therefore, the empirical formula for extra-long driven pile wasn't verified in this project. The main reason was the skin friction was partially activated, and the penetration after 1 hour was only 1,5 mm/blow.

6 Conclusion

The bearing capacity of driven piles was linear proportional to blow counts, and the penetration of the final stoppage was inverse proportional to bearing capacity. Because the pile cap entered under the water before final stoppage, the energy attenuation was deviation of around 10 % between the actual PDA measured energy and the initial output hammer energy. The traditional empirical formula for the bearing capacity of driven piles was only suitable for the length under 100 m. For the extra-long piles with the length over 100 m, the final stoppage criteria should follow the regulation of "Penetration mainly, Pile elevation supplementary"

Reference

[1] EN 10219-2: 2006: Cold formed welded structural hollow section of non-alloy and fine grain steels – Part 2: Tolerances, dimensions and sectional properties

[2] EN 1090-2:2008+A1 2011: Execution of steel structures and aluminum structures – Part 2: Technical requirements for steel structures

[3] Bullock, J.P.: The Easy Button for Driven Pile Setup: Dynamic Testing, Symposium Honoring Dr. John H. Schmertmann for His Contributions to Civil Engineering at Research to Practice in Geotechnical Engineering Congress, 2008, doi: 10.1061/40962(325)17

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EXTRA-LONG STEEL PILES PRODUCTION AND TRANSPORT OF BRIDGE MAINLAND – PELJEŠAC, CROATIA

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Abstract

Steel pipe pile as one of the foundation forms has the advantage of high bearing capacity, environment protection and installation, which were widely adopted to sea-crossing bridge, offshore wind power and marine oil rig at present. With the development of project scale, the length and diameter of the piles were increasing gradually. Meanwhile, civil engineers had to face the outstanding challenges in the production and transportation of extra-long and extra-large steel pipe piles. The design parameters of steel piles were controlled under the highest and strictest execution class EXC4 B+ in accordance with the European standard EN 1090-2. Additionally, the pile length of 130,6 m set a record for entire pre-fabricated longest steel pile in the field of civil engineering worldwide at that time. All the manufactured piles were delivered by cargo vessel after long voyage to Croatia. The accumulated experience of Pelješac Bridge could be as a reference for similar projects in future.

Keywords: extra-long, EXC4 B+, CNC, mechanized, long voyage

1 Project introduction

Bridge Mainland - Pelješac is located at a marine nature protection area in the south of Croatia. It will be connecting the separated two parts of the Croatian territory. By the construction of the bridge, the local economic development could be promoted rapidly. Bridge crosses the Mali Ston Bay with the length of 2440 m including approach spans. It is a multi-span extradosed cable-stayed bridge with a main span layout of 5 x 285 m. The superstructure applied steel box girders and strand stayed cables. The navigation clearance was 200 m x 55 m, as contractually agreed with Bosnia and Herzegovina. Steel pipe piles were adopted to deep water foundation. The lengths of steel piles ranged from 36 m to 130,6 m, and the proportion of pile length over 100 m exceeded 70 %.

2 Pile design work

The main steel pile structure consisted of pile head, pile body and pile shoe. Shear rings were welded around pile head with the distance of 30 cm, in order to enhance the stability of connection between pile head and pile cap. Pile body was composed of standard small segments with the length of 3,0 m, the pile body was made of raw material S355NH and the wall thickness is 40 mm. Meanwhile the 2-meter-long pile shoes were made of steel S460NH with the wall thickness of 60 mm. The longitudinal stiffeners were set and welded inside for improving the stability of steel tubular piles, and also enhancing the connection between steel tubular piles and concrete inside. Piles were divided into two main types according to

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pile shoe characteristics in this project. One type was steel driven pile with stiffening pile shoes, which were embedded into hard rock and the upper 40 m were filled with concrete, but the other type was reinforced-concrete filled steel piles with concrete sockets instead of stiffening pile shoes stand on the hard rock. see Figure 1.

Figure 1 Shear rings of pile head and pile shoes with stiffening pile shoes

The mean water depth on bridge site is 27 m, in order to ensure that the pile could be protected against corrosion of seawater and sludge, the upper 33,0-meter part of steel piles was painted with epoxy glass flake coating with dry firm thickness of 850μm, and the adhesion was as the requested minimum of 5,0 MPa. The adopted anti-corrosion system was consistent with the expected high durability in accordance with ISO 12944-5 [1], and the steel piles were equipped with the replaceable cathodic protection as well, so it is able to ensure the lifetime of the main structure of up to 100 years.

3 Preparation work of production

Before the production the selected Chinese manufactures had obtained the strict European Union (EU) and international certificates, and meanwhile gathered extensive actual valuable engineering experience of European standard EN 1090-2 [2] and international standard ISO 3834-2 [3]. In terms of personnel, the welders and the welding operators were qualified in accordance with ISO 9606-1 [4] and ISO 14732 [5] respectively.
According to approved detailed design drawings the manufacturers edited essential detailed technical documents before formal production, which were also approved by consulting engineer, such as the workshop drawings, manufacturing procedure specification (MPS), inspection and test plan (ITP), welding procedure qualification record (WPQR), etc. The welding procedure qualification test in the accordance with ISO 15614-1 [6] was organized and implemented by international welding engineer (IWE) of the manufacturers, and at the same time supervised and qualified by the consulting engineer and the third party during the whole procedure.
All of the raw materials were purchased from Chinese steel mills, which had acquired permission to produce steel plates of standard EN 10025 [7]. In the meantime, all the plates were qualified with the 3.2 inspection certificates of EN 10204 [8] by the EU notified body. The local steel mills were provided with the advantages of large plant capacity and short haul distance, so that it ensured the stable supply of steel plates during the production.

4 Manufacturing procedure

4.1 Standard segment production

As mentioned in chapter 2 the wall thickness is 40 mm and 60 mm respectively. For small diameter circular hollow section using thick steel plates was adopted. The pre-bending process was carried out for steel plate edges, and it was necessary to perform it by using a heavy CNC plate rolling machine to form the required curvature of the circular section. After that the plate was rolled backwards and forwards by the plate rolling machine to the "O" shaped cylinder and fixed with tack welding, then the curvature was checked by the templates in time. See Figure 2.

Figure 2 Standard segment rolling and forming

In order to decrease the difficulty of rolling and improve the processing efficiency, the total pile was separated into several small standard segments to achieve sectional manufacturing. After the forming, as approved by the welding procedure specification (WPS), the longitudinal seam was welded with mechanized welding process. Afterwards the re-rolling procedure was repeated to ensure the roundness of single standard segment. In order to avoid the deformation from welding residual stress, the press of automatic plate rolling machine needed to be strictly controlled.

4.2 Segments assembly

The steel piles were spliced from standard segments, one after another to three large segments with the length over 30 m by mechanized welding process. After that the large segments were fit-up and welded with circumferential seams together. See Figure 3. The circumferential seams were checked with non-destructive test (NDT) after the minimum hold time. The roundness and straightness as dominating dimensions were checked strictly for the standard segment, large segment and the total length, according to experience the internally permitted deviation were limited within mm and 0,05 % respectively, which were much stricter than the corresponding mm and 0,2 % referring to EN 1090-2 Annex D 1.9 Class B [2] and EN 10219-2: 2006 Table 2 [9]. The following Table 1 shows the actual measured value of driven piles in this project. In this way the final quality of steel piles could be ensured. The shear rings and pile shoes as additional production were installed when a large segment spliced completely.

Figure 3 Fit-up of standard segments

Table 1 Actual measured straightness and roundness of final product

Pile No.	Pile length [m]	Straightness [mm]		Out – of Roundness [mm]	
		Measured	Percentage [%]	Top	Bottom
S5.1	117,0	42	0,036	+2	+3
S6.4	126,6	59	0,047	0	-1
S7.4	128,1	55	0,043	+1	+2
S8.1	128,4	43	0,033	-3	-3
S9.1	122,0	51	0,042	+1	+1
TP7	130,6	34	0,026	+2	+2

4.3 Anti-corrosion protection

When the large segments were completed, the underwater 33-meter-long parts were protected against corrosion with epoxy coating before final assembly. In the field of offshore engineering the automatic rolling blasting machine with CNC system (computer numerical control) was widely adopted to steel piles instead of traditional manual sand blasting, which could greatly improve the working efficiency. The following Table 2 shows the comparison between them. The adopted CNC system in this project could treat a surface with maximum of 240 m^2 per hour, so that the uncoated pile could be shot blasted automatically in a total period of within 1 hour. As required the prime must be completed within 4 hours after blasting, in case the surface would be oxidized again. Except of high efficiency, the surface treatment was much more uniform than manual, the requirements of cleanliness Sa 2.5 and surface roughness 50 - 85μm could be achieved more easily.

Table 2 Comparison of work efficiency between CNC and manual

Item	Area [m^2]	Working efficiency	Blasting time	Labor
CNC-system	207,35	240 m^2 / hour	0,864 hour	1 operator
Blasting manual	207,35	6,4 m^2 / manhour	4 hours	8 workers

5 Quality control

The execution class of steel piles was EXC4 B+, as the strictest requirement of steel structures production in the EU. Compared to class EXC3, there was a noticeable difference located in the non-destructive tests, particularly in the visible inspection (VT) of quality level B+, as shown in Table 17 “Additional requirements for quality level B+” in EN 1090-2: 2008+A1 [2]. Undercut is not allowed and must be removed for EXC4 B+, and the other visible imperfections such as weld spatter, cracks, cavities and not permitted imperfections must be removed in time and couldn’t be deposited on the further processing steps.
The NDT tests including visible, ultrasonic, magnetic test (VT, UT and MT) were performed as approved proportion by NDT inspectors of the manufacturers, who had been qualified and certified as minimum level 2 according to ISO 9712 [10]. The full penetration longitudinal and circumferential welds were adopted 100 % percentage of ultrasonic and magnetic test, which is stricter than the corresponding requirements of Table 24 in EN 1090-2: 2008+A1 [2]. In addition, both of consulting engineers and the third party checked the products randomly as well.

6 Logistic and transport

After the final inspection the piles were moved to storage area and ready for uploading. Usually the flat barge as best choice is used for pile transport at short distance, but for this project there are two main difficulties, which are the extra-long piles and the long voyage. The vessel departed via the route from China, passing through Strait of Malacca, Indian Ocean and the Suez Canal and arriving to Croatia.

Figure 4 Cargo vessel with extra-long steel piles arrival on site

Facing such a great challenge the contractor considered the influence factors thoroughly. Firstly, the convenience and safety of uploading in the manufacturers and unloading on bridge site had to be taken into account. The draught on departure and on arrival should be deep enough for anchoring of vessel safely. As transport demanded, the vessel deck had enough strength to afford an approximate load capacity of 5.500 tons. Secondly, in order to minimize or avoid the rent of floating crane and other barges, the better option was for cargo vessel to be equipped with its own deck crane for up- and unloading. Thirdly, according to pile driving rate and production capacity of the manufacturers, it was necessary that the delivery time to destination be around month to ensure an uninterrupted pile driving. As a result, the vessel had to be equipped with adequate strong power. According to the mentioned comprehensive consideration, finally, the large heavy lift vessel was adopted to extra-long steel piles transport. The piles were banded and fixed on the deck with riggings and welded steel structure for safety. Meanwhile, the rubber mat and stow-wood were adopted to protect the contact areas from the damage of coating. Based on the dynamic plan the steel piles of 31.000 tons were sent to site in six batches respectively. See Figure 4.

7 Conclusion

In order to meet the great demand of the increasing infrastructure of sea-crossing bridges and offshore wind power worldwide, the underwater steel pile foundation has to now be developed at a rapid pace. The piles have to be longer and with a greater diameter gradually. The advanced CNC-system and mechanized welding process were used widely, on one hand it could reduce man-made effect on product quality, on the other hand, it improved the working efficiency obviously. The contractor has achieved a huge success in Pelješac Bridge Project by selecting a professional supplier of steel piles. It means that China's heavy industry is advancing forward at a global level with high-quality and efficient services.

Reference

[1] ISO 12944-5: 2018 Paints and varnishes – Corrosion protection of steel structures by protective paint systems – Part 5: Protective paint systems

[2] EN 1090-2: 2008+A1 2011 Execution of steel structures and aluminum structures – Part 2: Technical requirements for steel structures

[3] ISO 3834-2: 2005 Quality requirements for fusion welding of metallic materials – Part 2: Comprehensive quality requirements

[4] ISO 9606-1: 2012 Qualification testing of welders – Fusion welding – Part 1: Steels

[5] ISO 14732: 2013 Welding personnel – Qualification testing of welding operators and weld setters for mechanized and automatic welding of metallic materials

[6] ISO 15614-1: 2017 Specification and qualification of welding procedures for metallic materials – Welding procedure test – Part 1: Arc and gas welding of steels and arc welding of nickel and nickel alloys

[7] EN 10025: 2005 Hot rolled products of structural steels

[8] EN 10204: 2004 Metallic products. Types of inspection documents

[9] EN 10219-2: 2006 Cold formed welded structural hollow sections of non-alloy and fine grain steels – Part 2: Tolerances, dimensions and sectional properties

[10] ISO 9712: 2012 Non-destructive testing – Qualification and certification of NDT personnel

STUDY OF FILMING CONDITION FOR DEEP LEARNING BASED CRACK DETECTION METHOD

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Abstract

Recently, the study of extending the service life of bridges has gained attention. In Japan, there are about 730,000 bridges with a length of 2 m or longer, and many of these were built during a period of high economic growth, and have now reached the end of their service life. Therefore, their rebuilding and the extension of their service life must be considered. However, some local public organizations have problems that insufficient manpower relative to the number of bridges to manage, as well as insufficient funding for maintenance. Thus, these organizations are unable to perform routine close visual inspections. Specific problems include "notably less staff and consulting technicians relative to the number of bridges to be managed" and "high inspection cost preventing from funding for repair." As issues with the continuing close visual inspection of bridges are surfacing, the remote imaging system is expected to become a new inspection method that replaces close visual inspection. The practical potential of bridge inspections using images captured with a super-high-resolution camera was examined. A super-high-resolution camera enables us to take a wide area picture of a target bridge from a long distance. An image processing method could improve the efficiency of image-based inspection method. For example, a deep learning-based image processing method could extract a damaged area on a surface of a bridge automatically with high accuracy faster than human inspection. In general, the accuracy of an image processing method is affected by the quality of an input image. Filming conditions are one of the factors that determine the quality of a photo image. It is important to evaluate the effect of filming conditions to improve the reliability of an image processing method. In this paper, we evaluate the effect of the filming conditions for an image processing method by comparing the results of a deep learning-based crack detection method.

Keywords: bridge inspection, crack detection, image processing, filming conditions, deep learning

1 Introduction

Recently, the study of extending the service life of bridges has gained attention. In Japan, there are about 730,000 bridges with a length of 2 m or longer, and many of these were built during a period of high economic growth, and have now reached the end of their service life. Therefore, their rebuilding and the extension of their service life must be considered. An owner of a bridge is required to monitoring a bridge with close visual inspection per 5 years according to the national criteria in 2014 in Japan.

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However, some bridges owned by a local government have not completed the inspection due to a lack of an engineer of bridge inspection or budget. Such bridges are not expected to manage with aggressive preventive maintenance in the future.
One of the reasons for this problem is the expensive cost of close visual inspection for a bridge. Some bridges are hard to access for engineers to inspect. Such bridges need scaffolding or an expensive special car to perform a close visual inspection. It increases the cost of the inspection. In some case, bridge inspection needs traffic control and it makes economic loss. So more reduce economical cost and simplified process of inspection method is required for a future bridge maintenance inspection and already many novel inspection methods are proposed [1], [2], [3]. We focus on the remote imaging system which is expected to become a new inspection method that replaces close visual inspection [4]. In this system, an engineer inspects the bridge by photo image of the target bridge. So engineer no needs to closing the target bridge to inspect. This system can solve many problems of current close visual inspection.
The practical potential of bridge inspections using images captured with a super-high-resolution camera was examined [4]. A super-high-resolution camera enables us to take a wide area picture of a target bridge from a long distance. An image processing method could improve the efficiency of the image-based inspection method. For example, a deep learning-based image processing method could extract a damaged area on a bridge surface automatically with high accuracy and faster than human inspection. In general, the accuracy of the image processing method is affected by the quality of the input image. Filming conditions are one of the factors that determine the quality of the photo image. Setting appropriate filming conditions is one way to make the quality of the photo high. But the weather changes very often and all bridges are not located on a plane field. It is difficult to control filming conditions in real bridge inspection. So, it is important to evaluate the effect of filming conditions to improve the reliability of the image processing method. Unfortunately, there is no discussion about the effect of filming conditions for accuracy of image processing method using real crack on a concrete building.
In this paper, our purpose is not discovering appropriate filming conditions but measuring the degree of effect of filming conditions to image processing methods. We focus on two factors of filming conditions: the distance of the camera and the lighting. We evaluate these factors using a photo image of a real crack on the surface of a concrete building. We compare the results of the deep learning-based crack detection method to evaluate the effect.

2 Related works

There are many kinds of method for detecting crack on a concrete surface by image processing. Fujita et al. [5] proposes a crack detection method considering the effect of noise such as irregular shading and blemishes. They focus on the robustness of crack detection. They did not evaluate the effect of noise. A supervised machine learning method for crack detection is mainstream recently. Bu et al. [6] proposed Support Vector Machine based crack detection method. They applied three feature extract methods from the input image; Zenki moment, carbon filter, and wavelet transformation. Convolutional Neural Networks is one of the popular deep learning methods for image processing. Many researchers use it for the crack detection task. Cha et al. [7] prepared 40000 images for the training model of crack detection. This method can surround crack on a concrete surface image with a bounding box and has around 98 % precision for detection.

3 Evaluation

We evaluate the effect of the filming conditions by shifting the distance of a camera and change the light of a field. We compare the precision and recall of crack detection rate for the evaluation.

3.1 Image processing based crack detection method

We adapt the semantic segmentation method as an image processing based crack detection method. We use DeepCrack [8] model for crack detection. This model can output crack area with a pixel unit (Fig. 1). This model trained with a paired input image: raw image and annotation image (Fig. 2). We used 13,700 concrete bridge surface images as training data. All image has same size 256 × 256 pixel. 137 images are extracted from the photo image of real concrete bridge surface. Such photo images had taken by a super-high-resolution camera which has a resolution of about 100 million pixels count. The other images are created by augmentation with the deep learning method [9]. We have trained 137 augmentation models with the above 137 images to generate concrete surface images. We created augmentation images by 137 annotation patterns (Fig. 3).

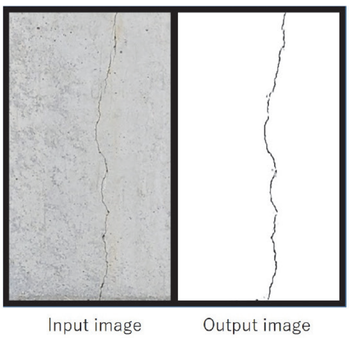

Figure 1 Example of crack detection by DeepCrack

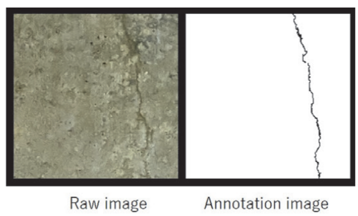

Figure 2 Example of training data set

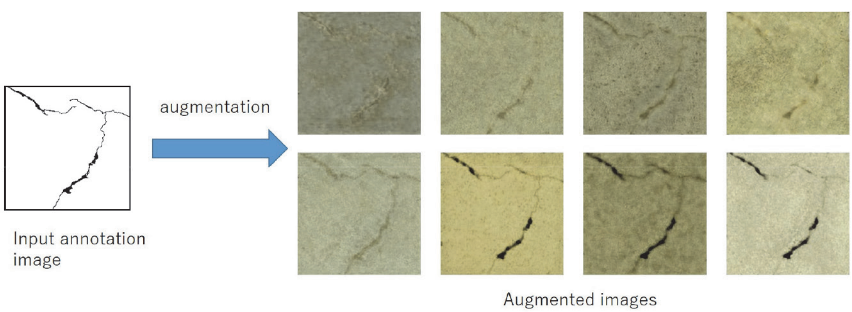

Figure 3 Example of image augmentation

3.2 Test dataset

We set four different cracks on a concrete building as a target crack of test data. We used a super-high-resolution camera which has a resolution of about 100 million pixels count to take a picture. We set six filming points to evaluate the effect of filming distance (Fig. 4). We put two floodlights (intensity is 5500 lumen) near the target crack (distance is 50 cm). To evaluate the effect of light, we set a four-light condition: both lights are turned off, the right one is running, the left one is running, both lights are running (Fig. 5).

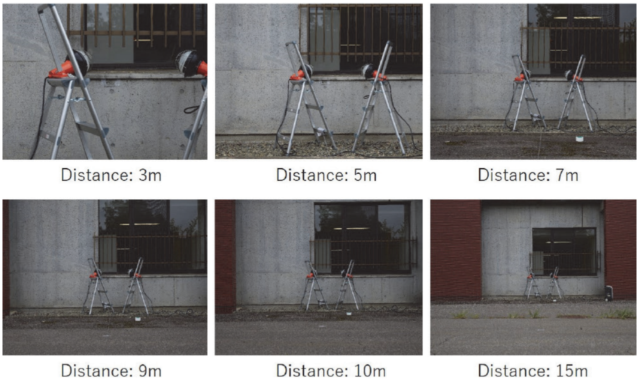

Figure 4 Example of each shooting distance images

Figure 5 Example of light conditions

In this paper, we focus on the effect of filming conditions. But, the target crack area is too small than the original photo image size. It would make decrease the precision of crack detection and make it difficult to evaluate the effect of filming conditions. All test images are cropped image of a crack area of each test data (Fig. 6).

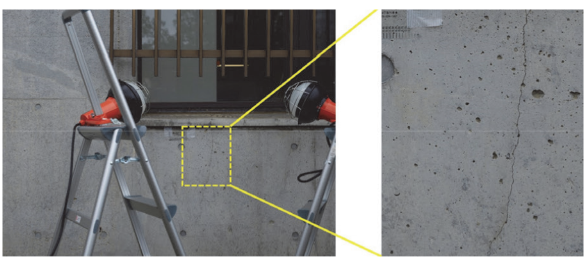

Figure 6 Example of test image cropped target crack area

The crack annotation data of test data made by tracing crack area on the cropped images at each shooting distance. Note that different shooting distance annotation data are not the same (Fig. 7). The annotator could trace small crack on image taken by close shooting distance. On the other hand, the annotator could distinguish only a big crack on an image taken by far shooting distance. The crack annotation data made by one annotator.

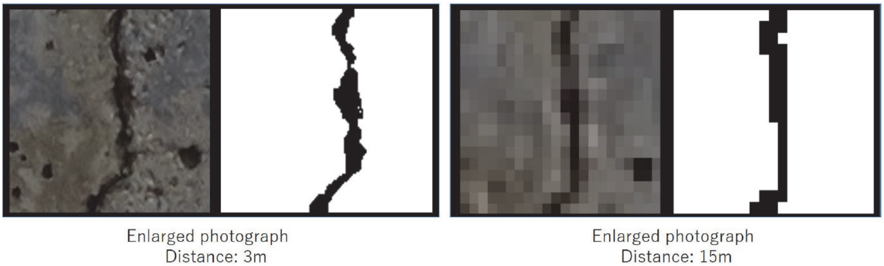

Figure 7 Comparison of enlarged raw crack images and annotated images

3.3 Results of crack detection

We evaluate the effect of shooting distance and light field by comparing the precision and recall. These values are results of image processing for detect cracks on the concrete surface of each condition. We show the average results of crack detection of four different crack images. The precision and recall of crack detection were calculated by the rate of concordance of the results of detection by the image processing method and annotator annotated crack in pixel unit.
Fig. 8 show the precision values of different shooting distance and light of field. When focusing on the change in shooting distance, the worst precision value is 0.280 and the best precision value is 0.423. The difference is 0.143. Note that the worst precision value is not the result of the farthest shooting distance and the best precision value is not the result of the nearest shooting distance. These results show that setting the best shooting distance with low overdetection is difficult. When focusing on the change in light of field, we compare the case of both lights off and the other case. When comparing the precision values, it decreases 0.036 in the worst case and increases 0.023 in the best case. The best light condition of each shooting distance is different.
Fig. 9 show the recall precision values of different shooting distance and light of field. When focusing on the change in shooting distance, the worst precision value is 0.542 and the best precision value is 0.859. The difference is 0.317. The recall value of far-shooting distance becomes higher than one of close-shooting distance. It is because of a difference in resolution of test image data. In the case of shooting distance is far, an annotator could not distinguish a small crack on the image. Because a big crack which is easy to detect by crack detection remains in far shooting distance data, the recall value becomes high when shooting distance is far (Fig. 10). Note that this result does not ensure any far shooting distance make high recall value always. When the surface of the target concrete has a narrow crack only, the recall value may become low. When focusing on the change in light of field, comparing the recall value as with comparing of precision value. When comparing the precision values, it decreases 0.084 in the worst case and increases 0.022 in the best case. As same as the precision values, the best light condition of each shooting distance is different. We should evaluate the mean error in future.

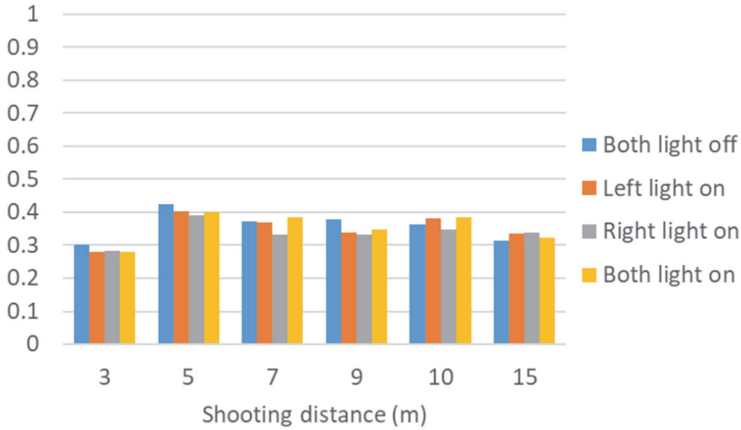

Figure 8 Results of precision

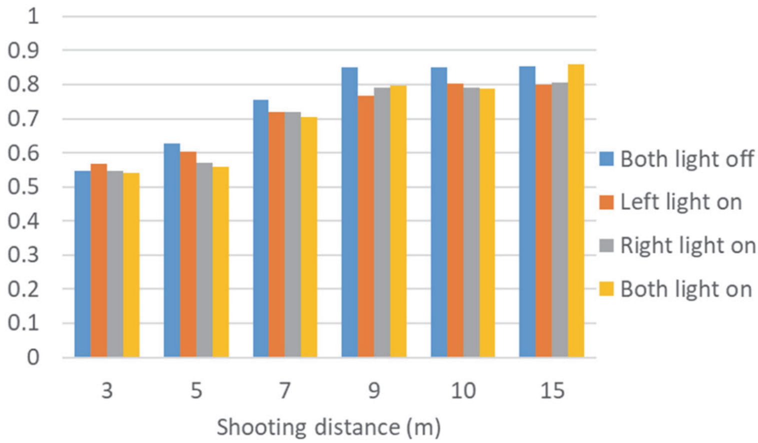

Figure 9 Results of recall

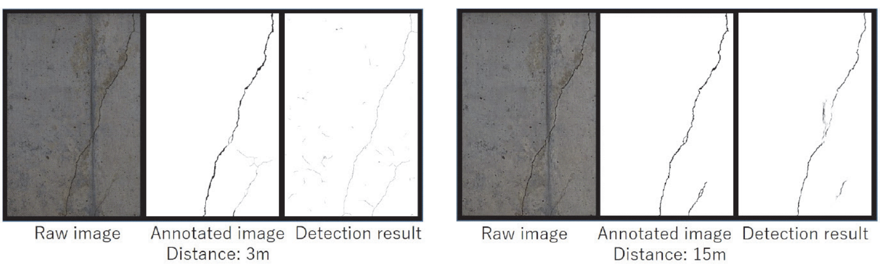

Figure 10 Comparison of different shooting distance

4 Conclusion

A bridge inspection needs much cost. So an efficient bridge inspection method is necessary for future japan. One of the alternative methods of the current close visual inspection is the image-based bridge inspections using images captured with a super-high-resolution camera. An image processing method could make efficient an image-based inspection but be affected by filming conditions of the input image.
We have evaluated the effect of shooting distance and light of field for deep learning-based crack detection by comparing precision and recall of crack detection results. The results of the evaluation show the concrete effect of filming conditions.
The results of this evaluation are not robust because of the small test dataset. In future work, we make a big test dataset by decreasing filming conditions and target crack on a concrete surface. We will make clear that a robust effect of filming conditions and the reason for filming conditions changes the results of crack detection.

References

[1] Menendez, E., Victores, J.G., Montero, R., Martínez, S., Balaguer, C.: Tunnel structural inspection and assessment using an autonomous robotic system, Automation in Construction, 87 (2018), pp. 117-126

[2] Pham, N.H., La, H.M., Ha, Q.P., Dang, S.N., Vo, A.H., Dinh, Q.H.: Visual and 3D mapping for steel bridge inspection using a climbing robot, SARC 2016 - 33rd International Symposium on Automation and Robotics in Construction, pp. 141-149, 2016.

[3] Xie, R., Yao, J., Liu, K., Lu, X., Liu, Y., Xia, M., Zeng, Q.: Automatic multi-image stitching for concrete bridge inspection by combining point and line features, Automation in Construction, pp. 265-280, 2018.

[4] Minami, H., Fujiu, M., Takayama, J., Suda, S., Okumura, S., Watanabe, K.: A basic study on diagnostic imaging technology for bridge inspection using super high resolution camera, Journal of Sociotechnology Research Network, 15 (2018), pp. 54-64

[5] Fujita, Y., Mitani, Y., Hamamoto, Y.: A method for crack extraction on concrete surfaces using Image processing techniques, Journal of The Japanese Society for Non-Destructive Inspection, 56 (2007) 7, pp. 371-377

[6] Bu, G., Chanda, S., Guan, H., Jo, J., Blumenstein, M., Loo, Y.-C.: Crack detection using a texture analysis-based technique for visual bridge inspection, Electronic Journal of Structural Engineering, 14 (2015) 1, pp. 41–48

[7] Cha, Y.-J., Choi, W., Büyüköztürk, O.: Deep learning-based crack damage detection using convolutional neural networks, Computer-Aided Civil and Infrastructure Engineering, 32 (2017) 5, pp. 361-378

[8] Qin, Z., Zheng, Z., Qingquan, L., Xianbiao, Q., Qian, W., Song, W.: DeepCrack: Learning Hierarchical Convo-lutional Features for Crack Detection, IEEE Transactions on Image Processing, 28 (2019) 3, pp.1498-1512

[9] Fukuoka, T., Minami, T., Urata, W., Fujiu, M., Takayama, J.: A data augmentation with Pix2Pix for deep learning method for detecting crack on an actual concrete, Journal of Japan Society of Civil Engineers, Ser. F4 (Construction and Management), 17 (2019) 2, pp. I_27-I_35

EVOLUTION OF BRIDGES WITH STEEL-CONCRETE COMPOSITE SUPERSTRUCTURE. WHAT COMES NEXT?

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Abstract

Bridges have evolved over time from the simplest forms made from materials found in nature – wood and stone - to the complex shapes of today, made of concrete, steel, steel-concrete, and composite materials. If in the past the large dimensions of an obstacle impeded building a bridge, today this problem can be solved by choosing suitable materials, an advantageous structural system, and an erection method that favors the chosen solutions. The composite superstructures made of steel-concrete have started to be used more often in the construction of bridges due to their advantages. The scope of this paper is to analyze the evolution of road bridges with steel-concrete composite superstructure. There can be distinguished mainly 4 stages in the evolution of these types of structures. In the first two stages during 1850-1925 the connection between concrete and steel was achieved by the adhesion between the contact surfaces of the two materials. Starting with 1932 (stage III), a connection was realized that takes over the forces of friction that develop at the contact between the two materials. These connecting elements took different forms: loops the reinforcement, U, L, or ⊥ metal parts, shear stud connectors, and more recently composite dowels. The advantages of different types of connectors have been highlighted by various calculation methods, practical applications, and high productivity. Nowadays the construction of an impressive bridge has become a source of pride at an international level, a way to demonstrate the technological progress in the field. But what does the future hold in the field of composite structures made of steel-concrete?

Keywords: composite, superstructures, evolution, future

1 Introduction

The steel-concrete composite structures offer extremely efficient solutions for bridges. The advantages of these types of structures result through the judicious placement of the constituent materials of the element - steel, concrete, and reinforcement - aiming, as far as possible, the concrete to be placed in the compressed area, and the steel in the tensile area, [1].

2 Evolution of road bridges with steel-concrete composite structure

The history of steel-concrete composite structures is as old as the history of reinforced concrete. The evolution of steel-concrete structures can be divided into 4 stages.

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2.1 Stage I (1850-1900)

The first concepts of steel-concrete composite structures were used on the floors of residential buildings. From the 1800s, different variants of metal used in combination with concrete began to appear. In 1848 Nathaniel Beardmore (1816-1872) patented a floor that used riveted I-beams with lost metal formwork and concrete filling between the I-beams [Fig. 1a], [2].

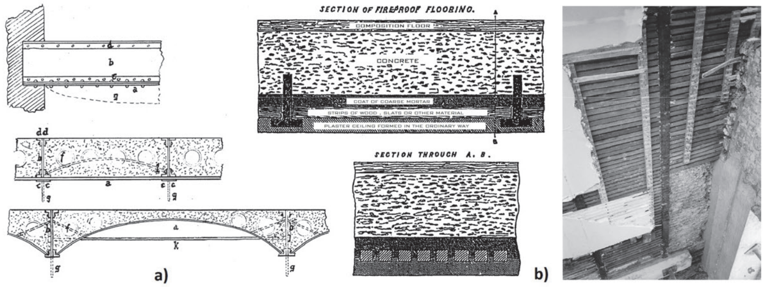

Figure 1 a) Suspended floor by Nathaniel Beardmore [2]; b) Suspended floor with cast-iron beams by Fox & Barrett [3]

On the fire-resistant floor patented by Henry Hawes Fox in 1844, cast iron was first used, but, from 1851 on, I or ⊥ steel beams became more common. The beams were partially positioned outside the concrete section, the compression force being taken over by the concrete section and the tensile force by the metal elements [Fig. 1b], [2].
In 1892 François Hennebique patented the use of flexible reinforcement for concrete elements and is considered the initiator of the use of reinforced concrete. However, it was necessary to use steel profiles to support the formwork or in the case of structures with larger spans. Although discussions were combining these two materials, only1902 that Fritz Pohlmann patented in Germany the steel-concrete composite structure, where the shear force between the two materials was taken over by the loops made of metal plate and the holes in the web of the metal beam [Fig. 2], [2].

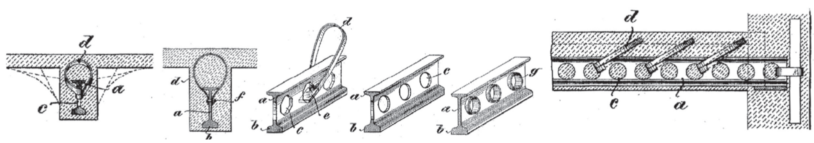

Figure 2 The Fritz Pohlmann floor [4]

2.2 Stage II (1900-1925)

As early as 1892, Mathias Koenen (1849–1924), made a structure where the tensile efforts were taken over by the flexible reinforcement and the compression efforts by the concrete section [Fig. 3]. He also used steel profiles embedded in concrete to make floors with large spans. Concrete has begun to be regarded as a composite material but no difference has yet been made between flexible reinforcement and rigid reinforcement (metal profiles) [2].

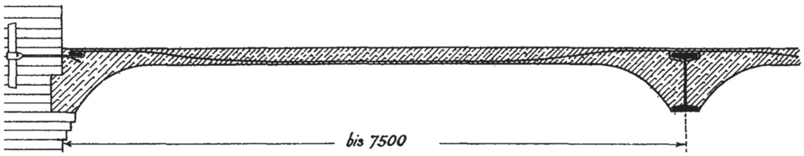

Figure 3 Section of Mathias Koenen's floor [2]

The tests carried out between 1907 and 1909 in Stuttgart by Carl von Bach (1847–1931) showed that as the load increased, in the elements having steel profiles, the cracks were more pronounced than in the elements that had flexible round bars. Also the phenomenon of dislocation of the concrete surrounding the metal profile appeared. Mathias Koenen draws attention to this dangerous phenomenon. Carl von Bach and Mathias Koenen admit that the adhesion between the concrete and the metal profile is less efficient than in the case of flexible round bars. However, in the design prescriptions developed during that period, no different rules are specified for the two types of reinforcement [2].

As early as 1904 Rudolf Saliger (1873–1958), an initial teacher in Kassel, then in Vienna, recognizes that once the adhesion between the contact surfaces of the two materials - concrete and metal - is lost, the resistance is reduced substantially. He recommended in 1920, as a special measure, the realization of a connection between the concrete and the metal profile by installing connectors in the form of plates bent at 45°. But his recommendation is not taken into account [2].

After understanding the vaulted floors, Joseph Melan (1853–1941) proposed a system that used spatial metal structures embedded in concrete. The steel structure was dimensioned to take over part of the wet concrete loading during the execution phase, and after hardening, both materials contributed to the bearing of the load [Fig. 4]. Thus, starting with the mid-1880s, the Melan system began to be used on bridges, and in1924 in the USA already being built over 500 bridges using the Melan system [5].

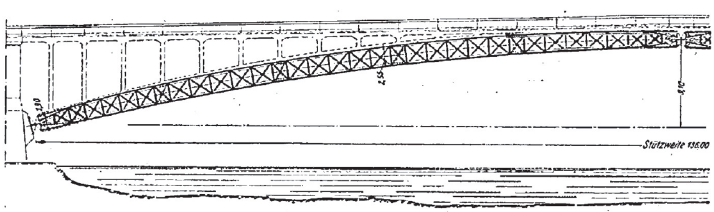

Figure 4 View of Elbbrücke Dresden – Melan System [5]

In 1902 José Eugenio Ribera patented a system similar to the system proposed by Melan, in which the entire weight of the concrete in the execution phase was taken over by the rigid reinforcement. After the hardening of the concrete, both materials contributed to the taking over of the loads [6].

For bridges with smaller spans, the solution of metal beams embedded in concrete was cheaper, especially for railway bridges. Otto Kommerell (1873–1967) proposed that this type of structure should no longer be considered composite and that the entire load would be taken over by the metal beams. The concrete should have the role of distributing the variable actions. He also proposed that where the height of the apron is greater, part of the web and the lower flange of the beams should not be concrete. In the drawings, he did not explain the role of the bars connecting the metal beams [Fig. 5] [8].

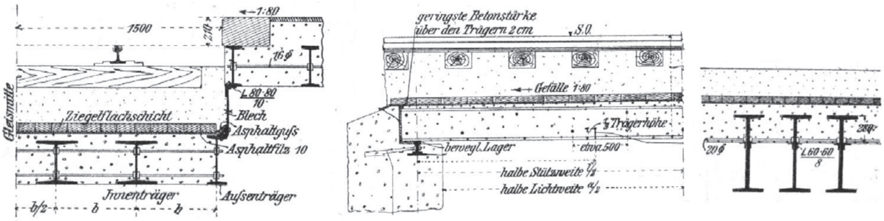

Figure 5 Typical detailing for railway bridge with rolled beams encased or partially encased in concrete [8]

Gradually the bridges made entirely of metal were replaced by those that included - in whole or in part - metal beams in concrete. In the first stage, the beams were fully embedded in the concrete, then only the upper flange and part of the web were integrated. The concrete slab was made of reinforced concrete and the bond with the steel beams was achieved by the adhesion between the contact surfaces of the two materials. One of the first bridges built in Europe (1914) with this solution is Achereggbrücke on Lake Lucerne [Fig. 6] [8].

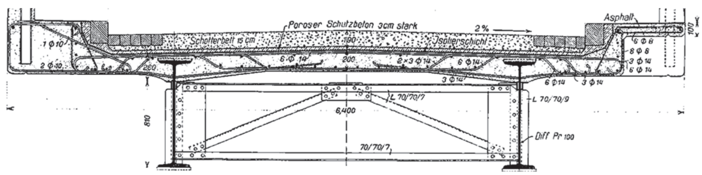

Figure 6 Section through Achereggbrücke on Lake Lucerne [8]

2.3 Stage III (1925-1950)

Even though in 1920, Rudolf Saliger recommended making a connection between the elements of the section by installing connectors, it was only in 1932 that connectors began to be assembled to achieve a connection between the two materials, going with the idea of a composite structure. At first, the connectors were arranged constructively in the form of round steel welded spirals. Gradually the connectors began to play a role of resistance, taking over the forces of friction that develop at the contact between the two materials [Fig. 7], [11].

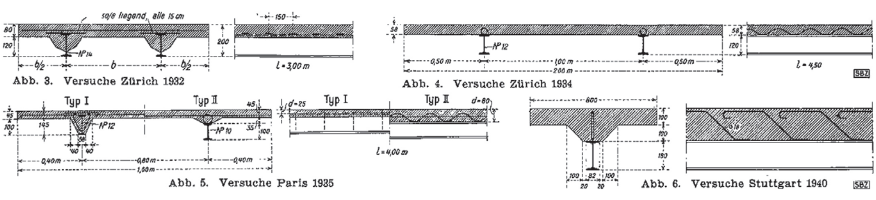

Figure 7 Various connection types [11]

Starting with 1932 the Verbandes der Schweizerischen Brücken- und Eisenhochbau-Fabriken (T.K.V.S.B.) decided to carry out experiments that first documented the elastoplastic behavior of the composite section. Several types of connectors were tested, but in principle, they were made of round bars bent at 45°, welded, and positioned in the longitudinal direction of the steel beams. Static and dynamic tests were carried out complementing the knowledge of the time [12, 14]. The Swiss become European leaders in the construction of composite structures. The Steinbach and Willerzell bridges over Lake Sihlee are the first European bridges that use specially dimension connectors to take over the shear between the two materials in the form of ⊥ welded to the upper flange [Fig. 8] [15].

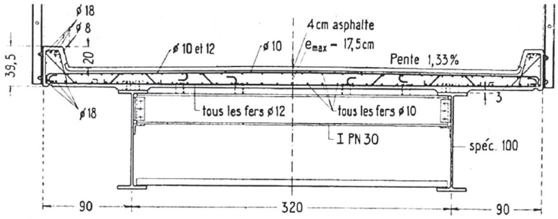

Figure 8 Section through Willerzell bridge on Lake Sihlee [15]

Typical for that period is the bridge over the Sava river in Zagreb. It has four spans and was built between 1938-1939. The connection between the main beam and the concrete slab was made by flexible connectors [Fig. 9] [16].

Figure 9 Bridge over the Sava river in Zagreb, Croatia [16]

In Spain, Puente de Tordera was completed in 1939. It was a bridge with a composite structure and had connectors in the form of round bars, [2].
In the U.S.A., bridges built in New York had to be light and resistant. That is why starting with Goethals Bridge (1928), George Washington Bridge (1931), and Tribourough Bridge (1936) the connection between the two materials (concrete and metal) has gradually improved, [2].

2.4 Stage IV (1950-today)

After the end of the Second World War, in Europe starts a program with the purpose of reconstructing the destroyed infrastructure during the war. That was the start of the development of composite structures. In 1950 the first set of rules for designing composite structures bridges was created and in 1956 DIN 1078 was adopted in Germany, [2].
Connectors made of round bars bent at 45° and welded on the upper flange of the beams were elastic. While those made of U, L, or ⊥ metal parts, completed by some loops in the reinforcement were rigid [Fig. 10a] [2].
Various discussions about connectors, in the end, led to the appearance of shear stud connectors in the 1960s. They were connected to the upper flange of the beams by welding. They proved to be very efficient because they had good behavior, high productivity, and they were easy to install due to the automation of the welding process [Fig. 10b] [2].

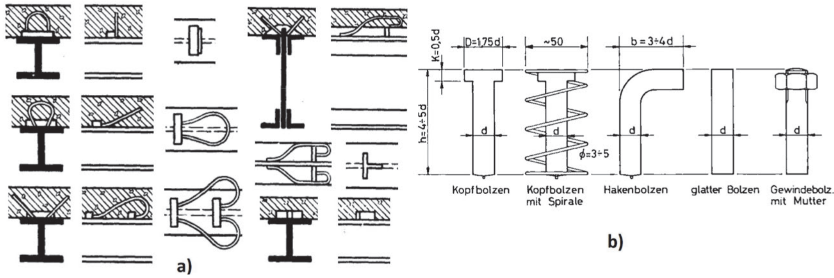

Figure 10 a) Typical shear connectors [2], b) Shear stud connectors [17]

Research has continued to focus on plastic analysis and how this type of structure influences the shrinkage and creep of concrete. The possibility of prestressing the concrete slab was also studied [2].

If until the 1950s composite structure bridges could not compete with those made of prestressed concrete, then this type of structure was used especially for bridges with large spans and thin decks [2].

Since that period, the evolution of steel-concrete composite structures has been closely related to the qualitative evolution of material characteristics, the improvement of design methods, and the development of manufacturing and execution technology. Figure 11 presents some typical composite bridge cross-sections which are used mainly nowadays.

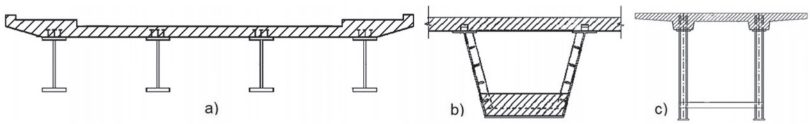

Figure 11 a) multi-girder section, b) double composite section for large span bridges above intermediate support, c) truss composite section [18]

Starting with 1998, prefabricated composite beams began to be used in the construction of bridges. This type of beams consists of a steel beam located at the bottom, thus taking over the tensile efforts, and at the top a formwork element made of reinforced concrete. The main advantage of these types of beams was the fact that they have a short time of assembly and fulfillment of quality standards due to their execution in superior conditions to those on-site [19].

The most expensive material in the composite section is steel, thus the engineers focused on optimizing the section based on internal stress distribution. Consequently, the upper flange was removed from the compressed area, the compression efforts being fully taken over by the reinforced concrete slab. The advantage of such a solution can be seen in Fig. 12 [8].

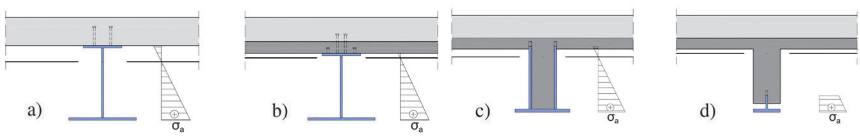

Figure 12 Stress distribution: a) conventional composite section, b) VFT® – construction method; c) VFT-WIB® – construction method; d) Section with external reinforcement (also VFT-WIB® section) [20]

By cutting the steel profile, an attempt was made to create a shape that would take the role of stud connectors - which were welded to the upper flange. The new shape of the steel profile is called composite connectors and they have had different shapes over time [Fig. 13]. This type of beam is called VFT® or VFT-WIB®. By cutting longitudinally an I type profile, according to a certain geometry, two identical T type profiles were obtained [20].
If the prefabricated beam has a length that cannot be transported, it can be formed of two sections that will be joined on-site and then mounted in the final position. The solution of joining the two sections is suggested in Fig.14 [21].

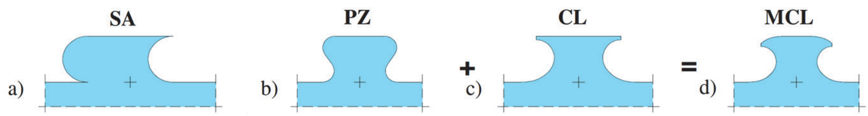

Figure 13 The shape of composite dowels: a) fin (SA), b) puzzle (PZ), c) clothoid (CL), d) modified clothoid (MCL) [20]

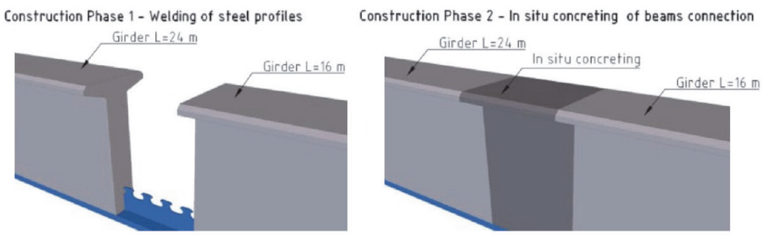

Figure 14 In situ connection of two sections of beams [21]

3 Conclusions

The use of steel and concrete in a unitary structural system took place long before the exact mechanical behavior of the composite elements was known. The connection was achieved by the adhesion between the contact surfaces of the two materials in the first two stages (1850-1925). Connecting the two materials has been discussed since 1932. There have been various variants of connectors studied over time. However, the development of calculation models and their validation in practice has revealed the advantages of using the two materials in a composite system using shear stud connectors and, more recently, composite dowels.
What does the future hold for us? The future will likely be of prefabricated elements because they can be made of superior quality, where it is most cost-effective. The composite connectors may have application not only in the case of beam bridges but also in the case of cable-stayed or suspended bridges, gradually replacing the stud connectors. It is not possible to say exactly what the direction will be, but one thing is sure, the progress made so far in the field has been conditioned by the technology of execution, the quality of materials, the improvement of dimension methods, and the disposition of governments to invest. Certainly, these factors will further determine the progress in the field of steel-concrete composite structures used in the construction of bridges.

Regerences

[1] Guțiu, Ş. I., Moga, C.: Structuri compuse oțel-beton, UT Press; 2014.

[2] Pelke, E., Kurrer, K. E.: Zur Entwicklungsgeschichte des Stahlverbundbaus, Stahlbau, 85 (2016) 11, pp. 764-780, doi: 10.1002/stab.201610429

[3] Sutherland, J., Sutherland, R. J., Humm, D., Chrimes, M.: Historic concrete: Background to appraisal, Thomas Telford, 2001.

[4] Concrete or like girder (United States Patent Office), https://patents.google.com/patent/US766899A/en, 03.03.2020

[5] Spangenberg, H.: Eisenbetonbogenbrücken für große Spannweiten, Der Bauingenieur, Heft 16, pp. 503-512, 1924.

[6] Un procedimiento de construcción aplicable a bóvedas y puentes de hormigón armado (Oficina Española de Patentes y Marcas), http://www.cehopu.cedex.es/hormigon/temas/H22.php?id_tema=11, 05.03.2020

[7] Eggemann, H., Kurrer, K.E.: Zur internationalen Verbreitung des Systems Melan seit 1892: Konstruktion und Brückenbau, Beton und Stahlbetonbau, 101 (2006) 11, pp.911-922, doi: 10.1002/best.200690163

[8] Kommerell, O.: Tafeln für eisenbahnbrücken aus Einbetonierten Walzträgern, Zentralblatt der Bauverwaltung, 78 (1911), pp. 481–484

[9] Rohn, A.: Neubau der Achereggbrücke über die Seeenge des Vierwaldstättersees bei Stansstad, Schweizerische Bauzeitung 66 (1915) 23, pp. 263–267, doi: 10.5169/seals-32326

[10] Voellmy, A.: Eisen-Beton-Verbundkonstruktionen Alpha, Schweizerische Bauzeitung, 103 (1934) 22, pp. 258–261, doi.org/10.5169/seals-83221

[11] Albrecht, A.: Der Verbundträger, Schweizerische Bauzeitung 125 (1945) 2, pp. 11–15, doi.org/10.5169/seals-83582

[12] Albrecht, A.: Der Verbundträger, Schweizerische Bauzeitung 125 (1945) 3, pp. 30–33, doi.org/10.5169/seals-83591

[13] Albrecht, A.: Der Verbundträger, Schweizerische Bauzeitung 125 (1945) 4, pp. 37–41, doi.org/10.5169/seals-83593

[14] Maier-Leibnitz, H.: Versuche über das Zusammenwirken von I-Trägern mit Eisenbetondecken, Die Bautechnik 19 (1941) 25, pp. 265–270

[15] Wichser, O.: Noveaux ponts – routes sur le Sihlsee, L'ossature Métallique 6 (1947), pp. 267-272

[16] Flaga, K., Furtak, K.: Application of composite structures in bridge engineering. Problems of construction process and strength analysis, Civil and environmental engineering reports, 2014, pp. 57-85, doi: 10.1515/ceer-2014-0035

[17] Schlaginhaufen, R.: Zur Anwendung der Bolzendübel: Vortrag, Schweizerische Bauzeitung, 83 (1965) 41, pp. 721–724, doi.org/10.5169/seals-68276

[18] Guțiu, Ş. I., Moga, C.: Poduri metalice: suprastructuri, UT Press, 2018.

[19] Schmitt, V., Seidl, G., Hever, M., Zapfe, C.: Verbundbrücke Pöcking–Innovative VFT Träger mit Betondübeln, Stahlbau, 73 (2004) 6, pp. 387-393

[20] Seidl, G., Petzek, E., Băncilă, R.: Composite dowels in bridges-efficient solution, Advanced Materials Research 814 (2013), pp. 193-206, doi: org/10.4028/www.scientific.net/AMR.814.193

[21] Petzek, E., Schmitt, V., Meteş, E., Ispăşoiu, G., Țurcan, A.: Romanian projects and integral bridge solutions based on composite dowels, Steel Construction, 9 (2016) 3, pp. 161-169, doi: org/10.1002/stco.201610022

A MODEL FOR ASSESSING THE PRIORITY OF THE BRIDGES WITHIN THEIR REPAIR STRATEGY

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Abstract

The efficiency of the road network operation significantly depends on ensuring the durability and operational reliability of road bridges. To prevent accidents on bridges, ensure their reliability and durability, it is necessary to perform a set of measures, including the following: inspection of bridges, determination of their operational condition, calculation of residual life, development of recommendations for their further operation, and performance of operational measures. Considering the significant quantity of bridges in Ukraine, most of these activities have to be performed using modern information technology. Therefore, an Analytical Expert Bridge Management System Software Complex (AESUM) for bridges of Ukraine on public roads was developed and implemented. An important component of this software complex is the procedures for development of a strategy of repairs in the system of road bridges operation. One of the components of the mathematical model, which is the basis for justifying the repairing strategy, is a system of priorities for bridges repairs, taking into account their importance. This scientific paper considers the current system of priority, which takes into account the technical and operational condition of bridges and the category of roads on which they are located. A new model for evaluation of the priorities of repairs is proposed which takes into account such factors as importance of the bridge, technical and operational condition of the bridge, traffic capacity of the bridge and the cost-effectiveness of repairs. The factors mentioned above consist of a set of sub-factors. For example, the bridge importance factor consists of such sub-factors as the average daily traffic volume, the bypass influence, the bridge's affiliation with international transport corridors, etc.

Keywords: road bridge, AESUM, bridges repairs strategy, bridges repairs priorities

1 Introduction

The maintenance service has an influence on the bridges condition and the amount of costs by choosing a certain sequence, nature and scope of repairs over time - the strategy of operation of the bridges. There is a big number of options for such strategies, so the issue of choosing the best strategy for one or more criteria is of the highest importance in justifying the strategy of bridges operation.

Providing the coordination of the degradation process and the rehabilitation process in space and time (rehabilitation strategy) under certain constraints is the content of the tasks (functions) of planning, organization, motivation and control of bridges conditions. Limitations in these tasks are the limit parameters of the degradation process and limitations on the resources (financial, labor, logistical, informational, etc.) (the principle of consistency).

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It is impossible to provide the optimal solution for one bridge without simultaneous consideration of other bridges on a certain network of roads, therefore, it is a question not only of the project of operation of an individal bridge, but, mainly, of the bridges operation program (principle of network level dominance). An important principle of rationale for the strategy of the technical bridges conditions rehabilitation is to take into account the external effects of of the bridge existence itself, i.e. its usefulness for consumers (consumer qualities) which can be assessed by traffic speed, permissible traffic loads and vehicles dimensions, safety and riding comfort, losses from closing the bridge for reconstruction, repair or replacement with a new bridge, etc. This principle should be called (utility principle). During implementation of this principle, the bridges are considered as integral parts of the road network, therefore, the deterioration of bridges causes inefficiency of the road network. Negative consequences include the cost of repair and reconstruction of bridges and increased costs for bridge users. Therefore, it is necessary to maintain bridges for securing their satisfactory values in their useful lifetime individually or as part of a road network [1]

2 One-level prioritization model

Bridges on the road network have different significance. Firstly, their significance is assessed by their operational conditions; secondly - by the impact on economic efficiency of transportation and traffic safety; thirdly, by the economic and social conditions of the adjacent region. So, they may have different priority (principle of priority) depending on a number of factors [2]. The list of factors can be quite wide, the main thing is to have access to information to calculate each of them, and that they meet the requirements for the expected result of their application. For example:
- average daily traffic volume;
- percentage of trucks in traffic;
- administrative importance of the road;
- category of the road;
- the nature of the bridge location;
- possible bypasses, length, traffic composition limitation;
- importance for defense;
- historical importance;
- compliance of the bridge dimensions with the traffic requirements.

The importance factor (priority) of the i-th bridge (Π_i), is determined by the formula:

$$\Pi_i = \sum_{k=1}^{m} w_k \cdot f_{ik} \tag{1}$$

where
w_k – specific weight of k-th factor $\sum_{k=1}^{m} w_k = 1$

f_{ik} – the dimensionless importance of the k-th factor of the i-th bridge takes values from 0 to 1.

During implementation of this principle, bridges are considered as integral parts of the road network, so the deterioration of bridges, in addition to safety issues, causes some inefficiency of the road network. This inefficiency can lead to significant negative economic consequences which include additional costs for repair and reconstruction of bridges and increased costs for bridge users. Therefore, it is necessary to maintain bridges for securing their satisfactory values individually or as part of the network [4].

Table 1 Samples of the score factors

Components of the importance factor	Score
Operational state of bridge	0.18
Daily traffic volume	0.15
Administrative importance of the road	0.12
Road category	0.11
The length of bypass when closing the bridge	0.10
Compliance of the bridge dimensions with the traffic requirements	0.14
Aggressiveness of the environment	0.05
Defense importance	0.10
Historical importance	0.05
Total	1.00

3 Two-level model of bridge prioritization

The proposed two-level model of bridge prioritization is based on the approaches presented in [5, 6]. According to these approaches, the priority of the bridge is a weighted indicator of five factors that are measured: the importance of the bridge, technical condition of the bridge, the design redundancy, the structure capacity and cost-effectiveness. The dimensionless value of each of these factors is multiplied by the score coefficient. The general form of the equation of priority П is:

$$П = a \cdot КВ + b \cdot РС + c \cdot КП + d \cdot КЗ + e \cdot КЕ \quad (2)$$

Where

a, b, c, d, e - score coefficients, a + b + c + d + e = 1.0;

КВ - coefficient of importance of the structure (Bridge Importance Factor) - the relative importance of the bridge in a particular network of roads;

РС (Condition State Factor) - a condition rating factor that characterizes the overall physical condition of the bridge based on the condition of each individual element;

КП (Design Redundancy Factor) - coefficient of adaptation (redundancy) - a measure of risk in the analysis of four important limit states: brittle fracture, emergency erosion, emergency destruction due to fatigue, destruction due to earthquake;

КЗ (Structure Capacity Factor) - traffic capacity of the bridge - the ability of the bridge to pass traffic, including the effect of restrictions on weight, bridge clearance and width dimensions;

КЕ (Cost-Effectiveness Factor) - cost-effectiveness factor characterizes the cost-effectiveness of a rehabilitation measure.

These factors indicate the relative importance of the structure (in the range from 0 to 1.0). For example, a structure with a score of 0.62 is more significant than a structure with a score of 0.43 for this.
The normalized score coefficients, a + b + c + d + e = 1.0, were selected for prioritization using the method of hierarchy analysis proposed by T. Saati [7].

The coefficients a, b, c, d, e for including five factors given in formula (2) are calculated using the so-called matrix of advantages (or comparisons) and the following formula is obtained:

$$П = 0.30 \cdot КВ + 0.25 \cdot РС + 0.15 \cdot КП + 0.10 \cdot КЗ + 0.20 \cdot КЕ \tag{3}$$

Table 2 Matrix of advantages for five factors (criteria)

	КВ	**РС**	**КП**	**КЗ**	**КЕ**	$\prod_{j=1}^{n} w_{i,j}$	$v_i = \left(\prod_{j=1}^{n} w_{i,j}\right)^{\frac{1}{n}}$	$v_i = \frac{v_i}{\sum_{1}^{n} v_j}$
КВ	1,00	3,00	2,00	2,00	1,00	12,0000	1,64	a=0,30
РС	1/3	1,00	2,00	4,00	4,00	10,6667	1,40	b=0,25
КП	1/2	1/2	1,00	1,00	1,00	0,2500	0,82	c=0,15
КЗ	1/2	1/4	1,00	1,00	1/9	0,0139	0,54	d=0,10
КЕ	1,00	1/4	1,00	9,00	1,00	2,2500	1,12	e=0,20
							Σ = 5,53	Σ=1,00

Importance of the bridge:

$$КВ=0,30 \cdot V_A+0,10 \cdot V_B+0,15 \cdot V_C+0,20 \cdot V_D+ 0,05 \tag{4}$$

Where
V_A - average daily traffic volume, car / day per traffic lane;
V_C - the same is for trucks, car / day per traffic the lane;
V_B - future average daily traffic volume, car / day per traffic lane, which is calculated taking into account increment rate of traffic volume;
V_D - the effect of bypassing the bridge;
$V_{Ед}$ - belonging of the bridge to the network of roads of state importance (yes - 1, no - 0);
V_F - belonging of the bridge to the transport corridor (yes - 1, no - 0).

Condition rating factor:

$$РС = 1.00\text{-}E / 100, \tag{5}$$

where E is the rating of the bridge (0 - 100)

Bridge traffic capacity:

$$КЗ=1,00-(0,35 \cdot ЗВ+0,50 \cdot ПГ+0,15 \cdot ШМ) \tag{6}$$

ЗВ - score reduction (from 0 to 1) - score characteristics of the structure relative to trucks;
ПГ - bridge clearance (from 0 to 1);
ШМ - correspondence of the width of the bridge to the width of the entrances of the highway ШМ (from 0 to 1).

Indices ЗВ, ПГ and ШМ in the absence of violations of technical parameters take the value of 1.0.

Cost-effectiveness factor

$$КЕ=C_з/C_{нм} \tag{7}$$

Where
$C_з$ - the cost of the operational measure
$C_{НМ}$ - the cost of a new bridge, UAH.

4 Application of priorities

Ranking and optimization are the two most widely used methods in selecting the repair project for the repair strategy. However, the concepts of these two approaches are very different. Rating methods assess simultaneously several related project factors and quantify the rating based on the assessment of these factors; therefore, all considered projects are evaluated according to their rating values.
Ranking methods do not necessarily give the optimal solution. Nevertheless, the rating approach is easy to use and provides a relative order of importance for different projects. It is most often used for short-term (annual) planning (distribution, prioritization) of repair works during the operation of bridges. In this case, the calculated priority of the building is included in the group of key indicators that determine an orderly list of solutions based on the rating indicators of the project.
On the other hand, optimization methods provide an “optimal” solution, where projects are selected subject to certain constraints. The optimal solution can be obtained by providing the maximum benefit from the bridge for the road network. Unlike ranking methods, optimization methods do not follow the rule of “selection of projects with the worst conditions”; instead of this, optimization methods identify the projects that best suit the entire road network, because all the constraints of the model are met simultaneously.
The optimization model for long-term planning uses a direct representation of the evaluation criteria determined by the target functions. The following indicators are used in AESUM: the cost of planned repairs and / or the weighted average level of degradation in terms of area and priority of bridges. In this optimization model, the priorities of bridges are not determining factors of calculation. However, they have a certain influence due to the influence on the amount of “fines” in determining the cost, and in the case of determining the weighted average degradation level it is included in the formula along with the structure area.

5 Conclusions

The proposed two-level model of prioritization of bridges on the road network has a number of advantages. Firstly, with a significant number of factors involved, their arrangement by groups allows for more influential control over defined impact factors and analyzes the results in their adjustment. Secondly, by implementing the mentioned model in the AESUM PC, it is possible to obtain a mechanism for tracking the influence of both individual factors and groups of factors on the results of calculations. The model of prioritization allows streamlining the consideration of bridges in the process of rehabilitation their technical condition and possible replacement in accordance with the priorities of bridges.

References

[1] Al-Assar, R., He, Z., Tighe, S., Haas, R.: Simplified User Delay Cost Model for Pavement and Bridge Life Cycle Analysis, Annual Conference of the Transportation Association of Canada, Alberta, 2000.

[2] Elkhalek, H.A.: Framework for Bridges Maintenance in Egypt, International Journal of Engineering, 6 (2016) 1, pp. 48-54

[3] Hatem, E.: Bridge Management System with Integrated Life Cycle Cost optimization, doctoral thesis, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario, Canada, 2007.

[4] AL-Subhi, K., Johnston, D., Farid, F.: Optimizing System-Level Bridge Maintenance, Rehabilitation and Replacement Decisions, Report, Department of Transportation, U.S., 1989.

[5] Benjamin, Jack R., Lind, N.C.: A Probabilistic Basis for a Deterministic Code, ACI Journal Proceedings, 66 (1969) 11, pp. 857–865

[6] Marková, J.: Reliability elements in the assessment of existing bridges, Reliability, safety and diagnostics of transport structure sand means 2008, University of Pardubice, Czech Republic, 25-26.9.2008.

[7] Saati, T.: Decision making, Method of analysis of hierarchies, Radio and Communication, 1993.

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DEVELOPMENT OF NEW BRIDGE INSPECTION SYSTEM USING 5G AND AI UNDER CLOUD CONDITION

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Abstract

In Japan, the deterioration of bridges constructed in the high economic growth period is progressing, and the maintenance of those bridges is a problem. Municipalities, as road administrators, are required to conduct close visual inspections of bridges (longer than 2m) once every five years, but municipalities face difficulties due to lack of financial and human resources. For this reason, the research of inspection and diagnosis technology is advanced for efficient inspection work. Especially, it is the new technology of ICT (information and communications technology), such as AI analysis of image data of bridge photographs.
In this study, we developed a bridge inspection support system that automatically detects cracks in concrete bridges from bridge photographs. This system uses AI of image processing by deep learning. By using AI, we will be able to detect cracks in a short time and inspect bridges more efficiently. However, it requires many photographs of huge amount of data for image analysis. And those images take time to upload to the system by mobile communication. Therefore, we verified the system operation using 5G mobile communication, which is characterized by high speed and large capacity.

Keywords: bridge inspection, ICT technology, image processing

1 Introduction

Bridges built during the high-growth period are deteriorating in Japan. 10 years later, approximately 52 % of the 720,000 bridges across Japan will be 50 years old or more [1]. In 2014, road administrators were required to conduct close visual inspections once every 5 years to maintain and manage road structures efficiently. Approximately 70 % of all bridges are managed by municipalities [1]. In the near future, municipalities will face financial difficulties due to the repair of bridges. Especially, it is considered that the cost of scaffold inspection vehicles and labor costs will be high [2]. In municipalities with insufficient financial and human resources, close visual inspection becomes impossible in the future. In addition, the result of the inspection differ depending on the inspector [2].
Therefore, a new inspection method using new technology is required to replace the close visual inspection. Alternative methods are expected to reduce inspection costs and operating time. There are many researches using new technology such as detecting cracks from image data of bridges [3-8].
In this study, we developed damage diagnosis support system, which analyzes high-resolution bridge images by AI (Artificial Intelligence) . This system is called "Systemized engineer's eye for Crack (SeeCrack)". When bridge images are imported into SeeCrack, it au-

tomatically detects the length and width of crack. SeeCrack can be operated by unskilled or inexperienced inspectors. Furthermore, SeeCrack was built on the cloud computing, and inspectors in the field and the engineers in the remote place were connected. It can perform inspection work efficiently in a short time.
The inspectors can upload images from bridge field via mobile communication and conduct AI analysis on the field. We used 100 million pixel images that has almost the same visibility as the actual visual inspection. The image data of 100 million pixels has a capacity of 600 MB per picture, and many images are necessary for the bridge inspection. Therefore, it takes time to upload images, and as a result, the inspection work time increases. To reduce the inspection work time, we utilized the 5th generation mobile communication system (5G) with features of high speed and large capacity.

2 System development

We developed the bridge inspection support system called SeeCrack. This system saves images of bridges, detects crack damage, supports soundness diagnosis, and manages them geographically. SeeCrack can automatically detect cracks, which is one of the inspection items of precise visual inspection, by image analysis using AI. Chapter 2 describes the outline of SeeCrack and the interview of a bridge inspection engineer about SeeCrack.

2.1 System overview

The four phases of SeeCrack are as follows:
Phase 1: To take photos of the bridge with the ultra high-resolution camera "iXU-RS 1000". The iXU-RS 1000 camera can take 100 megapixel (11,608 x 8708) photos. By enlarging the image, the visibility of small crack of the bridge can be easily recognized, which is like aclose visual inspection. Photographic image data taken by this camera has a capacity of about 600 MB per image. The camera is easy to carry because it measures 97.4 x 93 x 170.5 mm and weighs 930 g. Bridges with space for photography can be inspected more easily than bridge inspection car, a height work car, a scaffold or a boat for inspection. Figure1 shows the situation that take by an ultra high-resolution camera from 17 meter away from the pier. We were able to see cracks as small as 0.2mm from that image.

Figure 1 Situation of taking a photograph

Phase 2: Automatic detection of cracks by image processing technology using deep learning. The bridge images are predicted in pixel units "piece with cracks" and "piece without cracks" from the previously learned model. And the system calculates the length and width of the extracted cracks and dreates a "crack map".

Phase 3: Determining soundness in four stages based on image analysis results. Past crack detection results, site conditions, and past inspection records are also taken into consideration. As shown in Figure 2, scale, shape and position of crack are extracted from the image by pattern matching, and this information is useful for diagnosis. Although this phase is being studied and not yet implemented, it is not necessary for the purpose of this paper.

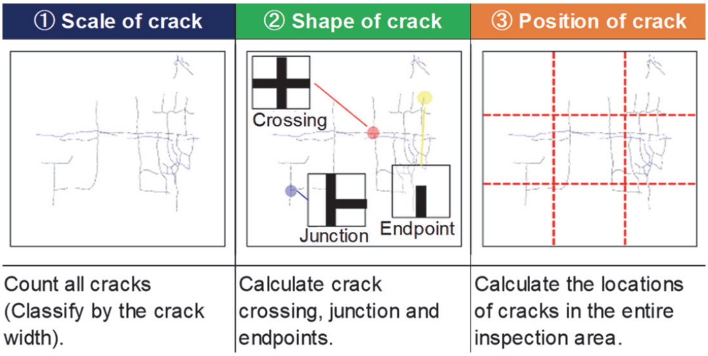

Figure 2 An example of crack feature extraction

<u>Phase 4:</u> The system marks the results of the bridge inspection on the map. These results are useful for the examination of bridge repair. In order to prioritize detailed inspection and repair, it is useful to group bridges by data according to items such as manager, diagnosis result, and inspection date.The image of SeeCrack screen is shown in Figure 3.

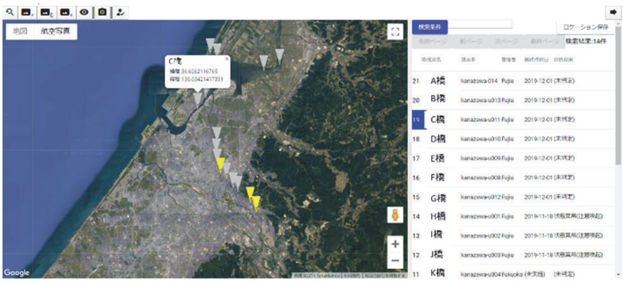

Figure 3 SeeCrack Screen Image

2.2 Evaluation of the system

We interviewed a bridge inspection engineer about the function and usefulness of SeeCrack. According to it, SeeCrack conducts objective damage detection, and workers in the field and engineers in remote places can share images of the bridge at the same time. In addition, it was able to find cracks in the photos which photographed bridge pier from the distance of 17 meter by the camera of 100 million pixels. It was valuable to be able to inspect with photos taken without getting close to the bridge. Additionally, we also got advice that if the system had a voice call function, it would be a better system. That's because field workers could receive guidance from skilled engineers in remote areas while checking images of bridge.
Furthermore, there was a comment to display not only the image of the damaged part but also which part of the whole bridge was damaged. The structure of the bridge and the damaged place also influence the damage decision. If the details of the damage and the damage part are easily identified, skilled engineers in a remote area can make a quick decision.
It is important to comprehend the bridge condition accurately by periodic inspection. Therefore, it is a problem that the diagnosis varies depending on the inspector. AI analysis is one way to detect and record cracks based on objective criteria. With an objectie crack record, we can accurately compare whether the original crack progressed or suddenly appeared with the

past data. Finally, human decision is indispensable, because the diagnosis of the damage greatly influences natural environment and structure and type of the bridge. If photos of the bridge and damage automatic detection by SeeCrack can be shared remotely, the quality of the inspection work will be improved. AI can detect the crack from images, and the engineers can decide easily. The combination of AI and human decision is expected.

3 Target bridge

3.1 Overview of the target bridge

The target U-bridge has a length of 344 m, a width of 16.5 m, and 3 spans of continuous PC cable-stayed bridge. Built in 2001, 9 diagonal members extend from 2 type A towers, 95 m in height and 54 m in height from the outer surface of the main girder.

3.2 Evaluation of the target bridge

Concerning the structure and close visual inspection method of the U-bridge, hearing was conducted with the bridge inspection engineer as described in Chapter 2 Section 2. The U-bridge needs to inspection with a vehicle for work at height or a vehicle. This bridge is not closed during the inspection, but the inspection is expected to be 5 ~ 6 people a day for 4 ~ 5 days. The inspection cost is estimated to be several millions yen, but this is based on the view that the U-bridge is new. In general, the older the bridge, the longer it takes time to inspect it, so the cost of this bridge inspection after 10 years will increase.

4 Demonstration experiment

We demonstrated automatic damage detection of U-bridge using SeeCrack. Images of the U-bridge taken in advance were uploaded to SeeCrack, and crack damage was automatically detected by AI analysis.

4.1 Experimental environment

In this study, we set up two categories: communication environments and cloud systems (Figure 4). Table 1 shows items by category.

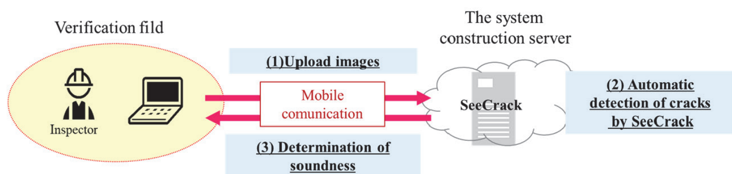

Figure 4 Diagram of the experiment

In the communication environment, there are 4 types, LTE (Mobile network carrier A), LTE (Mobile network carrier B), eLTE and pre-commercial 5G. Mobile communication systems started with 1st Generations (1G) in the 1980s and evolved every 10 years. 4th Generations (4G) has been used since the 2010s. 4G has LTE and eLTE (enhanced LTE). 5G commercial service was scheduled to start in the spring of 2020 in Japan. 5G has three features of high speed, large capacity, low delay, and simultaneous connection of other numbers, and is expected to be able to transmit large image data with super high-resolution. Since 5G had not started when we verified the demonstration, we conducted an experiment using pre-commercial 5G which was not a sufficient spec in February 2020.

We constructed SeeCrack on the on-premises server and the cloud server. The on-premises server is an original production including a graphics board with GALAKURO GAMING model NVIDIA GEFORCE RTX 2060. The cloud server is docomo Open Innovation Cloud.
The verification feilds are the pre-commercial 5G verification room (LTE(A), eLTE, pre-commercial 5G), the visual simulation room in the university (LTE(B)), and the bridge feilds in I-Prefecture(LTE(B)).

Table 1 Examination items of the demonstration experiment

Category	Item	Function
Mobile communication system	LTE (mobile network carrier A)	communication system, commonly called "4G"
	LTE (mobile network carrier B)	Verification by LTE communication of Company B for comparison with Company A.
	eLTE	It is means "enhanced LTE". "eLTE" evolved from conventional LTE.
	pre-commercial 5G	In the demonstration, we used "pre-commercial 5G" because "5G" was before it was launch.
The system construction server	On-premise	SeeCrack constructed on our University's servers. We manage it ourselves.
	Cloud	The cloud server that we used was docomo Open Innovation Cloud.

4.2 Evaluation object

Table 2 shows the processing time for each combination of survey items.The processing time was determined from the start of the registration of the object bridge to the system, to the upload of images, the AI analysis on the server, and the feedback of the analysis result. In the case of U-bridge, 15 images were uploaded and 11 of them were analyzed by AI. The 15 images uploaded were compressed in order to reduce the capacity.

Table 2 Processing time in SeeCrack

	On-premise saver [sec]	Cloud Sever [sec]
LTE (A)	1,446	1,712
LTE (B)	1,979	1,859
eLTE	1,385	1,439
pre-commercial 5G	1,241	1,388

5 Analysis of demonstration experiments

From the results of Chapter 4, we analyze the inspection time, upload time, and inspection cost in SeeCrack.

5.1 Inspection work time

In the case of a close visual inspection of a U-bridge, it takes 4 ~ 5 days to inspect for cracks in the concrete. However, if we use SeeCrack, it takes only 0.5 hours to detect the damage automatically by AI, and it is estimated to be 1.5 hours in total for the inspection including the photographing time.

The images analyzed by SeeCrack were prepared in advance and the photographing time was not measured, therefore it is not evaluated in this paper. Further experiment will be needed to comprehend the total working time for crack inspection including photographing time.

5.2 Upload time

It is necessary to upload a large number of high-resolution images (about 600 MB per image) to SeeCrack for the inspection of bridges. Since it takes time to upload large data images, it is beneficial to upload using high-speed mobile communication. By using 5G, we expect to reduce the upload time and make the inspection more efficient. In the future, 5G is said to be 100 times faster than 4G [9]. At the time of the experiment, 5G service was not started in Japan, and we used pre-service 5G, which was not a sufficient spec. Therefore, there was no significant difference in upload time compared to 4G. However, in the real service of 5G, it is expected that the upload time difference will increase rapidly in future. We will need to experiment with real 5G service to measure upload times.
We also compared two servers, On-premise and cloud. On-premise was faster than cloud, and We have concluded that it was the influence of the Internet environment. In the future, it is necessary to examine including the internet environment.

5.3 Inspection cost

The instecion cost of the U-bridge was estimated at several millions yen assuming the inspection work for 4 ~ 5 days with 5 ~ 6 workers a day by the close visual inspection. On the other hand, using SeeCrack, we assume the cost of the ultrahigh-resolution camera and using system, and 2 ~ 3 workers can do it in a few hours. From examining the findings, the inspection cost can be reduced by using SeeCrack.
Moreover, as the bridge becomes older, the time and labor required for inspection of damaged parts in the close visual inspection are increase. Therefore, even on the same bridge, the cost will increase after 5 or 10 years. On the other hand, the cost of usng SeeCrack does not change even after 5 or 10 years because it takes the same amount of time and effort to photograph the bridge and analyze by AI.

6 Conclusion and future work

In this study, we developed “SeeCrack”, a system to analyze crack damage of concrete bridges by AI and to support diagnosis, as an alternative technology to close visual inspection. SeeCrack is a system which can be utilized in the Internet environment, and it can be expected to be utilized not only in Japan but also in overseas bridge inspection.
In addition, the image data of the bridge was uploaded to SeeCrack using pre-commercial 5G, and the result of automatic damage detection was obtained from the system. Our data suggested that SeeCrack can be operated in an environment over the internet, and upload time are reduced in fast mobile communication environments.
Automatic damage detection by SeeCrack using 5G will reduce inspection time and cost and improve safety compared with convnentional inspection methods. And this new inspection method will promote the efficiency of preventive maintenance management of bridges.
We show the possibility that SeeCrack can improve the efficiency of bridge inspection work. However, in this study, it was not clear how much inspection time could be reduced depending on the size of bridges. In the future, It is necessary to measure the inspection time using SeeCrack in some bridge fields.

References

[1] Road Maintenance in Japan, Problems and Solutions (ministry of Land, Infrastructure, Transport and Tourism), http://www.mlit.go.jp/road/sisaku/yobohozen/torikumi.pdf, 03.03.2020

[2] Survey report on bridge inspection work in local government, (Japan Society of Next Generation Sensor Technology), http://www.socialinfra.org/p_technology/Apdf_H.pdf 03.03.2020

[3] Imai, M., Ohta, M., Tsuyuki, K., Imai, H., Miura, S., Murata, K., Takada, J.: Video image based dynamic behavior analysis of concrete structures by using digital image correlation method, Journal of Japan Society of Civil Engineers, Ser. A1 (Structural Engineering & Earthquake Engineering), 72 (2016) 1, pp. 279-289

[4] Nishimura, S., Kimoto, K., Matsuoka, N., Otani, H., Ogata, T., Matsuda, H.: The Development and Evaluation of Telemetry in Bridge Maintenance, Journal of applied survey technology, 24 (2013), pp. 52-61

[5] Okada, Y., Okatani, T.: Evelopment of UAV with Passive Rotating Spherical Shell for Bridge Inspection and its Evaluation of Inspection Capability in Real Bridges Journal of the Robotics Society of Japan, 34 (2016) 2, pp. 119-122

[6] Kimoto, K., Yamaguchi, K., Okumatsu, T. Kawamura, T. Matsuda, H. The Development of Inspecting Method about Bridge without Temporary Scaffold Using Optical Measurement Techniques, 47 (2017) 89, pp. 59-66, doi: http://hdl.handle.net/10069/37675

[7] Fujita, Y., Taguchi, T., Hamamoto, Y.: Image mosaicking and semi-automatic crack evaluation for visual inspection of concrete structures, Journal of Japan Society of Civil Engineers, Ser. F3 (Civil Engineering Informatics), 74 (2018) 1, pp. 18-32

[8] Minami, T., Fujiu, M., Takayama, J., Suda, J., Okumura, S.: A Study on Image Diagnostic Technology for Bridge Inspection Using Ultra High Resolution Camera, 5th International Conference on Road and Rail Infrastructure, pp. 79-85, 2018.

[9] Initiative to realize 5G in 2020 (Ministry of Internal Affairs and Communications), https://www.soumu.go.jp/main_content/000593247.pdf, 03.03.2020

GEOTECHNICAL ULS DESIGN ISSUES OF BRIDGE SHALLOW FOUNDATIONS

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Abstract

Some important issues referring to the Ultimate Limit States of geotechnical design of bridge shallow foundations are discussed using results of 2D and 3D FE analyses, as follows: (a) The effects of highly eccentric and inclined loadings on the bearing capacity of footings on cohesionless soils, (b) the effects of soil inhomogeneity in the special case of 2-layered clay, (c) the scour effects in case of abutment and piers in riverbed, from the geotechnical point of view.

Keywords: bearing capacity, cohesionless soils, FE analyses, layered clay, scour effects

1 Introduction

Spread footings continue to be an attractive type of bridge foundations due to their well known advantages, as the simplicity and low cost. However, deep foundations as bored or driven piles and drilled shafts are chosen in many cases by reason of the limitation of vertical and horizontal displacements. After systematic observations and evaluation of data, now it is widely accepted that bridges on spread footings can tolerate considerably larger displacements than those adopted at the past. Consequently, the Ultimate Limit States (ULS) criteria may decisively influence the foundation type and the estimation of the vulnerability of existing bridges on shallow foundations, as well.

In the present paper selected issues are presented and discussed based on FE results under 2D and 3D conditions: (a) The main factors affecting the bearing capacity of shallow foundations on cohesionless soils, especially in case of highly inclined and eccentric loads. (b) The effect of soil inhomogeneity due to the two layered clay system. (c) The effects of scour on the bearing capacity of footings in waterway. The case is of peculiar interest, since scour is the main cause of bridge geotechnical failures in many countries.

2 Factors affecting the bearing capacity of shallow foundations

The bearing capacity (BC) of footings based on homogeneous soil and subjected to combined loadings (V,M,H) has been extensively investigated. Such problems since today are analyzed by trinomial equations, loosely based on the solutions from the theory of plasticity for strip footings, using correction coefficients, to assess the effects of shape, eccentricity and inclination of loadings. In the important case of cohesionless soil, the characteristic resistance or ultimate vertical load ($R_k = V_u$), according EN 1997-1, Annex D [1] is given by the simplified equation (base inclination factors, $b_c = b_q = b_\gamma = 1$):

$$V_u = A'\cdot(q'\cdot N_q\cdot s_q\cdot i_q+0{,}5\cdot\gamma'\cdot B'\cdot N_\gamma\cdot s_\gamma\cdot i_\gamma) \quad (1)$$

where

A′ - the effective contact area,

B′ - the effective width,

N_q - and N_γ the BC factors,

s_q, s_γ - the shape factors and

i_q, i_γ - the inclination correction factors.

The key figures indicating the symbols in this paper are presented in Fig.1 (D is the embedment depth).

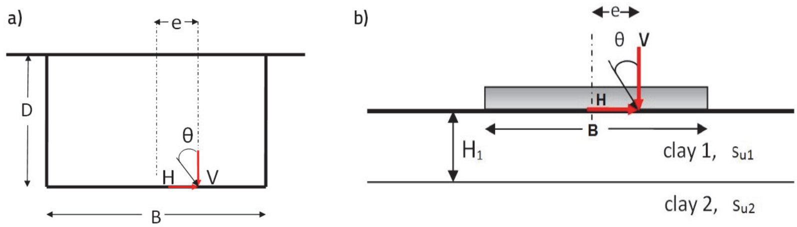

Figure 1 Combined loading on rectangular footing: a) Homogeneous soil, b) Two-layered clay

Even in simple cases, several uncertainties are related with the correction factors of Eq. (1). For example, the inclination factor i_γ for strip on cohesionless soil is given by different equations. Some proposals are compared in Fig. 2a. According to Eurocode 7.1 [1] and [2], the factor i_γ is related only with the inclination of the resultant load, tanθ, independently of the friction angle, φ′, thus it seems that the sliding for high ratios H/V is not taken into account. On the contrary, according to [3] i_γ depends on both the parameters tanθ and φ′. In order to separate the effects of i_q and i_γ, FE analyses are carried out for strip on the surface, which verify that i_γ depends also on the friction angle, φ′, according to Fig. 2. In any case, $i_\gamma \to 0$ for extremely high inclinations, as $\theta \to \varphi'$.

The depth effect on the BC, according to Eq. (1) is taken into consideration through the term q′, equal to the effective overburden pressure at the base of the footing. This simplification is usual, even in FE analyses, as for example [4]. In order to examine the contribution of this factor on the BC, FE analyses are carried out under 2D or 3D conditions, by the more realistic geometrical simulation shown in Fig. 1a. In the simple case of vertical centric load, the results from 2D analyses are presented in Fig. 3, in comparison with those from Eq. (1).

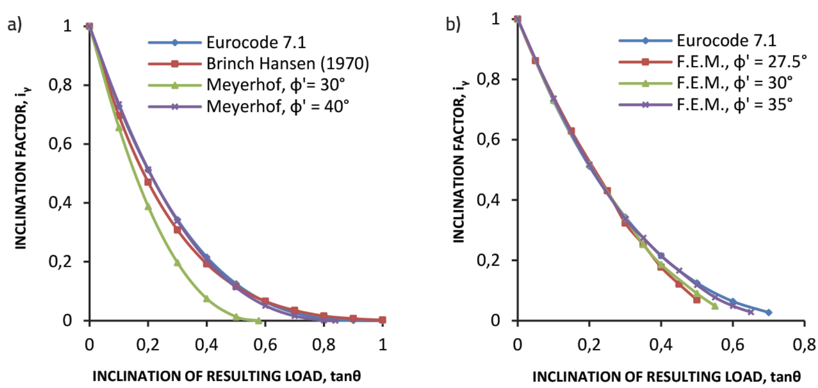

Figure 2 Inclination factor, i_γ: a) Comparison of proposal, b) FEM results (D=0)

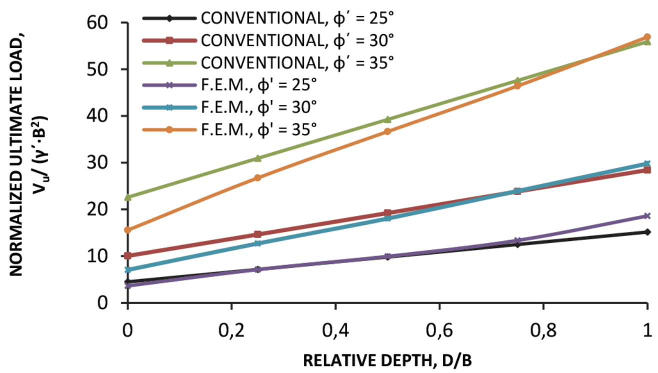

Figure 3 Effect of the foundation depth on the normalized ultimate load : cohesionless soils, vertical loads

The normalized ultimate load according to the conventional method increases linearly with the relative foundation depth (Eq. 2).

$$\frac{V_u}{\gamma' \times \mathrm{B}^2} = \left(\frac{D}{B}\right) \times N_q + \frac{1}{2} \times N_y \tag{2}$$

On the contrary, from the FE results considerably higher rate of increase is shown (Fig.3), which indicates that the foundation depth has a significant effect.

For the general loading case (V,M,H), it is well known that high eccentricities of the resultant on the foundation base decrease drastically the bearing capacity. Apart from these effects, several Codes impose limitation in eccentricity. DIN 1054 [5] directly correlates the biaxial eccentricity with the safety against overturning. For Load Case LC3 (corresponding to accidental design situations and seismic loadings), the verification of safety against overturning may be omitted if the bearing resistance is verified. Obviously, the effect of high eccentricities on the bearing capacity is of peculiar interest in such cases. According to AASHTO [6] for bridges, the restriction of normalized eccentricity is related with the safety against overturning for soils and rocks. Finally, according to [1], if e/B›1/3, special precautions shall be taken. In the spirit of this European Code, the investigation of bearing capacity has significant importance. From the extensive parametric analyses some assumptions or relationships incorporated in Eq.(1) are verified for both cohesionless or clayey soils under undrained conditions [7]. Nevertheless, these verifications refer only to simplified cases and not to the simultaneous effects of eccentric and inclined loadings, of shape and depth of foundations, as well.

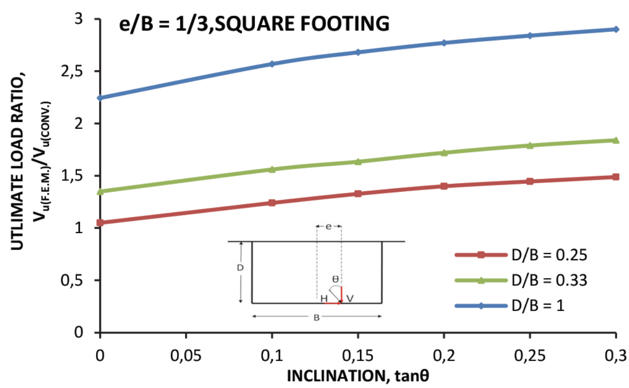

Figure 4 Effect of depth and inclination on the ultimate load ratio: Square footing on cohesionless soil, φ′= 40°

It is concluded that the conventional analyses result into significant higher reduction of the BC than the FEM, so it seems that the simultaneous effects cannot be approached by the product of individual parameters, as the effective width, inclination and shape factors. For example, in case of square footing (Fig. 4) under eccentric and inclined loading, the conventional equations considerably underestimate the BC, especially for higher values of depth and inclination. In the general case of loading of footing in homogeneous soil, the locus of all possible combinations of vertical, moment and horizontal loads, which lead to shear failure forms the BSS, i.e. the bearing strength surface, which reduce to BC lines in the M, V plane. Such lines (in homogeneous soils) have been examined, as for example [8], [9]. From the present FE analyses the interaction diagrams refer to the normalized values:

$$v = \frac{V_u}{V_{u,o}}, \quad m = \frac{M_u}{V_{u,o} \times B} \tag{3}$$

where $V_{u,o}$ the ultimate centric vertical load.

From Fig. 5a, it is observed that the curves from Eq. (1) and FE, for strip and θ = 0 are almost identical, while for tanθ = 0.20, the former was clearly underestimate the BC. The effects of the shape of foundations in this comparison are clear in Fig. 5b, where differences are shown for both cases θ = 0 and tanθ = 0.20.

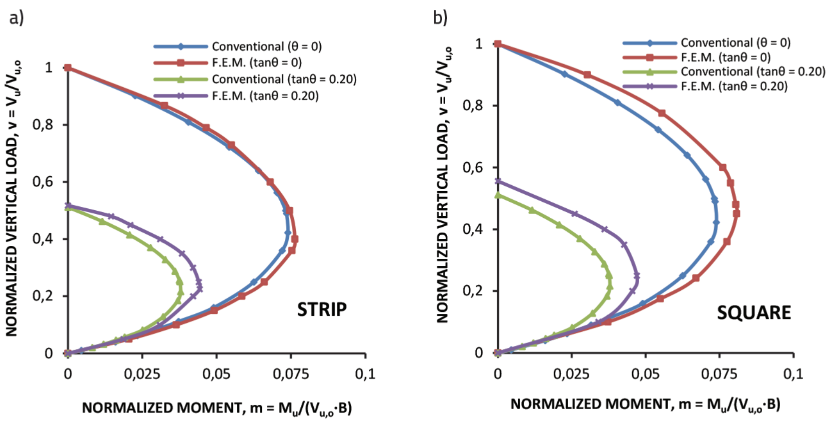

Figure 5 Comparison of interaction diagrams, cohesionless soil, φ′ = 30°

3 Inhomogeneity effects in clayey soils

Several results from FE analyses, in the special case of two-layered clay are presented for strip, rectangular or square footings. For eccentric loading on a rectangular area L·B (Fig. 1b), the ultimate vertical load can be expressed according to authors [10], in the following form:

$$V_u = N^*_{C1,e} \cdot s_{u,1} \cdot L \cdot (B\text{-}2e) \tag{4}$$

where $N^*_{C1,e}$ the BC factor depending on the normalized thickness of the upper layer, H_1/B, the strength ratio SR = s_{u2}/s_{u1} and the normalized eccentricity e/B, incorporating also the shape effects. In the special case, where e/B = 0, the corresponding ultimate vertical load is:

$$V_{u,o} = N^*_{C,1} \cdot s_{u,1} \cdot L \cdot B \tag{5}$$

The impact of soil inhomogeneity on the BC for eccentric loadings is clearly satisfied by the interaction diagrams. From Eqs (3), (4) and (5) the following relationship results:

$$m = \cdot v \cdot (1 - v \cdot) \tag{6}$$

For SR ‹ 1, $N^*_{C1,e}$ › N^*_{C1}, thus for any value v the ratio m is higher than this for the homogeneous clay. On the contrary, for SR › 1, $N^*_{C1,e}$ ‹ N^*_{C1} and consequently the value m is now lower than this for SR = 1 and a given v. It is expected that the curve v-m for SR = 1 comes in-between the lines for SR ‹ 1 or SR › 1. For the cases of $s_{u,2} = s_{u,1}/5$ and $s_{u,2} = 5s_{u,1}$ (SR = 0.2 or 5), the V-M failure envelopes, in Fig.7a (square) and Fig.7b (rectangular, L/B = 2), verify the above-mentioned. The maxm values for SR = 0.2 are significantly higher than these for homogenous soil, in both cases. Figure 6 refers to $H_1/B = 0.25$ and the differences between the envelopes for SR = 1 and SR = 5 are not very important, mainly in the case of square footing. For higher normalized thickness H_1/B these curves (SR = 1 and SR = 5) become almost identical.

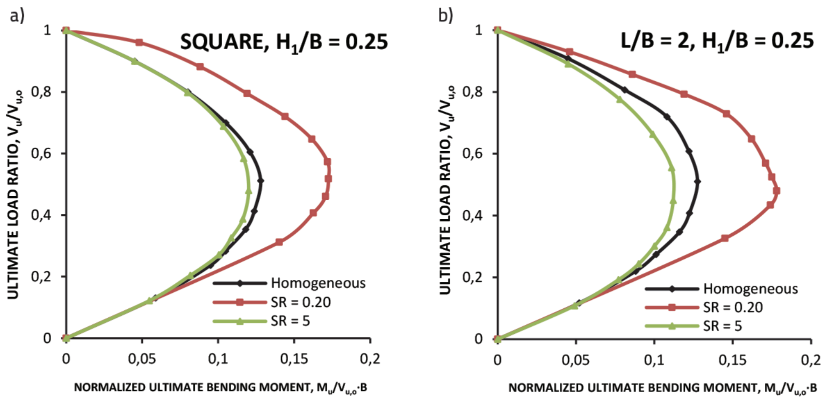

Figure 6 Interaction diagrams v-m of rectangular footing on two-layered clay

4 Scour effects on the BC of shallow bridge foundations

The erosion of the foundation soil in riverbed is associated with three distinct mechanisms, from which local scour is the most significant, since this may quickly reach great depths, causing the foundations instability. The hydraulic performance of shallow bridge foundations has been extensively investigated, i.e. [11]. However, according to [12], the procedures to appraise the vulnerability of river bridge piers often overlook the geotechnical factors. The same authors [12] presented a simple method to estimate vulnerability of such foundations.

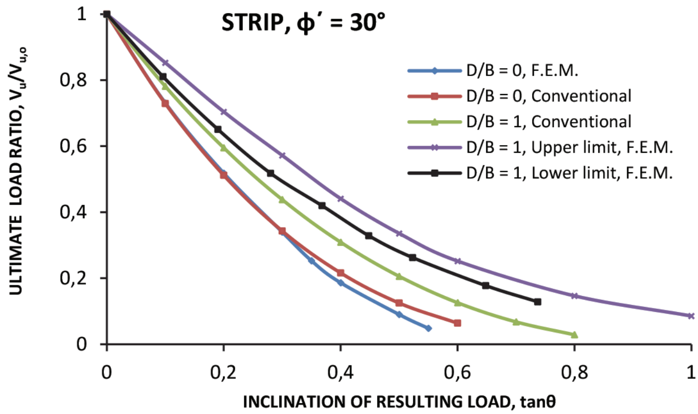

Figure 7 Effect of the inclination and depth on the ultimate load ratio, $V_u/V_{u,o}$

The simultaneous effect of the foundation depth and loading inclination on the normalized ultimate load is clearly defined in Fig. 7. Although for D = 0, the results from conventional methods and FEM are almost identical, for the higher foundation depth the former ones significantly underestimate the V_u values. A main factor affecting the vulnerability of the footing is the relative scour depth, D_s/D, where D is the initial foundation depth. The comparison of failure mechanisms for general erosion (corresponding to the remaining depth after the scour, $\Delta D = D\text{-}D_s$) and this for local scour is indicatively illustrated in Fig. 8.

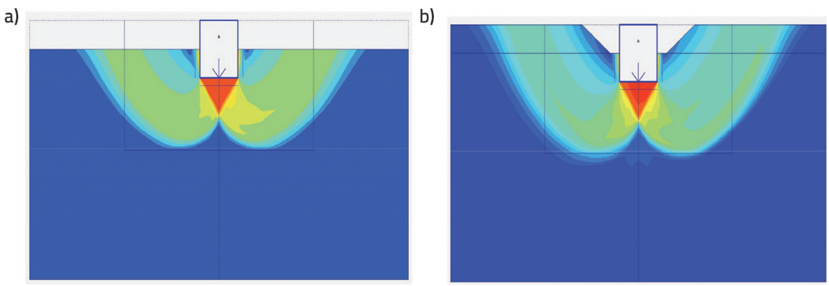

Figure 8 Comparison of failure mechanisms for two scour cases

The differences of failure mode for the two cases reflect on the diagrams of normalized ultimate load versus the relative scour depth, presented in Fig. 9. Although a representative geometry of the local scour is simplified, in order to carry out the FE analyses, it can be concluded that the case of general erosion is more unfavourable for a given ratio D_s/D.

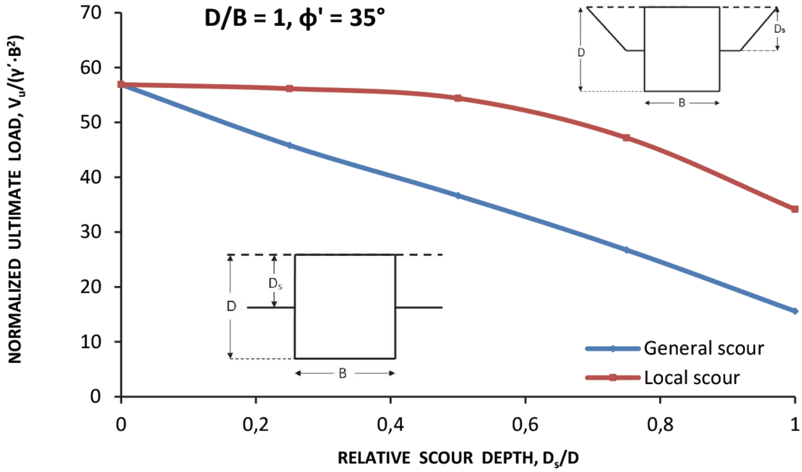

Figure 9 Effect of relative scour depth on the normalized ultimate load

5 Conclusions

The results from 2D and 3D FE analyses indicate that the simultaneous effects of high eccentricity and inclination cannot be approached by the product of the partial factors in the conventional equation of BC. As a result, it seems that in such cases, the BC is considerably underestimated. In the case of two-layered clay, the eccentricity of loading leads to moving up the failure mechanism, thus the effects of the second layer (either unfavourable or beneficial) tend to be less important. For the assessment of the vulnerability of bridge shallow foundations associated with scour, the simultaneous effect of all geotechnical data and parameters involved, should be taken into account.

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References

[1] EN 1997-1: Geotechnical Design-Part 1: General rules, CEN, 2004.

[2] Hansen, J.B.: A revised and extended formula for bearing capacity, Bulletin No 28, Danish Geotechnical Institute, Copenhagen Denmark, pp. 5-11, 1970.

[3] Meyerhof, G.G.: Some recent research on the bearing capacity of foundations. Canadian Geotechnical Journal, 1 (1963) 16, pp. 16-26

[4] Krabbenhoft, S., Damkilde, L., Krabbenhoft, K.: Bearing capacity of strip footings in cohesionless soil subjected to eccentric and inclined loads, International Jorunal of Geomechanics, 14 (2014) 3, doi: 10.1061/(ASCE)GM.1943-5622.0000332

[5] DIN 1054, 2005:1: Ground-Verification of the safety of earthworks and foundations

[6] AASHTO: Standard Specification for Highway Bridges, 16th Edition American Association of State Highway and Transportation Officials, Washington DC, 1996.

[7] Papadopoulou, K., Gazetas, G.: Geotechnical design issues of bridge shallow foundations, 8th Hellenic Conference on Soil Mechanics and Geotechnical Engineering, Athens, Greek, 6-9.11.2019.

[8] Georgiadis, M.: Load path dependent stability of shallow footings, Journal of Soils and Foundations, 25 (1925) 1, pp. 84-88, doi: https://doi.org/10.3208/sandf1972.25.84

[9] Pender, M.: Bearing capacity surfaces implied in conventional capacity calculations, Géotechnique, 67 (2017) 4, pp. 313-324

[10] Papadopoulou, K., Gazetas, G.: Eccentricity effects on the bearing capacity of strip and square footings on two-layered clay, Journal of Geotechnical and Geological Engineering, 37 (2019) 5, pp. 4099-4120

[11] Federal Highway Administration: Hydraulic performance of shallow foundations for the support of vertical-wall bridge abutments. Publication No FHWA-HRT-17-013, 2017.

[12] Federico, F., Silvagni, G., Volpi, F.: Scour vulnerability of river bridge piers, Journal of Geotechnical and Geoenvironmental Engineering, 129 (2003) 10, pp. 890-899

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PREDICTED AND MEASURED TIME-DEPENDENT BEHAVIOUR OF HIGHWAY EMBANKMENT ON COHESIVE SOIL STRATUM

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Abstract

Highway embankments are important structural elements in modern road infrastructure. If such a construction is built on cohesive low-permeability soils, it is necessary to perform a prediction of long-term settlements and excess pore pressures. The paper presents a numerical analysis of an instrumented embankment constructed in the Czech Republic using the finite element method. Two alternative constitutive models were employed throughout the analysis: standardly used linear elastic perfectly plastic model and elastoplastic model with volumetric and shear hardening with stress-dependent stiffness. A construction sequence was modelled in detail including durations of partial construction stages. Both the settlements of subsoil (in short-term and long-term conditions) and excess pore pressures measured in multiple depths were evaluated and compared with predictions. Results employing a more complex constitutive model show a reasonably good agreement with measurement both in terms of settlements and pore pressures. The application of a perfectly plastic constitutive model leads to an overestimation of settlements.

Keywords: embankment, settlement, excess pore pressures, finite element method, elastoplasticity

1 Introduction

Highway embankments are an integral part of modern road infrastructure. Due to space, ecological and economic constraints, they are often constructed in difficult geological conditions such as on highly compressible and low permeable fine-grained soils. In such cases, it is necessary to predict the development of settlement in time during construction and service life. Furthermore, the embankment construction rate must be controlled to avoid reaching soil shear strength due to the generation of excess pore pressure in the subgrade. Several such cases were reported in the literature ([1], [2]).
The finite element method is a standard computational tool in present geotechnical engineering. However, to obtain an acceptable response of a computational model, the finite element method must be combined with an appropriate constitutive model. Neglect the dependence of soil stiffness on stress and loading regime can lead to the overestimation of resulting displacements. On the other hand, if a pore pressure increase and consequently an effective stress decrease due to the contractant behaviour of fine-grained soil during undrained shearing is not involved in a particular constitutive model, the undrained shear strength might be overestimated.

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The proposed paper presents a back-analysis of highway embankment constructed in the Czech Republic. Apart from the monitoring of subgrade settlement, pore pressures in two boreholes were recorded via piezoelectric pressure sensors. Two alternative constitutive models were employed throughout the analysis: commonly used linear elastic – perfectly plastic model and more advanced elastoplastic constitutive model with shear and volumetric hardening.

2 Description of analysed embankment

The embankment is a part of the highway D47 Hrušov – Bohumín and situated on the west coast of Antošovice Lake. The maximum height and width of the embankment are 14,1 and 94 meters, respectively. The schematic cross-section of the embankment is shown in Fig. 1. The construction period was divided into 5 stages. Geotechnical monitoring consisted of hydrostatic nivelation between the shafts S1 and S2 in 31 points and pore pressure transducers situated in two boreholes (BH1 BH2), .
The top part of the subsoil to a depth of 1,8 m consists of anthropogenic materials. The natural Quaternary cover consists of sandy-silty clay (1,8 – 3,1 m BGL) and fluvial sandy gravel (3,1 – 6,9 m BGL).Stress-deformation behaviour of the embankment is most influenced by Tertiary silty clay with firm to stiff consistency locally with lens of sand. The Quaternary sediments were not involved in the FE model as they were partially excavated and their thickness is small compared to the underlying Tertiary clays. The embankment itself was built from a mixture of tailing material from local mine and a blast furnace slag with the following properties: r_d=2020 – 2100 kg/m^3, k=10^{-2} m/s, j´=37°, c´=3 kPa.

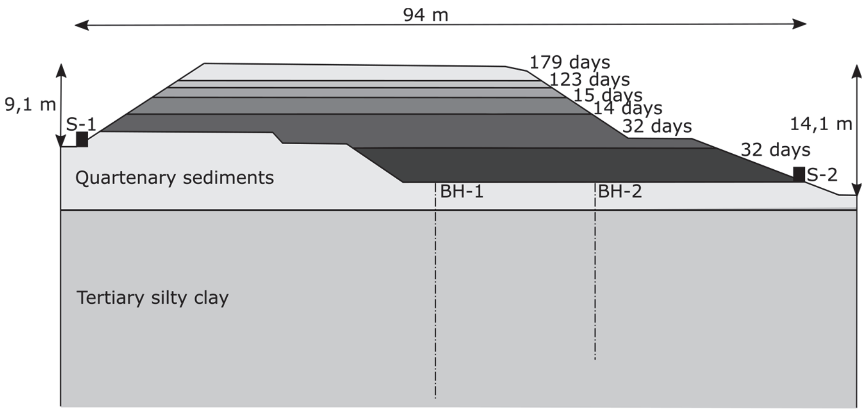

Figure 1 Schematic cross-section of the analysed embankment

3 Applied constitutive models

Two constitutive models were utilized throughout the analysis:
- The linear elastic – perfectly plastic Mohr-Coulomb (MC) model,
- The Hardening Soil model (HS) - elastoplastic model with shear and volumetric hardening [3].

The basic features of both constitutive models are graphically compared in Fig. 2 and Fig. 3 for compression and shear loading, respectively, where NCL is the normally consolidated line and URL is the unloading-reloading line.

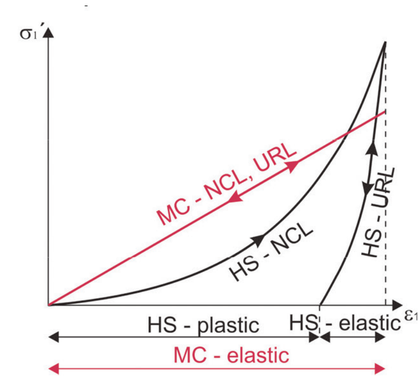

Figure 2 Compression loading – comparison of models

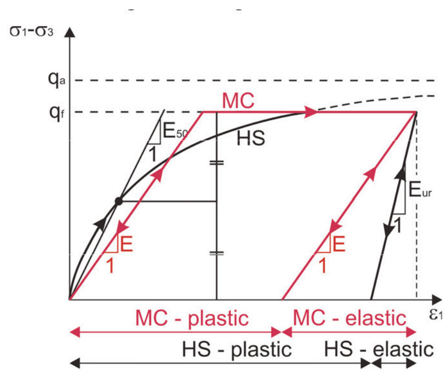

Figure 3 Shear loading – comparison of models

The HS model was based on the non-linear (hyperbolic) relationship between the axial strain and the deviatoric strain ([4], [5]). These non-linear elastic constitutive models were complemented by shear and compression hardening yield surfaces. An important feature of the HS model, especially in displacement analysis, is the stress dependency of soil stiffness. The stress-dependent oedometer stiffness modulus E_{oed} is given by Eq. (1). E_{oed}^{ref} is the reference value of the modulus valid for the reference vertical stress p^{ref}, j and c are the shear strength parameters, m is the power for the stress-level dependency of stiffness and K_0^{nc} is the K_0 value for normal consolidation (K_0^{nc}=1-sinj).

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\cos\varphi - \sigma'_1 \sin\varphi}{c\cos\varphi + p^{ref}\sin\varphi}\right)^m \tag{1}$$

4 Simulation of embankment construction

4.1 Computational model and values of input parameters

2D plain strain model was prepared using the Plaxis finite element code [6]. Due to the asymmetrical cross-section, the whole embankment was modelled. The construction sequence, as summarised in Tab. 1, was determined based on geotechnical monitoring report of the analysed cross-section [7]. The embankment height was held constatnt during the stages n. 5, 7, 9 in the particular section as construction activites took place on other sections of the highway. The total time-period analysed in this paper was 1800 days.

Construction of each layer is considered as the plastic stage (without the possibility of excess pore pressure dissipation) followed by consolidation analysis with a duration corresponding to the construction time of a particular sublayer.

Table 1 Construction sequence relevant to the analysed cross-section

ID	Name	Duration [day]
1	Initial (K_o) conditions	-
2	Construction of the 1st sublayer	32
3	Construction of the 2nd sublayer	32
4	Construction of the 3rd sublayer	14
5	Delay	27
6	Construction of the 4th sublayer	15
7	Delay	120
8	Construction of the 5th sublayer	123
9	Delay	151
10	Construction of the final 6th sublayer	179
11	Monitoring after construction	1107

Values of input parameters for the governing geological layer of Tertiary silty clay are summarised in Tab. 2. Poisson´s ratio during primary loading (n) and unloading-reloading (n_{ur}) is used in MC and HS model, respectively. Linear elastic behaviour before failure (MC model) is controlled by the Young´s modulus E_{ref}. Non-linear hyperbiloc stess – strain response during deviatoric loading in case of the HS model is controlled by the secant stiffness E_{50}^{ref} at 50 % of the maximum deviatoric stress.

4.2 Results

Measured and predicted settlement profiles for two various times since the beginning of construction are shown in Fig. 4. Settlements are significantly over-predicted in case of the calculation with the Mohr-Coulomb model in which stress-stiffness dependence is not involved. The prediction with the HS model provides more accurate predictions in terms of the final settlements in both times. However, the prediction underestimates the measured settlements on the left side of the embankment. This might be due to a greater thickness of Quaternary sediments in this particular area.

Table 2 Values of input parameters of Tertiary silty clay

Parameter		MC	HS
g_{unsat}	**[kN/m³]**	**16,21**	**16,21**
g_{sat}	[kN/m³]	19,86	19,86
$k_x=k_y$	[m/day]	5e-4	5e-4
E_{ref}	[MPa]	9,84	
n / n_{ur}^{*}	[-]	0,35	0,2
E_{50}^{ref}	[MPa]	-	16,08
E_{oed}^{ref}	[MPa]	-	16,08
E_{ur}^{ref}	[MPa]	-	44,66
m	[-]	-	0,55
j´	[°]	24,3	24,3
c´	[kPa]	40,33	40,33

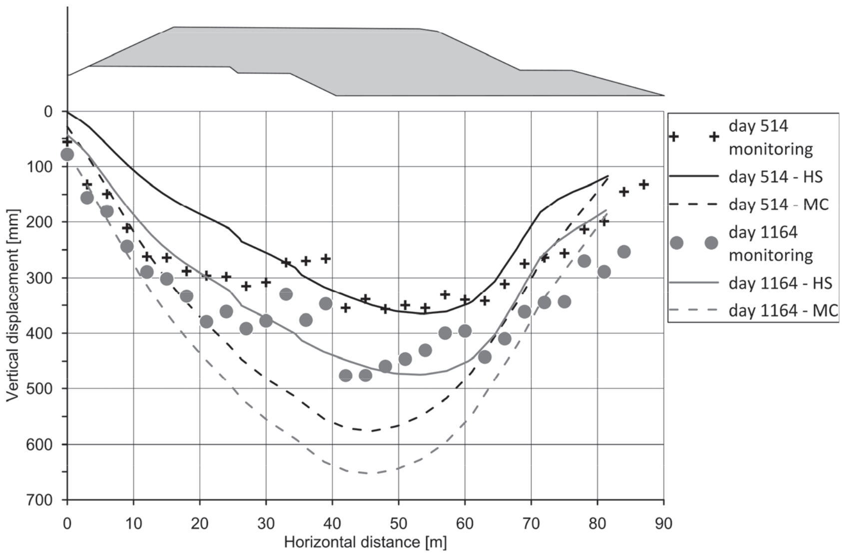

Figure 4 Measured and predicted settlement profiles

Measured and predicted time developments of excess pore pressures are shown in Fig. 5 (V1 – depth 5 m) and Fig. 6 (V2 – depth 10 m). Sharp increases in computed pore pressures arise due to the activation of corresponding sublayers. The times of these peaks approximately correspond to the times when the measured pore pressures increased. However, especially in the last stages which took much longer than the first stages, the activation of the sublayer with zero time interval (plastic analysis) followed by the consolidation analysis resulted in greater differences between the measurements and the predictions. The maximum values of pore pressures computed using the MC model are lower compared to the HS model. The Mohr-Coulomb model behaves elastically until reaching the failure state. In undrained shear loading, when no volume changes are allowed, no excess pore pressures are being generated and effective main stress remains constant.

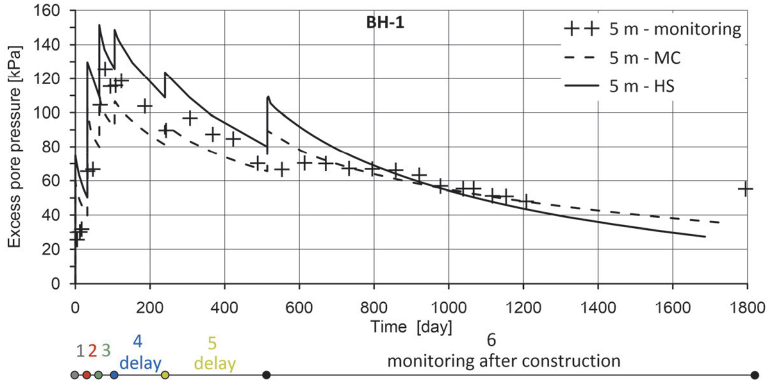

Figure 5 Development of excess pore pressures, borehole V1, depth 5 m

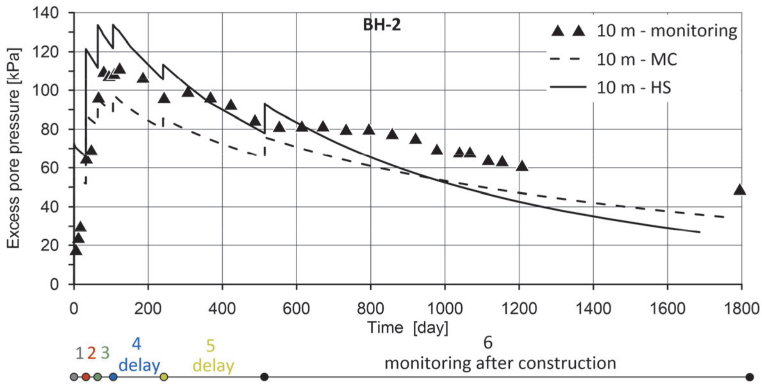

Figure 6 Development of excess pore pressures, borehole V2, depth 10 m

5 Conclusion

The performed analysis demonstrated that the combination of the finite element method and the appropriate material model provide sufficiently accurate embankment settlement prediction even in long-term conditions several years after the construction finished. Sufficient match is reached in case of the ultimate settlement and its position when using HS model. Predicted settlements are slightly underestimated on the left side of the embankment. This might be due to the higher thickenss of the Quaternary sediments in this area which were neglected in the presented analysis. Ignoring the stress dependency of the soil stiffness in case of the MC model leads to a substantial overestimation of predicted displacements. Measured excess pore pressures are predicited with a reasonable accuracy during the first 5 construction periods. However, the rate of the excess pore pressures dissipation during the subsequent monitoring is overestimated, indicating that the permeability of the Tertiary sediments is lower than anticipated.

Acknowledgment

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References

[1] Hopkins, T.C.: Stability of embankments on clay foundations, Kentucky Transportation Research Program, College of Engineering, University of Kentucky, 1986.

[2] Roy, D., Singh, R.: Failure of Two Embankments of Soft Soil Sites, 6th International Conference on Case Histories in Geotechnical Engineering, Arlington, pp. 1-9, 2008.

[3] Schanz, T., Vermeer, P.A., Bonnier, P.G.: The hardening-soil model: Formulation and verification, Beyond 2000 in Computational Geotechnics, Balkema, Rotterdam, pp. 281–290

[4] Kondner, R.L.: A hyperbolic stress strain formulation for sands, 2. Pan. Am. ICOSFE Brazil, pp. 289–324, 1963.

[5] Duncan, J.M., Chang, C.Y.: Nonlinear analysis of stress and strain in soil, ASCE J. of the Soil Mech. and Found. Div., *96* (1970), pp. 1629–1653

[6] Brinkgreve, R.B.J.: Plaxis 2D – Version 8, Finite element code for soil and rock analysis, User manual, TU Delft&Plaxis bv, Nederlands

[7] Hadacz, R., Zdrazil, K.: D47 47091/2 Hrušov-Bohumín, SO 199.2, Geotechnical monitoring – final report

METHODOLOGY OF GREEN RUNOFF DRAINAGE DESIGN FOR URBAN STREETS

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Abstract

The main provisions of the methodology for calculating and designing a "green drainage system" of surface runoff from the road network of settlements that are not equipped with an underground drainage and treatment system are given. Requirements for the "green drainage system" of surface runoff from urban streets that are not equipped with an underground drainage system are formulated. The requirements include the treatment degree of surface runoff, filtration rate, comfort of the visual environment, safety and convenience for pedestrians and bicyclists, technologies of winter maintenance. The main pollutants of surface runoff for different categories of Russian streets are identified. The composition and depth of filtration media, its operating life, types of green plants are determined depending on the composition of pollutants, their typical concentrations, the collection area of surface runoff and the composition of native soils. Examples of the most effective design solutions for the "green drainage system" and treatment of surface runoff from the road network are given.

Keywords: urban street, stormwater, rain garden, design, methodology

1 Introduction

The task of collecting and treating of surface runoff from the road network of settlements that are not equipped with an underground drainage system is extremely important for Russia. In Russia the water quality in most rivers is unsatisfactory and does not conform sanitary and hygienic standards. This is the result of the negative impact of uncontrolled sources, including stormwater. The flow of untreated stormwater to the relief leads to the accumulation of significant volumes of pollutants in soils and grounds [1]. Road dust, wear products from road surfaces, tires, brake pads, car discs, emissions from car engines and anti-icing materials for road ice prevention are the main pollutants in storm runoff. According to article 65 of the Water Code of Russian Federation the water protection zones and coastal protection zones are assigned to surface water bodies (seas, rivers, streams, canals, lakes, reservoirs). A special regime for the implementation of economic and other activities is established within these zones in order to avoid pollution of water bodies. In the absence of an underground drainage system the Water Code prescribes equipping transport infrastructure facilities with local treatment facilities (hereinafter - LTF). LTFs allow to the treat rainwater, watering and meltwater to ensure the most stringent standards established for water bodies of fishery importance. However, LTFs that are recommended by the industry road guidance document ODM 218.8.005-2014 [2] for roads are not suitable for the road network of settlements because LTFs require sufficient free space for their placement, their appearance does not match with the architectural environment of street space.

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In addition, the existing experience of operating LTFs on Russian roads has shown their low efficiency due to errors in design, construction and violation of the periodicity of routine maintenance [3], [4]. The listed circumstances require new designs of treatment facilities. These structures should fit harmoniously into the urbanized area, have high treatment efficiency and minimal costs for design, construction and operation.

2 Perspective LTFs for urban streets

2.1 Bioremediation LTFs

The practice of foreign countries are shown that LTFs are based on bioremediation technologies are the most perspective for the road network of settlements. Such facilities include filter strip, bioswale and a rain garden. The principle of operation of these constructions is based on the filtration of stormwater through a layer of soil (native or improved) and the plants. However, the construction of filter strip and bioswale requires favorable local relief. Moreover, the efficiency of surface runoff treatment by these LTFs is not high enough (40 ... 70 %) [5]. In addition, according to Russian set of rules SP 32.13330.2012 “Sewerage. Pipelines and wastewater treatment facilities” the open drainage of surface runoff is permissible only for settlements with low-rise residential buildings and for park areas and settlements in rural areas. In connection with the above, rain gardens should recognized as the most attractive and promising for urban streets. They have a high efficiency of storm water treatment (70...100 %) and do not have requirements for the relief [5].

2.2 The principal construction of rain gardens

The rain garden is a depression in the relief, which is designed to receive surface runoff and filled by a filtration media with planted moisture-loving higher plants that are able to restore the throughput of the filter (Fig. 1).

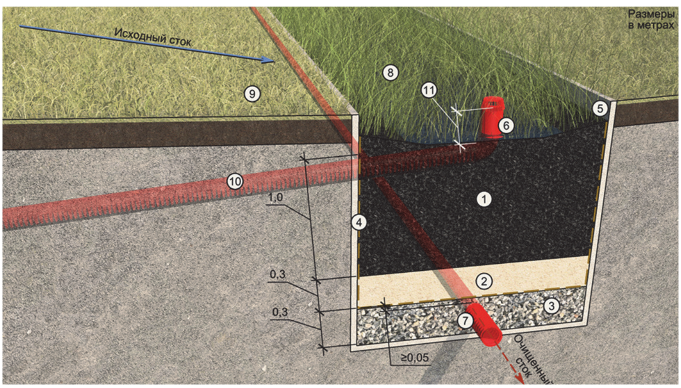

Figure 1 Rain garden operating principles [6] (1 - filtration media (native or improved soil), 2 - sand bed (optional), 3 - crushed stone, 4 - impermeable liner, 5 - geotextile, 6 - overflow pipe, 7 - drainage pipe, 8 - plants, 9 - drainage surface, 10 - outlet pipe, 11 - depth of the retention zone)

The catchment area is not more than 0.1 ha [6]. The surface runoff enters in the space provided by the rain garden design above the filtration media. This space serves for the temporary retention of water. The recommended depth of the retention zone is 0.3 m [6], [7], [8]. The incoming stormwater should not be in the retention zone for more than 24 hours for sanitary reasons [6], [8]. Stormwater slowly passes through a rain garden. Treatment is provided by passing in the filtration media together with bioretention provided by the plants. After passing through the rain garden, water is discharged either by infiltration to underlying soil, or is collected in a pipe and discharged into a storage system or sewer network. Maximum time of runoff water infiltration through the rain garden is 72 hours [7].

2.3 Adaptation of the rain garden design to Russian conditions

The use of rain gardens in Russia is constrained by the lack of guidelines for their construction and operation under local conditions. Simple copying of foreign guidance manuals for using is impossible due to differences in climate, soil, native plants and rhizosphere microorganisms. Russia is characterized by a clear division of the year into warm and cold periods, significant temperature fluctuations, and a stable snow cover is formed in most regions of the country during the cold season.Therefore, it is impossible to ensure efficient operation of the rain garden at low temperatures without introducing of sorption and ion-exchange materials into its filtration media [9]. Used plants should be native, perennial, moisture-loving, fast-growing, easy to care for, resistant to wintering, have a large biomass and thick deep roots, and have the ability to hyperaccumulate metal ions in the green mass. The most perspective solution is the combined use of plants and rhizosphere microorganisms (symbiotic plants and rhizosphere complexes) in LTFs [10], [11].

3 Methodological approaches of rain gardens design on urban streets in Russia

A rain garden designing must include specific steps.

3.1 Definition of main site parameters

The main parameters of the road network section include:
- urban street category;
- individual features of the section (plan elements, longitudinal and transverse profiles, number of lanes in each direction, width of lanes, width of the dividing strip, width of shoulders);
- soil types;
- weather and climate conditions.

According to [12] the weather and climatic characteristics of the area are determine from statistically processed data of long-term (at least 10...15 years) observations of meteorological stations in specific settlements or at the nearest representative meteorological stations.

3.2 Determination of the composition and concentration of pollutants in the surface runoff

At the design step of LTFs according to ODM 218.8.012-2019 [13], the predictive assessment of the concentration of pollutants in surface runoff from roads is performed for suspended solids, lead and oil and petroleum products in accordance with Table 1.
During the operation period of the road network section a qualitative and quantitative assessment of storm runoff for the presence of suspended solids and chemical pollution is carried out by sampling.

Table 1 Concentration of pollutants in the surface runoff from roads and urban streets [13]

Pollutant	Concentration of pollutants C [mg / dm³]	
	in rainwater	in meltwater
Suspended solids	390	810
Lead	0.084	0.09
Oil and petroleum products	7.2	7.8
Note: The table data may be specified depending on local conditions and for individual types of pollution.		

3.3 Assessment of runoff infiltration into the soil

The treated runoff can be discharged into the native soil under the rain garden (infiltrating rain gardens). If native soils have low water permeability or if other limitations exist, then rain gardens should designed with impenetrable walls, bottom and drainage infrastructure (non-infiltration rain gardens).
At the design stage, it is recommended to consider the possibility of partial or full use of treated storm water for household needs (for example, for road washing or plant watering) [12]. In this connection, it is advisable to design non-infiltration rain gardens for settlements.

3.4 Determination of the necessity for stormwater treatment

In accordance with ODM 218.8.012-2019 [13] the determination of the necessity for stormwater treatment should carried out by calculating of maximum allowable discharge in water body (MAD) for each pollutant separately, Eq. (1):

$$MAD(i) = q \cdot C_{\max}(i) \text{ kg/year} \tag{1}$$

q – stormwater vollume for treatment during a year, thousand m³ / year;
$C_{max}(i)$ – maximum allowable concentration of the i-th pollutant, mg/dm³ (It has been set by regulatory legal act of Russia).

A predictive assessment should be carried out for a prospective period. According to the Russian set of rules SP 34.13330.2012 “Automobile roads” this period is 20 years. The initial year of the calculated prospective period is the year when the road (or an independent section of the road) was put into operation. Therefore, effective treatment of stormwater by a rain garden without replacing the filtration media should be ensured during 20 years. Then during the calculation period the rain garden must hold a certain amount of pollutants from the surface runoff:

$$\Delta M_{20}(i) = 0.02 \cdot \left[q \cdot C(i) - MAD(i) \right] \text{ t/ period,} \tag{2}$$

C(i) – concentration of the i-th pollutant in the surface runoff, mg/dm³ (according to Table 1).

3.5 The calculation of the rain garden parameters

Calculation of the media area S is made by the following formula (3):

$$S = \frac{W}{h} \cdot 10^3 \quad m^2 \tag{3}$$

W – volume of the estimated rain or the estimated daily volume of meltwater is discharged for treatment (maximum value of them is accepted), m^3. The calculation is made according to metod [11];

h – maximum rain precipitation, mm.

The shape of the rain garden in the plan is chosed arbitrarily, based on considerations of harmonious fit into the street space. The LTF productivity Q is determined by the maximum infiltration time (24 hours), Eq. (4):

$$Q = \frac{W}{24} \quad m^3/h \tag{4}$$

Next, it is necessary to determine by calculation the composition and depth of the filtration media in the rain garden, select plants and rhizomicrobial associations, which together can provide the required treatment of stormwater for 20 years. Such calculations should be based on the results of own or others experimental research. The results of experimental research [8] can be recommended. They allow to determine the operating time of the rain garden before replacing the filtration media.It should be noted that the research [8] does not take into account the presence of rhizosphere microorganisms, which make it possible to increase the efficiency of stormwater treatment from oil products and heavy metals [11].

3.6 Comparative technical and economic assessment of the options for the technological scheme of stormwater treatment

When a technological scheme of rain garden is developed, it is advisable to consider several of its options. This will allow to choose the optimal scheme in the future. This choice is based on a comparison of the technical and economic indicators for different schemes in the conditions of construction and operation of LTF.

3.7 Development of the rain garden design and the requirements for its operation and maintenance

Based on the selected technological scheme, a LTF design is developed for the selected section of the street. Finally, the requirements for the operation and maintenance of the rain garden are determined, including the control of the infiltration rate.

4 Conclusion

The use of “green system” for drain and treatment of stormwater from urban streets without an underground drainage system is environmentally and economically viable. Such systems based on bioremediation technologies have already found application in many countries (USA, Canada, New Zealand, etc.). But these systems are not yet being developed in Russia, although there is a regulatory and methodological framework for this. The main technical issues preventing the widespread use of rain gardens for treatment of stormwater in Russia include defining the optimal filtration media composition and parameters, and selecting of plants, suitable for the climatic conditions of Russia. More research is also needed on the use of microbiota in rain gardens and the search for the optimal composition of plant and rhizomicrobial complexes for operating in various regions of Russia.

References

[1] State report: On the state and use of water resources of the Russian Federation in 2017, http://www.mnr.gov.ru, 20.02.2020.

[2] ODM 218.8.005-2014: Guidelines for the maintenance of treatment facilities on roads (ROSAVTODOR), https://rosavtodor.ru, 25.02.2020.

[3] Trofimenko, Yu.V, Grigoryeva, T.Yu: Development of a methodology for assessing environmental risks associated with the treatment of surface runoff from roads: a report on research, Moscow Automobile and Road Construction State Technical University (MADI), Moscow, 2013.

[4] Veremeev, A.M., Tomilov, A.A., Ruchkinova, O.I.: Surface wastewater from roads, Modern technologies in construction. Theory and practice, 1 (2016), pp. 393-402

[5] Melekhin, A.G., Schukin, I.S.: Analysis of existing bioengineering facilities for surface runoff treatment and the possibility of their application in the conditions of the Western Urals, Bulletin of PNIPU, Construction and architecture, 2 (2013), pp. 40-50

[6] On-site stormwater devices: description and standard design steps: On-Site Stormwater Management Guideline, New Zealand Water Environment Research Foundation. Wellington, New Zealand, October 2004, https://ru.scribd.com/, 29.01.2020.

[7] Stormwater Retrofit Guidance Manual, Philadelphia Water Department, https://www.phila.gov, 31.05.2019.

[8] Schukin, I.S.: Purification of surface wastewater using phytofilters, Dis. Cand. Tech Sciences: 05.23.04, Perm national researched Polytechnic un-t Permian, 2019.

[9] Melekhin, A.G., Schukin, I.S.: The use of bioengineering facilities for the treatment of storm and melt water from urban areas, Vestnik PNIPU, Construction and architecture, 1 (2012), pp. 122-132

[10] Evstigneeva, Yu.V., Trofimenko, Y.V., Evstigneeva, N.A.: Bioremediation technologies for runoff treating from the road network of settlements, European Journal of Natural History, 1 (2020), pp. 81-87

[11] Trofimenko, Yu.V., Lobikov, A.V.: Biological methods for reducing roadside pollution, Overview information. Car roads, 5 (2001), pp. 1-96

[12] Recommendations on the calculation of collection systems, diversion and treatment of surface runoff of residential areas, sites of enterprises and determining the conditions for its release into water bodies: method. guide (FAU FCC), https://www.faufcc.ru, 25.02.2020.

[13] ODM 218.8.012-2019 Guidelines (guidance) on the predictive assessment of environmental impact during the construction and operation of public roads (ROSAVTODOR), https://rosavtodor.ru, 25.02.2020.

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ROAD AND RAILWAY EMBANKMENTS AS FLOOD-CONTROL DIKES

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Abstract

When designing roads and railways in the vicinity of river flows, it is a common situation for road embankments to serve as flood-control embankments as well. Such solutions are not the best ones; however, they are often imposed. This paper presents examples of embankments designed in this way, considering both roads and railways. Geostatic calculations of slope stability are performed for the designed solution of a railway embankment. Based on the analyses, some recommendations and guidelines have been given, emphasising the aspects on which particular attention should be paid if there is a necessity to design this type of embankments.

Keywords: road and railway embankments, flood-control dike, slope stability, geostatic calculations

1 Introduction

The construction of contemporary roads and railways, in particular at high speeds, imposes the design of the route using larger radii of line curvatures. In addition, the routes of these traffic lines are often laid along river valleys, even up to the riverbed itself. In these cases, the road embankment represents, at the same time, a flood-control dike as well. Design of this type of structures is very complex and requires solution of many problems.

The first problem that needs to be comprehensively considered when designing these types of embankments is of hydrotechnical nature, i.e. the water level regime in the riverbed as well as in the hinterland. It is necessary to analyse in detail the flood wave over a period of time from the beginning of the flood to return to normal stage. The problem of sudden change of water level and its influence upon the embankment stability (settlement, stability of slopes of the embankment, and suffusion) is of particular concern in these cases.

Another problem related to the impact of water is the flooding of the embankment and withdrawal of water from the embankment, which can often lead to subsequent settlement. When it comes to high-speed roads and railways, the occurrence of subsequent settlements of embankments is considered to be unacceptable, in particular for the case of high-speed railways when not even the slightest settlements are allowed.

When designing an embankment as a traffic object and a flood-control structure, it is necessary to analyse in detail the stability of the embankment slopes, the settlement of the embankment, and the load from the embankment and the traffic to the subsoil. In addition, it is necessary to investigate the possibility of the occurrence of liquefaction, since saturated sands appear quite often in the vicinity of the riverbed.

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2 Design of embankments

When a roadbase (embankment) partially enters a riverbed (i.e., the road embankment is also the river bank protection), special measures should be taken to protect the embankment:

- The embankment toe must be secured against undercutting and erosion. There are several technical solutions as shown in Fig. 1. The embankment toe is secured up to the low water level. These are just examples of how the protection of an embankment can be performed. In principle, there is no essential difference in the protection of these embankments considering the railway and road embankments. The only differences are related to the type and intensity of load, as well as to the construction of pavement structure (permanent way).
- In addition to securing the toe, the protection of the slope of an embankment is also important. Ensuring the slope of the embankment should extend up to a minimum of 1.0 m above the high water level. In view of the data on the floods that have taken place in recent years and of the global warming, it is necessary to revise the high water levels (1 in 100 years' water event).

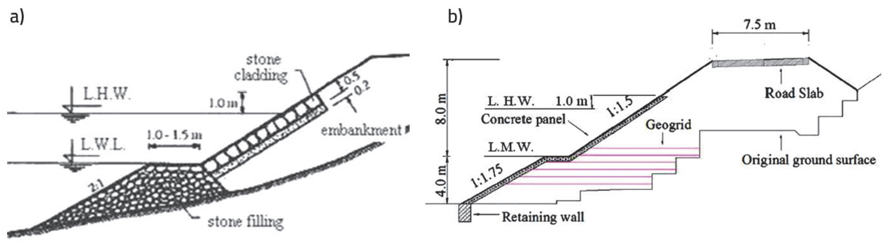

Figure 1 Protection of the embankment toe: a) on railways [1]; b) on roads [2]

When designing capital traffic lines (high-speed railways, highways, and roads of huge significance), in addition to the above mentioned measures, it is necessary to prevent water from entering the body of the embankment either from the protected or unprotected side of the embankment. In the present conditions, it is achieved by the application of waterproof foils. Nevertheless, in practice, when constructing these roads and railways, this is done in part or not at all, which can cause a number of negative consequences for the stability of the embankment and the safety of traffic. Therefore, in the design, it is necessary to analyse several factors in order to provide a safe solution from the aspect of the stability of the slopes of the embankment and its settlement. Factors influencing the design solution of a traffic communication embankment as a bank protection are:

- River morphology;
- Sudden decrease in water level;
- Embankment loading;
- Earthquake impact;
- Embankment settlement.

2.1 River morphology

The shape of a riverbed (cross-sections, longitudinal profile, river course) depends on the basic natural factors (hydrological, hydraulic, psamological, and other factors). The aforementioned forms are interdependent.

When designing an embankment, the impacts under which the riverbed is formed (water flow, properties of the material in the riverbed) should be analysed first.

The most significant influence is the flow of water. In addition to the basic flow of water in the river, which takes place under the influence of gravity, there are secondary flows in natural streams. The most important are centrifugal, frictional, and vortex flows.
Centrifugal flow occurs in curvity of river channel as a consequence of the uneven distribution of base flow velocities across the cross-section, as well as the uneven amount of water movement along the width and depth of the river stream. The flow is transverse to the base flow, so that with the longitudinal ones it creates a helical flow, as shown in Fig. 2. This flow plays a primary role in creating meanders and moving them downstream. At the bottom, the velocity of this stream is 1.5 higher than the velocity of the longitudinal stream, and at the surface it is much lower (about 15 % of the velocity of the longitudinal stream). Surface currents in the curvature plunge along the concave coast (downward flow), and they erupt to the surface in the convex coast zone (upward flow).
Friction flow is backflow, occurring at places of sudden widening of the river stream, behind sudden change in the longitudinal grade. The velocity of this stream reaches 30-50 % of the velocity of the basic stream, so that it also plays a very important role in the formation of the riverbed.
Vortex current is a consequence of friction flow. This stream draws the water with the deposit onto the surface of the stream and significantly influences the movement of the river deposit in the zones of the stabilisation structures – embankments. The velocity of the vortex current is the same order of magnitude as the velocity of the base stream, but may also be higher.
The properties of the material in the riverbed are also very significant. The geomechanical and geological composition of the material in which the riverbed was formed significantly influences its development and the morphological forms that occur. Therefore, geomechanical and geological data, obtained through proper investigations, are very important for the design of embankments.

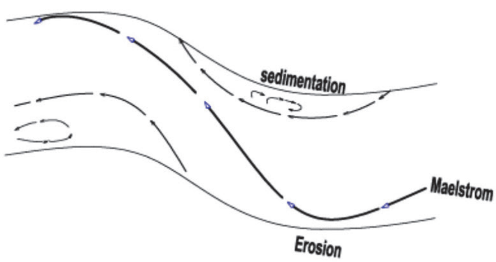

Figure 2 Helical flow in curvity of riverbed [3]

2.2 Sudden drop of water level

A sudden decrease in the water level has multifold effects: on the safety of the slope of the embankment, on the occurrence of suffosion, and on the settlement of the embankment.
The effect of a sudden decrease in water level on the stability of the slope of the embankment was considered by Morgenstern [4]. When the water level drops rapidly, the water from the embankment does not flow as fast as in the riverbed. Figure 3a shows a sudden decrease in water, based on which a diagram of values of the safety factor is given (Fig. 3b). The diagram is presented for one angle of inclination of the embankment slope.

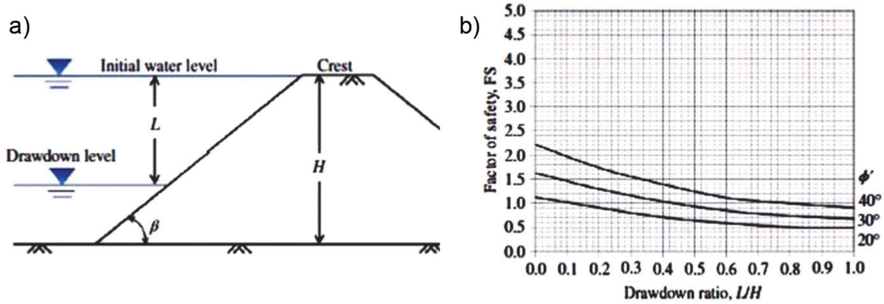

Figure 3 a) Illustration of rapid drawdown; b) Morgenstern charts for rapid drawdown for $c'/\gamma H = 0.0125$ and $\beta = 2{:}1$ [4,5] (c' is cohesion, γ is volume weight, and H stands for the height of embankment)

Suffosion of the soil (internal and external) can also be caused by a sudden decrease in water level. Internal suffosion is the redistribution of small particles that alter the local hydraulic conductivity of the material. External suffosion represents the extraction and evacuation of small particles, which yields an increase in hydraulic conductivity. This often leads to the degradation of the embankment, as well as to its settlement. Accordingly, when designing, it is necessary to analyse the criteria for the evaluation of suffusion, starting with the study of the size of the soil grains. The grain size distribution criteria should be linked to the hydraulic load rating. The hydraulic approach consists of estimating the load produced by the fluid flow to initiate the suffusion [6].
A sudden decrease in water level influences quick changes of the pore pressures in the embankment body, which in case of some embankment materials can lead to subsequent settlements of the embankment.

2.3 Embankment loading

In addition to constant load (embankment weight, traffic load), there are also incident loads (earthquake, flood). All these loads affect the stability of the slope of the embankment and the additional settlement of the embankment. Here, too, is necessary to emphasise the difference between road embankments which also serve as a river bank protection and embankments which are designed exclusively for protection against water. The former are characterised with significant traffic loads – static and dynamic, in which case no subsequent settlements are allowed. In case of latter ones, there is no traffic load, and the criterion for settlements is not so strict.

2.4 Influence of earthquake

The first impact of an earthquake is on the stability of the slopes of an embankment. Accordingly, in the design of roads and railways, it is necessary to take this influence into account. The effect of an earthquake reduces the coefficient of safety of the slope of the embankment. Another effect of an earthquake is possible occurrence of liquefaction. The soil, on which the embankment is located along the riverbed, is in most cases saturated and made of cohesionless material, and in some cases of soft loose materials. With this in mind, it is necessary to check all the criteria for the occurrence of liquefaction and to provide a solution in the design for its prevention.

2.5 Embankment settlement

One of the evident problems associated with the construction of an embankment is its settlement. Larger settlements are allowed in the case of flood-protection (bank-protection) embankments. In order to prevent settlements that would jeopardize the functionality of the embankment, the subsoil should be specially treated. The subsoil on the riverbank itself is usually soft soil, so it is necessary to take additional measures to improve it (soil replacement, construction of gravel piles, etc.). In addition, the quality of the material that is placed in the embankment must be in accordance with the required criteria. However, in the case of embankments–bank protections that are also used as embankments on roads or railways, settlements are not allowed (or possibly small values of settlements are allowed), which is especially true for railway embankments. In order to prevent their settlements, it is necessary to take the following steps:

- To ensure the subsoil of the embankment with the appropriate geotechnical measures that would prevent settlements of the subsoil due to the weight of the embankment, the impact of load, the impact of an earthquake excitation, as well as the effects listed in the second chapter of this paper;
- To build-in materials into the embankment body according to the technical criteria in terms of the quality of the material and the method of its placing;
- To protect the embankment from the effects of flooding and saturation of the embankment with water (especially for settlements, the predominant effect is a sudden drawdown of water level if water has infiltrated into the embankment body). This problem can be solved by installing waterproof foils in the toe and along the slopes of the embankment.

3 Geostatic calculations of slope stability

Due to the limited space of the paper, only the slope stability analysis is presented.
This section presents an analysis of the slope stability of an embankment on the Belgrade–Budapest railway line. In the Final Design of the railway line, a complete analysis was conducted based on an embankment model, which represents only one part of the route of the given railway line along the Danube River. The embankment is also a flood-control dike. This part of the analysis and the technical solution are not presented in this paper for the sake of the authorship protection. This paper presents a model of an embankment next to the Danube River that is not given in the Final Design of the railway. Information on the cross-section profile of the railway and soil characteristics was obtained from the Investor.
A cross-section of the embankment on one part of the Belgrade–Budapest railway line is depicted in Fig. 4. The slope stability calculation is shown for one case of water status in the Danube River, taking into account the total loading including earthquake effects. The seismic impact is considered for the 8th degree of seismicity (seismic coefficient $k_s = 0.06$).

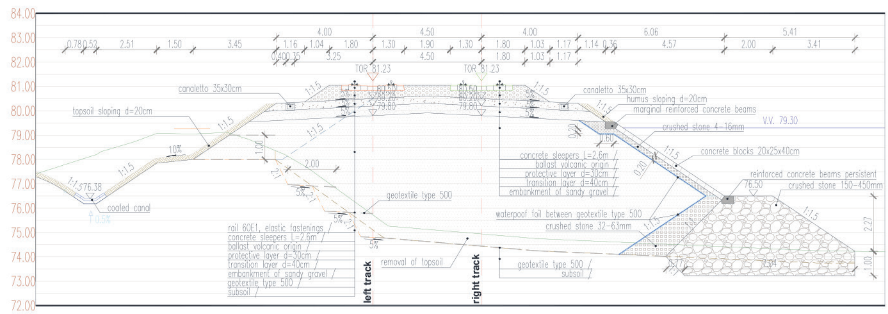

Figure 4 Embankment on the Belgrade–Budapest railway [7]

The designed embankment is ensured on the unprotected side (towards the river) by a waterproof foil, up to 20 cm above the high water level. In the analysis, two cases are considered: the case of the embankment with two loaded tracks and the case of one loaded track towards the river (Fig. 5). The earthquake action was taken into account in the calculation. The presented analyses are given for one water level in the river and the embankment, with a deeper slip surface, whereby different calculation methods were applied. The safety factor for the slope of the embankment considering two loaded tracks is in the range from 0.98 (Fellenius–Petterson) to 1.18 (Bishop). For the slope of the embankment loaded on a single track, the safety factor ranges from 0.94 (Fellenius–Petterson) to 1.22 (Bishop). The analysis of the slope stability of the embankment indicates that the safety factors are not satisfactory, i.e. an insufficient height of the embankment slope above the high water level was protected by a watertight foil.

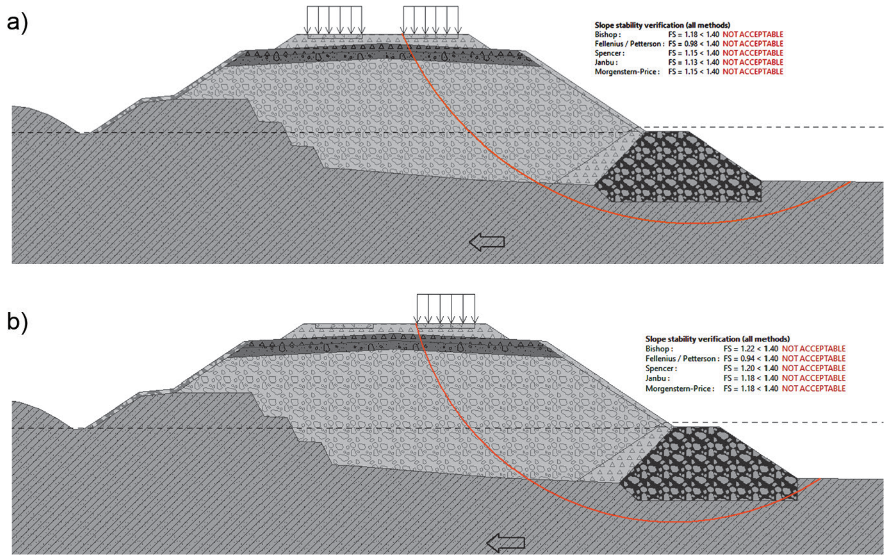

Figure 5 Slope stability analysis of railway embankment: a) two tracks loaded; b) one track loaded

This is the designed embankment solution within the Design of the high-speed railway line, for which the slope stability calculation has not been done in the designing stage. When the slope stability was checked within this study, as shown above, unsatisfactory safety coefficients were obtained. In order to achieve satisfactory safety coefficients, it is necessary to undertake the following:

- Redesign the geometry of the embankment by replacing the subsoil to the required depth and increasing the dimensions of the gabion in the riverbed;
- Install a waterproof foil on the unprotected side of the embankment, under the embankment, and on the protected side up to the required height;
- Provide a protection on the unprotected side up to the height of 1.0 m above the high water level with proper structures.

The presented example is highlighted with the aim of indicating to the designers that each type of embankment has to be analysed separately and comprehensively. Due to the limited length of the paper, the modified solution of the considered embankment is not shown here.

4 Concluding remarks

When designing roads and railways, in particular capital ones such as high-speed rail lines and highways, solutions of the road embankments that will also serve as flood-control dikes should generally be avoided. If, however, this cannot be avoided, the body of the embankment should be completely protected by watertight foils up to a height of minimum 1.0 m above the high water level. In addition, it is necessary to analyse all relevant factors, which affect the embankment stability and traffic safety on any basis.
The designed embankment of the railway line, presented in this paper, is only partially protected on the side down the river by a watertight foil of the height of only 20 cm above the high water level. The fact on inappropriately performed protective measures was confirmed by the slope stability analysis of the embankment, whose results indicate that the safety factors are not satisfactory. It is pointed out that it was necessary to redesign the embankment, but more importantly, the designers are told that each type of embankment on railways or roads that also serves as a bank protection should be analysed in detail, and not just make a cross-section of the embankment.
The presented analyses indicate the necessity for the comprehensive approach when designing this specific type of embankments.

Acknowledgement

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References

[1] Marusic, D.: Design and construction of railway lines, University of Split, Faculty of Civil Engineering, 1994.

[2] Wang, J., Li, Q., Yang, C., Zhou, C.: Dynamic response and damage character of road embankment under strong earthquake, International Journal of Distributed Sensor Networks, 14 (2015) 9, pp. 1-12, doi: 10.1177/1550147718794611.

[3] Mladenovic, B.M.: Arrangement watercourse, Institute of Water Management “Jaroslav Cerni”, Serbia, 2018.

[4] Morgenstern, N.R.: Stability charts for earth slopes during rapid drawdown, Geotechnique, 13 (1963) 2, pp. 121-133.

[5] Xiao, M., Barreto, D.: Geotechnical Engineering Design, John Wiley & Sons, 2015.

[6] Marot, D., Benamar, A.: Suffusion, transport and filtration of fine particles in granular soil, Erosion of geomaterials, (2012), pp.39-79.

[7] General design of the railway line Belgrade–Budapest, section Stara Pazova – Novi Sad, CIP, 2016.

SLOPE STABILISATION USING HIGH-TENSILE STAINLESS-STEEL WIRE MESH

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Abstract

Slope stabilisation systems with meshes made of high-tensile steel wire have been in use for 20 years and have proven to be reliable systems on loose rock and soil slopes. The optimization of the nailing pattern thanks to the high load bearing capacity of the system permits a reduction of the overall costs and represents an economical solution as well as an ecological solution. The expected service life with regards to corrosion depends, in addition to the corrosion protection used, on the environmental conditions and the corresponding chemical wear. The definition of aggressive corrosive environments for slope stabilisation projects are for example: coastlines by the sea, aggressive ground (low pH-value, sulphur content) and roads with de-icing (salt). If the micro-climatic conditions on site are known, systems made of stainless steel can be installed to counteract the aggressiveness and keep up a long service life. Stainless steel is a steel alloy, with a minimum of 10.5% chromium and less than 1.2% carbon content. The chromium produces a thin layer of oxide on the surface of the steel known as the 'passive layer'. This prevents any further corrosion of the surface.
In this contribution the pilot project for stainless steel-based slope stabilisation is presented, which has been installed 14 years ago in an aggressive environment, along the coastline in the UK. It was installed in 2007, with a stainless high-tensile steel wire mesh. Not only does the slope stabilisation mesh have to be made out of stainless steel, the additional components have to present the same protection to avoid the phenomenon of bi-metallic corrosion. Therefore, the nails, spike plates and press claws were as well made of stainless steel. After fourteen years, the slope is still undisturbed and the material in good conditions although exposed constantly to the sea breeze.

Keywords: slope stabilisation, stainless steel, high-tensile steel, natural hazards protection

1 Slope stabilisation using high-tensile steel mesh

1.1 What is a flexible high tensile steel mesh?

The so-called Tecco System is an engineered slope protection and stabilisation system which is used to stabilise steep slopes of unconsolidated soils and/or rocky material. It also prevents stones and blocks in disintegrated, loose or weathered rock faces from breaking out. Together with soil nails or rock bolts the mesh is fastened to the slope and pretensioned. The Tecco System consists of high-tensile steel wire mesh and associated with suitably adapted spike plates, clips for joining the mesh panels with full force transmission as well as boundary wire ropes and wire rope clips. The tensile strength of the mesh lies around 1'770 N/mm^2. This system has been validated by full-scale testing and is in use since 2000 (see Fig. 1).

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The nail grid can be optimized due to the high load bearing of the system. This reduces overall costs and thus represents an economical solution compared to conventional systems such as shotcrete and wire mesh with lower tensile strength. With the dimensioning concept RUVOLUM, the computational evidence can be provided. The level of safety and reliability are therefore significantly higher compared to constructive measures. The sustainability and cost-effectiveness of appropriate security systems can be determined taking into account the expected useful life.

Figure 1 1:1 large field test to verify the RUVOLUM concept (left) and installed slope stabilisation project (right)

1.2 Dimensioning concept

The design is based on the RUVOLUM concept for superficial instabilities. This online design tool is used to optimise the system by finding the most cost-effective combination of nail diameter, nail spacing and the mesh type.
The basis for the design is the support resistance of the individual components, which were determined in realistic, repeatable tests. The design concept RUVOLUM is described in detail by Rüegger [1] [2] and can be viewed and used under www.geobrugg.com. The design concept includes the investigation of near-surface, parallel instabilities as well as the investigation of local instabilities between the individual nails (see Figure 2).

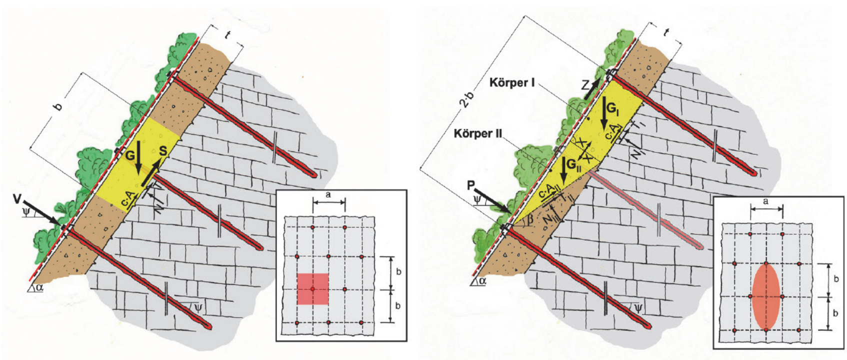

Figure 2 Near-surface, parallel instabilities (left). Local instabilities between the individual nails (right).

1.3 Long term experience of slope stabilisation using high tensile stainless-steel mesh

In this contribution two projects are described which lead to the development of a new material application by means of using high-tensile stainless steel to protect slopes in aggressive environments in terms of environmental corrosion. After material development, the first installation on site began in 2007, since then 14 years have passed and conclusion on its effectiveness can be drawn.

2 Environmental corrosivity and its effect on flexible steel protection measures against natural hazards

2.1 Corrosion protection on flexible slope stabilisation meshes

The design life of galvanized steel mesh products could be improved substantially since the introduction of zinc-aluminium coating in the 1980's. This kind of corrosion protection provides up to 3 – 4 times more durability compared to pure zinc coated wire products. In general, the corrosion protection of the Tecco System must be in accordance with the project-specific requirements. Geobrugg high-tensile wires come in Zinc-Aluminium galvanising as standard providing three times better corrosion protection than zinc galvanised wires. The corrosivity of the environment is described in classes, acc. to ISO 9223:2012-05, ranging from C1 to CX, where simply said C1 is unproblematic to a ZnAl coating, whereas this same coating would disappear in a matter of years in a CX environment.
However, with large quantities of the UK rail network near the coast, corresponding to C4 or C5 environments. Geobrugg undertook a research and development project in the early 2000s, developing stainless steel for the Tecco System, which had to still fulfil the definition of high-tensile steel. This was achieved around 2007.

2.2 Expected Useful Life of ZnAl coating

The expected service life in relation to corrosion depends on the environmental conditions and the corresponding chemical wear in addition to the corrosion protection used. DIN EN ISO 12944-2 [4] divides environmental conditions into 6 coronary conditions for the atmosphere and assigns these removal rates. Corresponding experiences were presented in 2013 at the FSR Continuing Education Seminar, on the basis of the valid standards and specialist literature [5]. The average values in a long-term assessment of ZnAl coating on high-tensile steel stabilisation projects obtained show a strong variation [6] and are plotted as diamonds in Figure 3 and extrapolated on the basis of the specialist literature [7] [8]. The sampling consisted in taken a sample of wire and analyse the remaining coating thickness after a certain amount of years, in the case of figure 3, between 20 and 25 years. This means that even within individual measures, different removal rates can be assumed, depending on the situation. In extreme cases, different categories of corrosivity may occur in an installation. On this basis, an expected useful life of greater than 50 years for a single sample and a larger 70 years for all other samples is estimated for the values determined (see Figure 3).

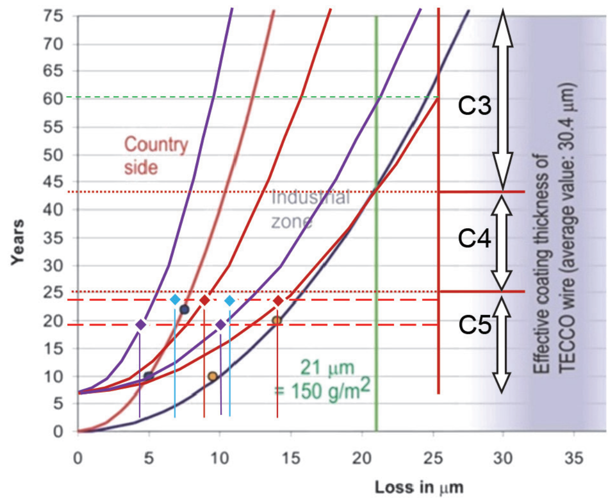

Figure 3 Graphic representation of the expected useful life for Tecco meshes in years (y-axis), based on the literature (circles) [7] and the sampling done in [6], represented by the coloured diamonds. The estimates refer to Nünninghoff, 2003 [7]. The green line represents the minimum thickness of ZnAl on a high-tensile steel wire, whereas the purple zone represents the average thickness of coating on the Tecco wires. On this basis, the removal process was extrapolated. The associated corrosivity categories are shown on the right.

2.3 The development of a high-tensile stainless-steel mesh

The pilot project, which will be highlighted in the next section, set out a "design working life" of 120 years for all new structures, therefore only marine grade stainless steel turned out to be the only suitable meshing solution. Therefore, Tecco G65/3 Stainless is a high tensile steel mesh with a 3mm wire diameter and a tensile strength of 1650 N/mm2, constructed from 1.4462 [3] marine grade stainless steel and has a tensile strength ≥ 140 kN/m. The Tecco Stainless is a fully compliant system with stainless steel spike plates, connection clips, boundary ropes and wire rope clips all made in the stainless-steel quality. Tecco Stainless steel is integrated in the online design tool RUVOLUM separately, due it slightly lower tensile strength and can be designed accordingly.

3 Over 10 years of experience in the field with high tensile stainless-steel mesh – the pilot project and a follow-up project

As previously mentioned, a pilot project between Network Rail and Geobrugg was set out to develop a high-tensile steel mesh in stainless steel quality, that withstands the aggressive environment of the seacoast with highly salty and humid air. Two sites of the pilot project will be discussed briefly. A third project undertaken a few years later is equally discussed to highlight the fact that coastal corrosive areas are as dangerous when being salty and dry instead of humid. Indeed, the salt itself is attacking the galvanized steel, but also humidity in general is a damaging factor, but the dryness has also to be considered, since the salt will then not be washed down regularly from the mesh.

3.1 The Cambrian Rock Cutting Campaign of Network Rail

This campaign is a multi-year pilot project for with a number of eight sites along two railway lines in West Wales, UK. Site investigations and historic rockfalls have indicated the requirement of protection measures. Five of eight sites are close to the sea and exposed to salt spray water, corresponding to a corrosion category at the high end of C3 [4, 9]. To achieve a 120-year design life a high quality of rockfall netting was necessary to withstand the high environmental corrosivity. In total 15,800 m² of Tecco Stainless and 2893 rock bolts were installed over 6 projects and 3 years.

3.1.1 Parton

The site comprises a 35m high slope above the eastern side of the railway line on the Cumbrian Coastline. The line is single tracked and a vehicle track runs parallel to the railway line, situated between the toe of the slope and the rail. The northern half of the site comprises a 35° slope with the cut being formed in rock for approx. the lower 3m. The remainder of the site consists of a sub-vertical rock slope of approx. 4m height with a sub-horizontal bench at its crest below a 35° soil slope of approx. 30m height. Rock fall netting was installed in 2002 to mitigate the risk of falling rock debris from the lower rock slope. Netting was also anchored on the soil slope for approx. 8 to 12 m above the top of rock slope as a temporary containment measure [10]. There was evidence that standard galvanized products do not stand the test of time and the rockfall netting had to be replaced. This was done in 2007 and it has proved to be successful: thanks to the corrosion-resistant system, the embankment right next to the railway line on the coast is still in perfect condition after 14 years in aggressive sea air (see Figure 4). The findings of this project were decisive for the development of Tecco Stainless and its expansion to several more projects worldwide.

Figure 4 Stabilised soil slope above the rails, with high-tensile stainless-steel mesh. The proximity to the see is evident as well as the exposure to constant salt spray.

3.1.2 Aberdovey

The site of Aberdovey belongs as well to the Cumbrian Rock Cutting Campaign, where the slopes around and above the tunnel portal were protected with stainless high tensile steel mesh, since it is equally exposed to the sea (see Figure 5).

Figure 5 The exposure of the rail and the slope at Aberdovey tunnel approach (left) and stainless-steel mesh installation at tunnel portal (right).

The success of this pilot project led to a new consideration of looking at the whole-life costing approach of a geotechnical meshing project. It can be shown that the increased material costs of Tecco Stainless are only a small percentage of the total package costs compared with that of a standard galvanised or plastic-coated mesh product, which needs regular maintenance and timely replacement.

3.2 La Gomera, Canary Islands, Spain

La Gomera is an island of the Canaries, on whose coasts numerous unstable embankments are in frequented locations. This was also the case in the popular hiking region “Pescante de Agulo”, where a practically vertical rock face had to be secured in the immediate vicinity of the shore. For the Spanish Coastal Protection Authority, it was clear that only a corrosion-resistant solution is possible.
Among other protection solutions, a surface of 5000m^2 was stabilized with this stainless-steel mesh. In order to ensure that the entire system can withstand permanently the saline and dry air, the connection clips between mesh panels, the spike plates and the wire rope clops were used in stainless steel quality [11].

Figure 6 Installation of stainless steel on a rocky outcrop on La Gomera in 2015

4 Conclusion

Several installations over the last decade in Italy, Gibraltar, United States of America, Costa Rica and more show the usefulness of the stainless high-tensile steel development. An estimate of the expected useful life for slope stabilisation protection systems is difficult and depends on many factors. In most cases, these can have a very large variation and the estimates become correspondingly inaccurate. For many applications a useful life can be assumed of greater than 50 years to greater than 70 years. Depending on the location, chosen corrosion protection and corrosivity category, a shorter service life has to be expected. If this is known, e.g. stainless-steel systems, with a significantly longer service life, should be provided. In comparison, a zinc coating on diagonal meshes, according to Krauter (1996), have rather a useful life of 20 - 25 years [12].

References

[1] Rüegger, R., Flum, D., Haller, B.: High strength meshes made of steel wire for surface protection in combination with anchorages and anchorages (detailed design instructions), Technical Academy Esslingen, 2nd colloquium Building in Soil and Rock, January 2002.

[2] Rüegger, R., Weingart, K., Bickel, M.: Flexible surface protection systems made of high-strength wire meshes in combination with ground and rock nails, 3 case studies. Technical Academy Esslingen, 3rd colloquium Building in Soil and Rock, January 2004.

[3] Definition and classification of grades of steel, German version EN10020:2000-07, 2000 Beuth

[4] DIN Paperback 286: Corrosion protection of steel by coatings and coatings 4, DIN EN ISO 12944-1 to DIN EN ISO 12944-8; 1998 Beuth

[5] Gröner E., Roduner, A.: Expected useful life of "stone impact protection nets": corrosion protection, long-term behavior, perspectives, Continuing education seminar of the Research Centre Rutschungen, Mainz 2013.

[6] Gröner E., Roduner, A.: Langzeiterfahrungen von Böschungsstabilisierungen mit hochfesten Drahtgeflechten. Continuing education seminar of the Research Centre Rutschungen, Mainz 2018.

[7] Nünninghoff, R.: long-term experience with Galfan, Wire 3/2003.

[8] Nünninghoff, R.: Comparative corrosion short-term tests on hot-dip galvanized and galfan-galvanized steel wires; 1998 University of Wuppertal

[9] Corrosion of metals and alloys. Corrosivity of atmospheres, Classification, determination and estimation. BS EN ISO 9223, 2012.

[10] Network Rail: LNW-001436 Parton (Bag of Rocks), Options Report, 2007.

[11] Tecco stainless ready for the island, https://www.geobrugg.com/en/TECCO-Stainless-ready-for-the-island-8274.html?markierung=lagomera, December 2015.

[12] Krauter, E., Scholz, W.: Long-termbehaviour of protectivenets against rockfall, Geotechnics 19 (1996) 2

TRACK BED DESIGN AND EVALUATION METHODS

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Abstract

Track bed consists of structural layers and subsoil, and in the case of the ballasted track, also the structure comprises ballast bed. Together, this structure creates a layered half-space in which the load from the rail supports is distributed. The paper is focused on static analysis of track bed as a layered half-space structure.
Currently, the so-called DORNII method, developed in the Soviet Union in the mid-1950s, is still used for the assessment of the track bed structure. The evaluation of this structure is based on the assessment of the deformation resistance. The deformation resistance is assessed through the value of the deformation modulus of the sub-ballast surface and the subsoil surface. The deformation modulus is measured both in the geotechnical survey and in the acceptance process of construction works. When designing the track bed, the deformation modulus is taken into account as an essential material characteristic.
Neither the stress values relative to the depth nor deflection is analysed in the currently used methodology. The DORNII method is empirical but allows calculation of vertical stress and deflection. The authors wondered whether this method would not be too inaccurate because of increasing train speed and axle load. Two methods, analytical and finite element, were chosen for comparison. The article describes the specific analysis procedures and compares their results.

Keywords: railway substructure, track bed, static analysis, substructure design, stress diagnostic

1 Introduction

At present, the trend of increasing the design speed of railway lines is quite evident, and in the case of new lines, the construction of high-speed lines prevails. As the speed of rolling stock increases the dynamic load on substructure increases. Therefore, it is necessary to pay attention to the entire track bed's sufficient deformation resistance during the design. This deformation resistance should guarantee support for the track structure, so that there are no significant, and especially not permanent, deflections under passing rail vehicles. Irreversible deflection of the track bed causes a permanent change in the track quality and thus, in turn, greater dynamic load on the railway superstructure and substructure, which, among other things, results in a significant reduction in the lifetime of the track [1].
Insufficient deformation resistance, therefore, leads to track undesirable settlement. To limit these settlements, it is necessary to well identify the processes that take place in the track bed due to loading. This is relatively difficult because the space under the track skeleton is not limited. Several different empirical and numerical methods attempt to describe these mechanical processes.

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One of the mentioned methods is the DORNII (Dorožnyj naučno-issledovateľskij institut) method [2]. It is a partly analytical and partly empirical method. It is included in the Czech infrastructure manager regulation and serves to design structural layers of the railway substructure concerning deformation resistance [3]. With its use, it is possible to determine the fields of stresses and displacements in the track. The question remains whether this is a realistic view and a suitable tool for designing a deformation-resistant structure, as this method simplifies and omits many things. The semi-analytical method of layered half-space (LHSM), used in road engineering, will be used for its comparison. The second one is the finite element method (FEM), a numerical method for solving differential equations with a relatively wide scope. Both of these methods provide a comprehensive overview of stresses and displacements in the substructure.
At the end of the article, the results of the calculation of displacements and stresses calculated according to DORNII, LHSM and FEM for a typical structural composition of the railway substructure in poor geotechnical conditions are tested.

2 Description of analysis methods

2.1 DORNII Method

Currently, in the Czech Republic, the DORNII method is used to determine the deformation resistance. The method is partly empirical and partly analytical. N. N. Ivanov developed it in the 1950s [2]. The method allows calculating the equivalent deformation modulus of a multi-layered track bed structure. In particular, such thicknesses or modulus of deformation of the track bed layers are sought so that the resulting modulus of deformation E_{eq} of the whole system guarantees the required deformation resistance.
In order to determine the required equivalent value of deformation of the whole structure E_{eq}, it is necessary to know the deformation characteristics of the materials used for individual layers (E_i, v_i). The DORNII method considers soil deformation only in the column of material that is loaded. This fact means that neither shear stress nor horizontal stress in the soil, which would otherwise arise at the two structural layers' interface, is considered. Based on the measurement of stress on the ground plane, M. I. Jakunin [2] recommended the following empirical relationship for the course of stress to a depth of 1.0 m:

$$\sigma_z = \frac{q}{1+\eta\left(\frac{z_e}{D}\right)^2} \tag{1}$$

in which: q is the pressure from the vehicle wheel acting on the ground surface [MPa], z_e is the design depth a quasi-homogeneous space [m], D is the diameter of the reference circular area of the wheel pressure [m], *n* is a dimensionless parameter, in a two-layer system that *n* = 1, in a single layer *n* = 2.5 (when $z_e = z$).

The equivalent depth z_e is determined according to the equation (see Fig. 1):

$$z_e = z + h_e - h_1 = z + (n-1)\cdot h_1 \tag{2}$$

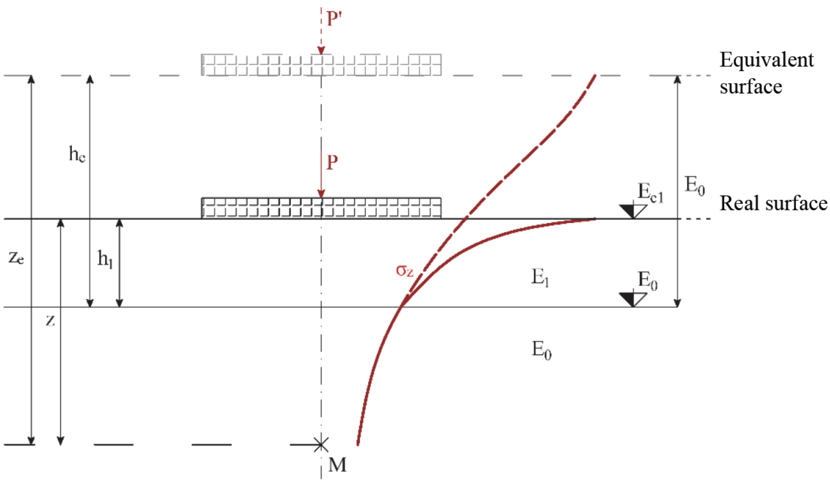

Figure 1 Pokrokovský's model [3] of equivalent layer

The equivalent deformation modulus and vertical deflection for a two-layer structure can be expressed:

$$E_{e1} = \frac{E_0}{n^{2,5}\left[1-\frac{2}{\pi}\left(1-\frac{1}{n^{3,5}}\right).arctg\left(k_2.n\right)\right]}; \quad n^{2,5} = \frac{E_1}{E_0} \tag{3}$$

$$y_1 = \frac{p \cdot D}{E_0}\left[\frac{\pi}{2}-\left(1-\frac{1}{n^{3,5}}\right)\cdot arctg\frac{h_1 \cdot n}{D}\right] \tag{4}$$

where: $n^{2,5}$ is the deformation characteristic of the system, E_0 is the deformation modulus of the lower layer material [MPa], E_1 is the deformation modulus of the upper layer material [MPa], E_{e1} is the equivalent deformation modulus of the whole structure [MPa], h_1 is the upper layer thickness [m], D is the diameter of the circular load plate [m].

2.2 Layered half-space method

The layered half-space method was initially developed for road engineering [4], [5]. Describes a situation where a vertical circular wheel load is applied to the surface of a multilayer structure. The circular load is idealized because other load components act on the road. In addition to this vertical load, which comes from the vehicle's weight, there is a horizontal surface load due to acceleration and braking and a shear load due to centrifugal forces in the directional curves.

In railway structures, the distribution of forces from railway vehicles' running occurs in a completely different way. However, the layers' design concerning the railway substructure's deformation resistance is based on applying pressure using a circular plate to the structural layer. The same task can be investigated by the layered half-space method.

The spatial problem of the pressure acting from a circular load plate of diameter D on the track bed's structural layer is transformed into a two-dimensional problem in the layered half-space method since it is an axially symmetric problem. That means with a suitable location of the origin of the reference axes in the centre of the loaded circular surface, the task is transformed from Cartesian coordinates to cylindrical coordinates, and thus axial symmetry and significant simplification are achieved.

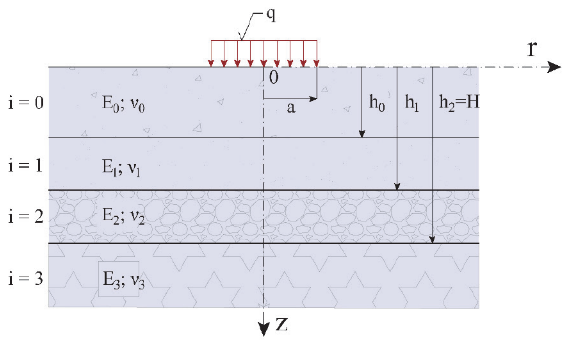

Figure 2 Layered half-space scheme

The calculation of the stress and displacement field is based on the solution of the compatibility condition, which is expressed by the following equation:

$$\nabla^4 \phi = 0 \tag{5}$$

in which ϕ is the stress function. This stress is considered for each of the layers. For a system with axially symmetric stress distribution, the differential operator ∇^4:

$$\nabla^4 = \left(\frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{\partial^2}{\partial z^2} \right)\left(\frac{\partial^2}{\partial r^2} + \frac{1}{r}\frac{\partial}{\partial r} + \frac{\partial^2}{\partial z^2} \right) \tag{6}$$

where r is the coordinate for the radial direction and z for the vertical direction. The initial equation (4) is a fourth-order differential equation, so the resulting equations describing stresses and displacements will contain four integration constants, which can be calculated from boundary conditions and compatibility conditions. It is assumed:

$$\rho = \frac{r}{H}; \qquad \lambda = z / H \tag{7}$$

where r is radial coordinate, H is the total thickness of the structure, and z is the defined depth. The stress function given by the Eq. (4) is:

$$\phi_i = \frac{H^3 J_0 (m\rho)}{m^2} \left[A_i e^{-m(\lambda_i - \lambda)} - B_i e^{-m(\lambda - \lambda_{i-1})} + C_i m\lambda e^{-m(\lambda_i - \lambda)} - D_i m\lambda e^{-(\lambda - \lambda_{i-1})} \right] \tag{8}$$

Function φ_i is the stress function for the i-th layer that satisfies the initial equation and in which J_0 represents a Bessel function of the first kind and zero-order; m is a parameter, A_i, Bi, C_i; D_i are integration constants that are determined from boundary conditions and compatibility conditions. The index i takes values from 1 to n and refers to the layer's order, starting with the surface layer. Substituting this equation into the Eq. (7), we obtain calculation equations for the quantities of normal stress, shear stress, vertical and radial displacement, generally R*.

We use the Hankel transform to find stresses and displacements for a uniform load q distributed on a circular surface, defined by radius:

$$\overline{f}(m) = \int_0^{\alpha} q\rho J_0 (m\rho)\; d\rho = \frac{q\alpha}{m} J_1 (m\alpha) \tag{9}$$

where α = a / H. Hankel's inverse transform $\overline{f}$ (m) is:

$$q(\rho) = \int_0^\infty \overline{f}(m) m J_0(m\rho) dm = q\alpha \int_0^\infty J_0(m\rho) J_1(m\alpha) dm \tag{10}$$

Assuming the load q negative, the corresponding quantity R is calculated:

$$R = q\alpha \int_0^\infty \frac{R^*}{m} J_1(m\alpha) dm \tag{11}$$

The layered half-space solution assumes that on all layers interface the same normal stress, shear stress, vertical displacement, and radial displacement occur. To calculate the 4n constant A_i, Bi, C_i; D_i boundary conditions are introduced:

$$\left(\sigma_z^*\right)_i = \left(\sigma_z^*\right)_{i+1}; \left(\tau_{rz}^*\right)_i = \left(\tau_{rz}^*\right)_{i+1}; \left(w^*\right)_i = \left(w^*\right)_{i+1}; \left(u^*\right)_i = \left(u^*\right)_{i+1} \tag{12}$$

On a surface for which i = 1 and $\lambda = 0$, the following boundary conditions apply:

$$\left(\sigma_z^*\right)_1 = -mJ_0(m\rho); \left(\tau_{rz}^*\right)_i = 0 \tag{13}$$

The stresses and displacements must logically disappear together with the increasing depth ($\lambda \rightarrow \infty$), therefore for the lowest layer i = n the following applies:

$$A_n = C_n = 0 \tag{14}$$

The solution was refined for the surface using Richardson's extrapolation, where the integral in Eq. (10) [6]:

$$I = \int_0^\infty f(\xi) d\xi \tag{15}$$

was modified to:

$$I = \lim_{b \to \infty} \int_0^\infty f(\xi) e^{-\xi^2 b} d\xi \tag{16}$$

2.3 Finite element method

The model for FEM analysis was created in the PLAXIS 3D program. The dimensions of the rectangular model in this analysis were set to dissipate the stress in its peripheral parts. The stress components expressing the stress and displacements did not change. The model's specific dimensions were set to 6 m in width and length and 7 m in depth. The load was placed in the centre of the model.

3 Example of track bed design

The particular analysis methods were compared for several specific track bed structures. Below is a specific case of such an analysis.

3.1 Input parameters

A uniform load q = 0.2 MPa was applied at the level of the bottom of the railway body to an area with a diameter of the circular plate D = 0.30 m. The deformation modules at the substructure plane level and the subsoil plane were determined to respect the required minimum deformation resistance, which must match regulation rules for the reconstructed line for speeds of 120 km.h^{-1} to 160 km.h^{-1}.

Oedometric modulus and Poisson's ratio were chosen as input deformation characteristics for the DORNII method. The oedometric modulus was selected for calculation because horizontal deformations are not considered when loading with a circular plate. Permanent deformations are included in the analysis the same way, as is the case with the oedometric test. The modulus of elasticity and the Poisson's ratio were used for the layered half-space method. A constitutive relation that links tension and transformation is Hooke's law for the LHSM. The modulus of deformation and the Poisson's ratio were chosen for the finite element method. A linearly elastic model was used as a constitutive relation.

3.2 Example of track bed analyses

Track bed with a layer of stabilized soil is specially designed when necessary to increase the subsoil's strength and deformation resistance, further if required reduction of the thickness of the structural (base) layer or improve the subsoil resistance to frost.

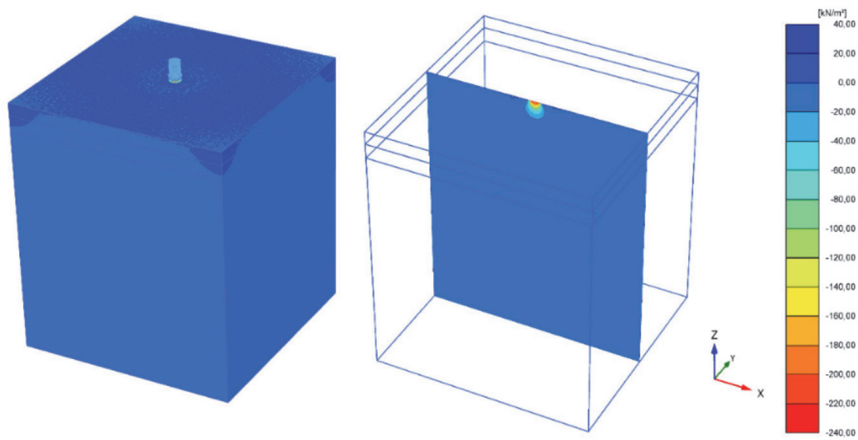

Figure 3 Vertical stress σ_z [kPa] calculated by FEM in PLAXIS software

Layers of following input parameters were considered: Fine crushed stone mixture 0/32, E_{def} = 80 MPa, E_{oed} = 96 MPa, E = 89.6 MPa, v = 0.25, h = 0.35 m; Lime soil stabilization, E_{def} = 70 MPa, E_{oed} = 84 MPa, E = 78.4 MPa, v = 0.25, h = 0.4 m; Clay with high plasticity, E_{def} = 8 MPa, E_{oed} = 17.1 MPa, E = 13.9 MPa, v = 0.4.

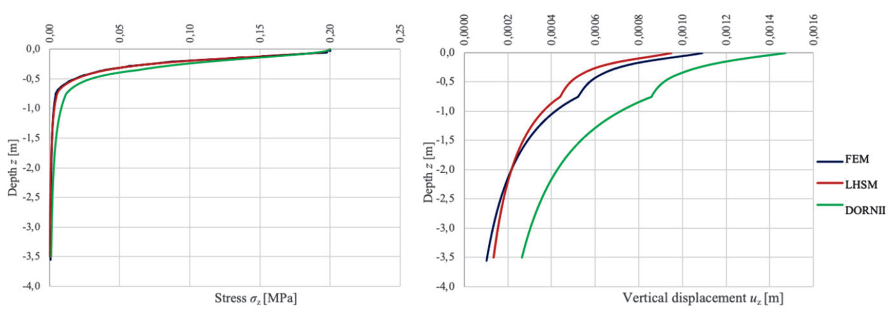

Figure 4 Comparison of vertical stress and vertical displacement under the centre of applied load for methods used

From the comparison of the results, it can be observed that there are no significant differences between the methods for stress results. The course of stress along the depth is not so smooth, which is given by the relative flexibility of the last layer; however, all three methods treat this fact in the same way. The stresses, determined by the methods of FEM, LHSM and DORNII, do not differ significantly in principle. Besides, stress is a theoretical quantity that only describes the state of the structure. The link between stress and strain is Hooke's law, so it is impossible to conclude purely based on knowledge of stress whether the strain resistance of a system, assessed by any method, is sufficient. The vertical deflection better expresses the deformation resistance. The more flexible the structure, the greater the deflections will occur. In this, DORNII was very consistent, and the largest decrease was always calculated for all types of structures compared to FEM and LHSM. If we wanted to limit the vertical deflection and get, for example, to the value found by the LHSM, which was the lowest for all cases, we would have to design based on DORNII a structure considerable rigid.

4 Conclusions

So DORNII does not correctly describe the processes that take place in the substructure. It neglects horizontal deformations, but as a tool for designing a sufficiently deformation-resistant railway substructure, it will stand on the safe side. The calculated equivalent modulus of the system, based on this theory, is very likely to be lower than the real one. So DORNII works on the safe side. In addition, it is incomparably computationally simpler.

Trends in the calculated results of stresses and displacement received by particular methods were compared up to a depth of 3.5 m. The stresses along depth did not differ almost for all three methods. The resulting deflection observed in the FEM was always greater than in the LHSM, however not significantly. The results of vertical displacement were always almost the same for both of these purely computational methods. In contrast, the total deflection at DORNII was up to a third larger for all investigated track bed structures compared to the LHSM calculations.

The disadvantage of the DORNII method is its limitations concerning the track bed structure. The method requires that the deformation modulus of the layers grow upwards in the structure; furthermore, it is not possible to include any geosynthetic reinforcement elements in the calculation. The DORNII method does not provide stress and displacement fields in the results and does not allow assessing the track bed structure using the limit state method. Since it does not provide a reliable calculation of the vertical deflection, it cannot be used to design the track elasticity.

The DORNII method is very simplified; for example, it omits horizontal strain and stresses. On the other hand, there are very good experiences with FEM in geotechnics, it provides very high-quality outputs and describes mechanical processes more comprehensively. Its results were very close to the layered half-space method. Most likely, DORNII does not have sufficient informative value about what actually happens in the substructure and does not describe these processes' consequences (too high deflection). However, as a methodology for designing a track bed structure, it guarantees greater deformation resistance than the case with FEM and Layered half-space method.

Acknowledgment

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References

[1] Lichtberger, B.: Track compendium. 2. vyd. Hamburg: DVV Media Group, 2011. ISBN 978-3-7771-0421-8.

[2] Babkov, V.F., Bykovskij, N.I., Gerburt-Gejbovič, A.J.: Gruntovedenie i mechanika gruntov, Moscow, Dorizdat, 1950.

[3] Ižvolt, L.: Železničný spodok: namáhanie, diagnostika, navrhovanie a realizácia konštrukčných vrstiev telesa železničného spodku, Žilina, Žilinská univerzita v Žilině, 2008.

[4] Huang, Y.H.: Pavement Analysis and Design, Prentic–Hall. 2004.

[5] Maina, J.W., Matsui, K.: Developing software for elastic analysis of pavement structure responses to vertical and horizontal surface loadings, Transportation Research Record, 2004, pp. 107–118, doi:10.3141/1896-11

[6] Maina, J., Kunihito, M.: Elastic multi-layered analysis using DE-integration. Publications of the Research Institute for Mathematical Sciences, 41 (2005) 4, pp. 853–867

CONCRETE CANVAS APPLICATIONS

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Abstract

This presentation reviews few different applications of Concrete Canvas and Concrete Canvas Hydro material. Concrete Canvas is a flexible, concrete filled fabric which provides a thin durable concrete layer when hydrated or fully waterproof layer in it's Hydro variant. It's possible to apply concrete Canvas in many different ways, some of them are slope protection, weed suppression, canal lining, lagoon lining and erosion control. We are going to present our application of material at 3 different projects: securing waterproofness of area around gas tanks in luka Ploče, preventing fire outbreaks along railway in Slovenia and water drainage at hardly accessible locations in Serbia. Each project had it's problems where we found Concrete Canvas suitable replacement for conventional concrete solutions. In that way we delivered Investor faster and high quality solution which in the end resulted in lower expanses. We will show comparison between Concrete Canvas and standard methods that are usually used.

Keywords: concrete canvas, concrete canvas Hydro, slope protection

1 Introduction

Concrete Canvas is modern concrete alternative for erosion control applications, providing erosion control, abrasion resistance and weed suppression. Also it provides excellent impermeability. Since today material is used all over the world for a range of applications. Some most used applications are: Channel lining, slope protection, bund lining, concrete repair, weed control, culvert lining, lagoon lining, gabion protection, etc.

1.1 Material parameters

Regarding the application there are two types of material that can be chosen from, Concrete Canvas and Concrete Canvas Hydro.

Concrete Canvas
Concrete Canvas consists of a 3-dimensional fibre matrix containing a specially formulated dry cementitious mix. Underneath 3D fibre matrix, a PVC backing that ensures complete waterproofness can be found. Material gets it's compressive strength after hydration with water. Fibres inside material reinforce the cementitious mix preventing crack propagation and providing a safe plastic failure mode. As a result Concrete Canvas provides good alternative to traditional concrete. Material comes in 3 typical values. It can be 5mm, 8mm or 13mm thick. 24 hour Compressive strength of cementitious mix gets up to 50MPa while 28 day Compressive strength totals 80MPa. Concrete Canvas is Freeze-Thaw, weather, chemical, root, fire and abrasion resistant.

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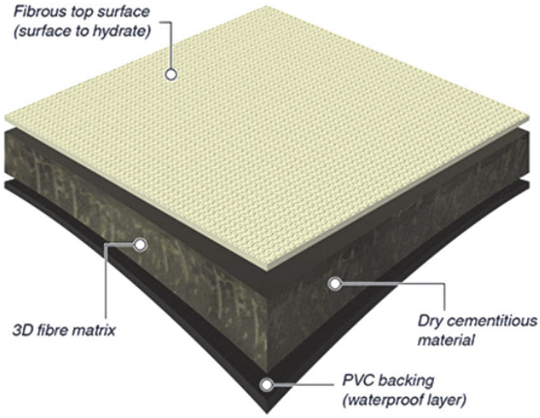

Figure 1 Concrete Canvas layers

1.1.1 Concrete Canvas Hydro

Concrete Canvas Hydro combines the concrete filled geotextile technology of Concrete Canvas with a highly impermeable, chemically resistant geomembrane liner as addition to regular PVC backing. The geomembrane liner allows joints to be thermally welded with a double or triple weld with a high-visibility welding strip that allows joints to be pressure tested easily on site. Material consists of fibrous top surface, dry cementitious material, 3D fibre matrix and PVC backing same as Concrete Canvas. Concrete Canvas Hydro has been independently tested to BS-EN-1377 to have a hydraulic conductivity better than $1x10^{-12}$ m/s. Also material has shown to have excellent resistance to a wide range of chemical reagents, including hydrocarbons, digestates and acidic leachates that makes it suitable for using within the Petrochemical and Oil and Gas industries.

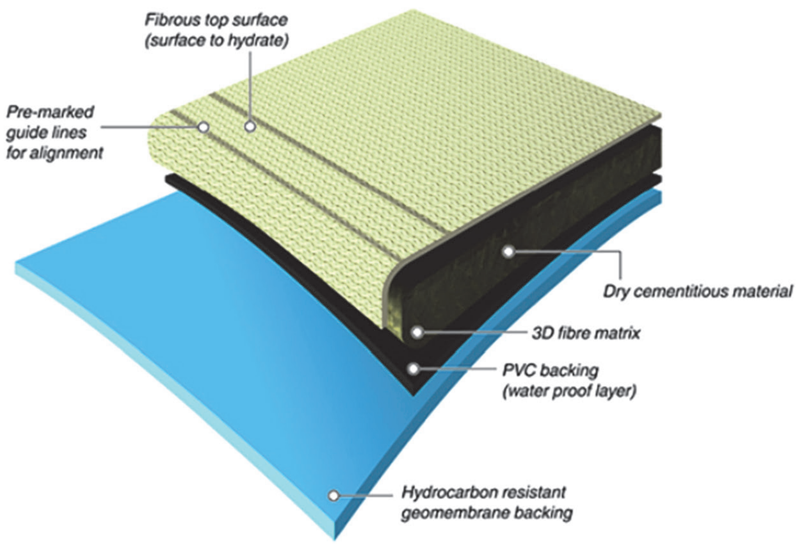

Figure 2 Concrete Canvas Hydro layers

2 Concrete Canvas projects finished by Monterra d.o.o. in Croatia, Slovenia and Serbia

2.1 Oil and Gas terminals in Luka Ploče (NTF), Croatia

During 2019 and 2020 Concrete Canvas Hydro was used to provide secondary containment to a total of approximately 10.000 m^2 concrete tanks in Luka Ploče operated by NTF (Naftni terminali Federacije).
There are 12 tanks of various sizes within the site that are used to store different petrochemical products: gasoline, jet fuel, gas oil, biodiesel and diesel. As part of ongoing improvements NTF had to improve secondary containment around 3 of it's storage tanks.
As a solution to a problem Monterra together with project designers gave Investor better, faster and after cost comparison cheaper solution than conventional concrete solutions.

2.1.1 Preparation of the substrate

Prior Concrete Canvas Hydro installation, base material inside concrete tanks had to be replaced. Replacing base material resulted in soil strength higher than 45Mpa while parameters of the former material showed strength lower than 20Mpa. After replacing base material with gravel, thin layer of fine sand and geotextile was made on top so Concrete Canvas Hydro can lay perfectly on the ground.

2.1.2 Deployment of the Concrete Canvas Hydro

Supplied in bulk rolls of up to 150m2, Concrete Canvas Hydro is deployed via excavator across tanks before being cut to length using basic hand tools.

2.1.3 Thermal welding and pressure testing of the joints

Welded in accordance to guidelines, CCH incorporates a high-visibility welding strip, allowing the joint to be thermally bonded with triple-track air channel.

2.1.4 Hydration

Following the welding and testing of the joints, Concrete Canvas Hydro was hydrated. CC Hydro cannot be over hydrated so procedure was repeated.

Figure 3 Step by step Concrete Canvas Hydro installation


Figure 4 Concrete Canvas Hydro after 28 days

2.2 Slope protection against fire along railway Kopar - Divača

During October 2017 Concrete Canvas was used to provide fire protection on slopes along railway Kopar – Divača. Before Concrete Canvas installation on this part of the railway there was risk of fire caused by sparkles during train traffic. Investor started project using classical shotcrete method which made contractor dig a lot of small canals underneath rail for using hoses, there was constantly needed for moving equipment along the railway and sometimes railway traffic had to be completely stopped. After numerous problems Investor agreed to Concrete Canvas 8mm.

2.2.1 Preparation of the substrate
Prior Concrete Canvas installation, all unstable/sharp rocks and larger vegetation was removed from the surface. On top part of the slope a small canal (dimensions were 15cm in width and 10cm in depth) was made so we could anchor top part of Concrete Canvas and stop it from twisting while it hardens. At the end of application canal was covered with soil.

2.2.2 Deployment of the Concrete Canvas
Before installation Concrete Canvas bulk rolls were cut to premeasured lengths. On site 4 workers covered 300m2 of slopes along railway in 2 days.

2.2.3 Connecting Concrete Canvas
Layers of Concrete Canvas were overlapped by 100mm and connected with screws.

2.2.4 Hydration
Following connecting of the joints, Concrete Canvas was hydrated with a water tank and hose (spray nozzle attached).


Figure 5 Slope before fire protection (left), slope after fire protection (right)

2.3 Slope protection – drainage channel lining, Serbia

During 2018-2020 in Serbia, Concrete Canvas was used to provide drainage and erosion control of perimeter canals on few projects. During huge slope stabilization on highway trough Momin Kamen because of difficult conditions and big heights, material was used as replacement of conventional methods like stone cladding or concrete channels.

2.3.1 Preparation of the substrate

Prior Concrete Canvas installation, all unstable/sharp rocks and larger vegetation was removed from the surface. On both edges of slope a small canal (dimensions were 15cm in width and 10cm in depth) was made so we could anchor top part of Concrete Canvas and stop it from twisting while it hardens. If substrate is rock, there was no need of making this step.

2.3.2 Deployment of the Concrete Canvas

Before installation Concrete Canvas bulk rolls were cut to premeasured lengths. Material was than deployed on the installation point by loader machine or crane.

2.3.3 Connecting Concrete Canvas

Layers of Concrete Canvas were overlapped by 100mm and connected with screws.

2.3.4 Hydration

Following connecting of the joints, Concrete Canvas was hydrated with a water tank and hose (spray nozzle attached).

Figure 6 Perimeter canals

3 Conclusion

Concrete Canvas in many ways provides replacement for conventional concrete solutions. It is faster, more reliable solution that saves time, money and usually avoids traffic disruption and big mechanization. All that above makes it a material that should be considered while designing modern road and railway structures, slope protection, weed suppression, canal lining or maybe oil tanks sanitation.

References

[1] Concrete Canvas website: https://www.concretecanvas.com/downloads/?types=product-brochures-acc,technical-certification-acc#toggle-id-499

TECHNICAL DESIGN AND STABILITY ANALYSIS PROCEDURE FOR HORIZONTAL STABILITY CONSTRUCTION OF ROADS AND RAILWAYS

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Abstract

Unstable sections of predominantly vertical roads and railways are usually stabilized by viaducts, while predominantly horizontal unstable sections of the same structures are regularly stabilized by special structures which have a common feature of spaciousness or massiveness, and which proportionally also require peculiarity in all aspects of the construction.
The goal of the new solution is to avoid the highlighted structural peculiarity, that is, to apply a solution that will be more of a constructive element of roads and railways, like a viaduct in an approximate sense. There is such a solution, and that is the low-rise stable structure, which in a naturally appropriate way counteracts horizontal instabilities on low-rise objects. The horizontal effect on the object is converted to a vertical direction via this construction by means of pile coupling, while this effect is greatly reduced due to the effect of static interaction between the components of the coupling. If, instead of various vertical structures with horizontal anchors or mass structure retaining walls, we apply the slope-pile coupling at an optimal angle in the range of 15 to 20 degrees, then, by activating the external horizontal effect, i.e. instability, the primary axial resistance in the oblique pile is simultaneously activated through circumferential friction. The vertical component of this resistance decreases the active horizontal component, while the horizontal does the same, provided that the pile has a transverse static EI feature. This approach has not been used thus far in engineering practice and therefore represents a novelty. Therefore, it can be argued that by constructing a low-rise stable structure, we can achieve at least approximately the same structural impression that we enjoy regarding the viaduct construction for predominantly vertical instabilities.

Keywords: retaining engineering structure, batter pile, vertical piling structure, active soil pressure, suspended weight, pile skin resistance, natural slops instabilities

1 Introduction

Batter piles and a vertical pile walls are coupled with a head beam, as a horizontal load-bearing structures. The batter pile is performed at an angle of 20˚, using “in situ” pile technologies. The batter pile achieves the load-bearing capacity with a part of its length as a kind of catenary and partially as a anchoring. Along the entire length “L_1“, the profiled steel core “A_S“ is installed.

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As a conclusion, the load-bearing capacity of the batter pile is not only conditioned by the value of activating its axial deformation, but by the activation of the weight suspended on it as well. There is no activation of the horizontal load of the sistem without the activation of the suspended weight on the batter pile.
Suspended weight on it, then no longer creates horizontal pressure but is displaced as a vertical pressure of the vertical element of the structure. With such a design and equal dimension quantity, the structure achieves the goal of reducing spatiality and increasing the load-bearing potential.
The batter pile can be considered as an element composed of two lenghts, the span “CA“ and the anchoring part “L_1 – CA“. One end of the span “CA“ is supported by the vertical structure while the other end is anchored by the axial tensile force “A_n“.
On the base of this statements , equilibrium equations for stability analysis can be performed for the system of structural elemets or for the system of the finite elements method , with characteristics “E_I , A_I , I_I “ , for batter pile and “ E_{II} , A_{II} , I_{II} “ for vertical structure...

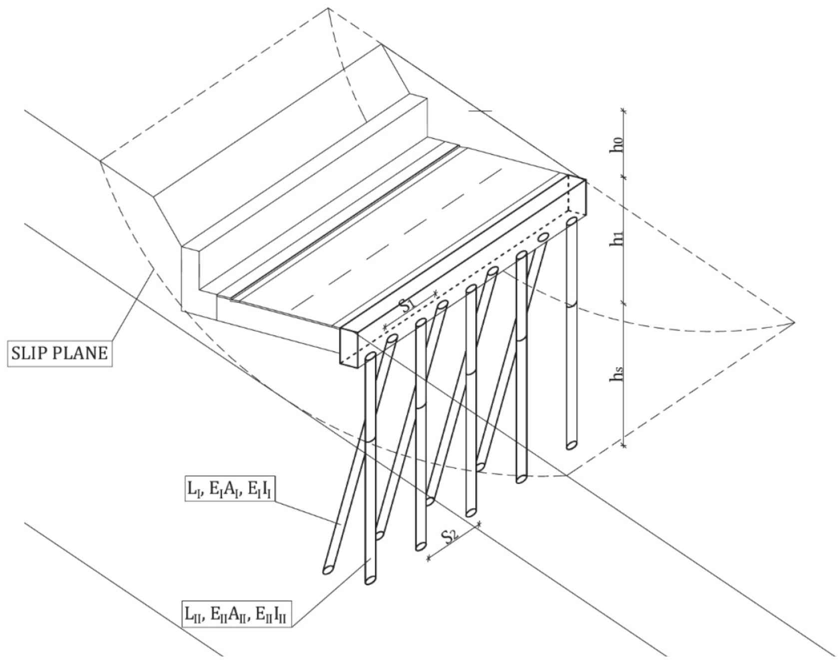

Figure 1 Road on slope - Construction model

Construction is modeled as 3-joint bracing, Figure 2.

a) Vertical load of element I, A_n‘cosβ
b) Horizontal resistance of element I, A_n‘sinβ
c) Bending resistance I, composite bearing capacity A_n‘,M_I‘ of the pile‘s section
d) Axial resistance in point A of element I, A_n‘
e) Horizontal load on the element II, P_a-A_n‘(K_a cosβ+ sinβ)
f) Transversal and axial resistance in point B of element II, B_t‘, B_n‘
g) Bending resistance of element II, composite bearing capacity B_n‘, M_{II}‘ of the pile’s section
h) Axial deformation of element I, ΔI_n
i) Axial deformation of element II, ΔII_n
j) Bending deformation of element II, ΔII_t
k) Horizontal deformation of the sistem, Δ_S

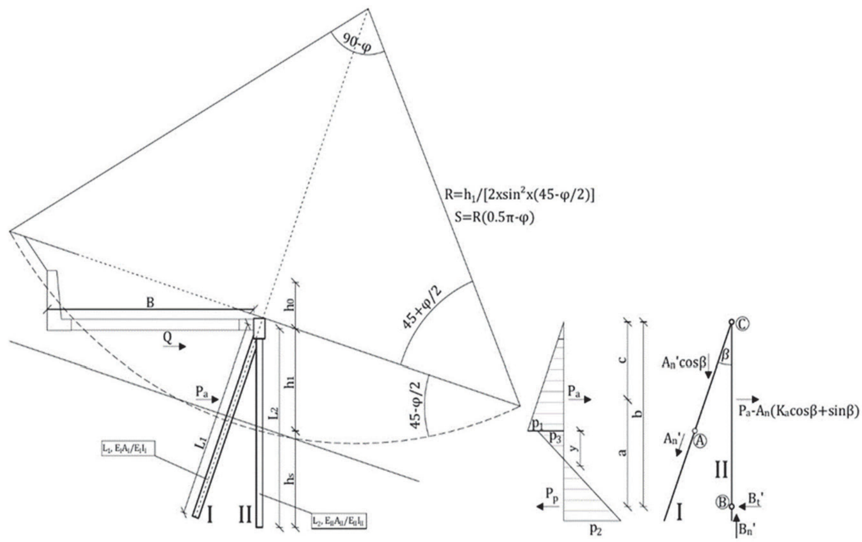

Figure 2 Road on slope - Stability model

2 Construction equilibrium equations

The equilibrium equations for the construction elements can be obtained from the stability model shown in Figure 2.

$$A_n' \times b \times \tan\beta + \frac{h_1 \times \tan\beta \times A_n' \times \cos\beta}{2} - a \times \left[P_a - A_n'\left(K_a \times \cos\beta + \sin\beta\right)\right] = 0 \tag{1}$$

$$B_n' \times h_1 \times \tan\beta - \frac{h_1 \times \tan\beta \times A_n' \times \cos\beta}{2} - \left(b - h_1\right) \times B_t' - \frac{h_1 \times \left[P_a - A_n'\left(K_a \times \cos\beta + \sin\beta\right)\right]}{3} = 0 \tag{2}$$

$$B_t' = \frac{c\left[P_a - A_n'\left(K_a \times \cos\beta + \sin\beta\right)\right]}{b} \tag{3}$$

$$C_n' = \frac{a\left[P_a - A_n'\left(K_a \times \cos\beta + \sin\beta\right)\right]}{b \times \tan\beta} \tag{4}$$

Activated horizontal load

$$p_1 = \gamma_{sat} \times h_1 \times K_a - p_{ca} \tag{5}$$

$$p_3 = p_1 - p_{cp} \tag{6}$$

$$y = \frac{p_3}{z} \tag{7}$$

According to the Figure 2:

$$b = \frac{2h_s + y}{3} + h_1 \tag{8}$$

$$c = \frac{2h_1}{3} \tag{9}$$

$$a = b - c \tag{10}$$

$$z = \gamma \times \left(K_p - K_a\right) \tag{11}$$

$$p_2 = z \times \left(h_S - y\right) \tag{12}$$

$$P_a = å(p_a \times h_{p_a}) = \frac{p_1 \times h_1 + p_3 \times y}{2} \tag{13}$$

$$P_p = å\left(p_p \times h_{p_p}\right) = \frac{p_2 \times \left(h_S - y\right)}{2} \tag{14}$$

3 Deformation equilibrium state

Kinematic equations can be obtained from the stability model shown in Figure 2.

$$\Delta I_n = A_n \times \left[\frac{s_1}{4h_{s1} d \pi G_s} + \frac{L_1 - h_{s1}}{A_I E_I}\right] \tag{15}$$

$$\Delta II_n = B_n \times \left[\frac{s_2}{4h_{s2} d \pi G_s} + \frac{L_2 - h_{s2}}{A_{II} E_{II}}\right] \tag{16}$$

$$\Delta s = \frac{\Delta I_n + \Delta II_n}{\tan \beta} \tag{17}$$

3.1 Composite bearing capacity of the pile’s section of element I

$$\Delta I_t \begin{cases} A_n \\ \Phi(u) M_{I\max} \end{cases} \tag{18}$$

$$\Phi(u) M_{I\max} = \Phi(u) A_n \frac{h_1}{12} \sin \beta \tag{19}$$

$$u^2 = \frac{A_n h_1^2}{4 E_I I_I} \tag{20}$$

3.2 Composite bearing capacity of the pile’s section of element II

$$\Delta II_t \begin{cases} B_n = n A_c f_c \\ M_{II\max} = m A_c h f_c \end{cases} \tag{21}$$

4 Stability evidence

4.1 Stability evidence of construction elements

$$A_n = A_n^{'} \times ds_1 < A_{n,q,R} \tag{22}$$

$$A_t = A_n^{'} \times \sin\beta \times s_1 < A_{t,q,R} \tag{23}$$

$$B_n = B_n^{'} \times s_2 < B_{n,q,R} \tag{24}$$

$$B_t = B_t^{'} \times s_2 < B_{t,q,R} \tag{25}$$

$$\Phi(u) M_{I\max} = M_{I,R} \tag{26}$$

$$M_{II\max} < M_{II,R} \tag{27}$$

$$\Delta s < \Delta s_{adm} \tag{28}$$

4.2 Stability evidence of road on slope

Stability of the system is ensured by setting the adding reactive shear forces $\Delta R_{,req}$ in the slip plane, after which the system is solved as Rankine's semi-infinite equilibrium condition. Calculation of the adding reactive section force R_{req}:

$$\Delta R_{req} < 0.3 \times R_{soil} = 0.3\left(c_d + \gamma_{sat} \times h_1 \times \tan\varphi_d\right) S \tag{29}$$

$$\Delta R_{req} < \left(A_{t,q,R} + B_{t,q,R}\right) \tag{30}$$

Main stress at slide plane,

$$\sigma_b = \gamma \times h_1 \tag{31}$$

Diference stress of ultimate resistence stresses and loads stresses,

$$\Delta R_R = \frac{\left[\left(\sigma_b \tan\varphi + c\right) RS + \left(A_{t,q,R} + B_{t,q,R}\right) R - H\left(R - h_1\right)\right]}{0.5B^2 + B\left(R - h_1\right)\tan\alpha} - p_0 \tag{32}$$

If, Δq_R, positive, then we have sliding – seizmic resistance potential,

$$H = P_a + Q \tag{33}$$

5 Construction elements characteristics

5.1 Element I

Batter pile lenght L_1, where:
d - pile diameter
s_1 - axial distance between piles
h_{s1} - anchor part of L_1
A_l - profiled steel core of a pile
$\gamma_R,\gamma_S,\gamma_C$ - partial safety factors
σ_q - soil pressure
σ_s - steel core strenght
K_p - coef. passive earth pressure
φ_d - angle of internal friction of the soil
$A_{n,q,R}$ - axial tensile bearing capacity of element I, with regard to the soil
$A_{t,q,R}$ - lateral bearing capacity of element I, with regard to the soil

$$N_{s,R} = \frac{\sigma_s A_l}{\gamma_s} \tag{34}$$

$$A_{n,q,R} = \frac{\sigma_q d \pi h_{s1} \tan \varphi_d}{\gamma_R} \tag{35}$$

$$A_{t,q,R} = \frac{\gamma \left(h_1 + h_{s1}\right) K_p d h_{s1}}{2\gamma_R} \tag{36}$$

5.2 Element II

Vertical pile wall length L_2, where:
d - pile diameter
s_2 - axial distance between piles
h_{s2} - anchor part of L_{12}
P_p - passive soil resistance
$B_{n,q,R}$ - vertical bearing capacity of element II in point B
$B_{t,q,R}$ - horizontal bearing capacity of element II in point B
N_q, N_c - bearing capacity of the soil

$$B_{n,q,R} = \frac{h\gamma N_q A_b + c_d N_c A_b}{\gamma_R} \tag{37}$$

$$B_{t,q,R} = \left(P_p + B_n^{'} \tan \varphi_d\right) s_2 \tag{38}$$

5.3 Element III

Connection element for connection and anchoring of the batter piles with vertical pile structure as a reinforced concrete head beam.

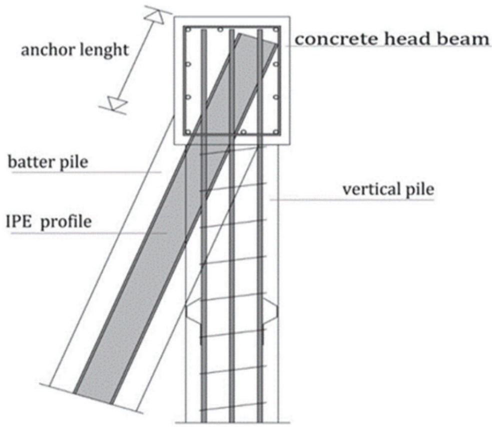

Figure 3 Connection element detail

6 Conclusion

According to the goals set and with the use of known static and structural settings, naturally acceptable and at the same time more rational and based on its function more stable construction is gained. This construction and its resistance elements give the optimal response to the state of strain caused by the construction of the road or railways on the slopes, without introducing new elements of instability, which are presently caused by the weight of massive elements or the anchoring position of anchor systems. In addition, interactions among the elements of our construction are optimized, so the rationality is substantially increased for the same safety factor. This will enable a single standard solution for construction on the slopes, bringing us closer to the goal set for the construction. Concerning stability analysis, carried out on the basis of equilibrium equations, based on the mobilized resistance of structural elements or less on the basis of finite structural elements equilibrium, in one and the other case, as can be seen in the stability analysis procedure, the influence of the wrong estimation of the calculation parameters is minimized. Therefore, this approaches leads to consolidation of constructions at horizontal instabilities, as well as slope sliding and earthquake

References

[1] Berezantzev, V.G., Khristoforov, V., Golubkov, V.: Load Bearing Capacity and Deformations of Piled Foundations, Proc. 5th Int. Conf. S.M. & F.E., 1961

[2] Bowles, J.E.: Foundations Analysis and Design, Fifth Edition, 1996.

[3] Broms, B.B.: Lateral Resistance of Piles in Cohensionless Soils, J.S.M.F.D., ASCE, 90 (1964), SM3:123-156

[4] Poulos, H.G., Davis, E.H.: Pile Foundation Analysis and Design, 1980.

[5] Simović, V.: Građevna Statika I, Građevinski institut, Zagreb, 1988.

[6] Šepac, Z.: Patent cooperation treaty, PCT, Int. Aplication No. PCT/HR2018/0000

[7] Terzaghi, K., Peck, R.B.: Soil Mechanic in Engineering Practice, 1967.

[8] Timoshenko, P.S.: Strength of Materials, Third Edition, 1958.

[9] Tomičić, I.: Priručnik za proračun armiranobetonskih konstrukcija, 1983.

[10] Whitaker, T.: The Design of Piled Foundations, Second edition, 1976.

A NOVEL ALGORITHM FOR VERTICAL SOIL LAYERING BY UTILIZING THE CPT DATA

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Abstract

Determination of the relevant soil stratigraphy is of the paramount importance for any geotechnical analysis. The cone penetration test (CPT) is the cost-effective, rapid, continuous, and reliable testing method for assessing soil layering and estimating in-situ mechanical properties of soil, and as such is especially useful for subsoil investigations along linear infrastructure networks, such as roads, highways, or railways. The design soil profile can be effectively determined using the CPT-based soil behaviour type (SBT) classification system. However, the soil profile consists of layers of various thickness and some layers can be only a few centimetres thick. Because the cone needs to penetrate to a certain depth to develop the cone resistance and to identify the presence of another layer, thin layers of soil cannot be properly detected. The soil layering algorithm, presented in this paper, merges these thin layers into the adjacent layers and thus overcomes the unreliable determination of the thin layers. The implementation of the proposed algorithm is demonstrated using a CPT carried out on the embankment test-site in north Croatia.

Keywords: CPT, soil layering, vertical variability, SBT soil classification

1 Introduction

Obtaining reliable information on the soil stratigraphy and corresponding soil parameters is critical for any type of geotechnical analysis. Traditionally, the geotechnical investigation works overrely on the soil drilling, sampling, and laboratory testing. As an alternative, cone penetration test (CPT), Fig 1, provides continuous and reliable information along the investigation depth. Considering its repeatability and reliability, especially the major advances in speed of use, the method can overcome the commonly encountered delay issues of drilling and laboratory testing on construction projects or emergency interventions (such as unstable slopes). This especially comes to fore when investigating the soil below linear infrastructure such as road, highway, railway networks, where the necessity for the optimization of investigation works is often highlighted. As such, CPT is increasingly incorporated into the portfolio of investigation methods of geotechnical engineers.

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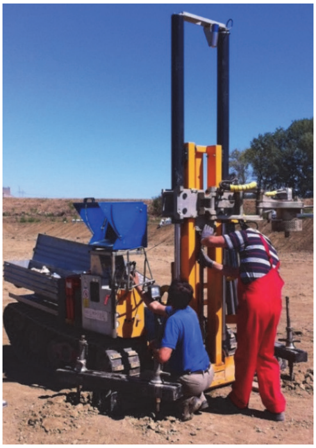

Figure 1 Cone penetration test (CPT) investigation

The method relies on pushing a specially designed cone (probe) into the soil, at relatively fast rate (20 mm/s), enabling continuous record of the cone tip resistance (q_c) and sleeve resistance (f_s), with the aim to delineate soil stratigraphy and to provide estimates of its in-situ physical and mechanical properties. The flexibility and applicability of testing can be aided by equipping the probe with the additional sensors. The method can also provide additional information on the groundwater pore pressure (CPTU), in-situ shear wave velocities (SCPT), liquefaction risk estimates, etc. Hence, as [1] notes, CPT can provide up to seven independent measurements in one cost-effective test, where a soil classification system should include all these measurements to be fully effective.
The well-known heterogeneity of most soils, along with many difficulties in obtaining not only undisturbed samples but also samples that can be considered as representative of soil mass [1], requires proper procedures of dealing with the inherent variability in both horizontal and vertical direction. The design soil profile, determined by the CPT procedures, usually consists of several layers of various thickness, where some identified layers can be only a few centimetres thick. However, since the CPT probe needs to penetrate to a certain depth to develop the cone resistance and to sense the presence of another layer, thin layers of soil cannot be properly detected. This paper offers the procedure to overcome the unreliable determination of these thin layers, within the framework of the CPT-based soil behaviour type (SBT) classification system.

2 The soil behaviour type (SBT) classification system

CPT-based classification of complex natural soil behaviour is convenient since the CPT can offer multiple, repeatable, independent in-situ measurements. As an alternative to the traditional soil classification schemes such as Unified Soil Classification System (USCS), which relies on laboratory classification tests, Robertson [2, 3] proposed a soil classification based on the CPT recorded data. This classification utilizes CPT-based charts, Fig 2a, to predict the Soil Behaviour Type (SBT), whereas author stress that the cone responds to the in-situ mechanical behaviour of the soil and not directly to soil classification criteria based on grain-size distribution and soil plasticity. However, as Molle [4] noted, there is reasonably good correlation between the soil in-situ mechanical behaviour and soil classification criteria, even though there are several exceptions [5] Originally, Robertson [2] proposed 12 SBT zones, and this was later reduced to 9 SBT zones [3], as shown in Table 1. To avoid further elaboration on the reasons for the reduction of the zone number, as well the differences between these SBT classifications, the reader is referred to the appropriate literature [5].

Table 1 Proposed unification between 12 SBT zones [2] and 9 SBT zones [3]

SBT zones [2]	SBT zones [3]	Proposed SBT description
1	1	Sensitive fine-grained
2	2	Clay – organic soil
3	3	Clays: clay to silty clay
4 and 5	4	Silt mixtures: clayey silt & silty clay
6 and 7	5	Sand mixtures: silty sand to sandy silt
8	6	Sands: clean sands to silty sands
9 and 10	7	Dense sand to gravelly sand
12	8	Stiff sand to clayey sand (OC or cemented)
11	9	Stiff fine-grained (OC or cemented)

However, it was noted that the CPT-based SBT classification systems can cause some confusion in geotechnical practice, since they use textural-based descriptions, such as sand and clay. Therefore, a CPT-based SBT classification system with behaviour-based descriptions for each soil group was proposed, where Robertson [6] provided latest update to use behaviour-based descriptions, which includes a method to identify the existence of microstructure in soils and its influence on this CPT-based classification. Within this work, Robertson differentiates seven (7) soil behaviour types, shown on Fig 2b. These include: (1) CCS: Clay-like – Contractive – Sensitive; (2) CC: Clay-like – Contractive; (3) CD: Clay-like – Dilative; (4) TC: Transitional – Contractive; (5) TD: Transitional – Dilative; (4) SC: Sand-like – Contractive; (5) SD: Sand-like – Dilative. CPT-based charts from [2, 3, 6] use the Q_t, F_r and B_q as normalized parameters to determine the SBT, and these can be expressed with:

$$Q_t = \frac{q_t - \sigma_{v0}}{\sigma'_{v0}} \tag{1}$$

$$F_r = \left(\frac{f_s}{q_t - \sigma_{v0}} \right) \times 100\% \tag{2}$$

$$B_q = \left(\frac{u_2 - u_0}{q_t - \sigma_{v0}} \right) = \left(\frac{\Delta u}{q_t - \sigma_{v0}} \right) \tag{3}$$

where q_t is the cone resistance corrected for water effects defined as $q_t = q_c + u_2(1\text{-}a)$, with 'a' being the cone area radio, u_2 being penetration pore pressure and u_0 being current in-situ pore pressure; is the current in-situ total vertical stress; and is the current in-situ effective vertical stress. Furthermore, the contours (thick lines) on the Q_t - F_r chart represent the boundaries of respective SBT zones, which are, for original Robertson chart (Fig 2a), given by the equation:

$$I_c = \left[\left(3.47 - \log Q_t\right)^2 + \left(\log F_r + 1.22\right)^2 \right]^{0.5} \tag{4}$$

where I_c is identified as the soil behaviour type index. The boundaries of updated version of Robertson chart (Fig 2b), which considers the behaviour based descriptions of materials, are given by the equations:

$$I_B = \frac{100\left(Q_{tn} + 10\right)}{70 + Q_{tn} F_r} \tag{5}$$

$$CD = (Q_{tn} - 11) \times (1 + 0.06F_r)^{17} \tag{6}$$

where I_B stands for modified soil behaviour type index and CD stands for contractive – dilative boundary.

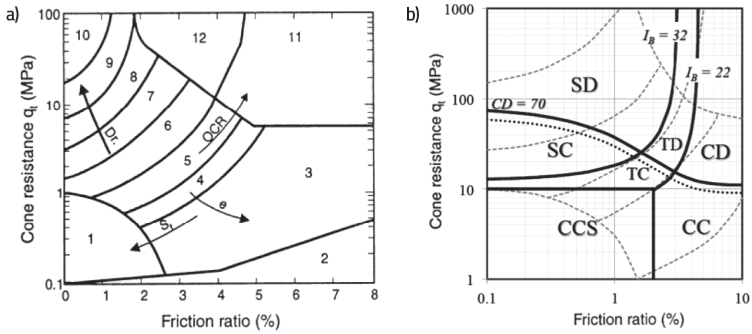

Figure 2 CPT-based charts: (a) from original Robertson [2] and (b) an updated version [6]

3 A novel algorithm for vertical soil layering

Several studies used the data from an individual CPT to generate a stratigraphic profile based on Soil Behaviour Type (SBT) chart. Ganju et al. [7] developed an algorithm to handle the occurrence of thin soil layers within a stratigraphic profile. The CPT identification of these layers in not reliable since the standard CPT cone cannot accurately sense layers with thickness below a certain limit. This can confuse a geotechnical practitioner who relies on the CPT-based soil stratigraphy for the various geotechnical analysis. Ganju et al. [7] used three different approaches to absorb thin CPT layers into thick adjacent layers, and these include:

1. SBT chart band approach: consolidation of thin layers into adjacent layers considering secondary soil type(s) classification;
2. Soil group approach: consolidation of thin layers into adjacent layers of the same soil group;
3. Average qc approach: consolidation of thin layers into adjacent layers with similar average q_c.

This paper will not elaborate each approach in detail, since their comprehensive description is given in the literature [7]. The algorithm presented in this paper is a novel protocol of merging thin layers with the adjacent thick layers, and as such being most similar to the above-mentioned 'soil group approach'. The algorithm is reliable for the everyday geotechnical practice which utilizes CPT for defining soil stratigraphy. The algorithm consists of two main phases. The first phase deals with the generation of initial soil profile, while the second phase merges thin layers into adjacent thick layers, by grouping soil behaviour types into soil groups of similar behaviour.

The initial soil profile generation phase consists of three steps. First step provides a CPT input on depth of each data (d_i), measured values of cone tip resistance (q_{ci}), and sleeve resistance (f_{si}), penetration (u_{2i}) and in-situ pore water (u_{0i}) pressures, as well an information on pre-drilling depth and the corresponding unit weight of the soil above the drilling depth. All these provide an input for step two, which calculates Q_{tni}, F_{ri}, CD_i i IB_i using the equations (1), (2), (5) and (6). To calculate the unit weight of the soil, a correlation proposed by Kovačević et al. [8] is used. Finally, in a third step of initial soil profile generation, SBT_{ni} value is calculated, thus enabling the classification of soil in one of the soil-behaviour types.

The second phase of grouping layers consists of two steps. First step requires the input on the distance (d_z) between two adjacent CPT results, which was determined prior the CPT execution. By providing a required input on the minimum soil thickness ($Thick_{min}$) a calculation of the number of adjacent CPT results (n_t), corresponding to the minimum layer thickness, follows. Finally, a code for merging thin layers into the adjacent thick layer is given, based on the comparison of behaviour types of thin layer of calculated thickness with the behaviour types of thick layers. A full algorithm, which can be implemented for any SBT chart available in literature, is given in Table 2.

Table 2 A novel algorithm for the vertical layering from CPT data

Phase	Step	Code
1 - initial soil profile generation	1.1 input data	d_i, qc_i, fs_i, $u2_i$ i $u0_i$ (i=1, 2, . . ., n), predrill_depth, gamma_predrill
	1.2 Calculating the Qtn_i, Fr_i, CD_i IB_i from: d_i, qc_i, fs_i, $u2_i$ and $u0_i$ (i=1, 2, . . ., n)	qt[i] = qc[i] + 0.001 * u2[i] * (1-0.8) Rf[i]=100*fs[i]/(qt[i]*1000) gama[i]=11.849+0.109*log(z[i])+2.595*log(fs[i])+0.561*log(qt[i]) sigv0[i]= sigv0[i-1] + gama[i]*(z[i]- z[i-1]) + predrill_depth * gamma_predrill sigv0c[i]=sigv0[i]-u0[i] Fr[i]=(fs[i]/(qt[i]*1000-sigv0[i]))*100 Qtn[i]=0.01*(qt[i]*1000-sigv0[i])*(100/sigv0c[i]) ^n[i] Ic[i]=sqrt((3.47-log(Qtn[i])) ^2 + (log(Fr[i])+1.22) ^2) n[i]=0.381*Ic[i]+0.05*(sigv0c[i]/100)-0.15 IB[i]=100*(Qtn[i]+10)/(Qtn[i]*Fr[i]+70) CD[i]=(Qtn[i]-11)*(1+0.06*Fr[i]) ^17
	1.3 Determining the $SBTn_i$ from: Qtn_i, Fr_i, CD_i and IB_i (i=1, 2, . . ., n)	if Qtn[i] <= 10. and Fr[i] <= 2.: SBTn[i] = 1 if Fr[i] > 2. and IB[i] <= 22. and CD[i] <= 70.: SBTn[i] = 2 if IB[i] <= 22. and CD[i] > 70.: SBTn[i] = 3 if Qtn[i] > 10. and IB[i] > 22. and IB[i] <= 32. and CD[i] <= 70.: SBTn[i] = 4 if IB[i] > 22. and IB[i] <= 32. and CD[i] > 70.: SBTn[i] = 5 if IB[i] > 32. and CD[i] <= 70.: SBTn[i] = 6 if IB[i] > 32. and CD[i] > 70.: SBTn[i] = 7 SBTn = 1: CCS - Clay-like - Contractive - Sensitive SBTn = 2: CC - Clay-like - Contractive SBTn = 3: CD - Clay-like - Dilative SBTn = 4: TC - Transitional - Contractive SBTn = 5: TD - Transitional - Dilative SBTn = 6: SC - Sand-like - Contractive SBTn = 7: SD - Sand-like – Dilative
2 - grouping of thin layers with the thick layers	2.1 Defining minimum soil thickness $Thick_{min}$	dz – the distance between two adjacent CPT results nt – the number of adjacent CPT results corresponding to the minimum layer thickness $Thick_{min}$ nt = int($Thick_{min}$/dz)+1
	2.2 Merging the thin layers within the adjecent thick layers	For j=1, 2, . . . , nt: For i=1, 2, . . . , n: Determining the number of adjacent CPT results nl_i, which have the same $SBTn_i$. Determining the average values of Frav and Qtnav for the nl_i adjacent CPT results. For k=j, j+1, . . . , n–j: If nl_k = j: Calculating the distance between the average (Frav,Qtnav) result and the last previous result n1 for which it is nl_k > j: ds1= sqrt((log(Fr[k])-log(Fr[n1]))^2 + (log(Qtn[k])-log(Qtn[n1])) ^2). Calculating the distance between the average (Frav,Qtnav) result and the first next result n2 for which it is nl_k > j: ds2= sqrt((log(Fr[k])-log(Fr[n2]))^2 + (log(Qtn[k])-log(Qtn[n2])) ^2). If ds1 <= ds2: The current result merges with the previous adjacent results for which it is nl_k > j: SBTn[k]= SBTn[n1] If ds1 > ds2: The current result merges with the next adjacent results for which it is nl_k > j: SBTn[k]= SBTn[n1] For k=1, 2, . . ., j: The first j results merge with the j+1 result: SBTn[k] = SBTn[j+1] For k= n-j+1, . . ., n: The last j results merge with the n-j result: SBTn[k] = SBTn[n-j]

4 An example of a CPT sounding from north Croatia

To demonstrate the effectiveness of the proposed algorithm in determination of soil stratigraphy, a single CPT record is selected from the test-site location of an embankment in north Croatia. The analysed CPT has the investigation depth up to 15 m, with the predrilling depth of 0.5 m. The continuous raw acquisition data, Fig 3, contains the probe tip resistance (q_c) and probe skin friction (f_s).
The input data was used to calculate the normalized values of cone resistance (Q_{tn}) and friction ratio (F_r). The recordings for each depth are plotted on the CPT-chart, proposed by Robertson [6], in order to get an insight into dispersion of the Q_{tn} - F_r plots in regards to the one of the seven soil behavior types, see Fig 4.

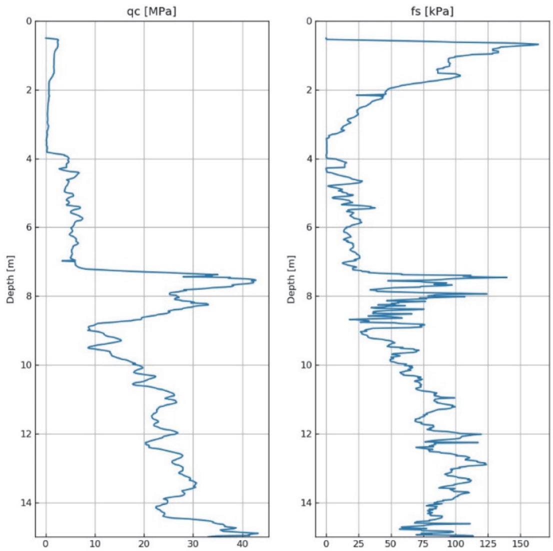

Figure 3 Analysed CPT raw recorded data: q_c and f_s

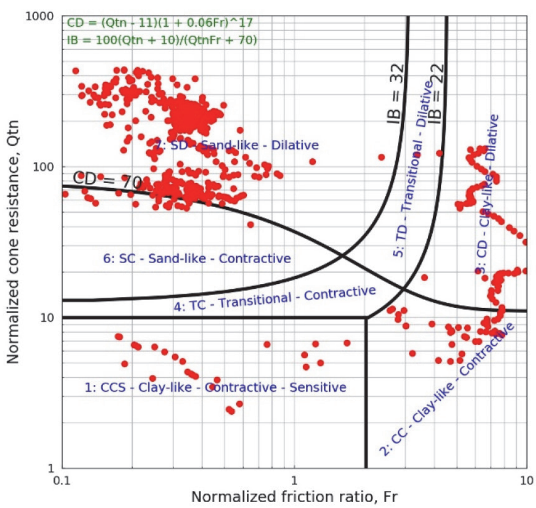

Figure 4 CPT results plotted on the Robertson [6] Q_{tn} - F_r chart

The determined soil stratigraphy, using the various input of minimum layer thickness, is shown on Fig 5. The selected minimum thickness are 15 cm, as suggested by [7] to be a minimum which can be properly detected by the standard CPT cone to avoid ambiguity in the assignment of soil type to such layers, 50 cm and 100 cm. The increase of the minimum thickness of the layer merges thin layers with the adjacent thick layers, leading to the reduction of the number of layers as well the number of SBT group types in the overall stratigraphic soil profile.

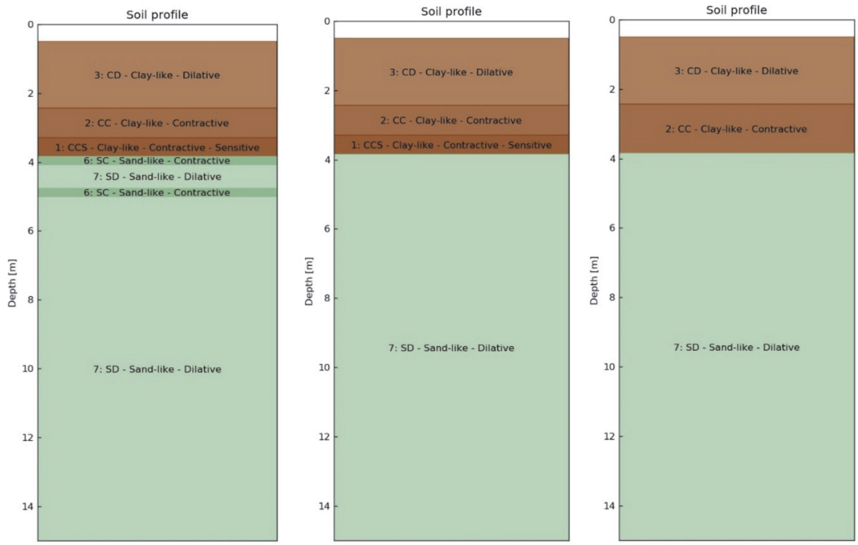

Figure 5 Obtained stratigraphy profile for minimum thickness of: 15 cm (left), 50 cm (middle) and 100 cm (right)

5 Conclusions

This paper presents the soil layering algorithm for the development of a design soil profile, determined using the CPT-based soil behaviour type (SBT) classification system. The algorithm merges thin soil layers into the adjacent layers and thus overcomes the unreliable determination of the thin layers. It consists of two main phases, where the first phase deals with the generation of initial soil profile, while the second phase merges thin layers into adjacent thick layers, by grouping soil behaviour types into soil groups of similar behaviour. Having in mind that the CPT is increasingly incorporated into the portfolio of investigation methods of geotechnical engineers, especially those dealing with construction and remediation of linear infrastructure such as road, highway or railway networks, the algorithm is sufficiently reliable for the everyday practice.

References

[1] Baloukas, I., Alamanis, N., Papageorgiou, G., Karipidis, C., Lokkas, P., Xafoulis, N., Chouliaras, I.: Implementing a Static Cone Penetration Test in Infrastructure Works, ASRJETS, 72 (2020) 1, pp. 214-229

[2] Robertson, P.K., Campanella, R.G., Gillespie, D., Greig, J.: Use of Piezometer Cone data, Use of In-situ testing in Geotechnical Engineering, pp. 1263-1280, ASCE, Reston, USA, 1986.

[3] Robertson, P.K.: Soil classification using the cone penetration test, Canadian Geotechni-cal Journal, 27 (1990) 1, pp. 151-158

[4] Molle, J.: The accuracy of the interpretation of CPT-based soil classification methods in soft soils, M.Sc. thesis, TU Delft, 2005.

[5] Robertson, P.K: Soil behaviour type from the CPT: an update, 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, USA, 2010.

[6] Robertson, P.K.: Cone penetration test (CPT)-based soil behaviour type (SBT) classification system — an update, Canadian Geotechnical Journal, 53 (2016) 12, pp. 1910-1927, doi: https://doi.org/10.1139/cgj-2016-0044

[7] Ganju, E., Prezzi, M., Salgado, R.: Algorithm for generation of stratigraphic profiles using cone penetration test data, Computers and Geotechnics, 90 (2017), pp. 73–84, doi: 10.1016/j.compgeo.2017.04.010

[8] Kovačević, M.S., Gavin, K.G. Reale, C.; Librić, L.: The use of neural networks to develop CPT correlations for soils in northern Croatia, 4th International Symposium on Cone Penetration Testing (CPT'18), pp. 377-382, 21-22 June 2018, Delft, The Netherlands

METHODOLOGY FOR TUNNEL RISK ASSESSMENT USING FAULT AND EVENT TREE ANALYSIS

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Abstract

The intense demand and construction of tunnels is accompanied by uncertainties. The reason for appearance of uncertainties are the complex solutions and conditions for these structures. Location and dimensions are becoming more challenging, and the construction is predicted in complexed geological conditions, leading to application of new approaches, methodologies and technologies by the engineers. Most of the uncertainties and unwanted events in tunnelling occur in the construction phase, which generally leads to economic consequences and time losses. For easier handling of the uncertainties, they should be anticipated and studied within a separate part of each project. One of the newer approaches to dealing with uncertainties is hazard and risk assessment and defining ways to deal with them i.e. management. Hazards and risks can be analysed qualitatively and quantitatively. The quantitative analysis, examines the causes and consequences in more detail way and gives explanation of the dependencies. With the quantitative approach, a more valuable information for decision-making can be provided. There are various models and methods used for the quantification of hazards and risks. This paper presents a methodology in which the fault tree analysis and event tree analysis are used in combination to obtain quantitative results. The fault tree analysis is used for assessment of various hazards and the different ways and reasons that cause them. The event tree analysis is a method for assessing the possible scenarios, which follow after a certain hazard i.e. the consequences that may occur in the project. These trees represent graphic models combined with a mathematical (probabilistic) model, which give the probability of occurrence of the risks.

Keywords: tunnels, risk assessment, fault tree analysis, event tree analysis

1 Introduction

The transport infrastructure has a great importance for the society. Project optimization and infrastructure construction can bring great benefits to any state and the wider region. In order to be successful, projects must meet certain technical, economic, safety and time requirements.

Tunnels as underground structures are an integral part of the transport infrastructure. They allow overcoming of complex obstacles and the fulfilment of technical parameters for the construction of modern roads and railways, where high speeds are developed. Tunnels minimize the impact of infrastructure on the environment, and in cities, their placement improves the quality of life. As urbanization continues and demands for quality and safety of life increase, the importance of underground structures is expected to increase even more.

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The nature of these underground structures indicates uncertainties. Because of the uncertainties, there is no project without a certain level of risk. The risk in its most basic form is defined as a potential for unwanted consequence due to an event or occurrence i.e. hazard. Risk assessment is an essential part for making the right decisions. Very often solutions that look cheaper and faster based on deterministic estimates are associated with greater uncertainties and risks. Tunnels can pose significant risks associated with cost overruns, delays in construction and environmental impacts. Also, as demonstrated in several historical records, tunnels have great potential for accidents involving people in the construction process.

2 Hazards and risks in tunnelling

Risk is associated with certain events that cause consequences, which are often negative. There are a great number of definitions for risk in the literature, but the main terms use in this area are usually the same (hazard, consequences and vulnerability).
In recent times, tunnelling hazards are mainly followed by economic and time consequences, and rarely with human consequences. In terms of environmental impacts, underground structures have many positive characteristics and in many cases are the best solutions to problems in this area.
The data collected from different sources, indicates various types of hazard and risks occurring in tunnel construction around the world. Tunnel collapse is one of the most often recorded event because of the major consequences it has on the construction process, workers safety and the environment. The collapse can be manifested in different ways such as: crown (roof) fall, daylight collapse, instability of the tunnel face, instability to the walls, etc. Most of the time this is caused by several factors, some of which are other types of hazards: excessive deformation, flooding (large inflow of water), rockfall, rockburst and other. According to the location, most of hazards and risk happen near the tunnel (excavation) face.

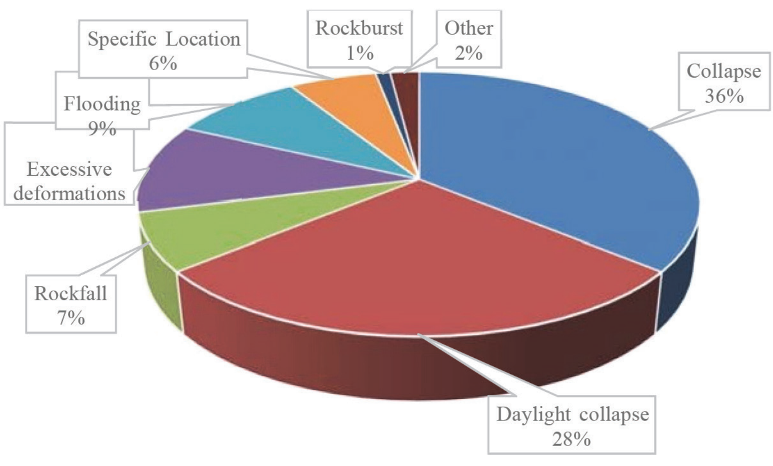

Figure 1 Distribution of hazards during construction of 132 tunnels around the world [1]

The exploitation phase of tunnels includes other types of hazards such as: fires, vehicle collisions, explosions, leaks of aggressive or toxic materials, natural disasters (earthquakes and floods) and specific events (characteristic for submerged tunnels). The largest number of hazards and risks occur in traffic tunnels, because usually people cause unwanted events during tunnel exploitation.

3 Methodologies for tunnel hazards and risks assessment

There are generally two approaches to assessing hazards and risks: qualitative and quantitative approach.

3.1 Qualitative analysis

In the initial project stages for the identification of potential hazards that pose a threat to construction activities, a qualitative risk analysis could be performed. The main goal of this analysis is to raise the awareness of all participants about the risks involved in the construction process and to provide a structural basis for decision making in the early stages of the project. This analysis should contain the following:

- Identification of hazards;
- Classification of hazards;
- Identification of adequate protection or preventive measures;
- Details of the risks in a so-called risk register.

3.2 Quantitative analysis

For more detailed analysis of hazards and risks, a numerical or quantitative approach is applied, which can be deterministic or probabilistic. The quantitative approach needs a detailed analysis of the causes and consequences and an explanation of the dependencies between the considered events and phenomena. This analysis provides valuable information for decision making in the case of uncertainty and unforeseen events, such as the selection of an appropriate project or construction technology, possible protection measures, impacts on third parties and the environment. It also allows the determination of prices and construction time.

The approach to quantifying uncertainties, hazards, and risks is often a combination of mathematical and graphical models or methods. In the literature, they can also be found as graphic networks or as risk management tools. Some of these methods are the Fault and Event tree analysis, Markov process, Bayesian networks, Failure Mode and Effects Analysis (FMEA), Hazard Operability Study (HAZOP), Hazard Analysis and Critical Control Points (HACCP), etc.

4 Tunnel risk assessment using fault and event tree analysis

The combination of fault and event tree with their probabilistic models is a methodology, which as a final product gives the risk in quantitative form. This approach can be used to assess hazards and risks in different types of tunnels under construction and exploitation.

The principle of this methodology begins with the formation of the fault tree, where an expected hazard in the tunnel, which is defined based on the available data, is presented as a top event. The branching of the tree is done in a logical order where the primary events are grouped into several main groups that represent them. Probabilities are given for each primary event describing the uncertainty of the occurrence or the impact on the top event. This gives the likelihood of a hazard (top event) occurring during the tunnel construction period.

The top event from the fault tree with its probability is then presented as the initial event in the event tree. In the event tree, the nodes indicate the adopted and proposed measures that affect the occurrence of the hazard, i.e. their success or failure determines the consequences.

In the fault tree probabilistic analysis, standard deviations are assigned to all primary events separately. With the use of logarithmic distribution and advanced Monte Carlo simulation with 1000 samples, the results are obtained in the form of cumulative probability distribution. The same concept of assigning standard deviations to preventive measures and the initial event is implemented in the event tree. The results of this analysis give the most critical direction (path) in the event tree, which is actually a sequence of failure of all preventive measures.

4.1 Railway tunnels

This process of combining the two trees has been used for risk assessment of several tunnels designed on the future railway line, which is part of the Pan- European transport corridor VIII in North Macedonia. Specifically, the section Kriva Palanka - border with R. Bulgaria, with length of 23,40 km is the most complexed part of the railway line from Skopje to the border with Bulgaria. Along the route, 24
tunnels with a total length of about 9,00 km have been designed. In this paper the results from the two longest tunnels (1,4 and 1,3 km) along the section are shown. The tunnels are placed in a horizontal curve where the railway slope is near the maximum (23,50 and 19,00 ‰). The excavation of the tunnels is predicted to be mostly with the Drill & Blast method.
For the risk analysis, different information have been used such as:

- number and length of investigative boreholes;
- classification, types and quality of ground materials;
- number and types of fault zones;
- level and quantity of groundwater;
- tunnel support.

Four hazard have been analyzed for these tunnels: unpredicted inflow of ground water, excessive deformation (swelling and squeezing) and instability of the excavation face (collapse). The results from the fault tree show that the unpredicted inflow of ground water (figure 2) is the hazard with the most probability of occurrence, QH = 0,2195 (21,95 %), but the biggest risk in the event tree comes from the instability of the excavation face (figure 3) QR = 0,0002530 (0,02530%).

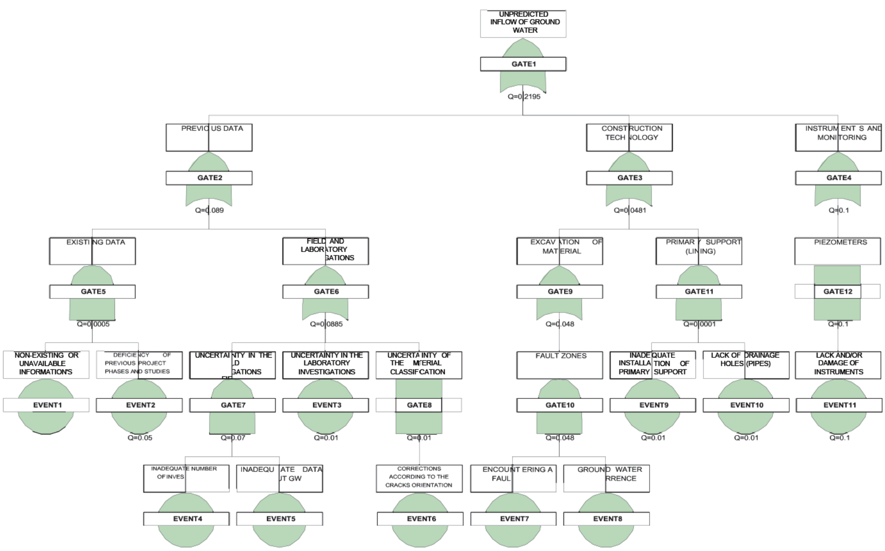

Figure 2 Figure 2. Fault tree analysis for unpredicted inflow of ground water

Table 1 Probabilistic analysis results in the fault tree for inflow of ground water

Parameter (probability)	Primary value	Mean value	Standard deviation	5%	50%	95%
QH	0,2195	0,2189	0,0671	0,1287	0,2090	0,3445

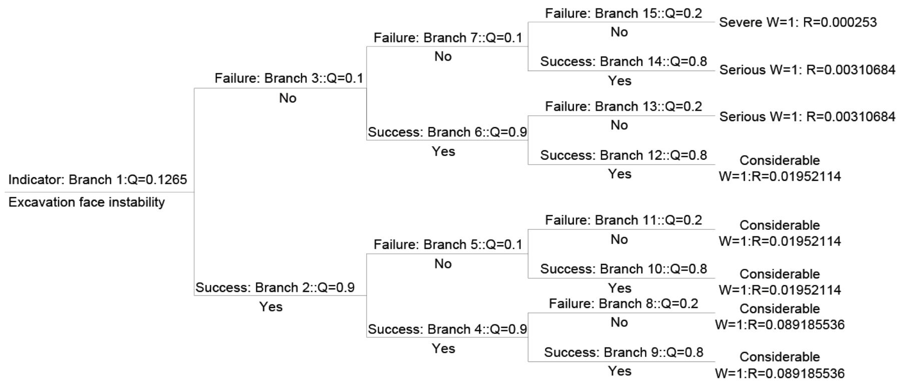

Figure 3 Event tree analysis for instability of the excavation face

5 Conclusion

In the current practice, the risks in the projects were mainly analysed on a qualitative basis. For greater effectiveness, certain changes are needed that focus on the application of quantitative methods.

The methodology shown in this paper a combination of fault tree, event tree and probabilistic model resulting in cumulative function distribution. The fault tree serves to define and asses the hazards and the event tree analyses the critical scenarios aSSnd the consequences that can occur from a defined hazard.

The results in this paper show that the hazards with highest probability of occurrence not always represent the highest risk. Further, the results from this methodology can be used for classification and definition of acceptable risk levels and the appropriate measures i.e. the management of risks.

References

[1] Sousa, L.R.: Risk Analysis for Tunneling Projects, Massachusetts Institute of Technology, Department of Civil and Environmental Engineering, PhD Thesis, 2010.

[2] Eskesen, D.S., Tengborg, P., Kampmann, J., Veicherts, H.T.: Guidelines for tunnelling risk management: International Tunnelling Association, Working Group No. 2, Tunnelling and Underground Space Technology 19, pp. 217-237, 2004.

[3] Hyun, C.K., Min, S., Choi, H., Park, J., Lee, M.I.: Risk analysis using fault-tree analysis (FTA) and analytic hierarchy process (AHP) applicable to shield TBM tunnels, Tunnelling and Underground Space Technology, 49 (2015), pp. 121-129

[4] Stille, H.E.: Geological Uncertainties in Tunnelling – Risk Assessment and Quality Assurance, International Tunnelling and Underground Space Association (ITA), Sir Muir Wood Lecture, 2017.

[5] Kikkawa, N., Itoh, K., Hori, T., Toyosawa, Y., Orense, P.R.: Analysis of labour accidents in tunnel construction and introduction of prevention measures, Industrial Health, 53 (2015), pp. 517-521

[6] Seidenfus, T.: Collapses in Tunnelling, Stuttgart University of Applied Sciences, MSc Thesis, 2006.

[7] Špačková, O.: Risk management in tunnel construction projects, Czech Technical University in Prague, Faculty of Civil Engineering, PhD Thesis, 2012.

[8] Yoe, C: Principles of Risk Analysis – Decision Making Under Uncertainty, Second Edition, CRC Press, 2019.

FIRE RESISTANCE OF CONCRETE LINING IN ROAD TUNNELS

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Abstract

Fire is an incidental load on structures. Experience has shown that in the event of a fire, extremely high temperatures are developed, as a result of which very often a colapse of the tunnel bearing structure happens, usually caused by spalling of concrete.
Road tunnel fires are usually caused by vehicles using the tunnel, but can also be caused by exidants, technical breakdowns in tunnel equipment or improper tunnel maintenance. The intensity and frequency of fires in the tunnels are function of several factors such as: length and geometry of the tunnel, density and type of traffic, vehicle speed, slope, availability of ventilation equipment and so on. All of these factors define the fire risk. Based on the defined fire risk, known fire load and location of the fire, it is possible to define the fire curve that defines the temperature in the tunnel versus time. Several fire curves, usually used in EU countries, will be described in this paper, the fire curves characteristics will be defined and the most proper fire curve for typical tunnel, as case study, will be recomended.
In the framework of this paper, a methodology for fire resistance analysis of road tunnels, based on the performance, is elaborated. A numerical procedure for defining the behavior of the tunnel lining in case of nominal fire curve (standard fire) is described and applied on one case study. The impact of the fire on the stress-strain state of the concrete structure of a tunnel is analyzed and measures for proper tunnel design in terms of increasing the fire resistance is proposed.

Keywords: tunnel, fire risk, fire curve, thermal analysis, stress-strain analysis

1 Introduction

Fires in road tunnels are usually result of ignition of vehicles using the tunnel and are mainly caused by: electrical defects (most common cars); overheating of breaks (about 60 % to 70 % of fires caused by trucks) and other defects leading to self-ignition of the vehicle [1-3].
Other reasons, which are very rare, but still exist, are: car exidants; technical defects (self-ignition) of equipment in tunnels and improper performance of maintenance work.
Theoretically, the frequency of tunnel fires is related to factors such as: the length of the tunnel, traffic density, speed control and slope. These factors should be taken into account when comparing different tunnels. Therefore, to include the effects of tunnel length and traffic density, the frequency of fires is estimated not only by the number of fires per tunnel, but also by the number of vehicles per kilometer. All these factors define the fire risk.
The safety of the tunnels in case of fire depends on the applied cladding system and the appropriate design procedures for taking into acount the fire as incidental load on the tunnel structure. The “design fire”, defined in terms of its increacing rate and duration, provides the basis for determining the necessary protection systems and influence on the operational

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measures that have to be set. In case of new tunnels, this may affect the choice of tunnel configuration, or the use of additional mitigation systems.
In general, according to the options given in Eurocodes, a nominal approach for designing fire-safe tunnels has been adopted. Normally, in the past, the rate of increase of fire was adopted in the range up to a maximum value of 30 MW. However, the experience of real large tunnel fires and the tests performed in real dimensions indicated that there could be much larger fires. For this reason, different fire models have been adopted in different countries, ranging from 20 to 300 MW.
Given the large range of fires with a varying intensity, it is evident that the choice of fire intensity for a particular tunnel cannot be precisely defined without including all relevant factors. Several factors need to be considered, such as the type of traffic, the ventilation system, the geometry of the tunnel and the fire-fighting system. Even in the nominal approach, when the fire is defined by a prescribed temperature-time curve, such considerations are taken into account.
In order to provide more guidelines for the selection process, a methodology for a performance-based approach has been developed in this paper and a numerical analysis of the tunnel lining behavior in the case of nominal (standard) fire has been conducted. The influence of the design fire on the stress-strain state of the tunnel was analyzed. Tunnels with and without reinforcement of the secondary lining and with different lining thickness were analyzed. Based on the obtained results, measures for appropriate design of tunnels from the aspect of increasing their fire resistance are proposed.

2 Fire risk and protection measures for tunnels

As a result of more intensive road traffic and the need of fast underground communications, the probability of accidents in tunnels caused by fire has increased. Additional factors that increase the fire risk are:
- the increased length of modern tunnels;
- increased vehicle speed;
- transport of dangerous goods;
- two-way traffic with physically undivided carriageways;
- increased fire load as a result of increased vehicle volume and increased transport capacity;
- mechanical defects in motor vehicles.

There are three reasons for taking protection measures against fire in a tunnel. The first and most important reason is the safety of the passengers. To fulfill this condition, on time evacuation is required, and it depends not only on the stability of the structure, but also on the functionality of the ventilation system, the accessibility of the evacuation exits, etc. The second reason is to enable the tunnel to function and the traffic to run smoothly. Very often, fires in tunnels cause explosive spalling of concrete and collapse of parts of the structure. The third reason are the economic losses caused not only by the structural damages, but much more by the non-functionality of the tunnel and the interruption of the traffic during the rehabilitation period, which is certainly a long period of time.
In order to ensure adequate fire safety of tunnels, it is necessary to pay attention to the following aspects: fire resistance of the load-bearing structure; air supply system; ventilation and smoke extraction system; construction of evacuation pats protected from smoke and flames; active and passive fire detection systems; fire extinguishing systems; fire doors and alarm systems.

3 Fire models for road tunnels

A special feature that makes tunnel fires different from other fires (for example, those that occur in buildings) is the sudden rise in air temperature under the vault, which can reach over 1000 ° C in just a few minutes. This phenomenon negatively affects both the fire extinguishing process (rapid extinguishing is almost impossible) and the structural system.
In order to be able to perform appropriate thermal and static analysis of the tunnel structure and to define its fire resistance, it is first necessary to define the temperature of the fluid (air) inside the tunnel. This is possible by implementing a numerical procedure that solves the differential equations of heat release and transfer based on the laws of fluid dynamics (CFD). The temperature inside the tunnel is influenced by: the length and geometry of the tunnel, the fire load, the burning time of the primary ignited vehicle, the materials that are built into the structure, etc. The ventilation has an effect on the HRR of the burning items and should be considered when designing the type of fire curve and the period of required fire protection.
Eurocode 1-1-2 [4], which treats fire as an incidental load for the structure, defines the fire load due to the increase in temperature in the fire sector in time. Nominal curves "temperature - time" are defined, but they are valid only for indoor fires (buildings). The European countries, in their national regulations for design of tunnels, define curves "time-temperature" for a nominal fire in a tunnel [1, 2]. They are defined on the basis of results from conducted fire risk analyses, laboratory tests and experiences of fires in tunnels.
The most characteristic and often used "time-temperature" curves are presented in Figure 1. The cellulose or ISO 834 fire curve is typical for fires in buildings. The Hydrocarbon curve is applicable where small petroleum type fires might occur, e.g. car fuel tanks, petrol or oil tankers, certain chemical tankers. The temperature rise in case of Hydrocarbon curve is far more rapid than in case of ISO 834 fire curve, but after the initial 30 min. the temperature follows an almost horizontal line. The RABT-ZTV curve was developed in Germany as a result of a series of tunnel fire tests, such as Eureka project. In RABT curve the temperature rise is very rapid, up to 1200°C within 5 minutes. The duration of this temperature is shorter than for other fire curves and drops after 30 minutes in case of road tunnel, and after 60 minutes in case of railway tunnels.
The RSW curve was developed in Netherlands. This fire curve is based on the assumption that in a worst case scenario, a fuel oil or petrol tanker with a fire load of 300 MW lasting up to 120 minutes could occur. This curve is usable for enclosed area, such as tunnel, where there is little or no chance of heat dissipating into the surrounding atmosphere. The RWS curve simulates the initial rapid growth of a fire in case of petroleum tanker source, and gradual drop in temperatures when the fuel burnt off.
In the Netherlands, this curve lasts 120 minutes, as it is estimated that by then the temperature will have dropped to a level that will allow firefighters to approach the vehicle and put out the fire. This curve is also used in Switzerland and Austria, but the time is 180 minutes as the tunnels under the mountain massifs are significantly longer. When this curve is applied, the criterion for failure of the structure is taken to be the moment when the temperature of the surface of the primary tunnel lining, which is protected by concrete secondary tunnel lining or by installing insulation materials, reaches 380 °C, and the temperature in the reinforcement does not exceed 250 °C. If high-strength concrete is used, which is more sensitive to explosive spalling, the surface temperature is limited to 250 °C.

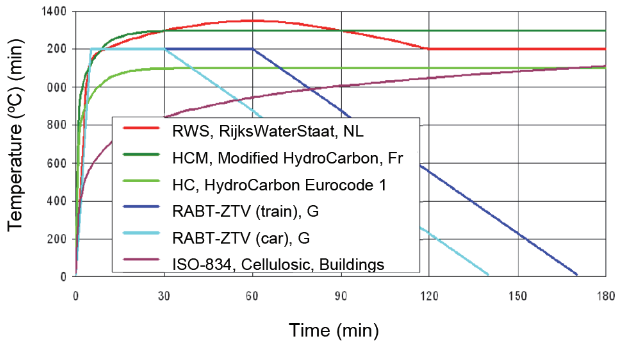

Figure 1 Standard temperature-time curves used to simulate a fire in a tunnel

Figure 2 compares these curves with the results of the experimental study, Eureka project [1]. The fire was caused by wooden and plastic pallets and 240 GJ of heat was released. The temperature was measured 10 m from the energy source. It is obvious that the RWS curve is closest to the real development of the temperature when this type of fire will happen.

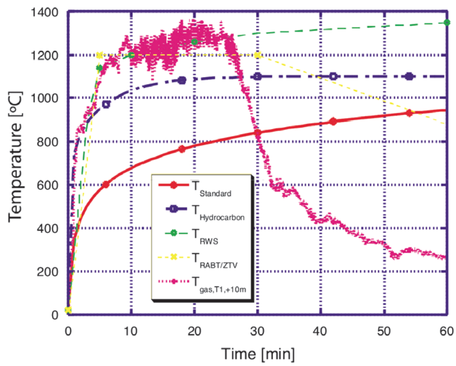

Figure 2 Comparison of Standard "temperature-time" fire curves and measured temperatures in a real fire in a tunnel - EUREKA project

4 Fire resistance analysis of concrete lining of road tunnel

As it was already mentioned, one of the factors that influence the fire safety of tunnels is the stability of the tunnel structure which is exposed to extremely high temperatures. This problem connects two parallel analyses: thermal analysis for defining the temperature distribution in the cross section of the tunnel linings and the stress-strain analysis for defining the structural response.

This paper presents the analysis results for a road tunnel exposed to RSW fire curve. For the analyses the program FIRE [5], based on Finite Element Method, was used. The tunnel length is about 900 m and belongs to the group of short tunnels. The road width is 3 x 3.50 m, the height of the tunnel is 4.70 m, with a slope of 2.5 % to 4.0 %.

On one part, the secondary tunnel lining is made of plain concrete, and on the other part as reinforced concrete arch structure. Concrete grade MB30 is applied. The minimum thickness at the top of the dome for section type 1 (plain concrete lining) is d = 30 cm, and for section type 2 (reinforced concrete lining) is d = 45 cm. The two cross-sections are shown in Figure 3.

The calculation of the fire resistance of the tunnel was performed in accordance with Austrian regulations. According to the characteristics of the traffic, the fire load is defined by the nominal fire curve RWS for 3 hours (180 min). The criterion for fire resistance is taken to be the

moment when the temperature of the surface of the primary tunnel lining, which is protected by concrete secondary tunnel lining, reaches 380 °C, and the temperature in the reinforcement does not exceed 250 °C (the lower value is adopted).
The failure time of the secondary concrete lining is defined as a third condition. Failure occurs as a result of temperature-induced stresses and in case of failure the primary lining is directly exposed to fire, so the surface temperature of the concrete is almost equal to the air temperature inside the tunnel.

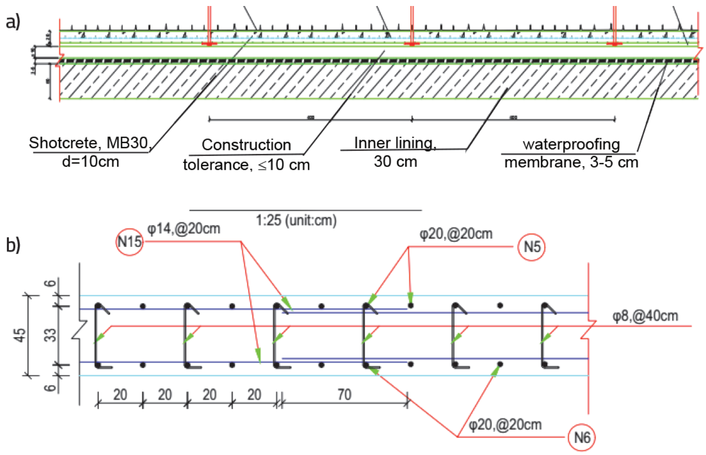

Figure 3 Secondary tunnel lining: a) plain concrete d=30 cm, b) RC lining d=45 cm

The thermal analysis was performed for a 1,00 m width strip of the arch structure. Due to the axial symmetry, only one half of the arch structure was analysed. In the thermal analysis, in addition to the concrete lining, the waterproofing layer with a thickness of d = 3 cm, the free air space d = 10 cm and the primary reinforced concrete lining with a thickness of d = 10 cm were included. The discretization of the cross section was performed with 994 finite elements with 4 nodes, of which 680 were used for the secondary lining. The time step was Δt = 0.01 hours = 0.6 minutes = 36 seconds.
The temperature inside the tunnel was defined by the fire curve RWS, Figure 1, as the most appropriate fire load. When discretizing the secondary tunnel lining, the symmetry of the tunnel cross-section and the loads was used. 11 elements with a width of 1 m were used and were placed in such a way to follow the curvature of the tunnel lining (Figure 4a). The first element was fixed in the terrain, while the rotation and the horizontal displacement at the end point of the last element (highest point of the tunnel) were restricted and only vertical displacement was free.
Temperature-dependent physical and mechanical characteristics of concrete and steel (coefficient of thermal conductivity, specific heat capacity, density, compressive strength and tensile strength of concrete, tensile strength of steel and modulus of elasticity) were adopted in accordance with the recommendations given in EN1992-1-2 [4], while the thermal properties for the waterproofing and the air in the cavity were taken in accordance with the data given in the literature. Isotherms in the cross section of the tunnel structure after 180 minutes of fire action are presented in Figure 4b, only for the case of tunnel lining type 1 (secondary lining d = 30 cm).
The non uniform temperature field in the initial moments of fire action (only the surface layers of the inner part of the lining are heated to more than 1000 °C, while the layers on the opposite side are at 20 °C) and the inability the thermal dilatation of the cross section to be freely realized, leads to additional bending moments.

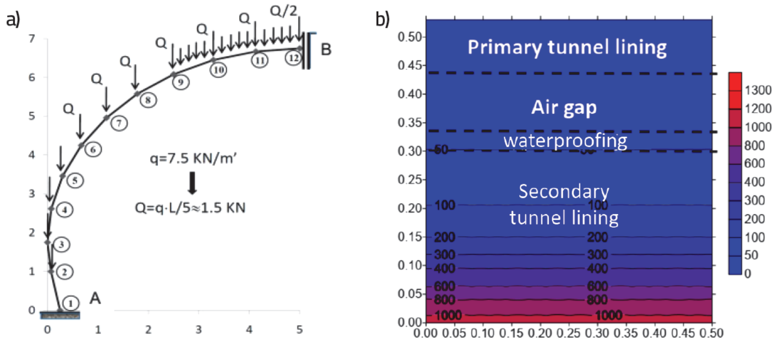

Figure 4 a) Discretization of the tunnel structure; b) Isotherms in the cross section of tunnel structure in case of lining d=30 cm, after 180 min of fire exposure

According to Figure 4b, it could be concluded that the criterion for fire resistance concerning the temperature at the surface of the primary tunnel lining is not reached after 180 minutes of fire exposure, as the temperature is lower than 380 °C. The temperature in the reinforcement in the primary lining does not exceed 250 °C, too. It is even better for concrete lining type 2 (d = 45 cm).

Additionally, the third criterion concerning the collapse of the secondary lining is controlled. If free thermal expansion of the secondary lining is possible, no axial compressive forces will occur and the bending moments caused by temperature difference will result in compression at the hot side of the cross section and tension at the opposite and cold side. In case when the concrete lining is not reinforced, there is no option for accepting the tensile forces and cracks will appear on the cold side of the lining. At the same time, in the hot inner zone, the compressive stresses reach up to 90 % of the concrete strength for the respective temperature. When the cracked zone expands to more than 80 % of the lining cross-section, the balance of the internal forces cannot be achieved and the cross-section will fail due to crushing of the "hot" concrete. In this case the failure occurs only after t = 0.3 hours = 18 minutes.

In order to solve the problem with the bearing capacity of the secondary lining in the initial moments of fire exposure, there are two solutions: to prevent free dilatation by anchoring the secondary lining to the primary lining, or to install a minimum percentage of reinforcement at the inner cold side of the lining, for accepting the tensile forces. In both cases, the secondary lining achieves fire resistance for more than 6 hours.

Figure 5 presents the time dependent stresses at the top and the bottom edge of the secondary concrete lining, in case of minimal reinforcement (μ = 0.1 %) placed in the tensioned (upper) zone of the cross section, expressed as a percentage of the bearing capacity of the concrete for the actual temperature ($\sigma_c/f_c(T)$). Due to the large temperature differences in the concrete (hot lower and cold upper part of the cross section) and the impossibility of free thermal expansion, after only 15 minutes of fire action the compression stresses in the lower zone increase significantly, while in the upper zone tensile stress occurs. Over time, as the temperature difference decreases (the temperature penetrates deeper into the cross section), the compressive stresses in the lower zone slowly decrease. The tensile stresses in the upper zone have to be accepted by the reinforcement, which, even minimal, ensures the balance of the internal forces in the cross section.

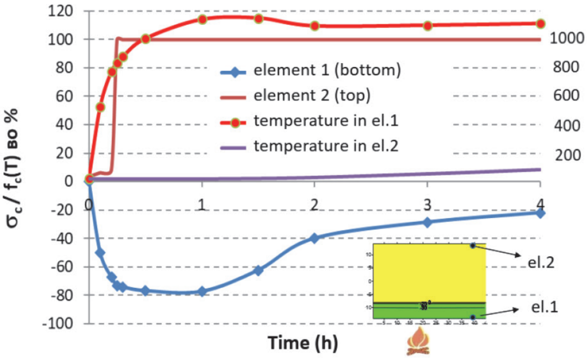

Figure 5 Time dependent stresses at the top and bottom edge of the secondary concrete lining, in case of minimal reinforcement, as a percent of the bearing capacity of concrete for current temperature ($\sigma_c / f_c(T)$)

5 Conclusions

Based on the numerical analysis, it was determined that in order to ensure stability of the tunnel structure in case of fire, a reinforced concrete secondary lining has to be constructed. Due to the tensile stresses on the cold side of the lining, if there is no reinforcement, the secondary lining will collapse in the first twenty minutes of fire exposure and will not be able to protect the primary tunnel lining during the required period of fire resistance. Reinforcement of the secondary lining, even with a minimum percentage, will delay the moment of failure and will additionally reduce the risk of explosive spalling of concrete, which is a special problem in tunnels and occurs not only in high-strength concrete, but also in normal concrete.

References

[1] ASFP, Tunnel fire protection, www.promat-tunnel.com

[2] Austrian Guideline Codes for Planning, Construction and Maintenance of Roads (RWS)

[3] Directive of the European Parliament and of the Council on minimum safety requirements for tunnels in the Trans-European Road Network.

[4] Eurocode 2: Design of concrete structures - Part 1-2: General rules - Structural Fire Design. Brussels : European Commitee For Standardization, 2004. EN 1992-1-2:2004

[5] Cvetkovska, M.: Fire resistance of Reinforced concrete structures, Doctoral thesis, Ss. Cyril and Methodius University in Skopje, 2002

IMPLEMENTATION OF STRUCTURAL HEALTH MONITORING INTO LIFE CYCLE MANAGEMENT OF TUNNELS: CASE STUDY TUNNEL BRAJDICA

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Abstract

This paper presents a case study focused on the Brajdica railway tunnel, which carries the Zagreb-Rijeka railway line into the port of Rijeka in Croatia and thus represents a critical node on the European TEN-T network. The tunnel is undergoing a major reconstruction project to increase its capacity. As part of this work an extensive embedded monitoring system comprising inclinometers, extensometers, micrometers and survey markers were installed to monitor the tunnel response. This data is supplemented with periodical laser scanning of the tunnel interior. Measurement profiles are set along the tunnel bore and the data collected is used for the development of a tunnel performance model. Long-term monitoring data from a neighbouring road tunnel was used to develop models tho predict the long-term response of the Karst bedrock in the area. Combining these models with the settlements measured during the construction phase of the works at Brajdica railway tunnel allow prediction of settlements and the future occurrence of damage in the rail tunnel. Based on different limit states, life cycle management scenarios are developed and used for maintenance planning, with the aim to decrease short and long-term risks. This work has been performed within European H2020 SAFE-10-T project.

Keywords:

1 Introduction

Tunnels are one of the most critical structures on transport infrastructure networks which provide vital links for society, having a large impact on the regional economy and environment. They are designed and built for a long design life (usually for more than 100 years) and therefore regular maintenance interventions are necessary in this period. Poor understanding of tunnel behaviour, in all life cycle phases, has a result that planning the scope and extent of any intervention is challenging and unnecessary and/or ineffective maintenance and repair activities can cause large financial and environmental costs. Calculation of total life time costs for different design alternatives, maintenance options and societal impacts can be used to compare different technical solutions and select the optimal design and maintenance alternative. Within the SAFE-10-T European research project embedded monitoring techniques and data analytics solutions were developed with aim to improve probabilistic analyses tools for major infrastructure objects (bridges, tunnels and earthworks) resulting in safer and more environmentally sustainable infrastructure. A case study of tunnel Brajdica

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presented in this paper demonstrates the implementation of embedded monitoring systems into long-term predictive performance models and finally life cycle planning for tunnels in general [1].

2 Life cycle management of tunnels

The geological and structural characteristics of a tunnel support system will undergo significant changes during the life-time of the asset. Long-term structural assessment of tunnels support elements is one of the key activities in maintaining the reliability and safety of a tunnel during its service life. The lining structure of a tunnel is subjected to both external loading (weight of retained ground and traffic) and environmental effects like leaking or frost damage for example, see Figure 1.
Generally problems related to tunnel degradation can be divided into those caused by external pressure and those caused by the deterioration of materials [2]. These problems change through all life cycle stages of a concrete structure such as a tunnel, therefore decisions about the timing and the type of maintenance should be based on degradation prediction models and monitoring of the structural performance or degradation processes. Uncertainties in the decision making process can be decreased by using structural health monitoring data and structural models, in order to determine triggering thresholds for the structure passing certain performance levels.

Figure 1 Leakage and cracking in tunnels due to the external forces

3 Case study

3.1 Description of the project

The city of Rijeka is the principal seaport and the third-largest city in Croatia. It is located in the northern coast of Adriatic Sea (131 km southwest of the capital Zagreb), in the Rijeka Bay, which is a part of the Kvarner Gulf. The port is part of the Baltic-Adriatic Corridor on the TEN-T network. The current facilities are being significantly enhanced through the development of a multimodal transport hub in the Port of Rijeka including a connection with the Adriatic Gate container terminal [3]. As part of this project, reconstruction of the Rijeka Brajdica railway station is carried out and an intermodal container terminal is built. Works on the reconstruction and expansion of the capacity of the Rijeka Brajdica freight terminal include the complete reconstruction of the existing nine tracks and an expansion of the existing Brajdica railway tunnel (Figure 2).

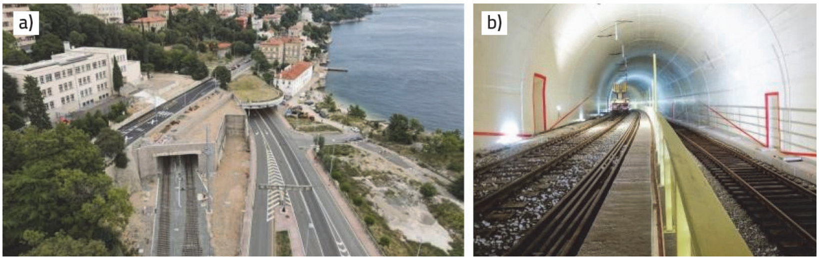

Figure 2 a) Entrance to the renovated and extended railway tunnel Brajdica beside the road tunnel Pećine, b) Finished expanded railway tunnel [3]

One of the unique features of the tunnel Brajdica project is that the railway tunnel is being reconstructed in close proximity to a modern highway tunnel, See Figure 3. The construction of highway tunnel Pećine started in 2005 and the tunnel was opened in 2008. The point at which the tunnels are closest together is at a distance of 11.5 meters between the main structural elements and the rock bolts required for stability of the area where both tunnels are overlapping. It is clear therefore that accurate prediction of the stress-strain response due to reconstruction works is essential for the safety of both tunnels [4 - 6].

3.2 Structural health monitoring of tunnels

Tunnel construction is by its nature an uncertain activity, with the volume of soil and rock being tested even during a comprehensive site investigation being very small in comparison to the volume of excavation. An additional complication in this project is that the rock in this region is formed of cretaceous deposits, breccias, dolomites and limestones, of relatively high permeability. These rocks are highly susceptibility to karstification processes, and phenomena including caverns, voids etc. are commonly encountered during tunnel construction. Recognizing these uncertainties the construction of the Pećine tunnel involved three interlinked phases; (i) ground investigation, (ii) numerical modelling and (iii) instrumentation. The instrumentation phase was seen as vital to confirming the ground model derived from the site investigation and to confirm the constitutive model parameters used in the finite element analyses. The monitoring system installed in Pećine tunnel included inclinometers, extensometers, micrometers and survey markers which supplemented periodical laser scanning of the tunnel interior to monitor ground movement. In addition load cells were placed on some rock bolts, shown in Figure 3 and strain gauges were embedded in the shotcrete lining to monitor bending moments.

The vertical deformation pattern measured directly above the crown of the tunnel Pećine during the 15 year period is shown in Figure 3 [4]. Kovacevic et al. [4] used this data in the development of a model to predict long-term vertical settlement performance of a tunnel in soft rock mass, through the inclusion of a Burger's creep viscous-plastic constitutive law to model post-construction deformations. To overcome issues related to the complex characterization of this constitutive model, a neural network NetRHEO was developed and trained on a dataset obtained using extensive numerical analyses. This model allowed to study the impact of uncertainties on the response of both tunnels to the reconstruction work being undertaken on tunnel Brajdica. This allowed the complex interaction of railway tunnel Brajdica and road tunnel Pećine to be evaluated prior to the reconstruction work on the former [4].

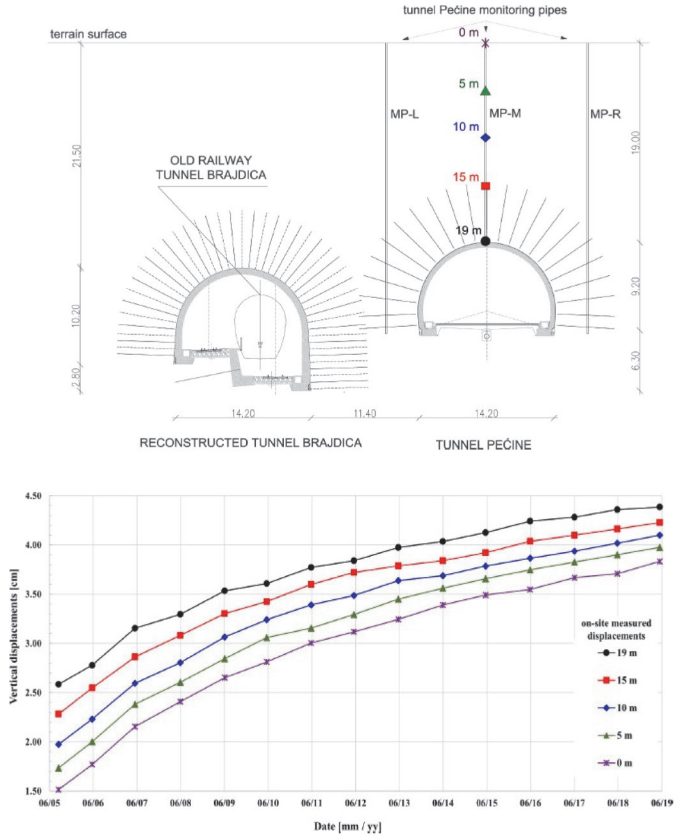

Figure 3 Location of deformation measurements along the vertical shaft and vertical displacement obtained by measurements during construction

3.3 Whole life cycle cost model

Whole life cycle cost (WLCC) analysis aims to support the management of physical assets, promoting informed decisions on the phases of design, construction, operation and end of service life. WLCC methodology gives great importance to the operational period, which includes maintenance, monitoring and inspection activities, with the direct and indirect associated costs they have on the overall economic performance of construction projects [1, 2]. The model developed within our project is integrating monitoring data, previous knowledge and long-term predictions of deformation, which can cause damage of the tunnel and increased risk of the failures. An example of how the impact of gradual deterioration of an ageing structure can be combined with monitoring data to establish threshold values which can then be used for determination of optimal timing for certain measures is shown in Figure 4. The experience gained from previous continuous monitoring systems installed for periods in excess of twenty years in several tunnels bored in the karst bedrock prevalent in Croatia is used for the development of long term performance models [4, 7]. Serviceability limit states (SLS) that can be linked to the continuum response of the tunnel are defined as follows:

- The admissible settlement value at the tunnel crown is exceeded.
- The admissible value of the axial force in anchors is exceeded.
- The compression capacity of the tunnel lining is exceeded due to combined bending moment and axial force.

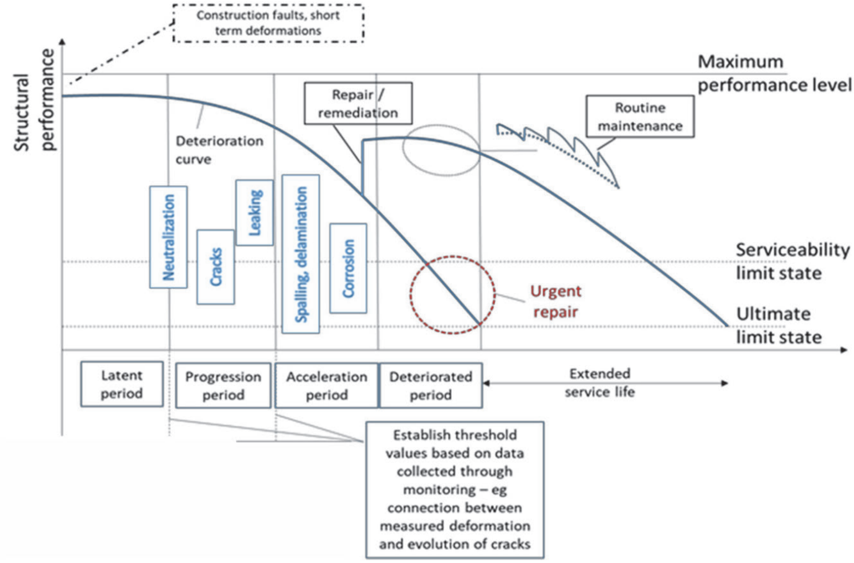

Figure 4 Life cycle model for tunnels using monitoring data and numerical prediction models

Based on the owner's experience with maintenance of similar structures, quantification of direct and indirect costs of certain maintenance activities (related to the SLS thresholds) is performed. Analysis of the whole life cycle cost is aimed for improving decision making process and does not necessarily mean choosing the longest service life or the minimum costs but rather choosing an optimal solution based on reliable data.

In order to determine the value of structural health monitoring, two different life cycle management scenarios are compared, one with the implementation of an embedded monitoring system and the other without. The frequency of maintenance activities is currently based on the expert judgment and previous experience with tunnels in the similar environment. In the future developments the long term deformations model will be used to predict the occurrence of damage and to plan the maintenance activities. The life cycle cost model therefore takes into account the following items:

$$TotalLCC = ICC + \sum_{t=0}^{T} \frac{MC_{t,nom}}{(1+r)^t} + \sum_{t=0}^{T} \frac{TDC_{fr,t,nom}}{(1+r)^t} \tag{1}$$

Wherein ICC = initial construction costs (€), MCt, nom = maintenance costs for year t (€), t = year in life cycle from 0 until end of life cycle T, r = the discount factor (%), TDCfr,t, nom = nominal freight traffic delay costs in year t (€).

The maintenance costs are calculated based on the quantity of maintenance activities, their accompanying frequencies and their estimated unit costs. The traffic delay costs occur due to the increased time spent on traveling or due to the unavailability of the network caused by the maintenance activities. They are based on the value of time for users, extra travel time, duration of the maintenance activity and average daily traffic [8, 9]. In the case of tunnel Brajdica which serves as a link to the port and containers terminal, freight trains are the only relevant type of users. The traffic delay costs can be then determined by:

$$TDC_t = ETT \times ADT_t \times VOT \times N_t \tag{2}$$

Wherein TDCt = traffic delay costs for year t (€), calculated separately for freight and for passenger cars, ETT = extra travel time per disruption / maintenance activity (hours), ADTt = average daily traffic in year t passing the analysed tunnel (tonne of freight/day), VOT = is a monetary value for the freight users (€/hour/tonne), Nt = duration of a certain maintenance activity (days).

4 Results

The WLCC model takes into account direct and indirect costs, as presented before for two different management strategies, one based on the historical performance and expert judgment, without embedded structural health monitoring and the other one with embedded structural health monitoring system, which enables data collection periodically during construction and operational stage. Monitoring data is used then for training the neural network model and for the prediction of the future deformations. The aim of the WLCC model is to provide to the infrastructure owner the insights into the impacts of different maintenance strategies and enable optimal decision making. In the model input parameters can be changed according to the decisions made. For the results presented here, we have used the discount rate of 1.5 %, traffic scenarios from [8] and the value of time based on the studies [9, 10]. Traffic regulations and duration of the maintenance activities are used from the current practice of Croatian Railways [11]. The outputs of the costs calculations are presented in Figure 5. The incorporation of SHM reduces both the direct and user delay costs significantly resulting in a 50 % reduction in overall costs over the 100 year-life span considered.

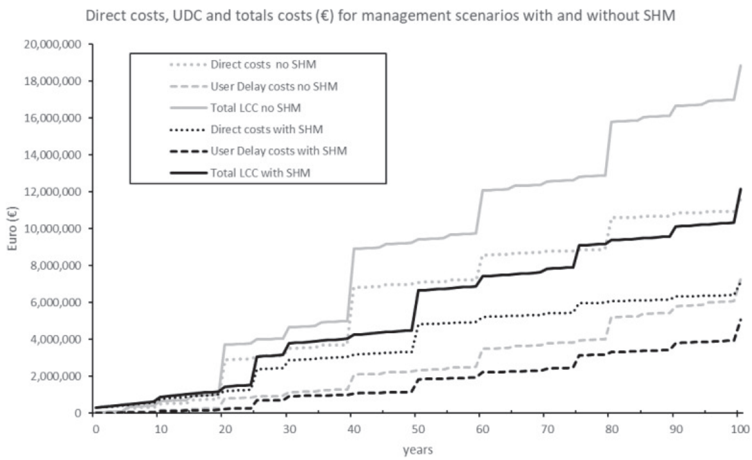

Figure 5 Results of the Life cycle model for two different tunnel maintenance scenarios

5 Conclusions

This paper presents a case study of Brajdica railway tunnel, which carries the Zagreb-Rijeka railway line in Croatia into the multi-modal port of Rijeka and thus represents a critical node on the TEN-T network. The tunnel was a part of a major reconstruction project, during which an extensive embedded monitoring system wasinstalled. The whole life cycle cost analysis model was developed for the management of the tunnel with the aim to support decision making process related to the phases of design, construction, operation and end of service life. The model developed within this project has integrated monitoring data and long-term predictions of deformation into the life cycle performance model. The experience gained from previous continuous monitoring systems installed for periods in excess of twenty years in several tunnels bored in the karst bedrock prevalent in Croatia has been used for the development of the long term performance model, which is capable of predicting forces and stresses in structural elements, thus predicting the occurrence of cracking. The performance model is then integrated into the long term maintenance planning, with the aim of developing optimal management strategies. Within this case study two management strategies were compared, one based on the historical performance and expert judgment without embedded structural health monitoring and the other one with embedded structural health monitoring system. The prediction of costs in the next 100 years clearly shows that the investments into SHM system pays off within first two decades and enables more than fifty percent of saving in the total costs after 100 years.

Acknowledgements

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References

[1] Stipanovic, I., Kovacevic, M.S., Bacic, M., Gavin, K., Skaric Palic, S.: SAFE-10-T Deliverable 4.2 Report on the results of Case Study 2 – Croatia, 2020, www.safe10tproject.eu/

[2] Skaric Palic, S., Stipanovic, I. Kovacevic, M.S., Gavin, K.: Life cycle management model for tunnels, CETRA 2018, 5th International Conference on Road and Rail Infrastructure, 17 - 19 May 2018, Zadar, Croatia, doi: 10.5592/CO/CETRA.2018.913

[3] Razvoj multimodalne platforme u Luci Rijeka i povezivanje s kontejnerskim terminalom Jadranska vrata (POR2CORE-AGCT), https://www.hzinfra.hr/rekonstrukcija- zeljeznickog-kolodvora-rijeka-brajdica-i-kontejnerskog-terminala-brajdica/

[4] Kovačević, M.S., Bačić, M., Gavin, K., Stipanović, I.: Assessment of long-term deformation of a tunnel in soft rock by utilizing particle swarm optimized neural network, Tunnelling and Underground Space Technology, 110 (2021), doi: https://doi.org/10.1016/j.tust.2021.103838.

[5] Hoek, E.: Reliability of the Hoek–Brown estimates of rock mass properties and their impact on design, International Journal of Rock Mechanics and Mining Sciences, 35 (1998), pp. 63–68, https://doi.org/10.1016/S0148-9062(97)00314-8

[6] Hoek, E., Carranza-Torres, C., Corkum, B.: Hoek-Brown failure criterion, 5th North American Rock Mechanics Symposium and 17th Tunnelling Association of Canada: NARMS-TAC, Toronto, Canada, 2002, pp. 267-273

[7] Kovačević, M.S., Gavin, K., Stipanović Oslaković, I., Librić, L., Rossi, N.: Monitoring of long term deformations in Bobova tunnel, CETRA 2018, 5th International Conference on Road and Rail Infrastructure, 17 - 19 May 2018, Zadar, Croatia, doi: 10.5592/CO/cetra.2018.844

[8] Davis Langdon Management Consulting, Guidance Document, Life cycle costing (LCC) as a contribution to sustainable construction, Ref. Ares(2014)1138398 - 10/04/2014.

[9] de Jong, G.: International Comparison of Transport Appraisal Practice, Annex 3 The Netherlands Country Report Project Funded by the Department for Transport, UK, Institute for Transport Studies, retrieved from http://ec.europa.eu/transport/themes/its/road/action_plan/doc/2012-its-plan-the-netherlands-2013-2017.pdf, 2013.

[10] Sundquist, H., Karoumi, R.: Life Cycle Cost Methodology and LCC Tools. ETSI. Retrieved from http://www.etsi.aalto.fi/Etsi3/PDF/TG3/LCC%20Description.pdf, 2012.

[11] HZ Infrastruktura, Annual Report for 2018, 2019, http://www.hzinfra.hr/wp- content/uploads/2019/07/Godisnje-izvjesce-HZI-2018.pdf

SUMMARIZE OF DETAIL DESIGN FOR PELJEŠAC BRIDGE PROJECT IN CROATIA

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Abstract

The Pelješac Bridge is a Extradosed cable-stayed bridge with the span arrangement 84 + 108 + 108 + 189,5 + 5 x 285 + 189,5 + 108 + 108 + 84 m = 2404 m under construction in Croatia at present. The purpose of this bridge is to achieve territorial continuity of the Republic of Croatia by connecting the southern exclave comprising the bulk of Dubrovnik-Neretva County with the remainder of the Croatian mainland. China Road and Bridge Corporation (CRBC) Joint Venture (JV) won the bidding of this project following the FIDIC Red Book Contract Condition in April, 2018. As the Contractor CRBC JV executed detail design with their own advantages based on the main design, which was issued by the Client Croatian Roads. In the detail design, the feasibility of construction technology of each element is firstly considered by CRBC JV technical staffs. After that, the other parts of detail design, i.e. the design for execution were prepared according to the construction technology and the main design. In the mentioned progress, the Contractor should communicate with the designer of the main design as closely as possible for higher efficiency. CRBC JV prepared the detail design for all elements of the whole bridge, including piles foundation, pile caps, pylons and piers, steel superstructures and etc. The main detail design ideas and details of each element will be introduced and described in this article for providing references for implementation of similar projects in the future.

Keywords: Pelješac bridge, detail design, foundation, pylon and piers, steel superstructure

1 Introduction

To be a national unity bridge, Pelješac Bridge will establish the connection between the mainland of Croatia and its geographically separated area, i.e. Dubrovnik – Neretva county. On 21 May, 2018, accompanied the effectiveness of construction contract between Croatian Roads and CRBC JV, a dream of unification of Croatian territory was about to come true. Following main design made by its Consultants, i.e. JV of University of Zagreb, Ponting d.o.o and Pipenbaher Inženirji d.o.o, the detail design is prepared by the Contractor, then reviewed and approved by the designer of main design, the auditor (Ramboll Group A/S) and the Engineer JV of Institut IGH d.d., Centar za organizaciju građenja d.o.o. and Investinženjering d.o.o..

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2 Main design scheme description

2.1 Location

The Pelješac Bridge (Mid-point coordinates 42º56'23" N and 17º32'38" E) crosses the Mali Ston bay of around 2140 m wide on the sea. Total length of the bridge between the both axes of the abutments is 2404 m, when the whole length of the bridge including abutment is 2440 m. It is located in a marine nature protection area, closed by Pelješac peninsula on one side and mainland on the opposite side.

2.2 Principle dimensions

The whole bridge was defined as extradosed cable-stayed bridge with the layout of 84 + 108 + 108 + 189,5 + 5 x 285 + 189,5 + 108 + 108 + 84 = 2404 m, in which 189,5 +5 x 285 + 189,5 m is the central cable stayed span part and 84 + 108 + 108 m is the approach span part on both sides (Fig.1). The 4,5m-high superstructure is made up of steel box girders, which were divided into 165 segments due to various erection methods, including 12m-long standard segments of main spans, 36~56-meter-long large segments and 12m-long segments of side spans.

Figure 1 Span arrangement of Pelješac Bridge [1]

2.3 Substructure

There were 6 pylons, 6 piers and 2 abutments on this bridge. For pylon No. from S5 to S10, the foundation was founded on 18 or 20 vertical grouped steel tubular piles in 4 rows, which were 2,0 m-diameter with the maximum length of 128,4 m. For the other piers at sea S3, S4, S11 and S12 were supported by 9 vertical grouped piles of diameter 1,8 or 2,0m, and of maximum length of 80,5m. In addition, the foundation of S2, abutments U1 and U14 were constructed by mass concrete, and the well foundation was adopted to S13, which consisted of 22 overlapped concrete or RC piles. The piles of S5 to S9 were end bearing piles, except that the other piles underwater were steel-concrete composite piles with concrete sockets. The pile caps dimensions of piers S3, S4, S11 and S12 were 17,0 x 17,0 x 4,50m, while the pile caps dimensions of pylons were 23,0 x 29,0 x 5,0m. All the piers were thin-wall hollow section with height range from 19,4 to 53,235m.

3 Foundation

3.1 Steel tubular piles

As requested in the main design, the Contractor completed the supplementary geological investigation reports in advance. The main designer determined the final pile length, the Contractor completed the corresponding detail design of pile foundation as well as the construction method statements. The worldwide largest pile driving barge from China were adopted to achieve the driving of extra-long steel pile (No.TP7) once successfully, and the length of 130,6 m was set the worldwide record for once pile driving. The final pile length was listed in Table 1.

Table 1 The general piles information of pier S3~S12

Pier No.	Pile length (From pile top) [m]		
	Steel tubular pile	**Concrete part**	**Total length**
S3	33,6~39,6	38,6~44,6	38,6~44,6
S4	72,1~77,1	78,1~83,1	78,1~83,1
S5	115,7~117	42,6	115,7~117
S6	122,6~126,6	42,6	122,6~126,6
S7	128,1	42,6	128,1
S8	128,4	42,6	128,4
S9	118,5~122	42,6	118,5~122
S10	83,3~84,6	89,3~90,6	89,3~90,6
S11	73,1	75,7	75,7
S12	49,3~53,3	56,3~61,3	56,3~61,3

In the main design, the over 80-meter-long piles were suggested to be driven in two parts, which had to be welded on site. It would bring high risk of quality at the connection of the two parts as the execution class of steel structure was the highest class EXC4 B+ referring to EN 1090-2 [2]. The Contractor evaluated the risks and their own pile driven capacity, then decided to drive each steel pile in one time. The whole extra-long piles were manufactured in workshop and transported to the site. Due to the wall thickness were 40mm and 60mm, the straight seam pipe processing technology were adopted finally. The raw materials were followed EN 10025 [3], and fabrication and quality control were in accordance with EN 10219-2 [4].

3.1.1 Pile head

The pile heads were extended into the pile cap for 2,6 m. For better connection between steel tubular pile and pile cap, external and internal shear rings were set and welded at the pile head. Each shear ring was separated into three elements with 50~75mm gap between each other for the convenience of processing.

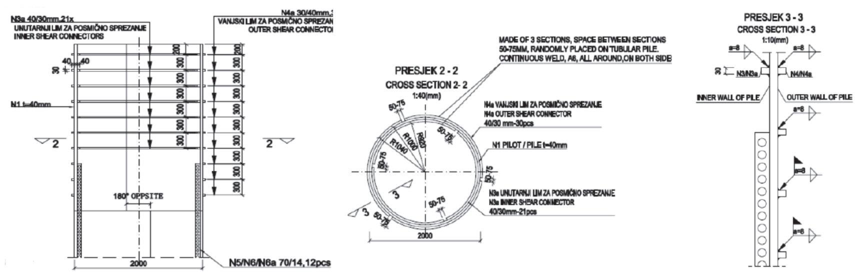


Figure 2 Details of pile head

3.1.2 Pile toe

Due to different bearing types and diameters of piles, there were three types of pile toes according to the main design. Type A and Type C were used for concrete-socket piles and Type B, which was strengthened by vertical stiffeners, was for end bearing piles. Chamfer of slope 4:1 was adopted at each pile toe for better penetration into rocks.

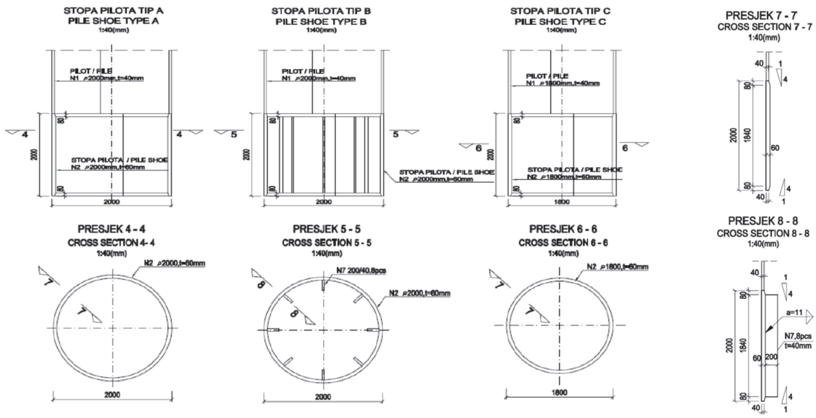


Figure 3 Details of pile toes

3.1.3 Connections between steel plates of pile body

The welding connection between steel plates of pile body were referred to EN 1993-2: 2006 [5] and EN1090-2 [2]. The butt welds of circumferential seams were adopted to the connection of pile segments. The longitudinal seams of all the standard segments on one pile body were totally staggered with the joints on adjacent segments, in order to ensure the reliable mechanical performance.

3.1.4 Pile driving

The output energy of driven hammer with 600 KJ was recommended in the main design. However, in the stages of detail design and construction, the Contractor finally adopted the hydraulic hammer with much higher kinetic energy (E_k) 800 KJ to improve the driving efficiency. Moreover, it could reduce the high risk for the construction, which the real measured skin resistance of soil and rock around the piles were underestimated just in case. The pile driving procedure and stoppage criteria were prepared by professional engineers. The stoppage criteria combined the maximum blow counts of certain penetration at required energy and allowable stress were shown below:

- Penetration ≥1500 blows / 1500 mm at $E_k \geq 650$ KJ
- Penetration ≥ 650 blows / 250 mm at $E_k \geq 700$ KJ
- Penetration ≥ 650 blows / 10 cm at $E_k \geq 700$ KJ
- Penetration ≥ 170 blows / 5 cm at $E_k \geq 700$ KJ
- The allowable stress at pile toe position was limited up to $0{,}9f_y/2 = (0{,}9 \cdot 460)/2 = 207$ MPa. (steel grade of pile toe S460)

3.2 Pile caps

In the main design, concrete cofferdams were used for the construction of pile caps. The connection between cofferdam and the steel tubular pile was below the cofferdam which will increase the construction risks. For minimizing the underwater construction, the monolithic precast concrete cofferdams were modified to the cofferdam composed by steel walls and precast concrete bottom slabs. The new solution could also reduce the risks of lifting irregular shape of cofferdam elements.

3.2.1 Cofferdam of pile cap

The bottom slabs of cofferdam were designed to prefabricate on precast yard beside the bridge. For S3, 4, 11 and 12, they were casted in one time. For S5-10, they were consisted of three parts and connected by post casted joints. See Fig.4.

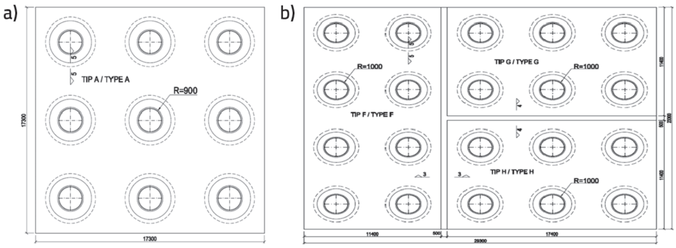


Figure 4 Configuration and joint detail of concrete bottom slab

The bottom slabs of cofferdam were lifted to the pier position above pile top and the joint parts (Fig. 4) were constructed. Then the steel walls and stiffen parts were installed and the whole cofferdam were moved down to the design level by hydraulic decentralized system. The gaps between steel piles and cofferdams were filled by sealing concrete subsequently. When the sea water inside cofferdam was pumped out completely, the pile caps could be constructed.

3.2.2 Pile caps reinforcements

Pile caps of No. S5-S10 were casted in 3 layers, S3-S4 and S11-S12 in 2 layers.

• Reinforcements arrangement

Due to the high seismic intensity, the reinforcement ratio of pile caps was rather high. Take S7 as example, 3-6 layers reinforcements D40 at bottom and 3-4 layers reinforcements D28 at top of each direction were adopted. The reinforcement was lengthened by couplers (D > 28 mm) or overlapping (D ≤ 28 mm). Due to the concrete pouring stratification, the typical stirrups were divided into three overlapped parts as shown below for the convenience of placing main reinforcements. (Fig. 5)

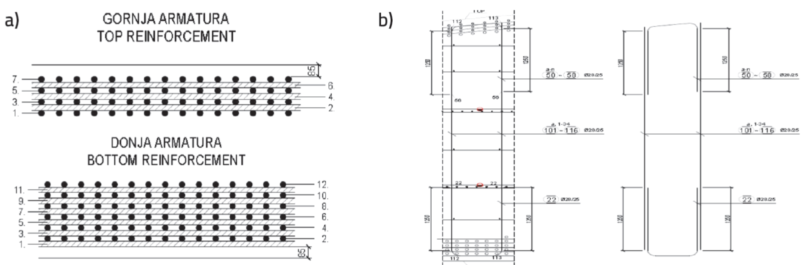


Figure 5 Arrangement of reinforcements

• Pier reinforcements embedded into pile caps

The pier reinforcements were embedded into pile caps for transferring loads from the superstructures. The embedded reinforcement bars were supported at the top of the first casting layer of pile caps. If there were conflictions between pier reinforcements and the piles, the reinforcements could be cut and welded on steel tubular piles or supported on the top surface of concrete piles.

4 Piers and pylons

The piers and pylons were both constructed by hydraulic climbing formwork system. In the main design, the segment length of piers and pylons were both 4m. In order to improve the construction efficiency, the pier segment length was adjusted to 4,5m, but the pylon segment length was kept as 4m because of the position of anchor links on them. Most of the main reinforcements from pylons were directly connected with the reinforcements from piers in concrete part of main girder. The reinforcements type D40/D50@150 mm were used for pylons, as well as D32/D40@150 mm for piers.

5 Stay cable system

There were 60 stay cables in single cable plane, and individual cable consisted of 55, 73, 91 or 109 galvanized strands of Y1860 S7-16.0. The upper pylon was 40m high above deck level. The cables were connected between deck anchorages, which were located at horizontal distances of between 22.5 m and 130.5 m from the piers, and the link systems in the pylons, which were located at between 19.25 m and 37.25 m above deck level (Fig. 6 a). The cable length was between 33 m and 137 m. The limited strength $0.55f_{uk} = 1023$ MPa was at construction stage, $0.45f_{uk} = 837$ MPa for SLS and $0.67f_{uk} = 1240$ MPa for ULS at operational stage.

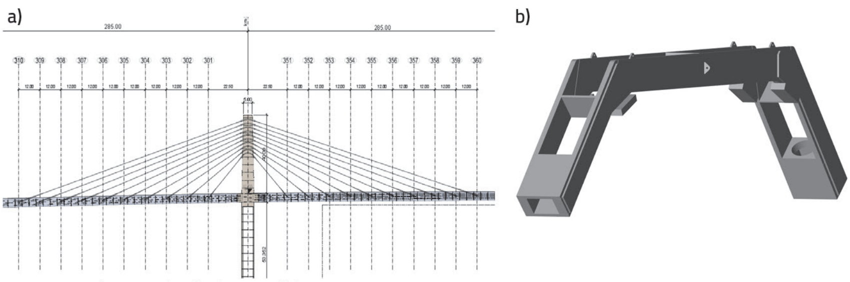


Figure 6 Layout of stay cables: a) Layout of stay cables, b) Link anchor box on pylons

For fulfilling requirements of main design, the VSL SSI 2000 Stay Cable System were adopted. The link anchor systems were made up of S355 steel plates and S235 + C450 shear studs, used as the anchors on pylons (Fig. 6 b).

6 Steel superstructure

6.1 General description

The steel superstructure was a three-cell and 4,5m-high orthotropic box girder. The top and bottom plates were 22,5 m (including the wind fairing) and 8,1 m wide respectively. The middle cell was 3,0 m wide for main spans (Fig.7) and 8,0 m wide for approach spans. The inclined bottom slab was 24° towards horizontal. The base segment at the pylon position will be casted in-situ, which was rigid connected with the pylons, connected with the steel box girder by shear studs. The base segments were strengthened in the longitudinal direction by post-tensioned strands.

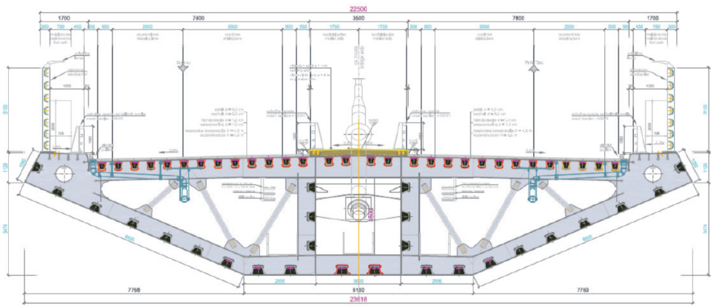


Figure 7 Standard cross section with stay cables

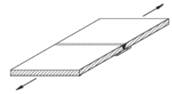
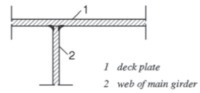
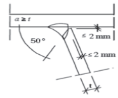
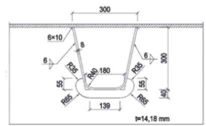
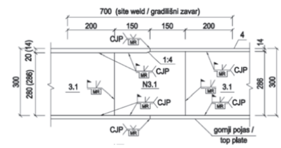
6.2 Erection method

The 165 steel superstructure segments would be erected by several erection methods. The standard 12 m-long segments of main span would be installed by derrick crane, and the segments of side spans on land would be lifted by floating crane and slipped forward to the final position by jacks on temporary steel frames. The segments of side spans at sea were 36~56 m long, which would be erected by floating crane with the lifting capacity of 1000 tons and assembled by cradles.

6.3 Overall calculation and welds design

Some minor modification of thickness of steel plates was adopted in detail design to improve the welds quality by using automatic welding sufficiently. During the construction stages, the maximum tensile stresses were 92 MPa and 80 MPa on the top and bottom plates respectively, and the corresponding minimum compressive stresses were -64 MPa and -112 MPa. And during the operational stage, the corresponding maximum tensile stresses were 247 MPa and 250 MPa, as well as the minimum compressive stresses were -158 MPa and -282 MPa respectively. Based on the calculation, all the welds types of girder were defined referring to EN 1993-2 [5] shown in Table 2.

Table 2 The main welding connections on steel box girder

Position	Welds type	Legend
Connection between deck plates / webs / bottom slabs	Single and Double V-welds with ceramic or metallic backing strip (CJP)	
Connection between deck plates and webs / bottom slabs and webs / cross frame and longitudinal slab	Twin fillet welds	
Stiffeners to the deck plates / webs / bottom slabs	T-Butt welds (partial penetration)	
Stiffeners to the web of cross frame	Fillet welds	
Continuity of the stiffeners	Butt welds	

6.4 Details of temporary matching connection

The erected segments could be temporarily positioned by site temporary matching elements, which located at the edges of segments, so as to control the fit-up gaps of the adjacent segments. The temporary - matching elements could be installed only if they meet accuracy requirements of the assembly test. When the connection of the adjacent segments has been welded totally on site, those elements could be removed. These elements were divided into 3 types, in which type D1 and D3 were connected with D40 rib bar and type D2 means connection with welded steel plates. These elements could also bear certain construction forces including “Bura” wind.

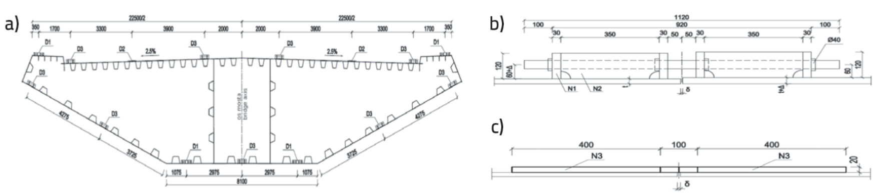


Figure 8 The configuration of site temporary matching on steel box girder: a) Arrangement of site temprary matching, b) Details of D1, D3, c) Details of D2

7 Conclusion

The detail design connected the main design and construction works. The designers of detail design must grasp the main theme of the main design and accurately refine the design documents for the construction on site. The designers of detail design communicated closely with the stakeholders mentioned in the first paragraph for obtaining ultimate consistent for the execution on site. Most of the detail design has been completed so far and the construction work is ongoing. Due to the great efforts of all stakeholders, the project is on the right way to completion.

References

[1] Pipenbaher, M., Radić, J.: Pelješac bridge from km 2 + 120 to km 4 + 560 – Construction Part, Final design

[2] EN 1090-2: 2008+A1 2011: Execution of steel structures and aluminum structures – Part 2: Technical requirements for steel structures

[3] EN 10025: 2005: Hot rolled products of structural steels

[4] EN 10219 - 2: 2006: Cold formed welded structural hollow sections of non-alloy and fine grain steels – Part 2: Tolerances, dimensions and sectional properties

[5] EN 1993 - 2: 2006: Design of steel structure – Part 2: Steel bridge

3
TRAFFIC: PLANNING AND MODELLING

POPULATION SYNTHESIS IN ACTIVITY-BASED TRAVEL DEMAND

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Abstract

A Synthetic Population is first part of creating travel demand model by using activity-based approach. Population synthesis is application of algorithms that expanded representative samples of people or household with characterises (such as gender, car ownership, age or ethnicity etc.) to entire area of researching. Because of complexity of people decisions before or during travel, one attribute is not enough to fully describe what factors have impact on them. Population synthesis iterate a set of attributes for each person in the sample and after expansion and assigning weights create simulated people or household with their characteristic. Basic components are marginal distribution targets of household and person attributes, household and person samples and algorithm for selecting the sample records into a synthetic population such that the attributes of that population match the marginal targets. Goal of this paper is to present population synthesis and her importance for activity-based approach in travel demand modelling. The paper will consist of introduction, literature overview, presenting benefits and complexity of population synthesis, discussion and conclusion.

Keywords: representative samples, set of attributes, marginal distribution, algorithm, marginal targets

1 Introduction

Activity-based concept present new way to handle predication of travel demand. Creating travel demand model is detecting relevant total of trips which occur at some specific area (for example large, middle or small city, various urban areas, rural areas or similar). Prediction of travel demand is determing total future trips with real possibility that will occur at some specific area. Most used method for travel demand modelling is four-step aggregate model. But that model has some limitations regarding to different traffic solutions. Influence of solutions that effect behaviour of people rather that infrastructure measures are hard to predict. That problem is trying to solve by activity-based approach. Activity-based approach is third and newest generation of travel demand model.
Main different between this to types of travel demand models, is level of aggregation and focus on trips or activities. Activity-based approach states that trips and mobility are sequence of some specific activity. Also, activity-based approach is focused on single household or people not zone. Product of activity-based approach is O-D matrix of single modes that are modelled. O-D matrix between zones are created by two separate steps. First steps are population synthesis and second is long-term, middle-term and short-term decisions. Population synthesis is method to create simulated population at some area by expanding relevant sample. Creating simulated population is achieving by various algorithms. In this paper will be presented basic definition of population synthesis and her role in for activity-based approach.

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2 Literature overview

Various authors in the past try to detect best and most applicable methodology for creating travel demand at some area. Activity-based approach in the current literature shows various advantages regarding to traditional four-step travel demand approach. But also, that advantages are detected primarily in urban areas where is more and more important how to affect on human behaviour in transport, rather than building new infrastructure or making big and complex interventions in space. First step of conducting activity-based approach is to collect data. Two type of Data is collected: relevant sample (based on number of all population at specific area) and marginal targets.
After collecting data, comes first step of activity-based approach, called population synthesis. Methods for creating baseline synthetic populations of households and persons using 1990 census data are given [1]. By using Public Use Microdata Sample (PUMS), and Iterative Proportional Fitting (IPF) authors are estimating the proportion of households in a block group or census tract with a desired combination of demographics. In paper is estimate the proportion of households in a block group or census tract with a desired combination of demographics [1]. New activity-based models require more detailed information on household demographics and employment characteristics [2]. Using Monte Carlo microsimulation these models aim at reproducing human behaviour at the individual level, i.e. how individuals choose between options following their perceptions, preferences and habits subject to constraints, such as uncertainty, lack of information, and limits in time and money [2].
Because of certain problems regarding to data availability, these kinds of approach must be generated that represents individuals in the form of households and household members [2]. While most existing procedures concentrate on iterative proportional fitting (IFP), this paper show combing different approaches [2]. Until this date, the conventional approach to synthesizing the base-year population has been based on the iterative proportional fitting procedure [3]. Paper show a new population synthesis procedure is presented that addresses the limitations of the conventional approach. Also, validation results indicate that the new procedure can produce a synthetic population that more closely represents the real population size than the conventional approach [3].
In agent-based microsimulation models for land use, the initial step is the definition of agents – usually, persons and households [4]. Authors in this paper are trying to summarize recent efforts to population synthesis for microsimulation. Authors also stated in past papers from various authors that they share two principle: adjustment of an initial population, taken from a past census or other survey data, to current constraints, and selecting households and optionally assigning them to geographic areas [4].
Goal of paper was to describe, analyse and evaluate the characteristics of the all mentioned approaches. Authors in paper [5] state that previous research have already developed a few techniques for generating a synthetic population: iterative proportional fitting and combinatorial optimization. This paper provides a guideline for using the synthetic population techniques by introducing terminologies, related research, and giving an account for the working process to create a synthetic population [5]. A method based on iterative proportional fitting (IPF) is developed for generating synthetic populations for the application of Albatross, a rule-based and activity-based model of travel demand [6].
A method is proposed to generate synthetic households based on data on distributions of individuals [6]. This method uses the concept of relation matrices to convert distributions of individuals to distributions of households in a pre-processing step [6]. Furthermore, a method is proposed to address differences in populations that relate to locational characteristics [6]. A crucial step in developing agent-based models is the definition of agents, e.g., household and persons [7]. This paper lists the most prominent techniques for population synthesis: iterative proportional fitting (IPF), iterative proportional updating (IPU), combinatorial

optimization, Markov-based and fitness-based syntheses, with other emerging approaches [7]. But, until present, authors stated that is no clear godliness for advises how and when use any of the available techniques. Also, authors claim that this paper present a comprehensive synthesis of available examples and literatures [7]. Population synthesis techniques are commonly used as alternative to supplement the lack of availability and completeness of microdata for microsimulation modelling [9]. This paper describes the process of generating a synthetic baseline population for Sydney Greater Metropolitan (GMA) using 2006 Population Census [9]. Those data were used to evaluate their representativeness with aggregated census data [9]. Paper [10] proposed a cross-entropy optimization model in which generalized constraints for different demographic characteristics of the synthetic population could be included. A quasi-Newton algorithm was used to solve the proposed problem [10].
Results from the model show that the proposed method held much promise for generating a more realistic synthetic population with different types of demographic characteristics and could be generally applied in different geographic areas [10]. Paper [11] show a new land use classification method for its explaining travel behaviour and as a new dimension in population synthesis. The method reproduces the four types of land use environments and improves ability to create a finer-grain geographic classification based on land use [11]. Paper also show similar indications about the difference between urban dwellers rural residents [11].
Techniques such as iterative proportional fitting (IPF) have been applied extensively to estimate data for the population, synthetically [12]. Paper proposes a binary linear programming model for tabular rounding in which the integer-converted table totals and marginal sums perfectly fit the input data [12]. The empirical comparison of the proposed method with eight existing methods demonstrates that the proposed model outperforms the tested methods [12]. Paper also show that in this paper, deterministic methods outperform stochastic methods in accuracy and perfect fit to census data [12].
Paper [13] endeavours to develop a synthetic population, based on the Simulated Annealing (SA) algorithm for the activity-based travel demand model. Hill climbing and cooling schedule are essential elements to be considered when applying SA into the synthetic population [13]. Also, Metropolis-Hasting Algorithm was employed to decide whether to select or dismiss the follow-up distribution so that hill climbing phenomenon can be prevented [13]. Based on this result, the current condition of micro sample and census data were utilized to compare the IPF (Iterative Proportional Fitting) of previous methodology with the establishment result of suggested algorithm [13]. Paper proved that the SA algorithm is valid and built with the synthetic population through statistical verification.

3 Population synthesis as basis of activity-based travel demand

Population synthesis is not exclusive to only activity-based models [14]. Synthetic population is used as the basis for forecasting the behaviour of the households and persons in the modelled area [15]. Most of aggregate and disaggregate models have a population that can be used with most of the relevant characteristics available for the base year because of data collected by national household travel survey [14]. That population needs to be synthesised for future years based on the parameters that have been forecast (number of people per zone/district, income, car ownership, etc.) [14].
Attributes, for example: distribution of household sizes, age distribution, school and university attendance, multiple vehicle ownership, etc. need to be estimated, usually at the level of the representative households (for each zone or district) [14]. The first part is to create a synthetic population and then simulate the behaviour of the households and persons in that population [14]. Population synthesis is creating by generating an artificial population by expanding the disaggregate sample data to mirror known aggregate distributions of household and person variables of interest [14].

The first input is marginal “control data,” and the second input is “sample data” [15]. The control data represent the attributes that are being explicitly accounted for in the generation of the synthetic population [15]. Control data must be provided at relatively detailed geographic levels. The second data input is sample data. Samples of households and persons to create a list of households and persons that matches distributions from marginal (control data) [15]. The process starts by creating a base year synthetic population from available data while using aggregate demographic and land use forecasts to create a synthetic population of future years.
First step is estimating a demographic distribution of households is for each TAZ or small census area, and then matching sample of households that is chosen from an available data [14]. Second phase is identifying person attributes from within each household [14]. The final output is a synthetic population in which each artificial household and its members have many clearly defined characteristics of interest and together they match the estimated demographic distribution within determined zones.

4 Conclusion

Implementation of population synthesis in travel demand modelling is visible in all of models created by using activity-based approach. Regarding to which methodology was used or creating during the phase of decision-making (which come after population synthesis), synthesis of population was base of that phase.
Population synthesis is created by two set of data: marginal “control data,” and the “sample data”. Purpose of marginal data is determing real distributions of data attributes on fine geographic level. Sample data is representing number of households and persons in population on specific geographic area. Combing two set of data, it is possible to create artificial population (which match real number of people in area) of some area by matching various set of attributes (age distribution, incomes, gender, etc.).
Population synthesis is first step of creating activity-based travel demand model after collecting data. All mentioned papers with their authors try to develop or detected best methodology or procedure to create population synthetic. Docent of methodology were detected or created. Most used principle is IPF (Iterative Proportional Fitting), while others are in phase of researching. Proposed methodologies for population synthesis have their advantages and disadvantages, but in total all of them need to be more researched to detect which is most suitable for what situation.

References

[1] Beckman, J.R., Baggerly, K.A., McKay, M.D.: Creating synthetic baseline populations, Transportation Research Part A: Policy and Practice, 30, pp. 415-429, 1996.

[2] Moeckel, R., Spiekermann, K., Wegener, M.: Creating a Synthetic Population, 8th International Conference on Computers in Urban Planning and Urban Management (CUPUM), pp. 1-18, Sendai, Japan, May 2003.

[3] Guo, J.Y., Bhat, C.R.: Population Synthesis for Microstimulating Travel Behaviour, Transportation Research Record Journal of the Transportation Research Board, pp. 92-101, 2014.

[4] Muller, K., Axhausen, K.W.: Population synthesis for microsimulation: State of the art, STRC 2010, Swiss Federal Institute of Technology Zurich, Zurich, Switzerland, 2010.

[5] Cho, S., Bellemans, T., Creemers, L., Knapen, L., Janssens, D., Wets, G.: Synthetic Population Techniques in Activity-Based Research, Data Science and Simulation in Transportation Research, pp. 1-23, 2014.

[6] Arentze, T., Timmermans, H., Horman, F.: Creating Synthetic Household Populations: Problems and Approach, Transportation Research Record Journal of the Transportation Research Board, pp. 85-91, 2014.

[7] Ramadan, E.O., Sisiopiku, V.P.: A Critical Review on Population Synthesis for Activity- and Agent-Based Transportation Models, Transportation Systems Analysis and Assessment, InTech, Ljubljana, 2019.

[8] Gargeet, D.: Population synthesis for travel demand forecasting, 2013.

[9] Siripirote, T., Sumalle, A., Ho, W.H.: A Statistical Synthetic Population Calibration for Activity-Based Model with Incomplete Census Data, Journal of the Eastern Asia Society for Transportation Studies, 10 (2013), pp. 742-761

[10] Lee, D.H., Fu, Y.: Cross-Entropy Optimization Model for Population Synthesis in Activity-Based Micro-simulation Models, Transportation Research Record: Journal of the Transportation Research Board, pp. 20-27, 2011.

[11] McBridem E.C., Davis, A., Goulias, K.: A Spatial Latent Profile Analysis to Classify Land Uses for Population Synthesis Methods in Travel Demand Forecasting, Transportation Research Record Journal of the Transportation Research Board, 49 (2018), pp. 158-170

[12] Choupani, A.A., Mamdooh, A.R.: Population Synthesis in Activity-Based Models: Tabular Rounding in Iterative Proportional Fitting, Transportation Research Record: Journal of the Transportation Research Board, 17 (2015), pp. 223-233

[13] Kim, J., Lee, S.: A simulated annealing algorithm for the creation of synthetic population in activity-based travel demand model, KSCE Journal of Civil Engineering, 20 (2016), pp. 2513-2523

[14] Ortuzar, J.D., Willumsen, L.G.: Modelling transport, 4th edition, Wiley, 2011.

[15] Castiglione, J., Bradley, M., Gliebe, J.: Activity-Based Travel Demand Models: A Primer, The second strategic highway research program, Transportation research board, 2015.

[16] Rich, J., Flotterod, G., Garrido, S., Pereira, F.: Review of population synthesis methodologies, Swedish National Road and Transport Research Institute, 2019.

DEVELOPMENT AND EVALUATION OF ANALYTICAL FORECASTING METHODS FOR VEHICLE OCCUPANCY, IN THE TRUNK ROUTES OF MOSCOW

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Abstract

Passenger transport plays an important role in large cities around the world. According with this, the administration of these cities pays much attention to urban transport, specifically, in the formalization and establishment of standard criteria for the quality of transportation. The main criteria are the "Regularity of transportation" and "Vehicle occupancy". Having standards is the first step to improve the quality of the transport service, the next step is the execution and application of these standards. In this article, the issue of improving the quality of passenger transport services is considered based on the forecast and control of the vehicle occupancy and the management of the transportation process based on the results of the forecast. The necessary condition for the implementation of these tasks is the use of telematic means as the Automated Passenger Counting System for monitoring passenger flow at bus stops. This information is the source data. In the article, we show the results of the study of the trunk routes of urban transport in Moscow. It is shown that the situation regarding passenger dynamics differs every time for each route, which requires different analytical methods for forecasting and control of vehicle occupancy for each case. Various methods for predicting vehicle occupancy were developed, and it is shown that the effectiveness of these forecasting methods depends directly on the characteristics of passenger flow. The control of the flow of passengers based on the established standards will avoid overloading the buses that circulate on the main roads of large cities, thus improving the quality of the transport service on the route.

Keywords: public transport, ITS, vehicle occupancy, passenger traffic, telematics

1 Introduction

The difficult transport and environmental situation in large cities and megacities, caused by the uncontrolled growth of motorization of the population, forces the administrations of these cities to look for alternative solutions that ensure high mobility of the population, which directly affects the economic situation. One of the most effective ways is to develop a system of public transport, [1,2]. It is possible to achieve a significant increase in the share of public transport in the overall structure of passenger transport by providing significant advantages for public transport only.

One of these advantages is a significant increase in the average speed of transportation, which is achieved by organizing public transport on main routes on dedicated lines and with a small number of stops.

The second important advantage to be achieved is the comfort of the ride. The comfort of the ride depends partially on the air temperature in the cabin. Manufacturers of vehicles have

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begun to produce city buses with air conditioning, which provides a normal temperature inside saloon at any time of the day at any season of the year. Since the trip time is considered by many citizens as wasted time, buses have started to organize access to mobile Internet, which allows most passengers to spend their travel time usefully. However, in this case the most significant factor affecting the level of service is occupancy of vehicle. In this regard, all the leading countries of the world have developed regulations on the level of service, which is evaluated by the vehicle occupancy.

In the United States, level of service is classified, depending on the vehicle occupancy. The main indicator is space (square meters) per one standing passenger [3, 4]. Vehicle occupancy is classified into six levels of service. The level of service "A" is provided when all passengers are sitting. This level of service is considered as the highest. Indeed, since passengers pass while sitting, these conditions are similar to those of a ride in a private car. At the same time, passengers of public transport on main routes are guaranteed not to get into a traffic jam during rush hour and will be transported at a significantly higher speed than passengers of private cars. Thus, in the future, the majority of passengers on main routes should be transported sitting and at a high speed. However, modern urban passenger transport vehicles are structurally designed for the situation when some passengers will pass standing. Therefore, the issue of maximum permissible vehicle occupancy is both economic, technical and of great social significance.

In Russia, the levels of service are also regulated, depending on the number of passengers per one square meter of cabin area. The level of service "A" is provided when all passengers pass sitting. The maximum permissible vehicle occupancy is 5 people per one square meter of the interior area. As shown in table 1, this corresponds to level of service "E" [5]. This indicator of the maximum permissible passenger load is approved by the regulations of the Ministry of transport of the Russian Federation transport [6].

Table 1 Levels of service for public transport in Russia [5]

Level of service	**Number of passengers per 1 sq. m inside the bus**	**Seats occupied [%]**
A	0	Up to 100
B	Not more than 1	100
C	1-3	100
D	3-4	100
E	4-5	100
F	More than 5	100

However, it is not enough to set level of service indicator, depending on the vehicle occupancy. It also needs to be controlled. It can be possible using the real-time data on the vehicle occupancy received by the public transportation dispatch system. The system needs these data to prognoses maximum occupancy value during the trip, and to prevent overloading (compared with permissible load) if necessary.

This article discusses the development of analytical methods for predicting vehicle occupancy on main routes, using statistics of Moscow public transport routes.

2 Calculation of vehicle occupancy according to the number of incoming and outgoing passengers at stops and assessment of the level of service

The method of forecasting for vehicle occupancy, considered in this article, is based on the assumption that each vehicle on the route is equipped with equipment for counting boarding and alighting passengers at stops, and the data transmitted to the dispatch center via mobile communication. In this case, vehicle occupancy ($c_i(t_{ik})$) in the vehicle performing the k-th trip and leaving the i-th stop of the route at the time of t_{ir} can be calculated using the formula [8]:

$$c_i(t_{ik}) = \sum_{j=1}^{i} b_j(t_{jk}) - \sum_{j=1}^{i} a_j(t_{jk}) \tag{1}$$

where, b_j (t_{jk})– quantity of passengers, boarding the vehicle at j-th stop of k-th trip at time t_{jk}; a_j (t_{jk}) – quantity of passengers, alighting the vehicle at j-th stop of k-th trip at time t_{jk}.
Let's consider as an example the city bus LiAZ-5292, the most common on the routes of Moscow. The bus has 28 seats. The maximum capacity is 108 passengers at the maximum load rate of 5 people per one square meter. According to table 1, this corresponds to level of service "E". Therefore, the maximum number of passengers passing standing at this level of service is 108-28 = 80 passengers. The floor area of the cabin (S_s) is equal to:

$$S_s = \frac{80 passengers}{5 passengers / 1 meter^2} = 16 meters^2 \tag{2}$$

Based on the data obtained, it is possible to calculate the number of passengers inside the compartment of the LiAZ-5292 bus for various levels of service according to [5], as shown in table 2.

Table 2 Number of passengers in the LiAZ-5292 bus compartment, corresponding to different levels of service

№	Level of service	The number of passengers in the bus	Number of standing passengers	Percentage of standing passengers [%]
1	A	Not more than 28	0	
2	B	Not more than 44	From 1 to 16	From 3.5 to 36 %
3	C	From 45 to 76	From 17 to 48	From 37 to 63 %
4	D	From 77 to 93	From 49 to 65	From 63.6 to 70 %
5	E	From 94 to 108	From 66 to 80	From 70.2 to74 %
6	F	More than 108	More than 80	More than 74 %

Real number of passengers in the bus can be compared with the qualified number, established in the standard for assessing the level of service provided on each stage of the trip. Using the length of each stage of the route, we can calculate the actual volume (V_{qi}) of transport work, performed with a certain level of service at a stage i:

$$V_{qi} = N_i LOS_i L_i \tag{3}$$

where, Ni - number of passengers at i-th stage of the route (after i-th stop); LOSi - level of service at i-th stage of the route, estimated using real vehicle occupancy at this stage; length of i-th stage of the route (kilometers).
Similar results can be obtained for each trip, performed for the route during the operational day.

3 Experimental research and selection of a method for predicting vehicle occupancy based on actual data of passenger occupancy on the bus route M10 "Kitai Gorod - Lobnenskaya ulitza"

As an object of experimental research the bus route number M10 " Kitai Gorod - Lobnenskaya ulitza" was chosen. This route is served by the "Mosgortrans" state enterprise, which uses LiAZ-5292 buses for the route. This bus model was described in section 1 of the article. The reason for choosing this route was that the route is a typical main trunk route, that runs from the center of Moscow to the Northern outskirts of the city. The time interval for this route during rush hour is 7-8 minutes. The tasks of experimental research were:

1) Evaluating the level of service on each stage based on actual data of the passenger compartment occupation;
2) Determining the busiest stop on the route based on the occupation analysis results;
3) Analysis of the possibility of various methods usage for predicting vehicle at a critical (the most occupied) stop of the route and selection of a forecasting method that allows dispatching system to make decisions on changing the mode of movement of vehicles on the route in order to prevent passenger overflow.

We collected and processed data on the number of boarding and alighting passengers at stops on the M10 route for several days, including weekdays and weekends. Calculations were made for the vehicle occupancy at each stage using the formula (1). An example of a time series that reflects the dynamics of the passenger occupation range of the M10 buses during the operational day January 5, 2020 at the critical stop «Rogachevskaya ulitza» is shown in fig. 1.

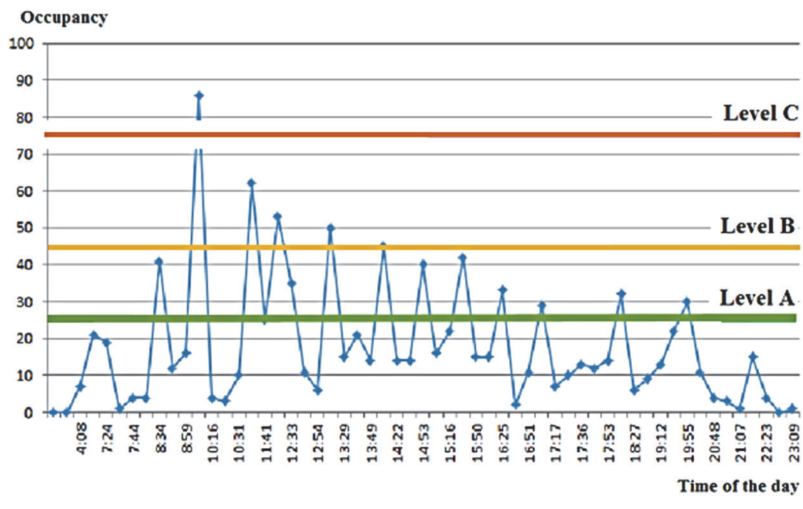

Figure 1 Example of the dynamics of the vehicle occupancy of the M10 bus route during the day at critical stop "Rogachevskaya ulitza" for January 5, 2020

The analysis of data on the vehicle occupancy showed that the highest congestion occurs after the stop "Rogozhskaya ulitza". In the article [7], such a stop is proposed to be called "critical". Bold type horizontal lines mark the boundaries of the various levels of service that are evaluated based on the vehicle occupancy of LiAZ-5292 bus.

Since the data for January 5, 2020 is shown for the most loaded bus stop, we can say the following:

- Morning trips (before 8:00) and evening trips (after 20:00) were performed with the service level “A”;
- The most part of the days’ time trips were operated with service level “ B “and only a few trips operated a level “C” at one segment of the route. Therefore, we can conclude that the level of service on the M10 route was high during most of the day January 05, 2020.

Let’s evaluate the data of the constructed time series, shown at figure 1, for the presence of outliers according to the Irwin criterion [8] for data suspected of being anomalous, the value of λ_t is calculated using the formula (4):

$$\lambda_t = \frac{| y_t - y_{t-1} |}{S_e} \tag{4}$$

where:

$$S_y = \sqrt{\frac{\sum_{t=1}^{n} \left(y_t - \bar{y} \right)^2}{(n-1)}}, \quad \bar{y} = \frac{\sum_{t=1}^{n} y_t}{n}$$

y_i - meaning of i level of time series.

Calculated standard deviation S_y= 14.4; average value =11

For our sample size (n=54), the value of the Irwin criteria is 1.1. Analysis of the data of the considered series showed that 14 of its values are anomalous according to the Irwin criterion. In our case, this indicates a high variability of the series levels. Therefore, forecasting, using a built-up trend [9] can lead to large errors. The exponential forecasting method [9], based on the use of actual data on vehicle occupancy at a critical stop, based on the forecast of the previous step and actual data on vehicle occupancy in the current trip, will also give significant errors. In this regard, we proposed to consider the trend of vehicle occupancy in each trip up to the critical stop. This section of the time series is sufficient if the task is to regulate interval of movement on the route depending on the forecast results in order to ensure the required level of service. Figure 2 shows the graphs of vehicle occupancy changes on the route stops of the trip, performed during the considered day on January 5, 2020. The horizontal axis shows the route stops in the order in which they occur on the route. The vertical axis indicates the number of passengers in the passenger compartment after each stop. Each polyline simulates the vehicle occupancy changes during the each trip during the day. Formally, this model can be described as random process realizations during each trip. The state of the process changes at each bus stop. In this figure, one can see that the occupancy of first few stops of each trip form a certain trend of the vehicle occupancy, which persists until the critical stop of the route. The presence of a the trend allows us to put forward a hypothesis that we can use the so-called “naive model” for predicting the vehicle occupancy at the next stop, which is used for predicting time series [9,10], and in which the forecast of vehicle occupancy at the i-th stop (O_i) is determined from the ratio:

$$O_i = O_{i-1} + \beta(O_{i-1} - O_{i-2}) \tag{5}$$

where:

O_{i-1} - fact of occupancy at previous (i-1) stop,
O_{i-2} - fact of occupancy at a stop before previous stop;
β - some correction factor determined empirically.

As one can see from the formula, the forecast is made only for the current flight starting from the third stop on the route.

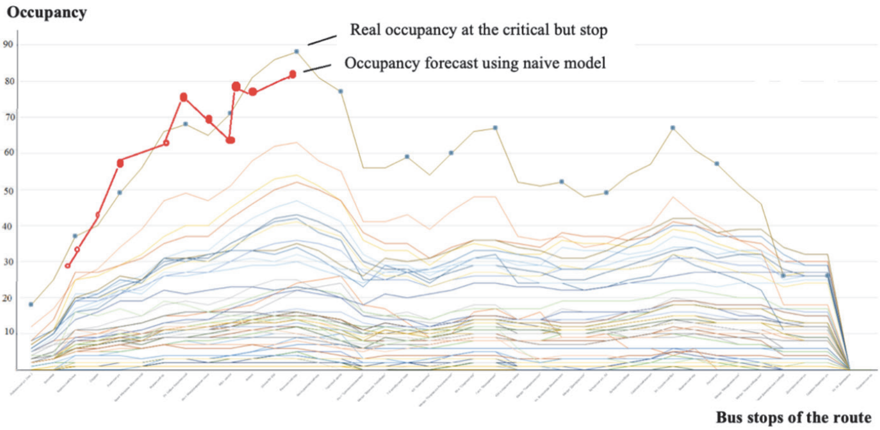

Figure 2 Dynamics of the vehicle occupancy process during the busiest trip

Figure 2 shows the trajectory of the vehicle occupancy process during the busiest trip, which according to the schedule began at 8:59. The red polyline is the vehicle occupancy forecast based on the naive model (4) with the coefficient $\beta=1$. As a result, the model rather accurately predicts the vehicle occupancy up to the critical stop. The error at each step is several passengers only, which is not important for large and articulated buses. However, as shown in figure 1, the process may follow a completely different trajectory on the next trip.

4 Conclusion

Analysis of data on passenger flows obtained by instrumental means on one of the trunk bus routes in Moscow shows that the passenger flow is unstable during "rush hours". This fact does not allow us to effectively use the classical methods, used in forecasting time series, for vehicle occupancy. At the same time, it is shown that the vehicle occupancy in a particular trip to the critical stop has a well-defined trend, which can be described by a linear trend. However, you can only use this information for a specific trip. Already on the next trip, the dynamics in the process of the vehicle occupancy may have a different trend.

References

[1] Bogumil, V., Efimenko, D.: Urban Transport Dispatch Control System Helps to Increase Intelligent Transport Systems Effectiveness, 11th European transport congress – ETC 2013, pp. 20-25, Prague, 2013.

[2] Vuchic, V., Wilew, J.: Urban Transit Systems and Technology, First edition, 2007.

[3] Transit Capacity and Quality of Service Manual, The national academies of Sciences, Engineering, and Medicine, Third edition, Washington, 2013.

[4] Highway capacity manual, Transportation Research Board of the National Academy of Sciences, Washington, 2010.

[5] ГОСТ Р 54723-2019. Глобальная навигационная спут-никовая система. Система управления городским пассажирским транспортом комплексная. Назначение, состав и характеристики решаемых задач подсистемы анализа пассажиропото-ков, Interstate Russian Standard

[6] Распоряжение Министерства Транс-порта России. от 31.01.2017 N НА-19-р (ред. от 13.04.2018) “Об утверждении социального стандарта транспортного обслуживания населения при осуществлении перевозок пассажиров и багажа автомобильным транспортом и городским наземным электрическим транспортом”, Order of the Ministry of Transport of Russia

[7] Duque-Sarango, M., Bogumil, V.: Mathematical model for predicting the vehicle occu-pancy for urban passengers transport on the route, International Conference on Advanced Technol-ogies for Communications ATC - 2019, pp. 248-251, Hanoi, Vietnam, 17-19 Oct. 2019.

[8] Irwin, G.: Analysis of stresses and strains near the end of a crack traversing a plate, Third edition, Trans. ASME, 1957.

[9] Degtyareva, N.: Models of analysis and forecasting based on time series, publishing house of JSC A. Miller Library, 2018.

[10] Lukashin, Yu.: Adaptive methods of short-term forecasting of time series: textbook for universities, Finance and statistics, 2003.

APPLICATION OF TRAFFIC SIMULATION MODELS FOR URBAN ROAD NETWORK ANALYSES – CASE STUDIES FROM RIJEKA CITY

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Abstract

The process of road network planning and designing in urban areas can be significantly improved by using microsimulation of traffic models. Traffic microsimulations are used for analyses and estimation of new proposals as well as for the reconstruction of existing infrastructure in order to reach optimum solution for defined problem. In this paper, applications of different analyses approaches are analyzed in two case studies. Both case studies are located in the city of Rijeka but in different parts of the city, in different traffic conditions and in circumstances where different changes in traffic network are planned. In both cases new solutions were tested through VISSIM traffic model and by application of SIDRA Intersection methodology. VISSIM is a stochastic, discrete, micro-simulation model designed for traffic analyses while SIDRA Intersection is a lane-based micro-analytical model. The results proved the suitability as well as advantages and disadvantages of both approaches. The paper contains suggestions for optimal application of selected models regarding different traffic problems.

Keywords: road traffic microsimulation, urban traffic, SIDRA Intersection, VISSIM traffic model

1 Introduction

Models used to analyze the capacity of a transport network or a section of it are generally divided into deterministic models where output is fully determined by the parameter values and the initial conditions, and stochastic models which possess some inherent randomness so the same set of parameter values and initial conditions can lead to different outputs. The application of microsimulation traffic models has been intensified in recent decades as it enables quality analysis and presentation of interventions in the transport network. Particularly interesting is the application of traffic simulations in complex urban conditions. Traffic microsimulations are used in the intersection optimization process at a specific location for traffic analysis of a section of the traffic network where changes occur as a result of planning of new contents, when planning changes in the traffic flow regime [1, 2, 3]. Recent studies related to the use of traffic microsimulation are aimed at the analysis of traffic safety [4] and simulation of traffic conditions in cases when in traffic, together with the standard vehicles, there is also a certain number of autonomous vehicles [5]. The quality of microsimulation results depends on how much detailed and precise are the input data) and on the extent to which they are calibrated to the local traffic conditions in a

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particular area (country, city). Calibration is defined as the process of comparing and minimizing the differences between modeling results and the real data obtained by counting and measuring in a local network [6].
In this paper, the aim is to analyze the application of deterministic and stochastic models of traffic flow on the example of two planned changes in the traffic network of the city of Rijeka. In both cases, the scope of changes in the traffic system is to create more favorable conditions for non-motorized modes of traffic which consequently affect the quality of traffic flow of motor traffic. In both cases, new solutions were tested through the VISSIM model and by the application of the SIDRA Intersection traffic model. VISSIM is a stochastic, discrete, micro-simulation model designed for traffic analysis while SIDRA Intersection is a lane-based micro-analytical model. Microsimulation tools are explained in section 2 and two case studies in section 3. The results implemented the suitability as well as the advantages and disadvantages of both approaches. The suggestions for optimal application of selected models regarding different traffic problems are given in the section Conclusions and recommendations.

2 Microsimulation tools

In this paper two approaches were analyzed– deterministic approach by application of SIDRA Intersection software and stochastic approach by application of VISIM traffic microsimulation model in order to compare results and select a suitable approach.
Sidra Intersection is a program package designed to analyze standard and roundabouts capacity. In addition to traffic and geometry, the program takes into account vehicle characteristics and driver behavior, and in addition allows entering of the pedestrian volume. After selecting the intersection type and entering the geometry data, the program generates a layout of the intersection. The data on vehicle speeds, vehicle lengths and distances maintained by drivers in relation to the vehicle in front, as well as the data about pedestrians on each approach are entered. The output data include the level of service expressed in delay as well as some other parameters (e.g. regarding CO emission etc.).
Vissim microscopic simulation model is one of the program tools in the field of microsimulation of traffic and is used in many countries of the world. In Vissim it is possible to simulate and analyze all traffic forms and modes. The results of Vissim can be displayed in 3D animations, which is a great manner of presenting planned infrastructure measures to the public [7]. Vissim is stochastic, it uses a random seed generator to generate different vehicle encounter scenarios, and uses distributions to simulate the set values of some input parameters (for example, the default speed of each vehicle category is the median of normal speed distribution). On the other hand, the precondition for valid scientific and expert experiment is reproducibility, and accordingly, for a specific value of the random number generator, the VISSIM model gives the same (reproducible) simulation result. In order to reconcile two opposing requirements – modelling of the stochastic nature of the real system and the reproducibility of the experiment, the model is quasi-stochastic in nature. The quasi-stochastic nature of the model enables comparability of results when analyzing variant solutions, because when setting the same initial generator value, of the same step and number of scenarios for each variant solution, we know that we analyzed and compared the same traffic scenarios of vehicle encounters. [3]

3 Case studies – the city of Rijeka

The city of Rijeka has 112.000 inhabitants and it is the most densely populated city in Croatia with average density of around 2600 inh/km^2. Population density as well as built-up density makes every intervention in space or traffic organization very complex. In this paper, two of the possible interventions in traffic organization that can improve traffic system by giving more space to non-motorized traffic are presented and analyzed. Analyzed case studies are:

- Case study 1: Improvement of overall traffic conditions in the city area Pećine – Rijeka
- Case study 2: Implementation of new pedestrian street - Ciottina street in the city center.

3.1 Traffic model for city area Pećine

The residential area of Pećine is located in the eastern part of the city of Rijeka, with an area of 45.6 ha and a population of around 3700 inhabitants. The backbone of this residential area consists of two two-lane one-way roads approximately 2 km long, which are categorized as major city roads and are also state roads (D8): Šetalište 13. divizije Street, extending from west to east (southern corridor) and J.P. Kamova Street (northern corridor) along which the traffic goes in opposite direction. Along both streets there are buildings specific to residential areas - residential houses, schools and kindergartens, as well as approaches to the city's swimming area, which make the pedestrian traffic particularly intensive during the summer. The basic characteristics of the aforementioned streets are shown in Table 1.

Table 1 Basic characteristics of streets in the analyzed area of Pećine

Street	Corridor traffic	Traffic lane [m]	Sidewalk [m]	ADT [veh/day]
Šetalište 13. divizije	Southern One-way	2x3,5 m	1,85-2,00m	5000
J. Polić Kamova	Northern Two-way	2x3,25	0-2,0 m	5000-8000*

* depending on the cross section at which it was measured

In both streets, the problem of lack of parking spaces is evident, and on the southern corridor (Šetalište 13. divizije) part of the vehicles are parked improperly along or on the pavement. There are two main intersections in the zone, standard three-legged intersections, one of which is signalized (Vulkan). At both intersections, there is a satisfactory level of service and somewhat impaired traffic safety conditions. Pedestrian traffic is most intense in the intersection zones.

Based on a detailed analysis of traffic and construction elements and traffic indicators of the zone [8], a proposal for improvement of traffic conditions in the zone was given. The proposed traffic solution for improvement of the conditions in the zone and the conditions for non-motorized traffic modes includes: a reconstruction of both intersections into roundabouts (carried out analysis of suitability according to technical standards), reconstruction of both main streets of the zone into one-lane one-way streets with bicycle lane and longitudinal parking where conditions allow, regulation of the most frequent pedestrian crossings by narrowing the road , organizing additional off-street parking lots. The solutions are shown in Figure 1. The VISSIM traffic model was used to analyze the traffic in the zone, which modelled and verified the entire zone (two road corridors, all local connections and two main intersections - Figure 1) for the existing condition to determine the reliability of the model, and then it was used to verify newly proposed traffic solutions. The data used to develop the model and calculate the capacity were collected in the field, with automatic traffic counts and measurements.

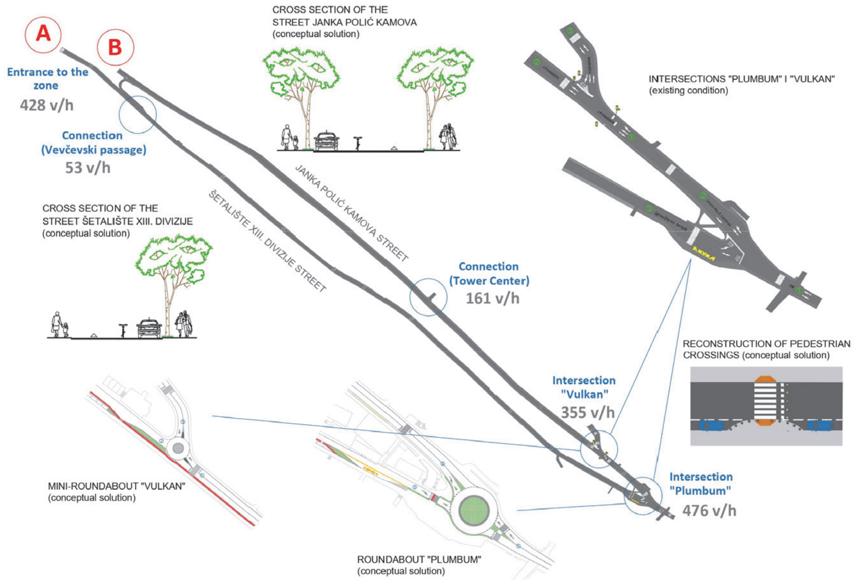

Figure 1 Input data for the VISIM model, with details of the existing and new solutions of the intersections and streets in the zone

Since Vissim is a stochastic model and uses random variables, three iterations were carried out for each solution, and in order to determine the validity of the model, the measured time of passage through the entire zone was calculated (from point A to point B in Fig. 1, total length approx. 3.5 km). The calculated and measured results of the passage are given in Table 2.

Table 2 Travel time for existing and new solution (Vissim) and field measurements

	Present solution			New solution			Field measurement		
Iiteration	1	2	3	1	2	3	1	2	3
Travel time [min]	4:59	5:18	5:23	5:13	4:59	5:07	5:11	4:54	5:30

Below are the traffic density results for the one and two lane variants (existing and new traffic solution) for both corridors.

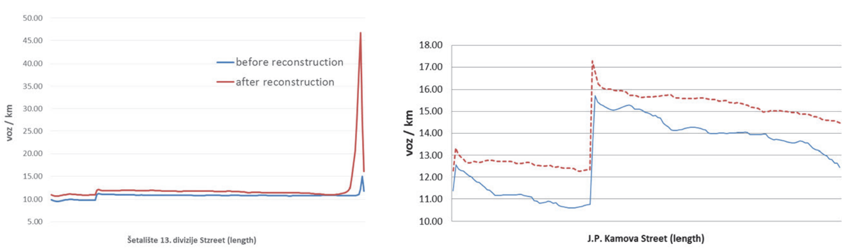

Figure 2 Results - traffic density (veh / km) before and after reconstruction (left-Šetalište 13 divizije, right - J.P. Kamova)

SIDRA Intersection software was used for intersection capacity calculation to analyze the level of service before and after the intersection reconstruction. Table 3 shows a comparison of the service level results obtained by the VISSIM traffic model and the mentioned SIDRA Intersection software, expressed in delay time s. For Visim, the average waiting time is shown according to three iterations, and for SIDRA according to the range that is used in the program for a certain level of service (A to F, or associated time losses). A very high level of compatibility can be observed between the results obtained by the mentioned two models.

Table 3 Delays in traffic flow calculated with SIDRA and VISSIM

	Intersection Plumbum delay (s)				Intersection Vulkan delay (s)			
Approach	**SIDRA (pres/new)**		**VISSIM (pres/new)**		**SIDRA (pres/new)**		**VISSIM (pres/new)**	
3-1	0-10	0-10	17,67	10,31	15-25	10-15	16,15	14,16
3-2	15-25	0-10	15,11	10,34	15-25	10-15	16,21	13,31
1-2	0-10	0-10	A*	7,1	0-10	0-10	10,38	6,68
2-1	0-10	0-10	A*	3,10	-	-	-	-
1-3	-	-	-	-	0-10	0-10	A*	6,82
*main direction – free traffic flow								

3.2 Traffic model for Ciottina Street Area in the center of the city of Rijeka

Ciottina Street is located in the very center of the city of Rijeka, in an area of exceptional built-up density and all-day heavy traffic. Due to the proximity of the Korzo pedestrian zone, interesting catering and shopping facilities in the street, and the proximity of schools, colleges and businesses, Ciottina Street has a very intensive pedestrian traffic and at the same time a very modest pedestrian infrastructure. The traffic lane is 3.5m wide and the pavement width varies from 70 cm to 1.5 m. In order to improve pedestrian conditions and widen the main pedestrian zone of Korzo, it is planned to reconstruct Ciottina Street into a pedestrian street, which would divert the motor traffic to the surrounding streets. The direct route through Ciottina Street from its entrance (point A) to its exit (point B) amounts to 330 m in total, while the bypass route that would take over the traffic load by closing Ciottina Street amounts to 650 m in total and includes driving through the streets of Ivana Pavla II and Erazma Barčića. (Figure 2). A section of Ciottina Street intended for closing of motor traffic is currently a single-lane, one-way street with ADT 4500 veh/day. As during a certain period, in the occasion of the Sustainable Transport Week (September 2018), Ciottina Street was closed for motor traffic, it was possible to collect real traffic load data for the case of an indirect route being activated. A VISSIM microsimulation traffic model was developed to determine the real impact of the additional traffic volumes on the surrounding intersections and the travel time through the zone, based on the data collected by field measurements. The level of service for existing intersections in changed traffic conditions, with increased traffic load, was also verified by SIDRA Intersection software. Figure 2 shows the direct and indirect route area and schematic level of service for key intersections taken from SIDRa Intersection.

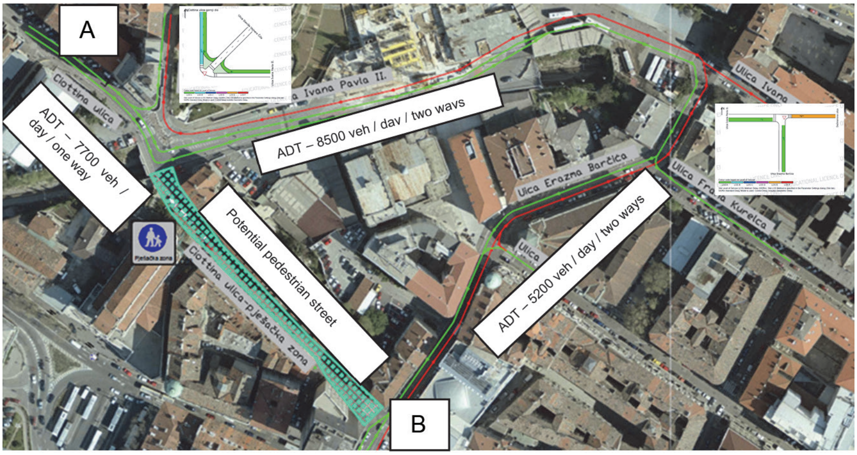

Figure 3 Ciottina Street Area - layout and main traffic inputs and results

The level of service analysis (SIDRA) or time losses (VISSIM) analysis was carried out for main intersections in the area and the results are well matched, in all cases the losses are up to 10 s (both before and after changes in traffic flows), which correspond to the level of service A according to SIDRA [9].

The VISSIM traffic microsimulation model for Ciottina Street and the surrounding area was first based on the VISSIM traffic microsimulation program settings and then calibrated to local conditions using a neural network method [3]. It was shown that the calibrated parameters for traffic conditions in Rijeka - average standstill distance, additive part of desired safety distance and multiplicative part of desired safety distance are significantly different from those in VISSIM settings. Table 4 shows the results of the travel time obtained for the direct and indirect route using the VISSIM model for the default and calibrated parameters as well as those obtained by direct measurement on the road network. The results clearly show that the precondition of the real values of the traffic indicators obtained by modeling is the calibration of the model to local traffic conditions.

Table 4 Travel time for direct and indirect route, model results and field measurements [3]

	Direct route – travel time (s)			Indirect route – travel time (s)		
	measured	VISSIM default	VISSIM after calibration	measured	VISSIM default	VISSIM after calibration
Travel time [s]	72,4	81,95	74,33	100	113,21	100,5

4 Conclusions and recommendation

In this paper the application of deterministic method for traffic flow analyses – SIDRA Intersection and of stochastic traffic model - VISSIM traffic microsimulation model is analysed in two case studies in the city of Rijeka, Croatia. Based on the two presented examples of the application of different methodologies of traffic flow analysis, it can be concluded that the deterministic approach (in this case SIDRA Intersection) sufficiently well describes a simpler traffic situation where, regardless of length, there are no many changes (interruptions) in traffic flows. For complex traffic situations where there is a series of intersections within less than 1 km and intensive pedestrian traffic that interrupts motor traffic flows in many places, a higher quality analysis is obtained by using the stochastic microsimulation models (in this case VISSIM). Both approaches provide the ability to calculate time losses, however using Vissim also allows verifying the travel time in a chosen section. In both cases, the quality of analysis is contributed by the fact that the actually measured input data were used (traffic volume) and in the second case (Ciottina Street) it was possible to test and calibrate them based on the existing and planned situation. The advantage of VISSIM is also the possibility of animated display of what is going on in the network.

Acknowledgement

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References

[1] Cvitanić, D., Breški, D., Ištuk, P.: Application of microsimulation models in planning and design of street network, The 4th Croatian Road Congress, Croatian Road Society – VIA VITA, Cavtat - Dubrovnik, 2007.

[2] Ištoka Otković, I., Zagvozda, M., Šraml, M.: Comparison between modelled and measured traveling time in urban roundabouts, 3rd International Conference on Road and Rail Infrastructure (CETRA 2014), Split, Croatia, 28 - 30 April 2014, pp. 269-274

[3] Ištoka Otković, I., Deluka-Tibljaš, A., Šurdonja, S.: Validation of the calibration methodology of the micro-simulation traffic model, Transportation Research Procedia 45 (2020), pp. 684-691

[4] Guido, G., Giofrè, V. P., Astarita, V., Vitale, A.: Using traffic microsimulation to evaluate potential crashes: some results, IEEE/ACM 23rd International Symposium on Distributed Simulation and Real Time Applications (DS-RT), 2019.

[5] Deluka Tibljaš, A., Giuffrè, T., Surdonja, S., Trubia, S.: Introduction of Autonomous Vehicles: Roundabouts design and safety performance evaluation, Sustainability, 10 (2018) 4, doi:10.3390/su10041060

[6] Ištoka Otković, I., Tollazzi, T., Šraml, M.: Calibration of microsimulation traffic model using neural network approach, Expert systems with applications, 40 (2013), pp. 5965-5974, doi: 10.1016/j.eswa.2013.05.003

[7] Šrami, M., Jovanović, G.: Mikrosimulacije u prometu, Univerza v Mariboru, Fakulteta za gradbeništvo, Maribor, 2014.

[8] Klasić, I.: Primjena mikrosimulacijskog prometnog modela u postupku unaprjeđenja prometnih uvjeta u gradskim naseljima - Primjer naselja Pećine u Rijeci, Master thesis, University of Rijeka Faculty of Civil Engineering, 2019.

[9] Mlinar, M.: Mogućnosti unaprjeđenja odvijanja pješačkog prometa u centru Grada Rijeke, Master thesis, University of Rijeka Faculty of Civil Engineering, 2019.

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THE IMPACT OF DIFFERENT SATURATION HEADWAY VALUES ON INTERSECTION CAPACITY

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Abstract

Elements of the city road network that determine its capacity are signalized intersections. Their capacity depends of many factors: traffic volume and distribution, traffic flow structure, signal timing, and number of bicyclists and pedestrians. However, the starting parameter for calculation of intersection capacity is saturation headway. This research explores the influence of weather conditions and purpose of trip on saturation headway. Saturation headways were determined on few intersections in the morning peak hour of working and weekend day, in good and bad weather conditions. The impact of different trip purposes and different weather conditions on intersection capacity is analysed, as well as the influence of using mean and median values of saturation headway when calculating the intersection capacity.

Keywords:

1 Introduction

Signalized intersections are elements of the city road network that determine its capacity due to recurring interruptions of traffic, numerous vehicle and pedestrian conflicts as well as limited number of lanes. Therefore, it is very important to accurately estimate their capacity. Factors affecting the capacity of signalized intersections are: traffic volume and distribution, traffic flow structure, signal timing, and number of bicyclists and pedestrians. However, the starting parameter for calculation of intersection capacity is saturation headway, which is not a constant value [1-3]. This research deals with dependence of saturation headway on weather condition and purpose of trip. Saturation headways were determined on few intersections in the morning peak hour of working and weekend day, in good and bad weather conditions (light rain). The influence of different trip purposes and different weather conditions on intersection capacity was analysed, in terms of different saturation headways measured. The impact of using the mean and median values of saturation headways when calculating the intersection capacity and delay was also analysed.

2 Saturation headway, start-up lost time and time-in-queue delay

Headways of vehicles that were in the queue from the beginning of the green phase to the appearance of the first non-passenger vehicle have been recorded at each lane. This provided the data needed to calculate the saturation headway and start-up-lost time. In addition, time-in-queue delay for each vehicle (the total time from a vehicle joined the queue to its discharge across the stop lane) has been recorded. The saturation headway h_s is the average value of the measured headways from the fifth to the last vehicle in the queue before the start of the green phase or until the occurrence of a vehicle that is not passenger (truck, bus, ...) [2, 3].

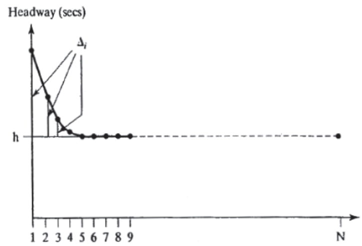

Figure 1 Flow from the queue at signalized intersection [4]

Figure 1 shows that first few vehicles (4) have greater headways because they have to react and accelerate. The additional time above and beyond headway h is defined as start-up lost time:

$$l_1 = \sum_i \Delta_i \tag{1}$$

l_1 – start-up lost time (sec/phase)
Δ_i – incremental headway (above 'h' seconds) for vehicle i

Delay is the most commonly used measure for describing the operation of a signalized intersections. Delay can be determined in many ways such as stopped delay, control delay, time-in-queue delay etc. Time-in-queue delay is the time from the vehicle joining an intersection queue to its discharge across the intersection on departure. Control delay is the delay caused by a control device. It is approximately equal to time-in-queue delay plus the acceleration-deceleration delay component.
Since control delay is difficult to measure in the field, the values of time-in-queue delays were quantified in this paper. Control delay was obtained by adding the time required to decelerate from the desired speed to 0 and the acceleration time. The deceleration and acceleration times were obtained using typical rates (2.5 m/sec^2) at intersections [4-6]. The average time lost for acceleration/deceleration was obtained by dividing the sum of lost times (of vehicles that came on red or came at the end of the green light but did not pass the intersection) with the total number of vehicles.

3 Field Survey

3.1 Locations and configurations of the analysed intersections

In order to examine the intersection functioning in different prevailing conditions (structure of traffic flow and driver behaviour), two intersections were analysed, one in Split (Croatia), and the second one in Prague (Czech Republic).
On both intersections, traffic data were collected with video camera in the morning peak hour. In order to analyse different driver's behaviour for different weather conditions and purpose of trip, recording was performed few times:
1. working day – good weather - dry carriageway
2. working day – bad weather - wet carriageway (light rain)
3. Sunday – good weather - dry carriageway
4. Sunday – bad weather - wet carriageway (light rain)

In morning peak hour on working days, most of the trips were working trips, while in the morning peak hour on Sundays, minor number of trips were working trips.
The measurements of saturation headway were carried out in the time period of 7:30 to 8:00 when all manoeuvres on approaches were operated at the capacity limit (occasionally, the queue remains at the end of the green but at the end of the analysed time interval queues were cleared). In this interval, signal cycles were recorded with a video camera and processed by Android application (Headway) developed by programming language Sketchware [7]. Using this application, headway and start-up lost time have been determined. Android application Delay Study (Aspen Technic) [8] was used to measure time-in-queue delay. Of the measured values, the average value of each parameter was obtained. Measurements were made for main movements, where the vehicles pass intersection without interaction with other turning vehicles.

3.1.1 The intersection in Split, Croatia

The first observed intersection is located in Split, at the crossroads of Bruna Bušića and Poljička cesta street (Figure 2). Poljička cesta is long main city road that extends from the entrance of the city to its centre and, therefore, has one of the most heavily traffic volume.

Figure 2 Observed intersection in Split

Direction Q1 is the northern approach and consists of 3 traffic lanes (short left lane and 2 full lanes). Direction Q2 is the eastern approach which consists of 4 traffic lanes (two for through movements, one shared lane for through and right turns, and one exclusive short left lane). Q3 is the southern approach and consists of 3 traffic lanes (one short left lane, one for through and one shared lane for through and right). The west approach Q4 has the same geometry as the Q3. The intersection is signalized in four phases. The approaches Q1 and Q3 have the same phase in which the compound phase of the left turn appears. The approaches Q2 and Q4 have the same phase duration, but there is a completely-protected phase for left turns. The cycle duration is 90 seconds, and the duration of each phase is shown in Figure 3.

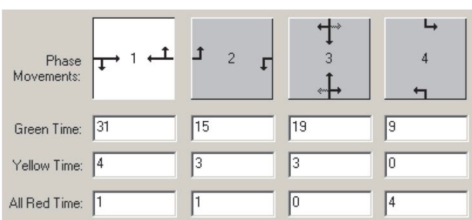

Figure 3 Phases duration

Traffic volume

Traffic data were collected with video camera in the morning peak hour on eastern approach through lane marked on Figure 2. Recorded number of vehicles in morning peak hour for different days and weather conditions is shown in Table 1, where PC denotes passenger cars, BUS is for buses and HV is for heavy vehicles.

Table 1 Table 1. Traffic volume for various conditions (the intersection in Split)

	PC	BUS	HV	TOTAL
Working day – dry carriageway	473	15	27	515
Working day – wet carriageway	467	19	22	508
Sunday – dry carriageway	297	14	16	327
Sunday – wet carriageway	262	13	11	286

3.1.2 The intersection in Prague

Location of second analysed intersection of Svatovske and Milade Horakove streets is in Prague, Czech Republic (Figure 4). It is located near the biggest castle in the world (Prague castle) which is the destination of numerous tourists, so this intersection has high traffic flow intensity.

Figure 4 Observed intersection in Prague

Direction Q1 is the northern approach which consists of 3 traffic lanes (two full left lanes and one right turn lane). The west approach Q2 consists of 3 traffic lanes (2 through lanes and one right turn lane). Direction Q3 is the southern approach which consists of 3 traffic lanes (2 left turn lanes and one right turn lane). This approach is one-way street, that is, there is no exiting traffic. The east approach Q4 consists of 3 traffic lanes (2 through lanes and one left turn lane).

This is signalized intersection with four phases and fully protected left turns from the approaches Q1, Q3 and Q4, while left turn from the approach Q2 is prohibited.

Recorded number of vehicles in morning peak hours is shown in Table 2.

Table 2 Traffic volume for various conditions (the intersection in Prague)

	PC	BUS	HV	TOTAL
Working day – dry carriageway	568	6	33	607
Working day – wet carriageway	440	6	35	481
Sunday – dry carriageway	507	-	10	517
Sunday – wet carriageway	426	3	7	436

4 The impact of applied values of saturation headway on capacity and delay

Tables 3. and 4. show average measured values (sec) of saturation headway (h_s) and average control delay (CD), for analysed intersections in Split and Prague.

Table 3 The values of h_s and CD for the intersection in Split

	Working day – dry carriageway		**Working day – wet carriageway**		**Sunday - dry carriageway**		**Sunday - wet carriageway**	
	h_s	**CD**	**h_s**	**CD**	**h_s**	**CD**	**h_s**	**CD**
Average	1.83	45.34	2.11	48.04	1.86	21.25	2.11	20.29
Median	1.74	n/a	2	n/a	1.77	n/a	2.01	n/a

Table 4 The values of h_s and CD for the intersection in Prague

	Working day – dry carriageway		**Working day – wet carriageway**		**Sunday - dry carriageway**		**Sunday - wet carriageway**	
	h_s	**CD**	**h_s**	**CD**	**h_s**	**CD**	**h_s**	**CD**
Average	1.72	76.2	1.97	77.41	1.84	63.39	2.06	68.77

From tables 3. and 4. it can be seen that saturation headway is higher in wet than in dry carriageway conditions for working and non-working day, in Split as well as in Prague. Saturation headway on working day, for wet carriageway in Split, is 15.3 % higher than in dry conditions (2.11/1.83), while in Prague, it is 14.5 % higher (1.97/1.72), that is, almost the same value. So it can be concluded that drivers react similarly in same prevailing conditions. Saturation flow rate of through lane in vehicles per hour, when the signal continuously show green light, is defined as s = 3600/h. It means that the capacity of intersection through lane in Split and Prague for dry carriageway is 15 % higher than for wet carriageway.
Saturation headway on Sunday for wet carriageway in Split is 13.4 % higher than in dry conditions (2.11/1.86), while in Prague, it is 12 % higher (2.06/1.84), slightly less than in Split. It means that the capacity is 12-13 % higher in dry conditions.
Saturation headway is 1.6 % higher on Sunday than on working day in Split for dry carriageway, while in Prague is 7 % higher. It can be explained by fact that most of drivers do not go to work on Sunday so they are more relaxed. In addition, traffic volume has less intensity, so vehicle queue and delay are smaller and drivers accept higher values.
The differences are smaller for wet carriageway, in Split there is no difference in saturation headway on Sunday and working day, while in Prague the difference is 4 % what is significantly less than for dry carriageway. It can be explained by the fact that saturation headway on Sunday is large enough because drivers are relaxed so the wet carriageway has a little impact on further increase of headway. Average (mean) value of saturation headway for analysed subsets of data is about 5 % higher than median value. In Table 5 is presented descriptive statistics of data on working day on dry carriageway.

Table 5 Descriptive statistics for a sample on working day on dry carriageway

Average	Median	Stand. error	Skewness	Kurtosis
1,84	1,74	0,06	0,68	0,58

The average value for all data subsets is greater than the median while the skewness of headways is positive which is consistent with [1]. It indicates that the distribution of queue discharge headway is likely unsymmetrical so the lognormal distribution should better fit data than normal [1].
The fact that average value of saturation headway h_s is greater than the median value means that more than 50 % drivers will use smaller headway. Therefore, traditional saturation flow rate estimation method would underestimate capacity, i.e. overestimate delay. Hence, it is reasonable to use the median value [1] to calculate the saturation flow rate.
In order to show consequences of using various values of saturation headway, ~~is~~ the analysis of delay for intersection in Split using software SIDRA [9] is conducted in this paper.
The data for working day on dry carriageway were used. Calculations were made for the same values of volume, but for the measured average and median values presented in table 5. Measured delay for analysed conditions was 45.34 sec, while resulting delays are shown in Figure 5.

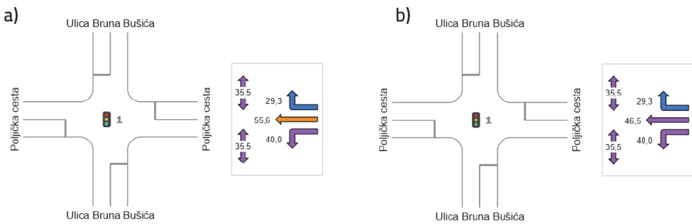

Figure 5 Resulting delays for the intersection in Split

It can be seen that the use of average value of saturation headway results in higher delay than measured (22 %), while using median value results in much better estimation (difference 2.6 %). Similar results were obtained for other conditions (wet carriageway, Sunday).
Considering that this research was conducted for just few hours on 2 intersections, general conclusions cannot be obtained. But, results indicate that there is a need for further research of using appropriate values of saturation headway when calculating capacity and delay in different prevailing conditions. Especially, it is important for different carriageway conditions.

5 Conclusions

This research points to the need for using appropriate values of saturation headway when calculating the intersection capacity and delay. Saturation headway is not a constant value; it depends about driver's characteristics, vehicle fleet, purpose of trip and weather condition, what is shown here.
For wet conditions, saturation headway is up to 15 % higher than for dry conditions on working day (up to 13 % on Sunday), both in Split and Prague. It means that the capacity is about 15(13) % smaller and resulting delay increases exponentially with small decrease of capacity for near saturated conditions.
On the other hand, either mean or median value of headway can be used, for every prevailing condition. Using mean value results with underestimation of capacity i.e. overestimation of delay because mean value of saturation headway is greater than the median value what means that more than 50 % drivers will use smaller headway. Hence, it is reasonable to use the median value [1] to calculate the saturation flow rate.
This research does not give general answers, just indicates a need for further research in this area and opens some questions. The main question is: Which saturation headway value is relevant, median or mean value; that for dry (traditional approach) or wet condition?

So choosing the appropriate value of saturation headway has a significant impact on traffic planning because the small error in capacity estimation results with significantly wrong estimation of control delay, i.e. level of service for near saturated conditions.

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References

[1] Chang-Qiao, S., Xiao-Ming, L.: Estimation of Saturation Flow Rates at Signalized Intersections, Discrete Dynamics in Transportation System, 2012, doi: 10.1155/2012/720474.

[2] Roess, R.P., Prassas, E.S., McShane, W.R.: Traffic Engineering (Fourth edition) Pearson Education inc. Pearson Prentice Hall, New Jersey 2011

[3] Transportation Research Board: Highway Capacity Manual, 3 (2010), Interrupted Flow

[4] Wang, J., Dixon, K., Li, H., Ogle, J.: Normal Acceleration Behaviour of Passenger Vehicles Starting from Rest at All-Way Stop-Controlled Intersections, Transportation research Record, 1883 (2004), pp 156-166, doi: 10.3141/1883-18.

[5] Wang, J., Dixon, K., Li, H., Ogle, J.: Normal Deceleration Behavior of Passenger Vehicles at Stop Sign-Controlled Intersections Evaluated with In-Vehicle Global Positioning System Data, Transportation Research Record, Journal of the Transportation Research Board, 1 (1937), pp.120-127, doi: 0.3141/1937-17

[6] Bokare, P.S., Maurya, A.K.: Acceleration-Deceleration Behaviour of Various Vehicle Types. World Conference on Transport Research – WCTR 2016, Transportation Research Procedia, 25 (2017) 4733–4749, Shangai

[7] Android programming language, http://sketchware.io

[8] Aspen Technic. Delay Study, https://play.google.com/store/apps/appinventor.ai.

[9] AKCELIK & ASSOCIATES: SIDRA intersection 7.0.

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THE EFFECT OF THE TRAFFIC COMPOSITION ON THE URBAN TRAFFIC CAPACITY. PASSENGER CAR EQUIVALENT COEFFICIENTS

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Abstract

Predicting the traffic capacity and its elements requires bringing the traffic flow represented by various vehicles to uniformity expressed in the equivalent number of passenger cars, through the use of the passenger car equivalent coefficients (PCE). The currently used in Russian Federation passenger car equivalent coefficients are taken on the basis of studies of the capacity of the Russian Federation, carried out in the 70s - 80s of the last century on rural roads, where most of the vehicles were heavy vehicles. Currently, the traffic flow is mostly represented by passenger cars. The riding qualities of cars, especially trucks, have changed significantly. This situation is especially common to Moscow. In this regard, the question of clarifying the traffic flow composition and revising the passenger car equivalent coefficients becomes relevant. The article presents the methodology and results of studies carried out on the route sections between road crossings to determine the passenger car equivalent coefficients and the traffic composition in Moscow.

Keywords: passenger car equivalent coefficient, capacity, traffic composition

1 Introduction

Predicting the level of traffic load to plan the development of a network and design individual objects requires modern techniques, one of the important components of which is the reduction factor of cars of various categories to a passenger car. It can vary depending on the behavior of drivers, which in turn depends on improvements in vehicle design, including active safety systems. In this regard, regular studies of traffic flows are needed to track changes in parameters that affect traffic capacity. Designing highways and city streets requires determining the capacity of traffic lanes on highways and various types of road crossings.

As the traffic flow consists of many types of cars, it is necessary to bring it to the equivalent traffic of cars to determine traffic capacity. Today, the Russian traffic flow mainly consists of cars, trucks with a carrying capacity of 2 to 14 tons, minibuses, buses of small, medium and large capacity, articulated buses and road trains with a carrying capacity of 12 to 30 tons. The reduction of a mixed flow of vehicles to a homogeneous one consisting of an equivalent number of cars is carried out using the reduction factors. Thus, the reduction factors are a fundamental component in determining the throughput of highways and roads. The research aim is to determine traffic composition and passenger car equivalent coefficients on the urban roads sections in modern conditions in Russian Federation.

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Section 2 presents literature review of PCE studies in Russian Federation and other countries. Section 3 contains information about the current research methods and data collecting. Section 4 presents the researches results and data analyses. In section 5 future recommendations are discussed.

2 Literature review

Most of the domestic studies devoted to the determination of the reduction factors were carried out in the 70s of the last century [1]. For this purpose, various methodological approaches can be used [1].

Method 1. The method is based on the analysis of distances and time intervals between successive cars of different types, in comparison with the movement of cars.
This method compares the dynamic dimensions of the vehicle under consideration and a passenger car. The value of the reduction factor PCE_{ij} is determined by the ratios:

$$PCE_{ij} = \frac{d_{ij}}{d_{ii}} \quad (1)$$

or

$$PCE_{ij} = \frac{\Delta t_{ij}}{\Delta t_{ii}} \quad (2)$$

Where
d_{ij} – distance between the considered vehicles, m;
d_{il} – distance between passenger cars, m;
Δt_{ij} – time interval between the considered vehicles, sec;
Δt_{ii} – time interval between passenger cars, sec.

Method 2. The method is based on the analysis of the “speed-intensity” relationship for various car flows in comparison with a similar relationship for a passenger car flow.
This method compares the average speed of the mixed vehicle flow and the speed of the passenger car flow, and analyzes the traffic intensities at the same flow speed. The reduction factors PCE_{ij} are determined by the ratio:

$$N_{ii} = (1 - p)N^{ij} + PCE_{ij} p N^{ij} \quad (3)$$

Where
N_{ii} – traffic intensity of a passenger car flow, vehicle/hour;
N^{ij} – traffic intensity of a mixed vehicle flow, vehicle/hour;
p – number of slowly moving cars.

Method 3. The method is based on the analysis of traffic capacity with different traffic flow composition.
It compares capacities of a road lane with homogeneous flows, consisting of the considered vehicles for the case of movement along a horizontal rectilinear section with the passenger car traffic capacity.

Method 4. The method is based on the analysis of the traffic density of mixed vehicle flows.
The values of the factors PCE_{ij} were determined from the ratio of the traffic densities of the considered vehicles and the traffic flow with the corresponding capacity. Density is determined by the maximum density of a standing traffic flow:

$$q=0.5q_{max} \quad (4)$$

where
g – density of the traffic flow with corresponding capacity;
q_{max} – density of the considered vehicles.

<u>Method 5.</u> The method determines the traffic flow with the maximum number of overtaking. It compares the traffic flow with the greatest number of overtaking of trucks by passenger cars. The average number of overtaking e with different traffic composition is determined by the formula:

$$e=q_i(V_i-V_j)q_j \quad (5)$$

where
q_i – density of freely moving cars;
V_i – speed of freely moving cars;
q_j – density of slowly moving cars;
V_j – speed of slowly moving cars.

The traffic composition and the passenger car equivalent coefficients obtained by methods 1-5 in 1970-s in Russian Federation are presented in Table 1.

Table 1 The traffic composition and the passenger car equivalent coefficients obtained by methods 1-5 in 1970-s in Russian Federation

Vehicles	**PCE_{ij} obtained by each method**					
	1	**2**	**3**	**4**	**5**	
Passenger cars	1.00	1.00	1.00	1.00	1.00	1.00
Motorcycles	0.75	0.70	0.68	0.40	0.72	0.65
Light trucks	1.20	1.60	1.70	1.40	1.68	1.52
Medium trucks	1.36	1.83	1.95	1.68	1.92	1.75
Heavy trucks	1.75	2.60	3.10	1.75	2.80	2.40

The passenger car equivalent coefficients depend on a large number of factors, the main of which are speed, traffic composition, and road conditions. Given the difficulty of obtaining reliable design relationships to find the passenger car equivalent coefficients, they should be used only to calculate the traffic capacity [1].
Today, the regulatory documents of the Russian Federation use the passenger car equivalent coefficients obtained on the basis of the ratio of the dynamic dimensions of vehicles excluding the differentially different traffic conditions (road sections of streets, various types of road crossings, etc.) [2].
The term “ passenger car equivalent coefficients (PCE)” was first introduced abroad in 1965 in the American Highway Capacity Manual. Since that moment, a number of foreign authors [4-8] have carried out a large number of studies to determine the passenger car equivalent coefficients for various elements of the road network.
One of the many foreign approaches to the determination of the passenger car equivalent coefficients on the street and road sections is a method based on comparing the headways of different vehicle types [8, 9].
The following intervals are determined when a car is moving behind another car, and when various vehicle types are following a passenger car (Figure 1).

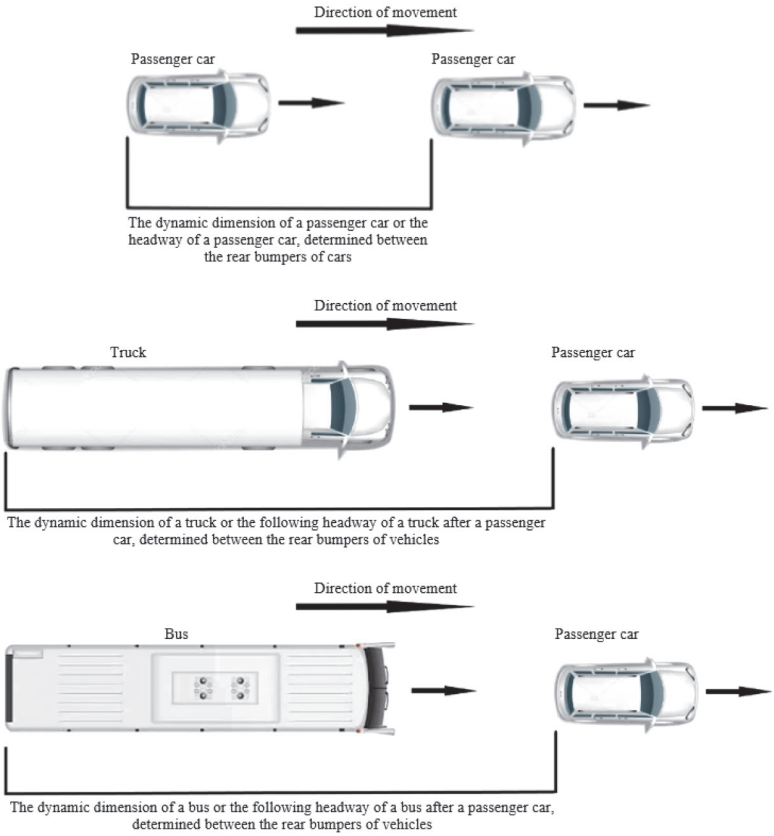

Figure 1 The following headways of different vehicle types

In this connection, the passenger car equivalent coefficients PCE_{ij} are determined by the formula:

$$PCE_{ij} = \frac{\Delta H_{ij}}{\Delta H_{ii}} \tag{6}$$

where
ΔH_{ij} – the following headway of the selected vehicle type behind a passenger car;
ΔH_{ii} – the following headway between passenger cars.

Over the past 10-15 years, redefining of the passenger car equivalent coefficients has been very actively studied in India. A number of authors [14, 15, 16, 17] carried out research to determine the passenger car equivalent coefficients on 2-lane urban roads, road sections and various types of road crossings.
The fundamental task in determining the passenger car equivalent coefficients is the traffic composition by vehicle types.
The regulatory documents of Russia use different traffic compositions and passenger car equivalent coefficients for urban streets and rural roads. For urban streets, 13 types of cars are accepted, for rural roads - 14. In studies devoted to the determination of the passenger car equivalent coefficients, a number of authors [1, 2, 3] use a simplified traffic composition, including 3-4 types of cars (cars, trucks, buses, and road trains). A simplified stream composition is also used in foreign studies [1-10].

The 2013 American FHWA guidelines adopted a very detailed traffic composition (13 classes, including 34 subclasses) based on the type of vehicle, its dimensions, and carrying capacity. However, the 2010 and 2016 HCM guidelines [12, 13] use the average PCE: 1.00 for passenger cars and 2.00 for all other vehicle types.
Comparison of the results of foreign authors and the passenger car equivalent coefficients used in the Russian Federation is shown in Table 2.
The above studies used various traffic composition and methods for determining the passenger car equivalent coefficients. At the same time, both foreign and domestic authors agree on the influence of a large number of factors on the passenger car equivalent coefficients and the need to introduce special coefficients for each element of the road network (sections of streets and roads, signalized intersections, unsignalized intersections, roundabounds), considering their traffic conditions (conflict points, direction of movement, traffic control regime, delays) [1, 2, 3, 5, 7].

Table 2 Comparison of the results of foreign authors and the passenger car equivalent coefficients used in the Russian Federation

<table>
<tr><th rowspan="2">Vehicle type</th><th colspan="6">PCE on the road sections according to various regulatory documents and authors</th></tr>
<tr><th>SP 34.13330. 2012</th><th>ODM 218.2.020-2012</th><th>SP 396.1325800. 2018</th><th>Mehar, Arpan & Chandra</th><th>Indian Roads Congress (IRC SP 41)</th><th>Ahmed Anwaar&Boxel</th></tr>
<tr><td>Cars</td><td rowspan="3">1.0</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td><td>1.0</td></tr>
<tr><td>Motorcycles</td><td>0.5</td><td>0.5</td><td>0.19-0.26</td><td>0.5</td><td>-</td></tr>
<tr><td>Minibuses</td><td>-</td><td>1.5</td><td>-</td><td>1.5</td><td>-</td></tr>
<tr><td colspan="7">Trucks, carrying capacity:</td></tr>
<tr><td>- max. 2 t</td><td>1.3</td><td>1.1</td><td>1.5</td><td rowspan="5">1.34-1.58</td><td rowspan="5">1.5</td><td rowspan="5">1.37</td></tr>
<tr><td>- 2-6 t</td><td>1.4</td><td>1.8</td><td>2.0</td></tr>
<tr><td>- 6-8 t</td><td>1.6</td><td>2.1</td><td>2.5</td></tr>
<tr><td>- 8-14 t</td><td>1.8</td><td>2.4</td><td>3.0</td></tr>
<tr><td>- over 14 t</td><td>2.0</td><td>2.5</td><td>3.5</td></tr>
<tr><td colspan="7">Heavy trucks, carrying capacity:</td></tr>
<tr><td>- max. 12 t</td><td>1.8</td><td>2.2</td><td rowspan="4">4.0</td><td rowspan="4">1.06-1.24</td><td rowspan="4">4.5</td><td rowspan="4">1.65</td></tr>
<tr><td>- 12-20 t</td><td>2.2</td><td>2.4</td></tr>
<tr><td>- 20-30 t</td><td>2.7</td><td>-</td></tr>
<tr><td>- over 30 t</td><td>3.2</td><td>3.3</td></tr>
<tr><td colspan="7">Buses, carrying capacity</td></tr>
<tr><td>- low</td><td>1.4</td><td rowspan="4">2.6</td><td rowspan="3">2.5</td><td rowspan="5">-</td><td rowspan="5">3.0</td><td rowspan="5">-</td></tr>
<tr><td>- medium</td><td>2.5</td></tr>
<tr><td>- high</td><td>3.0</td></tr>
<tr><td>Articulated buses</td><td rowspan="2">4.6</td><td>4.0</td></tr>
<tr><td>Trolleybus</td><td>-</td><td>3.0</td></tr>
</table>

3 Data collection

As the subject of research, we selected sections of urban motorways with continuous traffic, where the influence of ramps and main traffic inflows is absent.
This situation is common to the Moscow ring road. With a high traffic density in the two right-hand lanes, there is a flow that includes heavy vehicles, medium-tonnage vehicles moving along the 2nd and 3rd lanes, and light commercial vehicles in the 3rd and 4th lanes. The 4th and 5th lanes are occupied mainly by passenger cars.
The study was carried out on the section of the Moscow ring road: 31 km – between M-4 "Don" and Varshavskoe highway and 71 km between Putilkovskoe highway and Novokurkinskoe highway.
Before the study, the following PCE assessment method was considered. The method involved field studies by video recording of the intervals between different vehicles moving with the same speed.
Video recording is carried out using an unmanned quadcopter at an altitude of 50-100 m above the selected area to simultaneously cover all traffic lanes. The selected shooting height allows determining the type of a moving vehicle.
The database includes traffic flow, the number of cars with the selected traffic composition, the headways between cars, determined between the rear bumpers of cars.

4 Results

One of the objectives of the study is to determine the current traffic composition in Moscow. For this purpose, the traffic composition was adopted in accordance with the current regulatory documents. The research results are shown in Table 3 and in the diagram (Figure 2).
The analysis of the data obtained is based on the comparison theory of the lagging headways by equation (1) and equation (6) presented in section 2. The study found out that some vehicle types percentage is a very small value to reliably determine the drive ratios under given conditions.

Table 3 Traffic composition by research results in Moscow, Russia

No.	Vehicle type	%
1	Passenger car	83.85
2	Minibus	1.01
3	Truck with a carrying capacity of up to 2 tons	2.97
4	Small bus	1.71
5	Truck with a carrying capacity of 2-6 tons	7.49
6	Large bus	$<$ 1.00
7	Truck with a carrying capacity of more than 6 tons	1.50
8	Articulated bus	$<$ 1.00
9	Heavy truck	1.07

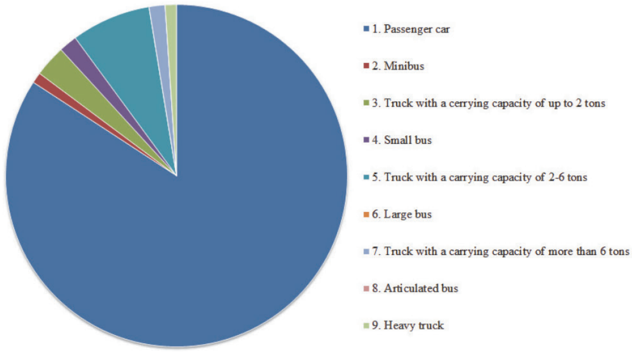

Figure 2 Moscow Ring Road traffic composition diagram

The results of PCE research on road sections in Moscow are presented in Table 4. The table presents determined average lagging headways between different vehicle types and calculated PCE. Comparison of the obtained research results with the passenger car equivalent coefficients specified in the current regulatory documents of Russian Federation* is shown on Figure 3.

Table 4 Lagging headway and PCE research results in Moscow, Russia

No.	Vehicle type	Average lagging headway, sec.	PCE
1	Passenger car	2.55	1.00
2	Minibus	3.52	1.38
3	Truck with a carrying capacity of up to 2 tons	3.47	1.36
4	Small bus	3.16	1.24
5	Truck with a carrying capacity of 2-6 tons	2.58	1.02
6	Large bus	-	-
7	Truck with a carrying capacity of more than 6 tons	3.48	1.37
8	Articulated bus	-	-
9	Heavy truck	-	-

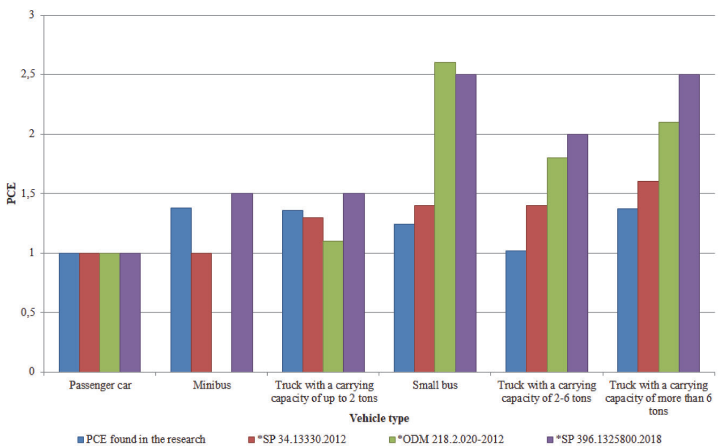

Figure 3 Comparison of the valid Russian and obtained PCE.

5 Conclusion

Many authors accept a simplified traffic composition in their research: cars, trucks, buses. The results of studies of the traffic flow on the sections of the city highway to Moscow (Moscow Ring Road) showed different passenger car equivalent coefficients obtained in the 70s, used at the moment and obtained in field studies. Given the very different nature of urban traffic on the road sections and various types of road crossings, the reduction factors should be considered for each situation separately. Within the framework of the above study, the passenger car equivalent coefficients were obtained for sections of continuous traffic on the city highway sections. Due to a huge number of factors affecting the passenger car equivalent coefficients, this issue should be considered in more detail in future studies.

References

[1] Silianov, V.V.: The theory of traffic in road design and traffic management, Transport, p. 303, 1977.

[2] Mikhailov, A.I., Golovnykh I.M.: Modern trends in the design and reconstruction of city street and road networks, Novosibirsk: Nauka, 3 (2004)

[3] Levashev, A.G., Mikhailov, A.I., Golovnykh, I.M.: Engineering of controlled intersections: Study guide, Irkutsk: Publishing House of Irkutsk State Technical University, 2007.

[4] Linzer, E., Roess, R., McShane, W.: Effect of trucks, buses, and recreational vehicles on freeway capacity and service volume, Transportation Research Record 699, Transportation Research Board, Washington, D.C., pp 17–26, 1979.

[5] Webster, N., Elefteriadou, L.: A simulation study of truck passenger car equivalents (PCE) on basic freeway sections, Transp Res Part B 33 (1999) 5, pp. 323–336

[6] Huber, M.J.: Estimation of passenger car equivalents of trucks in the traffic stream, Transportation Research Record 869, Transportation Research Board, Washington, D.C., pp 60–70, 1982.

[7] Sosin, J.A.: Delay at Intersections Controlled by Fixed Traffic Signals, Traffic Engineering and Control, 21 (1980) 8/9, pp. 407-413

[8] Van Boxel, D., Sinha, K.C., Bullock, D., Mannering, F.L.: An Exploratory Study of Vehicle Class Headway Ratios as Passenger Car Equivalence Values Using Three Stage Least Squares Estimation, 89th Annual Meeting of Transportation Research Board, Washington, DC, 2009.

[9] Ahmed, A., Boxel, D., Volovski, M., Anastasopoulos, P., Labi, S., Sinha, K.: Using Lagging Headways to Estimate Passenger Car Equivalents on Basic Freeway Sections, Journal of Transportation of the Institute of Transportation Engineers, 2 (2011), pp. 5-17

[10] Elefteriadou, L., Torbic, D., Webster, N.: Development of Passenger Car Equivalents for Freeways, Two-Lane Highways, and Arterials, Journal of the Transportation Research Board, 1572 (1997), pp. 51–58, doi: https://doi.org/10.3141/1572-07

[11] Highway Research Board, Special Report 87, Highway Capacity Manual, Washington, DC: National Research Council, Department of Traffic and Operations, Committee on Highway Capacity, 1965.

[12] Transportation Research Board, Highway Capacity Manual, Transportation Research Board, Washington D.C., 2010.

[13] Transportation Research Board, Highway Capacity Manual 6th Edition – A Guide for Multimodal Mobility Analysis, Transportation Research Board, Washington D.C., 2016.

[14] Raj, P., Sivagnanasundaram, K., Asaithambi, G., Ravi, S.: Review of Methods for Estimation of Passenger Car Unit Values of Vehicles, Transportation engineering journal of ASCE, 2019.

[15] Metkari, M., Budhkar, A., Maurya, A.: A Review of Passenger Car Equivalence For Indian Conditions, International Journal of Computer Applications, 2012.

[16] Mahidadiya, A.N., Juremalani, J.: Global Scenario on Estimation of Passenger Car Unit, A Review, 2016.

[17] Mehar, A., Chandra, S., Velmurugan, S.: Passenger Car Units at Different Levels of Service for Capacity Analysis of Multilane Interurban Highways in India, Journal of Transportation Engineering, 2014.

U-TURN CAPACITY AT SIGNALIZED INTERSECTIONS

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Abstract

The article presents the results of a study of the intersections capacity, at which the U-turn lane is organized, depending on the traffic flow volume and its organization. Signalized intersection capacity depends on traffic flow, geometry, traffic organization type, driver's behaviour and headways between drivers in each traffic lane. To analyse signalized intersection capacity it is necessary to determine traffic lanes saturation flow. The study presents field observation analyses of headways between drivers and saturation flow on U-turn lanes at signalised intersections in Moscow. To conduct research and identify patterns, intersections in Moscow were chosen with different conditions and different organization of the U-turns.

Keywords: intersection, critical headway, U-turn, capacity

1 Introduction

U-turn at the intersection is performed from the left lane, which can be designed for a left turn or be a shared lane for through and left-turn movements.
U-turn from the left lane can be carried out at the intersection itself, the queue to the left turn and the U-turn is accumulated before (downstream) the stop line. In this case, the U-turn is made through the upstream traffic or using a separate phase for left-turn traffic. The critical headway required for a U-turn exceeds the critical headway for a left turn, hence the presence of U-turn cars in the queue reduces the capacity of left-turn movement through the upstream traffic. U-turns can also be allowed before intersection from a left turn lane or a special lane for a U-turn. In the second case, the U-turn is made during the prohibition of the upstream traffic at the nearest signalized intersection.
At the selected intersections and sections of streets, surveys were carried out of the number of cars making a U-turn at different volume of the traffic flow, the percentage of cars making a U-turn, the delay before performing the manoeuvre and critical headway required for a U-turn.
This study aims to compare the saturation flow values of U-turn lanes at controlled intersections with different traffic arrangements. The types of the studied U-turn lanes are shown in Fig. 1

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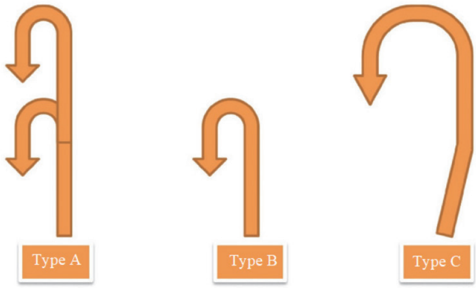

Figure 1 U-turn lanes traffic arrangements types at signalized intersections

The first type of arrangements for the allocated U-turn lane (type A in Fig. 1) allows drivers to make a U-turn in two flows.
The second type (type B in Fig. 1) allows drivers to make a U-turn in one flow.
The third type (type C in Fig. 1) suggests the following way of drivers movement. At first, drivers go along the lane allocated for the right turn, then cross the oncoming traffic flow and complete the maneuver, moving along the lanes allocated for the left turn.
Section 2 presents the research capacity and traffic flow methods. Section 3 presents data collecting and field observation process. Section 4 presents the research conclusions.

2 Research methods

Russian studies [3, 4] and guidelines [1] determine the capacity of the traffic lane at a signalized intersection depending on the saturation flow and the duration of the green phase as part of the traffic signal regulation cycle. The lane capacity P is determined by the formula (1):

$$P = S \cdot \frac{t_{green}}{T_c} \tag{1}$$

where
S - the saturation flow,
t_{green} - green signal duration, s,
T_c - cycle length, s.

The basic saturation flow corresponds to the number of vehicles moving freely in the forward direction, provided that the duration of the permissive traffic signal is one hour. This does not take into account the effects of longitudinal gradients, parking, bus stops, multi-lane roadways, pedestrians and cyclists. [1, 2]. The recommended value of a basic saturation flow with a lane width of 3.50-3.60 m is 1900 veh/hour [1]. The same value is adopted in the USA HCM 2016 Guidelines for cities with a population of more than 250,000 [2].
Reduction factors are applied for the turn lanes. In the Russian Guidelines [1], with a lane width of 3.50-3.60 m, the following factors are used: for turning left 1/1.75 or 0.571, for turning right 1/1.20 or 0.833. The US Guidelines for turn lanes management recommends using the value of 1.00 for an individual separate lane; if it is included in a group of lanes: to the left turn lane - 0.971, to the right turn lane - 0.885 [2].
Russian and foreign studies were conducted to saturation flow and capacity values determent at signalized intersections. The question of surface type and conditions were observed in Russia [10]. Saturation flow modelling at signalized intersections for modern traffic conditions was developed in India [7, 8]. A huge work was done in Japan [5], the values of modern and last decade saturation flow were compared with USA HCM 2016 [2] values and saturation flow reduction factors were established. The questions of queue length and left turn capacity

at signalized intersections were observed in China [6]. The questions of saturation flow regression at rainy weather conditions were observed in Shanghai [9].
When determining the saturation flow by field observations, Russian [1, 3, 4] and foreign methods [2, 5] suggest calculating the saturation flow by the formula (2). Provided that the mean value of the headway between vehicles takes into account the start delays of the first 4 cars when they are leaving the queue.

$$S = \frac{3600}{\Delta t_{cp}} \quad (2)$$

where
S - the saturation flow, veh/hour
Δt_{cp} - headway when the vehicles are leaving the queue, s.

3 Data collecting and field observation process

Signalized intersections in Moscow were selected as the object of research. The selected intersections are located on arterial roads with multi-lane carriageways. The width of the traffic lanes is 3.50 m. The intersections are equipped with traffic signals with different cycle lengths.
At the same time, traffic signal synchronization assumes a separate phase for the left turn and making a U-turn, thus there is no influence of the oncoming traffic flow on the drivers making a U-turn. During the study, the duration of the traffic signal control cycle and the duration of the green signal for turning left were determined.
Diagrams of typical intersections with allocated traffic lanes for a U-turn (types A, B, C) with indication of the traffic directions are shown in Fig. 2.
The theoretic cross-section for recording the vehicle passage is set along the left edge of the U-turn lane where the allowing changing lane road markings are applied.
For crossing the U-turn lane of type A, the cross-section length is 35.00 m, for type B - 25.00 m. For type C, the cross-section is set along the stop line of the allocated traffic lane for turning right, the section width is 7.00-12.00 m for two traffic lanes.
Field observations were carried out during daylight hours when the road surface was dry. The observations were carried out by video filming the traffic flow using stationary surveillance cameras located near intersections at a height of 6-10 m.
The video analysis allows obtaining the following database used in the study: lane traffic flow, headway between drivers, percentage of trucks in the traffic flow.

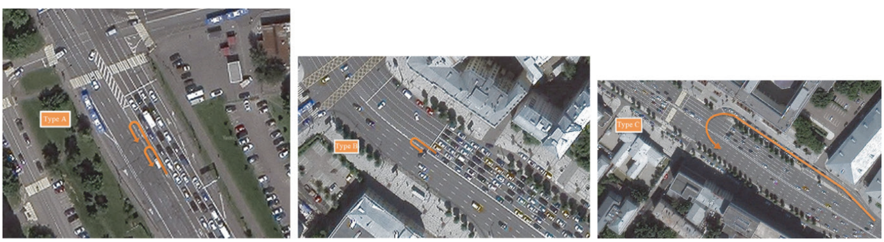

Figure 2 Layouts of the observation objects in Moscow. Type A - intersection of Lipetskaya St. and Lebedyanskaya St. Type B - intersection of Zubovskiy Boulevard and Prechistenka St. Type C - intersection of Zubovsky Boulevard and Zubovsky Proezd.

4 Conclusions

The result of the study is a comparative analysis of traffic flow parameters for three types of the U-turn lanes. The percentage of heavy vehicles is less than 15 %. Fig. 3 shows mean values of the headway between drivers when they are leaving the queue. Fig. 4 shows the actual values of hourly U-turn lanes traffic intensity. Fig. 5 provides a comparative analysis of the saturation flow of the U-turn lanes.

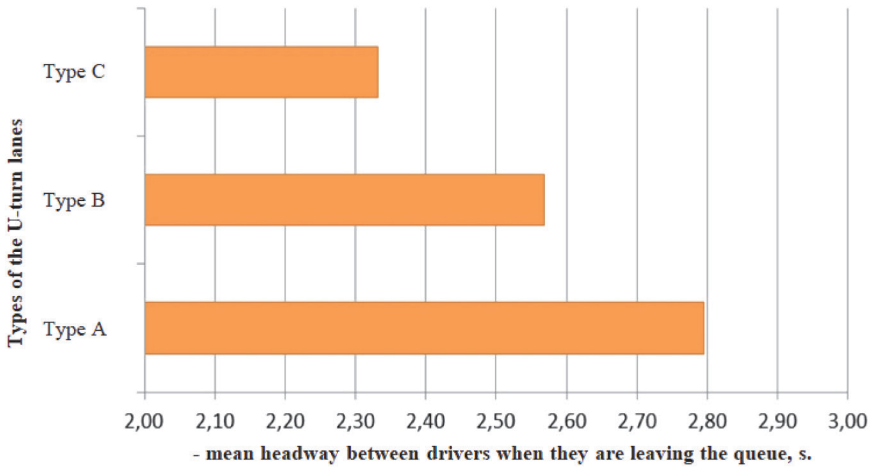

Figure 3 Mean headway between drivers

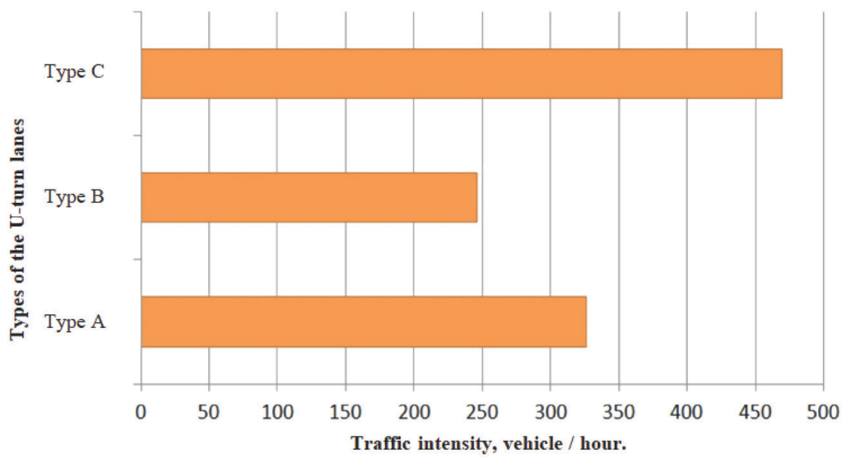

Figure 4 Traffic flow by field observations

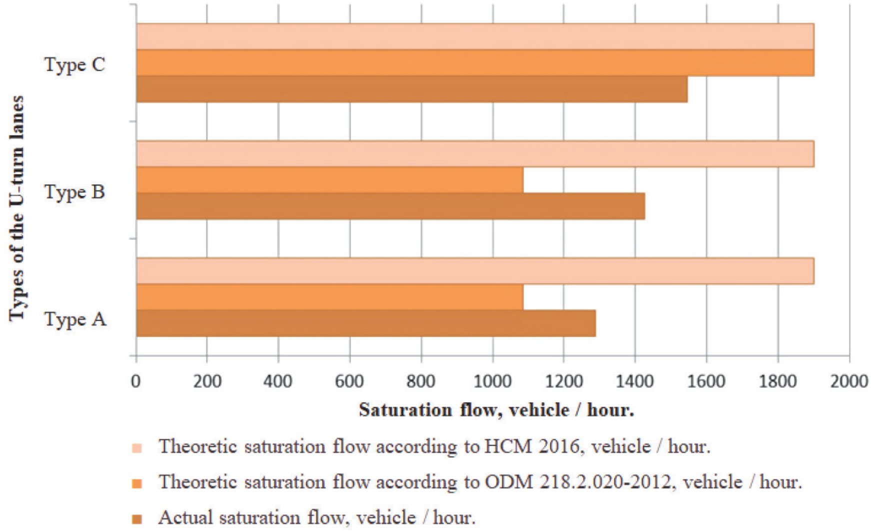

Figure 5 Comparative analysis of saturation flow values

The compare of theoretical values of saturation flow used in Russian Federation ODM [1] and used in USA HCM [2] with actual saturation flow shown the difference of 20-30 %.
The comparative analyses of average headways between leaving the queue drivers and saturation flow of different type U-turn lanes shown the more effective probability of type C U-turn lanes.

References

[1] ODM (Highway Industry Methodological): Guidelines for assessing the highway capacity, 218.2.020-2012, Moscow, Russian Federation, 2012.

[2] Highway Capacity Manual 2016.: Transportation Research Board, Washington, D.C., 2016.

[3] Silyanov, V.: The theory of traffic flows in the engineering of roads and traffic management, Transport, Moscow, Russian Federation, 1977.

[4] Kostsov, A.V., Bakhirev, I.A., Borovik, E.N., Martyakhin, D.S.: Transport planning of cities: Manual, A-project, Moscow, 2017.

[5] Aoyama, E., Yoshioka, K., Shimokawa, S., Morita, H.: Estimating saturation flow rates at signalized intersections in Japan, Asian Transport Studies, 6 (2020)

[6] Yang, Q., Shi, Z.: The queue dynamics of protected/permissive left turns at pre-timed signalized intersections, Physica A: Statistical Mechanics and its Applications, 562 (2021)

[7] Chand, S., Gupta, N.J., Velmurugan, S.: Development of Saturation Flow Model at Signalized Intersection for Heterogeneous Traffic, Transportation Research Procedia, 25 (2017), pp. 1662-1671

[8] Borigarla, B., Rakesh, P., Maddala, P., Mudda, L.P.: Effect of approach width on performance of signalized intersections, Materials Today: Proceedings, 2021.

[9] Sun, H., Yang, J., Wang, L., Li, L., Wu, B.: Saturation Flow Rate and Start-up Lost Time of Dual-left Lanes at Signalized Intersection in Rainy Weather Condition, Procedia - Social and Behavioral Sciences, 96 (2013), pp. 270-279

[10] Novikov, A., Novikov, I., Shevtsova, A.: Study of the impact of type and condition of the road surface on parameters of signalized intersection, Transportation Research Procedia, 36 (2018), pp. 548-555

IMPROVED APPLICABILITY DIAGRAM OF TWO-LANE ROUNDABOUTS

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Abstract

When reconstructing existing or constructing completely new intersections, the main problem is determining the type of future intersection. Capacity is one of the key indicators that influence the choice of traffic control type. In this paper, using different scenarios of theoretical traffic flow distributions and traffic volume scenarios, the authors have determined the applicability area of two-lane roundabouts. The results obtained were used to improve the existing applicability diagrams of the various intersection types presented in several issues of US Highway Capacity Manuals (US HCM). Capacity in each scenario is determined using HCM 2010 and Hagring methods with practically obtained values of the time gap acceptance parameters.

Keywords: two-lane roundabouts, capacity, gap-acceptance parameters, HCM 2010, Hagring

1 Introduction

In order to determine type of control of future (planned) intersection, based only on traffic demands or capacity, engineers can use applicability diagrams suggested in the US Highway Capacity Manual 2000 (HCM 2000) (Figure 1) [1] or in the newer edition called Planning and Preliminary Engineering Applications Guide to the Highway Capacity Manual 2016 (Figures 2 and 3) [2]. Although capacity is not the only criteria on which this decision should be made, it is the most proper regards to the desired level of service. Other relevant factors are space availability, type of adjacent intersections, environmental impact etc.
The capacity of one intersection is not an absolute category and highly depends on traffic flow distribution between different approaches. Based on HCM 2000 roundabouts are desirable solutions when major and minor streets have approximately equal traffic volume (Figure 1). Other traffic combinations are not detailed explained. In the new version of Highway Capacity Manual (HCM 2016) two different diagrams, depend on different traffic distribution, are proposed when it comes to roundabouts applicability. Furthermore, these diagrams recommend separately one-lane and two-lane roundabouts.
According to these diagrams, one-lane roundabouts are the right solution for almost every traffic volume combinations. It can be applied for major traffic volume between 500 and 1.600 (1.900) veh/h while minor traffic volume is up to 800 veh/h. At the same time, two-lane roundabouts can be applied for major volume higher than 1.050 (1.400) veh/h. Both types of roundabouts have limited application to conditions of a lower major (up to 1.000 veh/h) and minor (up to 500 veh/h) volumes. Under these conditions two-way stop intersection is highly preferred over all other types of intersection control. This can be related to the

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USA local policy and long-term familiarity with this type of intersections. Drivers are more accustomed to STOP sign-controlled intersections instead of roundabouts.
It is clear that applicability diagrams for several types of intersections in different HCM editions differ significantly. They differ not only in detail (roundabouts are far more present in the newer version) but also in traffic volumes to which roundabouts may apply. Because their application highly depends on traffic distribution, these diagrams have to be updated and expanded for more traffic volume combinations and capacity values.

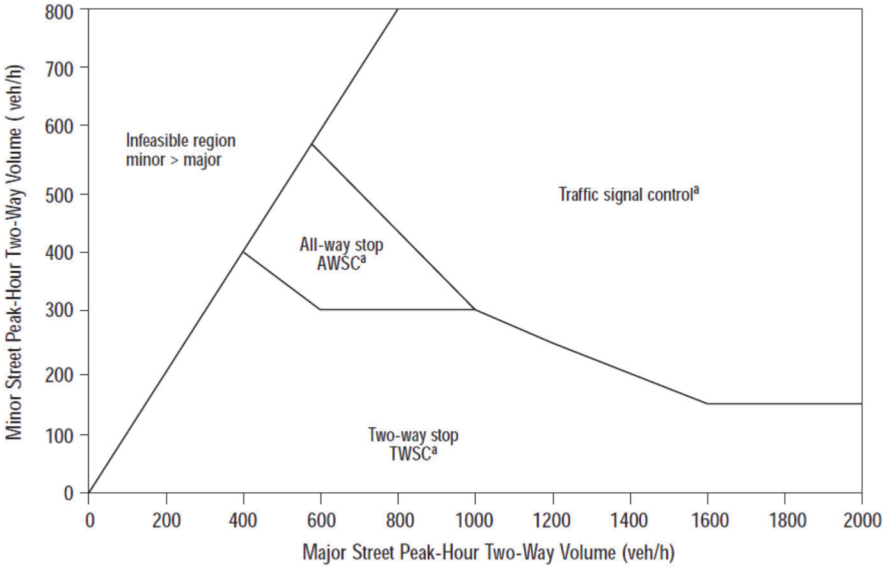

Figure 1 Intersection control type and peak-hour volumes (a. Roundabouts may be appropriate within portion of these ranges) [1]

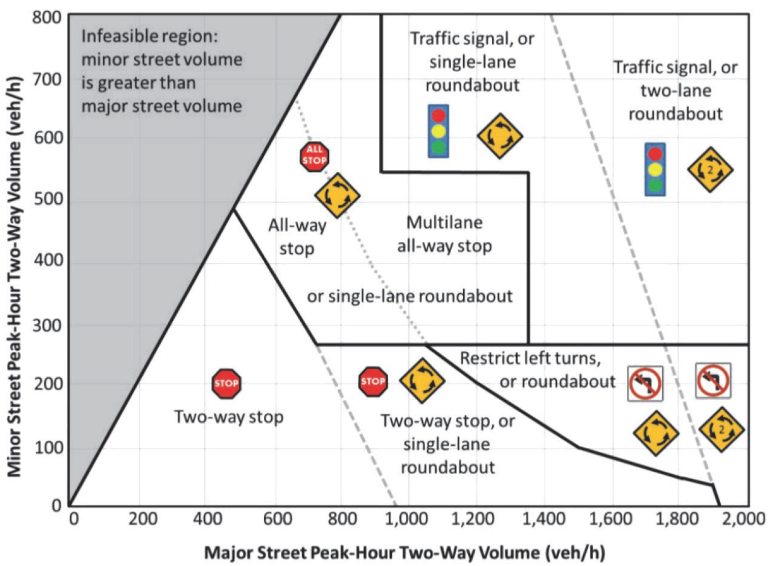

Figure 2 Intersection control type by peak hour volume - 50/50 Volume Distribution on Each Street [2]

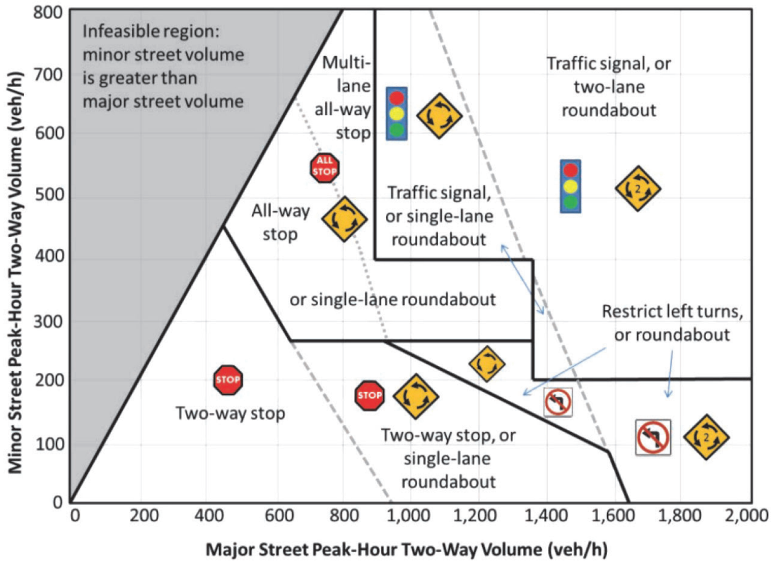

Figure 3 Intersection control type by peak hour volume - 67/33 Volume Distribution on Each Street [2]

2 Capacity calculation methods

Capacity calculation of any type of roundabout can be performed with different empirical regression and gap-acceptance based methods. Empirical regression methods are based on experiences of built roundabouts and their geometric and traffic conditions. Contrary to this, gap-acceptance methods represent a fully theoretical method. Two parameters are essential for this method: critical gap (t_c) and follow-up headway (t_f).

In this research, two analytical methods were used. First, Hagring method [3], is developed on conflict theory of traffic streams at roundabouts entries. Second, HCM 2010 method [4] can be consider as semi-analytical, but in this paper is calibrated with field obtained gap-acceptance parameters. Definitions, field investigation and calculation methods of critical gap and follow-up headway are not explained in detail in this paper. Instead, we refer to [5], [6] and [7].

In 1998, Hagring [3] proposed a complete formula for the capacity calculation that can be used for each multi-lane intersection. Driver behaviour is represented with a different set of gap-acceptance parameters on each conflict of entry lane and circular lane inside the roundabout. A common assumption is that gaps between vehicles in the circular lane have Cowan's M3 distribution which leads to the following Hagring's capacity equation:

$$C = \frac{e^{\left(-\sum_{i \in I_k} \lambda_i \cdot (t_{c,i} - \Delta_i)\right) \cdot \sum_{i \in I_k} \lambda_i}}{1 - e^{\left(-\sum_{i \in I_k} t_{f,i} \cdot \lambda_i\right)}} \cdot \prod_{i \in I_k} \frac{\alpha_i}{\alpha_i + \lambda_i \cdot \Delta_i} \tag{1}$$

Where are:

C - capacity of entry lane (veh/h),
α - proportion of free vehicles,
$t_{c,i}$ - is the critical gap for each entry lane (s),
$t_{f,i}$ - follow-up time for each entry lane (s),
Δ - minimum headway of circulating vehicles (s),
λ_i - Cowan's M3 parameter,
k - minor flow index,
I_k - set of major flows i that conflict with minor flow k.

For capacity calculation of two-lane roundabout previous equation can be expressed in more simple form: equation (2) for the conflict of one circular lane and one entry lane and equation (3) for the conflict of two circular lanes and one entry lane.

$$C = \frac{q \cdot \alpha \cdot e^{\left[-\lambda \cdot \left(t_c - \Delta\right)\right]}}{1 - e^{\left(-\lambda \cdot t_f\right)}} \tag{2}$$

$$C = \frac{e^{\left\{-\left[\lambda_1 \cdot \left(t_{c,1} - \Delta_1\right) + \lambda_2 \cdot \left(t_{c,2} - \Delta_2\right)\right]\right\}} \cdot \left(\lambda_1 + \lambda_2\right) \cdot \alpha_1 \cdot \alpha_2}{1 - e^{\left[-\left(t_{f,1} \cdot \lambda_1 + t_{f,2} \cdot \lambda_2\right)\right]} \cdot \left(\alpha_1 + \lambda_1 \cdot \Delta_1\right) \cdot \left(\alpha_2 + \lambda_2 \cdot \Delta_2\right)} \tag{3}$$

Parameters α, λ and Δ are the parameters of the Cowan's distribution. Parameter λ can be expressed by the following equation:

$$\lambda = \frac{\alpha \cdot q}{1 - \Delta \cdot q} \tag{4}$$

while α is represented as Tanner's model:

$$\alpha = 1 - \Delta \cdot q; \quad \Delta = 2\,\mathrm{sec} \tag{5}$$

The greatest advantage of this method is its lane by lane approach, which makes it suitable for multi-lane roundabouts or some special modification of standard types of roundabouts (e.g. turbo-roundabouts). For this purposes, it is necessary to make field measurements for every pair of entry lane and circular lane. For example, a two-lane roundabout with two-lane approach requires four different sets of gap-acceptance parameters.
HCM 2010 methodology is widely accepted and used method for capacity calculation. It is also an analytical method but with some data based on US experience on roundabouts. However, it can be calibrated with local gap-acceptance values. The capacity formula for two-lane roundabouts is given separately for the right (equation 6) and the left (equation 7) entry lane:

$$C_{e,R,pce} = 1130 \cdot e^{\left(-0{,}70 \cdot 10^{-3}\right) \cdot v_{c,pce}} \tag{6}$$

$$C_{e,L,pce} = 1130 \cdot e^{\left(-0{,}75 \cdot 10^{-3}\right) \cdot v_{c,pce}} \tag{7}$$

These two equations can be expressed in more general form (equation 8) which allows its calibration. Critical gap and follow-up headway can be used as part of equations 9 and 10.

$$C = A \cdot e^{\left(-B \cdot v_c\right)} \tag{8}$$

$$A = \frac{3600}{t_f} \tag{9}$$

$$B = \frac{t_c - \frac{t_f}{2}}{3600} \tag{10}$$

Fully description of the HCM methodology for the capacity evaluation of roundabouts can be found in HCM 2010: chapters: 21. Roundabouts, and 33. Roundabouts Supplemental.

When it comes to traffic distribution pattern and its influence on capacity, one more thing is significant for two-lane roundabouts. At least two approaches should be double lane on two-lane roundabouts. This geometric characteristic means that vehicles can use both right and left lane before entering the roundabout. Traffic distribution between these two lanes is often unequal because most drivers use right entry lane and outside circular lane, no matter which exit they will take. This is due to safety uncertainty when driver wants to exit from the inner circular lane, especially if that exit has only one lane. In practice, this misbalance is often ignored. HCM 2010 gives too simple recommendations which in most cases lead to 53-47 % traffic distribution in favour of the right lane. However, HCM approach is used in this paper for the sake of simplification of numerous capacity calculation.
In both methods, local and field-determined gap-acceptance values were used (Table 1). These values are obtained in our previous researches and have been published in paper [8].

Table 1 Table 1. Gap acceptance parameter for two-lane roundabout [8]

Left entry lane				Right entry lane			
Inner circular lane		**Outside circular lane**		**Inner circular lane**		**Outside circular lane**	
t_c (sec)	t_f (sec)	t_c (sec)	t_f (sec)	t_c (sec)	t_f (sec)	t_c (sec)	t_f (sec)
3,84	2,92	3,84	2,92	2,80	2,60	3,26	2,97

3 Capacity evaluation based on different traffic flow distribution

Using both described methods capacity was calculated for the standard type of four-leg two-lane roundabout (Figure 4). Different traffic flow distribution scenarios were used. In total, seven theoretical traffic conditions were implemented as origin-destination matrices (Figure 5). Elements of these matrices are the percentage of each traffic movement between two approaches. In every scenario, major and minor traffic flow started from 200 veh/h with step of 200 veh/h. The upper limit for both flows was 1.000 veh/h or when the degree of saturation was above 0,90. This leads to 60-80 capacity values for every scenario. Major or minor flow represent the sum of traffic volume on two approaches, and U-turns were not considered.
Seven scenarios were chosen in order to cover most of the usual distribution of traffic movements. In scenario 1, traffic volume is equally distributed on all approaches. Scenarios 2 and 3 have dominant through movement on all approaches, which is typical for arterial roads. In scenarios 4, 5 and 6 major flows are retained as in the previous ones, while minor approaches have dominant right movement excluding left (scenarios 4 and 5) or through movement (scenario 6). These situations are common, for example, near shopping centres. In scenario 7 both major and minor approaches have dominant right movement.
Detailed results of the capacity calculation for each scenario and traffic combination will not be presented. All results are summarized and presented in the form of applicability diagrams (Figures 6, 7 and 8). The upper limit for capacity calculation under different traffic flow distribution is set at volume to capacity (v/c) ratio of 0,90.

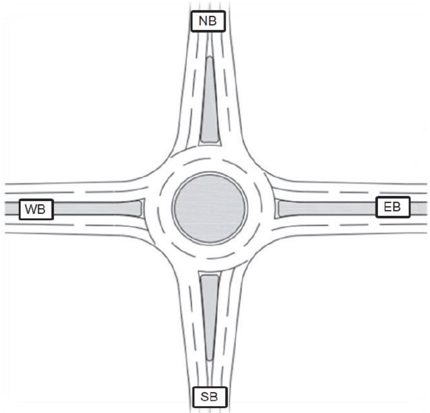

Figure 4 Two-lane roundabout

	Scenario 1				Scenario 2				Scenario 3				Scenario 4			
	WB	**EB**	**SB**	**NB**	**WB**	**EB**	**SB**	**NB**	**WB**	**EB**	**SB**	**NB**	**WB**	**EB**	**SB**	**NB**
WB	0	0.33	0.33	0.33	0	0.7	0.15	0.15	0	0.8	0.1	0.1	0	0.80	0.10	0.10
EB	0.33	0	0.33	0.33	0.7	0	0.15	0.15	0.8	0	0.1	0.1	0.80	0	0.10	0.10
SB	0.33	0.33	0	0.33	0.15	0.15	0	0.7	0.1	0.3	0	0.6	0.00	0.50	0	0.50
NB	0.33	0.33	0.33	0	0.15	0.15	0.7	0	0.1	0.3	0.6	0	0.50	0.00	0.50	0

	Scenario 5				Scenario 6				Scenario 7			
	WB	**EB**	**SB**	**NB**	**WB**	**EB**	**SB**	**NB**	**WB**	**EB**	**SB**	**NB**
WB	0	0.8	0.1	0.1	0	0.8	0.1	0.1	0	0.30	0.60	0.10
EB	0.8	0	0.1	0.1	0.8	0	0.1	0.1	0.30	0	0.10	0.60
SB	0	0.8	0	0.2	0.2	0.8	0	0	0.10	0.60	0	0.30
NB	0.8	0	0.2	0	0.2	0.8	0	0	0.60	0.10	0.30	0

Figure 5 Traffic flow distribution scenarios

On the applicability diagrams, two different zones are marked:

- Green zone – two-lane roundabout can be applied regardless of the capacity calculation method and traffic scenario,
- Blue zone – two-lane roundabout can be applied conditionally, which means that for some scenarios and calculation method degree of saturation is higher than 0,90
- Field data for one two-lane roundabout operating in Sarajevo were used to compare those w

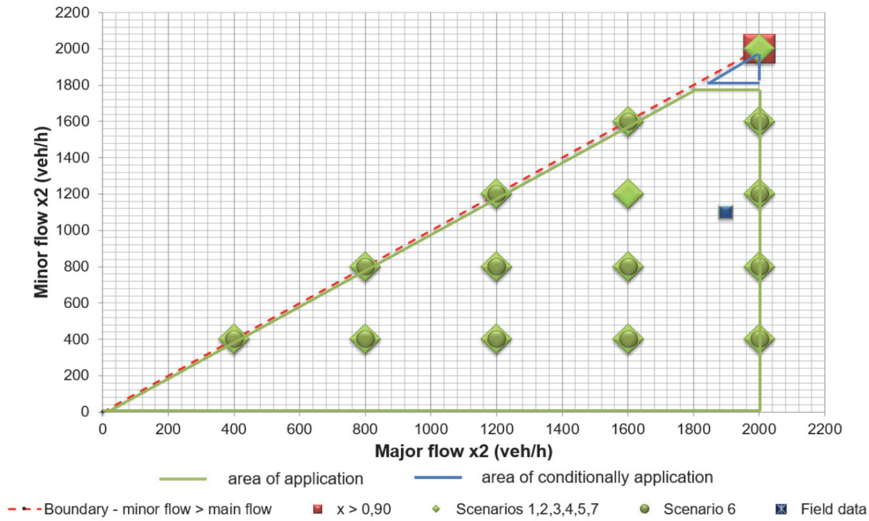

Figure 6 Applicability of two-lane roundabout based on HCM methodology

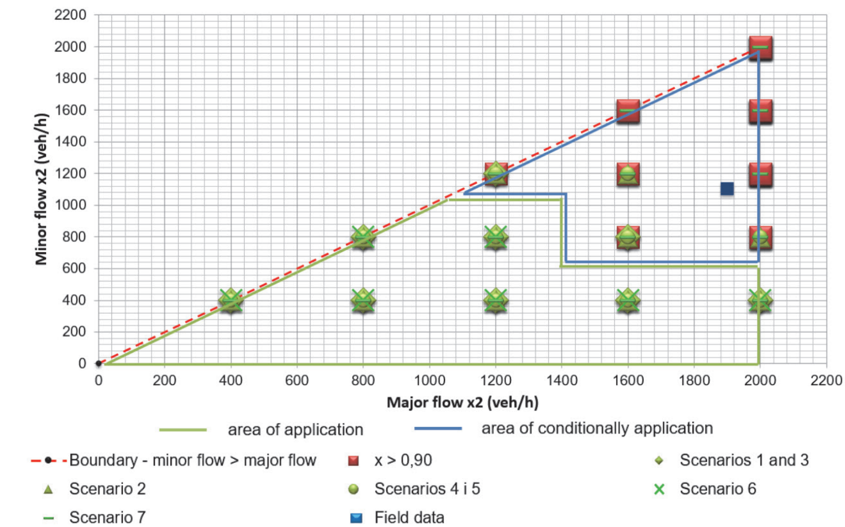

Figure 7 Applicability of two-lane roundabout based on Hagring methodology

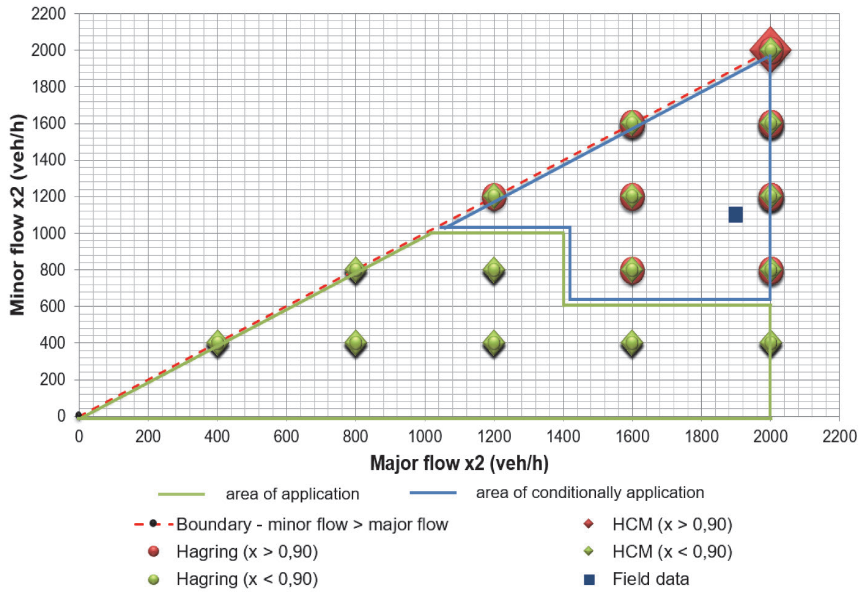

Figure 8 Applicability of two-lane roundabout based on both methodology

Based on presented figures, several conclusions can be drawn:
- HCM methodology allows the application of two-lane roundabouts in almost all traffic scenarios, because only small part of the diagram, where both flows are 2000 veh/h, is conditionally applicable.
- Hagring method is more conservative compare to the HCM methodology and leads to a smaller overall capacity in several scenarios. Conditionally application area starts at 600-800 veh/h for major flows and 600-1000 veh/h for minor flows.
- In both methodologies, there is no unapplicable area which means that at least one traffic scenario did not reach an upper limit of degree of saturation.

Compare to the diagrams in HCM 2016, it is clear that results from this study fully cover the proposed applicability area. However, looking only HCM methodology application of two-lane roundabouts is possible regardless of traffic distribution pattern or traffic volume. Calculation based on Hagring method is more strict than the HCM method, which leads to smaller capacity values and application area. The main reason for this is the usage of four different sets of gap-acceptance parameters in order to describe driver behaviour better.
Overall, two-lane roundabouts can be applied unconditionally for major flows up to 1400 veh/h and minor flows up to 1000 veh/h, and for major flows up to 2000 veh/h and minor flows up to 600 veh/h. For the same major flows values and higher values of minor flows (upt to 2000 veh/h) two-lane roundabouts can be applied with some restrictions due to traffic distributions. In all scenarios with a clear unequal traffic distribution, regardless of the type of dominant movement, two-lane roundabouts have limited application for simultaneously high values of both major and minor flows.
In addition to theoretical scenarios, capacity was calculated for one existing two-lane roundabout. Based on obtained diagrams and existing traffic volumes on this roundabout, it was to be expected that it would be the conditionally right solution. Results of the capacity analysis showed that HCM 2010 provides a satisfied degree of saturation for all approaches. At the same time, according to Hagring methodology, not all movements have a degree of saturation below 0,90. This is consistent with the final diagrams presented of Figure 8.

4 Conclusion

Roundabouts are a widely accepted solution dealing with the different traffic problems. Their capacity is still one of the key element for their introduction in urban areas. Capacity calculation method and applicability of two-lane roundabouts in a particular situation are two connected problems.
Most of the existing researches suggest that fully gap-acceptance method is more suitable for this type of intersections. At the same time, a single value of future capacity is not sufficient to decode on building two-lane roundabouts. It is necessary to examine different traffic volumes and different traffic distribution patterns to make a full capacity image of one intersection. In this paper, we tried to solve that problem and to overcome the shortcomings of existing manuals, with a set of theoretical traffic distributions.
The results of the capacity analysis showed that the traffic distribution from scenario 6 gives the most unfavourable results according to both methods. In scenario 6, the main WB-EB direction is predominantly loaded, while from the minor approaches, 20 % of vehicles turn left. The results showed that a relatively small number of left-turning vehicles is sufficient to affect the quality of traffic of the entire roundabout significantly. Using the HCM method in all other scenarios, the degree of saturation was less than 0,90 and even for a very high volume of over 1000 veh/h in both directions. The most favourable scenario is 7 in which the degree of saturation is a maximum of 0,60 regardless of traffic volume, which is a consequence of a specific distribution of movement dominated only by right turns that do not conflict with other flows. Compared to the HCM method, the Hagring method shows significantly lower results in terms of capacity. Again, scenario 6 is the most unfavourable and 7 the best, however, oversaturated conditions also occur in all other scenarios for loads higher than 1000 veh/h on the major approach and 400 veh/h on the minor approach.

References

[1] Highway Capacity Manual: Practical Applications of Research. U.S. Dept. of Commerce, Bureau of Public Roads, 2000.

[2] National Academies of Sciences, Engineering, and Medicine: Planning and Preliminary Engineering Applications Guide to the Highway Capacity Manual, Washington, DC: The National Academies Press, 2016, doi: https://doi.org/10.17226/23632

[3] Hagring, O., Rouphail, N.M., Sørensen, H.A.: Comparison of capacity models for two-lane roundabouts, 82nd annual meeting of the Transportation Research Board, January 2003 Washington, DC.

[4] Highway Capacity Manual: Practical Applications of Research, U.S. Dept. of Commerce, Bureau of Public Roads, 2010.

[5] Hagring, O.: Estimation of critical gaps in two major streams, Transportation Re-search Part B 34, 2000.

[6] Giuffrè, O., Granà, A., Tumminello, M.L.: Gap-acceptance parameters for roundabouts: a systematic review, European Transport Research Review, 8 (2016)

[7] Çalişkanelli, P., Özuysal, M., Tanyel, S., Yayla, N.: Comparison of different capacity models for traffic circles, Transport, 24 (2009) 4, pp. 257-264

[8] Šarić, A., Albinović, S., Čaušević, J.: Practical Determination of Gap-Acceptance Parameters on Roundabouts, 5th International Conference on Road and Rail Infrastructure – CETRA 2018, Zadar, Croatia, 17-19 May 2018.

CHARGING POWER OPTIMISATION FOR ELECTRIC BUSES AT TERMINALS

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Abstract

Charging infrastructure has a key role in the operation of electric buses in public transportation. In this paper, mixed-integer linear programming was used to model the bus service and capture the relationship among the network characteristics, vehicles, and charging unit attributes. The model supports the charging power optimisation at terminals to reduce the total operating costs of electric buses and charging units. The model was applied for the bus network of Kőbánya, Budapest. It was found that despite using more expensive high-power chargers, the total cost is lower because of the lower number of electric buses. It was also found that higher charging power does not affect the total cost significantly if it is higher than 350 kW.

Keywords: electric bus, static charger, charging power, optimisation

1 Introduction

Despite the technological improvements in vehicle drivetrains, the emission of road transport increased in recent years. The operation of pure electric vehicles is an efficient tool to decrease the local carbon dioxide (CO_2) emission (e.g., [1-3]). In the case of city buses, the CO_2 reduction is especially significant because of the high mileage. Accordingly, public bus service providers are making an effort to reduce the carbon footprint by operating electric buses. Between 2014 and 2019, the number of electric buses in Europe has increased from around 200 to 2200 [4]. The overarching objective of this paper is to elaborate an electric bus charging infrastructure optimization tool for public transport operators that aids in reducing the cost of electrification. Electric buses are mainly charged in a stationary position. This paper focuses on stationary charging; however, electric buses may be charged in movement using dynamic chargers (e.g., catenary). On the base of the location, the following stationary (static) charging strategies are distinguished:
- charging at the depot,
- daytime charging at terminals,
- daytime charging at terminals and stops.

The buses are usually charged at the depot overnight. Thus, the long charging time does not have an adverse effect on the operation, and there is no need for high-power charging units. On the other hand, the high battery capacity increases the purchase cost of electric buses, and additional charging significantly during the daytime may increase dead mileage. Fur-

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thermore, 1 charging unit can serve only 1 bus overnight. Therefore, charging at the depot is proposed in the case of low electric bus numbers, and daytime charging strategies are more favourable in the case of high electric bus numbers. In general, conductive charging units are used at terminals because of the higher dwelling time, and wireless chargers are used at the stop because the bus may be charged immediately as soon it is in a stationary position. Furthermore, wireless chargers have a higher purchase price and lower energy efficiency. According to the experiences, conductive charging is considered mature, and the maximum charging power of conductive chargers is significantly higher than inductive chargers [5].
The charging power of static chargers varies on a wide scale. High charging power may reduce the charging cost significantly but increase the deployment cost. On the other hand, low charging power decreases the deployment cost and decreases the utilisation rate of buses. Therefore, the optimal charging power should be derived from the characteristics of the bus service. Accordingly, a mathematical model was elaborated to optimise the charging power at the terminals. The structure of this paper is the following: after a brief literature review in Section 2, the model of charging infrastructure deployment is elaborated in Section 3. In Section 4, the application of the model is presented, and the result is discussed. Finally, the conclusion has been drawn, and directions for future research are given.

2 Literature review

Several papers focus on technology, environmental effects, energy management, and cost-benefit analysis (e.g., [6-10]). The number of studies dealing with the charging infrastructure of electric buses increased significantly recently. It was noted that most of the papers either focus on a static (e.g., [11-12]) or dynamic (e.g., [13-14]) charging infrastructure. Furthermore, the optimal charging infrastructure was determined for specific bus lines instead of the bus network in several papers (e.g., [15]). Albeit charging infrastructure deployment based on the network characteristics may significantly decrease electrification cost [11]. In general, charging infrastructure deployment is based on the modelling of the electric bus service. A mixed-integer linear program was elaborated to determine the optimal fleet composition considering battery electric buses and other fuel alternatives, such as biodiesel and biogas [12]. Major public transport hubs and terminals were considered as candidate sites. The relationship between charging power and the number of buses was not investigated. The deterministic and robust planning of dynamic wireless charging infrastructure was elaborated, considering the uncertainty of energy consumption and travel time in [14]. It was found that the deterministic model may effectively determine the allocation of charging infrastructure. Separated models were elaborated for static, dynamic charging, and battery swapping strategies in [16]. It was found that static charging is less cost-effective. However, the comparison was performed assuming low power at static chargers (90kW). The results also suggested that the service frequency, circulation length, and operating speed of a transit system may significantly impact various charging strategies' cost competitiveness. Mixed-integer second-order cone programming was used to formulate static charging stations' deployment at candidate sites into an optimization problem in [17]. Joint optimization of the bus service characteristics and the power grid was conducted. The effect of various charging power on fleet size was not analysed. A stochastic program was developed to optimize the fleet size and charging stations' locations for electric buses in [18]. The charging demand was aggregated at bus terminals. Electric load-dependent tariff and the uncertainty of weather and traffic conditions were considered. A mixed-integer linear program was elaborated to optimize the charging infrastructure at depots and determine the optimal fleet size and composition considering various electric bus types in [19]. However, the charging power of buses was different; the relationship between charging power and fleet size was not investigated in detail. Papers dealing with electrification on a higher level can also be found. The fleet electrifica-

tion problem was formulated into an integer linear program based optimization problem in [20]. Besides purchase and operational costs, various charging technologies, such as slow and fast plug-in stations, catenary, and wireless chargers, are considered. However, the model does not support decisions regarding where to locate the charging infrastructure.
According to state of the art, the relationship between the charging power at static chargers and the fleet size was not investigated. On the one hand, the infrastructure cost of a high-power charger is significantly higher [21]. On the other hand, a high-power charger may decrease the total time spent with charging significantly. Thus, the less electric bus may be enough. This study hypothesises that the lower fleet cost of electric buses exceeds the higher infrastructure cost of high-power chargers.

3 Model

The model of public bus service was elaborated to optimize the charging power. The aim was to define the cost of electrification as a function of charging power. In other words, the optimal charging power is where the cost of electrification is the lowest. In the physical model, the assumptions and limitations of the operation were defined. In the mathematical model, the electric buses' charging was modelled in consideration of the energy consumption and capacity limitations.

3.1 Physical model

The focus was put on the bus lines. One turn along the entire route was analysed for each bus line. The following assumptions were made:
- Each bus line is served with a homogeneous bus fleet.
- Buses operating on various lines may differ.

The dwelling times at terminals are given by the schedule. The specific arrival and departure times were not considered. The aggregated energy consumption of one turn was considered. The following limitations were applied for the charging units:
- The charging power at a terminal is constant.
- The aggregated cost of a charging unit may contain both the deployment and operational costs.
- The total cost of charging units at a terminal is the aggregated cost multiplied by the number of units.
- The capacity limitation of the power network was not considered. In other words, the cost of power network development was not considered.

3.2 Mathematical model

Since the operation of buses is controlled by the schedule, a deterministic modelling approach was applied. The model of public bus service was formulated into a mixed-integer linear program based optimisation problem. The parameters of the bus service are summarised in Table 1.

Table 1 Model parameters

Category	Parameter	Description
Terminal specific	c	Cost of charging unit [€]
	cu	Number of charging unit, integer [-]
	μ	Effective charging time [-]
	P	Effective charging power [kW]
Bus line specific	e^-	Energy consumption of a turn [kWh]
	f	Number of departures in peak hour [-]
Terminal and bus line specific	e^+	Amount of charged energy
	t	Dwelling time at terminal [h]

The schedule and the technology significantly influence the effective charging time at a charging unit. The arrival and departure times determine the periods when a bus may be charged. The uneven distribution of the charging periods may decrease the effective charging time. Connecting and re-connecting times at conductive charging units also have an adverse effect on the effective charging time. Therefore, the effective charging time parameter (μ) is introduced to consider these phenomena. The charging infrastructure is determined based on the highest energy demand, which occurs during the peak hour. Dwelling time is the available time to recharge a bus at a terminal between arrival and departure.
The parameters are either one- or two-dimensional. The one-dimensional parameters are either terminal or bus line specific. These are row and column vectors, respectively. Two-dimensional parameters are both terminal and bus line specific. These parameters are matrices. The objective of the optimisation is to minimise the total cost of charging units. Accordingly, the objective function is given in Eq. (1).

$$min\left(\sum_{j=1}^{m}\left(c_j \cdot cu_j\right)\right) \tag{1}$$

Where c_j is the cost of a charging unit at terminal j, cu_j is the number of charging units at terminal j, and m is the number of terminals.

3.3 Constraints

The solution of the optimisation is valid if the following constraints are satisfied:
- Eq (2): the total charged energy is higher than the energy consumption.
- Eq (3): the total energy demand is lower than the charging capacity.

An additional electric bus should be assigned to the bus line if the following condition is not met:
- Eq. (4): the charging time is lower than the dwelling time.

$$\sum_{j}^{m} e^+_{i,j} = e^-_i \quad \forall i = 1..n \tag{2}$$

$$\sum_{i}^{n}\left(f_i \cdot e^+_{i,j}\right) \le cu_j \cdot P_j \cdot \mu_j \quad \forall j = 1..m \tag{3}$$

$$e^+_{i,j} \big/ P_j \le t_{i,j} \quad \forall i = 1..n \quad and \quad j = 1..m \tag{4}$$

Where i indicates the bus line and n is the number of modelled bus lines.

4 Case study

4.1 Simulation

The model was applied for the bus network of Kőbánya, Budapest. The peak hour is between 7 and 8 AM. The aim was to minimise the cost of electrification. The cost of electrification consists of the cost of electric buses and the cost of charging infrastructure. 5 terminals were considered as candidate sites for charging units. In sum, 19 bus lines were considered. 15 bus lines may be charged at 1 terminal (strict demand), 4 lines may be charged at 2 terminals (flexible demand). The total strict and flexible demand were 1607 and 340 kWh at the peak hour, respectively. Homogeneous charging infrastructure was assumed. Namely, the charging power is equal for each charging unit. The effect of schedule adjustments on the number of electric buses was not analysed. The number of charging units (cu) and the amount of charged energy (e^+) were the variables. The parameters of the optimisation are as follows:

c - $444 per kW [15],
μ - 0.7,
P - several runs were performed with charging power varying between 100 and 450 kW,
e^- - estimated based on the length of the entire route. Solo bus: 1.2kWh/km, articulated bus: 1.5kWh/km,
f - given by the schedule,
t - time spent at the terminal (given by the schedule) minus 2 minutes because of terminal movements.

The utilization of the charging infrastructure (u) was calculated as the rate of total charging demand and the charging capacity (Eq. (5)).

$$\left(1607 \text{ kWh} + 340 \text{ kWh}\right) \Big/ \left(\sum_j cu_j \cdot \mu_j \cdot P_j\right) = u \tag{5}$$

The built-in intlinprog function was used in MATLAB.

4.2 Results and discussion

The total number of charging units, the number of additional electric buses, and the utilisation are summarised in Table 2.

Table 2 Simulation results

P [kW]	100	150	200	250	300	350	400	450
Σcu	29	20	15	12	11	9	8	7
Additional electric buses	15	12	12	8	3	1	1	1
u [%]	96	93	93	93	84	88	87	88

It is noted that there is a strong relation between charging power and the number of additional electric buses. Namely, the dwelling time at terminals is not enough to recharge an electric bus. In other words, in the case of low charging power, more electric buses are needed to replace a conventional diesel bus. It is also noted that the charging power does not influence the utilization of the charging infrastructure significantly.

The cost of electrification consists of the purchase price of additional electric buses and the cost of charging infrastructure. The cost of an electric bus was 580000 € [22]. The number of additional electric buses significantly influences the cost of electrification. Accordingly, it is advised to increase the charging power if the number of additional buses decreases. In this case, the optimal charging power is 350 kW. The hypothesis of the study has been confirmed. The relation between the cost of electrification and charging power is given in Fig. 1.

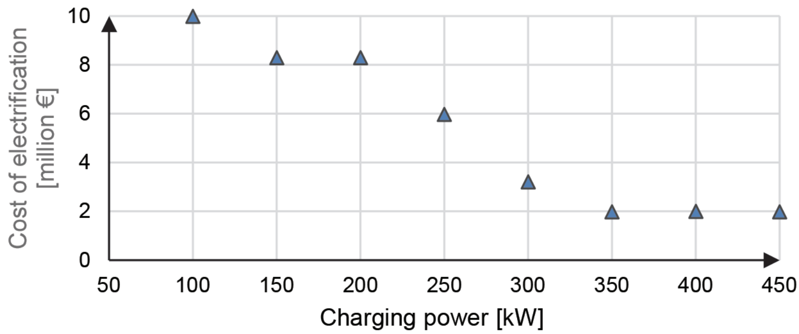

Figure 1 Cost of electrification according to various charging power

5 Conclusion

A mixed-integer linear program was elaborated to support the deployment of charging infrastructure at terminals in consideration of the bus service characteristics. The application of the model indicates that the model supports the minimalization of the cost of electrification. The paper's key finding is that it is advised to increase the charging power if the number of electric buses decreases. Although the trend is to increase the charging power, charging power higher than 350kW did not affect the operation of electric buses, according to the case study. The future direction for the research is to consider other charging technologies, such as wireless and dynamic chargers. Furthermore, other candidate sites, such as bus stops and sections, should be modelled.

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References

[1] Li, C., Cao, Y., Zhang, M., Wang, J., Liu, J., Shi, H., Geng, Y.: Hidden benefits of electric vehicles for addressing climate change, Scientific Reports, 5 (2015), 9213 doi: 10.1038/srep09213

[2] Ensslen, A., Schücking, M., Jochem, P., Steffens, H., Fichtner, W., Wollersheim, O., Stella, K.: Empirical carbon dioxide emissions of electric vehicles in a French-German commuter fleet test, Journal of Cleaner Production, 142 (2017), pp. 263-278

[3] Kolbe, K.: Mitigating urban heat island effect and carbon dioxide emissions through different mobility concepts: Comparison of conventional vehicles with electric vehicles, hydrogen vehicles and public transportation, Transport policy, 80 (2019), pp. 1-11

[4] Busworld, https://www.busworld.org/articles/detail/4813/increasingly-more-electric-buses 27.02.2020

[5] Xylia, M., Silveira, S.: The role of charging technologies in upscaling the use of electric buses in public transport: Experiences from demonstration projects, Transportation Research Part A, 118 (2018), pp. 399-415

[6] Ye, L., Liang, C., Liu, Y., Li, D., Liu, Z.: Performance analysis and test of a novel eddy-current braking & heating system for electric bus, Energy Conversion and Management, 183 (2019), pp. 440-449

[7] Laib, F., Braun, A., Rid, W.: Modelling noise reductions using electric buses in urban traffic. A case study from Stuttgart, Germany, Transportation Research Procedia, 37 (2019), pp. 377-384

[8] Du, J., Zhang, X., Wang, T., Song, Z., Yang, X., Wang, H., Ouyang, M., Wu, X.: Battery degradation minimization oriented energy management strategy for plug-in hybrid electric bus with multi-energy storage system, Energy, 165 (2018), pp. 153-163

[9] Jwa, K., Lim, O.: Comparative life cycle assessment of lithium-ion battery electric bus and Diesel bus from well to wheel, Energy Procedia, 145 (2018), pp. 223-227

[10] Bartłomiejczyk, M., Kołacz, R.: The reduction of auxiliaries power demand: The challenge for electromobility in public transportation, Journal of Cleaner Production, 252 (2020) 119776

[11] Bi, Z., Keoleian, G., Ersal, T.: Wireless charger deployment for an electric bus network: A multi-objective life cycle optimization, Applied Energy, 225 (2018), pp. 1090-1101

[12] Xylia, M., Leduc, S., Patrizio, P., Kraxner, F., Silveira, S.: Locating charging infrastructure for electric buses in Stockholm, Transportation Research Part C, 78 (2017), pp. 183-200

[13] Jang, Y.J., Suh, E.S., Kim, J.W.: System Architecture and Mathematical Models of Electric Transit Bus System Utilizing Wireless Power Transfer Technology, IEEE System Journal, 10 (2016), pp. 495-06

[14] Liu, Z., Song, Z.: Robust planning of dynamic wireless charging infrastructure for battery electric buses, Transportation Research Part C, 83 (2017), pp. 77-103

[15] Lajunen, A.: Energy consumption and cost-benefit analysis of hybrid and electric city buses, Transportation Research Part C, 38 (2018), pp. 1-15

[16] Chen, Z., Yin, Y., Song, Z.: A cost-competitiveness analysis of charging infrastructure for electric bus operations, Transportation Research Part C, 93 (2018), pp. 351-366

[17] Lin, Y., Zhang, K., Shen, Z.J.M., Ye, B., Miao, L.: Multistage large-scale charging station planning for electric buses considering transportation network and power grid, Transportation Research Part C, 107 (2019), pp. 423-443

[18] An, K.: Battery electric bus infrastructure planning under demand uncertainty, Transportation Research Part C, 111 (2020), pp. 572-587

[19] Rogge, M., van der Hurk, E., Larsen, A., Sauer, D.U.: Electric bus fleet size and mix problem with optimization of charging infrastructure, Applied Energy, 211 (2018), pp. 282-295

[20] Pelletier, S., Jabali, O., Mendoza, J.E., Laporte, G.: The electric bus fleet transition problem, Transportation Research Part C, 109 (2019), pp. 174-193

[21] Li, C., Huang, W., Cao, R., Bu, F., Fan, C.: An Integrated Topology of Charger and Drive for Electric Buses, IEEE Transactions on Vehicular Technology, 65 (2016), pp. 4471-4479

[22] Potkány, M., Hlatká, M., Debnár, M., Hanzl, J.: Comparison of the Lifecycle Cost Structure of Electric and Diesel Buses, NASE MORE International Journal of Maritime Science & Technology, 65 (2018), pp. 319-328

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PREREQUISITES OF THE SUCCESSFUL TRAM-TRAIN SYSTEM AS A PART OF THE REGIONAL RAILWAY NETWORK

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Abstract

The idea of connecting the two of the closest urban and suburban rail-based public transport systems was proposed more than 40 years ago and it boasts a proclaimed support from the regional authorities. However, many of the systems either did not achieve their goals or became financially inefficient as they replaced the conventional railway systems in a wider range than it had initially been planned. That resulted in an inappropriate competition between the public transportation modes. This paper aims to define the prerequisites needed for the establishment of a successful, both financially and operationally sane tram-train system as a substantial part of the regional railway network An analysis of the various current tram-train systems in the selected European cities is made and special attention is paid to their distinctive features including the offered capacity, density, technical compatibility, and operational aspects. Using the conclusions of the analysis and the current technical and operational requirements to be met, a recommendation for the design and organization of the planned tram-train lines and networks is stated, which may help the transport planners design such costly systems in a way it can use its overall advantages in favor of the passengers and an increase of the public transportation modal share.

Keywords: tram-train, suburban transport systems, regional railways, interoperability, urban tram systems, mixed operation

1 Introduction

The predecessors of the current tram-train systems have been present to urban and suburban public transport networks since their creation in the second half of the nineteenth century. As there often existed a rivalry between the railway company and the urban rail transport operator, there were separate rail-based links between the main point of interest within the agglomerations. At the same time, additional private railway companies built their cheaper local railways to connect the populated areas that could not be properly served by the mainlines and to accommodate the goods flows in the regional relations. Some of the previously mentioned tramlines survived and kept its railway-like character and sometimes their historical railway legislative status concerning their urban or suburban character (e. g. the relation Mannheim – Heidelberg in Germany or Most – Litvínov in the Czech Republic). On the contrary, the local railways suffered from their inability to face the new supply chain requirements and became slow and unattractive for both passenger and freight transport.

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Some of the perspective ones were adjusted to meet the local tram standards and accommodated light rail tram-based vehicles with the operation connected to the urban tramway system (e. g. the so-called Albtalbahn in Karlsruhe, Germany).
Although these systems might share some features with the tram-train systems, none of them can be called a proper tram-train. However, its successful operation supported the idea of connecting both rail systems with mixed operation and combining the advantages of the two different systems in one product. After a decade of planning and creating suitable legislation and financing models, the first proper tram-train vehicles emerged on the refurbished regional line between Karlsruhe and Bretten in 1992. Since then, numerous applications of the system mostly in Europe were opened and each of them differs and adds specific regional features and technical solutions [1]. Concerning the evolution of the relevant legislation, all the approaches are to be analyzed to define the cities suitable for tram-train systems introduction and its optimal technical parameters.

2 Prerequisites for the tram-train system implementation

All the statements made in this chapter as well as the following ones are based on the analysis of the selected important tram-train systems in Europe opened since 1992. As not only the different networks but also the single lines within one network are quite distinctive, the overview of the analysis results available in Table 1 and Table 2 distinguish even between the different lines within one tram-train system.

2.1 Urban and settlement structure

The cities listed in Table 1 are mostly middle-sized cities with a population between 100,000 and 350,000. However, several exceptions may call a conclusion of a middle-sized city as a tram-train system prerequisite into question. The Hague has over 500,000 inhabitants but the tram-train serves as a mere urban tram line in the city and then connects the city of Zoetermeer with almost 125,000 inhabitants. The cities in the Paris conurbation form a polycentric grid so the overall area of operation may be considered one functioning middle-sized city. Thus, two of the main urban structure models of middle-sized city and its urban agglomeration may be followed as a prerequisite – the compact regional center with mainly radial or diametric journeys and a polycentric grid with the line linking several populated places and switching from the railway to tram mode whenever it is desirable and suitable.

Table 1 Overview of operational characteristics of the chosen tram-train lines

Line	Route	Travel time [min]	Peak hour frequency [min]	Tram section	Mixed operation tram	Mixed operation trains
S41	Heilbronn – Mosbach	58	60	Yes	No	Yes
S42	Heilbronn – Sinsheim	65	30	Yes	No	Yes
C13	Chemnitz Technopark – Chemnitz Hbf – Burgstädt	38	60	Yes	Yes	Yes
C14	Chemnitz Technopark – Chemnitz Hbf – Mitweida	40	60	Yes	Yes	Yes
C15	Chemnitz Technopark – Chemnitz Hbf – Hainichen	52	60	Yes	Yes	Yes
S31	Karslruhe Hbf – Odenheim	45	20	No	No	Yes
S32	Karlsruhe Hbf – Menzingen	48	20	No	No	Yes
S4	Karlsruhe – Heilbronn – Öhringen	170	15–30	Yes	Yes	Yes
S5	Wörth (Rhein) – Karlsruhe downtown – Pforzheim	90	10/20/30	Yes	Yes	Yes
S51	Germersheim – Karlsruhe downtown (– Pforzheim)	73/118	60	Yes	Yes	Yes
S52	Germersheim– Karlsruhe Hbf – Karlsruhe Tullastraße	51	60	Yes	Yes	Yes
S6	Pforzheim – Bad Wildbad	35	30	Yes	No	Yes
S7	Karlsruhe – Durmersheim – Achern	67	30–60	Yes	Yes	Yes
S71	Karlsruhe Hbf – Malsch – Achern	50	60	No	No	Yes
S8	Karlsruhe – Durmersheim – Freudenstadt – Bondorf	168	60	Yes	Yes	Yes
S81	Karlsruhe Hbf – Malsch – Freudenstadt	87	30	No	No	Yes
RT1	Kassel city – Hofgeismar-Hümme	64	30	Yes	Yes	Yes
RT4	Kassel city – Wolfhagen	69	30/60	Yes	Yes	Yes
RT5	Kassel city – Melsungen	46	30	Yes	Yes	Yes
-	Lyon St. Paul – Saint-Bel	42	30	No	No	Cargo
-	Lyon St. Paul – Brignais	25	30	No	No	Cargo
TT	Mulhouse station – Mulhouse downtown – Thann St. Jacques	44	30	Yes	Yes	Yes
T1	Nantes – Chateubriant	67	30/60	Yes	No	No
T2	Nantes – Clisson	29	30	No	No	Yes
T11	Épinay-sur-Seine – Le Bourget	15	5	No	No	Possible
T4	Aulnay-sous-Bois – Bondy, Gargan – Montfermeil	21/27	6/12	Yes	No	No*
3	The Hague city – The Hague downtown – Zoetermeer downtown	67	10	Yes	Yes	Subway
4	The Hague city – The Hague downtown – Lansingerland-Zoetermeer	58	10	Yes	Yes	Subway
S1	Lebach-Jabach – Saarbrücken downtown – Saregueminnes	71/40	7.5/15/30	Yes	No	Yes

* the infrastructure of the line remained at the railway standards, but the actual operation is conducted according to the tram standards as far as signals and crossings are concerned

Table 2 Overview of technical characteristics of the chosen tram-train lines

Line	Propulsion	Platforms	Vehicle category	Railway line type
S41	750 V DC / 15 kV, 16.7 Hz AC	high	3-section, 37–37.6 m long, high and middle floor	mainline
S42	750 V DC / 15 kV, 16.7 Hz AC	high	3-section, 37–37.6 m long, high and middle floor	mainline
C13	600 V DC / diesel	low	3-section, 37 m long	middle-frequented regional line
C14	600 V DC / diesel	low*	3-section, 37 m long	mainline
C15	600 V DC / diesel	low*	3-section, 37 m long	mainline, low-frequented regional line
S31	15 kV, 16.7 Hz AC	high	3-section, 37–37.6 m long, high and middle floor	mainline, low-frequented regional line
S32	15 kV, 16.7 Hz AC	high	3-section, 37–37.6 m long, high and middle floor	mainline, low-frequented regional line
S4	750 V DC / 15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	middle-frequented regional line
S5	750 V DC / 15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	mainline
S51	750 V DC / 15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	mainline
S52	750 V DC / 15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	mainline
S6	750 V DC / 15 kV, 16.7 Hz AC	high	3-section, 37–37.6 m long, high and middle floor	low-frequented regional line
S7	750 V DC / 15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	mainline
S71	15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	mainline
S8	750 V DC / 15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	mainline, middle-frequented regional line, low-frequented regional line
S81	15 kV, 16.7 Hz AC	high/low	3-section, 37–37.6 m long, high and middle floor	mainline, middle-frequented regional line, low-frequented regional line
RT1	600 V DC / 15 kV, 16.7 Hz AC	low	3-section, 36.8 m long	mainline
RT4	600 V DC / diesel	low	3-section, 36.8 m long	middle-frequented regional line
RT5	600 V DC / 15 kV, 16.7 Hz AC	high/low	3-section, 36.8 m long	mainline
-	1500 V DC	low	4-section, 42 m long	low-frequented regional line
-	1500 V DC	low	4-section, 42 m long	low-frequented regional line
TT	750 V DC / 25 kV, 50 Hz AC	low	5-section, 37 m long	middle-frequented regional line
T1	750 V DC / 25 kV, 50 Hz AC	low	4-section, 42 m long	abandoned regional line
T2	25 kV, 50 Hz AC	low	4-section, 42 m long	mainline
T11	25 kV, 50 Hz AC	low	4-section, 42 m long	newly built line
T4	750 V DC / 25 kV, 50 Hz AC	low	5-section, 37 m long + 4-section, 42 m long	middle-frequented regional line
3	750 V DC	low	3-section, 36.8 m long	high-frequented regional line
4	750 V DC	low	3-section, 36.8 m long	high-frequented regional line
S1	750 V DC / 15 kV, 16.7 Hz AC	low	3-section, 37.9 m long	mainline, abandoned regional line, middle-frequented regional line

2.2 Current existence of an urban tram system

Most of the existing tram-train systems have a functioning and branched urban tramway system as their backbone for the core city. If one excludes the systems that do not use the advantages of both modes (e. g. Lyon), using the tramway infrastructure within the city borders not only enables the passengers to reach directly their goals but also saves public costs as the most expensive and complicated urban infrastructure is either already completely built or requires only small linking adjustments. Also, the existing tramway infrastructure usually enables the operator to reduce some of the existing urban lines along the tram-train route in the city (Karlsruhe, Mulhouse). Finally, contrary to the standard regional railway in the middle-sized cities, it may serve as a welcome reinforcement for the urban tram lines in the busiest sections without the need to purchase additional urban tramcars as the tram-train units are needed anyway (Chemnitz).

Therefore, the existence of an urban tramway system is an important prerequisite for the new system introduction. The only exceptions to using both tram and train mode for a longer diametric line without existing tram systems are Saarbrücken where the newly-built tram line through the city serves as a substitution of the urban tram system and Heilbronn with its creation of the urban rail system using the existing tram-train line and vehicles from the Karlsruhe system. However efficient, the coverage and frequency needed for the urban trams may not be reached with the tram-train lines alone (see Section 3.4).

2.3 Types of suitable railway lines

Among the analyzed tram-train lines, the most frequent railway line type is a local railway with poor or even no passenger railway transport and initially questionable significance to the system. The line may be typically accustomed to a tram-train operation and upgraded so the travel time reduction effects caused by higher operational speeds, better acceleration, and direct connection to the city center can be synergic [2]. The freight transport on these lines is usually conducted only during the off-peak hours or at night. Apart from this, the tram-train units are the only vehicles on that kind of line, so its capacity does not restrict the desired tram-train operational concept.

The second type of railway line is a middle-frequented line where there is a potential for the tram-train to form the slow operation layer whereas the conventional railway vehicles serve as a fast or semi-fast train. This two-layered or zone-oriented operational scheme [3] requires a higher level of coordination between the two modes and leaves less room for modifications of the transport supply (e. g. line S5 between Karlsruhe and Pforzheim).

The last railway line type is the mainlines with operational speeds often up to 160 $km.h^{-1}$. Their use is limited as the slow tram-train units are not fully able to utilize the line speed potential and are present only when the dynamic aspect of the tram-train vehicles make a difference with the frequent stopping of the slowest operational layer (line RT5 Kassel) or when there is a railway connection to the more distant local branch line via mainline (line C15 Chemnitz, S31/S32 Karlsruhe). A special category is a newly built mainline reserved only for the tram-train operation (line T11 Paris) that denies its financial and operational advantages.

To sum up, with few exceptions in most of the listed systems the tram-train lines serve as a useful complement to both urban and regional public transport grid. In the urban area, it creates desirable enforcement for the local tram system (if applicable) and sometimes even takes over some of the urban passenger transport requirements (line S5 Karlsruhe or the Hague tram-train lines). Only if the tram-train is built as a substitution of a tram system (Saarbrücken, Paris T4) it may serve as an arterial system within the city borders. In the region, however, it can only fit into the regional railway scheme when applied to those railway lines where it can use its agility and dynamics and surpass traditional railway vehicles. As there is a very limited number of these lines in each candidate region, it is to be expected that an attempt of creating a tram-train network using each railway line according to Karlsruhe is not likely to be realized again. According to most of the analyzed lines, a new tram-train may stay an unconventional complementary way how to improve the regional railway network but even for the candidate cities, it may not form the backbone of the rail transport grid.

3 Analysis of the tram-train system operation

3.1 Travel time

As seen from Table 1, most of the lines have the overall travel time close to 60minutes whereas the average travel is 62.28 min. It covers the well-known isochrone of the daily commuting which is about an hour door-to-door travel time. The line may even exceed this value as the terminus is usually located in the suburbs after having reached the city center (Kassel, Chemnitz, Mulhouse). Only a few Karlsruhe lines have significantly higher travel time as it substitutes the regional railway on long line branches and creates an undesirable connection unattractive for commuting to the core city. On the contrary, the former Kassel line to Treysa was already closed and replaced by conventional railway as the travel time was over an hour and therefore unattractive for the commuters.

3.2 Operation frequency

Concerning the earlier mentioned complementary role of the tram-train lines in the transport system, most lines have the corresponding frequency of 30 minutes during peak hours and 60 min off-peak (or at least 60 min all the time). However, there is an exception if the systems serve as a substitution of the urban tram line accommodating an individual urban transport flow [4]. In this case, the requirements of the inner-city section are much higher and the frequency of 10 min (line S5 Karlsruhe) or even less (Saarbrücken, line T4 Paris, lines 3 and 4 in the Hague) are provided. The major disadvantage of the tram-train substituting the urban tram is the much bigger number of vehicles needed for the operational concept in the comparison with the regional line only. As the rather complicated vehicles are the most expensive part of the tram-train operation, it creates an inadequate expensive tramline in the core city.

3.3 Utilization of the tram-train advantages

Tram-train vehicles are due to their complexity more expensive than either regional railway units of the appropriate capacity or tramcars. The higher purchase price must be compensated by the utilization of its advantages over both more conventional systems. Two main tram-train operational features should be applied if the tram-train system is planned – the operation in both modes (tram and railway) and mixed operation with the railway vehicles (at least regarding the same infrastructure). However, not all the analyzed systems can fulfill that condition.
The fulfillment of the conditions is to be seen mainly by the oldest system in Karlsruhe but not with all lines as some operate only on railway infrastructure and do not have any tram sections. A good example is set by the systems in Kassel [5], Chemnitz, or Mulhouse where there is a mixed operation with both other modes and the tram-train serves as a complement to the regional network and a reinforcement of the urban tram network. In the Hague, the tram-train is in mixed operation with urban railway mode, the Rotterdam subway line E. In Saarbrücken on the line T4 in Paris with their absence of urban trams both tram and train sections are present. On the other hand, there are several cities with undesirable isolation

of the system, most of which are in France. The Lyon system is operated only on refurbished railway lines, it is not connected to the city´s existing tram system and it does not bring any travel time reductions or direct connections to the city center whatsoever. A similar example can be found in Nantes where there is one short tram section with no connection to the tram system and no mixed operation with the trains and regarding the second line, the operation is conducted on mainline with no difference to the already operated regional trains. Such examples deny the tram-train system function and do not bring the desired improvements concerning the higher investment made into the vehicles or line refurbishment and may be fully substituted by cheaper regional rail.

4 Recommendations for new systems

The recommendation given in this section is based on the analysis of the current systems conducted in the previous sections. The technical parameters of the future systems correspond to the environment of the Czech Republic with its tram networks suitable for a tram-train system introduction [6] and its general principles can be used also in similar central European countries without tram-train operation (e. g. Poland or Slovakia).

4.1 Urban agglomeration size and residential structure

Although there are seven tram networks in the Czech Republic, some may be excluded in advance. The capital city of Prague is too populated, and its capacity demands cannot be satisfied by radial or diametric tram-train line. A similar situation is in Brno or Plzeň where all railway lines heading to the city are suitable for modern regional trains. However, two examples of the urban structures mentioned in Section 2.1 are to be followed. There is a compact city with the solid urban structure and prevailing radial journeys and suitable regional and middle-frequented regional lines in the Olomouc Urban Agglomeration. The Ostrava Conurbation with its industrial railways connecting different cities of its polycentric structure may serve as an example of another suitable location.

4.2 Railway line types

As the main railway lines are often busy and sufficient modern railway vehicles for the slowest operational layer are currently being introduced, it is recommended to use this kind of lines only if there is no other way how to reach the linked branch and regional lines. Even then, it is expected for the tram-train to divert from the mainline on the suburb reaching the built-up areas in tram mode and continuing to the city center. Such a concept may be implemented in the Ostrava region.

More important railway line types are the middle-frequented lines with one or two-layered operation. The dynamic tram-train vehicles may compensate for the lower speeds and the tram-train may be used in mixed operation with electric or diesel-powered long distance or semi-fast trains using the zone-oriented timetable and operating in the inner zone. This concept is suitable for example for the railway line Olomouc – Opava [7].

The most promising application of the tram-train seems to be on the poorly used or even disused regional and industrial railway lines. Due to the higher tram-train frequency, the mixed operation is expected only in off-peak or night hours and only with cargo trains. As there is the biggest freedom of operational concept that kind of lines are to be preferred in Ostrava (industrial lines heading to Orlová) and Olomouc (regional limes to Senice na Hané and Litovel).

4.3 Technical aspects

The platform height should be adjusted to the platform heights reachable in the current urban tram systems with the usual height up to 240 mm above the track level. With the usual wear and tear of the wheels considered, the acceptable urban platform height is 300 mm. In the suburban sections [8], the height should be 350 mm and if the mixed operation with passenger trains is present, the number of stations with dual platform height or separate platforms should be minimized.
All sections are to be electrified for economic, dynamic, and ecological reasons. The voltage of the tram-train system should correspond to the voltage of the tram system (600 V) and the local voltage (3 kV DC or 25 kV, 50 Hz AC) should be applied. If the conversion to the alternative current is foreseen (as it is the case with the Czech Republic and Slovakia), the AC electrification is recommended regardless of the prevailing voltage in the core city railway node.
From the vehicle categories, the 3-section 8-axle 37 m long unit is recommended as it fits into the existing tramway infrastructure and its capacity is suitable for regional lines. Also, its layout enables the separation of urban and suburban passengers.

5 Conclusion

The paper provides a very detailed and current overview of the technical characteristics of the chosen European tram-train operations. As it was experienced, a tram-train system is a rather diverse means of transport that is always uniquely adjusted to the needs of a region. Although a certain decline of the new systems was to be spotted, with the right parameters the system can utilize its advantages and complement the remaining urban and suburban public transport network. With a huge degree of adaptation to the new interoperability and ETCS regulations, the tram-train introduction will remain a support act of the more efficient and comfortable rail-based alternative which enforces the regional public transport modal split and helps to compete against the growing car ownership in the region. Appropriately created operational concept of the tram-train system, using best-practice approaches from existing operations, can thus form a very effective part of the transport system within Smart-Regions and the frequently discussed concept of Smart-Cities [9, 10].

References

[1] Naegeli, L., Weidmann, U., Nash, A.: Checklist for Successful Application of Tram-Train Systems in Europe, Journal of the Transportation Research Board, 2012, pp. 39–48, doi: https://doi.org/10.3141/2275-05

[2] Kampf, R., Gašparík, J., Kudláčková, N.: Application of different forms of transport in relation to the process of transport user value creation, Periodica Polytechnica Transportation Engineering, 40 (2012) 2, pp. 71-75,doi: 10.3311/pp.tr.2012-2.05

[3] Vávra, R., Janoš, V.: Comparison of Different Conceptions of Suburban Railway Transport, Smart City Symposium Prague, May 2019, pp. 1-5

[4] Lorenz, F., Teichmann, D., Farana, R.: Network synchronization of periodic multiple source flows, 16th International Carpathian Control Conference (ICCC), pp. 300-303, Szilvasvarad, 2015, doi: 10.1109/CarpathianCC.2015.7145093

[5] Holzapfel, H., Meyfahrt, R.: The regional tram-train of Kassel, Germany: how regional responsibility leads to local success, Handbook on Transport and Development, 2015, doi: 10.4337/9780857937261.00050

[6] Kołoś, A., Taczanowski, J.: The feasibility of introducing light rail systems in medium-sized towns in Central Europe, Journal of Transport Geography, 54 (2016), pp. 400-413

[7] Metelka, S.: Možnosti zavedení vlakotramvají v aglomeraci města Olomouce, master thesis, Praha: ČVUT, 2019.

[8] Hofer, K., Fellendorf, M., Smoliner, M., Walter, S.: Strategic Multimodal Assessment of Suburban Transport Infrastructure, 7th Transport Research Arena TRA 2018, Vienna, Austria, 2018, doi: 10.5281/zenodo.1491585

[9] Vinoj, J., Gavaskar, S., Rameshkumar G.P., Priya, R.: Smart city's vehicle login system with sensor chip, IEEE International Conference on Advances in Computer Applications (ICACA), pp. 217-223, Coimbatore, 2016, doi: 10.1109/ICACA.2016.7887954

[10] Janoš, V., Kříž, M.: Smart Urban Transport. In: Smart City Symposium, 2018, doi: 10.1109/SCSP.2018.8402664

EVALUATION OF THE BASIC CHARACTERISTICS OF THE TRAFFIC FLOW BY MATHEMATICAL ANALYSIS

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Abstract

Today, traffic is an integral part of every active human life. People prefer individual car transport to public transport. This fact increases the number of cars on the roads. This article focuses on the basic characteristics between moving vehicles on the road, traffic flow characteristics - intensity, speed and density. Their values are obtained from traffic survey and radar Sierzega. In addition, a simulation was made from the measured values and the basic characteristics of the traffic flow were evaluated using mathematical statistics.

Keywords: simulation, junction, traffic, road

1 Introduction

Transport in cities is usually realized via roads in the traffic flows of vehicles or pedestrians. From the life of citizens, it is the realization of the movement of persons or material, i.e. their transfer from sources to goals. Transportation is a complex stochastic process.
At present, transport is an integral part of each of us, whether travelling for work, leisure, transportation of goods, etc. This factor has a particular impact on the growth of vehicles on the roads, which often causes congestions in cities. Nevertheless, it can be managed and regulated, through measures and laws. However, an important task is to design and build roads that handle the traffic flow as well as future traffic intensity. Before the actual construction of roads, it is necessary to conduct traffic surveys and find out the current intensity of traffic on a given section of the road network. Each transport survey aims to obtain quantitative and qualitative data on transport [1]. It is also important to know the basic characteristics of the traffic flow, i. j. intensity, speed and density and their interrelationships.
Theoretically, traffic flow can be understood as a flow composed of different types of vehicles that have their own specific properties. These properties distinguish it from similar phenomena (flows) known, for example, from physics, and therefore it is necessary to examine it separately. A number of factors affect the traffic flow [1](1).

2 Traffic flow

Traffic flow theory is a very broad area. In mathematics and traffic engineering, it is the observation of the interaction between passengers (includes pedestrians, cyclists, drivers and their vehicles) and infrastructure (motorways, roads, markings) to understand and develop an optimal transport network with efficient traffic movement and minimal congestion problems.

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The traffic flow is understood as the unity of temporal and spatial conditions, but at the same time traffic and movement conditions of vehicles on the traffic area of the road. Traffic flow is the flow of individual vehicles that move in certain conditions and in a certain direction. It, therefore, depends on the width arrangement and the overall guidance of the route. The movement of vehicles in traffic flow is affected by other vehicles in the traffic and is therefore monitored as a whole and not as the movement of an individual vehicle. His research is a very interesting and current issue. Traffic flow is a nonlinear dynamic phenomenon. This nonlinear behaviour is most noticeable at high traffic flow densities. The traffic flow can be divided into homogeneous and inhomogeneous in terms of composition. Homogeneous traffic flow is characterized by the homogeneity of types of vehicles that have the same driving characteristics (passenger cars, trucks, heavy, light, slow, fast). The inhomogeneous traffic flow contains proportions of different types of vehicles with different driving characteristics. The ratio of these vehicles is expressed as a percentage of the total number of vehicles in the traffic flow. Traffic engineering calculations can convert all vehicles in the traffic flow into unit vehicles and thus obtain a theoretical homogeneous traffic flow.

2.1 Basic characteristics of the traffic flow

Basic characteristics of the traffic flow are three interdependent characteristics, namely speed v, intensity M and density H. All these quantities depend on place and time. The quantity of the traffic flow is its intensity and quality expresses the speed and fluency in the given conditions. A change in its acceleration and deceleration, a speed gradient or a change in the ripple of a traffic flow can be considered as the flow of a traffic flow. This ripple is intended as a change in intensity and density over time CITATION LEI \l 1051 [2](2). The search for and description of the relationships between these three quantities is the basis of the traffic flow theory.
The speed v depends directly on the path s and indirectly on the time t. It is most often stated in the basic units of the SI system, i.e. in $m.s^{-1}$, but more commonly used units are $km.h^{-1}$ CITATION Kap \l 1051 [3](3). A formula for calculation of speed (1) is simple.

$$v(s,t) = \frac{\partial s}{\partial t} \quad \left[km.h^{-1}\right] \tag{1}$$

Where:
v - the speed of traffic the flow [$km.h^{-1}$],
s - the distance of the monitored section [km],
t - the time [s].

Intensity is the most important characteristics of the traffic flow because the greatest intensity that the road can transmit is the capacity of the road. This is important for its design and assessment. Intensity is defined as the number of vehicles that pass a given profile per unit time, in either one or both directions. Number of vehicles and the time is necessary for calculation (2).

$$M(s,t) = \frac{N}{t} \quad \left[veh.h^{-1}\right] \tag{2}$$

Where:
N - the number of vehicles on section at certain moment [vehicle],
t - the time [s].

Density is the number of vehicles located in the unit section l at a given time t (3). At low densities, vehicles are allowed to move freely and drivers can choose the speed of their choice. Conversely, at high densities, the driver is affected by other road users and congestion occurs.

$$H(s,t) = \frac{N}{l} \quad [veh / km] \tag{3}$$

Where:
N - the number of vehicles on section at certain moment [vehicle],
l - selected section [km].

2.2 Basic characteristics of the traffic flow

There is a relationship between the basic characteristics and it is given by the equation of continuity relationship (4),

$$M = \nu \cdot H \tag{4}$$

the natural relationship of speed to density is verified, because there is a maximum speed at the minimum density and, conversely, a maximum density at which speed is zero. It follows that density also depends on intensity.
Fundamental diagrams are commonly used to express the relations in steady traffic flow, i.e. to describe the relationship between the basic quantities of the traffic flow given by the continuity equation (4).

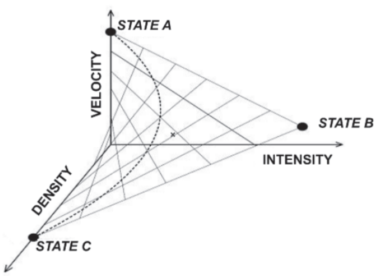

Figure 1 Three-dimensional model of traffic flow characteristics CITATION Kap \l 1051 [3](3)

The relationship of these characteristics is well illustrated in figure 1. This representation in the 3D graph is not entirely clear, and therefore the representation using three graphs is used, which are the projection of this curve into all three planes. The basic diagram is the relationship between density H and speed v, from which the relationships density H - intensity M and speed v - intensity M are derived (figure 2).

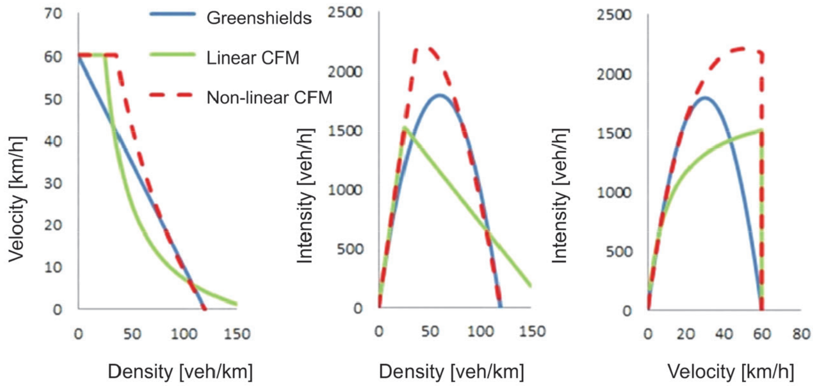

Figure 2 Comparison of fundamental diagrams CITATION MIK \l 1051 [4](4)

Stationary models of homogeneous traffic flow working with the stationary hypothesis. According to it, the speed field v (H) has rapidly stabilized at a constant value identical for all vehicles, and the same applies to the density or the distances between the vehicles. The system does not evolve over time. The hypothesis does not address the mechanism of stabilization, although it would probably not be as trivial as the trivial assumption of stabilization itself. Both empirical and simulation results suggest that the stationary hypothesis is too strong a simplistic assumption that is inconsistent with reality CITATION VKo \l 1051 [5](5). Nevertheless, it is very often used [3]. The most commonly used models are CITATION Hab \l 1051 [6](6):

- Greenshield model - assumes a steady and homogeneous flow, where the speed of individual vehicles and the density of the traffic flow are constant. It is characterized by a linear dependence of the speed of the traffic stream on its density.
- Constant time interval model - this model is not based on a theoretical description of the traffic flow, but on recommendations for safe driving. A well-known rule states that the vehicle should maintain a safety distance Δt from the previous vehicle of two seconds, regardless of speed.
- Linear CFM - A linear “car-following model” (CFM) is equivalent to a constant time interval model. It is based on the assumption that the acceleration of the i-th vehicle depends on its relative speed to the previous i-1 vehicle.
- Non-linear CFM - linear CFM can be improved by including the assumption that the driver is more sensitive to the behaviour of the previous vehicle at a smaller relative distance.

3 Modelling of traffic flows

Modelling of traffic flows by means of suitable software equipment represents an effective working method in the fields of traffic engineering as well as traffic construction. However, such modelling does not only include traffic simulation, but represents a wide range of auxiliary tools, from relatively simple single-purpose applications to complex tools for performing complex analyzes of transport networks and processes on them. One of the most important theoretical tools of these specialized software products are traffic flow models. These models form one of the basic input parameters in the analysis and optimization of the target behaviour of road users CITATION HEN \l 1051 [7](7).
The TSS (Transport Simulation System) simulation program - Aimsun was used to create the simulation. Aimsun software helps thousands of international users model future smart mobility networks. Aimsun is traffic simulation software that allows users to model everything from a single bus lane to an entire region in both 2D and 3D views.

4 Traffic survey

The analysed data come from a survey performed on Tuesday, September 17, 2019 near the city of Martin on road I/65. It was cloudy with occasional rain in the morning and this continued until the afternoon, when the clouds decreased and partly cloudy. The temperature ranged from 8 to 15 °C. The survey lasted a total of 12 hours, from 6 o'clock a.m. to 6 o'clock p.m. In addition to the use of measuring technology, counters were also present at the measuring site. Based on this fact, it is possible to compare the intensity of vehicles recorded by measuring equipment (radar) and counters (persons). The maximum permitted speed on the monitored section is 50 km.h^{-1}.
During the traffic survey the intensity reached 13,505 vehicles in both directions in 12 hours. The highest intensity (peak) for a quarter of an hour (326 vehicles per 15 minutes) was recorded between 6:30 and 6:45, for an hour (1248 vehicles per hour) it was determined between 6:30 and 7:30 (Table 1).
According to a survey carried out by counters, the composition of the traffic flow is as follows: 76.3 % of passenger cars, 10.5 % of trucks, 12.5 % of heavy trucks, 0.37 % of motorcycles, 0.25 % of buses and 0.07 % of bicycles. In this case we speak on inhomogeneous traffic flows, as it is assumed that a homogeneous traffic flow must make up at least 80 % of vehicles that have similar characteristics, e.g. passenger cars.

5 Results

The following chapter shows the processed results from the measured values using Sierzega radar and simulation.

5.1 Results from traffic survey

The ideal diagram showing the behaviour of the transport system is the so-called fundamental diagram. This is a graphical dependence between the basic characteristics of the traffic flow (intensity, speed, density) was determined for each 15-minute interval for each lane separately and together. These dependencies are shown in the figure 3.

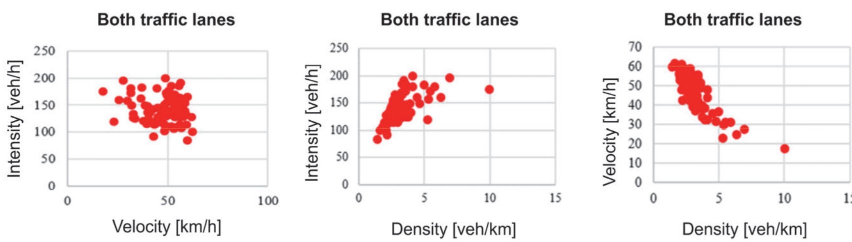

Figure 3 Graphical dependence between basic characteristics of traffic flow as output from measured values

According to the relationship between intensity and speed (figure 3), the dependence should form a parabola. However, with increasing speed, the intensity should also increase, which would represent a linear course. It is clear that each intersection and each road section has the specific maximum capacity CITATION MPo1 \l 1051 [8](8). If it is exceeded, the intensity should logically still increase, but this fact causes a decrease in speed because the capacity has been exceeded. The maximum intensity is achieved at the optimum speed. The reduction of speed leads to a reduction in the distance between vehicles, which is reflected in increasing density. Therefore, the relationship between intensity and speed is expressed as a parabola, that is, as the square of speed. The same rule applies to the relationship between

intensity and density. The correlation coefficient has the linear dependence. Its value was around - 0.037, which corresponds to a negative very weak linear dependence.
It is interesting that the dependence between intensity and density reached a positive linear dependence with a high correlation coefficient of 0.653, so it is the medium correlation. The dependency can also be read from the graph CITATION Ond \l 1051 [9](9).
According to the general diagram, the relationship between speed and density is expressed as a linear dependence. However, the fact is that in this case, the dependence between density and speed has an exponential course with a degree of reliability of almost 80 %. In principle, if the speed of vehicles decreases, the density in the monitored section will increase, and the opposite is also true. The correlation coefficient was also calculated for the course of speed and density. In this case, the value of the coefficient of dependence between the two variables is 0.950, which corresponds to a very strong negative correlation CITATION Ryb \l 1051 [10](10).

5.2 Results from simulation

The simulation lasted 12 hours, as did the traffic survey. The load on the lanes was based on traffic surveys and Sierzega radar records. A total of 10 simulations were performed, from which an average was made, which was the basis for the resulting values, namely the resulting values of intensity, density and speed.

5.3 Probability of vehicle occurrence

The figure 4 is a graphical representation of the incidence of the number of vehicles per minute resulting from a survey that follows the Poisson distribution CITATION Med \l 1051 [11](11). According to statistics, this is a discrete distribution of the probability of occurrence of rare events in a series of large numbers of independent experiments. Based on the survey, it was found that an average of 18.8 vehicles was intercepted in 1 minute. Thus, the highest probability is that in 1 minute 18.8 (λ) vehicles will pass through the monitored section of the road in both, and with the increasing or decreasing number of vehicles, this probability decreases CITATION MPo \l 1051 [12](12).

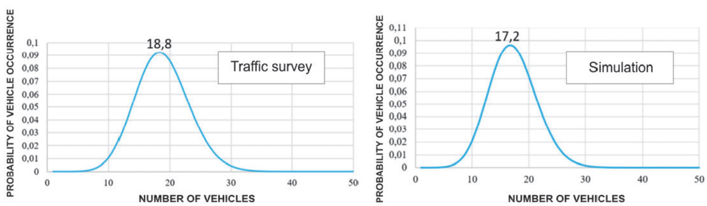

Figure 4 Occurrence of vehicles during 1 minute

In contrast to the actual measurement, the results from the simulation showed that in this case, the highest probability of arrival is that in 1 minute it will pass the monitored section on average 17.2 vehicles. Thus, the decrease represents 1.6 vehicles less than in the evaluated measurement results.

6 Conclusion

As the number of road traffic increases, the density increases and the speed of the traffic flow decreases. This fact has the effect of increasing the residence time, standing and travel time and other variables. The mentioned characteristics of the traffic flow vary significantly depending on the time and volume of traffic, often reaching maximum values. Intersections are an important point of road network permeability.
The evaluation of the traffic survey showed that the largest number of vehicles was recorded between 6:30 and 6:45 with an intensity of 326 vehicles. The average intensity according to the traffic survey is 1200 vehicles per hour and according to the simulation it is 1035 vehicles. The strongest correlation was calculated in the relationship between density and speed with a value of -0.95, which represents a strong linear dependence.
Our results confirm difference in probability of vehicle occurrence. The difference between simulation and real measurement were 1.6 vehicles.

Acknowledgment

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References

[1] Novák, M.: Popis charakteristik dopravního proudu, Vysoké učení technické v Brně, Brno, Diploma thesis, 2015.

[2] Leitner, B., Sventeková, E., Novák, L.: Možnosti testovania parametrov dopravného prúdu pri strate funkčnosti vybraného prvku cestnej siete, Perners Contacts, Pardubice, 9, 2014.

[3] Ondruš, J., Karoń, G.: Video System as a Psychological Aspect of Traffic Safety Increase, International Conference on Transport Systems Telematics, Springer, Cham, 2017, doi: 10.1007/978-3-319-66251-0_14

[4] Mikolášek, I.: Analýza dejov pri spájaní dopravných prúdov, Vysoké učení technické v Brně, Brno, Bachelor thesis, 2014.

[5] Konečný, V., Berežný, R., Bartoníková, M.: Research on the Impact of Quality on Demand for Bus Transport, 18th International Scientific Conference-Logi 2017, MATEC Web of Conferences, 2017, doi: 10.1051/matecconf/201713400024

[6] Habtemichael, F.G., Cetin, M.: Short-term traffic flow rate forecasting based on identifying similar traffic patterns, Transportation research Part C: emerging technologies 66 (2016), pp. 61-78, doi: 10.1016/j.trc.2015.08.017

[7] Petro, F., Konečný, V.: Calculation of emissions from transport services and their use for the internalisation of external costs in road transport, 12th International Scientific Conference of Young Scientists on Sustainable, Modern and Safe Trans, doi: 10.1051/matecconf/201713400026

[8] Poliak, M., Mrnikova, M., Simurkova, P., Medvid, P., Poliaková, A., Hernandez, S.: Social law in road transport like tool safety road transport, New York, IEEE (2018), doi: 0.1109/AUTOSAFE.2018.8373317

[9] Ondruš, J., Kolla, E.: Practical Use of the Braking Attributes Measurements Results, MATEC Web of Conferences, 134 (2017), EDP Sciences, doi: 10.1051/matecconf/201713400044

[10] Rybicka, I., Caban, J., Vrábel, J., Šarkan, B., Stopka, O., Misztal, W.: Analysis of the safety systems damage on the example of a suburban transport enterprise, XI International Science-Technical Conference Automotive Safety, IEEE (2018), doi: 10.1109/AUTOSAFE.2018.8373323

[11] Medviď, P., Marián, G., Kubaľák, S.: Occupancy of Public Transport Vehicles in Slovakia, Transportation research procedia, 44 (2020), pp. 153 – 159, doi: 10.1016/j.trpro.2020.02.022

[12] Poliak, M., Hammer, J., Reuter, N., Poliaková, A.: The impact of the transport route on the cost of the transfer, New York: IEEE (2018), doi: 10.1109/AUTOSAFE.2018.8373319

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THE ANALYSIS OF SOLUTIONS OF STATIC TRANSPORT IN THE SLOVAK REPUBLIC AND ABROAD

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Abstract

This article is focused on problems related to parking in the Slovak Republic (SK) and abroad. The examples of individual solutions are mostly from spa cities or towns. Spa cities are attractive from the transport view and they attract traffic more. The most used means of transport is an automobile and in relation to this, there is a greater demand for parking in these cities. The number of visitors in spa cities has been increasing every year and therefore, the number of vehicles has been increasing too. The cities often do not have enough space to build new parking spots. Improper parking on the streets, public space and green areas is not unusual and it presents a negative impact on the city dwellers (e.g. restricted traffic). In our article, we analyse and compare individual solutions for parking in the Slovak Republic and abroad.

Keywords: static transport, vehicle, parking regulation, parking app, parking house

1 Introduction

On average, a vehicle is not used 90 % of the time – it is parked. In connection with the increasing number of vehicles and the need for parking, it is necessary to ensure an adequate number of parking spots, which are a few in present. It is difficult to find a free parking spot and the needed time to find a free parking spot has been increasing. Possibly the best way to decrease the demand for parking is to ensure that people will begin to use the public transport more, walk or cycle. There are several ways how to decrease the ratio of IAT (individual automobile transport) in favour of the public transport, cycling and walking.

One of the most significant ways is realization of driving lanes reserved for public transport, so-called bus lanes. A travel time has been reduced in several cities in the world due to that. The second way how to increase the demand for other modes of transport is to make roads dedicated just for pedestrians, cyclists, or public transport. As a result, certain areas of the city will be closed for IAT and passengers will be forced to use other modes of transport than automobile one. The best example of this solution is Groningen city, in which ~~is~~ one of the biggest ratios of bicycle transport of the whole modal split is.

Regulation of parking is the easiest viable way and also the most significant one. Vacant sidewalks and regulation of parking are the base of improving traffic situation according to Enrique Peñalosa – worldwide honoured expert on urban development. Regulation of parking and number of parking spots effectively decrease ratio of IAT. A good example is a building 33 Central in London, which has only 2 parking spots in total. Both parking spots are reserved for customers with disabilities.

Parking can be regulated by 2 basic ways. One way is by legal norms, which define a number of parking spots and the second way is paid parking. Slovak norms define just a minimal number of parking spots and due to this, they do not support decreasing of demand for IAT, but the other way around. For comparison, it is suitable to mention the neighbouring Czech Republic, where there are valid Prague civil engineering prescriptions, which determine minimal and maximal number of parking spots. The regulation of parking in the form of charging is in the Slovak Republic at a low level. Drivers often park their vehicles at places, which are not defined for parking – sidewalks, grassy areas, and roads. The food for thought is that they can park for free like this and in many cases, they restrain other participants of traffic or they destroy green areas. The individual examples of solutions given in the article are mainly from spa cities or towns. A spa town is a specialized resort town situated around a mineral spa. The term spa is used for towns or resorts offering hydrotherapy which can include cold water or mineral water treatments and hot thermal baths. [1,2,4]

2 Static transport in SK

Modal split in SK achieves the ratio of 74 % (IAT) to 26 % (public transport) in a numerical expression. The current situation regarding the modal split is unfavourable due to the lack of accessible parking spaces in most cities in the SR. It is necessary to solve the parking problem complexly from the beginning of requirement to its enforcement, so that it keeps the necessary city mobility. [3]

2.1 Parking houses

One of the most common problems in SK is the lack of space to build new parking spots. The most suitable solution to this problem is to build new parking spots under or above the ground, which does not take up as much place as regular terrestrial parking spots. This way will ensure a large amount of parking spots without a need for a large area. Currently, there are not many parking houses in SK. Most of them are a part of shopping centres, where underground parking on several floors is provided. The following parking houses are already in operation, however, they are not in the spa cities, because parking houses are only located in these type of cities (we do not consider shopping centres).

2.1.1 Zvolen

Parking house with a capacity of 177 parking spots is in Zvolen in a housing estate called Západ. These parking spots are not free of charge, but drivers must pay for parking. The investor of this parking house is a private company in Banská Bystrica. There is also a car wash, bowling, pizzeria and a fitness centrum apart from parking spots. This Parking house was built in this part of the town because of the cars parking on sidewalks and green areas. [5]

Figure 1 Parking house in Zvolen – housing estate Západ

2.1.2 Trenčín

Near the centre of Trenčín city a parking house with 150 parking spots, which are charged is situated. Few shops and a club are a part of this parking house too. The operation is nonstop and the whole building is supervised by a camera system and staff. Occupancy of the parking house is not too high. At night it is at a level of 10 % and during a day the occupancy is higher. The reason for a low occupancy is a fear of parking in the parking house and free regular terrestrial parking spots near the parking house. [6]

Figure 2 Trenčín - parking house

2.2 Parking apps

Increasing number of vehicles every year and a lack of parking spots wreak a problem to find a free parking spot. Parking applications can truncate a time of searching for a free parking spot by 43 % and travelled distance by 30 %, according to the results of realized measurements. Few applications can be used in several cities in SK – e.g. ParkDots, Parkio, Zaparkuj.to, EasyParking, parking4disabled, CVAK. [7]

2.2.1 ParkDots

It is possible to use ParkDots application in the several cities in SK – Bratislava, Piešťany, Trenčín, Trnava, Dolný Kubín, etc. The application offers data about the occupancy of a parking lot and a parking fee. It is using IoT (internet of things) sensors. It will send a notification to the user 15 minutes before the end of the validity time. In the case of a need, it is possible to prolong the time of parking from anywhere. Apart from that, the application offers detailed parking statistics – monitoring of payment discipline, actual occupancy of parking spots. ParkDots Enforcement application serves policemen and other delegated persons for control of payment for parking and time of parking for which was paid (it is carrying out with the help of a license plate number). They carried out a research on a sample of 696 questioned persons:

- 82 % of questioned persons would use a parking application to find a parking spot and pay for parking,
- 74 % of questioned persons were willing to offer their parking spot for other drivers to park when they are not using it,
- 75 % of drivers regularly use the navigation application while driving,
- 80 % of drivers said they would like to pay for parking by mobile payment. [8,9]

2.3 Parking cards

Parking cards are one of the solutions of parking regulation. These cards are given to residents and also for visitors of the city. A validity of these cards is for a certain period. The owner of a parking card does not have to pay for parking on paid parking lots at the area of the city.

2.3.1 Piešťany

The town of Piešťany is regulating parking in the form of paid parking on the streets Rázusova, Nálepkova and in the central area of the town. Parking in paid parts of the town is possible ~~just~~ only with non-transferable parking card or by buying a parking ticket. Non-transferable parking card is valid only when it is located in the vehicle, which license plate number is listed on the particular card. Three kinds of non-transferable parking cards can be procured in Piešťany - weeklong parking card – 6 €, yearlong parking card – 35 € and yearlong parking card for resident – 25 €. [10]

3 Static transport in abroad

In most western countries, several options have been introduced to solve problems with parking. In these countries, they are trying to change a human´s mind and persuade them that ownership of an automobile is a burden. They have been trying to restrict the movement of automobiles through various regulations (no entry into a certain area, prolongation of travelling time – red wave on crossroads with traffic lights, traffic calming) and continually improving the quality of public transport. San Francisco is an interesting city concerning the charging of parking. If demand is high, the parking fee is increasing, if demand is low, the parking fee is decreasing.

3.1 Parking house

In developed European countries (e.g. Germany, Austria, etc.), parking houses are not exceptional, but the common way of ensuring a sufficient number of parking spots on the minimal area. They are trying to ban parking in the streets and as a result ensure larger space for other kinds of transport (public transport, bicycle transport and walking).

3.1.1 Mariánské Lázně

Mariánské Lázně is a spa town in the Czech Republic. A spa area is marked with traffic signs with restrictions. Entry to this area with motor vehicle is possible just with a permission.

Figure 3 Parking house – Mariánské Lázně

In a side part of the town, it is possible to park a vehicle (up to 3,5t) in a parking house with 360 covered parking spots. Occupancy of the parking house is 100 % during holidays and weekends. During other days, the occupancy is approximately at a level of 60 %. Yearly costs including repairs and debentures present approximately 1 million CZK. Minimal parking fee is 20 CZK for an hour and maximal parking fee is 140 CZK for a day. [11]
3.1.2 Bad Hersfeld
Bad Hersfeld is a spa town in Germany, where four parking houses were built by private companies. The construction of the Schilde-Parkhaus is made of steel and unprocessed spruce wood. Due to the open wood construction, natural lighting and ventilation is ensured. The architects considered various aspects when designing to minimize interference with the surrounding landscape. [12]

Table 1 Price list and information about parking houses

	Capacity	Operation	Price list	
			min.	max.
Schilde-Parkhaus	180 spots	nonstop	1 € / hour	4 € / day
City-Parkhaus	285 spots	Mon – Sat (7:00 - 21:00)	0,50 € / 30 minutes	15 € / day
City-Tiefgarage	230 spots	Mon - Sat (7:00 - 19:00)	0,50 € / hour	10 € / day
Parkhaus Altstadt Bad Hersfeld	162 spots	nonstop	0,50 € / 30 minutes	10 € / day

Figure 4 Schilde-Parkhaus

3.2 Automated parking systems

Currently, automated parking systems (APS) have been progressing and are able to park a vehicle without the driver´s assistance. Many variants of these systems exist (for example - car tower, puzzle parking system, etc). This system of parking helps drivers eliminate the needed time of finding a free parking spot. The vehicle is parked without the assistance of the driver. APS are in operation in the Czech Republic (Prague, Liberec, Slaný), China, Germany and other countries.

3.2.1 Slaný

This town has a population of 15 800 habitants and a parking house with APS (149 parking spots) is located here. Although it is not a spa town, it can be an example for small spa cities, which do not have enough place for parking.

Figure 5 Parking house with APS in Slaný

The total price of this parking house with APS was 89 million CZK. Yearly costs are more than 1,5 million CZK (information of 2018). Parking house with APS has 6 overground floors and 1 underground. Parking house with APS is utilised on 60 % according to the information from the previous year (2018). The operation is nonstop. Parking fee is 10 CZK per hour. Between 17:00 and 8:00 on weekdays, the maximum fee is 50 CZK. During holidays and weekends, the maximum fee is also 50 CZK. [13]

3.3 Parking applications

3.3.1 Parkopedia

In Great Britain, Germany and the USA a survey with the following result was conducted – a driver spends 44 hours yearly finding a free parking spot. Parkopedia can be used in 75 countries and 8 000 cities. It incorporates approximately 60 million parking spots. The application allows drivers to find the nearest parking lot near their destination, informs how much it will cost to park in the parking lot and whether there is a free parking spot. Real-time parking monitoring is available in over 2 000 cities (using special sensors). For example, it is possible to park a vehicle using this application in spa towns - Bad Hersfeld (Germany), Karlovy Vary (Czech Republic), Piešťany (Slovak Republic), etc. [14]

3.4 Parking cards

The regulation of parking in the form of parking cards and charging of parking is a usual way abroad. Parking for residents and abonents is the most frequent differentiate. Residents are inhabitants with residence in the area of a certain city. Abonents are regular visitors to the city.

3.4.1 Karlovy Vary

In the spa town of Karlovy Vary, there are 4 types of parking cards – a one-off, resident, abonent and discounted abonent parking card (100 CZK/year). In the case of the loss or theft of a parking card, it is requisite to pay a fee of 100 Kč. [15]

Table 2 Price list of parking cards – Karlovy Vary

Number of vehicles	Fee	
Parking card for residents		
1. vehicle	480 CZK / year	240 CZK / half-year
2. vehicle	3 500 CZK / year	1 750 CZK / half-year
3. vehicle	7 000 CZK / year	3 500 CZK / half-year
Parking card for abonents		
1. vehicle	2 400 CZK / year	1 200 CZK / half-year
2. vehicle	9 000 CZK / year	4 500 CZK / half-year
3. vehicle	18 000 CZK / year	9 000 CZK / half-year
One-off parking card		
1. vehicle	3 000 CZK / month	100 CZK / day

4 Conclusion

Parking problems occur all over the world. Solutions are often costly, but it is still necessary to apply them. It is important to change people's mind to increase the demand for public passenger transport or other alternatives (bicycle transport and walking).
The analysis of the current situation in Slovakia shows that parking is a big problem and solutions have been arising slowly. The regulation of parking is not developed sufficiently and is not expensive. The situation is already very problematic, so it is not possible to constantly wait. In the introduction, it was mentioned that when analysing the current situation in the Slovak Republic and abroad, we will try to give examples of solutions from spa towns/cities. Only two solutions are mentioned from the spa towns in SK - parking cards and the parking app, which can be used in Piešťany. Other cities have more solutions for parking - for example, parking houses, parking apps, SMS payment, etc. In comparison to spa cities/towns, they attract less traffic (visitors) and do not have such problems with parking. Spa towns in Slovakia have problems with parking and have been implementing solutions to improve the current situation slowly. It is necessary to solve parking problems also in terms of air pollution (reduce emissions from traffic). It is possible to say that spa towns in Slovakia which attract traffic do not have sufficient solutions in place to deal with the current situation of parking. That is the reason why we focused on spa towns/cities. Unless they introduce the necessary measures and solutions soon, this will be a major problem for these towns. It is better to avoid these problems than to wait for them to occur. Of course, some solutions require a change in people's mind, as we have already mentioned.
Several solutions from abroad spa towns/cities are mentioned in the article. It is necessary to introduce a system of public transport decreasing demand for IAT. There are several solutions and successful examples how is possible to solve the parking and set up a better direction for the future, which will help change the modal split in favour of public transport.

References

[1] Kalašová, A., Kubíková, S., Kapusta, J.: The relation between driver behavior and intelligent transport system, Acta Polytechnica CTU Proceedings 12 (2017), pp. 50-55, doi: 10.14311/APP.2017.12.0050.

[2] Dula, I., Surový, R.: Mobilita a parkovanie, Urbanita, pp. 32-33, 2018.

[3] Palúch, J., Veterník, M., Kalašová, A.: Impact of Public Transport Priority on Traffic in Chosen Part of the City of Martin, Intelligent Transport Systems – From Research and Development to the Market Uptake, INTSYS 2017, pp. 3-11, doi: https://doi.org/10.1007/978-3-319-93710-6_1

[4] Milosavljevic, N., Simicevic, J.: Parking regulation (Chapter), Sustainable Parking Management, ed. Romer, B., Elsevier (2019), pp. 165-207, doi: https://doi.org/10.1016/C2017-0-03917-4.

[5] Parking house Zvolen, www.parkovacidom.sk, 14.03.2020.

[6] Parking house Trenčín, www.parkovacidom-trencin.sk, 14.03.2020.

[7] Parking apps, www.mobilmania.cz/clanky/diky-aplikacim-uz-parkovani-v-cizim-meste-neni-peklo/sc-3-a-1333464/default.aspx, 15.03.2020.

[8] Parking app – ParkDots, www.parkdots.com, 15.03.2020.

[9] Subramani, A., Solanki, V.K.: Smart City: IOT Based Prototype for Parking Monitoring and Management System Commanded by Mobile App, Second International Conference on Research in Intelligent and Computing in Engineering, ACSIS, 2017, pp. 341-343, doi: http://dx.doi.org/10.15439/2017R23.

[10] Parking cards – Piešťany, www.sluzbymesta.sk/parkoviska-1/, 15.03.2020.

[11] Parking house Mariánské Lázně, www.zaparkujauto.cz, 15.03.2020.

[12] Parking houses in Bad Hersfeld, https://www.parkinglist.de/parkplatz/Bad-Hersfeld/tiefgarage-city-tiefgarage-stadthaus-bad-hersfeld-8519, 15.03.2020.

[13] APS in Slaný, www.slanske-noviny.cz, 23.03.2020.

[14] Parkopedia, www.parkopedia.com, 29.03.2020.
[15] Parking cards – Karlovy Vary, www.mmkv.cz, 29.03.2020.

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ANALYSIS OF ENERGY EFFICIENCY OF SUBURBAN RAILWAY TRANSPORT NETWORK

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Abstract

Rising numbers of agglomeration residents cause increased need for people movement on daily basis. Because of congestion of local roads, air pollution and limited parking space, providing mass transit based on electric traction is reasonable. While the electric rail vehicles are considered highly efficient in themselves, they need to be analyzed as a part of a transport network, because energy consumption depends on operating conditions as well. Information about energy efficiency of whole system operating under realistic conditions could be helpful for modernization of traction power supply, timetable planning or while ordering new rolling stock. This paper presents approach to analysis of energy efficiency of a suburban rail network, using specialized software developed on Matlab/Simulink basis. For the sake of analysis, simple transport network consisting of three lines was considered. Vehicles, assumed as uniform electric multiple units, operate according to the set schedule, taking into account varying electric drive efficiency and mass dependent on passengers' number. Vehicles are supplied by four substations with nominal voltage of 3000 V DC, using overhead contact line. Developed model includes calculation of energy losses in power supply, therefore it is possible to determine efficiency of the whole network as a relation of mechanical energy of vehicles movement to electrical energy fed from public power system. Mean useful voltages for vehicles and substations are computed as well. Program structure allows for further expansion, e.g. with optimization algorithms.

Keywords: computer simulation, urban transport network, railway electric traction, energy efficiency, traction power supply

1 Introduction

Suburban transport networks are constantly changing in order to provide adequate service for passengers. Reliability, comfort and operation cost are the key factors forcing rolling stock and infrastructure modernization, as well as building new lines. Because such investments are expected to work efficiently in long-term perspective, precise planning is needed. This is especially important when entirely new infrastructure or vehicle is designed – because it is impossible to test object that does not exist yet. While basic parameters can be determined using empirical equations and averaged statistic data found in literature or norms, computer simulations could provide more accurate results. Moreover, a well-developed model should be capable of producing comprehensive data, allowing for precise determination of power supply and vehicle drive parameters, unveiling weak points (with highest voltage drop) or energy losses assessment.

Numerical simulations of vehicle movement in transportation are widely used, mostly for timetabling and energy consumption calculations. Most of them, however use simplified models [1-3] – assuming one vehicle on route, constant values of line voltage and drive efficiency factors or idealized regenerative braking. While such simplifications will not noticeably affect basic results like travel time or movement dynamics, they have an impact on electrical parameters, especially values of current and line voltage [4, 5]. There is also need for calculation of voltage drop caused by power supply elements resistance, and mean useful voltage for vehicle and substations [6]. In order to achieve that, some applications compute these values assuming single vehicle run, then calculating parameters of whole system using superposition. The most complex models simulate multiple vehicles operating under realistic traffic conditions – which allow in-depth analysis of such system. However, complexity and stability of such programs are the major drawbacks that often narrow their usage to single line or section [7,8]. While it is possible to simulate substation load caused by vehicles on other sections (or branch lines) as additional current, analysing transportation system as a whole will be more accurate.
This paper presents a novel approach to suburban railway transport network simulation using model developed on the basis of Matlab/Simulink. Efficiency maps computed for both traction drives and substations (power transformer with rectifier) were used to approximate losses dependant on both mechanical and electrical load. For the sake of the analysis, theoretical simple railway network consisting of three lines was considered (Fig. 1).

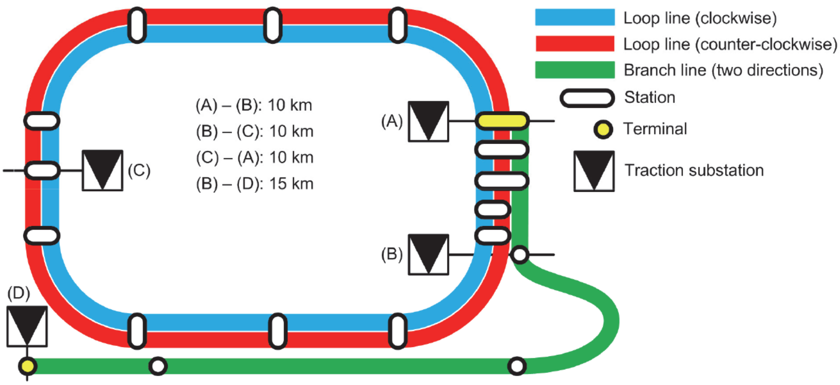


Figure 1 Map of analysed system

Vehicles operate according to set schedule, with 10 minutes tact on the loop line (in both directions) and 20 minutes tact on the double track branch line (similar to airport connections in some agglomerations). Simulation was conducted for timeframe of one hour of network operation, allowing assessment of energy consumption, losses and overall system efficiency under realistic circumstances.

2 Model design

Model presented in this work was developed aiming for maximum versatility, allowing for easy parameter editing and expansion for future analyses. Because of that, Matlab/Simulink was chosen as a basis for program design – Simulink block diagram synergizes well with modular/layered structure concept (Fig. 2), while it is still possible to run additional operations using coded functions and scripts, improving computation performance.

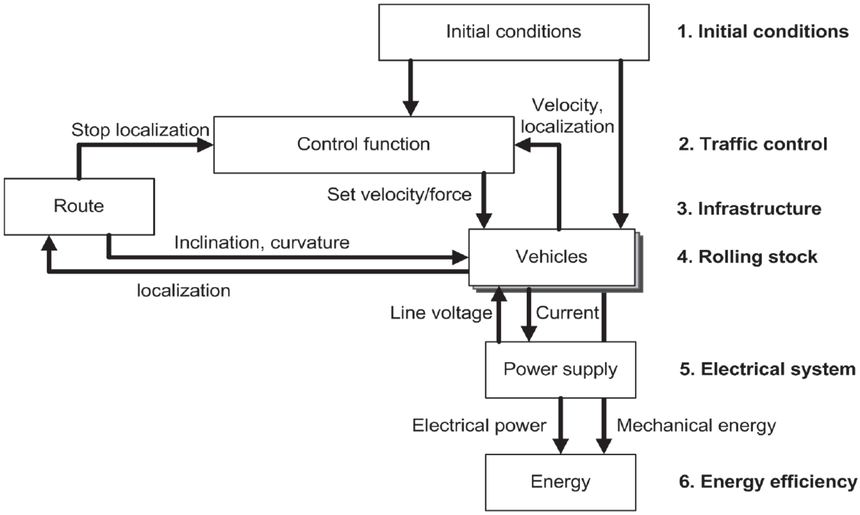


Figure 2 Simplified block diagram of the developed program

It is worth noting, that every module contains standalone model, which can be copied and executed as a part of different program as long as sufficient parameters are provided. Also, it is possible to modify input parameters to simulate various types of rolling stock, different catenary or number of working power transformers in substations.

2.1 Vehicle movement

Vehicles were assumed as uniform, six-section electric multiple units (EMUs), similar to vehicles used by suburban rail operators worldwide (e.g. Newag Impuls, Siemens Desiro, Urban-Liner Next etc.). Each EMU is powered by eight induction motors with total power of 3 MW, allowing for top speed of 44,5 m/s (160 km/h). Detailed vehicle data are shown in Table 1.

Table 1 Parameters of analysed vehicles

Parameter	Value	Unit	Comment
Axle layout	-	-	Bo'Bo'+2'2'+2'2'+2'2'+2'2'+Bo'Bo'
Max. tractive effort	250	kN	Acceleration/braking
Vehicle mass	195/220	Mg	Empty/full load
Rotating mass coeff.	1,15	-	
Passenger places	300	-	Assumed 75 kg per passenger
Auxiliary power	320	kW	Assumed constant

Vehicle movement dynamics are computed using basic physical equations. Acceleration is calculated by division of motive force by vehicle mass and rotating mass coefficient. Velocity is computed by integrating acceleration, while distance is the result of velocity integration. The motive force is dependent on drive parameters and values set by control function, to ensure execution of velocity profile. Braking is performed according to braking curves, with constant deceleration value of 0,9 m/s^2. Adequate brake force is achievable at all speeds because combined brake system is implemented (electrodynamic and friction brakes) [9]. Because value of motive force generated by motors is needed for correct regenerative brak-

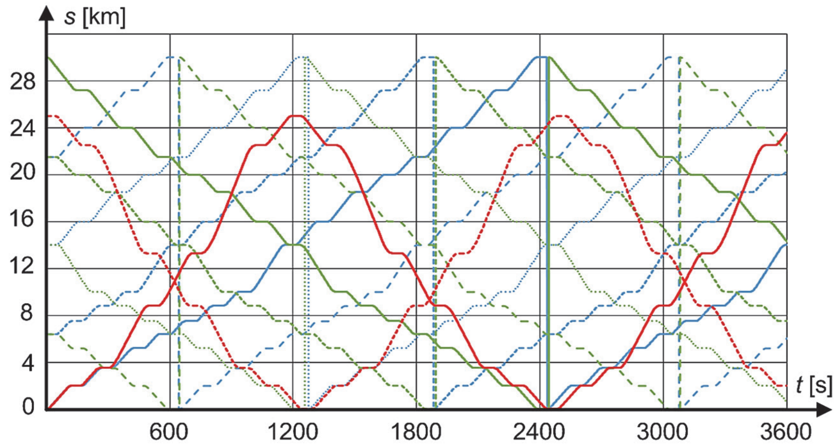
ing analysis, friction braking force is excluded from energy calculations. Motion resistance depends on route geometry and vehicle construction.
Control function is responsible for determining values of set velocity (or motive force), braking curves calculation and station stops. While in this situation network simplicity allows for open-loop movement regulation (every vehicle is running according only to pre-set schedule and velocity profile), control function is designed to cooperate with global regulator allowing for simulation of signalization or realistic traffic scenarios in future iterations.

2.2 Electrical system

Electrical system of developed program contains models of traction substations, catenary and vehicles' drives. Whole transportation network in 3 kV DC system can be depicted using equivalent electrical circuit, with substations modelled as real voltage sources, vehicles as controlled current sources and other elements (overhead contact line, rails, power cable) as resistances [6-8]. Layout of this circuit, however, is constantly changing due to vehicles movement, so parametrization of such system might be challenging [1]. Because of problematic modelling of train running through multiple sections using basic Simulink libraries like Power Systems or Simscape, dedicated functions were developed. Those were designed to allow for simultaneous simulation of the whole network, where vehicle nodes are not confined to single sector.
Electrical power of vehicle is calculated by adding mechanical power divided (multiplied if vehicle is braking) by variable drive efficiency factor and power of auxiliary needs (lighting, air conditioning, passenger information etc.). Energy consumed by vehicle is computed by integrating electrical power in time domain. Vehicle current is a result of division of electrical power and line voltage. Initial line voltage value is set as 3600 V, which in analysed system equals no-load voltage.
Model structure enables detailed analysis of energy losses in every element of the traction power supply. Insight in such data allows to identify when and where the most energy is lost. Losses in substations (power transformers and rectifiers) are calculated using efficiency map. Values of resistance of contact line, rails and power cable, parameters of substations and number of active power transformers are set independently for each section/element. Layout of power supply system depends solely on connections between each model, and sequence of sections which each of vehicles travel through is set by the control function.

3 Energy efficiency of analysed network

In this analysis, one hour of transport network operation was considered. Trains operated according to set schedule (Fig. 3) – delays and varying station dwelling time were disregarded. Round trip for every line takes about 40 minutes.



3.1 Energy and power losses

Analysis shown, that most energy is lost in vehicles' traction drive (Fig. 4). For a single train, momentary losses may reach up to 600 kW. Losses in substations are slightly higher than losses in catenary mostly because situations of multiple trains accelerating with full power happen in proximity of the substation. Losses in power cables are negligible – because of relatively short length of the cable.

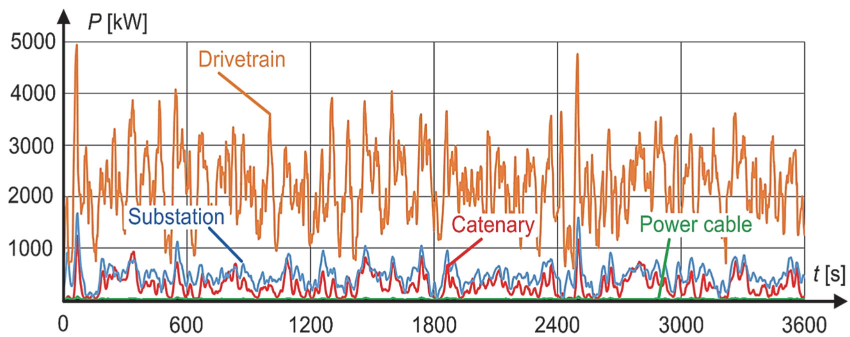


Figure 4 Waveforms of losses' power

Sometimes, losses in contact line get close or even exceed losses in substations – it is possible, when energy of regenerative braking is transmitted through catenary to other vehicles over longer distances.

3.2 Voltage assessment

Values of voltages on substations' output (Fig. 5), line voltage on vehicles' pantograph and mean useful voltages for substations and vehicles were computed.

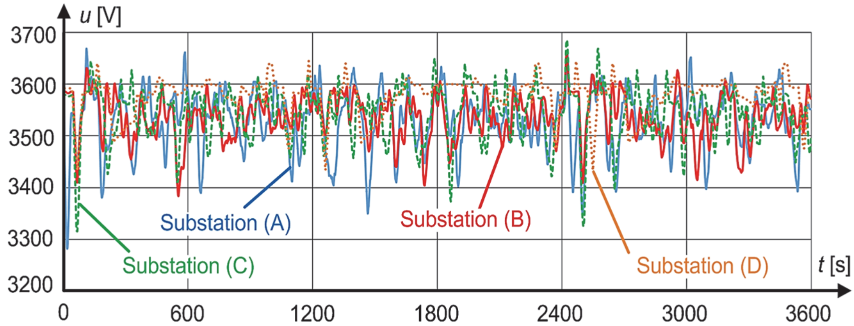


Figure 5 Waveforms of substation output voltages

It can be concluded, that analysed system met the requirements of EN 50163 norm [10]. Most of the time, voltages fluctuated between 3400 V and 3600 V with occasional peaks of about 3700 V and drops below 3300 V. Mean useful voltage values also complied with the norm, being at the level of about 3500 V for the substations and 3300 V – 3400 V for the vehicles.

3.3 Global system efficiency

One of the aspects of transportation network energy efficiency analysis is how much of energy fed from public power system is actually used to move the trains. Conducted simulation allows to compare values of both mechanical power of vehicles movement and electrical power of the substations (Fig. 6).

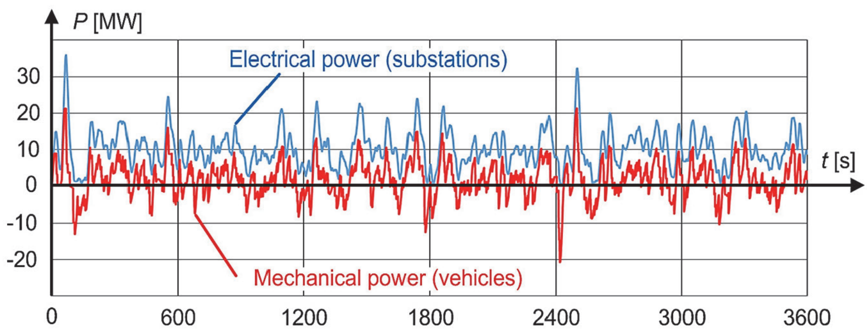


Figure 6 Comparison of mechanical and electrical power (fed to substations from public power system)

Mechanical power, constitutes for slightly less than half of total electrical power fed to the substations from public power network. It can, however, achieve negative values during braking phase, sending energy back to the drive. Situation, when multiple vehicles accelerate or brake translates to peak in losses' power and waste of regenerative braking energy. About 50 % of energy fed from public power system is used for trains' movement (Table 2).

Table 2 Energy consumption summary (values in MJ)

Parameter	Value	Efficiency
Total energy fed by power system	37240	-
Energy consumed - movement	18550	49,8 % (of fed energy)
Auxiliary needs	11520	80,7 % (with movement)
Drivetrain losses	7952	-
Substation losses	1718	-
Catenary losses	976	-
Power cable losses	26	-
Total consumed	40742	-
Regenerative braking	11790	-
Regenerated energy	3502	29,7 %

Large amount of consumed energy is used for powering auxiliary needs, including systems responsible for passenger comfort. If those would be considered as a part of transport system, then about 81 % of energy is used for passengers' sake. Efficiency of regenerative braking is slightly lower than 30 % – it could be improved by using optimized schedule, reversible substations or energy storages [9, 11].

4 Conclusions

Conducted analysis show, that significant part of energy consumed by transport network is dissipated in form of heat losses, and only about half is used to move vehicles. However, it is possible to determine, which element of system causes highest energy losses, and under what circumstances it happens. Developed model also provides insight in line voltage values, allowing for localization of points of highest voltage drops and reliability of power supply, measured by mean useful voltage. Such information can be helpful while planning modernization of power supply, timetabling or considering order of new rolling stock. Program structure allow further modifications, e.g. implementation of optimization algorithms or energy storage analysis.

References

[1] Alnuman, H., Gladwin, D., Foster, M.: Electrical modelling of a DC railway system with multiple trains, Energies, 11, 2018.

[2] Abrahamsson, L.: Optimal Railroad Power Supply System Operation and Design, KTH Stockholm, 2012.

[3] Douglas, H., Roberts, C., Hillmansen, S., Schmid, F.: An assessment of available measures to reduce traction energy use in railway networks, Energy Conversion and Management, 106 (2015), pp. 1149-1165

[4] Jakubowski, A., Jarzębowicz, L.: Constant vs variable efficiency factor in train run simulations, 26th International Workshop on Electric Drives: Improvement in Efficiency of Electric Drives (IWED), Moscow, Russia, 30 January – 2 February 2019.

[5] Bartłomiejczyk, M., Mirchevski, S., Jarzębowicz, L., Karwowski, K., How to choose drive's rated power in electrified urban transport?, EPE'17 ECCE Europe, Warsaw, Poland, 11-14 September 2017.

[6] Szeląg, A.: Electric traction – basics, Oficyna Wydawnicza Politechniki Warszawskiej, 2019.

[7] Tian, Z., Hillmansen, S., Roberts, C., Weston, P., Zhao, N., Chen, L., Chen, M.: Energy evaluation of the power network of a DC railway system with regenerating trains, IET Electrical Systems in Transportation, 10 (2015), pp. 1-9

[8] Tian, Z., Zhang, G., Zhao, N., Hillmansen, S., Tricoli, P., Roberts, C.: Energy Evaluation for DC Railway Systems with Inverting Substations, IEEE International Conference on Electrical Systems for Aircraft, Railway, Ship Propulsion and Road Vehicles and International Transportation Electrification Conference, ESARS-ITEC 2018.

[9] Koseki, T.: Technologies for Saving Energy in Railway Operation: General Discussion on Energy Issues Concerning Railway Technology, IEEJ Transactions on Electrical and Electronic Engineering, 5 (2018), pp. 285-290

[10] EN 50163: Railway applications – Supply voltages of traction systems, European Committee for Electrotechnical Standardization (CENELEC), 2004.

[11] Frilli, A., Meli E., Nocciolini, D., Pugi, L., Rindi, A.: Energetic optimization of regenerative braking for high speed railway systems, Energy Conversion and Management, 129 (2016), pp. 200-215

PREFERENCE FOR PUBLIC TRANSPORT VEHICLES IN SELECTED AREA OF PÚCHOV

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Abstract

The paper deals with ensuring the preference for public transport vehicles in the selected area of Púchov.The main aim of the paper is to analyze the current traffic situation within the selected area in the cadastral area of Púchov, near the "Nástupište, MŠK (Makyta)" bus stop, focusing on the delay of public transport vehicles due to the large number of bus lines arriving and departing from the stop, and emerging delays of these vehicles at entry and exit. Moreover, the partial aims of the paper are to evaluate traffic surveys, to design alternative solutions to the new organization of transport, and to select the optimal option which will predict the most significant time savings of public passenger vehicles. The necessary basis of the paper was the elaboration of several traffic surveys. The application of preferential tools enables to improve the throughput and fluency of problematic sections, to eliminate large delays achieved in the most stressed sections, to improve conditions for passengers, to increase attractiveness for public transport, also to observe timetables, reduce road accidents and driving safety, and ultimately reducing financial costs.

Keywords: preference, public transport, traffic survey, delay time

1 Introduction

In most large cities today, efforts are being made to make urban public transport (UPT) systems more efficient, in particular by removing the negative effects of individual bus transport (IBT) by gradually introducing elements of urban transport preference. To mitigate these impacts, dedicated lanes for public transport vehicles are being built in individual cities, streets are being unified or IBTs are being banned from entering central areas of cities. One of the effective measures to reduce public transport delays is to introduce a preference at light-controlled intersections, which cause large time losses for public transport [3, 6]. Movement of public passenger transport buses (PPT) in the street area, resp. over the local road, networks re generally quite complex. Preferential measure for buses On the infrastructure side, PPTs are a basic tool for minimizing the negative effects of individual road transport as a tool for ensuring the functions, quality, and attractiveness of bus transport as a reliable part of the integrated transport system. Maximizing the effects of PPT preference is only achievable with the effective and consistent application of preferential measures [4].

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2 Public transport in the monitored area

The current organization of transport in the assessed area is ensured using three uncontrolled intersections, two of which are used for entry and exit of public passenger transport (PPT) vehicles directly to/from the Makyta stop, and the third intersection is situated in the direction of the city center on the road I/49. At present, the entry and exit of vehicles to the Makyta stop re provided from two directions (Fig. 1). The yellow arrows in fig. 1 show the directions from which public transport vehicles enter the stop. The red arrows show the directions in which the vehicles leave the stop and join the street on 1. Mája. The assessed area includes 22 routes of suburban bus lines and 8 routes of public transport lines.

Figure 1 Map showing the distribution of individual inputs (left panel), Current organization of transport in the assessed area with indication of the direction of public passenger transport lines (right panel) [own study]

3 Traffic survey and analysis

Several traffic surveys were carried out in the assessed area since we wanted to approach the current situation in the assessed area during the simulations of individual models and we also wanted the model to be as accurate as possible. The performed traffic surveys were:
- directional traffic survey at the intersection of 1. Mája and Športovcov street,
- survey of the delay of public transport and regular bus service vehicles at the Makyta bus stop (MŠK platform) in the town of Púchov,
- processing of timetables,
- measurement of selected parameters= Detector = SIERZEGA SRA 5.4.1.

4 Proposal for a new transport organization through alternative solutions

The main goal of the changes in the organization of transport is to achieve an increase in the smoothness and safety of passage of all road users in the assessed area. However, in many cases, the proposal to change the organization of transport is not beneficial for all road users, including public passenger transport vehicles (PPT). For this reason, these calculation procedures can be replaced by the virtual reality of the modeled transport network - the transport model [8].
It was the use of the transport model that assessed the effects of several variants of the change in the organization of transport on public passenger transport vehicles in a defined area near the central urban zone of the town of Púchov. When modeling the traffic flow, we create real situations of a certain traffic problem. The proposal of changes in the organization of transport in the assessed area was made using the transport model Aimsun [7].

4.1 Assessment of the traffic situation using a traffic model

The transport model was developed in the transport - modeling software of the Spanish company TSS (Transport Simulation Systems) Aimsun. For each simulated variant, 36 simulations were performed (taking into account the standard deviation generated by the traffic model, corresponds to the accuracy of the results at 99 % confidence level with a confidence interval of 6). From the number of simulations, average values were calculated for the monitored indicators. The results were monitored and evaluated for public passenger transport vehicles and individual transport vehicles specifically with measurements on buses, to determine the variant that will achieve the best results in a given comparison of average values [7].
In order to better orient ourselves in the simulation model, I am using the function „Subpaths“ divided the intersection into individual entrances and exits. (Fig. 2). The given division is clearer when evaluating the individual characteristics of traffic flow for the current state and the proposed solutions.

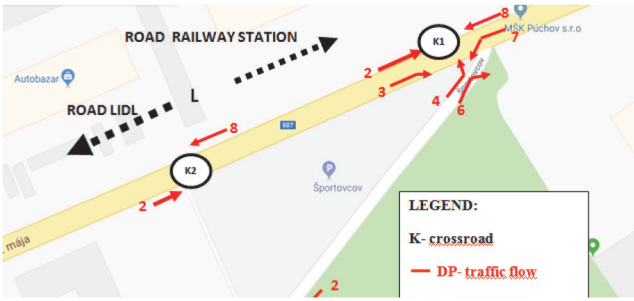

Figure 2 Division of individual entrances and exits [own study]

The current traffic situation was assessed in the transport model and several proposals were made to change the organization of traffic. The monitored indicators were evaluated separately for each means of transport. The results were evaluated for public passenger transport vehicles and buses. The following variants were simulated in the transport model (the current organization of transport in the area under consideration):

- Proposal 1: Design of light control at the intersection of streets on 1. Mája – Športovcov (entrance A),
- Proposal 2: Design of light control at the intersection of on streets Športovcov – 1. Mája (entrance B),
- Proposal 3: Reserved lane for public passenger transport vehicles at the intersection of streets on 1. Mája - Športovcov,
- Proposal 4: Change of transport organization of street on Športovcov - entrance to the Makyta stop.

5 Analysis of the obtained results and selection of the optimal variant

As already mentioned, 36 simulations were performed for each variant, from which the average values for individual monitored indicators were subsequently calculated, the monitored indicators include: waiting time, number of stops, total travel time, standing time, average speed, section speed and number of stops. These indicators were monitored separately for public passenger transport vehicles on the routes of public passenger transport lines passing through the assessed area. These parameters were also monitored for individual bus transport vehicles, with an evaluation compiled for the whole area under assessment, as shown in Tables 1 and 2.

Table 1 Overall comparison of public passenger transport lines with the current situation [own study]

Proposal	Line	Direction	Delay time [s]	Number of stops [st./veh.]	Total travel time [s]
current situation	Line1	Regular bus service from Lidl - directly	2,38	1	243,36
proposal 1			2,38	1	243,67
Proposal 2			2,33	1	243,36
proposal 3			2,31	1	244,09
proposal 4			1,96	1	243,76
current situation	Line2	Regular bus service from the railway station - directly	4,95	0,25	120,8
proposal 1			4,95	0,25	120,8
Proposal 2			4,72	0,24	120,13
proposal 3			5,19	0,26	123,55
proposal 4			4,23	0,2	120,3
current situation	Line 3	Public transport from the railway station - Makyta - back	25,34	2,19	118,27
Proposal 1			17,08	1,78	113,1
Proposal 2			25,13	2,17	117,93
proposal 3			25,95	2,25	118,89
proposal 4			26,52	2,39	114,54
current situation	Line 4	Regular bus service from Lidl via Makyta	10,6	1,42	123,88
proposal 1			11,38	1,47	124,81
Proposal 2			10,53	1,39	123,98
Proposal 3			10,06	1,42	123,45
proposal 4			11,27	1,36	119,88
current situation	Line 5	regular bus service from the station via Makyta	25,44	1,78	126,54
Proposal 1			24,87	1,72	118,02
Proposal 2			11,73	1,17	113,5
proposal 3			25,79	1,78	127,93
proposal 4			26,66	1,89	126,21

Based on the overall comparison of individual results (Tab.1), the most appropriate proposal, in terms of achieving the lowest value of the residence time for public passenger transport (PPT) vehicles at the entrances to intersections, can be considered proposal 2. The proposal changed the organization of transport construction of traffic lights inclusion (TLI) proposed at the intersection of streets on 1. Mája - Športovcov, which was to contribute in particular to reducing the waiting time of vehicles turning left from the main road of the street Športovcov. The waiting time was found to be 15.11 sec. The comparison of the results achieved on the individual assessed lines shows that in the case of all lines in the case of proposal 2 an improvement of the values of the assessed parameters compared to the current state was found. The second-lowest delay time was found in the case of proposal no. 1, for which a value of 17.37 sec. was found. The results of the performed simulations show that proposal 3 and proposal 4 do not affect the movement of PPT vehicles within the assessed area. In both cases, the values of the assessed parameters were compared with the current state, or worse.

The reserved lane itself (proposal 3) was not located in a place where there would be significant delays in PPT vehicles. The benefit of such a lane would rather be manifested at the entrance to the intersection, where PPT vehicles have to wait in a row of stationary vehicles in front of the intersection (controlled or uncontrolled), and thus there is a delay. In such a case, a reserved lane would allow the segregation of PPT vehicles from other road users and would contribute to shortening the delay time and thus the travel time itself. The same conclusion can be drawn in the case of a change in the organization of transport by redefining the main road at the entrance to the stop Makyta from Športovcov street (proposal 4). The intensity of vehicles on a given street is not so high as to significantly affect the departure of vehicles from the stop.

Table 2 Overall comparison of proposals for buses [own study]

Comparison	Delay time [s/km]	Stop time [s/km]	Number of stops [st./veh./km]	Speed [km/h]	Harmonic speed [km/h]	Total travel time [s]
current situation	18,91	10,23	1,03	25,15	21,41	0,20
proposal 1	17,37	8,59	0,99	25,37	21,79	0,20
proposal 2	15,11	6,97	0,93	25,40	21,87	0,20
proposal 3	18,96	10,45	0,93	25,16	21,46	0,20
proposal 4	18,64	10,16	1,03	25,48	21,69	0,20

A comparison of the individual proposals shows that the application of proposal 2 should achieve the highest time savings for public passenger transport (PPT) vehicles. This fact is influenced by the fact that at the intersection in question the number of vehicles turning left from the main road gradually increases (due to increasing the attractiveness of this area), while for this intersection no separate lane is built on the main road, as stated in the description of the current state. This affects not only the movement of PPT vehicles but also the movement of other vehicles in the superior current. The equipment of the traffic lights inclusion (TLI) junction will allow vehicles to turn left to leave the junction more quickly and thus also contribute to the shortening of delay times for PPT vehicles. To avoid these negative effects, it is necessary to consider the equipment of TLI exits and the preference of PPT vehicles, as based on the results of individual simulations in proposal No. 2.

6 Application of the selected optimal variant

Based on the comparison and evaluation of the obtained results, the optimal variant is proposal 2 = Design of light control at the intersection of on streets Športovcov – 1. Mája (entrance B). In this part of the article, we will present the application of the selected variant in the selected part of the town of Púchov. Figure 3 shows a view of the routing of public passenger transport (PPT) lines and the designation of the place where we propose a change in the organization of transport. Figure 4 shows there commendation proposal 2, where there would be a change in the organization of transport at the intersection of the street of Športovcov – 1. Mája. In the above, the equipment of the intersection is considered by a light signaling device.

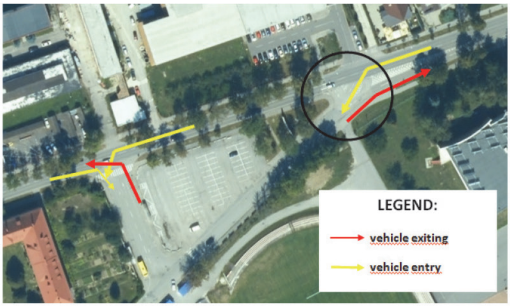

Figure 3 Map with the proposal of design 2 together with the indication of the direction of public passenger transport lines [own study]

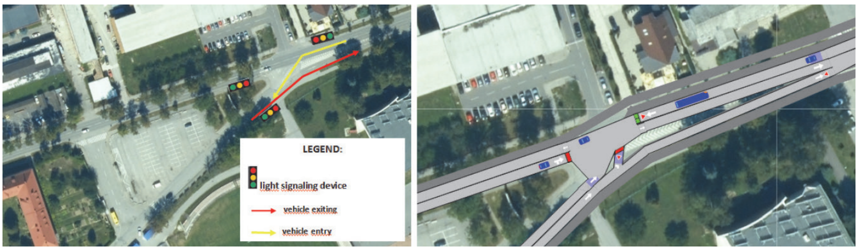

Figure 4 Recommended proposal to change the organization of transport (left panel), the changed state model in Aimsun (right panel) [own study]

The proposal considers that the traffic lights inclusion (TLI) will not manage traffic at the intersection at all times, it will be used only if a call is detected from a detector located at the entrance to the intersection in the direction away from the city center. Therefore, if a public passenger transport (PPT) vehicle arrives at the intersection, which wants to cross the intersection by turning left onto Športovcov street, by crossing the detector it will cause a signal change (it will cause a red signal) in all collision directions. This change will ensure that the vehicle passes smoothly through the intersection without stopping. If no challenge is detected on the detector, the intersection is not controlled by the TLI, but vehicles pass through the intersection as if they were at an uncontrolled intersection (as at present).

In case the traffic at the intersection in question would be controlled by traffic lights throughout the day (via the established signaling plan), the preference of PPT vehicles can be ensured by detectors located at individual entrances (since PPT vehicles pass through the intersection from all entrances). In such a case, however, it is necessary to ensure dynamic control of the intersection and we propose the following principle of operation, which is shown in Figure 5. [1, 5].

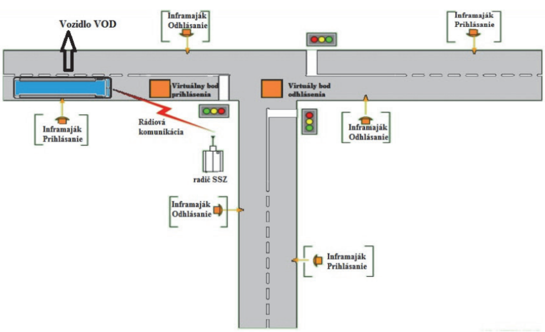

Figure 5 Principle of active preference of public passenger transport vehicles [own study]

7 Conclusion

The article aimed to analyze the current traffic situation within a selected area in the cadastral area of Púchov, near the stop „Nástupište, MŠK (Makyta)“ with a focus on delays of public transport vehicles due to entry and exit. The partial goal of the article was to evaluate traffic surveys, propose alternative solutions of the new transport organization and select the optimal variant, which will be expected to significantly save public passenger transport vehicles, while such a proposal can be considered in terms of simulations 2, as mentioned above in the text. In conclusion, we can note that the application of preferential instruments (measures) allows to improve the passability and smoothness of problem sections, eliminate large delays achieved in the busiest sections, improve driving conditions for passengers, increase attractiveness for public transport [2, 3].

References

[1] Kalasova, A., Cernicky, L., Kupculjakova, J.: The Impact of Public Transport Priority on the Traffic in the Choosen Part of the City of Zilina, Transport Problems, 9 (2014) 2, pp. 19-26

[2] Berezny, R., Konecny, V.: The impact of the quality of transport services on passenger demand in the suburban bus transport, 12th International Scientific Conference of Young Scientists on Sustainable, Modern and Safe Transport, eds. Bujnak, J., Guagliano M., Procedia Engineering, pp. 40-45, Amsterdam, Elsevier Science Bv, 2017.

[3] Medviď, P., Marián, G., Kubaľák, S.: Occupancy of Public Transport Vehicles in Slovakia, Transportation research procedia, 44 (2020), pp. 153 – 159, doi: https://doi.org/10.1016/j.trpro.2020.02.022

[4] Kalasova, A., Mikulski, J., Kubikova, S.: The Impact of Intelligent Transport Systems on an Accident Rate of the Chosen Part of Road Communication Network in the Slovak Republic, Challenge of Transport Telematics, Tst 2016, Communications in Computer and Information Science, Cham, 2016, pp. 47-58

[5] Culik, K., Harantova, V., Kalasova, A.: Traffic Modelling of the Circular Junction in the City of Zilina, Advances in Science and Technology-Research Journal, 13 (2019) 4, pp. 162-169

[6] Poliak, M., et al.: The Competitiveness of Public Transport, Journal of Competitiveness, 9 (2017) 3, pp. 81-97

[7] Manser, P., Becker, H., Hörl, S.W., Axhausen, K.: Designing a large-scale public transport network using agent-based microsimulation, Transportation Research Part A: Policy and Practice, 137 (2020), pp. 1-15

[8] Kupculjakova, J.: Scientific - peer-reviewed online journal: Active detection of public transport vehicles using ITS, www.svetdopravy.sk, 20.03.2020.

4

INFRASTRUCTURE AND VEHICLES: DESIGN, MODELLING, MONITORING AND CONDITION ASSESEMENT

ROAD WIDENING IN CURVES ACCORDING TO CROATIAN, REGULATIONS, GERMAN GUIDELINES AND COMPUTER SIMULATION OF VEHICLE MOVEMENT

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Abstract

Road widening is necessary in curves because the rear wheels of the vehicle are usually not steerable and their trajectory is described with a smaller radius than that of the front wheels. The maximum value of the road widening depends on the turning angle, the radius of the curve and the length of the vehicle, the wheelbase and the front overhang. The required road widening increases gradually with a higher turning angle and decreases gradually as the vehicle leaves the curve and continues in the direction. This paper presents the determination of road widening in curves according to the Croatian regulations, German guidelines and computer simulation of the vehicle movement. The road widening determined according to the above mentioned guidelines and regulations for certain design vehicles (truck with trailer, semi-trailer truck and bus), different radii of circular curves (from 12.5 to 500 m) and different turning angles (from 45 to 270°) is compared with the road widening determined by computer simulation of vehicle movement with the Autodesk software Vehicle Tracking.

Keywords: road widening, circular curves, design vehicles, vehicle trajectories

1 Introduction

When the vehicle travels through a horizontal curve, it occupies a greater width of road than when it travels in one direction. The front wheels turn and the rear wheels do not follow their trajectory and describe an arc of smaller radius than the front wheels (Figure 1). Therefore, it is necessary to widen the road in curves, depending on the size of the radius of the horizontal curve and the dimensions of the design vehicle [1]. As a rule, the widening of the road in curves for all lanes is performed on the inside of the curve, in exceptional situations it can be performed on the inside and outside of the curve or only on the outside of the curve (for example, roads with hairpin curves) [1]. Usually the transition from the non-widened road to the widened road is performed along the length of the transition. Along the entire length of the arc, the road should be widened to the full amount [1-3]. This paper gives an overview of the currently valid regulations and guidelines from Croatia [1, 4] and Germany [2, 5], which refer to the determination of the dimensions of road cross-section elements (width of lanes and width of curbs), dimensions of design vehicles operating on public roads and other methods for determining road widening in circular arcs.

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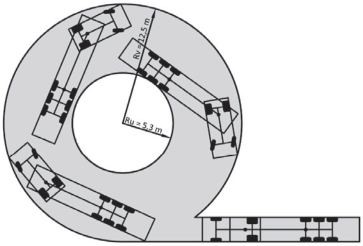


Figure 1 Simulation of vehicle movement in circle

Based on the formulas defined in the above-mentioned regulation and guidelines, the values of required widening in curves were calculated for different radii of circular arcs from 12.5m to 500 m. The calculated values of roadway widening in circular curves were compared with the values obtained from the simulation of vehicle movement truck with trailer, semi-trailer truck and bus. Vehicle movement simulation was performed using Autodesk software Vehicle Tracking [6]. The vehicle trajectories were drawn for radii from 12.5 to 500 m and for different turning angles from 45° to 270°. A total of 162 combinations were processed. The research was carried out to determine the relationship between the required values of widening obtained according to different criteria described in regulations and guidelines [1, 2] and the values obtained based on the simulation of movement of design vehicles [3].

2 Guidelines, regulations and computer simulation of vehicle movement

Extent of road widening on curves are usually specified in guidelines and regulations. The only accurate method to determine the road widening is by driving the vehicle in a real traffic situation or on a test site [7, 8]. However, conducting such a test is time-consuming, organizationally demanding and expensive, so computer simulations have been developed. The following text describes how the widths of the traffic and shoulder lanes are chosen, how the road widening along the entire arc is determined, and how the dimensions of the design vehicles are chosen according to regulations and guidelines from Croatia [1, 4, 9] and Germany [2, 5]. The characteristics of the software used to simulate the vehicle movement are also briefly described.

2.1 Croatian Regulations

Traffic lanes are part of the roadway and must be wide enough for vehicles to move freely. The width of the traffic lanes S_{pt} (Table 1) depends on the design speed V_P [1]. The shoulder lanes S_{rt} (Table 1) are made continuous on the whole road section and their width depends on the road category and the width of the lane [1].

Table 1 Traffic and shoulder lane widths depending on the design speed [1]

V_P [km/h]	≥ 120	100	90	80	70	60	50	40
S_{pt} [m]	3.75	3.50	3.50	3.25	3.00	3.00	3.00	2.75
S_{rt} [m]	0.50	0.50	0.50	0.30	0.30	0.20	0.20	-

According to the regulation [1], the widening of a lane along an arc of a circle for radii R ≥ 45 m is determined according to the following formulas:

$$\Delta s = 10/R \tag{1}$$

$$\Delta s = 32/R \tag{2}$$

$$\Delta s = 42/R \tag{3}$$

Δs [m] - lane widening,
R [m] - curve radius.

Equation (1) is used to determine the lane widening for a passenger cars, eqn (2) for trucks and buses, and eqn (3) for trucks with a trailers, semi-trailer trucks and articulated buses. The detailed dimensions of the design vehicles (axle distances, front and rear overhang lengths, attachment points, etc.) for determining the lane widening in circular curves are not defined in regulation [1]. According to the regulation [9], the maximum permissible lengths of vehicles in traffic on Croatian roads are: 12.0 m for trucks, 16.50 m for trucks with semi-trailer, 18.75 m for trucks with trailer, 13.5 m for buses with two axles, 15.0 m for buses with three axles and 18.75 m for articulated buses. The only recent document in Croatia that defines design vehicles with all dimensions is the "Guidelines for the Design of Roundabouts on State Roads" [4], only trucks with trailers and semi-trailers.
Road widening values are usually determined for both lanes for the same design vehicle. In the circular arc, road widening values have a constant value, and the road is usually widened for both (or more) lanes on the inside of the curve. Based on equations (1) (2) and (3), the road widening values can also be determined for curves with a radius of 25 to 45 m when the turning angle of the curve is smaller than the right angle of 90° [1]. If the radius of the curves is less than 45 m and the turning angle is greater than or even 90°, the road widening should be determined based on the trajectories of the design vehicle [1]. Road widening is not provided if the sum of individual lane widening Δs for the total number of lanes in the curves is not more than 0.2 m and the width of the roadway (traffic and shoulder lanes) is less than or equal to 6.0 m, and if the sum of individual lane widening Δs is not more than 0.3 m and the width of the roadway is greater than 6.0 m. The transition from the non-widened road to the widened road in the circular arc is carried out along the length of the transition and the change is non-linear.

2.2 German guidelines

In the German guidelines [2], the lane widths are related to given road cross-sections and are in the range of 2.50 to 3.50 m. A lane width of 2.50 m applies to a two-directional road without a dividing line, for road class EKL 4 and a design speed of 70 km/h. A lane width of 3.25 m represents the width of the second or third lane at the cross sections for roads of class EKL 1, EKL 2 and EKL 3 for the design speed of 90, 100 and 110 km/h. A lane width of 3.50 m is the usual lane width at the cross sections for roads of classes EKL 1, EKL 2 and EKL 3 for the design speed of 90, 100 and 110 km/h. The usual width of the shoulder lane is 0.50m and can be increased to 0.75 m on sections of three-lane roads to provide a stopping width for maintenance vehicles [2]. According to the German guidelines [2], in curves with radius less than 200 m, the road must be widened by a corresponding amount, whereby the widening must take place over the entire length of the arc at the inner edge of the curve. The roadway widening is calculated according to the following formula [2]:

$$i = 100/R \quad (4)$$

i [m] - roadway widening,
R [m] - curve radius.

The transition from the non-widened road to the widened road in the circular arc is carried out along the length of the transition and the change is usually linear [2]. The dimensions of 13 relevant vehicles (passenger cars, buses, trucks, etc.) are defined in the guidelines [5].

2.3 Computer simulation of vehicle movement

With the development of information technology in the last 15 years, numerous software have appeared on the market to simulate the movement of vehicles. The two most famous programs are Vehicle Tracking (Autodesk) [6] and AutoTURN (Transoft Solutions) [10]. These programs are used as additional applications in the interface CAD. The main advantage in computer simulation of vehicle movements is that it is relatively easy and fast to define different types of vehicles with arbitrary dimensions, set different guidance lines and vary the turning angles, and the representation of the trajectory of the vehicle movement is fast and accurate. The distributors of the software available on the market point out that they are not responsible for errors and damage that may occur due to their use. Since the method of computer drawing of the vehicle trajectory is the basis for conducting this research work and the manufacturer did not provide evidence of the accuracy of the software in relation to the actual behavior of the vehicle, an accuracy analysis was conducted for Vehicle Tracking. The results [7] showed that the deviations are greater than the actual measured values on the polygon by a maximum of 14 cm. According to [8], the average deviation between the values obtained by field measurements and those obtained by simulating the vehicle motion in the software Vehicle Tracking is -7 cm. In 95 % of the cases, the simulation resulted in larger widths of the surfaces crossed by the vehicles than the real ones, which should be considered from the point of view of safety design. The results of the T-test [8] showed that with 95 % confidence it can be claimed that there is no difference between the observed variables.

3 Research methodology

To investigate relations between the required widening values, circular arcs with the following radii were selected: 12.5, 25, 45, 75, 120, 150, 175, 250, 350 and 500 m. Research procedure consisted of the following steps:
- determination of road widening in all above mentioned curves for a single lane according to Croatian regulations [1] for the longest vehicles,
- determination of road widening in all above mentioned curves for a single lane road according to German guidelines [2],
- determination of road widening in all above mentioned curves for a single lane according to the software Vehicle Tracking for three design vehicles (three-axle bus, truck with trailer and truck with semi-trailer) and different turning angles from 45 to 270° (Figure 2). During the test vehicles must follow curves shown on Figures 1, 2 with the front left outermost point.
- comparison of results.

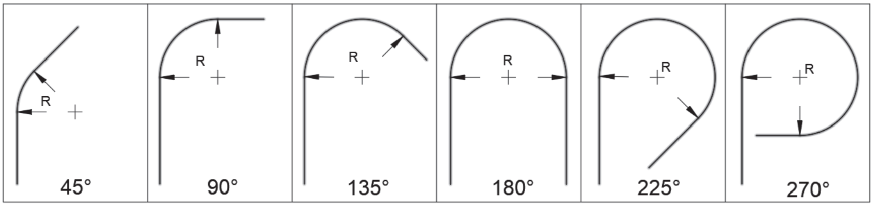


Figure 2 Curves for different turning angles

Three axle bus with a length of 15.0 m from German guidelines [5], a semi-trailer truck with a length of 16.5 m and a truck with a trailer with a length of 18.75 m from Croatian guidelines [4] were selected for testing in Vehicle Tracking software (Figure 3). These vehicles are selected because preliminary tests have shown that a 15.0 m bus is less favorable than an articulated bus, and trucks with a trailer and semi-trailer from Croatian guidelines are less favorable than German ones [5].

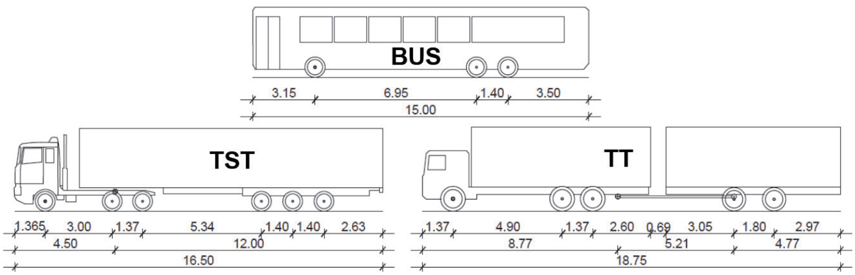
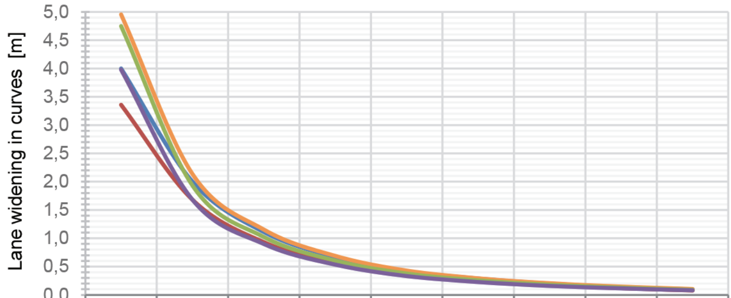


Figure 3 Design vehicles [4]

4 Research results

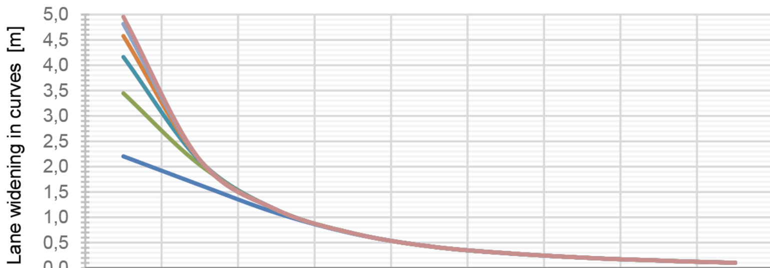
Research results showed the following (Figures 4, 5, 6, 7):

- eqn (3) gives smaller widening values of compared to eqn (4) and can only be used to determine the widening for a truck with a trailer for radii of curves greater than 25 m,
- eqn (4) gives higher widening values than simulation for both trucks and for all radii of curves greater than 25 m,
- the bus occupies the largest surface width for all radii of curves and all turning angles,
- the largest widening value is required for a turning angle of 270 ° and a circular arc radius of 12.5 m for all design vehicles,
- the influence of turning angles on the required widening values is more significant for radii less than 45 m, ie the higher the turning angle, the greater the widening value is required.



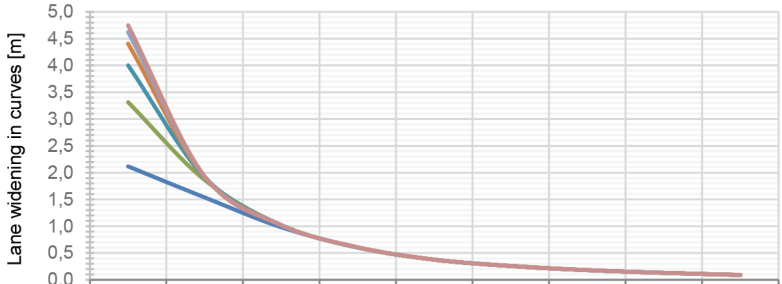
	12,5	25,0	45,0	75,0	120,0	175,0	250,0	350,0	500,0
GER (i/2)	4,00	2,00	1,11	0,67	0,42	0,29	0,20	0,14	0,10
CRO (Δs)	3,36	1,68	0,93	0,56	0,35	0,24	0,17	0,12	0,08
BUS-270°	4,96	2,14	1,15	0,69	0,43	0,29	0,21	0,15	0,10
TST-270°	4,75	1,93	1,02	0,61	0,38	0,26	0,18	0,13	0,09
TT-270°	3,98	1,68	0,90	0,53	0,33	0,23	0,16	0,11	0,08

Figure 4 Lane widening according to Croatian regulations [1], German guidelines [2] and software simulation of design vehicles movement



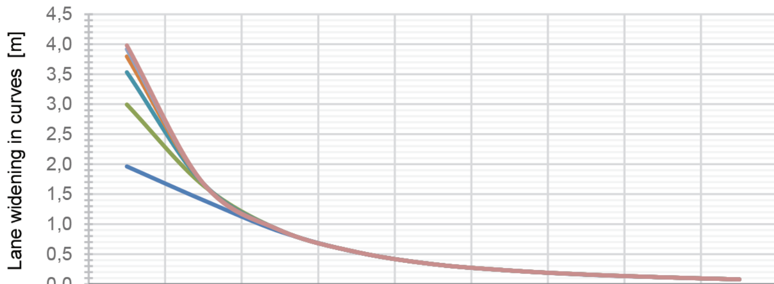
	12,5	25,0	45,0	75,0	120,0	175,0	250,0	350,0	500,0
45°	2,20	1,64	1,09	0,68	0,43	0,29	0,21	0,15	0,10
90°	3,45	2,04	1,15	0,69	0,43	0,29	0,21	0,15	0,10
135°	4,16	2,12	1,15	0,69	0,43	0,29	0,21	0,15	0,10
180°	4,58	2,13	1,15	0,69	0,43	0,29	0,21	0,15	0,10
225°	4,82	2,13	1,15	0,69	0,43	0,29	0,21	0,15	0,10
270°	4,96	2,14	1,15	0,69	0,43	0,29	0,21	0,15	0,10

Figure 5 Lane widening in curves for three-axle bus, l=15.0 m



	12,5	25,0	45,0	75,0	120,0	175,0	250,0	350,0	500,0
45°	2,12	1,54	0,98	0,60	0,38	0,26	0,18	0,13	0,09
90°	3,31	1,86	1,02	0,60	0,38	0,26	0,18	0,13	0,09
135°	4,00	1,92	1,02	0,60	0,38	0,26	0,18	0,13	0,09
180°	4,41	1,93	1,02	0,61	0,38	0,26	0,18	0,13	0,09
225°	4,62	1,93	1,02	0,61	0,38	0,26	0,18	0,13	0,09
270°	4,75	1,93	1,02	0,61	0,38	0,26	0,18	0,13	0,09

Figure 6 Lane widening in curves for truck with semi-trailer, l=16.5 m



	12,5	25,0	45,0	75,0	120,0	175,0	250,0	350,0	500,0
45°	1,96	1,40	0,88	0,53	0,33	0,23	0,16	0,11	0,08
90°	3,00	1,65	0,90	0,53	0,33	0,23	0,16	0,11	0,08
135°	3,53	1,67	0,90	0,53	0,33	0,23	0,16	0,11	0,08
180°	3,80	1,68	0,90	0,53	0,33	0,23	0,16	0,11	0,08
225°	3,92	1,68	0,90	0,53	0,33	0,23	0,16	0,11	0,08
270°	3,98	1,68	0,90	0,53	0,33	0,23	0,16	0,11	0,08

Figure 7 Lane widening in curves for truck with trailer, l=18.75 m

5 Conclusions

The research showed that the widening of the road for the longest vehicles according to the German guidelines corresponds better to real vehicle movement than the Croatian regulations. Furthermore, the research shows that when the width of the non-widened roadway is 3.00 m, no widening is required for curve radii greater than 250 m, and when the width of the non-widened roadway is 3.50 m, no widening is required for radii greater than 120 m either. For curve radii smaller than 45 m, the widening must be determined based on the vehicle movement trajectories due to the high influence of the turning angle on the required size of the widening. All these facts indicate the need to update the existing regulations and guidelines in this respect.

References

[1] Pravilnik o osnovnim uvjetima kojima javne ceste izvan naselja moraju udovoljiti sa stajališta sigurnosti prometa (NN, 110/01)

[2] Richtlinien für die Anlage von Landstraßen (RAL), Ausgabe 2012. Forschungsgesellschaft für Straßen- und Verkehrswesen, (FGSV Verlag), Köln 2012.

[3] Tomić, I.: Proširenje kolnika u kružnim lukovima, diplomski rad, Sveučilište u Zagrebu, Građevinski fakultet, 29.06.2017.

[4] Smjernice za projektiranje kružnih raskrižja na državnim cestama, Građevinski fakultet Sveučilišta u Rijeci, Rijeka, srpanj 2014.

[5] Bemessungsfahrzeuge und Schleppkurven zur Überprüfung der Befahrbarkeit von Verkehrsflächen, FGSV Verlag, Köln, 2001.

[6] Autodesk Vehicle Tracking. Autodesk, San Rafael, CA, 2020.

[7] Korlaet, Ž., Dragčević, V., Stančerić, I. Designing Criteria of Acute Angle Four-Leg Intersection At-Grade. Proceedings of the 4th International Symposium on Highway Geometric Design. Ed by Harwood, D.W.; Garcia Garcia, A. Valencia: Editorial Universitat Politecnica de Valencia, 2010.

[8] Džambas, T.: Oblikovanje standardnih turbokružnih raskrižja na temelju zakonitosti kretanja mjerodavnog vozila– disertacija, Sveučilište u Zagrebu, Građevinski fakultet, 11.07.2018.

[9] Pravilnik o tehničkim uvjetima vozila u prometu na cestama (NN, 85/16, 24/17, 70/19, 60/20)

[10] AutoTURN, Transoft Solutions Inc. Richmond, BC, Canada, 2020.

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THE SPEED FACTOR IN SWEPT PATH ANALYSIS

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Abstract

The determination of the geometric vehicle movement is significant for the appropriate design of a road element, such as an intersection or a parking lot, because it ensures safe, smooth and without abrupt changes movements especially for heavy vehicles. Consequently, the accurate and correct swept path analysis of the vehicles determines the geometry of the horizontal alignment. Also, the selection of the design vehicle is a factor that affects the geometric characteristics of the analysis. The AASHTO Green Book presents the minimum turning paths, the maximum steering angle and the minimum centerline turning radius (CTR) for typical design vehicles. In order to simplify the geometrical problem of swept path analysis, the speed in sharp curve road is considered to be low and more specifically less than 15 km/h. However, this condition does not represent the actual vehicle movement, gap that the present paper aims to bridge by performing swept path analysis for increased travel speeds. There are only few cases, especially along urban road network that the lateral force applied on the vehicles that traverse horizontal transition curves are neglected due to low travel speed. On the contrary, in other road projects the transition curve is an integral design element and have advantages in geometric regularity of heavy vehicles movement because of their steering mechanism. Based on the literature review, in this study the design vehicles paths which are considered as clothoid shapes are correlated with their corresponding travel speeds. The implemented methodology considers various design vehicles which travel in various speeds, performing U-Turns.

Keywords: swept path, turning path, speed

1 Introduction

Swept path analysis is predominantly a geometry-based approach due to the fact that tire mechanics, in most situations have insignificant effect to offtracking simulation. Apparently offtracking is highly correlated with the travel speed of the vehicle under investigation. Particularly the more the speed travel the more the centrifugal forces acting on the turning vehicle. Nevertheless, this effect results in less offtracking distances and hence low-speed offtracking analysis errs on the side of safety [1]. Consequently, in low-speed offtracking approach, the problem is transformed to a pure geometrical one whilst in high-speed approach more variables have to be taken under consideration. The fundamental assumption in low-speed swept path analysis is that the truck driver is capable of steering from a straight line to a circular arc immediately without traversing a transition path, for instance a clothoid. This assumption however is not valid in high speeds and hence a comprehensive analysis should take into account the path curvature as well as the travel speed of the design vehicle [2], [3]. It is very well known that the path of any point of the front axle of a vehicle traversing a circular curve is an arc whereas the path of any point of the rear axle is a tractrix. Between the

tangent and the arc, the shape of the vehicle's path the so-called transition path, depends on two primary conditions: the steering speed and the speed of the vehicle itself. Although many researchers stated that a clothoid describes better the said transition path, it has been documented that this is valid only under certain circumstances [4]. Therefore, the knowledge of the steering and travel speed of any vehicle negotiating a roundabout is of paramount importance in order to design a safe circular intersection or evaluate an existing one. Besides as inferred by Mussone et al [5], the use of simulation software packages, which provide efficient swept path analysis, combined with proper speed prediction models is a very handy tool to investigate the performance of a roundabout. They also mention that design consistency evaluation of road elements like roundabouts are predominantly based on the fluctuation of the vehicles' operating speed [5].
The interaction between travel speed and traversed path was one of the subjects examined in a research study conducted by Wolfermann et al [6] who aimed at developing a model in order to predict the speed profiles of turning vehicles at signalized intersections. They concluded that the approach, exit and minimum travel speed is related to the traversed path of the vehicles and consequently to the intersection's properties. The developed model encompassed the influence of the driving behaviour and other parameters to the speed profile of the vehicles [6]. Likewise, few years later Park et al [7] concluded that the assumption of identical traversed paths in roundabouts is not realistic. Case studies in several intersections in US and Japan revealed that as the turning manoeuvres increase so do the path variation [7].
In 1986 Sayers [8] summarized the research studies conducted until then regarding the developed offtracking models. He also proposed a graphical method to simulate swept path analysis of any type of vehicle negotiating any type of turn at low speed using Apple II computer. However, it was stated that the designers employed methods which were not capable of calculating low speed offtracking for transient paths [8] e.g. horizontal curves without transition segments where truck rollovers might occur [9].
The magnitude of speed is highly correlated with accidents frequency especially when it is accompanied with steering manoeuvres either abrupt or not e.g. curved road sections, intersections, roundabouts etc. The combination of speed and steering, results in rearward amplification which poses a great problem to multiunit vehicles. When the rearward unit of the vehicle is overamplified the vehicle overturns [10]. Multi trailer trucks are more likely to exceed rollover thresholds compared to semitrailers [11]. Apparently, the contribution of superelevation and suspension system is substantial with regards to the prevention of this phenomenon [12]. Moreover, poor braking efficiency of unload trucks in conjunction with the lower frictional capabilities of truck tires compared to passenger car's ones pose a great risk for the truck drivers who negotiate roundabouts in higher speeds [13].
A study conducted in Kentucky [14] revealed that double trailers trucks are more likely to be involved in overturning accidents compared to single unit trucks. However, the latter have higher accident likelihood at intersections [14]. The size and the number of the articulation points affect the offtracking and safety performance of large vehicles. For instance, a very effective measure to increase the stability of the large vehicles is the enlargement of their width which enhances the rollover resistance [15] whereas long vehicles of few articulation points are associated with greater low-speed offtracking [11]. The latter is critical when designing roundabouts, intersections and sharp horizontal curves where low speeds are to be expected [16].
The three different types of offtracking analysis i.e. low speed, high speed steady and high speed transient offtracking, are implemented individually and in combination, in an 8-axle vehicle in a research study from New Zealand [17]. The authors showed that depending on the travel speed and the radius of the turn the offtracking of the vehicle might be more than twice its width. In general, low travel speeds and small radius of turn force the vehicle to

off-track more. It is also confirmed that the magnitude of offtracking also depends on the number of the articulation joints of the vehicle meaning that the more the number of the joints the less the offtracking [17].
In the present paper, the offtracking in low speed and high-speed steady will be investigated. In a U- Turn the large vehicles travel at low speed because of the small turn radii. Therefore, in order to achieve increased speed, a U- Turn form with a clothoid transition curve will be used.

2 Methodology

2.1 U-Turn and clothoid paths

In the present paper, a U- Turn form is used, namely a 180º turn that has 14.46 m radius of circular arc. The geometry of the form is based on the minimum turning path template of single-unit truck (SU-12 [SU-40]) according AASHTO Green Book 2011 [18]. The choice of this form was made, because it can accommodate different types of vehicles, such as passenger cars, single unit trucks, semi- trailer trucks and buses, without overstepping the max steering angle of each design vehicle.
Generally, the speed of vehicles that move in U- Turn, is less than 15 km/h according AASHTO Green Book 2011 [18] due to constant change of deflection angle. Hence, in order to increase vehicle's speed, a steering path configuration using straight lines, clothoids and a circular arc is considered. Clothoid is a transition curve with linear change of its curvature relative to its length. Because of its geometry clothoid also allows the increasing change of deflection angle and therefore the low maintenance of turn rate. In addition, clothoid permit the smooth application of the centrifugal force and consequently the driving comfort. For that reason, in this research clothoids are used as the transition sections between straight lines and circular arcs in order to approach the path that the vehicle drivers probably choose to travel in high speeds in U- Turns.

2.2 Swept path analysis

As design vehicles a passenger car (P-Car), a single-unit truck with total length 9.14 m (SU-9), a single-unit truck with total length 12.04 m (SU-12) and an intercity Bus with total length 13.86 m (BUS-14) are used according AASHTO Green Book 2011 [18]. Two turning paths are assumed in a U- Turn form that are the same for all design vehicles. The first steering path has the following form: two straight lines in the entry and the exit of the form that tangentially connect with a circular arc of 14.46 m radius. The second steering path has the following form: two straight lines in the entry and the exit of the form that tangentially connect with clothoids with parameter A = 15 and length L = 16.50 m, which are tangentially connected to a circular arc of 13.64 m radius.
The steering path concurs with the design of the centerline of the front axle. The description of the path that the rear axle of a vehicle follows for a given steering curve is called tractrix of the steering curve and is described by a complex curve. The swept path analysis estimates the paths of the front and the rear wheels and ultimately the off- tracking of the vehicle.
In the present paper, the swept paths are generated in Anadelta Tessera software that is considered the most accurate according to the researchers [19]. The basic theory of the program is that the rear axle is computed on the straight line that connects the middle of the step distance with the old location of the rear axle (Fig.1) [19].

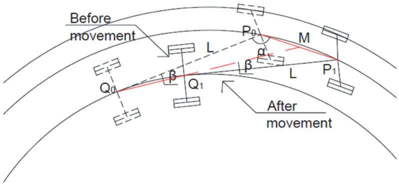


Figure 1 Calculation method of offtracking

2.3 Speed estimation

The implementation of the swept path analysis requires the calculation of the steering angle in each step. Subsequently, in each path position the turn rate R is estimated using the following equation:

$$R = \frac{STA_2 - STA_1}{PL_2 - PL_1} \tag{1}$$

where STA_1(⁰) and STA_2(⁰) are the steering angles in the initial and in the final position of the step path respectively, and PL_1(m) and PL_2(m) are the path length angle in the previous and in the present position of step path respectively.

Subsequently, the maximum turn rate maxR of the path is selected and the design speed V (km/h) can be determined by the following equation [20]:

$$\max R = \frac{\max STA}{V \cdot t} \Rightarrow V = \frac{\max STA}{\max R \cdot t} \tag{2}$$

where maxSTA (⁰) is the max steering angle path and t (s) is the lock to lock time.
Lock to lock time t is considered for all types of design vehicles equal to 6 s.

3 Results & Analysis

3.1 Swept path analysis

The swept path analysis described in Chapter 2.2 depends on the design of the steering path. The Fig. 2 and Fig. 3 show the swept path analysis for each design vehicle using a steering path form with and without clothoid.

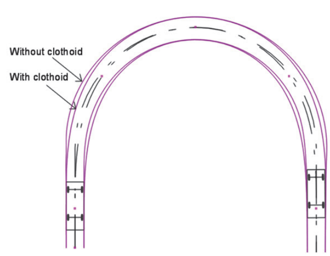
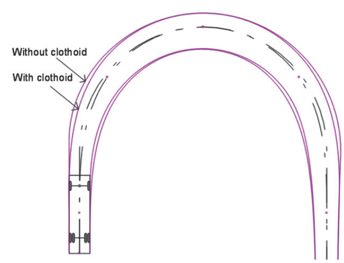
Swept path analysis for P- Car	**Swept path analysis for SU-9**
	

Figure 2 Swept path analysis for P-Car and SU-9

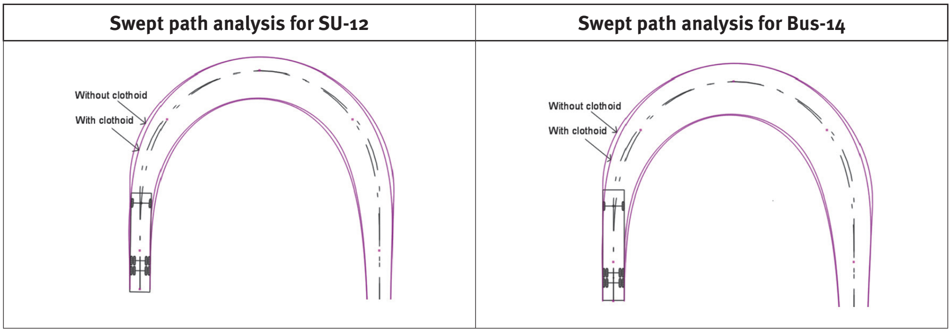


Figure 3 Swept path analysis for SU-12 and BUS-14

Swept path analysis of the four design vehicles show that the off tracking in the steering path without the use of a clothoid is greater than the offtracking in the steering path with the use of a clothoid. In addition, in this U- Turn form the intercity bus (BUS-14) takes up the greatest width when turning compared to the other design vehicles because of its great length (13.86 m).

3.2 Speed analysis

The design speeds are calculated according to the swept path of the design vehicles (Fig. 2, Fig. 3) i.e. the maximum turn rate is defined for each steering path (with and without clothoid). The speeds of the design vehicles are estimated according the Eq. (2). as well, the difference of the speeds between the U-Turn form without the use of clothoid and with the use of clothoid.The results are encompassed in Table 1.

Table 1 Speed analysis for design vehicles

Design vehicle		U- Turn form without clothoid		U- Turn form with clothoid		
		maxR	Speed [km/h]	maxR	Speed [km/h]	Dif. (%)
Passenger Car		3.82	9.93	0.86	44.27	3.46 %
maxSTA[º]	31.60					
SU-9		3.88	9.83	1.47	25.91	1.64 %
maxSTA[º]	31.8					
SU-12		3.90	9.79	1.75	21.86	1.23 %
maxSTA[º]	31.8					
BUS-14		3.91	13.89	1.91	28.37	1.04 %
maxSTA[º]	45.2					

According to the Table 1, the estimations of speeds in U- Turn form with clothoid are greater than in U- Turn form without clothoid. The passenger car has the greatest difference between the speed values. The intercity bus develops the highest speed of the large vehicle's despite of its great length.

4 Conclusion

According to the preceding investigation, the following conclusions can be drawn for the movement of vehicles in U- Turn and their speed estimation:

- The steering path using a clothoid allows higher speeds than that of the steering path without a clothoid.
- The use of a clothoid reduces the off-tracking distance in all of the design vehicles.
- The application of a clothoid permits the large vehicle to follow the steering path at speeds greater than 21km/h with an increase in the turn rate. Hence, the maximum steering angle has a significant impact in the speed estimation.
- Clothoid geometry permits the increased change of the deflection angle and therefore the turn rate kept small. The smaller the maximum turn rate, the higher the design speed.

The preceding analysis reveals that the precise tracking of a vehicle performing a U-Turn has not be determined yet and consequently further research is recommended on this topic. Finally, this research can be very helpful to designers, who deal with the movement of vehicles at roundabouts, U-Turns, vehicle parking and intersections, as well to developers of swept path analysis simulation software tools.

References

[1] Carrasco, M.S.E.: Turning vehicle simulation: Interactive computer-aided design and drafting application, Transportation Research Record, 1995.

[2] Harwood, D.W., Mason, J.M., Glanz, W.D., Kulakowski, B.T., Fitzpatrick, K.: Truck characteristics for use in Highway Design and Operation, U.S. Department of Transportation, 1990.

[3] Dragcevic, V., Korlaet, Z., Rukavina, T., Lakusic, S.: Three-leg Intersection at-Grade – The Right Edge Forming Test, 3rd International Symposium on Highway Geometric Design, Chicago Illinois, United States, 29.6.-1.7.2005.

[4] Rolland, F., Spacek, P.: The Effects of Legal Changes on Highway Geometric Design, 3rd International Symposium on Highway Geometric Design, Chicago Illinois, United States, 29.6.-1.7.2005.

[5] Mussone, L., Matteucci, M., Bassani, M., Rizzi, D.: An innovative method for the analysis of vehicle movements in roundabouts based on image processing, Journal of Advanced Transportation, 47 (2011) 6, pp. 581–594, doi: 10.1002/atr.184

[6] Wolfermann, A.W., Alhajyaseen, W.K., Nakamura, H.: Modeling speed profiles of turning vehicles at signalized intersections, 3rd International Conference on Road Safety and Simulation, Indianapolis, USA, 14-16.9.2011.

[7] Park, H.J., Alhajyaseen, W., Iryo-Asano, M.: Comparative Analysis on the Maneuver of Turning Vehicles at Crosswalks in United States and Japan, Road Safety & Simulation International Conference, pp. 952-964, Orlando, Florida USA, October 2015.

[8] Sayers, W.M.: Vehicle offtracking models, Symposium on geometric design for large trucks, 1986, pp. 53-62

[9] Smith, L.B.: Existing design standards, Transportation Research Board, 1986.

[10] Freitas, D.M.: Large-truck safety research, 1986.

[11] Federal Highway Administration: Guide for monitoring and enhancing safety in the National Truck Network, 1988.

[12] Glauz, D.W., Harwood, W.D.: Superelevation and body roll effects on offtracking of large trucks, 1991.

[13] Olson, L.P., Cleveland, E.D., Fancher, S.P., Kostyniuk, P.L., Schneider, W.L.: Parameters affecting stopping sight distance, National Highway Research Program, Report 270, 1984.

[14] Agent, K.R., Pigman, J.G.: Evaluation of highway geometrics related to large trucks, Kentucky Transportation Cented and Federal Highway Administration, 1991.

[15] Ervin, D.R., Nisonger, L.R., MacAdam, C.C., Fancher, S.P.: Influence of size and weight variables on the stability and control properties of heavy trucks, Federal Highway Administration, 1986.

[16] Transportation Research Board, Providing acess for large trucks, 1989.

[17] Transport Engineering Research New Zealand, Offtracking of 23 m HPMVs compared to Standard Vehicles, TERNZ Limited, Auckland, 2013.

[18] AASHTO A Policy on Geometric Design of Highways and Streets, American Association of State Highway and Transportation Officials, Washington D. C., 2011.

[19] Kaliabetsos, G., Lemonakis, P., Gkoutzini, A., Eliou, N.: Parametric Investigation of Vehicle Minimum Turning Path, 9th International Congress on Transportation Reaserch, Athens, Greece, 24.-25.10.2019.

[20] Savoy Computing Services Limited, User Manual for AutoTrack - Advanced Vehicle Swept Path Analysis, England, 2008.

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DESIGN OF MEDIAN ENDS AT AT-GRADE INTERSECTION LAYOUT PLANS

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Abstract

Median represents a portion of an urban road separating opposing directions of the traveled way. Due to their function to provide enough space for separate traffic lane and storage of left-turning vehicles, the medians are highly desirable on city arteries carrying two or more lanes per travel direction. Type and dimensions of turning vehicles significantly affect the design of a median openings and median ends. The design solution and the geometry of median ends should be based on the composition of all left-turning movements occurring simultaneously at at-grade intersection. After selection of design vehicles for swept path analysis of the characteristic turning maneuvers in intersection, larger vehicles should be further checked for their ability to turn without undue encroachment on adjacent traffic lanes. The critical design parameter is the median width, especially at the ends of the median openings. Dragging paths of left-turning vehicles shape the median ends geometry. In traditional design practice established in Europe, three-centered compound curves with radii rations R1 : R2 : R3 = 2 : 1 : 3 and R1 : R2 : R3 = 2.5 : 1 : 5.5 are used to approximate dragging pahs of turning vehicles. In this paper, considering the median width, the deficiencies of the traditional design approach are revealed and new findings regarding the design of the median ends at at-grade intersections are presented.

Keywords: At-grade intersection, layout plan, median end, three-centered compound curve

1 Introduction

Left-turn lanes design elements of channelized four-leg intersections with medians and raised islands used for the separation of intended vehicle paths are analyzed in this paper. Traditional design procedure for the four-leg at-grade intersection usually includes several stages in which different geometric elements of layout plan are designed first and then integrated into unique civil engineering design. The selection of adequate design vehicle and swept path analysis represent one of the key stages in the intersection design process, especially at the end of the process when adopted design elements should be checked and poorly shaped roadway and traffic islands edges finally corrected. Although existing Serbian design guidelines [1] comply with relevant German [2] and Swiss [3] design standards, intersection design procedures and methodology defined in these standards are based on the traditional design approach. In recent years, only small number of papers [4, 5] have been published in which new design procedures for four-leg channelized intersections were presented. In addition, even rarer was the research [6, 7] focused on the composition of geometric elements in three-centered compound curves used for the formation of the pavement edges geometry in right and left-turn channelization.

Traditional design methodology implies that geometric elements of intersection layout plan are separatelly designed and later integrated following the “INSIDE-OUT“ principle. In essence, this means that at the beggining vehicle movement trajectories for simultanious left-turning maneuvers in the center of intersection conflict area are defined (Figure 1), based on which roadway edges of left-turn auxulury lanes and the locations of median ends are determined. In relation to the previously positioned median ends, crosswalks at the major and minor street are than set. Crosswalks endings in the direction of original roadway edges dictate the position of triangular islands for right-turns channelization. At the end of the procedure, swept path analysis for right-turning vehicles are performed and the final geometry of outer roadway edges in relation to the modified triangular channelizing islands is set.

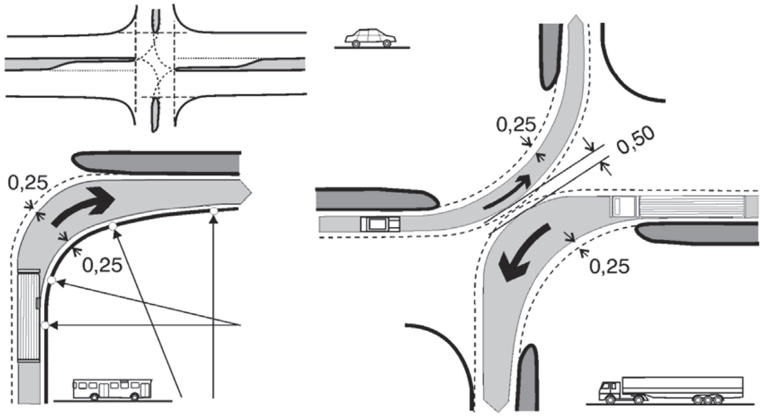

Figure 1 Minimal protective lateral widths along dragging paths of turning vehicles performing right and simultaneous left-turn maneuvers [8]

The above described procedure imposes as a major issue: How the shaping of median ends impacts on the size of intersection conflict zone and consequently on the scale of the whole intersection layout plan? Naimly, this issue was the key motivation for the research presented in this paper.

2 Definition of the problem

After setting of vehicle movement trajectories for the left-turning vehicles in intersection conflict area, the precise position of median ends (noses) could be defined. The essential dimension which impacts on the placement of crosswalks and further on the geometry of outer roadway edges is the distance between median end and the center of intersection (L_{ostrva}) where the axes of the crossing streets intersect (Figure 2).

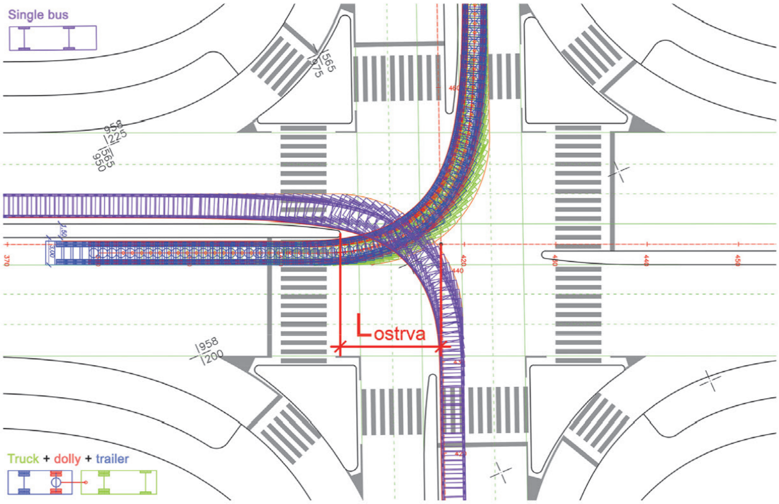

Figure 2 Determining the position of the median end in relation to the left-turning vehicles dragging paths

In traditional design approach, dragging paths of left-turning vehicles are usually aproximated by three-centered compound curves with well known radii rations R1 : R2 : R3 = 2.5 : 1 : 5.5 and R1 : R2 : R3 = 2 : 1 : 3. Starting and ending segments of this curves (circular arcs of radius R1 or R3) physically represent edges of raised curbs at median ends (Figure 3). Adopted geometric elements of three-centered compound curves for shaping of median ends imply that minimal protective lateral widths along dragging paths of left-turning vehicles are secured.

When placing crosswalks in relation to median ends, pedestrian safety should be carefully considered, especially for intersections located in urban areas. Bearing this in mind and according to Serbian guidelines for urban intersection design [8], minimum width of 1.5 m between opposite edges along adjacent crosswalk marks at median ends should be provided. This width guaranties enough space for temporary stopping of disabled persons in wheelchair and women with baby strollers when crossing the street.

However, when intersecting angles deviate from 90°, especially from the recommended range 60° - 120°, shaping of median ends and correct placement of crosswalk marks becomes even more complex (Figure 4). By further movement of crosswalks away from the median ends, triangular islands for right-turning vehicles channeling get bigger and bigger, and the whole area covered by the intersection layout plan increases too (Figure 5). So, as the key question arises: What is the optimum ration for the circular arcs and their corresponding central angles in three-centered compound curves which can provide precise shaping of median nose and consequently rational dimensions of triangular islands and intersection conflict area? It is already known from the literature [9, 10] that as the intersection conflict area gets larger in the signalized at-grade intersections, the duration of the red light signal for all vehicles consequently extends, thus the intersection capacity decreases. Hence, one of the main goals of road designers is to reduce as much as possible the intersection conflict area.

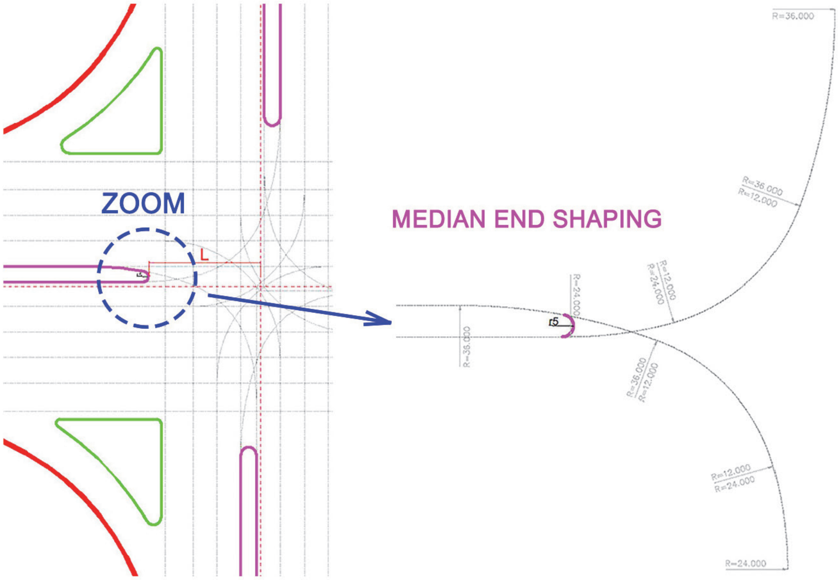

Figure 3 Shaping the median nose by three-centered compound curves used for the aproximation of left-turning vehicles dragging paths

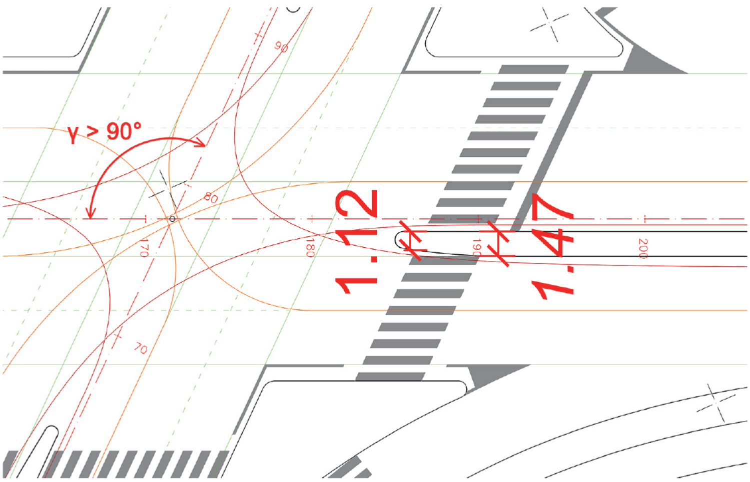

Figure 4 Median nose shaping when the intersection angle deviates from 90°

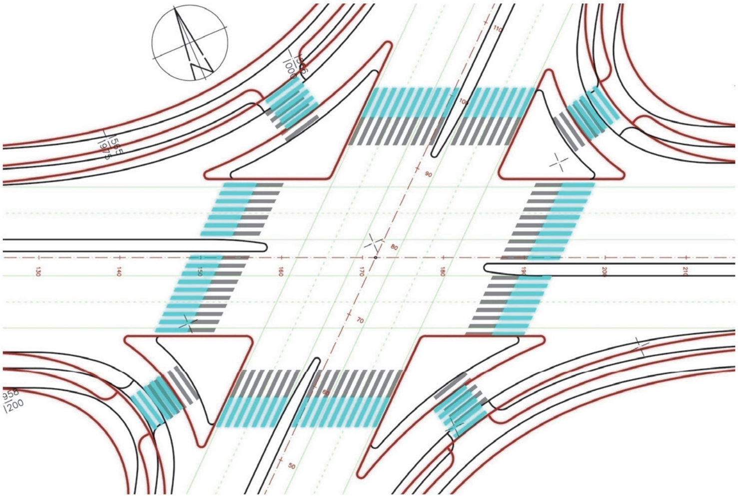

Figure 5 Displacement of crosswalk marks due to insufficient width of median ends (< 1.5 m) and consequently increase of the intersection area

3 Application of traditional geometric forms of three-centered compound curves for vehicle dragging paths approximation

Special “simulation” experiment in AutoCAD was performed in order to estimate the “geometric” eligibility of three-centered compound curve to approximate dragging paths of selected design vehicles. Tractor-semitrailer truck and single bus were selected as design vehicles for vehicle turning simulation run under GCM++ software [11] in AutoCAD environment. Steering path alignment was configured as a set of simple road curves with turning angles ranging from 60° to 120°. When setting the steering path alignment, minimum turning radius of selected design vehicles were drawn first. Then these minimum radii were offset towards their corresponding curve centers for the half of design vehicle width including mirrors and used later, together with the accompanying entrance and exit tangents, for the steering path alignment shaping. Both test vehicles follow previously set steering path alignment by the vehicles’ front bumper midpoint. As an example, one of the simulated turning maneuvers for single bus following steering path shaped as 90° circular curve is presented in Figure 6. In the same figure, the position of three-centered compound curve, drawn with the recommended value of central radius R2, in relation to the single bus dragging path is shown.

Three centered compound curves, used for the approximation of vehicle dragging paths, were constructed with the conventional radii ratios (R1 : R2 : R3 = 2 : 1 : 3 and R1 : R2 : R3 = 2.5 : 1 : 5.5), using diagram displayed in Figure 7 for determing the value of central radius R2 as a function of vehicle minimum turning radius R_s and intersection angle γ. This diagram, described in Serbian guidelines, is based on the similar diagram originally presented by Krenz and Osterloh [12].

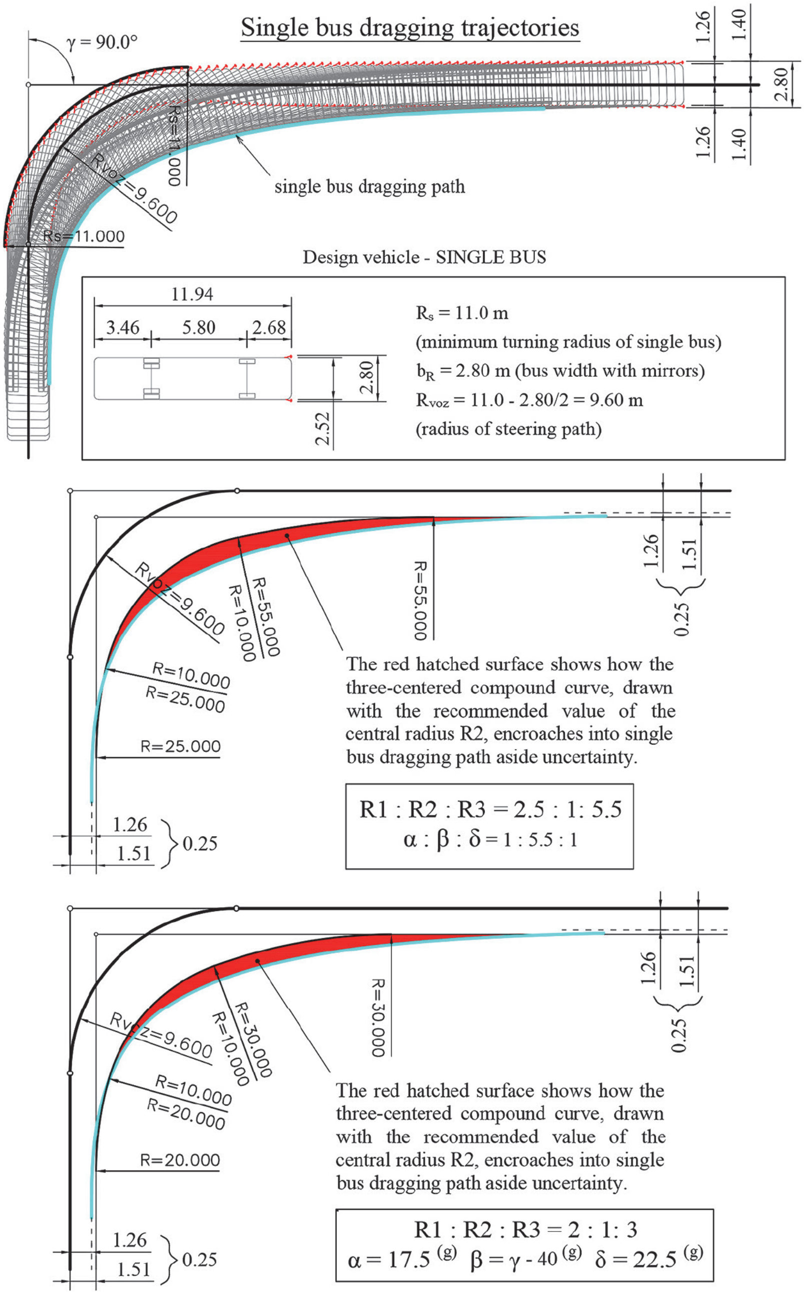

Figure 6 The position of three-centered compound curve drawn with the recommended value of radius R2 in relation to vehicle dragging path

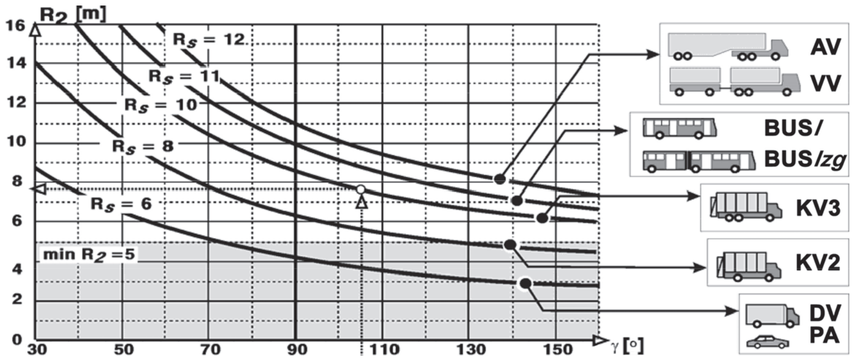

Figure 7 Central radius R2 of 3R compound curve expressed as a function of design vehicle minimum turning radius R_s and intersection angle γ [8]

Tangents for the construction of three-centered compound curves were drawn parallel to the original tangents of circular steering paths at the distance of the half of the vehicle width increased for the protective lateral width of 0.25 m. The Figure 6 clearly shows for both conventional radii ratios (R1 : R2 : R3 = 2 : 1 : 3 and R1 : R2 : R3 = 2.5 : 1 : 5.5) that three-centred compound curves, drawn with the central radius R2 adopted from the diagram in Figure 7, encroache into single bus dragging paths aside uncertainity. For the second longer test vehicle tractor-semitrailer truck, this encroachment is even wider. In practical terms, this phenomenon will be manifested as shortage of auxiliary traffic lane surface for left-turning vehicles. Due to this traffic lane surface shortage, the wheels of longer and wider vehicles will run into raised median ends and curbs along median nose edges will be phisically damaged.

4 Conclusions and future research

The shaping of median ends represents crucial stage of four leg at-grade intersections layout plan design. Critical dimension for placing of crosswalk marks and consequently repositioning of triangular channeling islands is the width of median nose. Therefore, besides providing sufficient clearance for unobstructed left-turning maneuvers, optimal geometrical formation of median nose edges is essential in order to get compact intersection conflict area with rationally programmed traffic signal timing.
Results of the simulation study obtained in this paper clearly indicate that widely used geometrical form of three-centered compound curve with the conventional radii ratios R1 : R2 : R3 = 2 : 1 : 3 and R1 : R2 : R3 = 2.5 : 1 : 5.5 could not be applied for the precise approximation of design vehicles dragging paths around median ends. Also, existing diagram in national intersection design guidelines, used for the selection of central radii R2 values in three-centered compound curves should be revised and corrected for various types of design vehicles. In addition, new ratios of circular arcs R1 : R2 : R3 and their corresponding central angles α : β : δ should be established and verified through comprehensive simulation tests for different turning maneuvers.
Revised geometrical form of three-centered compound curves will provide optimal shaping of median ends, decrease of the intersection conflict area, as well as rationalization of expropriated surface occupied by the whole intersection area. The final goal is to get reliable diagrams for the selection of three-centered compound curve design elements in order to eliminate iterative process of swept path controls and consequent road edges geometry adjustments after simulation of vehicle turning maneuvers in the final stage of intersection layout design.

However, besides optimal composition of geometric elements in three-centered compound curves, other specific curve forms (two-centered compound curves, curves shaped as fillet designs at airport runways and taxiways, etc.) potentially applicable as retraction curves, should be tested too. Bearing in mind a significant shortage of free space for the construction of new transportation infrastructure in urban areas, design of compact intersections and efficient space utilization are absolute imperative for all urban road planners and designers.

References

[1] JP Putevi Srbije, 2011. Pravilnik o uslovima koje sa aspekta bezbednosti saobraćaja moraju da ispunjavanju putni objekti i drugi elementi javnog puta, Javno preduzeće Putevi Srbije, Beograd, https://www.putevi-srbije.rs/index.php/регулатива, 22.05.2020.

[2] Richtlinien fur die Anlage von Landstrasen (RAL), Ausgabe 2012. Forschungsgesellschaft fur Strasen- und Verkehrswesen (FGSV Verlag), Koln 2012.

[3] Vereinigung Schwiezerischer Strassenfachleute (VSS), Schweizer Norm (SN 640262), Knoten - Knoten in einer Ebene (ohne Kreisverkehr), Zurich, 1999.

[4] Kirk, A., Jones, C., Stamatiadis, N.: Improving Intersection Design Practices. Transportation Research Record, No. 2223, pp. 1-8, 2011, https://doi:10.3141/2223-01

[5] Stančerić, I., Korlaet, Ž., Dragčević, V.: New design procedure for four-leg channelized intersections. Građevinar, 69 (2017) 4, pp. 257-266, doi.org/10.14256/JCE.1927.2016

[6] Dragčević, V., Korlaet, Ž., Rukavina, T., Lakušić, S.: Three-leg Intersection at-Grade - The Right Edge Forming Test. 3rd International Symposium on Highway Geometric Design June 29 - July 1 2005. Transportation Research Board, Chicago, Illinois.

[7] Ilić, V.: Analitička metoda za kontrolu prohodnosti i proračun elemenata situacionog plana površinskih raskrsnica. PhD thesis. University of Belgrade, Faculty of Civil Engineering, Belgrade, 2019.

[8] Maletin, M. et al.: Tehnička uputstva za projektovanje površinskih raskrsnica (PGS-PR/07), Građevinski centar, Beograd, 2010.

[9] Maletin, M.: Planiranje i projektovanje saobraćajnica u gradovima, III izdanje, Orion-Art, Beograd, 2019.

[10] Teodorović, D., Janić, M.: Transportation Engineering - Theory, Practice and Modeling, First Edition, Elsevier, Oxford, United Kingdom, 2017.

[11] Gavran, D.: Gavran Civil Modeller - GCM++. [Computer software for transportation infrastructure modeling]. Belgrade, Serbia, 2013.

[12] Krenz, A., Osterloch, H.: Die Bordsteinführung an Straßenkreuzungen und Knotenpunkten - Taschenbuch für Entwurf und Absteckung. Bauverlag GmbH, Wiesbaden und Berlin, Deutschland, 1971.

RISK RANKING ON EXISTING TWO-LANE RURAL ROADS WITH RESPECT TO ALIGNMENT AND AT GRADE INTERSECTIONS

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Abstract

The objective of this paper is the elaboration of a suitable methodology capable of deploying elements that characterize the existing infrastructure on two-lane rural roads, regarding road geometric elements (horizontal and vertical alignment, superelevation, sight distance), as well as location and geometric elements of intersections with the scope to evaluate the built-in road safety. The overall target is the utilization of these elements to lead to evaluation coefficients that point out the risk level of the examined road sections, as well as the critical areas, in terms of safety levels, curved sections and intersections. In this context, a hierarchy of critical parameters affecting the risk rating of a road network with respect to geometric design and at-grade intersections was primarily carried out. Appropriate mathematical equations were created for each parameter, based on current road design guidelines and the corresponding Crash Reduction Factors (CRF) presented in the literature. Initially, case studies were conducted, out of which special diagrams were produced to assess the risk level of each parameter. Subsequently, a basic statistical analysis of the results was carried out , using regression factors aiming at developing an appropriate mathematical model for the estimation of the risk level. The same analysis was performed for each parameter separately and appropriate weight coefficients were sought in order to obtain a combined rating that characterizes each road section, while at the same time identifies those critical intersections that present high probability of accident occurrence. Finally, data were collected from a significant sample of the existing two-lane road network of Greece, about 1000km of road network with more than 4000 intersections (unpaved road intersections were also included), in order to assess the results of the proposed methodology.

Keywords: two-lane rural roads, horizontal and vertical alignment, superelevation, sight distance, road safety

1 Introduction

This research was inspired by the fact that around 51 % of the fatalities in Greece [1], 55 % in Europe [1] and 51 % in USA [2], relate to incidents occurring in rural two-way highways. A characteristic of the majority of accidents in these road sections are the inconsistent, for the road condition, traffic velocities, which in combination with the inconsistency of geometric design often lead to violation of driver's expectation and increase the likelihood for potential accident occurrence. Moreover, many existing intersections studied and constructed many years ago are characterized by a poor geometric design, while the lack of adequate roadway

signing and the inadequacy of required sight distances, further increase the occurrence of incidents in these locations. The existing intersections that are not complying with the updated regulations need to be demonstrated through a quick evaluation in order to upgrade them.
In this framework the objective of the present research is to develop a methodology that exploits a great amount of data regarding the existing infrastructure on intersections collected in a very short time, while appropriate mathematical calculations are arising and rating intersections with an increased chance of road accident occurrence, aiming at proper care and prevention.

2 Literature review

A fundamental parameter influencing the safety of a road section is the operating speed of moving vehicles. It is noted that the V_{85} operating speed has been used in the literature in order the road construction to be evaluated on the basis of its deviations from the design speed V_e and also from the deviation shown by the V_{85} operating speeds between two consecutive and independent geometric elements of the road (two successive curves, a curve and an independent tangent, etc.) [3-8]. The limits established internationally are defined as follows:

- Correlation between operating speed V_{85} and design speed V_e

<u>Case 1:</u>	Good quality design	$\lvert V_{85} - V_e \rvert \leq 10$ km/h
<u>Case 2:</u>	Medium quality design	10 km/h $< \lvert V_{85} - V_e \rvert \leq 20$ km/h
<u>Case 3:</u>	Poor quality design	$\lvert V_{85} - V_e \rvert > 20$ km/h

- Correlation between the operating speeds V_{85} of two consecutive geometric elements on the road

<u>Case 1:</u>	Good quality design	$\lvert V_{85i} - V_{85i+1} \rvert \leq 10$ km/h
<u>Case 2:</u>	Medium quality design	10 km/h $< \lvert V_{85i} - V_{85i+1} \rvert \leq 20$ km/h
<u>Case 3:</u>	Poor quality design	$\lvert V_{85i} - V_{85i+1} \rvert > 20$ km/h

Based on the above methodology each individual curve of every road section is evaluated (by considering the long tangents as an independent element) and the spots with a deviation of more than 20 km/h are defined in order to propose improvement measures. The philosophy of the methodology lies in the fact that roads with better consistency lead to smaller operational speed differences occurring along each road section and therefore to an improved road safety level, based on crash rates collected from historic research.
The utilization of the above methodology is based on the determination of the operating speed V_{85}, either by actual on-field measurements, if the road is in operation, or otherwise by appropriate mathematical expressions that are being presented in the literature. The estimation of operating speed has been adopted by many researchers in the past and many mathematical expressions have been presented that estimate operating speed V_{85} [9-13]. All mathematical relationships include the horizontal curve radius R (or the curvature 1/R) and several additional parameters that have been utilized, such as CCR (Curvature Change Rate), road width, length of the circular arc, deflection angle, longitudinal gradient, etc. In many cases, the above methodology and the mathematical expressions have been integrated into the State regulations, and is used until today.

3 Contribution of operating speed V_{85}

In this framework the research has tried to combine all the evaluation parameters of intersections, with the magnitude of speed and at the same time enable the execution of the method for any speed, that the researcher desires to use (design speed, allowable speed, operating speed etc.).
This research adopts the use of the V_{85} operating speed as being more appropriate, considering that it incorporates most of the travelling vehicles (85 % of the vehicles) and that has often been adopted in the bibliography by a mathematical expression in relation to the CCR (Curvature Change Rate) of the road. Because the present research took place by utilizing data from Rural two-lane highways located in Greece, the mathematical expression of operating speed presented in the Greek Regulations [1] is used, incorporating the longitudinal gradient, as resulted from recent measurements. The mathematical expression used in this research is presented as follows:

$$V_{85} = \frac{1000000}{10150.10 + 8.529 \cdot K_E} + 5 \cdot (b - 3.75) + 25 \cdot \overline{s} \quad (1)$$

where:
V_{85} (km/h) - the operating speed;
K_E (grad/km) - the CCR of the considered road section;
b (m) - the width of the traffic lane;
$\overline{s}$ (-) - the mean gradient of the considered road section

It is noted that the operating speed resulting from this mathematical expression refers to Rural two-lane highways in free flow conditions. In intersection areas the operating speed is assumed to be lower [14], but at this time the relative research is rather poor, in order to define a new mathematical expression concerning the operating speed levels in the functional area of an intersection. By deploying the operating speed levels of the highway outside the functional area of the intersection it is anticipated that the analysis presented herein will be on the conservative side.

4 Critical parameters that are taken into account

In this new attempt it was not possible to exploit and evaluate all the parameters that may affect the provided road safety at intersections and road alignment. In this context, specific parameters have been selected, which are considered to be the most critical in this section. For each parameters an attempt was made to extract a mathematical expression of hazard level based on the bibliography as presented in Greece and worldwide. In the present study, in order to examine the hazard level of an intersection, the following parameters were evaluated:
1. Required stopping sight distance and intersection sight distance
2. Required time for safe passing
3. Adequacy of right-turn and left-turn lanes
4. Existence or absence of triangular or dividing islands
5. The vertical and horizontal signage both along the major and the minor road.
6. The appropriate road lighting of the intersection
7. The operating speed V_{85} of the major road, as calculated by expression (1).

It is noted that roundabouts have not been considered as they eliminate the concept of free and uninterrupted flow on the major road, a parameter which is a basic prerequisite in this methodology. Hereafter follows a brief description of mathematical expressions that characterize the hazard level as determined by Greek and international bibliography, using the basic expressions of science and traffic dynamics. Also, in order to examine the hazard level of a road section, the following parameters were evaluated:

1. Minimum radius of horizontal curve
2. Radius of consecutive horizontal curves
3. Longitudinal gradient of the road (influences only the operating speed V85)
4. Required superelevation in the horizontal curve
5. Required stopping sight distance
6. The operating speed V85 of the major road, as being estimated by expression (1)

5 Methodology application

The methodology for evaluating the geometric elements and intersections of a road network, as presented in the previous paragraphs, took place on approximately 1000 kilometers of existing Rural two-lane highways in Greece. To extract the geometry of each road axis, a topographic survey was made using appropriate instruments placed on the roof of a moving vehicle and synchronized to take a speck every 3-5 meters. The vehicle made a go and return on each road section to capture the right and left boundaries of each road. With proper processing of the X, Y, Z coordinates, the axis of the road was produced as the geometric mean of the two boundaries. Finally, through the generated X, Y, Z coordinates, the horizontal and longitudinal elements were extracted to be used in the calculation of the operating speed, as shown by the expression (1). In order to accelerate the procedure, the FM17 road design software (upgrade of H12 road design software used in the past [15]), has been used. It should be noted that for the majority of the road axles that were examined there were no data of existing superelevations available and for this reason the coefficients concerning the superelevations were ignored.

5.1 Geometry evaluation

After determining the horizontal and vertical alignment, as well as the superelevations in the curves (wherever possible), there were all the necessary elements to complete the evaluation of the geometry of each road.

5.1.1 Rankings of individual horizontal curves

- Rating from 0 to 50 corresponds to high level of road safety.
- Rating from 50 to 100 corresponds to medium level of road safety.
- Rating from 100 to 150 corresponds to low level of road safety.
- Rating above 150 corresponds to very low level of road safety.

Each road section, depending on the final rating, was evaluated over the level of road safety, according to limit values, which resulted from correlation with reported road accidents:

5.1.2 Rankings of road sections

- Rating from 0 to 150 corresponds to high level of road safety.
- Rating from 150 to 300 corresponds to medium level of road safety.
- Rating from 300 to 450 corresponds to low level of road safety.
- Rating above 450 corresponds to very low level of road safety.

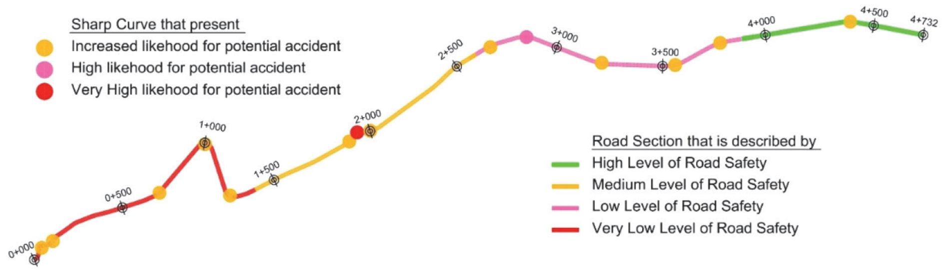

Figure 1 Graphic Colour Presentation of Road Sections Depending on the Level of Road Safety/Highlighting of Hazardous Horizontal Curves

5.2 Intersection evaluation

In order to evaluate each intersection, an attempt was made to collect all the elements described in the previous paragraphs. The angle of intersection, the geometric features of the left and right-turn lanes and the existence of the dividing island were extracted through the Google Earth maps, while the adequacy of lighting was evaluated through the Street View feature that provides the same software. The existing vertical signage and the assessment of traffic volumes were made by appropriate video recordings.
At the same time, the results of the evaluation of each intersection were summed up to determine the total coefficient of each road section, as divided into segments of 1-2 kilometers length. The purpose of this process is to evaluate furthermore the number of intersections in a road section. This marks the road sections that are highly rated due to the high frequency of intersections even if these intersections have a low score. In this context, all the hazard coefficients of individual intersections located within the same road section are summed up. The resulting total coefficient is divided by the length of each road section L and a weighed factor per kilometer is calculated and characterizes each road segment.

5.2.2 Methodology for an independent intersection

Many parameters can be estimated in a very short time by using the information taken by the Google Earth images, as shown in figure 1, like the intersection angle, the existence of a left or right turn lane, the width of the left turn lane, the existing lightning poles and the vertical signage (through Street View operation). Other parameters need to be taken from the site, like the slope of the minor road, the traffic volumes, while the geometry of the major road can be exported from topography measurements. Operational speed can be measured or calculated by the equation (1).

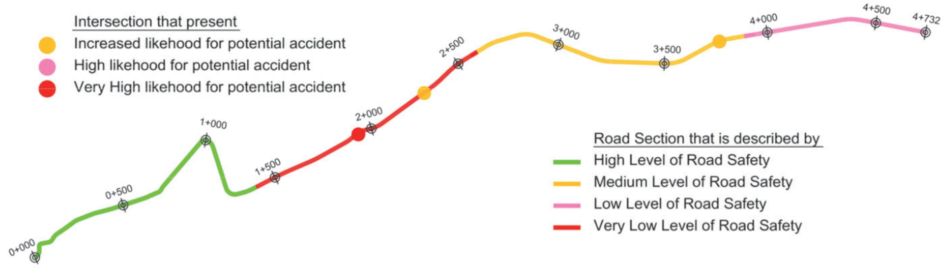

Figure 2 Graphic color presentation of road sections depending on the level of road safety/Highlighting of hazardous intersections

5.3 Total score

Finally, the sum of the above scores determined the final hazard level of the road sections, while the critical positions of the road network were determined from the previous stages. The color presentation based on the hazard level of both geometry and intersections is shown in Figure 3.

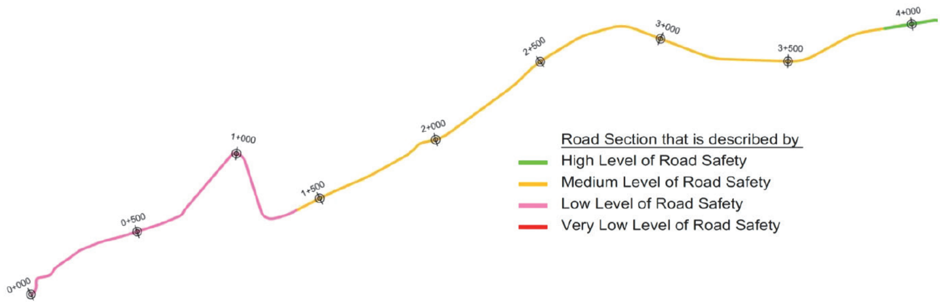

Figure 3 Graphic color presentation of road sections depending on the level of road safety, regarding both geometry and at-grade intersections.

6 Conclusions

The present study aimed to present a new approach of evaluating the road safety of a Rural two-lane highway, based on the geometric elements of the road, introducing the concept of the hazard score and ranking. In this context, an integrated methodology has been presented that evaluates and takes into consideration several critical parameters that affect the road safety in a road section in terms of geometry. For each of these parameters, appropriate mathematical expressions were identified using data from Greek and international literature, while simultaneously the results were evaluated based on the results of the IHSDM software and recorded accidents. The created mathematical expressions lead to the estimation of the hazard level, so to extract a total score for each individual horizontal curve and a total score for each road section. The present methodology and mathematical approach to the hazard level offers the following advantages:

Each parameter is calculated individually, but every curve and every road section is ranked according to a final score summed from all the parameters considered critical to the road safety. It is noted that it is possible to add any parameter considered critical and has not been utilized, as long as a mathematical expression that determines the hazard level of the parameter results from a research. With the distinct score for each design parameter, the parameters responsible for the high score of the black spots are highlighted by demonstrating immediately the improvement treatments that have to be implemented.

Individual curves of a road network are evaluated and ranked, while the entire road segment get evaluated by a weighted rating that characterizes each road section in terms of road safety.

It has built a background based on the concept of ranking due to hazard level, where additional parameters critical to the level of road safety, such as geometry and configuration of intersections, pavement conditions, the roadside clear zone widths, road insurance, road lighting, etc. can be included and evaluated. The hazard level of these parameters could be integrated into this methodology in order to provide a wider model of road evaluation that would highlight the black spots not only due to geometry.

Based on the proposed methodology approximately 1000 kilometres of rural road network were investigated. For faster data processing and simple and immediate export of the final scores, a new computer software by the research team was created. This new software enables the evaluation of a large number of road sections in a very short period of time and at the same time to correlate the results with recorded accidents and the results of other software like Interactive Highway Safety Design Model (IHSDM).

References

[1] National Technical University of Athens - Road Safety Observatory - Road Safety Data https://www.nrso.ntua.gr/data/

[2] Rural/Urban Comparison - Crash Stats - National Highway Traffic Safety Administration (NHTSA) https://crashstats.nhtsa.dot.gov/Api/Public/.../812301

[3] American Association of State Highway Transportation Officials (AASHTO), A Policy on Geometric Design of Highways, 2011.

[4] Lamm, R., Psarianos, B., Mailaender, T.: Highway Design and Traffic Safety Engineering Handbook, McGraw-Hill Companies, Inc, New York, 1999.

[5] Anderson, I.B., Bauer, K.M., Harwood, D., Wand Fitzpatrick, K.: Relationship to Safety of Geometric Design Consistency Measures for Rural Two-Lane Highways, Transportation Research Record 1658, Transportation Research Board, Washington DC, pp. 43-51, 1999.

[6] Anderson, I.B., Bauer, K.M., Harwood, D.W., Fitzpatrick, K.: Relationship to safety of geometric design consistency measures for rural two-lane highways, Journal of the Transportation Research Board, 1658 (1999), Transportation Research Board, National Research Council, Washington DC, pp. 43-51

[7] Cafiso, S.: Experimental Survey of Safety Condition on Road Stretches with Alignment Inconsistencies, 2 International Symposium on Highway Geometric Design, Mainz, Germany, 2000, pp. 377-387

[8] Hassan, Y., Sayed, T., Tabernero, V.: Establishing a Practical Approach for Design Consistency Evaluation, ASCE Journal of Transportation Engineering, 127 (2001) 4, pp. 295-302

[9] Hassan, Y.: Highway Design Consistency - Refining the State of Knowledge and Practice, 83 Transportation Research Board Annual Conference, Washington DC, January 2004.

[10] Taragin, A.: Driver Performance on Horizontal Curves, 33 Annual Meeting of the Highway Research Board, National Research Council, Washington, D.C., 1994.

[11] McFadden, J., Elefteriadou, L.: Formulation and Validation of Operating Speed-based Design Consistency Models by Bootstrapping, Transportation Research Record, Washington, D.C, 1997, doi: http://dx.doi.org/10.3141/1579-12

[12] Andueza, P.J.: Mathematical Models of Vehicular Speed on Mountain Roads, Transportation Research Record, Transportation Research Board, Washington, D.C, 2000, doi: http://dx.doi.org/10.3141/1701-13

[13] Cardoso, J.L.T., Kabbach-Junior, F.I., Suzuki, C.Y.: Analisis of Speed Prediction for Design Consistency of Two-lane Rural Highways, International Symposium on Highway Geometric Design, Valencia, Spain, 2010.

[14] TRB Operational Effects of Geometrics Committee: Modeling Operating Speed: Synthesis Report, Transportation Research Circular E-C151, Transportation Research Board of the National Academies, Washington, DC. 2011.

[15] Apostoleris, K., Vardaki, S., Mertzanis, F.: Identification of Safety Hazards on Existing Road Network Regarding Road - Geometric Design, Implementation in Greece, Proceedings RSS Conference, 2013.

RELIABILITY OF VEHICLE MOVEMENT SIMULATION RESULTS IN ROUNDABOUT DESIGN PROCEDURE BASED ON THE RULES OF DESIGN VEHICLE MOVEMENT GEOMETRY

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Abstract

Previous studies have shown that a valid roundabout design approach should include a determination of design elements based on the position of design vehicle's movement trajectories obtained by swept path analysis in early project stages, and not a conduction of swept path analysis at the end of design process. Several software which enable such significant progress in the design practice (optimal design of roundabout elements based on the results of vehicle movement simulation) are currently available on the market. Consequently, it is of great importance to know their accuracy. The reliability of vehicle movement simulation results is usually verified by field tests in which the distances between the test vehicle's movement trajectories are measured by means of a meter, which is a dilatory and time-consuming process. Within the scope of this study, a new approach for determination of the position of test vehicle's movement trajectories at the test site using a precise GNSS (Global Navigation Satellite System) device is described. The test vehicle was conducting a critical manoeuvre (left turn for 270°) for ten times, and the distances between its movement trajectories were determined by means of a meter and a precise GNSS device. The situation on the test site was then simulated on a computer and the assessment of the accuracy of chosen software for vehicle movement simulation was made.

Keywords: roundabout, swept path analysis, vehicle movement simulation, real drives, GNSS device, comparison

1 Introduction

According to existing roundabout design guidelines, roundabout planning and designing is an iterative procedure consisting of three main steps: (1) one of the available roundabout types is chosen depending on traffic conditions (e.g. single-lane roundabout, two-lane roundabout, turboroundabout); (2) the elements of the chosen roundabout type are designed in accordance with design rules (approaches, circulatory roadway, central island); (3) when all roundabout elements are designed, horizontal swept path and fastest path vehicle speed analyses are carried out [1-8]. If the analyses show that applied elements do not fulfil the swept path and/or speed requirements a redesign of roundabout elements is required. Main disadvantages of this procedure are reflected in the following: the design solution which fulfils the swept path and speed requirements is adopted and no further optimization of the design elements is made; detailed instructions on assigning input parameters for the swept path testing procedure are not provided, so the designer may come to the conclusion that the applied elements are successfully designed if the design vehicle in a simulated drive

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passes through the roundabout along the arbitrarily selected path in any way (drive with difficulties or with extra space for unhindered movement) [9]. In the light of the above, this design procedure can lead to oversized or undersized roundabout solutions i.e. low capacity, poor traffic safety, low driving comfort, and high construction costs [10].
Long term studies performed at the Department for Transportation of Faculty of Civil Engineering, University of Zagreb [10-15] have shown that a valid roundabout design approach should include a determination of design elements based on the position of design vehicle's movement trajectories obtained by swept path analysis in early project stages, and not a conduction of swept path analysis at the end of design process - such an approach ensures the usage of optimal roundabout element dimensions and an unhindered path for the design vehicle through the intersection. Thereby, the design vehicle's swept path becomes a key factor in the roundabout geometric design [16]. Several software which enable such significant progress in the design practice (optimal design of roundabout elements based on the results of vehicle movement simulation) are currently available on the market [17-19]. Consequently, it is of great importance to know their accuracy.
The reliability of vehicle movement simulation results is usually verified by field tests in which the distance between the test vehicle's movement trajectories is measured by means of a meter. This is a quite dilatory and time-consuming process which requires a large number of participants in field tests and leads to a rather demanding subsequent data analysis on computer [20]. In this study, a new approach for determination of the position of test vehicle's movement trajectories at the test site using a precise GNSS (Global Navigation Satellite System) device is proposed. The main assumption was that the use of this precise GNSS device would greatly speed up and facilitate the previously described process.

2 Methods

In traffic networks in suburban areas where different types of roundabouts are usually planned (single-lane roundabouts, two-lane roundabouts, turboroundabouts etc.) a significant number of heavy vehicles (truck-semitrailer combinations and trucks with trailers) and intercity buses (two- and three-axle buses) is present [9]. Therefore, these are the groups of vehicles from which the least favourable one regarding swept path width is chosen as a design vehicle when suburban roundabouts are designed [1-8]. Due to limited financial resources, field tests described in this paper were conducted using only one of the aforementioned vehicles. This vehicle was a two-axle truck IVECO STRALIS 460 EEV EURO 5 with a three-axle semitrailer KRONE SDP 27 ELB4-CS shown in Figure 1.
As stated in Introduction, the distance between the test vehicle's movement trajectories was observed in this research. These movement trajectories are a path of front overhang, which is defined by the front most prominent point of the test vehicle, and a path of right rear overhang, which is defined by the inner endpoint of the test vehicle (Figure 2) [21].

Figure 1 Test vehicle used in the research

Figure 2 Water tanks at front most prominent point and inner endpoint of the test vehicle

Before the test drives begun, a path consisting of a 40 m long entry straight, a 58.9 m long circular arc with radius of 12.5 m, and a 40 m long exit straight (which represented a critical manoeuvre i.e. left turn for 270°) was marked at the pavement using a reinforced adhesive tape and a marking spray (Figure 3). The test vehicle followed this marked path for ten times with its front most prominent point, and its movement trajectories were drawn by a water traces drained through the thin pipes from the water tanks installed at its front most prominent point and the inner endpoint (Figure 4).

The distances between the test vehicle's movement trajectories, i.e. test vehicle's swept path widths w_i, were determined by means of a meter and a precise GNSS device Trimble R8 [22] in 32 cross sections after each drive (Figure 4). Consequently, the 320 swept path widths wi were determined by means of previously described procedures (Figure 5). However, the use of a precise GNSS device proved to be significantly faster and less demanding method for determining so many swept path widths wi compared to a meter.

It is important to stress that these field tests were conducted in the spring period when weather conditions were favourable: the day was sunny, there was no wind, and the air temperature ranged from 15°C to 20°C. Such weather conditions were required in order to prevent the water traces from drying during the measurements of vehicle's swept path widths after each drive.

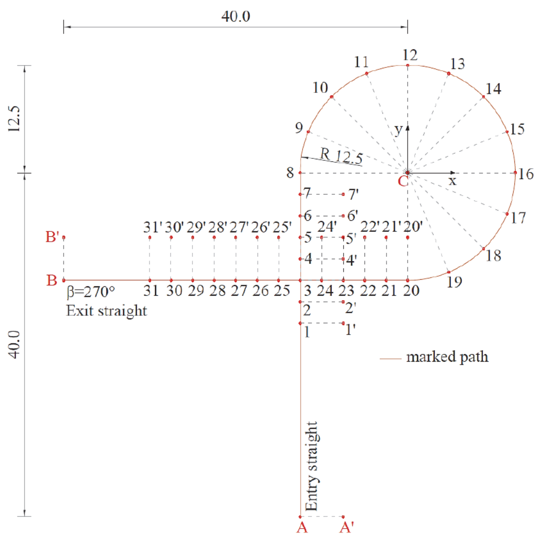
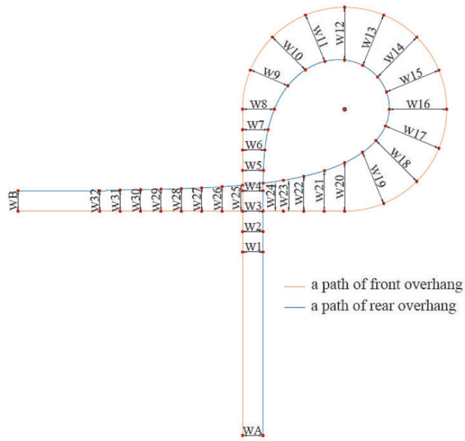


Figure 3 Elements of the path marked at the pavement and 32 cross sections in which the design vehicle's swept path widths wi were measured



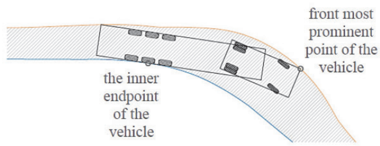


Figure 4 Movement trajectories and most prominent points of the test vehicle used in the research

Furthermore, it should be noted that this was not the first field test of this kind carried out by the research team from the University of Zagreb i.e. several trial tests were conducted in order to define the optimal measuring procedure which will result in high-accuracy data collection obtained by a dual-frequency GNSS device. GNSS observations were conducted using the high-precision real-time positioning service provided by the Croatian national network of continuously operating reference stations CROPOS (Croatian Positioning System). The obtained test vehicle's movement trajectories were referenced to the ETRS89 (European Terrestrial Reference System 1989) i.e. to the GRS80 ellipsoid (Geodetic Reference System 1980). The change of test vehicle's positions was captured by the single-epoch measurements at 32 cross-sections with the position precision expressed by the standard deviation of 0.56 cm. All measurements were checked for the outliers.

Figure 5 Test drive and measurements of test vehicle's swept path widths

3 Results of field tests

The analysis of data obtained by field tests was carried out using the Autodesk AutoCAD software. Firstly, the path that test vehicle followed, the 32 cross sections in which swept path widths were measured, and the test vehicle's movement trajectories were positioned using the geo-referenced points obtained by a precise GNSS device at the test site. All these elements (the path, the cross sections, and the test vehicle's movement trajectories) were then approximated with AutoCAD's cubic splines, i.e. interpolation polynomials of the third degree. Finally, the test vehicle's swept path widths w_i in 32 cross sections were measured and compared with those determined by means of a meter (Figure 6). Following conclusions were made based on this comparison:

• while conducting a critical maneuver, the test vehicle occupied the largest surface at the very end of the circular part of the path (cross section no. 18);
• the average difference between the swept path widths determined by means of a meter and the swept path widths determined by means of a precise GNSS device (in all cross sections and for all test drives) amounted +1 cm.

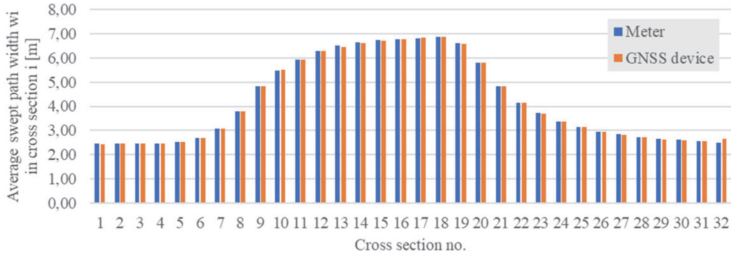


Figure 6 Results of field tests

Accordingly, measuring using a precise GNSS device is not only a fast, but also, a very precise method for determining the test vehicle's swept path widths at the test site. Moreover, the data obtained by this precise GNSS device is a rather simple to analyse.

4 Vehicle movement simulation

After the accuracy of results of field testes obtained by a precise GNSS device was confirmed, the situation on the test site was simulated in Autodesk's Vehicle Tracking software. Firstly, a virtual vehicle with dimensions equal to those of the test vehicle used in the field tests was created. The exact dimensions of the test vehicle were determined based on the data provided in catalogues of its manufacturers [23-24] and checked by hand measurements with a meter after the field tests finished (Figure 7).
This virtual vehicle followed the test vehicle's outer movement trajectories obtained by a precise GNSS device at the test site (outer cubic splines), and the swept path widths obtained by vehicle movement simulation and real drives were compared. Finally, the assessment of the accuracy of Vehicle Tracking software for vehicle movement simulation was made.

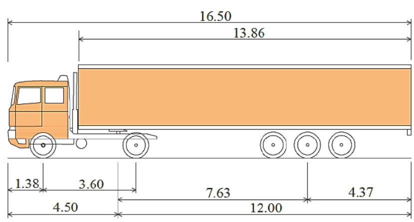


Figure 7 Dimensions of the test vehicle

4.1 A comparison of simulated and real swept paths

A comparison between the swept path widths determined by means of a vehicle movement simulation and the swept path widths determined by means of a precise GNSS device at the test site is given in Figure 8. Following conclusions have been made:
• largest differences between the swept path widths obtained by vehicle movement simulation and the swept path widths obtained by real drives occurred at the very beginning of vehicle's path (cross section no. 1) and amounted up to +12 cm;

- the average difference between swept path widths obtained by vehicle movement simulation and real drives (in all cross sections and for all test drives) amounted +7 cm;
- the simulation resulted in greater swept path widths in 95 % of cases, which is favourable in terms of roundabout planning.

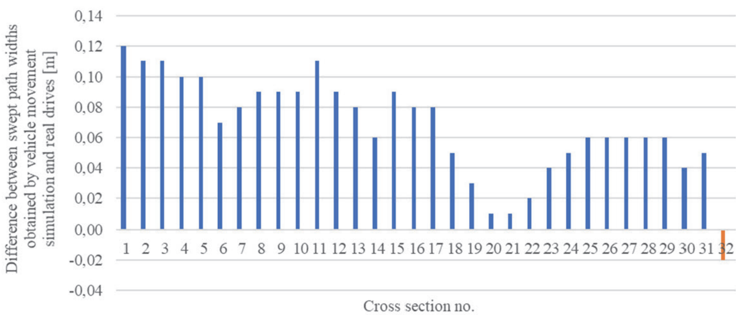


Figure 8 The differences between the swept path widths determined by means of a vehicle movement simulation and the swept path widths determined by means of a precise GNSS device at the test site

4.2 Assessment of the reliability of chosen software for vehicle movement simulation

The reliability of chosen software for vehicle movement simulation has been evaluated using the T-test for the significance of the difference between the means of two independent samples. The null hypothesis H_0 was as follows: the swept path widths determined by field measurements μ_0 are equal to swept path widths determined by vehicle movement simulation μ_1 (Equation 1). The alternative hypothesis H_1 was: the swept path widths determined by field measurements μ_0 differ from swept path widths determined by vehicle movement simulation μ_1 i.e. one method resulted in greater swept path widths than the other (Equation 2). The significance level was α = 0.05.

$$H_0 : \mu_0 = \mu_1 \tag{1}$$

$$H_0 : \mu_0 \neq \mu_1$$
$$H_0 : \mu_0 > \mu_1 \tag{2}$$

T-test for 95 % confidence interval has shown that the swept path widths determined by field measurements are equal to the swept path widths determined by vehicle movement simulation. As shown in Table 1, p-values were greater than the significance level α in the case of all test drives.

Table 1 T-test results

Result	**Test drive no.**									
	1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
p-value	0.435	0.431	0.435	0.434	0.439	0.426	0.450	0.447	0.463	0.422

Normality of average differences between the swept path widths determined by field measurements and swept path widths determined by vehicle movement simulation was tested using the Shapiro-Wilk's and QQ tests.
The null hypothesis H_0 in the Shapiro-Wilk test was that the distribution of these average differences is normal (Equation 3) and the alternative hypothesis H_1 that the distribution is not normal (Equation 4). The significance level was $\alpha = 0.05$.

$$H_0 : \mu_0 = \mu_1 \tag{3}$$

$$H_0 : \mu_0 \neq \mu_1 \tag{4}$$

Shapiro-Wilk test for 95 % confidence interval has shown that the distribution of average differences between the swept path widths determined by field measurements and vehicle movement simulation is normal (p-value amounted 0.370) (Figure 9).

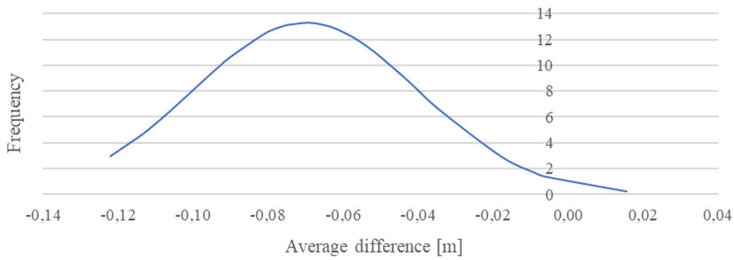


Figure 9 Normal distributions of average differences between swept path widths obtained by field measurements and vehicle movement simulations

Above results were confirmed by the results of QQ test (Figure 10). The linearity of the points suggests that the analysed data are normally distributed.

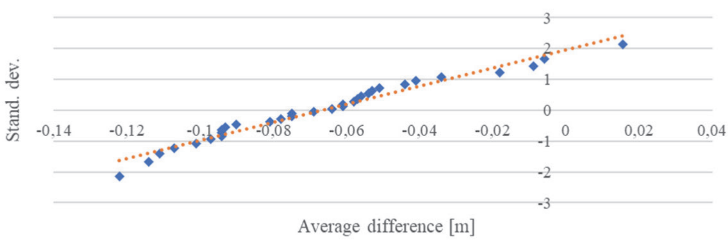


Figure 10 Results of QQ tests

5 Conclusions

A valid roundabout design approach should include a determination of design elements based on the position of the design vehicle's movement trajectories obtained by swept path analysis, and not a conduction of swept path analysis at the end of design process. Thereby, the design vehicle's swept path becomes a key factor in roundabout geometric design.
The reliability of software for vehicle movement simulation, which enables such significant progress in the design practice, is usually verified by field tests in which the distance between the test vehicle's movement trajectories is measured by means of a meter. However, the results of field tests carried out within the scope of this research have shown that the use of a precise GNSS device significantly simplifies and speeds up this process and therefore

leads to a lower total costs of field tests (lower rental costs of test polygons, lower rental costs of test vehicles, lower driver service costs). Moreover, this testing approach requires fewer people to participate in the field surveys and facilitates the subsequent analysis of test results on a computer.
A comparison of swept path widths determined by means of vehicle movement simulation in Autodesk's Vehicle Tracking software and real swept path widths determined by means of precise GNSS device at the test site has shown the following:

- the simulation results in greater swept path widths in 95 % of cases, which is favourable in terms of roundabout planning;
- from a statistical point of view these swept path widths do not differ.

In view of the above, the procedure described in the paper could be used, not only in the assessment of the reliability of one software for vehicle movement simulation, but also in the definition of the optimal software for vehicle movement simulation depending on the chosen design vehicle and intersection type.

References

[1] Smjernice za projektiranje kružnih raskrižja na državnim cestama (Guidelines for Roundabout Design on State Roads). Croatian Authority for Roads, Zagreb, 2014.

[2] Smjernice za projektiranje kružnih raskrižja sa spiralnim tokom kružnog kolnika na državnim cestama (Guidelines for Design of Roundabouts with Spiral Circulatory Roadway on State Roads). Croatian Authority for Roads, Zagreb, 2014.

[3] Eenheid in rotondes (Uniformity in Roundabouts). CROW Publication No. 126, 1998.

[4] Turborotondes (Turbo Roundabouts). CROW, Publication No. 257, 2008.

[5] Krožna križišča (Roundabouts). Ministry of Transport Republic of Slovenia, TSC 03.341, 2011.

[6] Krožna križišča s spiralnim potekom krožnega vozišča (Roundabouts with a Spiral Course of the Circulatory Roadway). Ministry of Transport Republic of Slovenia, TSC 03.XXX, draft version, 2011.

[7] Priručnik za projektovanje puteva u Republici Srbiji, Dio 5.3 Kružne raskrisnice (Manual for Road Design in Republic of Serbia, Part 5.3 Roundabouts). Serbian Authority for Roads, Beograd, 2012.

[8] Roundabouts: An Informational Guide. Second Edition, NCHRP Report 672, Transportation research Board, Washington, D.C., 2010.

[9] Džambas, T. Oblikovanje standardnih turbokružnih raskrižja (Design of standard turboroundabouts). PhD Thesis, Faculty of Civil Engineering of University of Zagreb, Zagreb, 2018.

[10] Džambas, T., Ahac, S., Dragčević, V.: Geometric design of turbo roundabouts. Technical Gazette, (1)2017. 24: 309 – 318. https://doi.org/10.17559/TV-20151012162141

[11] Džambas, T., Ahac, S., Dragčević, V.: Geometric design of turbo roundabouts according to Croatian and Dutch guidelines. Proceedings of 4th International Conference on Road and Rail Infrastructure (CETRA 2016), Zagreb, 2016, 893 – 900.

[12] Džambas, T., Ahac, S., Dragčević, V.: Design of turbo roundabouts based on the rules of vehicle movement geometry. Journal of Transportation Engineering, (7)2016, 142: 1– 10. https://doi.org/10.1061/(ASCE)TE.1943-5436.0000850

[13] Bezina, Š., Stančerić, I., Ahac, S.: Swept Path Analysis on Roundabouts for Three-axle Buses - Review of the Croatian Design Guidelines. Proceedings of 4th International Conference on Road and Rail Infrastructure (CETRA 2016), Zagreb, 2016, 901 – 908.

[14] Stančerić, I., Korlaet, Ž., Vesna, D.: New design procedure for four-leg channelized intersections. Građevinar, (4)2017, 68: 257 – 266. https://doi.org/10.14256/JCE.2029.2017

[15] Stančerić, I., Ahac, S., Bezina, Š., Vlaović, F.: Design limits for intersection angles between approach legs of suburban roundabouts. Građevinar, 71 (2019) 5, pp. 389 – 399. https://doi.org/10.14256/JCE.2029.2017

[16] Chan, S., Livingston, R.: Design Vehicle's Influence to the Geometric Design of Turbo Roundabouts. Teach America. http://teachamerica.com/RAB14/RAB14papers/RAB14ppr147_Chan.pdf. Accessed September 4, Accessed September 5, 2019.

[17] Vehicle Tracking features and functionality. Autodesk. https://www.autodesk.com/products/vehicle-tracking/overview. Accessed October 1, 2019.

[18] AUTOTURN. Transoft solutions. https://www.transoftsolutions.com/vehicle-swept-path/autoturn-select/autoturn/. Accessed October 1, 2019.

[19] TORUS roundabouts. Transoft solutions. https://www.transoftsolutions.com/road-design/torus-roundabouts/. Accessed October 1, 2019.

[20] Auswirkungen von neuen Fahrzeugkonzepten (Impact of new vehicle concept). BASt VI, German Federal Highway Research Institute, Bergisch Gladbach, 2008.

[21] Harwood, D.W., Torbic, D.J., Richard, K.R., Glauz, W.D., Elefteriadou, L.: NCHRP Report 505: Review of Truck Characteristics as Factors in Roadway Design. Transportation Research Bord, Washington, D.C., 2013. https://doi.org/10.17226/23379

[22] Trimble R8. Trimble. https://geospatial.trimble.com/products-and-solutions/trimble-r8s. Accessed October 4, 2019.

[23] IVECO Stralis. IVECO vehicles. https://www.iveco.com/sweden/Documents/Configurator/Brochure/Stralis_SE.pdf. Accessed May 22, 2018.

[24] KRONE. Krone Trailer. https://www.krone-trailer.com/english/download/data-sheets/. Accessed May 22, 2018.

SYSTEMATIC ARRANGEMENT OF INTERSECTIONS ON THE PRIMARY ROAD NETWORK IN BANJA LUKA

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Abstract

The City of Banja Luka experienced construction of eleven roundabout intersections of different type and functional level in the recent period. Previously adopted planning documents and conclusions of the City's Expert Team provided a stronghold for these activities, while positive experiences influenced the commitment to continue with similar activities in the future. In order to answer this question in a quality way, it is necessary to lay down strategic foundations for the systemic arrangement of intersections on the primary network of the City. The paper presents initial research regarding the definition of potential locations for the application of this type of intersections in the Banja Luka urban area as a basis for a strategic development document, i.e. an urban plan.

Keywords: urban road network, intersections, roundabout, strategy, location

1 Introduction

The role of urban traffic is to integrate city amenities, direct and synchronize activities and set the pace of urban life. In addition, urban roads limit the space for the development of physical structures so that traffic is an inevitable factor in the spatial organization of the city. Therefore, the city and its traffic present unique planning and design complex with the same temporal and spatial dimensions [1].

Urban transport network needs to justify its functionality by the rapid and reliable transportation of people and goods within the impact zone. For this, urban street network must be divided into primary and secondary networks that provide traffic connectivity and support urban content, respectively. Primary routes, due to their role of connecting remote contents and driving-dynamic requirements, should occupy a position that will lean to boundaries of consolidated urban entities. In order to ensure the functioning of the network and connect different levels of urban roads, intersections are formed at nodal points. Their distance and access control aim at maintaining certain level of service for particular urban road, as well as for the urban road network as a whole.

In the past period, eleven roundabouts of different types and functional levels have been built in Banja Luka. This number does not include three roundabouts built as temporary solutions from assembly/dismantling elements at slightly less busy, but still problematic intersections from the safety point of view. The justification for these activities was found in the previously adopted planning documents (regulation plans), and partly in the conclusions adopted by the City's Team for Defining and Designing New Roundabouts at Urban Roads.

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Thus, the action was guided by the goal to increase the degree of safety and to solve traffic flow problems at individual intersections, to the contrary of a conceived concept derived from serious traffic research in the City area. Still, given the positive effects of the implementation of such solutions, the City's commitment is to continue with similar activities in the future.
This paper, in the absence of an official and serious traffic study, and planning basis for organization and management of urban traffic, is an initial proposal for further analysis and consideration of strategic issues in the development of the Banja Luka urban road network and planning documents. The authors intend to present their thoughts regarding the arrangement of the Banja Luka urban road network, primarily in relation to recently performed activities that were not based on basic research on traffic flows, safety, accessibility and environmental impacts.

2 Banja Luka transport network

In the past two decades, Banja Luka has witnessed a significant expansion of individual motorized traffic, accompanied by a high degree of urbanization and negligible correction of the capacity and/or organization of traffic areas. At this point, it is no longer questionable whether the City's road network will collapse, but only when it will happen. Over the last few years, it has been noticeable that morning and afternoon peak loads are being extended to more and more streets, and that in some parts of the central City area peak loads have a practically continuous duration from early morning to early evening [2].

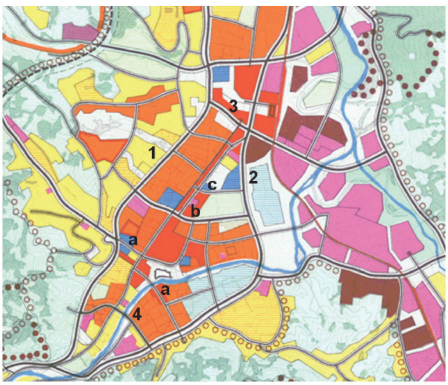

Figure 1 Primary urban roads within the 1975 Banja Luka Urban Plan [3]

Following the general principles of urban road network organization, the 1975 Banja Luka Urban Plan [3] (Figure 1) defined that West (1) and East (2) Transit roads are primary urban roads - City's magistral roads interconnected into a ring by streets Ivana Gorana Kovačića (3) in the North and Gavrila Principa (4) in the South. Beside the ring, transverse connections were planned at the perimeter of the inner City center, namely Bulevar Cara Dušana - Cara Lazara Street in the South (a) and Vuka Karadžića Street - Aleja Svetog Save in the North (b). The same consistency in development still exists, except that the role of Aleja Svetog Save is taken over by the new Olimpijskih pobjednika Street (c) with a part of Vidovdanska Street as a consequence of the new contents distribution in the area.

However, in the absence of the updated Urban Plan in these days, the only valid implementation planning documents that form the basis for spatial planning are regulation plans. Several documents of such type exist for the City area. Although the regulation plans need to define the conditions for equipping certain spatial units with traffic and municipal infrastructure, and accordingly define the function and rank of individual traffic (infrastructure) corridors, this is not the case in practice. All regulation plans are generally similar to one another, and the main focus is on the layout and design of structures (residential and administrative buildings), while at the same time missing to define the conditions for improvement and/or development of traffic infrastructure.

As a consequence, planners involved in the development of spatial planning documents for implementation (regulation plans) do not have a valid information base that clearly defines the overall concept and hierarchy of the City's transport network (urban plan and related traffic study). Even more, it is not clear when it will be available. Therefore, the primary task of the City is to define its strategic commitments in this regard and to make such commitments a firm planning basis.

By defining the concept of primary urban roads (in addition to improving the service and facilities for other modes of transport such as public transport, bicycle and pedestrian traffic), followed by creation of relevant planning documents, corridors, interconnections and connections will be clearly established, thus setting the scope of public areas required for the realization of the outlined concept (regulation line). In addition, such primary corridors will also define individual spatial blocks/scope for the development of regulation plans.

The following Figure 2 presents one vision/concept for the development and organization of the primary urban road network in the City of Banja Luka which was created based on previous experience and knowledge of past plans of higher order. The concept also defines the inner City center within which the movement of passenger cars would be restricted, with the priority given to public transport, pedestrian and bicycle traffic. The functioning of the concept is supported by the construction of capacity public parking garages on the perimeter of this zone (G1-G6), as well as by the proper positioning of roundabouts (R1-R24). The above mentioned, already built, roundabouts are: R1 and R4 at the West Transit, R7, R10 and R11 at the East Transit, and R13, R17, R18, R20, R23 and R24 as part of transversal primary links. The potential roundabouts at the most important locations will be analyzed in more detail below, individually.

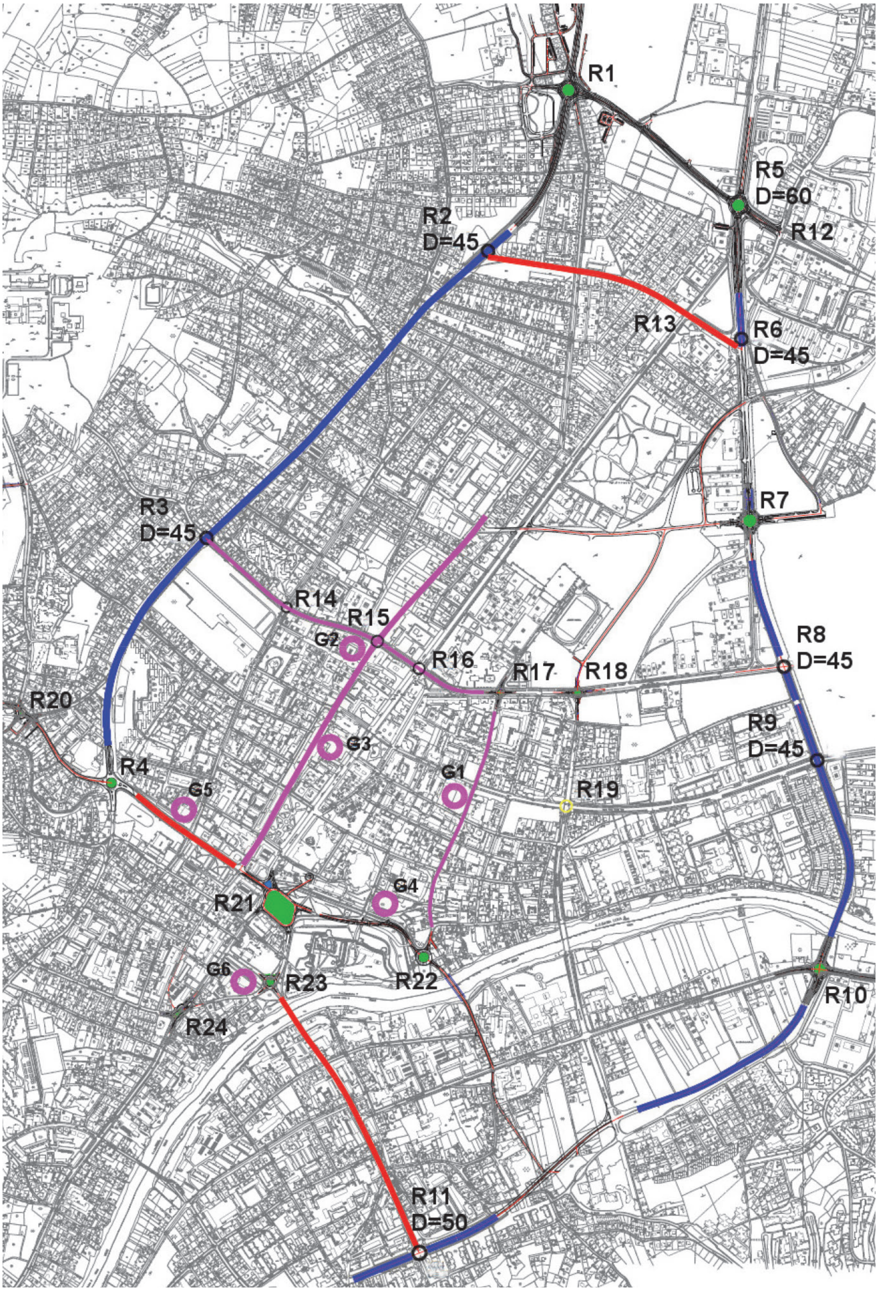

Figure 2 Concept of the primary urban road network and intersections [authors]

3 Roundabout locations for the primary network

The main urban magistral roads (MUMR) in the City of Banja Luka, as already mentioned, are the West and East Transit. The process of roundabouts construction on these MUMRs started in the previous period, showing individually good results, but also creating problems in some other sectors of these, as well as the surrounding streets. An important fact, which must be especially emphasized as an element of not so good practice, is that none of the solutions made was based on research of traffic loads, capacity of intersections and free sections, safety levels and environmental impacts. All designed and so far applied solutions are based on general conclusions about the level of traffic volume and possible ways of managing transit traffic.

In order to complete the solution and utterly resolve the congestion, the process along the West Transit must continue with the construction of two new roundabouts at the intersections with streets Trive Amelice (R2) and Vuka Karadžića/Ranka Šipke (R3) (Figure 3). The continuity of traffic flow would be achieved in this way, the causes of traffic jams at traffic lights would be eliminated and the level of service would be raised. In both cases, this is a 2:1:2:1 type of roundabout, however having only three branches at the intersection with Trive Amelice Street. The peculiarity of the intersection with the Vuka Karadžića/Ranka Šipke Street (R3) is that it is necessary to perform additional analyzes from the pedestrian movement point of view for the West/East direction, which in this case is intense, and to consider the need for the denivelation of pedestrian movements in the said direction.

Figure 3 Intersections at Trive Amelice (left) and Vuka Karadžića/Ranka Šipke streets (right) [authors]

One of the first locations and priorities in respect to solving problems at the East Transit is the intersection with Ivana Gorana Kovačića Street (R5 in Figure 2). The design for this intersection has been already prepared (Figure 4, left), so the implementation phase can start quickly after updating the existing documentation and resolving property-legal issues. In addition, jointly with these activities, it would be good to analyze the possibility of introducing another roundabout at the intersection of Braće Podgornika and Pilanska streets (R12 in Figure 2; Figure 4, right), given the fact that the stretch between the new roundabout (R5) and Braće Podgornika Street is rather short, and that, in addition to access for bus and train stations, and to business zone in Pilanska Street, the construction of a new business zone in the Livnica complex is planned in the near future.

What is important for this location is that in parallel with the creation of conditions for the realization of the R5 intersection, it is necessary to modernize Ivana Gorana Kovačića Street as a connection with the main intercity road M16 (West Transit) through two roundabouts. Its cross section requires redesign involving an additional lane for diversion into the local trade zone (Tropic) and pedestrian paths, which would significantly increase the level of service and safety on this traffic route.

Figure 4 Intersections of East Transit and Ivana Gorana Kovačića Street (left) and of Braće Podgornika and Pilanska streets (right) [authors]

The following location for the roundabout would be the intersection with Trive Amelice Street (R6) (Figure 5). In this case, one might consider that the junction of Frane Supila Street and the extension of Gundulićeva Street (x-x in Figure 5), due to existence of another roundabout at the intersection with the Olimpijskih pobjednika Street (R7), could be reconstructed on a right-right basis, which would significantly contribute to continuity of flow.

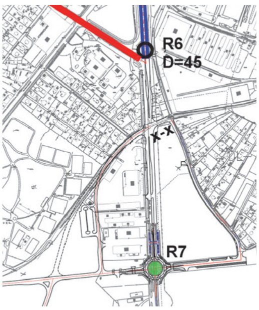

Figure 5 Intersection of Trive Amelice Street and East Transit [authors]

Further South, after the construction of the roundabout at Rebrovac (R10 in Figure 2), it became apparent that traffic lights at the intersections of East Transit with Aleja Svetog Save and Bulevar vojvode Živojina Mišića caused the peak hour crowds at the East Transit and these two streets (Figure 6). These are two typical examples of how independent solving of intersection can do more harm than good. Roundabouts should also be built at these two locations.
In this way, the entire East Transit route would be made passable with a high level of service, better access to the inner City center and better transit flows in the North-West - South-East direction. As this is a MUMR-ranked street with priority for motor traffic, there is a need for additional analysis of transverse pedestrian movements at both intersections. Underground pedestrian passage already exists at the University Campus (Aleja Svetog Save), but it is certainly necessary to inspect the condition of the underpass since it was never opened for use, and assess the possibility of reconstruction in order to obtain an eventually better crossing solution. An above ground pedestrian crossing would be one of the options at the intersection with Bulevar vojvode Živojina Mišića. Such structures can, in addition to their functionality, be very attractive in space.

According to the established approach, construction of roundabout was completed at the intersection of East Transit with Cara Lazara and Krfska streets (Figure 6). Since East Transit is MUMR and Cara Lazara Street is the magistral urban road, with two lanes in each direction, it was necessary to build the roundabout at the highest functional level for this location. However, end part of Krfska Street is a public transport terminal, and the major deficiency of the implemented solution is that the opportunity to establish a quality solution for locating and organizing the terminal was missed.

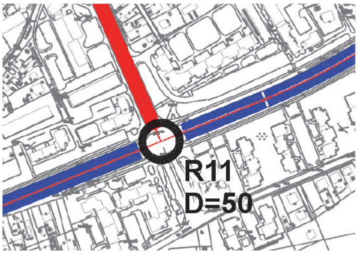

Figure 6 Intersection of East Transit with Cara Lazara and Krfska streets [authors]

In addition to systemic arrangement of roundabouts along West and East Transit roads, for good quality access to the inner City center, taking into account the consistency in solving intersections, it would be desirable to reconstruct the existing classic surface intersections along the Vuka Karadžića Street - Aleja Svetog Save stretch into roundabouts, at the locations of crossroads with Jovana Dučića, Vidovdanska and Kralja Petra I Karađorđevića streets (R14, R15 and R16 in Figure 2) in addition to the already existing roundabouts with Vase Pelagića and Gundulićeva streets (R17 and R18 in Figure 2). Considering that there is intensive pedestrian and bicycle traffic around the inner City center, and given the intensive construction of residential and administrative buildings in the area immediately adjacent to this zone, it is necessary to further consider and analyze pedestrian flows and options for cross-communication through street profiles.

Finaly, an extremely significant location in the City of Banja Luka is the intersection of Kralja Petra I Karađorđevića Street and Bulevar Cara Dušana (R21 in Figure 2). Initially adopted concept [4], fully in line with and supporting the planned concept of street network development, was to build a square with a park at this location (Figure 7). The “square” intersection, with virtually roundabout traffic, would thus become an important point of acquisition and distribution of flows for various purposes through which other parts of the City (South and East) would be connected to the central zone. At the same time, the square would also represent an initial step towards the calming, i.e. destimulation of motorized traffic at the entrance to the inner City center.

Figure 7 Square (R21) at the intersection of Kralja Petra I Karađorđevića Street and Bulevar Cara Dušana [4]

4 Conclusion

A quality answer to the question of designing the urban road network and solving the main flows is obtained only through a specific strategic development document in which a significant segment would be traffic. The urban plan is therefore a basic document since it contains a traffic study that defines the strategic choices for the further development of the street network. Bearing in mind that the adoption process of the Banja Luka Urban Plan has no definite end, it is considered necessary to adopt a certain strategy for the benefit of further activities, and in that sense to adopt a certain document that would clearly define the relationship between the City and the interurban road network, as well as complete the concept of primary urban roads from the aspect of hierarchy and corridors. The possible systemic arrangement of intersections on the primary network, presented in this paper, represents the authors' contribution to the considerations regarding the arrangement of the urban road network in the absence of planning documentation and relevant traffic analysis. In this way, several goals would be achieved, of which the most important is the identification of a basic traffic matrix, which defines urban blocks and entities of different contents and purposes. Additionally, a balance could be established between the use of space, movement, traffic needs, traffic base, accessibility and therefore the value of urban land.

References

[1] Maletin, M.: Planiranje i projektovanje saobraćajnica u gradovima, Orion art, Belgrade, 2019.

[2] Grujić, Ž., Jokanović, I., Đorđić, V., Topić, D.: Koncept drumske mreže u centralnoj zoni grada Banja Luka, Put i saobraćaj/ Journal of Road and Traffic Engineering, 65 (2019) 4, pp. 11-16, doi: 10.31075/PIS.65.04.02

[3] Banja Luka Urban Plan, Urban Planning Institute of Banja Luka, Banja Luka, 1975.

[4] Preliminary design of intersection between Kralja Petra I Karađorđevića Street and Bulevar Cara Dušana, Putevi i ulice Ltd., Banja Luka, 2010.

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DEVELOPMENT OF A CONTACTLESS SENSOR SYSTEM TO SUPPORT RAIL TRACK GEOMETRY ON-BOARD MONITORING

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Abstract

This paper is focused on the ongoing research, within a work package of the Shift2Rail project Assets4Rail, related to the development of an on-board contactless sensor system able to measure the wheel's transversal position in relation to the rail in order to support track geometry measurements. In particular, this research work focuses on developing a sensor system to support track geometry monitoring performed by the master system under development in other Shift2Rail projects. The aim is to develop a sensor system to detect the relative transversal position between the wheelset and the rail, suitable for the use on commercial (in-service) vehicles. In fact, a possible track geometry monitoring system alternative to the sophisticated and expensive optical/inertial systems and suitable for use on commercial vehicles, could be based on the measurement of accelerations. However, some parameters of the track geometry, such as lateral alignment, are extremely difficult to determine through the measurement of accelerations. In this case, it is necessary to find an innovative sensor system able to determine the wheel's transversal position in relation to the rail. For this reason, this project intends to focus on innovative systems that allow the detection of the wheel-track position by avoiding the optical/inertial systems already used on diagnostic trains. After a state-of-the-art overview on the potentially applicable technologies for the sensor system to be developed, a corresponding analytical tool for comparison of contactless sensors to choose the most suitable technology has been developed and two candidate technologies (stereo and thermal cameras) have been selected and assessed by means of a test platform in the facilities laboratory of VGTU (Vilnius Tech). This work will be the basis for developing a concept design of the sensor system together with a montage solution, which will be finally tested on a vehicle in real operation conditions.

Keywords: track geometry, monitoring, sensors, transversal displacement, commercial trains

1 Introduction

Assets4Rail project, founded within Shift2Rail Joint Undertaking [1], begun in December 2018 and will last 30 months if not longer. The project's main objective is to achieve cost-efficient and reliable infrastructure, developing a set of cutting-edge asset-specific measuring and monitoring devices. To achieve that, Assets4Rail follows a twofold approach, including infrastructure (tunnel, bridges, track geometry, and safety systems) and vehicles. The project is

structured into two Work Streams (WS): the WS1 related to monitoring and upgrading solutions addressed to bridges and tunnels, the WS2 regarding monitoring solutions for three railway assets: trains, track geometry and data collection from fail-safe systems [2].
This paper reveals some preliminary results achieved in the EU Shift2Rail project Assets4Rail, which aims to contribute to the modal shift towards rail by exploring, adapting and testing cutting-edge technologies for railway asset monitoring and maintenance, in order to ensure proactive and cost-effective maintenance.

2 Background

This paper focuses on track geometry monitoring, which is an essential activity to ensure the safety of railways operation, nowadays performed by Infrastructure Managers by diagnostic vehicles capable to analyse track conditions and detect potential problems at an early stage. The research in the field of these diagnostic activities is mainly aimed at the improvement of maintenance strategies, in combination with the reduction of infrastructure management costs.
The focus is on track geometry monitoring systems based on contactless optical/inertial technologies (Fig. 1). Optical sensors are used for the rail profile measurements and the rail location, while the inertial unit makes available the linear and angular accelerations. The combination of optical and inertial data allows the determination of the track geometry quality through the measurement of track geometric parameters, the main five of which are: gauge, cross level/cant, longitudinal level, alignment and twist. The definitions for the principal track geometry parameters, their measurement requirements and the analysis methods are given by European Standard EN 13848-1:2019 [3].

Figure 1 Track geometry monitoring system composed by an inertial unit and two optical sensors (source MERMEC)

Infrastructure Managers (IM) commonly deploy a dedicated Track Recording Vehicle (TRV) or hauled Track Recording Coach (TRC) running along with the network gathering track geometry data for inspection and general measurement purposes [4]. The measurement by TRV and TRC is a mature technology, standardised in the EN 13848-2:2006 [5], which covers several aspects concerning the characterisation of track geometry and measurement devices and methods. Track geometry parameters can be measured by using either an inertial or a versine system, which is also the measurement principle of TRV and TRC.
Monitoring of track geometry with in-service trains is more and more of interest for the IMs and RUs. In recent years, IMs attempted to deploy Unattended Geometry Measurement Systems (UGMS) on in-service vehicles without interrupting the regular traffic [4].
To avoid the sophisticated and expensive optical/inertial systems, acceleration measurement and advanced data processing techniques are the most popular option due to the robustness of accelerometers [6]. In comparison, optical sensors, such as laser based, camera based, etc., must be cleaned regularly to keep them working and thus need special treatment to avoid getting dirt when applied on commercial vehicles. Robustness is an essential factor for the applications on in-service vehicles, as the monitoring system should not require additional maintenance, affecting the reliability and the availability of the vehicles themselves.

Longitudinal level and vertical rail profile can be obtained by double integration of vertical accelerations. In terms of feasibility on commercial vehicles and accuracy, the best measurement is probably by the acceleration on the bogie over the axle-box and the vertical displacement from the accelerometer to the axle-box, despite displacement transducers are more vulnerable and expensive than accelerometers [4]. To avoid involving displacement transducers, axle-box mounted accelerometers, in conjunction with dedicated signal processing techniques, can also be used. Monitoring of the track's longitudinal levelling is a straightforward method thanks to the strong relationship between the vehicle reaction (e.g. vertical acceleration of axle boxes) and the vertical track defects. Therefore, monitoring track geometry with accelerometers is mainly done for this track geometry parameter only, like in the ICE2 for several years [4].
Other parameters of the track geometry, such as twist or lateral alignment measured by inspection cars and are also essential for maintenance and safety issues, are much more complicated to be monitored with commercial trains using accelerometers. Besides this, an accurate position of the measurements is important to verify the reproducibility, provide sufficient information for trend analysis, predict the degeneration, and identify the root causes.

3 Measurement of the transversal wheel/rail relative position

3.1 Drivers, benchmarks and emerging solutions

The initial results of Assets4Rail research in the field of track geometry monitoring allowed to identify some candidate technologies for the development of the on-board sensor system able to measure the transversal position of the wheel in relation to the rail. They have been identified in the fields of:

- direct measurement systems, such as lasers, high speed cameras, stereo cameras and thermographic cameras;
- indirect measurements, such as accelerations and ultrasonic reflection.

In this large set of systems, for the specific target of wheel-rail relative position measurements, the most promising technologies to be taken into consideration for next developments seem to be stereo cameras and thermographic cameras. In particular, thermographic cameras have already been tested with good results in two experiments [7], [8], even if for a restricted speed range and without facing most atmospheric conditions that may occur during normal operation of a commercial train.

3.2 Specifications of requirements

The next step for developing the envisaged robust and cheap sensor system to determine the wheel-rail relative position was the definition of a System Breakdown Structure (SBS) by considering the candidate technologies identified in state-of-the-art recognition described above. In general, the system for wheel-rail relative position detection is articulated into the following modules:

- Hardware technology (the image monitoring system should be composed of a camera, protection case, illumination device, on bogie montage solution, image processing unit and data storage);
- Communication interfaces (data acquired by the sensor system should be synchronised with the whole monitoring system);
- Software post-processing (algorithms should determine the transversal position of wheelset in relation to the rail by determining the wheel-rail contact points and the angle of attack);

- Energy supply system (should be constituted by dedicated batteries powered both by sustainable and permanent technologies - e.g. harvesting systems using the vibration of bogies - and/or low voltage power supply system of the wagon).

For the whole system and each module, requirements have been identified and classified into Functional, Operational, Performance and Safety.
In general, the system for determining wheel-rail contact position should determine the transversal position of the wheel in relation to the rail continuously, must be integrated and synchronised with the primary on-board system for detecting the track geometry to be developed by Shift2Rail and should be able to be installed on different typologies of bogies. Moreover, the system should be able to work on in-service trains at 60÷200 km/h under a large set of weather and environmental conditions (dust, rain, snow, etc.). It must withstand the stresses due to vibrations in the entire operating speed range. Finally, the system should be easily maintained and should be powered by the low-voltage power supply system of the wagons or by dedicated batteries powered both by sustainable and permanent technologies (e.g. harvesting systems using the vibration of bogies).

4 Identification of the most suitable technology for sensors

4.1 Specific measurement issues due to the wheelset movement on the track

For developing devices to measure the transversal displacement of the wheelset, it is necessary to evaluate the features of the wheelset movement on the track. In fact, due to the wheelset oscillation phenomenon, the wheelset not only moves linearly along track longitudinal axis X, but also rotates about axis X and vertical axis Z as well even on tangent track, resulting in a sinusoidal motion, known as Klingel motion. The wheelset movement trajectory becomes even more complex and difficult to describe when the rolling stock is running on track curves and/or along track superelevation (inclined track).
Hence, the wheelset's multifaceted movement on the track, the distance from the sensor to the point to be measured on the wheelset (circular line of the rolling profile) is constantly changing both vertically and horizontally. This can lead to measurement inaccuracies, hard-to-reach repeatability and reproducibility, which needs to be considered.

4.2 Developing analytical tools for comparison of sensors to choose the most suitable technology

The most suitable sensor technology for transversal position estimation of the wheel to rail was defined through multi-criteria decision making (MCDM) analysis, considering the outcomes of track geometry measurement state-of-the-art study provided in previous sections.
Proposed MCDM method allows evaluating indicators that cannot be compared using classical mathematical-statistical methods. This significantly increases the practical value and novelty of the work.
Questionnaires were provided for the partners of Assets4rail project, and they made proposals about factors that are important for wheel-rail transversal position monitoring technologies. Eight categories of criteria were formulated.
It was found that the most important factors of the technology used for measuring the transversal wheel position are: 1. Dimensions and Weight; 2. Energy Consumption; 3. Life Cycle Cost (LCC); 4. System Robustness; 5. Measurement Accuracy includes sampling rate, system dependency on train speed; 6. Technical Compatibility (Interoperability); 7. Output Data; 8. Measurement Repeatability includes the reliability of results.

To decide the most important criteria, a questionnaire was established, the rankings were made by twelve experts and driver weights were defined. These procedures allowed the weights to be identified in two distinct ways: using AHP (Analytical Hierarchical Process) method and using direct evaluation. In conclusion, it was found that the most four important criteria are:
• System Robustness;
• Measurement Accuracy;
• Life Cycle Cost;
• Technical Compatibility.

4.3 Evaluation of candidate technologies

The results obtained in this study were evaluated using TOPSIS (The Technique for Order of Preference by Similarity to Ideal Solution). It defines alternatives by the distances between the indicators' best and worst values. If the result equals one, it is the best solution, if the result equals zero, it is the worst solution.
Using the estimated values of the criteria and their weights, applying the AHP (Analytic Hierarchy Process) and Direct evaluation methods, and evaluating the selected technologies was performed. The TOPSIS method was applied and the results are presented in Tab. 1. The technologies' ranking is similar using the two different calculations methods; only the ranks 7 and 8 were identified differently. It has no considerable significance since only the best one is needed.

Table 1 Measurement Technology Ranking

	Technologies							
	Reference technology Plasser Optical Gage Measuring System	Laser profile measurements	Distance sensor laser, time of flight	Time of flight camera	Digital Image Correlation (DIC) of high-speed filming	Laser based systems, displacement	Thermal vision	Stereo vision
TOPSIS DE	0,816	0,761	0,830	0,730	0,789	0,661	0,847	0,879
Ranking	4	6	3	7	5	8	2	1
TOPSIS AHS	0,886	0,808	0,888	0,655	0,863	0,801	0,910	0,920
AHP	4	6	3	8	5	7	2	1

It was found out that technology based on stereo vision is the best for this specific task. The technology-based on thermal vision came in second place. Therefore, it was decided to conduct the experimentation by investigating two sensors representative of these two technologies through laboratory and on-track testing.

4.4 Test platform for real-time testing of sensor solutions

During the experimental investigation, two technologies were under review:
a) Based on stereo camera (the Zed stereo camera developed by STEREOLABS);
b) Based on the thermal imaging camera (the T420 thermal imaging camera developed by FLIR).

Figure 2 Sensors under investigation: Stereo camera ZED by STEREOLABS (left image) and Thermal camera T420 by FLIR (right image)

Algorithms for required geometrical data evaluation from the image were created. The developed algorithms are suitable for both selected technologies. Methodology for measurement accuracy calculation was developed, and measurement accuracy for the devices under investigation was calculated, for the stereo camera it is ±0.7 mm and ±1.4 mm for thermal imaging camera. Using the developed algorithm measurement accuracy can be calculated for different devices that system developers may use in the future.
Test platform for real-time testing of sensing solutions initially in the facilities laboratory of VGTU was developed. It included cameras connected with power-efficient embedded AI computing device Nvidia Jetson TX2 and test rig. Data processing algorithm was validated using wild experimental data measured in Lithuanian Railways. Maximal power consumptions of measurement equipment with the camera is less than 20W for both technologies. Data can be transmitted to the data warehouse via Wi-Fi and Ethernet or stored in hard disk installed in Nvidia Jetson TX2. The algorithms were developed during the task implementation so output data format can be freely chosen.

5 Conclusion

This paper focuses on developing a sensor system to support track geometry monitoring that was carried out within the EU Shift2Rail project Assets4Rail. The initial results of Assets4Rail research for track geometry monitoring by systems suitable for installation on commercial trains, allowed to identify the System Breakdown Structure (SBS) and the requirements of an on-board sensor system for wheel-rail relative position detection. The most appropriate and promising candidate technologies to realise was found stereo cameras and thermographic cameras. Algorithms for required geometrical data evaluation from the image were created. The developed algorithms are suitable for both selected technologies.
A methodology for theoretical measurement accuracy estimation was developed and measurement accuracy for the devices under investigation was calculated: a value of ±0.7 mm was calculated for the stereo camera and ±1.4 mm for thermal imaging camera (e.g. for track gauge measurement, the EN 13848-1:2019 standard requires an accuracy of ±1.0 mm). Using the developed algorithm measurement accuracy can be calculated for different devices that system developers may use in the future.
Test platform for real-time testing of sensing solutions initially in the facilities laboratory of VGTU was developed. It included cameras connected with power-efficient embedded AI computing device Nvidia Jetson TX2 and test rig. Data processing algorithm was validated using wild experimental data measured in Lithuanian Railways (LTG).
Finally, the main findings of the first step of experimentation described above are shown below. These results will be the basis for subsequent development of the sensor system design and a second experimental phase foreseen on a diagnostic train made available by the Italian Railways (Ferrovie dello Stato Italiane).

Both stereo vision and thermal imaging cameras used in the investigation can provide sampling frequency up to 60 Hz. Hardware used in the investigation allows processing at up to 120 frames per second and is suitable for real-time work. Selected technologies can be applied in trains performing at high velocity ranges, selecting devices with higher sampling frequencies. About mounting the sensors on the bogie frame, it was found that additional housings are required to prevent cameras from ambient conditions. Housings should be placed on bogies via dampers, which will protect the equipment from overloads up to 20g. Additional airflow should be provided to the housings; such a system requires about 3W and can prevent the system from dust, rain, snow, and ensure system performance at low temperatures.
Algorithm for wheel-rail transversal position monitoring requires precise positioning of thermal imaging camera during installation. Technology users and developers should consider if there is no way to guarantee the desired distance to the measuring object correction in the algorithm should be provided. In a stereo vision, this is not required, as the camera automatically calculates the distance to the object. However, the stereo camera should be calibrated before the installation. They are using technology based on a stereo vision; an additional light source is needed for wheel-rail contact.

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References

[1] https://shift2rail.org/

[2] http://www.assets4rail.eu/about/

[3] EN 13848-1:2019: Railway applications - Track - Track geometry quality - Part 1: Characterisation of track geometry.

[4] Weston, P., Roberts, C., Yeo G., Stewart, E.: Perspectives on railway track geometry condition monitoring from in-service railway vehicles, Vehicle System Dynamics, 53:7, 2015, 1063-1091, DOI: 10.1080/00423114.2015.1034730.

[5] EN 13848-2:2006: “Railway applications - Track - Track geometry quality - Part 2: Measuring systems - Track recording vehicles”.

[6] De Rosa, A., Alfi, S., Bruni, S.: Estimation of lateral and cross alignment in a railway track based on vehicle dynamics measurements, Mechanical Systems and Signal Processing, Volume 116, 2019, pp. 606-623, ISSN 0888-3270, https://doi.org/10.1016/j.ymssp.2018.06.041.

[7] Burstow, M., De Podesta, M., Pearce, J.: Understanding wheel/rail interaction with thermographic imaging - Proceedings of the 22nd IAVSD Symposium, paper N. 105, Manchester, 2011.

[8] Yamamoto, D.: Improvement of Method for Locating Position of Wheel/Rail Contact by Means of Thermal Imaging. Quarterly Report of Railway Technical Research Institute (RTRI), 60 (2019) 1.

OPTIBOX - SOFTWARE TOOL FOR THE OPTIMAL DISTRIBUTION OF HOT BOX AXLE DETECTORS

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Abstract

Axle bearings may constitute a critical component with regards to safety due to the fact that they can present sudden failures. Hot box detectors are wayside devices that aim at identifying axle bearings with a high potential of failure. Therefore, it is important to place these sensors along the network in order to minimize the risk of axle bearing failures that could derive in train derailments. How many and where to install these wayside devices depends on the requirements of each country and on the available investment capacity. However, there is no tool in the market that helps the Infrastructure Managers to prioritize locations for hot box detectors. In this context, the OPTIBOX tool that is presented in this article appears as useful and easy-to-use tool to guide Infrastructure Managers in the selection of the most appropriate locations for hot box detectors according to historical data of the line and its main relevant characteristics, such as speed, type of trains or volume of traffic.

Keywords: axle bearing, hot box detectors, optimisation, OPTIBOX

1 Introduction

The monitoring of the axle bearings of rolling stock can be performed either by wayside systems either by onboard monitoring devices. Nowadays the most used monitoring systems are the Hot Axle Box Detectors (HABD), a wayside device that detect axle bearing faults based on the measurement of in-service axle bearing temperatures.
In terms of managing axle box detectors, the initial investment on the acquisition of HABD as well as its maintenance during the lifetime have to be taken into account by the Infrastructure Managers considering safety and economic aspects.
Currently, there is no tool available for the Infrastructure Managers to help deciding the physical distribution of the monitoring systems, namely hot axle box detectors, in a railway network. Therefore, within the MAXBE (Interoperable Monitoring, Diagnosis and Maintenance Strategies for AXle BEarings) project a software was developed to overcome this absence. The MAXBE project, funded by the European Commission and more details on the project results can be consulted on the website www.maxbeproject.eu. The developed software tool allows to define and it is a decision-aid support system, which assists the infrastructure manager in the decision of the physical distribution HABD within the railway network considering their own criteria regarding safety, quality of service and also taking into account the main guidelines for the installation of these devices in the railway network of each country. The tool, developed for the OPTImal distribution of hot BOx aXle detectors, was named OPTIBOX and it allows to consider historical and statistical data of the railway network, the risk

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associated to certain indicators and the importance assigned to each one of the pre-defined indicators in order to be able to identify the most critical aspects regarding the axle bearing failure in a railway network. The software is available in excel format programmed with Visual Basic to be a user-friendly one and it is easy to implement and very flexible to the end-users needs. At the end, the OPTIBOX is able to suggest the most adequate places to install a wayside monitoring system, a hot box axle detector or other type of device, considering a priority list that results from the historical data and also from the infrastructure managers experience and the risk assessment, which is included in the definition of the risk criteria and the importance attributed to each one of the defined indicators.
In this paper, a summary of the tool requirements is indicated. Then, the main features of the software tool along with the methodology employed in its development are explained. Furthermore, a case-study of the application of the tool into a realistic scenario is presented.

2 Requirements of wayside monitoring devices installation

Regarding the wayside monitoring devices installation, the European countries follow the recommendations of the European Standard EN 15437-1 [1]. Nevertheless, as this standard does not give recommendations for the strategic location of HBDs, usually each country has its own national requirements. Accordant to the EN 15437-1 [1], the requirements for wayside installation can be classified in four groups:

- requirements to assure a steady and clean recording of the signal; this aspect refers to the need of a stationary running of the train, which means that the wayside must be placed in a section where train runs at constant speed, preferably on straight alignment, far from switches and crossings, etc.
- requirements to minimize operational impact; this issue accounts for the aim of reducing as much as possible the traffic disruption in case of a detection of a hot axle box by the HBD; or this reason, HBD should be placed close to stations, a higher number of HBD should be installed in lines with higher traffic, etc. These requirements also include those related to safety issues, such as installing the HBD before tunnels or bridges to detect any failure before entering the tunnel or the bridge.
- requirements to minimize interferences with electrical/electronic equipment, giving special attention to the signaling system and the overhead line return current.
- other requirements, such environmental conditions (such as to avoid high changes of temperatures, etc.), or the risk of damage of theft.

3 Software tool

The main objective of the OPTIBOX tool is to help the user to identify the most suitable locations to install wayside devices to detect axle bearing failures such as the Hot Axle Box Detectors. The software (SW) was defined in an excel file and programmed with Visual Basic in order to be an easy-to-learn, easy-to-use tool and a flexible tool, straightforwardly implemented by potential users and adapted to satisfy the user's needs.
OPTIBOX is organised in two main parts, each one divided in several steps. The first part is the core part of the S tool and deals with the selection of the most suitable line segment to install the wayside diagnostic devices. It starts with the data input, where all the information regarding the network under analysis and its characteristics are introduced. Once the initial information is included, the following step is weighting. In this phase of the procedure, the user define which parameters are the most important ones, aspect that is important since criteria can vary from one Infrastructure Manager to another, However, the toll also proposes recommended values in order to guide the user in the definition of appropriate values.

Then, the user can proceed to the last step of the first stage: prioritisation. OPTIBOX creates a ranking of the line segments, according to the scores The second stage of the procedure starts from there and it aims to help the user to define the best location to install the wayside devices within the identified most suitable line segments. This stage can be split in two phases: definition of the general requirements from each country and definition of specific requirements, such as maximum distance to signalling lights, etc. Figure 1 shows the methodology of the software tool.

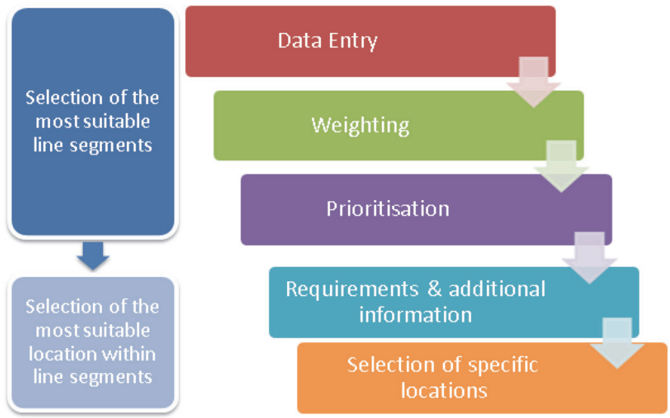


Figure 1 Methodology of the SW tool

4 Case study

A case study was developed to highlight the benefits of the OPTIBOX tool for the definition of the optimal physical distribution of wayside diagnostic systems (HBD). The example is based on realistic data (not real) of the Portuguese Railway Network. Real data are not used due to confidential reasons.

Section 1 – Input Information

In the first section of the tool, the preliminary information is included in the software tool by the infrastructure management company including the country in order to take into account the requirements of the installation of the wayside systems for that country identified previously, in this case study, the country is Portugal. Then, the railway line in the network to be analyzed needs to be selected in order to consider its specific features. In the present case study, a double railway line with a total length of 336 Km is considered and the aim is to find an optimal distribution of HABD for that line. The information to proceed with the analysis and included in the tool by the end user is the following:

- number of segments that should be considered in the analysis of that specific line.
- maximum distance of the segments into which the line is divided. This parameter can be defined by the user, but the distance of the segments should be smaller than the maximum distance between HABD defined in the standards available for the specific country.

Number of singular points defined according to the number of singular points that should be considered in that specific line for the purposes of this software tool, in which singular points can be considered as connection of a branch line or any other element that may cause any modification in the parameters defined for each segment, such as traffic, accidentability, speed and type of trains. Additionally, the information related with the "kilometric point" (KP) of each singular point of the line defined by the correspondent kilometre, should also be provided. In Table 1, the input included in section 1 is presented.

Table 1 Information included in the 1st stage

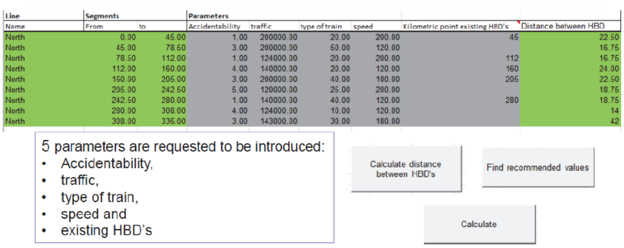
Information	
Country	Portugal
Line	Northern Line
Length of line (in km)	336
Segments to divide the line (length)	50
Number of singular points	5
Kilometre of each singular point (in Km)	45; 112; 160; 205; 280

Section 2 – Parameters

In the second section, several segments are generated automatically based on its initial and final KP and according to the information provided in the previous section 1. OPTIBOX will produce the segments considering the lower value between the maximum distance of each segment and the distance considering the singular points of the line. Then the following four parameters are used to characterize each segment:

- Speed (km/h) – maximum commercial speed at each section (considering the train that circulates with the highest speed). By considering this parameter, a higher level of priority is indirectly being given to high speed lines. Moreover, in many occasions higher speeds means less distance between HABD, and therefore, the worst case scenario regarding safety (maximum speed) should be considered. In the present case study, the speed is 220 km/h.
- Traffic (Tonnes/year) – the total tonnage of the line per year in each section, including freight and passenger trains. With this parameter, it is possible to give a higher level of importance to main lines usually with higher level of service (higher traffic). Furthermore, higher volume of trains means higher risk of axle bearing failures taking place in this line and more operational impact if a train has an accident or its travel has to be interrupted. In the case study, the railway line is a mixed one, and therefore, the total tonnage of freight and passenger trains in each section is included.
- Type of Trains (% of freight trains over total traffic) - ratio between the percentage of freight trains and the total traffic in each section. This aspect allows distinguishing the main use of the line and giving different levels of importance to the railway lines
- Accidentability (incident/year) - number of incidents that occur per year in each segment line.

In Figure 2, the parameters input interface are shown.



Line	Segments		Parameters					
Name	From	to	Accidentability	traffic	type of train	speed	Kilometric point existing HBD's	Distance between HBD
North	0.00	45.00	1.00	200000.00	20.00	200.00	45	22.50
North	45.00	78.50	3.00	200000.00	50.00	120.00		16.75
North	78.50	112.00	1.00	124000.00	20.00	200.00	112	16.75
North	112.00	160.00	4.00	140000.00	20.00	120.00	160	24.00
North	160.00	205.00	3.00	200000.00	40.00	100.00	205	22.50
North	205.00	242.50	5.00	120000.00	25.00	200.00		18.75
North	242.50	280.00	1.00	140000.00	40.00	120.00	280	18.75
North	280.00	308.00	4.00	124000.00	10.00	120.00		14
North	308.00	336.00	3.00	143000.00	30.00	180.00		42

Figure 2 Second input stage:parameters

The OPTIBOX estimates the most suitable location for implementation of new HBD as a decision support system, but considering the existing wayside monitoring systems already installed in the railway line. Therefore, in the second part of the SW, the information related to the number of existing HBDs is introduced mentioning the respective exact kilometric point. As an example, if there is a HABD installed in KP 45, the user should include 45 in the row of the segment with the correspondent interval (KP 50-60). In Table 2, the results calculated by OPTIBOX are presented.

Table 2 Mean and deviation parameters calculated by the SW

	Mean	Desv.
Accidentability	2.78	1.48
Traffic	154555.56	35025.39
Type of train	28.33	12.75
Speed	151.11	42.56
DIST between WDDs	21.78	8.25

Section 3

The attribution of weights allows defining the importance given to the parameters and it is divided in two steps. The software tool automatically produces a table with recommended values estimated based on the mean and deviation of each parameter. Considering this results (Table 3), the limit values of the intervals considered for each parameter are recommended as well as the correspondent weights. However, the user is able to define different values for the intervals and also the weights taking into account their experience and preferences. In Table 3, the estimated weigh parameters are presented for the case study.

Table 3 Example of weight of parameters

	Range		Total Weight	Points
Accidentability	From	To	30	
	0	2	20	6
	2	5	60	18
	5		100	30
Traffic	From	To	20	
	0	142690	10	2
	142690	355310	40	8
	355310		80	16
Type of train	From	To	10	
	0	40	30	3
	40	80	80	8
	80		90	9
Speed	From	To	20	
	0	100	10	2
	100	180	90	18
	180		100	20
DIST between WDDs	From	To	20	
	0	20	20	4
	20	60	80	16
	60		100	20

Section 4
Considering the information previously included in the SW tool, it is possible to estimate a priority list that presents the ranking of segments ordered by the segment that presents the highest importance in the installation of a HABD system to the segment with less priority. It should be noticed that this priority list is based on the weights defined by the user, and any modification of those parameter weights implies a new estimation of the priority list. The "Priority List" is estimated through the "Calculate" button presented in the second sheet of Excel SW tool. Part of the resulting priority list is presented in Table 4, where the segment with higher urgency to install a HABD is from KP 205.00 to KP 242.50.

Table 4 Example of priority list

Line ID	From	To	Descending order (priority for HABD installation)
North	205.00	242.50	58
North	308.00	336.00	56
North	45.00	78.50	54
North	160.00	205.00	52
North	112.00	160.00	41
North	280.00	308.00	41
North	0.00	45.00	35
North	242.50	280.00	34
North	78.50	112.00	29

After the identification of the track segments with higher priority, the user can determine a suitable HABD location regarding the recommended minimum distance from a HABD to a main signal.
In order to determine a suitable location within the selected line segments (those with higher score in the ranking), the OTIBOX tool considers significant parameters as the train speed, location of main signal, danger points, minimum distance from main signal, average distance between wayside devices, the train length and the braking distance diagnostics systems.
In all cases, it has to be guaranteed that the distance between HABD to be installed and main signal is large enough to allow the train to stop. For this minimum distance, a reaction time is included in the formulation in order to make sure that the alarm of a hot box can be noticed by the safety system and can be transferred to the signals along the line. This has been estimated in 120 seconds. On conventional lines the train driver must have the chance to see that a signal changes from "Drive" to "Stop".
As a result, the OBTIBOX tool provides a diagram where the user can see in a visual way which locations are possible with regards to the position of the main signals.

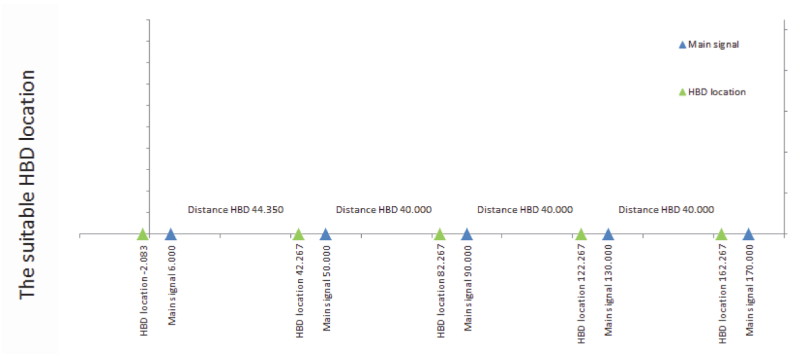


Figure 3 Diagram of HBD location

5 Conclusions

Unexpected axle-bearing failures can cause derailments, that should be prevented not only to avoid the disruption of traffic but specially to guarantee the safety of passengers. To the definition of the location to install the new wayside monitoring systems in order to obtain reliable monitoring data is fundamental to assess the correct condition of the axle bearing. Currently, European rail networks are provided with Hot Axle Box Detectors that detects axle bearings faults by the measurement of the in-service temperature. All the European countries follow the recommendations of standard EN 15437-1 [1] on the installation of HABD monitoring systems. Nevertheless, since the standard does not give recommendations for the strategic location of HBDs, each country follows its own national requirements, which means that significant differences can be found when it concerns to the distance between two consecutives HBD.
Within the MAXBE project, a software tool (OPTIBOX) was developed. In this paper, the main features of the software tool are described and a case-study is presented.

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References

[1] EN 15437-1 Railway applications, Axlebox condition monitoring, Interface and design requirements Track side equipment and rolling stock axlebox, 2009.

[2] EN 15654 Railway Applications - Measurement of wheel and axle loads Part 1: Interoperable “In-service” rail vehicles, 2015.

[3] EN 50125-3 Railway applications, Environmental conditions for equipment Equipment for signalling and telecommunications, 2003.

[4] ISO 3095:2005 Railway applications - Acoustics - Measurement of noise emitted by railbound vehicles

[5] UIC 518 Testing and approval of railway vehicles from the point of view of their dynamic behaviour, 2005

[6] MAXBE Project Deliverables 2.8: Testing site(s) in Belgium, MAXBE FP7 project, 2012, www.maxbep-roject.eu
[7] MAXBE Project Deliverable 2.9: Testing site(s) in Portugal, MAXBE FP7 project, 2012, www.maxbep-roject.eu
[8] Réseau Ferré de France (RFF), Document de référence de réseau, Annex 6.14. Détecteurs de boites chaudes, 2012.
[9] EPSF (Etablissement Public de Sécurité Ferroviaire), Document technique, Dispositif de surveillance des essieux, 2010.
[10] Eisenbrand, E.: Hot box detection in European railway networks, RTR Special, 2011
[11] Dings Habd, P., Wild, A.: scan of the situation Europe, Delta Rail, 2010.
[12] MAXBE Project D5.5- Diagnostic systems: Optimal Distribution and Interval Rates, April 2014, www.maxbeproject.eu
[13] MAXBE Project D5.6- Development of a Software Tool for the Optimal Physical Distribution of the Diagnostic Systems, April 2014, www.maxbeproject.eu

COMPARATIVE RIDE COMFORT ANALYSIS OF IN-SERVICE TRAMS ON EXTREME ALIGNMENT CONFIGURATIONS USING SMARTPHONE-BASED SENSING

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Abstract

In this paper, a cost-effective method for monitoring and evaluating the tramway passenger comfort and ride quality are presented using motion sensor data of smartphone fitted to in-service vehicles. Running vehicles experience a broad spectrum of vibrations and oscillations that occur in response to excitation inputs of vehicle-track coupled dynamics. Android mobile platform-based acquisition software is developed (CAFat) for commercial use, and the data from smartphone built-in sensors such as accelerometer and gyroscope are processed by sensor fusion and are coupled with local and global positioning using GNSS data to identify sections with poor ride quality. Results are promising and demonstrate that poor ride quality can be accurately localized on a tramway network. The proposed method enables infrastructure monitoring done by conventional passenger cars and makes the possibility of comparing the ride quality of traditional old-designed and the modular multi-articulated in-service vehicles.

Keywords: tram, smartphone, ride comfort, inertial sensors, track quality

1 Introduction

The railway is a guided transportation system that requires several track inspections and examinations to ensure safe operation. In the design process, the values of the track alignment parameters are chosen to ensure a safe riding with at least a minimum comfort level. A good compromise has to be found between train dynamic performance, maintenance of both the vehicle and track, as well as construction costs.

Tramways overcome horizontal curves with much smaller radii than the conventional railway vehicles, as well as the passengers in a tramway vehicle are more likely to be standing supported or moving around within the vehicle, therefore the risk of passengers losing their balance and falling is increased [1]. The standing passengers in tramway transport can have significant lateral force on entering both the small radius curve and the diverging direction of turnouts depending on the vehicle speed and the structural design of the vehicle running gear. However, the limit values of the lateral passenger comfort parameters (lateral acceleration and its rate of change) are not supported in the present Hungarian regulation [2] by the measured kinematic movement characteristics of the new tramway vehicles operated in Budapest.

The attention of this paper is focused on the measurement of tram irregular movements and vibration using smartphone motion sensors fitted to traditional and modular designed in-service vehicles operated in Budapest tram network. To measure the kinematic motion

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characteristics of these vehicles, an application are developed called "CAFat" in the android software platform that is capable of timing synchronized recording of all phone sensor data and GPS location information. After sensor calibration, the virtual transition length and the representative cross-sections of investigated tramcars were determined in terms of lateral passenger comfort (using the data of yaw-rate gyroscope and lateral acceleration) and then the line tests were carried out only in the relevant vehicle cross-sections. During the kinematic analysis, the lateral accelerations and the yaw-rate gyroscope data recorded on the car body were investigated. The peak values and the main characteristics of the recorded signals were analyzed.
In the next Section, details are given about the measurement setup adopted for experiments, the newly developed android application for data acquisition and the applied data processing methods. Then, Section 3 is intended to face the line test results and gives the main conclusions on the considered possibilities of mobile sensing for tramway track and vehicle condition monitoring.

2 Applied measurement system using mobile phone sensors

2.1 Android mobile-platform based sensor data acquisition (CAFat)

Today's smartphones include a range of sensing and communication capabilities, in addition to computing which can be used to infer the vehicle kinematics characteristics. We developed an application called "CAFat" running on the Android mobile software platform.
The program (Fig. 1.) has a "Start" button that starts the measurement, which changes to "Stop" after measurement begins. It also shows the time elapsed since the beginning of the measurement and the accuracy of GPS location data. Furthermore, the program allows for the operator to label each event encountered in real-time by pressing a button on the phone screen every time the impact of one class of event was felt.
The synchronized sensor data is logged to a Comma Separated Values (CSV) file. The sensor data in the phone are read out at the maximum possible sampling rate. Depending on the phone model, the available sampling frequency may vary from 100 Hz (for mid-range phones) to 600 Hz (for premium-category phones). To determine the irregular vehicle movements and oscillation, the 100 Hz sampling rate may be already sufficient, because the railcar body oscillations are generally between 0 and 20 Hz, but the investigation of rail surface defects requires higher sampling frequency. Only high-end phones were used in the measurements, whose sampling frequency was more than 400 Hz.

2.2 Measurement setup

The axis arrangement of sensors in smartphones is standardized, independent of the manufacturer. Three-axis accelerometer and gyroscope are suitable for describing spatial motion and the measuring axes follow a right-handed coordinate system. If the phone is positioned horizontally with a display facing upwards so that the charger connector is closer to you, the 'x' axis is to the right, the 'y' axis along the long side of the phone towards the camera, and 'z' the positive direction of the axis points vertically upwards. The positive direction of rotations around these axes is also determined by the right-hand rule. During the measurements, the phone is placed on its long side fixing to the wall of the car body so that the positive direction of the 'y' axis is the same as the travel direction (Fig. 1.). In the measuring arrangement used, the axis 'y' records the longitudinal acceleration, the axis 'x' is the vertical acceleration, while the axis 'z' records the lateral acceleration of the car body. The pivoting around the 'x' axis is the yaw movement of the vehicle, the movement around 'y' axis is rolling, and the pitching movement is around the axis 'z'. For most of the measurements, the device was attached to the glass of vehicle window using a silicone pad, which arrangement partly filters the higher frequency components, but keeps the low frequencies relating to the irregular oscillatory motion of vehicle.

2.3 Data processing

A) Processing and interpreting acceleration data
The vibration, sudden impact shocks and the orientation in a motionless position can be sensed with accelerometers. The acceleration data measured on the car body is extremely noisy and can not be used directly, prior processing is required. To remove vibrations of higher frequency components that are irrelevant for the kinematic test, a two-way moving average method is applied.

B) Processing and interpreting gyroscope data
The gyroscope is suitable for describing irregular and oscillating movements of the car body. The track alignment and the car body tilt of the train can be derived from the yaw-rate and roll-rate gyroscope data respectively. During measurement, the yaw-rate gyroscope data highly influenced by both the track alignment layout and the design of the vehicle running gear. Therefore, this data accurately describes the steering mechanism of the vehicle on curved track. For each evaluation, a 2 Hz low-pass filter was used uniformly on the raw gyroscope data that complies with the relevant requirements of the EN 12299 standard [3].

C) Relationship between lateral acceleration and yaw-rate gyroscope data
The Fig. 2. compares lateral acceleration and the yaw-rate gyroscope data recorded by Xiaomi Pocophone F1 on tram line 49 in Budapest. The top chart shows the velocity recorded by GPS, the second one introduces the raw (grey-colored graph) and the filtered yaw-rate gyroscope data (black- colored graph), while the third one shows the non-filtered (grey) and filtered (black) lateral accelerations. The measurement was performed on the car body of the Ganz type, articulated tram. The track horizontal alignment can be obtained from the filtered raw acceleration data (using a 0.5 Hz low-pass filter or a moving average with 1.00 s windows width, see the third graph in Fig. 2.), while in the case of the gyroscope, the raw data contains this information.

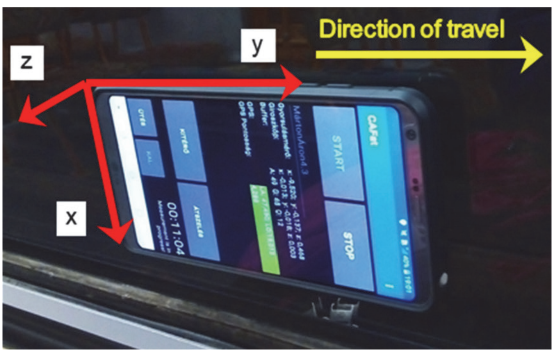

Figure 1 Screenshot of the developed Android smartphone application ("CAFat") and interpretation of the measurement axes

It is important to mention that when examining vehicles with good running properties, the raw acceleration data also clearly shows the horizontal alignment.
Filtered lateral acceleration and gyroscope data are highly similar (see graphs 2 and 3 in Fig. 2.), which is due to the fact that the data provided by the gyroscope can be used to calculate the quasi-static lateral acceleration using the Eq. (1):

$$a_0 = v \cdot \omega = v \cdot \frac{v}{R} = \frac{v^2}{R} \tag{1}$$

where

a_0 [m/s^2] - quasi-static lateral acceleration,
v [m/s] - velocity,
ω [rad/s] - angular velocity (yaw-rate gyroscope),
R [m] - radius of the curve.

The car body tilting acceleration can be computed by the difference between the filtered lateral acceleration and the calculated quasi-static lateral acceleration. On the section of "Szabadság" bridge (between the position of 3300 and 3600 m) the track is built with 22 mm superelevation. Fig. 2. clearly shows that the yaw-rate gyro is not sensitive to car body roll, while the recorded lateral acceleration contains both the tilting-, and the centrifugal acceleration. The fourth graph on Fig. 2. shows the calculated tilting acceleration compared to the nominal value of track cant measured by TrackScan track geometry measuring trolley (red-colored graph). The reference tilting acceleration is calculated from the measured track cant value.

D) Sensor fusion between magnetometer, accelerometer and the gyroscope data

To calculate device absolute orientation sensor fusion is applied. Generally, the accelerometer and magnetometer outputs include a lot of noise. The gyroscope in the device is more accurate and has a very short response time. Its downside is the dreaded gyro drift, which is accumulated when making the sum of angular velocity to get actual orientation.
To avoid both, gyro drift and noisy orientation, the gyroscope output is applied only for orientation changes in short time intervals, while the accelerometer data is used as support information over long periods of time. This is equivalent to low-pass filtering of the accelerometer and magnetic field sensor signals and high-pass filtering of the gyroscope signals.

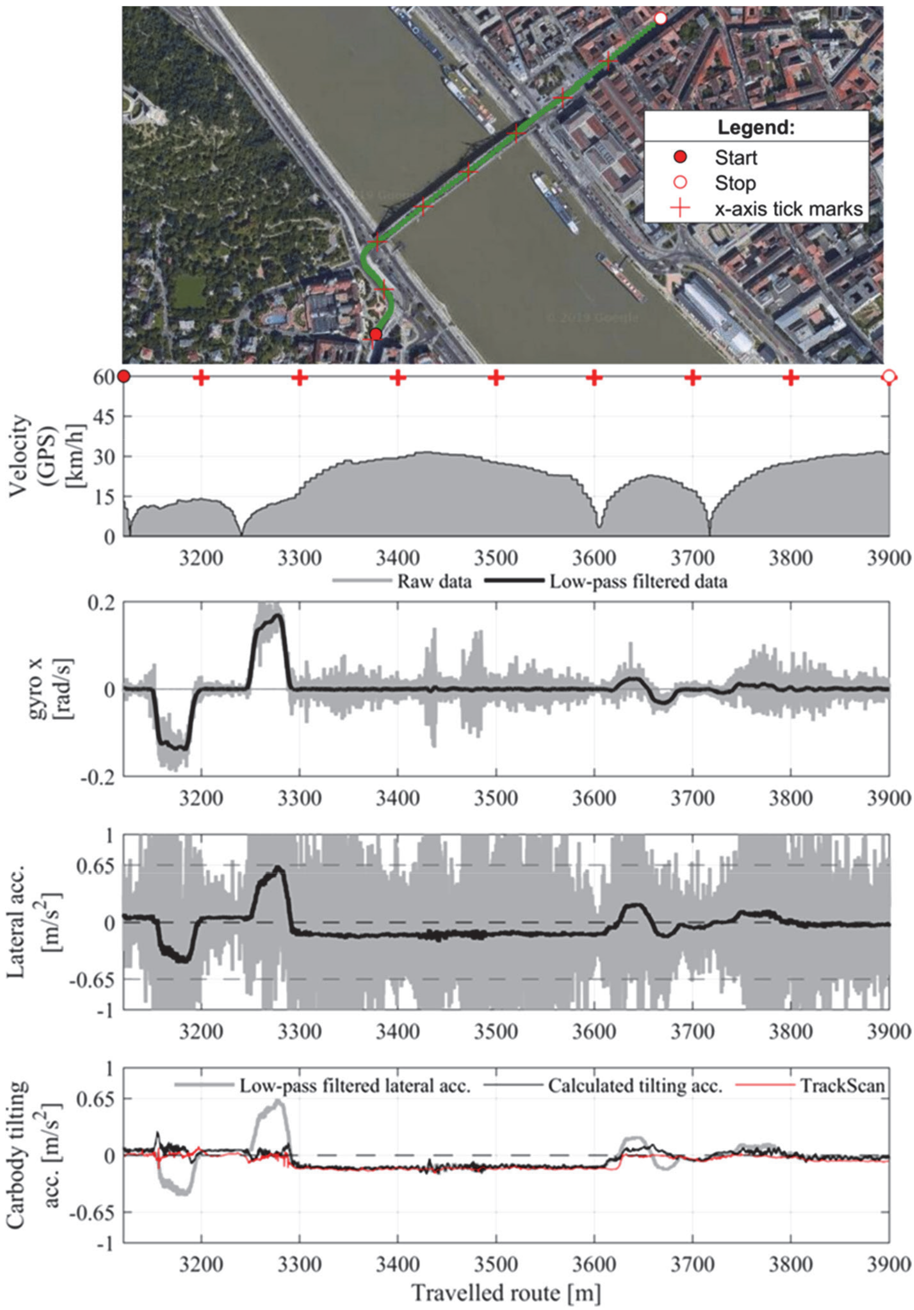

Figure 2 Determining the tilt of the car body using roll-rate gyroscope (gyro_x) and lateral acceleration data of in-service vehicle-mounted smartphone

3 Measurement results

Line test measurements were performed under real traffic conditions with passengers on different classes of tramway vehicles. The current fleet of vehicles operated in Budapest's tram network is not considered homogeneous, apart from standard vehicle configuration, i.e., a car body on two bogies, in modern tram designs various arrangements are applied. The running gear of the modular designed low-floor trams are based on a highly sophisticated axlebridge component and integrated completely into the car body. This structural design produces significant additional stress for the track compared to the traditional bogie vehicles.
The primary purpose of the tests was to determine the ride comfort, therefore the representative position of the passengers was decisive when selecting the measurement location within the vehicles. In determining the critical cross-section, several cross-sections were simultaneously measured on a vehicle and then the line tests were performed only in the determined relevant cross-section.
During the selection of measurement places for line test, the primary consideration was to quantify the extent of irregular vehicle movement (lateral sway and oscillation) and to search for a relationship with the track alignment parameters.

3.1 Curving behavior of investigated tramcars

The virtual transition length of the vehicles was investigated on entering circular curves or on a reverse curve with an intermediate straight section. In the case of abrupt change in curvature, the approximate formulas for determining the lateral acceleration do not take into account the virtual transition length of the vehicle, so curvature function is defined containing both the nominal track alignment parameters and curving behavior of the investigated vehicles. This curvature function must be matching line to the filtered yaw-rate gyroscope data. It is important to note that the value of the virtual transition length determined during the measurements depends greatly on the accuracy of the velocity data, from which the traveled distance was calculated. Nevertheless, the curving behavior of the full vehicle and its parts or modules could be properly separated. The virtual transition length of vehicles validated by measurements is summarized in Table 1.

Table 1 Virtual transition length (d) of tram fleet operated in Budapest

CAF Urbos3 (5 modules)		**Siemens Combino Supra**		**TATRA T5C5**	**TW6000**	**Ganz ICS**
Module	d [m]	Modul	d [m]	d [m]	d [m]	d [m]
C1, C2	1,800	1-6	1,800	6,700	6,400	6,000
S1, S2	<u>6,745</u>					
R2	1,850					
Legend: Wheelbase (italic font); bogie pivot distance (bold font); <u>module length</u> (underline and italic font)						

The virtual transition length of traditional bogie vehicles and articulated vehicles equal their pivot distance. However, the curving behavior of modular designed low-floor trams can vary per modules according to the design of their running gear. In the case of Combino tram, each module is supported by axlebridge and its virtual transition length equal to the wheelbase (1,80 m). The CAF vehicle consists of suspended (non-folded) and driven car body parts. The length of the virtual transition that significantly affects the curving behavior varies by vehicle modules: wheelbase distance for driven modules, and module length for suspended ones.

Due to the different virtual transition length, the most sensitive parts of the vehicle are the front and rear modules.
Fig. 3. compares the curving behavior of a conventional bogie and modern low-floor vehicle based on the angular rotation about the vertical axis measured on the car body. The blue-colored diagrams show the low-pass filtered yaw-rate gyroscope data ($gyro_x$) of CAF and TATRA trams respectively, while the black graphs show the defined curvature function, which contains the identified virtual transition lengths of the vehicle or investigated module. In both cases, the measurements are performed at the rear of the trams using Samsung Galaxy S8 mobile phone. The irregular vehicle movement of modular low-floor CAF trams in low-radius curves without transition is evident.

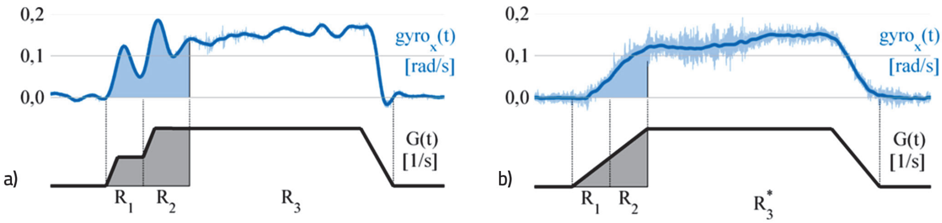

Figure 3 Comparative analysis of a) CAF Urbos3, b. TATRA T5C5 vehicles curving behavior at same velocity on VPh50/25 single track crossing turnout and connecting curved track using yaw-rate gyroscope data of Samsung Galaxy S8 fixed to the rear part of the trams

4 Conclusion

This paper presents measurements of tram kinematic movements and vibration using Smartphone Motion Sensors, as well as qualifies the curving behavior of different classes of tramway vehicles and estimates the vehicle dynamic response on track/rail irregularity.
Smartphones could be mounted basically anywhere on the vehicle to assess ride comfort. The peak oscillatory accelerations recorded at the inside corners of passanger cabin are often significant according to the vehicle structural design, but for qualifying the vehicle curving behaviour the cabin parts close to running gears (bogies) are the most suitable measurement setup.
The performance and sensitivity of sensors in high-end smartphones have significantly improved over the past few years, enabling them to provide useful information to track management due to their reliability and lower cost compared to industrial solutions. The accuracy of the high-end smartphones built-in sensors influenced by many factors (running gear design, track alignment layout, vehicle condition, velocity) and only partially ensure the “mm” precision required for the railways industry, but the real-time measurement of vehicle vibrations during commercial operation allows for rapid response, such as emergency track inspection and maintenance, in situations when vehicle vibration observations detect irregularly large deviations from standard control values. Thus, the use of this monitoring system to perform continuous monitoring of vehicle vibrations allows early detection of deterioration or other track irregularities, thus enabling railway operators to conduct effective maintenance work.
The research reported in this paper and carried out at BME has been supported by the NRDI Fund (TKP2020 IES,Grant No. TKP2020 BME-IKA-VIZ) based on the charter of bolster issued by the NRDI Office under the auspices of the Ministry for Innovation and Technology.

References

[1] Baasch, B., Roth, M., Havrila, P., Groos, J.C.: Detecting singular track defects by time-frequency signal separation of axle-box acceleration data Detecting singular track defects by time-frequency signal separation of axle-box acceleration data, 2019.

[2] Budapest Public Transport Ltd (BKV), HandBook for design and construction of tramway Tracks, (In Hungaruian) Közúti vasúti pályaépítési és fenntartási műszaki adatok és előírások, Budapest: Budapesti Közlekedési részvénytársaság, 2000.

[3] EN 12299: Railway applications, Ride comfort for passengers, Measurement and evaluation, Brussels, Holland: European Committee for Standardization (CEN), 2009.

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TRACK GAUGE MONITORING SCOPE OPTIMIZATION ON SMALL URBAN RAILWAY SYSTEMS

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Abstract

Urban transport plays a key role in the sustainable development of large cities. Urban railway systems, as eco-friendly mass transport systems, are becoming the basis of urban traffic development. Maintaining a high-quality service with continuously increasing traffic demand places an additional burden on public transport operators. Track geometry control has a major impact on availability and maintenance costs of public transport. Good management of rail infrastructure involves continuous monitoring of track geometry (track gauge, cant, twist, horizontal and vertical irregularities) where surveying should be done up to several times a year. Measuring of track geometry in chosen track cross-sections can be done automatically with relatively expensive equipment, or manually which is cheaper but takes longer. Therefore, the question arose as to whether it is possible on small urban railway networks to reduce monitoring scope by increasing of sampling distance, and if so, what should be recommended sampling distance. This paper presents, on the example of the City of Osijek tramway system, how changes in sampling distance effects on track gauge parameter. The results of the conducted analyses are presented and discussed. The recommendations on track gauge monitoring scope optimization on small urban networks are made.

Keywords: track geometry, track gauge; monitoring scope, sampling distance, urban railway system

1 Introduction

Two main objectives in the planning of modern urban transport systems are efficiency and sustainability. As rail systems are being recognized as eco-friendly mass transport systems, they are once again becoming the basis for urban area development and the backbone of urban transport systems [1]. The most popular urban railway transport system is the tramway transport system. Trams are electrically powered, usually lighter, and shorter than light or conventional rail vehicles, and they can share their route with other vehicles. These features make them more accessible for passengers in the historical city centres with narrow and winding streets.

With the annual increase in traffic demand, tram infrastructure needs to bear more load. This results in higher rates of tram track degradation. To ensure quality, and therefore safety and reliability of the system, the tracks must be continuously monitored and cost-effectively maintained. The key parameter in track quality assessment is the quality of track geometry. According to European Standard EN 13848-1 [2], it is presented as an assessment of deviation from the mean or designed track geometrical characteristics in the vertical and lateral plane which can raise safety concerns or have a correlation with the ride quality. To assess

track geometry quality, in total five geometric parameters need to be measured along the tracks: track gauge, longitudinal level, cross level, alignment, and twist. These parameters must be recorded as a consecutive set of data sampled at a constant distance-based interval not larger than 0,5 meters [2].
Track irregularities are usually measured with track-recording vehicles or cars (TRV or TRC) equipped with inertia-based measurement systems, which use accelerometers, gyroscopes, and lasers to record the track geometry and irregularities. Track irregularities can be estimated through numerical methods that are based on data collected with accelerometers mounted on the axle boxes, bogie frames, and in the car bodies of in-service trains [3].
Small urban railway system administrators usually do not have such sophisticated and expensive track maintenance machines, but the recording of the track geometry is carried out manually using a measuring trolley or a measuring rod. For manually operated devices each measurement needs to be recorded as a single value [1]. Unlike measuring vehicles, which can record the track geometry at relatively high speeds during operating hours, manual measurements are relatively slow and can affect the timetable if the measurements are not carried out without interruption of regular operations.
To ensure optimal allocation of resources for the railway infrastructure maintenance, it is necessary to continuously monitor the track geometry quality. However, given that monitoring is both time-consuming and costly [4, 5, 6], the main question is how frequently, both in time and space, measurements of the tram track geometry should be performed. The focus of this paper is on the evaluation of this sampling distance, defined by [2] as the traveled distance between any two consecutive measurement points on the same rail, for tram track geometry measurements done with manually operated devices. The main objective is to propose monitoring scope optimization for small urban rail systems by increasing the track geometry sampling distance. For this purpose, the geometry parameters of tram tracks in the City of Osijek were analysed. The data was collected in November 2016 for "Tramway track condition analysis on GPP Osijek tram network" study [7]. Although track geometry quality assessment is based on five track parameters, in this paper the focus was only on track gauge. The analysis was carried out separately for 3 identified network sections, and a total of 140 200-meters-long track segments using different sampling distances. By comparing the results of the analysis conducted for initial and different increased track gauge sample distances, the conclusions were made, and the recommended sample distance for this track geometry parameter is further elaborated.

2 Data collection and processing

Before measurements of tram tracks geometry, the tram network in the City of Osijek was divided into sections. This was performed in two steps. In the first step, the direction of tram traffic was taken into consideration. Then, an additional division was made due to the specific tracks layout, and organization of the lines. As a result, network was divided into 3 sections: Line L1: Zeleno polje–Višnjevac–Zeleno Polje; Line L2A: A.Starčevića–Mačkamama–A. Starčevića; Line L2B: Bikara–Mačkamama. Lines L1 and L2A are part of the double-track tram network, while L2B is part of the single-track tram network with passing loops at tram stops. Measurements were carried out in November 2016 by manually operated trolley TEC-1000 (GRAW product) that meets the requirements of the European Standard EN 13848-4 [8]. Measuring elements of the trolley include inductive linear motion sensors and a data logger [9].

Figure 1 The TEC-1000 trolley

Collected track gauge values are expressed as relative values in relation to the normal track width, in this case, 1,000 mm, i.e. as gauge deviations. The measurements were recorded with a sampling distance of a 1-meter. Data collected along 27,520 measurement points was georeferenced along track sections Line L1, Line L2A, and Line L2B and structured into a single database.
The geometry quality assessment is based on track geometry deviations or irregularities. There are around ten statistics, called track quality indices (TQI), adopted throughout the world to assess the track quality of a track segment. The main difference between them is the base on which TQI is calculated, as standard deviation, an average value, or weighted value over a track segment [10]. According to the results, TQI can be sort into two main categories: (1) objective or single-track quality indices, which are expressed separately for each geometry parameter, and (2) artificial or combined track quality indices, which try a different combination of track geometry parameters [11]. In addition to different approaches, TQIs are expressed for different track segment lengths, usually over 3–25 m, 25–70, and 70–200 m long segments [3].
For this research, in accordance with the European Standard EN 13848-1, track sections were segmented into 200-meter-long segments. As a result of this segmentation process, the tram network in the City of Osijek was divided into 140 segments. In Figure 2. track gauge deviations are displayed for each segmented section,
The track gauge deviation value ranges from minimum -10,8 to maximum +24,0 millimetres. Although the track gauge deviation values on most segments range between 0 and 10 millimetres, there are certain segments where the track gauge deviation value is around or above 10 mm.
Overall, the average track geometry deviation value is 4,9 mm with a standard deviation of 3,3 and a coefficient of variation of 0,3 %. The median value is 4,1 mm. The frequency of the track gauge deviation value is shown in Figure 3.

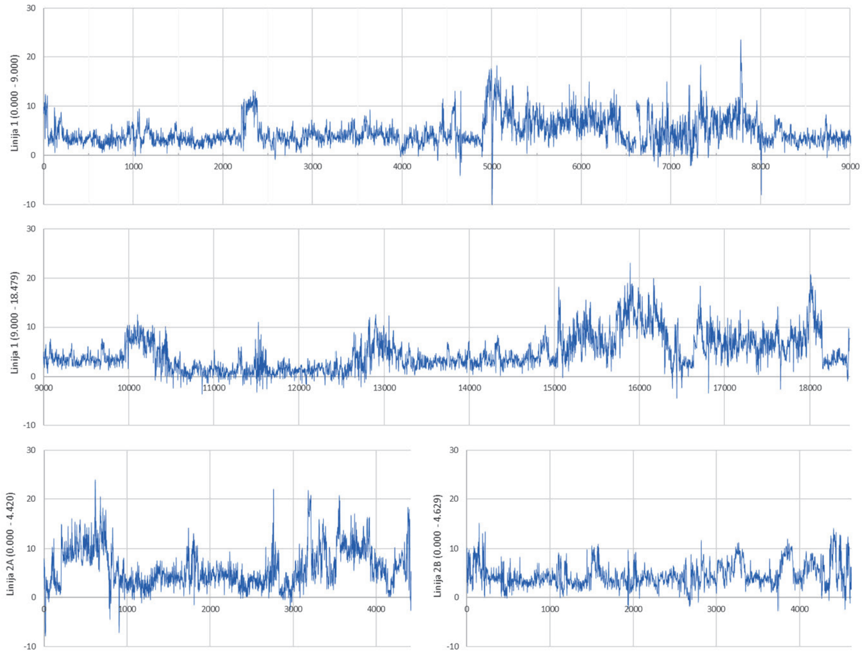

Figure 2 Track gauge deviations diagram, segmented, by sections

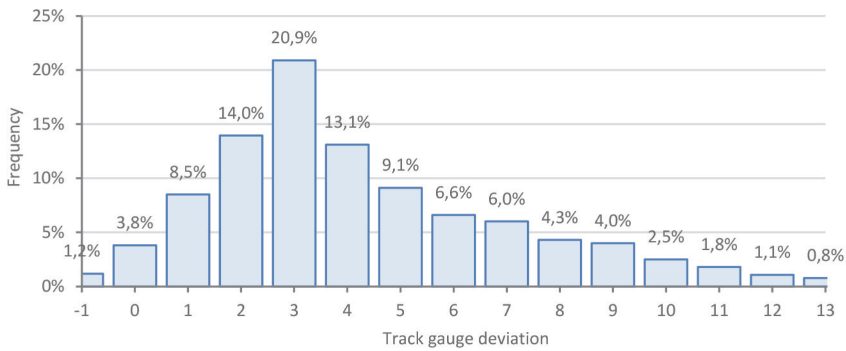

Figure 3 The frequency of the track gauge deviation value.

3 Track gauge data analysis

Track gauge deviation analysis was carried out for 200-meter-long segments using different gauge deviation sampling distances; 1, 2, 5, 10 15, 18, and 25 meters. A sampling distance of 1 meter represents the benchmark sampling distance. A bigger sampling distance means a smaller number of samples in a single 200-meter-long segment. For example, a sampling distance of 5 meters means that for each 200-meter-long segment there are 40 track gauge deviation values.

The analysis included calculation of track gauge deviation average values, standard deviation, and coefficient of variation for each 200-meter-long segment and different sampling distances. A linear regression analysis was then conducted to determine the level of correlation between and benchmark values calculated for sampling distance of 1 meter and values calculated for sample distances of 2, 5, 10 15, 18, and 25 meters. For each measure and sampling distance, correlation coefficient (r), coefficient of determination (R^2), and root-mean-square error (RMSE) were calculated.

The correlation coefficient is a statistical measure of the strength of the relationship between two variables and ranges from -1 to +1, where value +1 indicates that there is a positive relation, value -1 indicates that there is a negative relation, and value 0 means that there is no relation between two variables. Calculated correlation coefficients larger than 0.879 are showing that there is a positive relationship between two variables in all cases, but the relation is weakening with the increase in the sampling distance.

The goodness of fit between two variables is expressed with the coefficient of determination, which ranges from 0 to 1, where value 0 indicates that there is no relation between two variables and value 1 indicates the strongest possible relation of the variables. An increase in the sampling distance from 1 to 25 meters has a small effect on a change of the track gauge deviation average values but has a significant effect on the other two measures, track gauge deviation standard deviation and coefficient of variation. For example, the coefficient of determination calculated for the track gauge deviation average values changes from 0.997 for a 5-meter sampling distance to 0.988 for a 10-meter sampling distance. For the same change in a sampling distance, the coefficient of determination calculated for the track gauge deviation standard deviation values changes from 0.982 to 0.863 while the coefficient of determination calculated for the track gauge deviation coefficient of variation values changes from 0.982 to 0.862.

RMSE indicates how close the values calculated for a benchmark sampling distance are to the values calculated for increased sampling distance, where a value of 0 would indicate a perfect fit. An increase in the sampling distance from 1 to 25 meters has a significant effect on a change of all three measures, the track gauge deviation average values, standard deviation, and coefficient of variation. For a change in a sampling distance from 5-meters to 10-meters the RMSE calculated for the track gauge deviation average values changes from 0.142 to 0.263, the RMSE calculated for the track gauge deviation standard deviation values changes from 0.139 to 0.386, and the RMSE calculated for the track gauge deviation coefficient of variation values changes from 0.014 to 0.038. The results of linear regression analysis are presented in Table 1.

Table 1 A linear regression analysis result

Measure 2 m		A sample distance					
		5 m	10 m	15 m	18 m	25 m	
Average	r	1.000	0.998	0.994	0.988	0.985	0.971
	R2	1.000	0.997	0.988	0.976	0.970	0.943
	RMSE	0.030	0.142	0.263	0.381	0.426	0.571
Standard Deviation	r	0.999	0.991	0.929	0.946	0.908	0.879
	R2	0.998	0.982	0.863	0.896	0.825	0.774
	RMSE	0.044	0.139	0.386	0.387	0.519	0.548
Coefficient of Variation	r	0.999	0.991	0.929	0.946	0.908	0.879
	R2	0.998	0.982	0.862	0.895	0.824	0.772
	RMSE	0.004	0.014	0.038	0.038	0.052	0.054

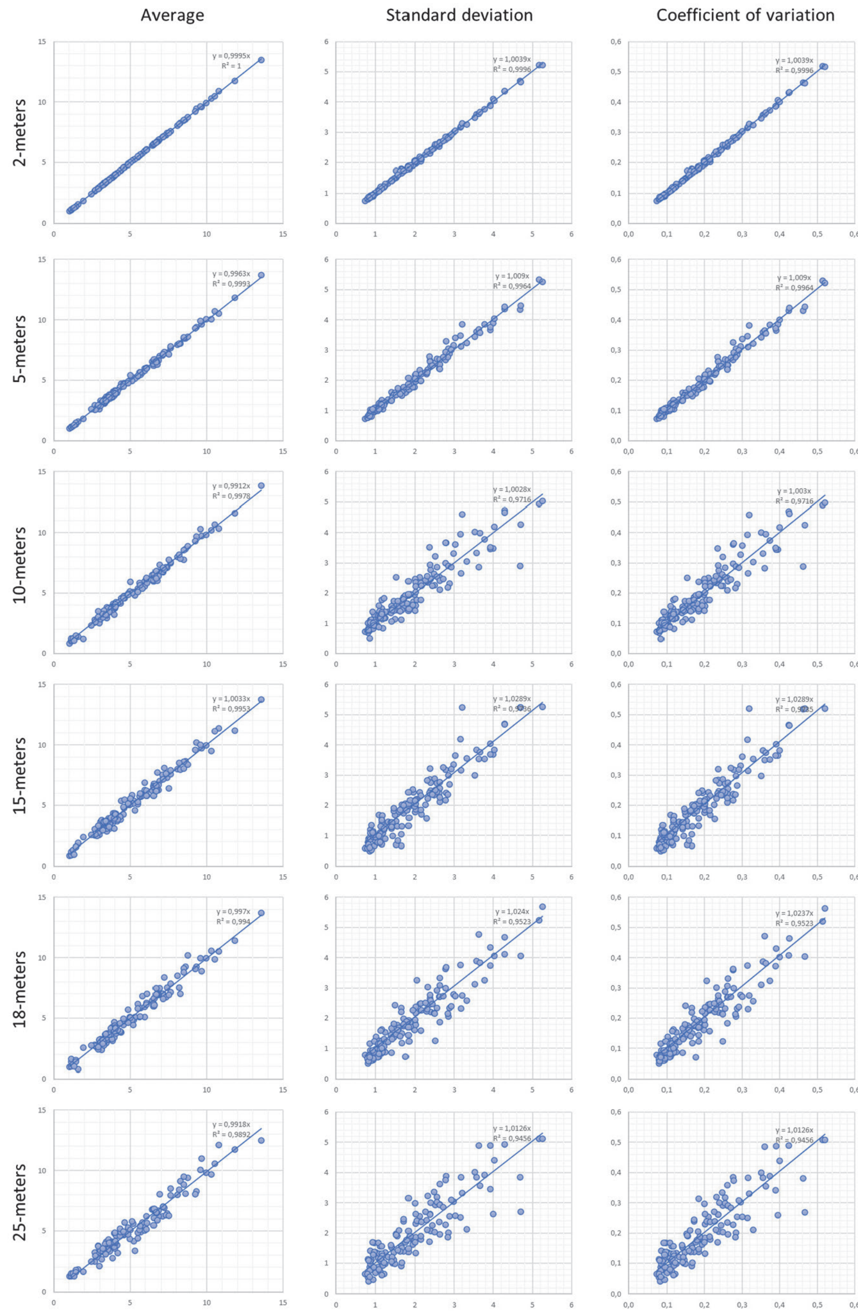

Figure 4 A linear regression analysis scatterplots

Scatterplots for the track gauge deviation average values, standard deviation, and coefficient of variation calculated for each segment with different sample distances are presented in Figure 4. The abscissa shows the measuring values calculated for each segment and benchmark 1-meter sampling distance while the ordinate shows the measuring values calculated for each segment with a larger sample distance.
As expected, an increase in the sampling distance, in relation to the benchmark 1-meter sampling distance, increases the dispersion of the sample data and therefore an error in the track gauge deviation analysis.

4 Conclusion

Decisions on track reconstruction are very often determined based on the track geometry irregularities, the key parameters in track geometry quality assessment. Track irregularities are usually measured with track-recording vehicles. Small urban railway system administrators usually do not have such sophisticated and expensive track maintenance machines, but the recording of the track geometry is carried out manually using a measuring trolley or a measuring rod. Such measurements are time-consuming and can lead to traffic disruptions.
The possibility of tram track geometry monitoring scope optimization presented in this paper was examined on the sample of the Osijek tram network gauge deviation values. The goal was to determine how to increase the monitoring frequency on the sections with a higher degree of track degradation by reducing the number of values measured along the tracks, ie by increasing the sampling distance, and still maintain the desired monitoring accuracy.
The result of the linear regression analysis shows that by increasing a sampling distance to 5 meters on a 200-meter-long segment it is possible to maintain the desired accuracy of the track gauge deviation value and therefore to optimise track gauge monitoring scope on small urban railway systems.

References

[1] UEMI Solutions: SOLUTIONS Factsheet 1.13 Tram systems, 2017.

[2] European committee for standardization: Railway applications/Track - Track Geometry Quality, Part 1: Characterisation of track geometry, EN.13848-1, 2003.

[3] Karis, T.: Correlation between Track Irregularities and Vehicle Dynamic Response Based on Measurements and Simulations, Thesis, 2018.

[4] Jovanović, S., Božović, D., Tomičić-Torlaković, M.: Railway infrastructure condition-monitoring and analysis as a basis for maintenance management, GRAĐEVINAR 66 (2014) 4, pp. 347-358, doi: https://doi.org/10.14256/JCE.959.2013, 2014.

[5] Arasteh Khouy I., Per-Olof L., Nissen A., Juntti U., Schunnesson H.: Optimisation of track geometry inspection interval. Proceedings of the Institution of Mechanical Engineers Part F Journal of Rail and Rapid Transit. 228. 546-556. 10.1177/0954409713484711, 2013.

[6] Muinde, M.: Railway track geometry inspection optimization, Dissertation, 2018.

[7] Ahac, M., Lakušić, S., Madejski, J., Haladin, I.: Application of Osijek's tram tracks geometry analysis results in their reconstruction strategy development, 5th International Conference on Road and Rail Infrastructure-CETRA 2018, doi: https://doi.org/10.5592/CO/cetra.2018.902, 2018.

[8] European committee for standardization: Railway applications/Track - Track Geometry Quality, Part 4: Measuring systems - Manual and light weight devices, EN 13848-4, 2012.

[9] TEC-1435/TET-1000 Track gauge User Manual, ver. 2.05e, GRAW, Gliwice, 2004.

[10] Liu, R., Xu, P., Sun, Z., Zou, C., Sun, Q.: Establishment of Track Quality Index Standard Recommendations for Beijing Metro, Discrete Dynamics in Nature and Society 1-9, 2015.

[11] Movaghar, M., Mohammadzadeh, S.: Intelligent index for railway track quality evaluation based on Bayesian approaches, Structure and Infrastructure Engineering, 16:7, 968-986, DOI: 10.1080/15732479.2019.1676793, 2020.

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ASSESSMENT OF TRACK AND TURNOUT CONDITION BASED ON GEOMETRY MEASUREMENT AND RAILHEAD CONDITION DATA

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Abstract

The paper presents the procedure of track and turnout geometry condition assessment, taking into account also the deterioration of the rail running surface. Track geometry measurements are made using manual tools, microprocessor-based portable instruments, and geometry cars. Methods of collecting track and turnout geometry data are discussed, and an exemplary equipment design features are presented. Maintaining and possible improvement of the technical condition of the permanent way call for regular inspections providing voluminous data requiring detailed analysis. The approach based on track line-speed dependent geometry parameters analysis is explained. Several synthetic track condition assessment coefficients are described, and analysis of the temporal trend of the track and turnout geometry change. Train operation safety is also affected by changes on the running surface of the rails. In addition to the track geometry, the significant reasons for train operation safety are the railhead wear being affected by the type of transport, traffic intensity and maximum allowable axle load. Determining the permanent way condition with the continuous design and maintenance characteristics is possible if measured on the minimum 200-300 m length with the measurement steps of ca 0.5 m. Comments on employing the Artificial Intelligence tools for track and turnout condition analysis are provided. Most of the inspection data collected using various equipment, like track and turnout geometry measurement data and video inspection information, can be analysed automatically by the dedicated software agents. Such an approach yields analysis results equivalent to the standard inspections, except that the trains and self-propelled trolleys can record data at higher speeds, railways staff could achieve.

Keywords: track condition, turnout condition, visual inspection, automatic defect detection, virtual templates

1 Introduction

Maintenance contracts are usually based on condition level nowadays. Specifying the condition requires defining the objective, transparent and reproducible parameters, tolerances and quality indices for turnout geometry. Therefore, the currently used systems should provide data that can fulfil the requirements mentioned above regarding the maintenance, safety and lifecycle of switches. Changing the approach from the time-based to the condition-based maintenance is needed to achieve effective and efficient maintenance. Such an approach requires reliable data sources, and its processing methodology yielding trustworthy support for the maintenance decisions.

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1.1 Geometrical data collection

Track and turnout geometry measurements may be carried out using the analogue manual track gauges and templates. However, such measurements require skilled staff. The measurement results are subjective, dependent on the attention paid to them, weather conditions and demand extra paperwork to store the results either in the forms or typing them into some files in the computer system. Moreover, this process takes time if it has to be done accurately. This approach focuses on the particular measurements with no way to easily compare the measurement results to determine the pace of the ongoing turnout geometry deterioration.
Digital track gauges improve the data collection process, as the data is stored in their memory which saves much time and possible data entry errors during the measurement report generation. Some gauges may also support the operators with the turnout specification saved in memory, suggesting the successive switch characteristic points at which the geometry has to be measured. The measurement data is next transferred to the PC. Digital trolleys have all advantages of the digital track gauges and make the work safer for the staff (no continuous bending, which is detrimental for the spine and knees – resulting in more frequent sick leaves, reducing the available staff productivity).
Track and turnout measurement cars (TRC) feature the next level in the diagnostic data collection's efficiency and scope. These cars travel at a speed corresponding to the trains operated on the railway lines, minimising their service disruptions. They provide the information on the track geometry under load as they are the full-sized rail cars (except for some self-propelled geometry measurement trolleys). TRCs can provide measurements of the track and turnout geometry, corrugation, permanent way bench shape, track clearance, catenary, accelerations, visual inspection, generating a considerable amount of data to be processed.

1.2 Measurement data processing

Safe train operation requires the railway lines maintenance being planned based on detailed inspections, scheduling improvements, safety, funds and workforce availability, and customer expectations. The relevant work planning can only be done by performing on-site inspections record defects, identifying issues, adding images, automatic identification of track irregularities, an inspection of rail profile, cracks, irregularities and missing components [1], [2], [3].
Data fusion is the process of directly combining raw data streams from different sensors of the same type. The data is subject to aggregation before it is subject to further processing. Property fusion requires the determination of a vector of these properties based on data from each sensor. The obtained features are initially associated with each other and then combined into one common vector - Fig. 1. The vector of connective features constructed in such a way, which characterises a given object globally, is transformed into the identity declaration domain (classes) using various fusion methods, e.g. neural networks, grouping methods – Fig. 2 [4].

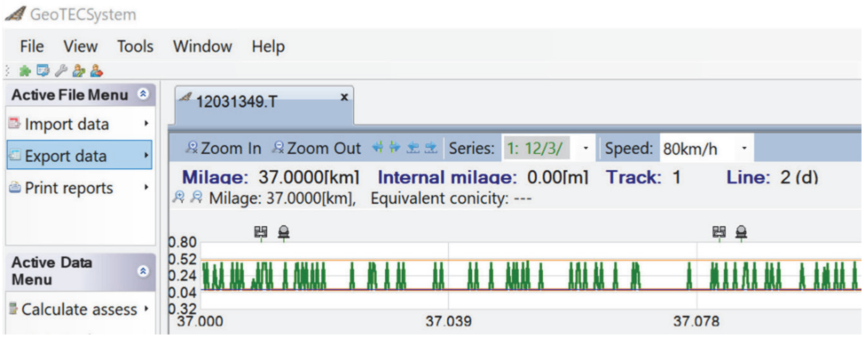

Figure 1 Equivalent conicity assessment based on railhead wear measurement for left and right rails, the back-to-back distance of wheels in a wheelset, and their tread profiles

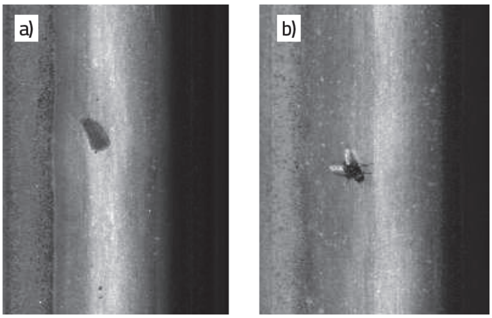

Figure 2 Automatic detection of imprints and elimination of the false ones

2 Laser measurement of track and turnout geometry

Laser measurement capability is demonstrated here by two exemplary systems: a TSP trolley with laser cameras [5] and the TMS track recording car [6]. Their capabilities are shown for the measurement of the railhead wear assessment and the turnout geometry. In essence, the measurement in their case is done by measuring the rail/turnout surface profile and comparing it to the reference profile.

2.1 Railhead wear measurement

Two laser cameras carry out the railhead wear measurement for each rail in the track, installed in a way that makes them immune to the ambient lighting conditions. The measured parameters are as follows – reported by the trolley's PC software (Fig. 3). This is an important parameter fotrack quality assessment [7-10].

- vertical consumption – understood as the difference between the nominal profile and the profile measured after the profile has been adjusted by taking into account the rail foot location,
- side wear,
- assessment of the running edge profile,
  - the angle of inclination of the side surface of the rail head α,
  - calculation of the equivalent conicity (in the office system),
  - measurement of the inclination of the rails.

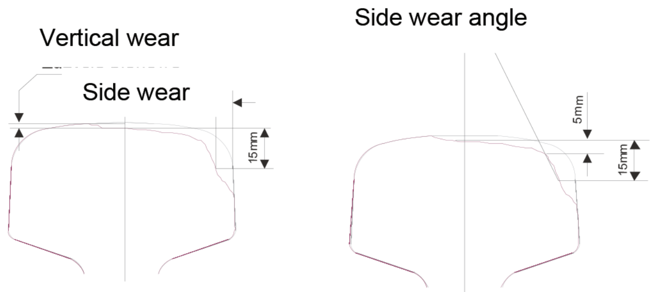

Figure 3 Definition of the main cross-section parameters of the rail

2.2 Turnout geometry measurement

An example of the TRC - TMS [6] can carry out track and turnout measurement autonomously. However, two persons have to be present on-board because of the regulations on some railways (operator and his assistant). After switching the TMS to the turnout mode, the measurements are made with the set of 8 measurement sensors (4 for each side) – Fig. 4.

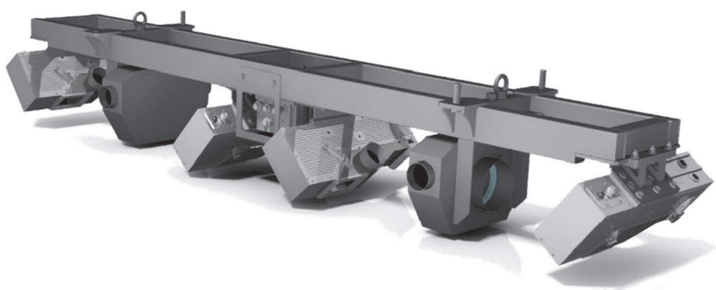

Figure 4 Laser track and turnout measurement system installed under the TMS vehicle body

The use of TMS requires careful planning as the vehicle passes some paths in the switches in one direction of its measurement ride and will complete measurement of their other legs (paths) during its nest measurement rides. A significant advantage of using the TMS is that it is treated as a train by the automation systems. Therefore, it does not require granting track possession to the measurement teams working in the track either with the manual track gauges or with the trolleys.

The off-line measurement data processing system provides the detailed turnout condition reports for the specific regions (or even country-wide) and the condition change over time, which can also be related to maintenance/repair costs. The exemplary measurement results, including the use of the reference profiles in the form of the virtual templates, are shown in Fig. 5 and Fig. 6. The automatic turnout geometry analysis includes, among others:

• Detection of turnout section (e.g., frog, blade, end)
• Detection of turnout characteristic points,
• Comparison with the mechanical templates,
• Checking measured parameters against tolerances in sections and at points.

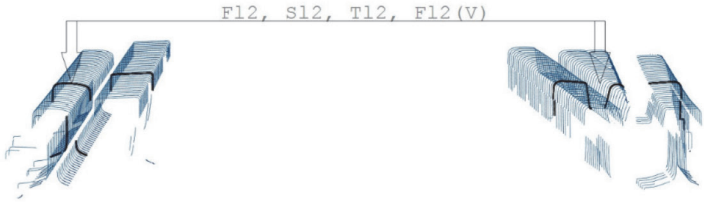

Figure 5 Optical measurement of turnout and rail cross-section

Figure 6 Virtual templates applied automatically during measurement data analysis

3 Visual inspection system

The visual inspection system allows recording the infrastructure condition during a train's passage and then carrying out analysis off-line. Such an approach is equivalent to the standard inspections, except that the train can record data at high speeds, without disturbing the typical traffic of trains or without affecting the railway line safety systems. Collecting the results by the inspection train and subsequent image processing in the office makes the inspection much more efficient and eliminates the need to maintain significant infrastructure staff [11], [12], [13]. The occurrence of particular damage types on the Polish Railway Lines (PRL) network was analysed To determine the causes of wear of railway rails. Attention was paid to three damage types that occur only on running surface of the railhead. The damages classified in the Catalogue of Rail Damages of PRL were quantitatively defined, taking into account their occurrence on chosen railway lines. The digital visual inspection (Fig. 7) system provides the possibility of saving the digital images of the rolling surfaces of both rails and rail surrounding (sleepers and ballast). Its resolution makes an assessment of the surface defects possible. For the track areas, the digital image resolution makes visual inspection possible of civil engineering objects in the track infrastructure. The main system components included the lighting subsystem for clear images of high contrast in any environment and lighting conditions, high-resolution cameras for rail surface defects recognition, and high-resolution cameras for the area surrounding the rails, covering the entire sleeper length. Data acquired by this system made automatic detection possible of defects like:

- rail surface anomalies,
- head check defect,
- rail edge anomalies,
- corrugation
- missing fastening elements,
- misplaced fastening elements,
- wheel burn on rail,
- break/discontinuity,
- periodical imprints.

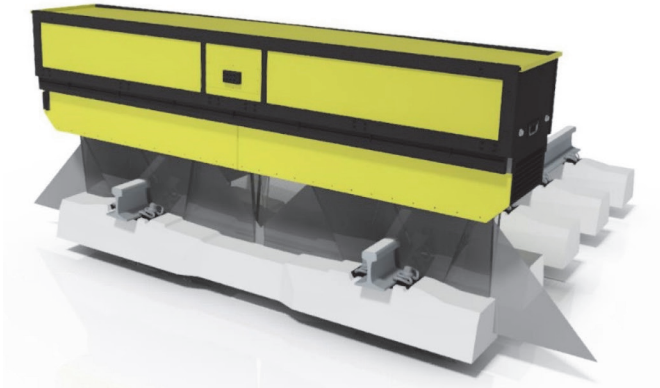

Figure 7 Track and turnouts visual inspection module

The images saved for the visual inspection needs are displayed on several monitors in real-time to enable the operators checking the infrastructure condition continuously. Detailed analysis is done in postprocessing the visual inspection data – Fig. 8, Fig. 9. Although most automatic detection algorithms generally try to minimise both false positive and false negative ratios, our approach focuses on minimising false negatives even at the cost of increasing the false positive detection rate to provide 100 % certainty that no major faults would be missed.

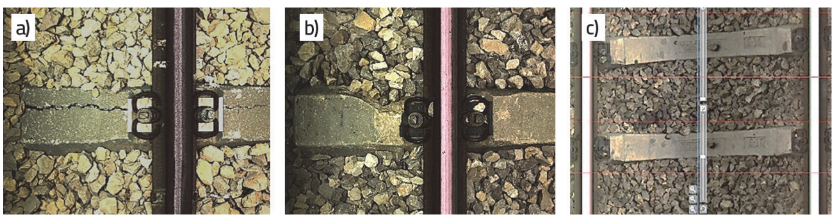

Figure 8 Examples of automatically recognised track sleeper defects: a) Sleeper cracks, b) Sleeper chipping, c) Displaced sleepers

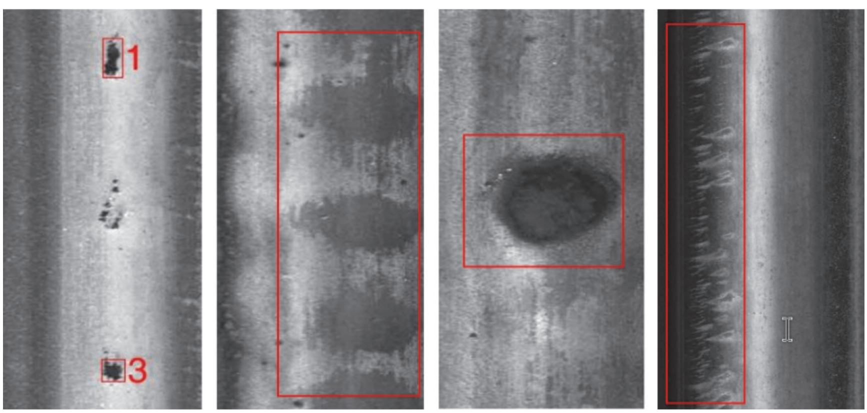

Figure 9 Example of automatic defect detection (imprint, short waviness, squat, HCH)

Track overview camera provides a vision of the track bedding and additional infrastructure elements like fasteners or fishplates. Missing or damaged fasteners detection can be done using images from those cameras. This method is robust and easily extendable since any given infrastructure element with a known location on the track can be found.
The images from the cameras will be recorded and fully synchronised with the measurement systems. Also, the selected camera's preview may be displayed on the control monitor installed on the vehicle. The recorded images can be transferred to the office system, and they can be made available in the operators' off-line software. The operator can specify the picture-taking distance increment.

4 Conclusions

Currently available digital track and turnout geometry measurement devices and systems provide a vast amount of diagnostic data that have made the predictive maintenance possible. Complex condition indices can be used now involving the measured geometrical parameters and also obtained from the automatic visual inspection data analysis.

References

[1] Bałuch, H.: Zagrożenia w nawierzchni kolejowej, Instytut Kolejnictwa Warszawa, 2017.

[2] Ahac, M., Lakušić, S.: Tram track maintenance-planning by gauge degradation modelling, Transport, 30 (2015) 4, pp. 430-436, doi: 10.3846/16484142.2015.1116464

[3] Madejski, J., Grabczyk, J.: Continuous geometry measurement for diagnostics of tracks and switches, International Conference on Switches: Switch to Delft 2002, Delft, The Netherlands, 19-22.3.2002.

[4] Sroka, R.: Fuzja danych w zastosowaniach pomiarowych, Pomiary Automatyka Kontrola, 5/6, pp. 15-19, 2002.

[5] GRAW GOLDSCHMIDT, www.graw.com/en/track-measurement/self-propelled-trolley-for-track-geometry-and-rail-wear.html

[6] GRAW GOLDSCHMIDT, Turnout Measurement System: TMS, www.graw.com/en/track-measurement/turnout-measurement-system-tms.html

[7] Jovanovic, S.: Railway track quality assessment and related decision making, 2004 IEEE International Conference on Systems, Man and Cybernetics, The Hague, Netherlands, 2004, pp. 5038-5043, doi: 10.1109/ICSMC.2004.1400992.

[8] Hakan, G.: Prediction of railway track geometry deterioration using artificial neural networks: a case study for Turkish state railways, Structure and Infrastructure Engineering, Maintenance, Management, Life-Cycle Design and Performance, 10 (2014) 5, pp. 614-626, doi: 10.1080/15732479.2012.757791.

[9] Sadeghi, J., Askarinejad, H.: Application of neural networks in evaluation of railway track quality condition, Journal of Mechanical Science and Technology, 26 (2012), pp. 113–122, doi: 10.1007/s12206-011-1016-5.

[10] Hitihamillage, L.: Tram Track Degradation Prediction - A thesis submitted in fulfilment of the requirements for the degree of Master of Engineering, Bachelor of Engineering, RMIT University, School of Engineering, College of Science, Engineering and Health, RMIT University, 2018.

[11] Madejski, J.: Potential of Multi-Agent Systems for Operation of Transportation Systems, Pomiary Automatyka Robotyka, 1 (2016), pp. 31–36, doi: 10.14313/PAR_219/31

[12] Shen, J.F., Xu, Y.D., Li, H.F.: Establishing of track quality level management system for ballastless track of high-speed railway, 22nd International Conference on Urban Transport and the Environment (UT 2016), pp. 39-50, Crete, Greece, 22-23.6.2016,

[13] Randika, K., Vithanage, W., Harrison, C., DeSilva, A.: Importance and Applications of Robotic and Autonomous Systems (RAS), Railway Maintenance Sector: A Review, Computers, 8 (2019) 56, doi:10.3390/computers8030056.

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DEVELOPMENT OF THE NEW "DIV" RAIL FASTENING SYSTEM

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Abstract

The primary role of the rail fastening system is to position and fasten the rails to sleepers and transfer the vehicle load from the rails to the track substructure. The type and characteristics of the fastening system are usually chosen depending on the required elasticity of the track, the design load, and the type of rail. The rail fastening system has also a significant effect on the emission of noise and vibration that occurs during the operation of rail vehicles, so the right choice of fastening system contributes to noise and vibration mitigation. Worldwide, there are many types of rail fastening systems, which differ in design, construction and technical characteristics. The most used rail fastening systems are W-clip, E-clip, Nabla clip, etc. Various types of fastenings are a result of effort of both independent institutions, rail equipment manufacturers and a significant number of research centres of the developed railway authorities. When optimizing the railway track, it is necessary to choose the right properties of the fastening system, which will ensure safe and reliable operation of railway vehicles with a minimum of noise and vibration emission. The characteristics of the fastening clip and the rail pad have the greatest influence on the mechanical behaviour of the fastening system. The Faculty of Civil Engineering of the University of Zagreb cooperates with DIV d.o.o., a manufacturer of railway equipment, on an R&D project "Development of the elastic fastening system DIV". With the aim of developing a new fastening system, this paper primarily analyses the properties of existing fastening systems. A meaningful evaluation of the rail fastening systems requires the understanding of the geometry, materials, mechanical properties and the utilisation of the clip and the rail pad. The new fastening system should meet the requirements defined in the standards EN 13146 and EN 13481.

Keywords: railway track, rail fastening system, rail clip, rail pad

1 Introduction

The dynamic forces acting on the structure have increased significantly due to the increase in rail vehicle speeds and the development of high-speed railways in recent decades [3]. Subsidence of individual layers and degradation of the track structure are challenges that occur on modern railway tracks. To reduce the dynamic forces and improve the track quality, an elastic fastening system is usually installed to connect the rails and sleepers. The main functions of the fastening systems are: transfer forces from the rails to the sleeper, ensuring a constant clamping force over time, invariable elastic behaviour over time and a durability of all elements, low cost and ease of installation and maintenance. Moreover, fastening system should provide: passenger comfort, damping of vibrations and shock loads caused by rail traffic, maintenance of track gauge within certain tolerances, provision of electrical insulation between rails and sleepers, torsional resistance to rail rollover, restraint against longitudinal rail displacements [4]. In the case of increased stresses in the rail due to dynamic effects caused by the railway vehicle passing over the rail, the elastic force of the fastening clip on the rail plays a vital role. It provides elastic support of the rail during vertical movement, absorbs vibrations and achieves high resistance to longitudinal movement and lateral rotation of the rail. Furthermore, it ensures constant contact between the rail and the sleeper and must be sufficient for all load cases, including wear of individual components [3]. Depending on the fastening system and customer requirements, the clamping force varies in the range of 7.5 to 12.5 kN and deflections of the clip toe between 10 and 15 mm. The clip needs to have a large deflection during the installation phase so that the clamping force is not significantly affected by variations in the thickness of the pads, insulators and rail. The rail clamping force requirement is determined by the rail size, vehicle weight and speed, the nature of the track, curve radii, temperature range and so on. According to European standards, the minimum resistance force against longitudinal movement of the rail by the fastening system is 7 kN for most mainline tracks and 9 kN for high-speed rail and heavy freight lines. This results in a nominal clamping force per clip of a minimum of 8.5 kN for mixed traffic line and 10 kN for the more severe applications [5].The main components of the elastic fastening system are therefore an elastic rail clip and an elastic rail pad.

The elastic rail clips used in the railways of the world are usually divided into two categories: the first uses a threaded nut/screw with a bolt to apply a force to the clip steel (W-14, Nabla), and the second category are the self-tensioning clip (E-2000) [6].

An elastomeric rail pad is an element of track fastening accessories that is placed under the rail at the fastening point. Its surface may be flat, plugged, or grooved. A rail pad installed between the rail and the sleeper prevents wear of the sleeper top and protects it from the effects of loading. Its effect is also manifested through the reduction of high-frequency vibrations caused by the passage of rail vehicles. The thickness of the rail pads varies from 4.5 mm to 15.0 mm. For use on 60E1 rail fastening systems, 180 mm long and 148 mm wide rail pads are used [7]. Materials from which the rail pad is most commonly made are ethylene vinyl acetate (EVA), high-density polyethylene (HDPE), thermoplastic polyurethane (TPU), High Sylodyn (HS), natural rubber (NR), etc. The materials used to make the rail pads are characterized by very high elasticity, low temperature-dependent stiffness, low dynamic stiffness, good noise damping of the track, good aging and weathering resistance, low water absorption and very good resistance to UV radiation and ozone. Some of the rail pads used with their associated stiffnesses are: Zw700a (53 MN/m), Zw900a (56 MN/m), Zw 661-6 (347 MN/m), Zw 687 (315 MN/m), Zw700 (68 MN/m) [8]. The stiffness of the 4.5 mm thick rail pad of the Nabla fastening system is 1300 MN/m, while the 9 mm thick rail pad has a stiffness of 200 MN/m [9]. The stiffness of a fastening system is critical to the long-term performance of the fastening system under repeated axle loading. Stiffness is closely related to the degree of wear to which the fastening system components are subjected, and the resulting life of the system. More elastic fastening systems tend to accelerate component wear, while stiffer fastening systems may cause problems such as rail breakage, pumping sleepers, or ballast crushing [10]. By reducing the stiffness of the fastening system, the vehicle load is transferred to a larger number of sleepers. This reduces the load on the individual sleeper and the rate of load growth and prolongs the life of both the sleeper and rails. Reducing the stiffness also lowers the rail frequency, which has a positive effect on the overall design of the track and reduces traffic noise and vibration [11].

The project development of the elastic fastening system “DIV” consists of 2 phases: industrial development of the clip and experimental development. The industrial development is divided into the following steps: development of clip models (development of 9 models and selection of the top 3), development of tools for the production of the elements of the “DIV” fastening system, internal testing of experimental series, laboratory tests, development of procedures and machines for the assembly and disassembly of the “DIV” clip, development of a trial section length (main project of the test section length 200 m - 100 m reference section with the fastening system W-14, and 100 m section with the “DIV” fastening system), and intellectual property protection. Steps of experimental development are: construction of a test section and installation of measuring equipment, testing of “DIV” elastic clip on test section and preparation for commercialization.

2 Characteristics of the W-14 and Nabla fastening systems

The most important properties of the fastening system in the process of the development of a new type are defined in [1], [2]. These standards for fastening systems for concrete sleepers consist of the following parts: determination of rail longitudinal restraint, determination of torsional resistance, determination of attenuation of impact loads, effect of repeated loading, determination of electrical resistance, effect of severe environmental conditions, determination of clamping force and uplift stiffness, in-service testing, determination of stiffness, and proof load test for pull-out resistance. W-14 and Nabla clip’s great advantage is the possibility of adjusting the clamping force on the rail-foot by tightening the nut/screw. The characteristics of the W-14 and Nabla fastening systems will be presented. Better understanding of current system characteristics leads to optimization of the new “DIV” fastening system.

2.1. W-14 fastening system

According to the technical specifications of the W-14 fastening system, the rail is fastened with an approximately 12 mm deformed W-14 clip and tightened with a fastening force of 2x9 kN. The central bending of the clip, where it makes a second stop (after the clip has deformed by about 14 mm), protects the rail from lateral rotation [12]. Structural Testing Laboratory of the Faculty of Civil Engineering of the University of Zagreb performed laboratory testing of W-14 fastening system [14]. The tests of the rail fastening system for one-piece prestressed concrete sleepers were carried out according to the requirements given in standards [1], [2]. The specified fastening system belongs to category C with a maximum design axle load of 260 kN and a minimum track curve radius of 150 m. Comparative analysis of fastening system W-14 characteristics from literature [13] and from own laboratory testing has been performed, Table 1.

Table 1 Tested parameters of the fastening system [13], [14]

Parameters	Fastening system	
	W-14 (Poland) [13]	**W-14 (Croatia)** [14]
Dynamic vertical stiffness	100,4 MN/m	109,4 MN/m
Static vertical stiffness (before cyclic loading)	86,6 MN/m	89,5 MN/m
Static static stiffness (after cyclic loading)	104,6 MN/m (switch ‹25%)	110,3 MN/m (switch ‹25%)
Longitudinal resistance (before cyclic loading)	14,0 kN	12,47 kN
Longitudinal resistance (after cyclic loading)	13,2 kN (switch ‹20%)	11,29 kN
Clamping force (before cyclic loading)	19,7 kN	16,96 kN
Clamping force (after cyclic loading)	18,6 kN (switch ‹20%)	14,92 kN (switch ‹20%)
Torsion resistance	1,27 kNm/1°	0,86 kNm/1°
Suppression of impact loads	47,30 %	

2.2 Nabla fastening system

The elastic clip Nabla is a trapezoidal elastic plate with two axes of elasticity: one perpendicular to the rail and the other parallel to the rail, called the ridge. It achieves the clamping force by tightening the screw up to a certain degree of tightening. Between the plate and the rail foot, there is an elastic plate made of plastic. Advantages of Nabla are the very good elastic properties and the always constant clamping force. Disadvantages are the impossibility of using it with the B70 sleeper and the necessity of retightening the screw (regular retightening of the screw to ensure the clamping force). The load-deflection curves of the W-14 clip and the Nabla clip are shown in Figure 1. together with fastening clip without secondary stiffness [15].

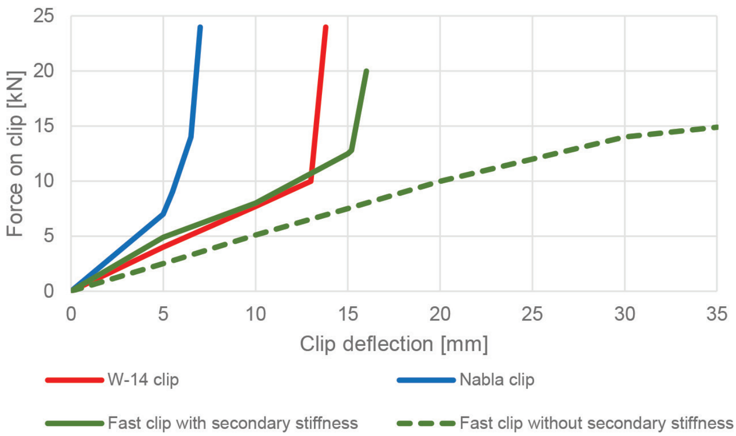

Figure 1 Figure 1. Force on clip and deflection diagram [15]

3 Research carried out on the development of the "DIV" fastening system

As part of the project to develop a new fastening system "DIV", the following tests have been carried out so far for the W-14 and Nabla fastening systems: the steel tensile testing, 3D scanning of the existing fastening systems, static stiffness test of the elastic clips, static stiffness test of the rail pads. The conducted tests of the existing fastening systems, which are necessary for the development of a new system that is currently in the modelling phase are presented in chapter below. Detailed numerical models of the new "DIV" fastening system are created. A numerical model was developed for each variant, on which two types of simulations were performed. The first simulation is the assembly of the fastening system by pushing the clip from the unmounted to the mounted position, and for this analysis the diagrams "default forced displacement of the clip - the achieved tensile force in the screw" are required. A second simulation was performed to demonstrate the resistance of the clip to the lifting of the rail, simulating the passage of a railway vehicle along the rail. All numerical models include data on the material model of the clip obtained through testing, i.e. the same material model was used as in the variants of the previous phases. Yielding point of the clip should be precisely determined.

3.1 Tensile testing of the steel

The samples were taken and shaped in accordance with requirements of HRN EN ISO 6892-1: 2016 and tested at room temperature with no deviations from the standard. Three specimens of the hardox-400 without weld, three welded hardox-400 specimens and four specimens extracted from Nabla with additional welded hardox-400 were tested. The test was performed on a Z600 universal static testing machine. The force measuring device is class 1, according to the standard HRN EN ISO 7500-1. The strain measurement of the test samples in both the elastic and plastic range was conducted using an class 1 extensometer. The obtained average tensile strength of the samples without weld is 1124.23 MPa, of the welded samples is 1184.27 MPa, and of the Nabla samples is 1178.53 MPa.

3.2 3D scanning of the existing fastening systems

3D scanning was performed using an industrial high-resolution 3D scanner ATOS – GOM. The 3D scanning of the rail fastening elements resulted in files of type “STL”. The STL file stores information about the geometry of the 3D model without colour and texture. In addition, STL files are suitable for later manipulation in a CAD tool and FEM software packages such as Abaqus software. Fastening systems over which 3D scanning has been performed are Nabla and W-14. The purpose of 3D scanning is to achieve high accuracy of numerical models of current fastening systems. Figure 2. shows laboratory setup for the purpose of the 3d scanning.

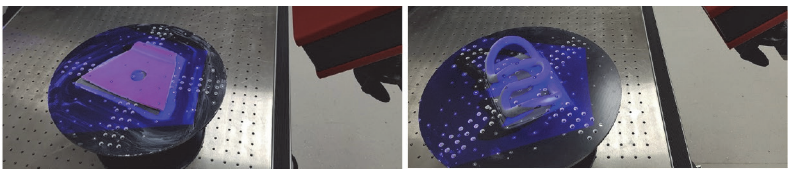

Figure 2 3D scanning Nabla clip (left); 3D scanning of W-14 clip (right)

3.3 Static stiffness testing of elastic clips (Nabla and W-14)

The test was performed on a Z600 static testing machine of 600 kN in capacity. The test was performed using a displacement control with load retention every 500 N, as shown in Figure 3. The test load measurement was performed using a 50 kN load cell, and the 3D displacement and strain field measurement was performed using the “Aramis” stereophotogrammetry system. During the test, the displacement and strain field of the surfaces of the clips were measured in the increments of the application of vertical load of 500 N. The load was applied to the samples up to the moment when the contact of the clip with the adapter is established, i.e. until the “second contact”, which is manifested during the measurement by a sudden increase in the amount of force without an increase in the deformation of the clip.

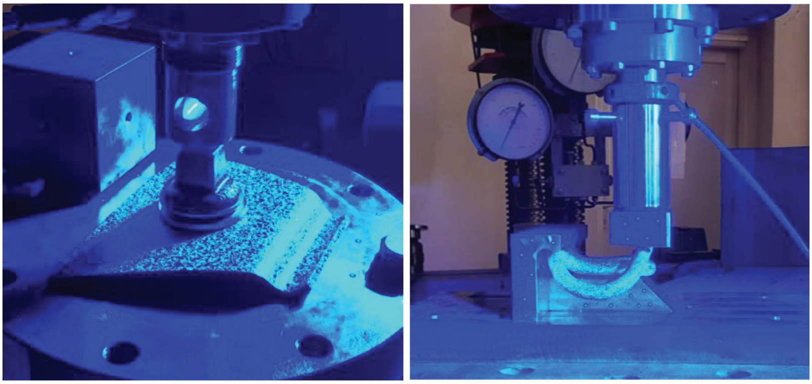

Figure 3 Laboratory testing of static clip stiffness: Nabla (left) and W-14 (right)

Diagrams of the vertical displacements of selected points of the clips as a function of the applied load are shown below in Figure 4. The same diagrams show the direction whose slope represents a certain static stiffness of the clips.

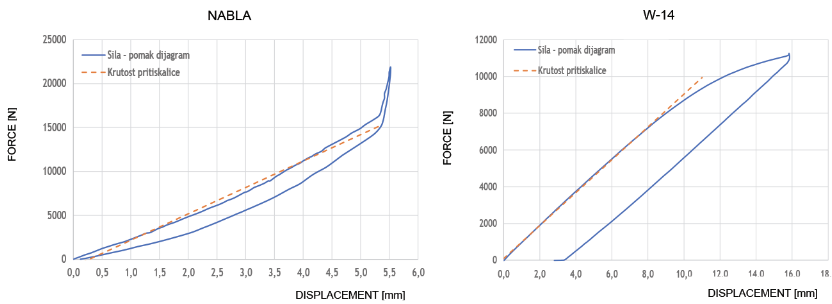

Figure 4 Vertical displacements depending on the applied load: Nabla (left) and W-14 (right)

From the linear part of the graphs shown in the previous figure, the stiffness of the clips (dashed line) is determined and amounts 3.00 kN/mm for Nabla clip and 0.90 kN/mm for W-14 clip.

Comparison between obtained static stiffness of the Nabla clip according to the model and according to Aramis system is shown in Figure 5.

In order to determine the stiffness of the Nabla clip, a simple numerical finite element model (FEM) was made, consisting of a Nabla clip, a screw made of steel and two plate components. One of the plate components is made of polymer material PA66, and the other, which represents the rail foot, is made of steel. The pressing element is accompanied by material obtained by testing on Nabla clip samples.

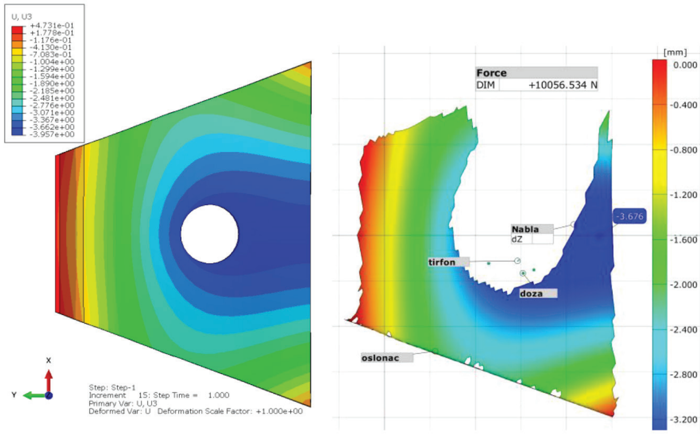

Figure 5 Results of the static Nabla clip stiffness test: according to the Abaqus model (left), and according to static test result analysed using Aramis system (right)

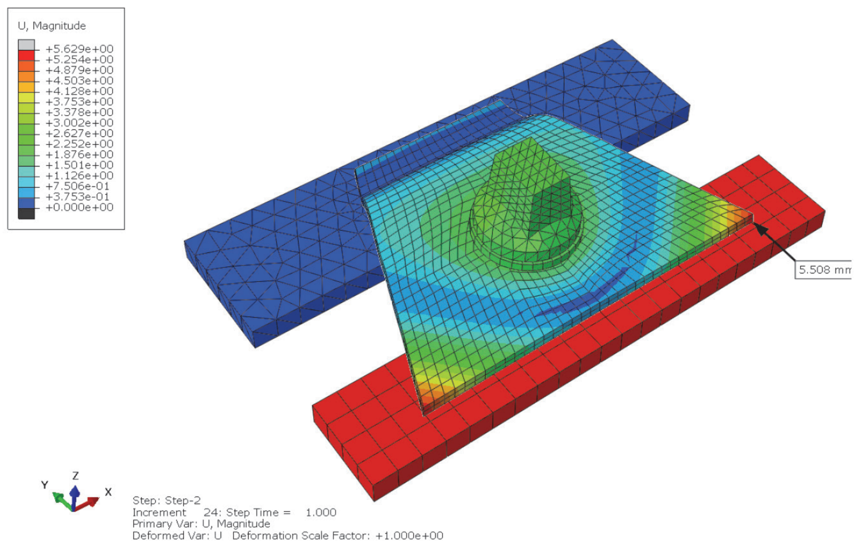

Figure 6 Strain plot of the Nabla surface due rail lifting

Figure 6. shows strain plot of the Nabla surface due rail lifting. To be able to determine the behaviour of the clip when the rail is lifted, a diagram of the relationship between the lifting force and the clip displacement (Figure 6.). On the same graph, changes in the slope of the curve and changes in the stiffness can be observed. The first such change is identified as the "second point of contact ", which represents the moment when the curvature of the clip foot in the xz plane in contact with the rail foot is completely flat (full contact phase). The second change in the slope of the curve represents the moment when the screw begins to take over the bending moment caused by the lifting force.

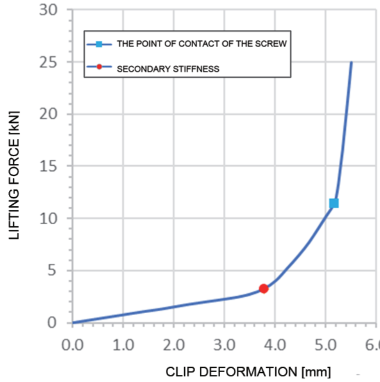

Figure 7 Nabla - Lifting force and clip deformation graph

3.4 Rail pad static stiffness testing

Static stiffness tests of rail pads were carried out at room temperature according [1]. Five samples of ZW 661-6, ZW 687, ZW 700 (used for the application of W-14 fastening system) and three MRE (used for the application of Nabla fastening system) rail pads were tested. Test conditions and load values were determined for the corresponding fastening category according to [2]. The static stiffness test of the rail pad was performed on a universal static testing machine with a load cell of 600 kN in capacity. The force measuring device is class 1. A rail pad is inserted between two 10 mm thick metal plates through which the load is distributed. The abrasive cloth was also inserted between the rail pad and the plates. The compressive load is applied to the pad using a rigid metal plate. The displacement of a rigid metal plate was measured with four inductive transducers (LVDT), 2xWA10 mm and 2xWA 50 mm. First, the maximum force F_{SPmax} is applied then the force is reduced to the value of F_{SP1}. Then two more cycles are repeated with the same load and unload at a loading rate of 120 kN / min. After that, the force is kept at the value of F_{SP1} for 30 seconds, and then the sample is loaded again with the force $F_{SP2} = 0.8*F_{SPmax}$. The static stiffness of the k_{SP} is determined based on the 4th cycle as secant stiffness at the values of the force F_{SP1} and F_{SP2} and the corresponding displacements.

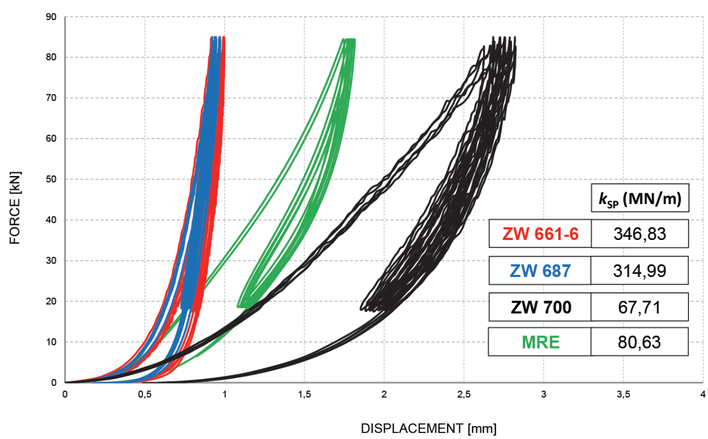

Figure 8 Measured static stiffness of the rail pads

4 Conclusion and further discussion

According to [15], a modern railway track should have a low static stiffness coefficient ρ_{static} < 100 MN/m for the fastening, as derived from the Load-Deflection curve of the pad. It should also have a compatibility of the clip and the rail pad of the system as it is derived from the combination and comparison of the load-deflection curves of the clip and the rail pad (toe-load in each case >8 kN). The secondary stiffness of the clip should reduce the "rail tilt" and keep it below a maximum limit of approximately 2 mm. The development of a new fastening system is a very comprehensive process consisting of modelling, laboratory testing, field trials, marketing and adoption by track operators. Once optimal models have been determined and the selected model has been manufactured, it is necessary to conduct laboratory tests on the properties of the new fastening system. The laboratory measurements that provide the required mechanical properties of the new fastening system were firstly conducted on an existing (W-14 and Nabla) systems. Such an approach is crucial to establish a testing methodology that can be used to define the characteristics of the newly developed system. Further investigations for the development of a new fastening system are mainly to test the dynamic properties in the low and high-frequency spectrum of the elastic clip and the rail pad. It is also necessary to investigate the static and dynamic properties of the whole assembly. After performing the dynamic tests in the high-frequency spectrum, the vibroacoustic properties of the fastening system will be better known. Finally, after conducting static and dynamic laboratory tests on the assembly and each individual element (rail clip and rail pad), a 12-month monitoring period will be carried out in the field to evaluate the properties of the new "DIV" fastening system.

References

[1] Railway applications - Track - Test methods for fastening systems - Part 1-9: Test methods for fastening system (EN 13146-4:2020)

[2] Railway applications - Track - Performance requirements for fastening systems - Part 2: Fastening systems for concrete sleepers (EN 13481-2:2012), 2012.

[3] Lakušić, S., Bartoš, D., Bajić, G.: Analiza elastičnih pritiskalica za pričvršćenje tračnica, ŽELJEZNICE 21, 7 (2008) 3, pp. 7-12

[4] Kraskiewicz, C., Oleksiewicz, W., Płudowska-Zagrajek, M., Lipko, C.: Overview of vibroacoustic isolators used in railway tracks, MATEC Web Conference, 219 (2018) October

[5] www.Precast Concrete Railway Track Systems: State-of-art Report.com, 04.03.2021.

[6] Mundrey, J.S., Wingler, F.A.: "Rail to Sleeper Fastening Systems, pp. 1–41, 2016.

[7] Sol-Sánchez, M., Moreno-Navarro, F., Rubio-Gámez, M.C.: The use of deconstructed tires as elastic elements in railway tracks, Materials, 7 (2014) 8. pp. 5903–5919

[8] Hasan, N.: Rail Pad Stiffness and Classification System, J. Transp. Eng. Part A Syst., 145 (2019) 5

[9] Thompson, D.: Railway Noise And Vibration, 2009.

[10] Romero, M.J.G., Edwards, J.R., Barkan,C.P.I., Wilson, B., Mediavilla, J.: Advancements in Fastening System Design for, 618 (2010), pp. 1–24

[11] Lakušić, S., Haladin, I., Ahac, M.: The Effect of Rail Fastening System Modifications on Tram Traffic Noise and Vibration, Shock Vib., 2016 (2016), pp. 1–15

[12] Highly elastic rail fastening for high speed and conventional rail – the established solution for ballast track with concrete sleepers–system W 14, Vossloh Fasten. Syst. Inc., 2019.

[13] Chudyba, L.: Fastening systems to concrete sleepers – comparison of the operating characteristics of fastening systems: SB and W14, Transp. Overv., 6 (2017), pp. 5–9

[14] Izvješće o ispitivanju kolosiječnog pričvrsnog pribora za betonske pragove, pp. 1–31, 2017, University of Zagreb, Faculty of Civil Engineering, Department of Engineering Mechanics, Structural testing laboratory.

[15] Giannakos, K.: Secondary Stiffness of Fastening Clips: Influence on the Behavior of the Railway Track Panel, Mater. Infrastructures 2, 5B (2014) January, pp. 115–129

STRAY CURRENT MEASUREMENT AT THE TRAMWAY INFRASTRUCTURE IN OSTRAVA, CZECH REPUBLIC

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Abstract

In most transit systems, rails are used as return conductors for the current from the vehicle to the electrical substations. If the rails are not fully insulated from the ground, some of this current would leak and become stray current, causing stray current corrosion on the rails and metal objects (such as pipelines) in the immediate area. It is very difficult to measure stray current directly, but stray currents can be calculated by measuring other parameters. Stray currents were measured on a 1.3 km section of tramway infrastructure in Ostrava. The potential between rail and earth was measured on the basis of the standard EN 50122-2, where two reference electrodes were placed at an appropriate distance from the tram track at three measuring points in the ground - the first point was located at the beginning of the section, the second in the half of the section and the last at the end of the section. Rail currents were measured at two measurement points - the first point at the beginning of the section and the second point at the end of the section. Using the rail-to-earth potential and the equation from the standard EN 50122-2:2011, the rail-to-earth conductance per length was calculated. The conductance per length was also calculated using Ohm's law, where the current difference is a difference between two measurement points. Since the results obtained using the standard and Ohm's law did not agree, a detailed analysis of the tram section was performed and electrical drainage was found. The drainage represents an electrical connection of the protected metal structure in the area of the tram track by a cable with stray current source. Through the drainage, the stray currents are directly returned to the rail. In this measurement section, the drainage has influenced the current difference between the measurement points - without drainage, this difference would be much smaller.

Keywords: *tram track, stray current, rail-to-earth potential, rail-to-earth conductance, rail current*

1 Introduction

In most electrified rail traction systems, the catenary system is used as the conductor of current from the electrical substation to the vehicle and the rails are usually used as the conductor of current back to the substation, which means that the catenary system is positive with respect to the rails [1,2]. Because it is very difficult to completely insulate the rail, and because of the finite longitudinal electrical resistance of the rails, some of the return current leaks from the rail and becomes stray current, which causes stray current corrosion on the rails [1,3-5]. Current leaks because it finds another, less resistant path to return to the source, such as metal pipelines near the rail infrastructure. The current flows through the metal pipe-

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line until it reaches the vicinity of the electrical substation. Then it leaves the pipeline and flows through the ground back to the rail (Figure 1). According to the authors [6], in a DC traction system, 5% of the current flowing through the rail becomes stray current. Stray current corrosion occurs at any point on a metal object (rail, metal pipeline) where the current leaves the object and enters the electrolyte (soil, concrete, etc.) [7].

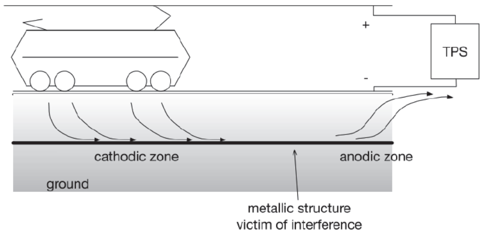

Figure 1 Stray current path in electrified tramway system [6]

The value of the stray current depends on the resistance between rail and ground. The higher the resistance between rail and earth, the less current flows from the rail. The resistance between rail and ground is determined by the insulation of the rail, the type of sleepers, the type of fastening system, and the quality of the ballast for ballasted tracks or the electrical resistance of the concrete layer for slab tracks. Methods for reducing stray current can be divided into two groups [8]:

• Increasing the resistance of the rail-to-ground leakage path,
• Decreasing the electrical resistance of the return path.

According to the standard [9], if following values of the conductance per length G'_{RE} and average rail to ground potential U_{RE} are not exceeded during the system lifetime, further investigations does not need to be performed:

• $G'_{RE} \leq 0.5$ S/km per track and $U'_{RE} \leq +5$ V for open formation
• $G'_{RE} \leq 2.5$ S/km per track and $U'_{RE} \leq +1$ V for closed formation.

2 Measurements at tram track infrastructure in Ostrava, Czech Republic

Tram track in Ostrava is ballasted tram track. Rails are fastened to the concrete sleepers on 1 m distance and laid on crush stone ballast. Since tram track in Ostrava is open formation, it was very easy to connect measurement equipment to the rails. In Ostrava tram track network, the positive pole is located on the rail, while the negative pole is at the catenary system (figure 2) [1].

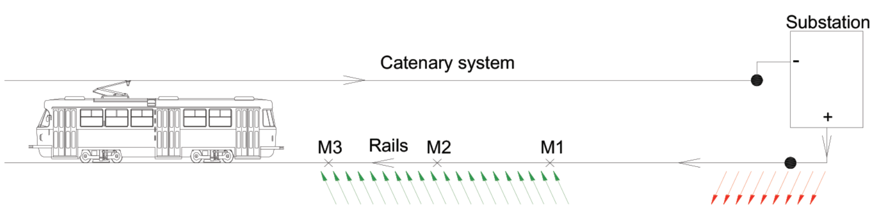

Figure 2 Current distribution at tram track infrastructure in Ostrava

It is very difficult to measure the stray current directly, but by measuring other parameters, the stray current on a section of rail infrastructure can be calculated. In the case of the tram infrastructure in Ostrava, measurements were made on a 1.3 km single-track section with relatively little traffic in the non-urban area at three measurement points - M1, M2 and M3 (Figure 3).

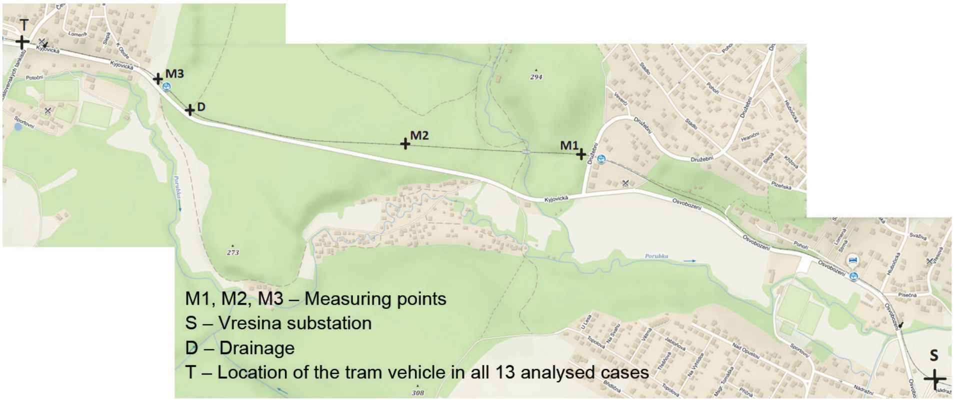

Figure 3 Map of the measuring segment with all necessary points

During the measurements, one electrical substation was shut down, so this section was supplied only by the Vresina substation. This substation is located 1.6 km east of measurement point 1 (marked with the letter S in Figure 3). All measurements were taken continuously and simultaneously for two and a half hours.

2.1 Rail-to-earth potential

At all three measurement points (M1, M2, and M3), the rail-to-earth potential was measured using two copper-copper sulphate reference electrodes as described in the standard EN 50122-2 (Figure 4) [9]. One electrode was placed in the ground at a distance of 7 m from the rail, the other at a distance of 40 m from the rail.

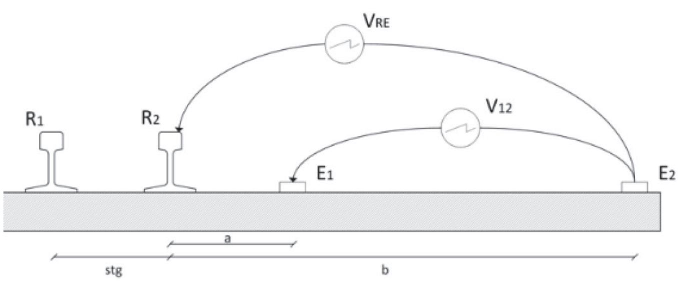

Figure 4 Measurement of the rail-to-earth potential [10]

At all three measurement points, the electrical resistance of the ground was also measured using the Wenner method. In the Wenner test, four electrodes are inserted into the ground at equal distances. The two outer electrodes inject current into the soil and the two inner electrodes measure the voltage. The ground resistance is calculated using Ohm's law [11].

2.2 Rail current

At the first and third measurement points (M1 and M3), the rail current was measured using the software on the computer. By measuring the rail current at these two points, it was possible to calculate the difference between measured values in MM1 and MM3. This difference should represent the stray current.

3 Results analysis

After the measurement was completed, the data were transferred to the computer and 10 different cases were selected for detailed analysis. In all cases, the tram vehicle passed through the measurement section or it approached the measurement section (Figure 3). Average values were calculated for each measured parameter in all 10 cases. The results are presented in Table 1, where:

- I_1[A] and I_3 [A] represent average current at the measuring point 1 and 3,
- U_{re1}, U_{re2}, U_{re3} [V] represent average value of the rail-to-earth potential measured at all measuring points,
- G_{re1}, G_{re1}, G_{re1} [S/km] represent calculated value of the rail-to-earth conductance based on the equation given in standard,
- G_{re} [S/km] represent average value of the rail to earth conductance for the whole measuring segment calculated using Ohm's law.

Table 1 Average values of the measuring results

Number	Measurements results Based on standard Based on Ohm's law					Rail-to-earth conductance [S/km]			
	I1 [A]	I3 [A]	Ure1 [V]	Ure2 [V]	Ure3 [V]	G're 1	G're 2	G're 3	G're
1	256	291	3.5	5.4	8.9	0.077	0.418	0.561	4.424
2	248	286	4.0	5.9	9.3	0.082	0.375	0.564	4.474
3	458	536	6.6	10.2	16.5	0.075	0.377	0.540	5.173
4	323	369	3.5	6.0	10.3	0.080	0.376	0.547	5.136
5	349	398	3.9	6.6	11.3	0.077	0.368	0.554	5.021
6	251	294	4.4	6.3	9.8	0.075	0.407	0.540	4.684
7	461	528	7.8	11.5	18.0	0.063	0.398	0.535	4.062
8	439	507	4.3	7.7	13.7	0.063	0.390	0.542	5.903
9	459	531	4.6	8.1	14.3	0.067	0.385	0.547	5.925
10	570	666	6.1	10.6	18.5	0.059	0.393	0.542	6.100

3.1. Rail to earth conductance using standard HRN EN 50122-2:2011

Rail-to-earth conductance was calculated using the equation (1):

$$G'_{RE}\left[S/km\right]=\frac{m_{sr}\cdot\pi\cdot 2000}{\rho_E\cdot\left[ln\cdot\left(b\cdot\left(b+s_{tg}\right)\right)-ln\cdot\left(a\cdot\left(a+s_{tg}\right)\right)\right]} \quad (1)$$

Where

m_{sr} – stray current transfer ration
ρ_E – electrical resistance of the soil
a – distance from the rail R2 and reference electrode E1
b – distance from the reference electrode E1 and E2
s_{tg} – tram track width.

The rail potential gradient DU_{1-2} was plotted as a function of the rail potential DU_{RE}. The slope of the linear regression of this function represents stray current transfer ratio (figure 5).

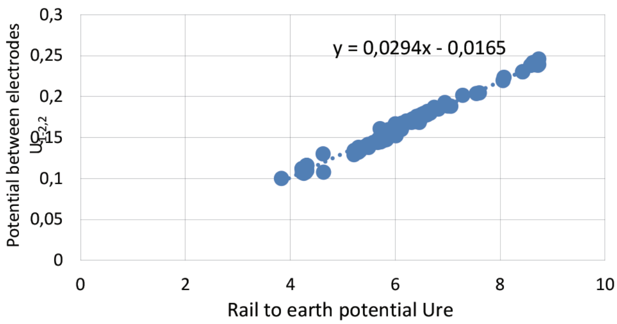

Figure 5 Example of the linear regression, stray current transfer ration for the measuring point 2, in case number 2; m_{sr} = 0.294

The HRN EN 50122-2:2011 standard [9] specifies maximum values for rail-to-earth conductance and rail-to-earth potential at which the structure is not at risk from stray currents. In the case of open construction of railway or tramway tracks, such as the construction of tramway tracks in Ostrava, the maximum rail-to- earth conductance is 0.5 S/km and the maximum rail-to- earth potential is +5 V [9]. As can be seen from Table 1, the maximum permissible values of the rail-to-earth conductance are not exceeded, but the potential is higher than the maximum permissible.

3.2.Rail to earth resistance using measured current values and Ohm's law

Based on the current difference between two measuring points and rail-to-earth potential between rail-to- earth, the conductance between rail and earth can be calculated using Ohm's law, equation (2):

$$G'_{RE}=\frac{\Delta I}{\Delta U\cdot l} \quad (2)$$

Where DU [V] represents average value of the rail-to-earth potential, DI [A] current difference between first and third measuring point and l [km] section length.

4 Discussion

The current value at measurement point 3 is higher than at measurement point 1, from which it can be concluded that current flows back into the rail at this measurement point. The results of the rail-to-earth potential also show that current flows back into the rail. At the part of the rail infrastructure where current flows back into the rail, the rail-to-earth potential should be negative. Since in these measurements the rails were considered negative and the electrodes were considered positive, the positive values of the rail potential mean that the rails are negative.
The values of rail-to- earth conductance calculated according to the equation given in the standard are different at each location. The average value at the first measurement point is 0.072 S/km, at the second measurement point 0.399 S/km and at the third measurement point 0.564 S/km. The values of the rail-to-earth resistance determined according to Ohm's law are significantly higher than the values determined according to the standard. The average value is 4.059 S/km. Due to the large difference between the rail-to-earth resistances, a detailed analysis of the tram track section was carried out and electrical drainage was found. The electrical drainage is used to protect metal structures located near the railway infrastructure from stray current. Drainage represents an electrical connection of the protected metal structure (e.g. pipeline) through a cable with stray current source [12]. Drainage bonding is based on a metallic connection between the metal structure and the rail. In this way, stray currents can flow directly back into the rail [2]. Since in this measurement segment a part of the stray current has flowed back into the rail through the electrical drainage (marked with the letter D in Figure 3), the current in measurement point 3 is significantly higher than without drainage.
As can be seen from Table 1, the values of the rail to earth conductance at measurement points 1 and 2 are less than the maximum permissible values, so that no further investigations or measures to reduce the stray current are necessary at this section.
Using Ohm's law, the stray current can be calculated, equation (3):

$$I_S = U_{RE} \cdot G_{RE} \tag{3}$$

Where:
U_{RE} [V] – rail potential, calculated using the equation given in standard EN 50122-2:2011
G_{RE} [S/m] – rail-to-earth conductance.

If the value of the stray current is known, the loss of material from the rail in a given period of time can be estimated using Faraday's law, equation (4) [13][14]:

$$m = k \cdot I_S \cdot t \tag{4}$$

Where:
m [kg/km] – weight of the material lost
k [kg/Ayear] – electrochemical equivalent (for steel is 9.1 kg/Ayear),
I_S [A/km] – average value of stray current
t [year] – time.

Stay current can be calculated using Ohm's law and the equations given in the HRN EN 50122-2-2011 standard. However, for this calculation, a continuous measurement of the potential between rail and earth must be carried out over a period of 24 hours.

5 Conclusions

In this paper, two stray current measurement methods are described. The current in the rail was measured at two measurement points. The difference between the measured currents at the two points should represent the values of stray currents. This measurement can be made in cases where tram traffic is relatively sparse and the measured section is supplied by only one electrical substation. In cases where the traffic load is higher, the measurement should be performed with a measurement vehicle while the tram traffic is stationary. In this way, the values of the current in the rails can be determined in situations where the vehicle accelerates, decelerates and travels at the same speed. After the measurement, electrical drainage was detected, so the current difference does not represent the exact value of the stray current in this section. The author's next step is to make the same measurement but with the disconected drainage. In this way, the influence of the drainage can be determined. Many stray current monitoring systems are based on continuous potential measurement between rail and earth. If the potential between rail and earth has changed, it means that the value of the electrical resistance between rail and earth has also changed, resulting in stray current leakage. Measuring the potential between rail and earth with a reference electrode is difficult to perform in urban areas because of the difficulty of placing the electrode due to asphalt surfaces.
The results of field measurements of stray currents are sensitive to many external influences, so it is very difficult to determine the actual values of stray currents. Stray currents must be considered in the design of new track systems. Good electrical insulation of the rails and rail fastening must be ensured so that the value of the resistance between rail and earth corresponds to the value specified in the standard.

Reference

[1] Kolář, V., Hrbáč, R.: Measurement of ground currents leaking from DC electric traction, 15th International Scientific Conference on Electric Power Engineering, pp. 613–617, Brno, Czech Republic, 12-14 May 2014, DOI: 10.1109/EPE.2014.6839423

[2] Transport for NSW: Guide: Electrolysis from stray DC current, version 1, 2014.

[3] Peelen, W.H.A., Neeft, E.A.C., Leegwater, G., Van Kanten-Roos, W., Courage, W.M.G.: Monitoring DC stray current interference of steel sheet pile structures in railway environment, HERON 56 (2011) 3, pp. 107-122

[4] Ogunsola, A., Sandrolini, L., Mariscotti, A.: Evaluation of stray current from a DC-electrified railway with integrated electric-electromechanical modeling and traffic simulation, IEEE Transactions on Industry Applications 51 (2015) 6, pp. 5431-5441, DOI: 10.1109/TIA.2015.2429642

[5] Dolara, A., Foiadelli, F., Leva, S.: Stray current effects mitigation in subway tunnels, IEEE Transactions on Power Delivery, 27 (2012) 4, pp. 2304–2311, DOI: 10.1109/TPWRD.2012.2203829

[6] Zaboli, A., Vahidi, B., Yousefi, S., Hosseini Biyouki, M.M.: Effect of control methods on calculation of stray current and rail potential in DC-electrified railway systems, 4 th International Conference on Recent Advances in Railway Engineering, Tehran, Iran, 17-18 May 2015

[7] Charalambous, C.A., Cotton, I., Aylott, P.: „Modeling for preliminary stray current design assessments: The effect of crosstrack regeneration supply, IEEE Transactions on Power Delivery, 28 (2013) 3, pp. 1899–1908, DOI: 10.1109/TPWRD.2013.2259849

[8] Memon, S., Clarner, G., Fromme, P.: Stray Current Mitigation and Collection Techniques Adopted by a DC Transit Agency and Its Effectiveness in Controlling Stray Currents, International Conference on Transportation and Development, pp. 640–650, Houston, Texas, June 26–29, 2016

[9] EN 50122-2:2010: Railway applications -- Fixed installations -- Electrical safety, earthing and the return circuit -- Part 2: Provisions against the effects of stray currents caused by d.c. traction systems

[10] Bongiorno, J., Mariscotti, A.: Track insulation verification and measurement, MATEC Web of Conferences, 2018

[11] Manual: Performing a Wenner Soil Resistivity Test with the AEMC ® Model 6472, 2016.

[12] Riskin, J., Khentov, A.: Electrocorrosion and Protection of Metals, 2009, DOI: https://doi.org/10.1016/B978-0-444-53295-4.X0001-4

[13] Winston, R.R., Herbert, U.H.: Corrosion and corrosion control, fourth edition, Ottawa, 2008.

[14] Charalambous, A.C., Demetriou, A., Kokkinos, N.: Impact of Photovoltaic-Oriented DC Stray Current Corrosion on Large-Scale Solar Farms' Grounding and Third-Party Infrastructure: Modeling and Assessment IEEE Transactions on industry applications, 51 (2015) 6

VEHICLE DESIGN – INFLUENCE ON OPERATIONAL QUALITY

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Abstract

The design of rail vehicles, in particular the boarding area and the interior, has a significant influence on the passenger exchange time. The vehicle design is overlaid with passenger-specific characteristics such as age, mobility restrictions and, in particular, baggage. Poorly designed vehicle layouts mean that some of the seats cannot be used and the degree of capacity utilization achievable and thus customer comfort are noticeably reduced. In particular, however, poorly designed vehicles significantly prolong passenger exchange times, which leads to frequent delays and thus to a decline in operating quality. Based on almost twenty years of intensive research in the field of passenger behaviour in passenger trains, the technical paper will show clear approaches to solutions, how a good design of rail vehicles can increase the degree of utilisation and at the same time the customer comfort, how the passenger exchange time and thus the dwell time can be significantly reduced and thus the operational quality can be significantly increased in addition to other important advantages.

Keywords: efficient rail vehicles, interiors simulation, occupancy rate, dwell time

1 Introduction

The design and development of passenger coaches often follows the principle of seat maximisation, with the aim of transporting as many passengers as possible and thus increasing efficiency and economy. However, the intensive scientific investigation of passenger wishes and needs as well as actual passenger behaviour under real conditions prove that instead of the expected increase in efficiency, in reality there is a loss of efficiency! In addition to a decline in passenger satisfaction, in reality the achievable seat occupancy rate even decreases and the passenger changeover time increases noticeably in the opposite direction. This in turn leads to more delays, which have to be reduced with higher energy consumption, and to a decrease in operational quality.

2 Methods

Based on the twenty years of know-how which is based on extensive investigations and data collection in the field of passenger behaviour in rail vehicles (observations of the behaviour of around 300,000 passengers in about 100 different types of vehicles, surveys of about 50,000 passengers and analysis of about 20,000 passengers during passenger change over), the simulation software TrainOptimizer® at www.TrainOptimizer.com was developed by TU-Vienna and netwiss. By using TrainOptimizer® you can determine with just a few clicks the most efficient layout variants in terms of best possible baggage stowage, highest possible seat occupancy and shortest possible passenger change over times, which has also beein used for this paper.

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3 Passenger behaviour versus operational quality

In two situations during a train journey the interaction of passengers with the existing vehicle layout has a significant impact on the quality of operation. This concerns boarding and alighting as well as baggage storage during the journey. Both influences are directly related to each other.

3.1 Luggage storage

Two factors have a significant impact on baggage accommodation. On the one hand, sufficient capacity must be available for the storage of baggage, on the other hand, the passenger needs must be considered extensively with regard to baggage storage, which, if disregarded, will lead to negative behaviour from an operational point of view. Passenger needs are simple and understandable, but can become complex challenges when designing vehicles. The two main needs are:

- Passengers do not want to lift larger pieces of luggage
- Passengers want a visual contact with their luggage

3.1.1 Lifting and manipulating luggage

The primary distinction to be made is which luggage is to be lifted. Smaller and lighter items of luggage are more likely to be lifted than larger and heavier items. Parallel to the basic willingness to lift luggage, a distinction must be made as to the height to which luggage is lifted. In this respect, readiness to lift is differentiated into two heights, one to a height of about one metre, which is used for luggage racks, and the other to the height of an overhead rack, usually about 1.8 metres.

Furthermore, the willingness to manipulate luggage must also be considered. This refers to whether passengers are willing to tilt or turn luggage. This is particularly important for all those stowage spaces into which baggage must be “threaded”, such as under seats or between seat backrests when seat spacing is tight. In general, passengers do not wish to manipulate their luggage for accommodation purposes and in practice do not do so. Trolleys transported in an upright position on two or four wheels must be parked in an upright position. Travel bags should be stored in a horizontal position if possible, ideally at a height of approx. one meter, which corresponds to the middle compartment in luggage racks. Smaller or medium-sized trolleys, which are more willing to be lifted, are often stored lying down in luggage racks, for example.

3.1.2 Visual contact with luggage

For about 90 % of passengers it is important for reasons of subjective security to have their luggage in view during the journey. Approximately 75 % of passengers are also explicitly prepared to place their luggage disturbingly (e.g. on or in front of seats or in the aisle) in order to establish visual contact.

3.1.3 Storage space dimensioning

In addition to the above-mentioned principles, it is also essential to provide sufficient luggage storage capacity. For this purpose, precise knowledge of the average luggage volume in the intended area of use of the vehicles is important. Furthermore, baggage must always be viewed in three dimensions. In practice, the volume of baggage is often taken as a basis, but this corresponds to a one-dimensional view. All volumes are summed up to form a total volume for luggage storage in the vehicle, which makes large luggage storage capacities seem likely. In practice, many of these areas cannot be used at all, as the dimensions of the luggage are larger than the respective areas!

3.2 Effects of insufficiently dimensioned baggage racks

Failure to comply with the above-mentioned requirements for baggage storage means that travellers are either unable to store their baggage at all because there is too little storage space available in practice, or they do not make sufficient use of the available shelves because they do not meet the basic needs of visual contact or avoidance of lifting operations. This leads, for example, to overhead shelves remaining partially unused and yet luggage not being stowed properly. Pieces of luggage that cannot be stowed are placed close to the passengers on or in front of seats or in the aisle. Non-stowable baggage results in seats being blocked. On average, two to three pieces of luggage that have not been properly stowed will result in the effective loss of a seat.

3.3 Passenger exchange

Passenger exchange is a complex interaction between passenger characteristics and the overall vehicle layout. Passenger-specific influencing factors are age and gender, any physical restrictions and the luggage carried, which in turn depends on the chosen purpose of the journey. The vehicle layout gives rise to three main areas with different influences. These are the entrance door, the entrance area, which can also serve as a catchment area, especially in local traffic, and the entire interior, which essentially corresponds to the seating area. At the entrance door, the door width, the gap between platform and vehicle and the number of steps have a significant influence. The design of the boarding area determines how well the passengers can continue into the seating area and how many passengers can remain in it in case of a backlog so that the train can still depart.
There are several influencing variables in the interior. The stowability of the baggage has a significant influence. As described above, pieces of luggage that cannot be stowed are sometimes parked in the aisle area, where they block the flow of passengers. Another influence is the simplicity of luggage storage. Ideally, if passengers can deposit their luggage “in passing” and then go straight to the nearest seat, the flow of passengers is faster than if passengers have to manipulate their luggage several times for storage. Especially when luggage has to be lifted to be stored in the overhead storage or the distance between two seat backrests is too short and the luggage can only be stored by tilting, if at all, the passenger flow slows down considerably.
Furthermore, the aisle width and possible alternative spaces have an important influence on passenger flow. The width of the aisle is important for the ease of movement with luggage, as well as when people are busy putting down luggage and others may pass by.

4 Design principles in vehicles

In order to achieve a high degree of seat occupancy and at the same time the shortest possible dwell time, the following principles must be observed:

4.1 Baggage racks

The baggage racks shall comply with the principles of avoidance of lifting of large pieces and visual contact. When dimensioning baggage racks, reference may be made to the actual willingness of different passengers to lift baggage for reasons of efficiency. Smaller and medium-sized pieces of luggage, which tend to be lighter, are also placed in the overhead rack by the passengers to a larger extent, larger pieces of luggage only to a small extent of approx. 20 %. This means that the calculation may well be based on overhead storage, but only to the extent that the passengers are willing to use it and not per se for all luggage.

Luggage racks must be well distributed in the seating area. This applies in particular to luggage racks and the space between the seats. A good distribution leads to appropriate use, as most travellers can see their luggage. At the same time, distribution also means that luggage can be more easily stored by passengers, thus allowing passengers to take seats more quickly. Luggage racks must not be placed in the boarding area of vehicles, as these are only used up to approx. 30 % for safety reasons. The same applies to luggage racks in the interior of the vehicle, which are located immediately after the entrance to the seating area. If they are used, the luggage being stowed very close to the boarding door, what leads to a backlog of boarding passengers. In addition to the arrangement of the luggage racks, it is essential to know the exact quantity and type of luggage to be expected. The type of baggage or the appropriate mix determines the required dimensions of the baggage racks. Storage racks that are too narrow by only a few centimetres often mean that certain pieces of luggage cannot be stored at all or only in such a way that there is no further useable free space, which makes the racks inefficient.

4.2 Overall vehicle concept

The entire vehicle concept has a significant influence on the passenger change over time. This already starts with the car bodies. Shorter and thus wider car bodies have the advantage of allowing wider aisles, in addition to the advantage of up to 50 % lower tare weight per seat and the resulting large energy saving effect. Aisles with a width of more than 60cm allow up to 25 % shorter passenger change over times than those with a width of 50cm.
Another important factor is the arrangement of the doors. The classic arrangement at the two ends of the car means that an average of 50 % of the passengers per car have to enter through one door walk through the same interior. Since the boarding time in the respective car interiors essentially follows a square parabola, a higher number of passengers passing through a cross-section leads to a disproportionate increase in passenger changeover time. If, on the other hand, the doors are arranged in such a way that the passenger flow can be divided up when passengers enter the boarding area, the passenger change over time can be significantly reduced. On the one hand, the number of people entering the respective passenger compartment through a cross section is halved if the doors are well located, which leads to a noticeable reduction of the boarding time. On the other hand, the division of passengers also reduces tailback effects from the seating areas. In the seating area, it must be considered to ensure good division and correct and adequate planning of the luggage racks. In addition, well-distributed spreading spaces should be created. This can be done by ensuring that tables in vis-á-vis seating groups do not reach as far as the aisle, but are approx. 10 to 15 cm shorter. Likewise, luggage racks should be moved away from the aisle, this creates equally good alternative space.

5 Example layout comparison

In the following, two layouts, which are deliberately similar in structure, are compared with each other in order to illustrate the effects that the correct consideration of luggage racks has on the achievable seat occupancy rate and passenger changeover time. If the overall concept is fundamentally revised, for example by shortening the car body and changing the arrangement of the entrance doors, even more significant differences can be seen.
The two layouts are fictitious examples and are not in actual use in the form shown. In both cases they are classic passenger coaches, in layout 1 with 100 seats and, except for two small racks, mainly overhead racks for luggage storage. In layout 2, only 88 seats are available, and there are more suitable baggage storage options to meet passenger requirements (see Figure 1 and Figure 2). The baggage racks have three compartments in all cases, measured from below at a height of 80 - 40 - 40 cm, with the overhead rack above.

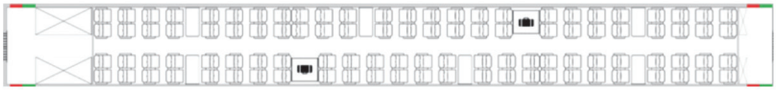

Figure 1 Example layout 1: netwiss - Layout creation in Beta - TrainOptimizer

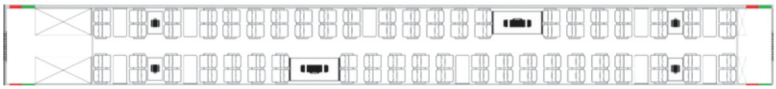

Figure 2 Example layout 2: netwiss - Layout creation in Beta - TrainOptimizer

Three equally fictitious travel purpose mixes are given below. In one the majority of business travellers use the train, in a second example these are mainly holiday travellers and in a third example it is additionally assumed that an international airport is served, which on average also induces a 20 % higher luggage volume.

5.1 Baggage accommodation and seat occupancy rate

Figure 3 shows that 31 pieces of baggage are stowable in layout 1 and 66 in layout 2, on days with a higher proportion of business travel every second piece of baggage is not stowable in layout 1, but all pieces of baggage are stowable in layout 2. On days with a higher proportion of vacationers, two out of three pieces of baggage are not stowable in layout 1, while only 17 pieces of baggage are not stowable in layout 2.

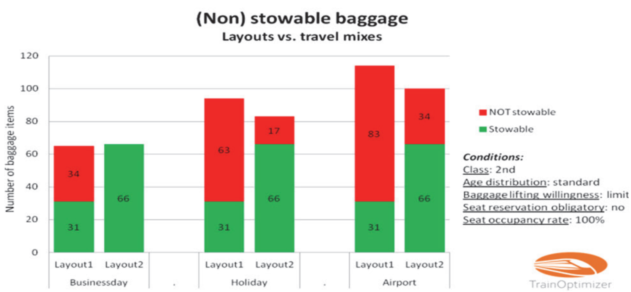

Figure 3 (Non-) stowable aggage in layout comparison: netwiss - Evaluation with Beta - TrainOptimizer

In addition to severe comfort restrictions and general problems caused by pieces of luggage that cannot be stowed properly, such as security problems or delays in passenger transfer, the pieces of luggage that cannot be stowed result in the fact that with layout 1 not all seats can be used in any of the travel purpose scenarios. Even on days with a higher proportion of business travel, only 89 of the 100 seats are available, and in layout 2 all 88 seats are available. On days with a higher proportion of vacation travelers, only 77 seats are actually available on average for layout 1 and at least 82 seats for layout 2 (see Figure 4).

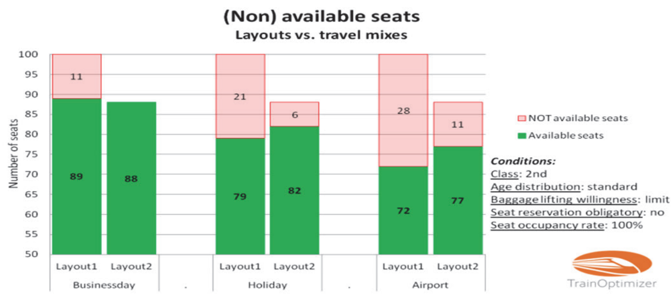

Figure 4 (Non-) available seats in layout comparison (depending on travel purposes): netwiss - Evaluation with Beta - TrainOptimizer

This analysis clearly shows that there is no added value in maximising the number of seats, since in any case no more than 89 seats can ever be used. A reduction in the number of seats therefore not only leads to a noticeable gain in comfort for the travellers, but on the majority of the time even to a higher proportion of available seats.

5.2 Passenger changeover time

The figure 5 shows that the time required for boarding increases more than linearly as the number of passengers boarding increases. The calculations are based on a travel purpose mix with a higher proportion of holidaymakers. Furthermore, it can be seen that the time required for layout 2 with improved baggage accommodation (higher capacity, better distribution in the vehicle) and better siding possibilities increases less strongly. For example, 40 boarding passengers need on average 210 sec for layout 2, whereas the time required for 40 persons for layout 1 is already 50 % higher with an average of 310 sec.

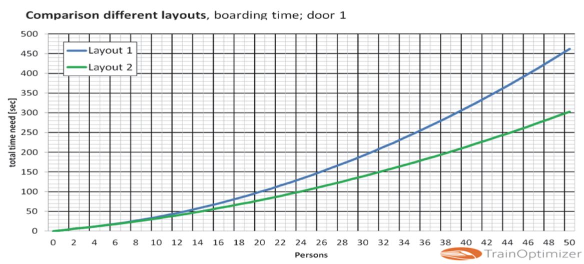

Figure 5 Time required for boarding passengers during layout comparison: netwiss - Evaluation with Beta - TrainOptimizer

The figure 6 shows the time required for a so-called 60 % passenger exchange. This includes both boarding and alighting passengers. A 60 % passenger change is a frequently requested comparative value for calculations which states that 60 % of the passengers of a fully occupied wagon get out and the same number of passengers get back in. With layout 2, the lower number of seats per door also results in a three person lower number of passengers.
The boarding time for the 60 % passenger exchange is just over three minutes, which is about 30 % higher than for layout 2, and the total passenger changeover time, including passengers getting off, still differs by 25 %!

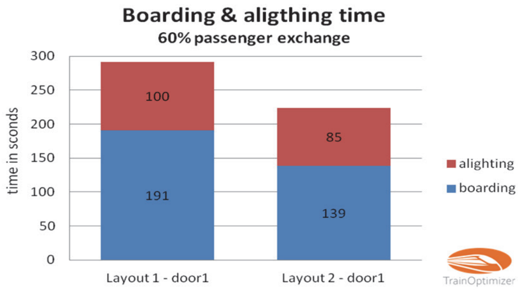

Figure 6 Time required for a 60 % passenger exchange in the layout comparison: netwiss - Evaluation with Beta - TrainOptimizer

6 Conclusion

The vehicle layout and the associated interior design have an influence on the operating quality in many ways. The correct and sufficient dimensioning of luggage racks has a significant influence. Baggage racks that do not meet passengers' basic needs, such as the desired visual contact with luggage or avoiding the lifting and manipulation of larger items of luggage, result in many items of luggage being stored in a disturbing manner. This leads to a decreasing seat occupancy rate and significantly longer passenger changeover times. A lower number of seats, if properly planned, leads to more seats being available in total, even in absolute terms, and to reduced dwell times.

The differences shown in this essay between the two TrainOptimizer® simulated variants, which differ from each other essentially only in the area of improved luggage accommodation, make it clear that with 12 % fewer seats, the proportion of usable seats remains at least the same or is even higher than in the seat-maximized variant with 100 seats. At the same time, the passenger changeover time is approx. 25 % less with a 60 % passenger exchange! If, in addition to the improvement of the baggage systems, further principles for optimisation are taken into account, such as shorter car bodies with the resulting wider aisles or the arrangement of the entrance doors with short car bodies in the middle, further significant improvements are possible, especially in the passenger exchange.

References

[1] Frederich, F.: The future of the railway, ZEV (Glasers Analen) 110, May 1986, S124-134

[2] Plank, V.: Dimensioning of luggage racks in passenger trains, diploma thesis TU-Vienna, 2008

[3] Feiel, R.: Luggage accommodation in long-distance trains - luggage racks; Bachelor Thesis, FH-St. Pölten, Department of Rail Technology and Mobility (2015)

[4] Feiel, R.: Luggage volume in intermodal long-distance traffic; Master Thesis, FH-St.Pölten, Department of Rail Technology and Mobility 2017

[5] Simulation with TrainOptimizer simulation software

STRATEGIC EVALUATION OF THE RAILWAY TRACTION ENERGY SUPPLY DEVELOPMENT ON THE HUNGARIAN RAILWAY NETWORK

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Abstract

Transformer substations are hidden elements of the railway infrastructure, they have a long service life and reliable operation. On this reason reconstruction of substations are often left out of railway development projects. The board responsible for railway development in the Ministry of Innovation and Technology has decided to set up a project dedicated to substation development. The purpose of our work was to assess, examine and supervise the current technical status, network role and future sustainability of railway substations in order to choose a set of substations to be reconstructed in the given cost framework. We completed traction energy simulations to explore the weak points of the traction energy supply system and to provide basic data for the planning process. In our feasibility study we chose 20 of 38 substations to be reconstructed in a multi-step decision process. On Level 1 we assessed professional and operational aspects with multi-criteria analysis (MCA) regarding capacity shortages, energy efficiency, existence of remote control, characteristics of environmental protection and climate resilience, age-related failures, unit performance and network assessment. Based on the multi-criteria analysis we formed feasible technological options. To quantify and compare their long-term financial effects, on Level 2 we have chosen cost-effectiveness analysis methodology considering investment cost and the operational costs incurred during the estimated evaluation period. After option analysis we conducted cost-benefit analysis (CBA). Savings at social level are considered benefits in economic terms. As the type of the intervention did not fit the relevant CBA guide, we had to elaborate a special methodology for the assessment of economic benefits of the project. After all we have set up three project packages (6 or 9 or 20 substations) depending on available funding sources – and all three project packages can be regarded as economically viable and eligible for financing and implementation.

Keywords: project evaluation, railway, power supply

1 Introduction

Main beneficiaries of the EU funds available for transport in the 2010s were railway lines. With the expansion of the electrified railway track sections and the increase in the number and performance of the electrified traction vehicles, the demand for electricity supplying the traffic is growing steadily. The accumulating effect of the increasing power requirement within the feed section of a traction transformer station may easily lead to local overload.
The figure below shows the network of the traction substations of the Hungarian State Railways Plc. The 38 orange discs stand for the sub-stations.

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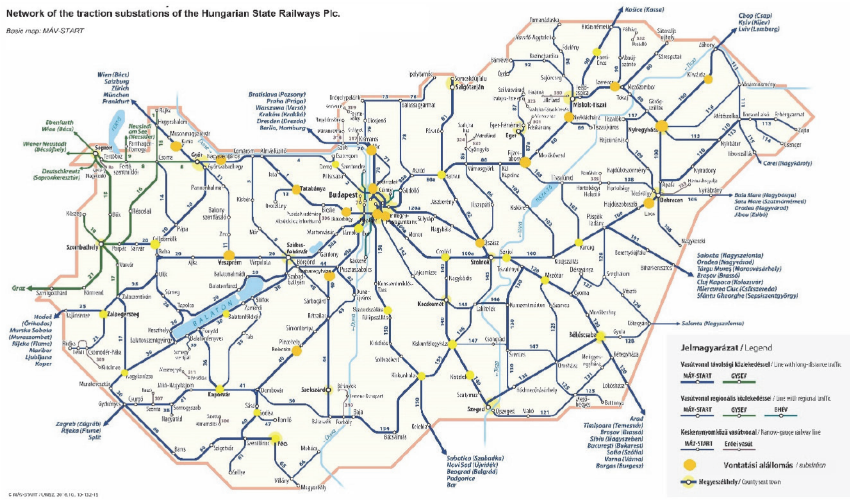

Figure 1 Network of the traction substations of the Hungarian State Railways Plc.

Although transformer substations are invisible elements of railway infrastructure, their proper technical standard is essential for the high-quality operation of rail transport. In our paper we present a comprehensive study covering the national railway network, that sought intervention points for rail traction energy supply network and suggested a prioritization of investments.
Our paper is based on a feasibility study with the title "Development of railway traction energy supply on the network of the Hungarian State Railways Plc." [1].
The targeted financial resource of the implementation project is provided by the Integrated Transport Development Operational Program (ITOP) [2], so we have followed the methodological guidelines of the ITOP program [3].
Now, the project is already in the implementation phase. Due to the developed project packages, growing number of substations can be financed by redeploying resources released on other projects.

2 Objectives and goals of the project

Significance of rail transport projects is emphasised in the European strategic documents like EU White Paper [4] and other studies and reports [5]. Furthermore, the project is in line with the goals of the National Transport Development Strategy [6], as it is promoting resource-efficient ways of transport and improving the quality and the efficiency of the transport services.

2.1 Main problems and the baseline scenario

The main problems stem from the age of the equipment and the obsolescence of the technology. On 10 sub-stations out of the 38 there is no remote-control system. These locations are sub-stations functioning with an operator. Three of them can be remotely controlled on the spot, so they are not connected to the power SCADA control panel. The age of the substation equipment is extremely high and most of the tools have exceeded their life expectancy. If not only active but passive components have to be improved as well, the reconditioning only extends the operating time of the transformer by 5 to 10 years, but the costs of the repair approximate the price of a new transformer.

The number and severity of age-related failures is expected to increase significantly in the future without infrastructure development, therefore the increased consumer demand can no longer be satisfied reliably. In addition, the operating, maintenance and replacement costs are also expected to increase.

2.2 Objectives of the project

The purpose of this project is to provide development suggestions and implementation for the selected substations by means of traction energy supply testing of the substations and their associated energy routes (i.e. normal and emergency power lines). Aiming to improve the inappropriate traction energy supply, we were looking for technical solutions that can provide the right quality and quantity of traction energy to comply with the relevant standards of the EU Regulation 1301/2014. In order to guarantee the proper utilization of the capacity and the safety of operation, we will also determine which traction energy supply development tasks need to be performed, including all the environmental intervention related to development.

3 Option analysis

3.1 Demand analysis

In railway infrastructure development projects, peak demand needs to be met, so the modelled traffic is also assessed for the heaviest traffic period. This was the summer workday afternoon. The traction energy simulation model considered a current timetable (A), a future timetable (B), and a “theoretical maximum” load (C). The model covers the entire electrical transmission path (overhead power line network) belonging to the power supply section of a given substation.

3.2 The method and process of option analysis

In Level 1 analyses, we examined the main features (i.e. the technical status, the network role and the load conditions) of the power supply network that currently comprises 38 substations and their powered sections. The purpose of the Level 1 analysis was to select the 20 development sites. Based on the network characteristics we have performed a multi-criteria analysis (MCA) of the present network at the 38 substations. The MCA revealed professional and operational aspects: capacity shortages, energy efficiency, remote control, environment protection and age-related failures, unit performance and network assessment. When determining the weight of the certain aspects, we also sought to ensure that the evaluation criteria reflect the network role in addition to the technical and performance characteristics. Substations that reached the highest total score were selected based on this evaluation. On the selected substations and their powered sections – based on their technical parameters, scheduling structure and the traction vehicle flee – traction energy simulation was used to determine the electrical capacities required for future loads and expected improvements, as well as to specify the technical content of the project. After traction energy simulation two technical variants (option “A” and “B”) were worked out at each location. Although the option features vary by sub-station, the following considerations have been prioritized:

- aspiration to complete refurbishment
- in the case of common buildings with other special services, only renovating the substation part
- development in own territory, without land acquisition
- enhancing environment protection: developing transformer bases

The following three areas of intervention were specified:
• Placement of the equipment of 25 kV fields (outdoor / indoor)
• Placement of the equipment of 120/25 kV fields (new place / original place)
• Placement of the equipment of 25 kV test resistor (outdoor / indoor)

At Level 2 option analysis we conducted a cost-effectiveness analysis of the two feasible technical options. In the cost-effectiveness analysis we considered the investment cost and the operational (maintenance and replacement) costs incurred during the estimated evaluation period. To support the decision process and to provide flexibility in the financing, three project packages were formed:
• ITOP Basic package: with 6 substations
• ITOP Extended package: with 9 substations
• Full package: with 18 substations

4 Environmental and climate effects

The impacts of the project on the environmental compartments were examined, namely: impacts on soil, water base, ground water, protected natural environment, populated areas, and noise. From a soil, surface water and groundwater quality perspective, the relevant environmental load of the substations may be due to the water pollution caused by regular or occasional oil spill. Nevertheless, it can be stated that the planned technologies, utilities and equipment do not allow the possibility of contamination risk based on the oil- and water sealing indicators. From noise protection aspect, the impact of the construction and operation is acceptable, not significant, thus noise protection measures are not required.
Based on the preliminary studies, it can be concluded that in case of any planned development on the site, the extent of the load on the nearabout environment is not expected to be significant in terms of land protection, water protection, air quality and landscape protection. In the case of those substations selected for the implementation, a separate plan for approval and environmental study has been prepared, which was submitted for a construction authorization procedure. The environmental study is intended to identify and record the environmental protection measures required during the implementation.
As the effects of the climate change, it can be stated that the vulnerability of the planned investment and the level of risk posed by climate change are moderate. [7] Furthermore, the impact of the planned investment, due to its climate change volume, has a neutral effect. Appropriate measures to decrease the effects of the climate change can significantly mitigate the expected negative impacts.

5 Financial and economic analysis

5.1 Financial analysis

The project's financial plan (i.e. the cost of project activities) was broken down into the main cost categories for the investment period (2017-2025) referring to the different project sizes. If the planned substation developments are implemented, the net investment cost without financial reserve:
• for the ITOP basic package: 38,655 million EUR,
• for ITOP extended package: 58,575 million EUR,
• for the full project package: 103,812 million EUR.

The project has not any non-eligible activities. Investment costs were determined partly based on the executive and framework contracts already signed and on the investor's cost estimates. The investment period ends in 2025, so the year 2026 is the first full year of the operating period.
Future operational and maintenance costs were calculated from the actual data of recent years, i.e. the 2014-2016 period. Our assumptions are that permanent fixed costs will not change. For running and maintenance costs we have set the percentages for two periods: between 2017-2025 (% change in value compared to 2014-2016 average), and % after 2025. With the addition of more powerful, more modern transformers, the energy loss also changes. After replacing the existing 2x6, 6 or 12 MVA capacity transformers up to 16 MVA transformers, the idle loss values are reduced but short-circuit losses are increased. Table 1 contains the overall values and the additional operating costs of the replacement of the transformers.

Table 1 Energy losses and additional operating costs [1]

Project package	Idle losses (kWh)	Short-circuit losses (kWh)	Total losses (kWh)	Additional operating cost (EUR)
ITOP basic	-402 960	1 752 000	1 349 040	49 401
ITOP extended	-911 040	2 479 080	1 568 040	57 421
Full	-1 200 120	4 140 560	2 940 440	107 678

In the baseline case, it is inevitable that major refurbishment activities will start in 2025, due to accelerating degradation processes, as further aging equipment is expected to require substantial intervention by then. It was assumed that renovations will also take place in 2030 and 2035, during which only the most inevitable repairs and equipment replacement works are performed. If the equipment in obsolete technical condition cannot be repaired during the maintenance operation, it can only be maintained by the replacement of larger units and thus a significant replacement cost. In case of realizing the project, replacement costs were taken into account, based on the life expectancy and the investment costs of the built-in infrastructure elements. The revenues of the Hungarian State Railways originate mainly from infrastructure charges (network access charges), but the implementation of this project does not affect these charges.

5.2 Economic analysis

For the determination of economic costs, the financial costs have to be adjusted by the following factors:
• fiscal adjustments
• the correction of market price
• external impacts.

Savings at social level as a result of the project are considered benefits in economic terms. The quantified economic benefits of the project are the differences between ”with” and “without” project costs of the change in travel time and the environmental cost savings.
In addition to the inputs provided by the simulation model, unit costs set out in the Hungarian CBA Guide [8] were also taken into account to quantify the benefits. In terms of manifesting time saving in money, train delays, potential time savings for the model trains of the simulation, percentages of current overloads, passenger counts, and unit value of travel time (VOT) were used. Time-saving is due to downsized train delays caused previously by short-circuits and substation errors, as well as the impact of the timetable adjustments on

passenger hours. The performance limit constantly affects the trains on the affected line: it has constant timetable effect by evolving a sort of electric slow-motion signal. The decisive part of all social benefits comes from travel time costs.
Based on the facts above, development of the substations may result in saved travel time costs during the 30-year evaluation period.
Changes in environmental impacts were performed by a detailed estimation methodology in which the impacts on climate change were quantified in accordance with the relevant guidelines [8], [9]. The calculation of the environmental costs was based on determining the effect of GHG on energy use resulting from the change in idle and short-term losses.
Based on the results of the economic analysis of the project, it can be stated that the economic net present value (ENPV) of all three project sizes is positive, the economic internal rate of return (EIRR) is higher than the applied social discount rate (5 %) and the cost-benefit ratio is also higher than 1. Thus, the project can be regarded as economically viable and eligible under the initial conditions. The best return is made by ITOP-basic packages, but the ITOP expansion, as well as the implementation of the entire development package is equally effective.

6 Conclusion

Energy supply system (including substations) is a special segment of the railway transport infrastructure with long service life and reliable operation, but reconstructions of these elements are often left out of railway development projects.
The purpose of our work was to assess, examine and supervise the current technical status, network role and future sustainability of railway substations in order to choose a set of substations to be reconstructed in the given cost framework. We used multi-level assessment for strategic and technical option analysis; and we have set up three project packages (6 or 9 or 20 substations) depending on available funding sources.
After option analysis we conducted cost-benefit analysis (CBA). As the type of the intervention did not fit the relevant CBA guide, we had to elaborate a special methodology for the assessment of economic benefits of the project. Based on the financial and economic analysis all three project packages can be regarded as economically viable and eligible for financing and implementation.
As a conclusion it can be stated that sustainability and viability of the project are affected by the relevant technical requirements, the results of the traction energy simulation, the social-economic background, and the availability of the funding sources.

References

[1] Transinvest-Budapest Ltd., TP Infra Ltd., AXON 6M Ltd.: Development of railway traction energy supply on the network of the Hungarian State Railways Plc., Feasibility study, 2018

[2] National Ministry of Development: Integrated Transport Development Operational Program (ITOP), 2014-2020, 2014

[3] Trenecon Ltd.: Guidelines for the preparation of a feasibility study of the projects financed by the Integrated Transport Development Operational Program, 2016

[4] European Commission: White paper, Roadmap to a Single European Transport Area – Towards a competitive and resource efficient transport system; COM(2011) 144; 2011

[5] European Commission: Electrification of the Transport System, Studies and reports, 2017

[6] Strategic Consortia: National Transport Strategy, Budapest, 2013

[7] Kelemen, Á., Malatinszky, É., Kisgyörgy, L., Mátyás, L., Buzás, K.: Detailed methodological guide for Climate Risk Assessment. Made on behalf of the Prime Minister's Office, Budapest, 2017.

[8] Trenecon Ltd.: Feasibility study and cost-benefit analysis template for individual applications of the Integrated Transport Development Operational Program - For projects with a total eligible cost of at least 1 mEUR, 2016

[9] European Commission: Guide to Cost-benefit Analysis of Investment Projects – Economicappraisal tool for cohesion Policy 2014-2020, 2014

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DEVELOPMENT OF CONTACTLESS MEASUREMENT DEVICE FOR OVERHEAD CONTACT LINE

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Abstract

In order to reduce maintenance labor of overhead contact lines (OCL), a contactless measurement device for OCL was developed. This device is mounted on a roof of a vehicle of a train and measures static three-dimensional positions of wires of OCL and detects positions of OCL fittings without touching the OCL while the train is running. We proposed hybrid sensing method that combines stereo measurement by image processing with structure measurement by laser range scanners and it realized to measure OCL geometry with high-precision even in sections with complicated OCL structure. In addition, we developed position detection method of the OCL fittings that can cope with changes in height and stagger of OCL by using machine learning. Measurement data of OCL contactless measurement device is static position of OCL without influence of a probe such as a pantograph. With this device, the OCL static position can be measured continuously instead of at each support point or dropper point. For maintenance of OCL, the criterion of OCL is defined as a static position. The device is utilizable for OCL maintenance and it sophisticate maintenance of OCL. For example, this device realize the height difference measurement of the crossing section, which has been conventionally measured by maintenance workers. In addition, the device are utilizable for OCL fittings inspection. Maintenance workers can check the image of OCL fittings without on foot into the field. Furthermore the static position data of OCL can be used to create simulation model of OCL dynamic behavior. Using this model, it is possible to know the dynamic response of cases where various pantographs pass at various speeds. Running tests was conducted on commercial line, and the performance of the device was verified when running at a speed of 130 km/h. The results shown that the repeated measurement accuracy is within 10 mm, and the OCL fitting detection rate is 90 % or more.

Keywords: OCL, maintenance, OCL fittings, machine learning

1 Introduction

Overhead contact lines (OCL) are installed along a railway track for a long distance, and their inspection requires a lot of maintenance labor. In order to reduce maintenance works such as walking patrols and close-up inspections, automated inspection by railway vehicle is one of the solutions. Therefore, we developed a contactless measurement device for OCL [1]. This device can measure static positions of OCL wires. OCL static position measurement using this device replace manual inspection of overlap or crossing section configurations and reduce maintenance work. The device is mounted on a roof of a vehicle of a train and can measure static three-dimensional positions of wires of OCL and detect positions of OCL fittings while

the train is running. We proposed hybrid sensing method that combines stereo measurement by image processing with structure measurement by laser range scanners and it realized to measure OCL with high-precision even in sections with complicated OCL structure.
In this paper, we report prototyped OCL contactless measurement device can measure the three-dimensional position of the OCL wires and OCL fittings at 130 km/h speed on a commercial line. In addition, we developed position detection system of the OCL fittings that can cope with changes in height and stagger of OCL by using machine learning.

2 Specification of contact measurement device for OCL

2.1 Basic concept

Using a hybrid sensing method that uses two line cameras and two laser scanners together, the OCL contactless measurement device measures the three-dimensional position of not only contact wires, but also messenger wires and auxiliary messenger wires. In addition, the device measures position of OCL fittings. This is achieved by detecting OCL fittings from two images captured by the two line cameras.
Two laser scanners are installed on the left and the right of the vehicle and look up at the line from the roof, because the messenger wires are hidden behind a contact wire when the overhead line is directly above the laser scanner. Even if one laser scanner cannot detect the wire, the other laser scanner can detect the wire to ensure redundancy.
Two line scan cameras are installed on the right and the left sides in the traveling direction to continuously capture the image of OCL and perform stereo measurement of the wires, and the line scan camera can also capture images of the OCL fittings from both sides. When performing stereo measurement, even if the image is in backlight condition, it is possible to detect the position of the line. Even if one of the two cameras are in a backlight state with respect to the contact wire, the other camera is in normal light state. Therefore, at least one camera is in normal light state regardless of the sunlight direction.

2.2 Device configuration

We consider the hardware configuration of the OCL contactless measurement device for 130 km/h running. It is required to achieve both high-speed capturing and a wide depth of field. However, there are trade-offs between capture speed and depth of field. Increase capture speed decrease depth of field. As a countermeasure, it is proposed to mount two sets of cameras with different focus ranges to take images that are divided the lower and upper parts of OCL.
However, consideration of the aim of mounting this device on the roof of a commercial vehicle in the future requires to downsize the device. Therefore, it should be used only one set of cameras. In addition, the lower and upper parts of OCL fittings such as connectors and droppers are captured as separate images with different focus ranges. It is difficult to detect fittings and diagnose abnormalities without whole images of the fittings. On the other hand, the fisheye lens has a large image distortion but a wide depth of field. The distortion of the image can be corrected by image processing. Therefore, we used fish-eye lens without divide image.

Sizes of OCL fittings decided resolution of the image. It requires higher than 2 mm / pixel for detection and diagnosis of OCL fittings. It is necessary to use 8k line scan cameras because of wide field of view of fish-eye lens. In addition, at a resolution of 2 mm / pixel in the traveling direction, that is, at a scan rate of 18 kHz at 130 km/h, the exposure time per line is about 50 μs. The image data can be recorded uncompressed, only the central 6144 pixels image data of the 8k line scan camera was recorded due to the restriction of the data transfer bandwidth.
The pulse signal was obtained from the speed generator of the vehicle axis to calculate kilometrage. The kilometrage is recorded together with the image captured by the line scan cameras and the data measured by the laser scanners.
Figure 1 shows the OCL contactless measurement device. To capture clear image of OCL fittings, it is required that the elevation angle of the line scan cameras to the OCL are small. So that the line scan cameras were placed outsides and the laser scanner was placed inside. In addition, eight white LED lighting units were equipped. The weight of the device on the roof, excluding cables, is about 55 kg. The device is waterproof. However, it cannot use in rainy weather because water adheres to the lens cover surface and causing irregular reflection of the light.

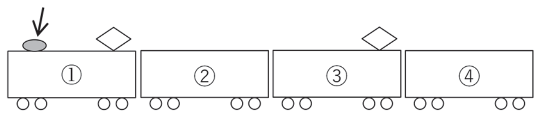
Figure 1 Contactless measurement device for OCL

3 Accuracy verification by field test

3.1 Outline of field test

In order to verify the OCL contactless measurement device accuracy, we installed the prototype device on a railway vehicle and run it on a conventional line to measure static position of OCL. We also measured the static height of the contact wire using the conventional method, and compared the two data. The accuracy of the device at low speed have been confirmed [1]. In the low speed verification, the device was mounted on a truck and measured on the OCL test bench in Railway Technical Research Institute. As a result, the error in the static height of the contact wire was within 2 mm.
In this section, we describe the field test performed to verify the measurement accuracy of OCL contactless measurement device mounted on a railway vehicle running at high speed. In the field test, the vehicle travelled several times in a section of about 40 km each way on a conventional line and recorded images. Table 1 shows the outline of test run. The maximum speed is 130 km/h. The OCL contactless measurement device was mounted on the roof of the first vehicle. Distance from the nearest pantograph is 13 m, and when the first car travelling at the head, the measurement can be performed with less effect of the uplift of the contact wire by the pantographs. In this paper, the measurement data obtained in the OCL contactless measurement device is treated as static structure measurement.

Table 1 Outline of test run.

Season	2 days in November
Weather	Sunny (both days)
Test section	40km, Three return trip / day
Speed	Max. 130 km/h
OCL type	Simple catenary
EMU configuration	OCL contactless measurement device 

3.2 Captured image

Figure 2 shows an example of a captured image of a section where the vehicle travels at 130 km/h in the running test. Although the distortion caused by the fisheye lens is observed, it is confirmed that equipment such as feeders, high-voltage power distribution lines, hinged cantilevers and catenary poles are recorded. The images of each component are focused and have enough resolution. We found out that proposed configuration of the device is suitable for OCL inspection.

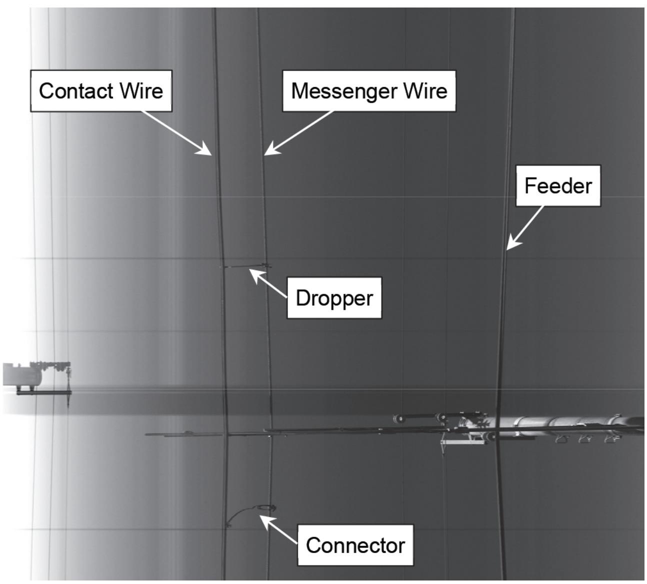


Figure 2 Example of captured image

3.3 OCL 3D structure measurement

After the test run, OCL 3D static geometry was measured with stereo measurement method. Figure 3 shows the measurement results of the height of the OCL in the test section running at 130 km/h, and Figure 4 shows the measurement results of the contact wire stagger in the same section. Figure 3 and 4 include the data obtained in the first and the second days of this test campaign, the measurement result of contact wire using a contact wire measuring instrument [2], and the contact wire inspection data measured from the inspection car that ran in the similar period of time. Note that the distance axis misaligned due to slippage of the wheels, etc., so axis was manually aligned based on the waveform.
The comparison between the contact wire position measured by the device on the 1st and 2nd days and the contact wire static height measured on a maintenance vehicle shows that difference is within about 20 mm. In test section, the train was running with the first car at the head, and that the effect of the contact wire lifts due to the pantograph located behind the device was small. The comparison between the contact wire heights measured by the device on the 1st and 2nd days shows that the contact wire heights agree within 10 mm, confirming that the repeat measurement accuracy is practically sufficient. Regarding the stagger of the contact wire, the comparison between those of the first day and the second day shows that they agreed within 20 mm. Since the contact wire stagger in the contactless measurements is not corrected the influence of the vehicle body rolling, it is considered that the repeat measurement accuracy is lower than the height of the contact wire.

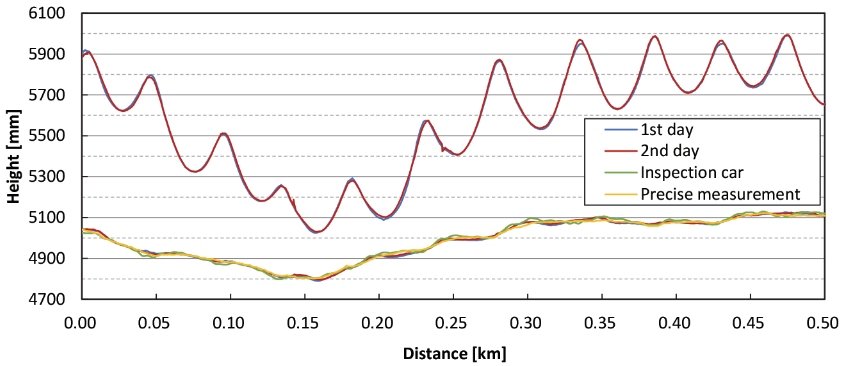


Figure 3 Static height of OCL

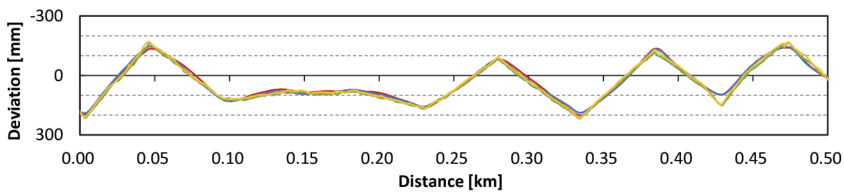


Figure 4 Static stagger of contact wire

4 OCL fittings detection

Dropper detection from the image of test section (approximately 1.6 km) of the simple catenary was performed by a machine learning algorithm. As learning data for machine learning, about 2000 annotated images were used, and a used transfer learning method. Detection processing was performed with area limitation. The limitation area was determined based on the wire position information.
Table 2 shows the detection results of dropper positions. The OCL contactless measurement device acquired OCL image by two cameras on the right and the left, if the OCL fitting can be detected from the image of either camera, it is determined to be detection (True Positive: TP). When neither camera detect the OCL fitting, it is determined to miss detection (False Negative: FN), and if false detection is made despite the absence of OCL fittings, it is judged as over detection (False Positive: FP). The judgement was made manually.
The precision (TP / (FP + TP)) and the recall (TP / (FN + TP)) are index of detection performance. The higher the recall value becomes, the less oversight is. Both the precision and recall were over 90 %, achieving highly accurate OCL fitting detection.
Figure 5 shows the position of the detected droppers and the three-dimensional positions of the OCL wires. By measuring the static three-dimensional geometry of a real OCL using the OCL contactless measurement device, we can digitize real OCL. The digitized OCL can be used as a simulation model to predict the behaviour when passing through a pantograph. It also is expected to be used as a tool to realize digital twins.

Table 2 Detection results of droppers

	Clipped Image
True number of droppers	94
True Positive	87
False Negative	7
False Positive	9
Precision	90.6 %
Recall	92.6 %

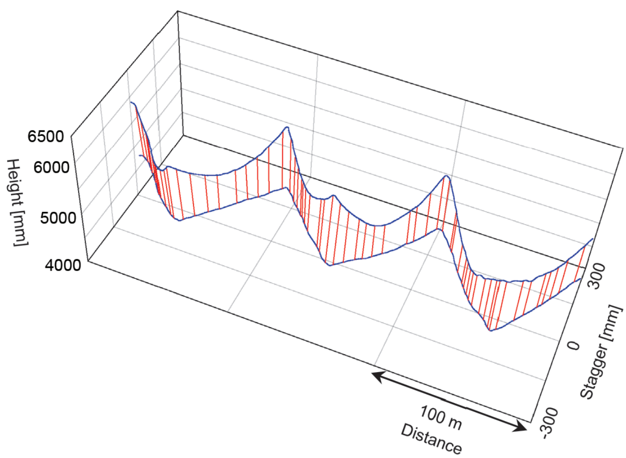


Figure 5 Measurement result of OCL 3D structure

5 Conclusion

We prototyped an OCL contactless measurement device and mounted on a vehicle of a train to perform a running test on a commercial line. Proposed “hybrid sensing method” realized the height and stagger measurement of OCL within 10 mm repeat accuracy of contact wire height at a speed of 130 km/h. In addition to this, we developed position detection system of the OCL fittings with machine learning algorithm. The detection rate of OCL fittings was over 90 %. Further research aims to develop diagnosis system of OCL fittings. Combined system with the OCL contactless measurement device and the OCL fittings diagnosis system will realize automated and frequent visual inspection of OCL. It will contribute to sophistication of OCL maintenance.

References

[1] Nezu, K., Matsumura, I., Aboshi, M., Niwakawa, M., Kawabata, T., Tabayashi, S.: Contactless Measuring Method of Overhead Contact Line Positions by Stereo Image Measurement and Laser Distance Measurement, Quarterly Report of RTRI, 56 (2015) 3, pp. 181-186.

[2] Shimizu, M., Fujii, Y.: Improvement of Structure of Contact Wire on Overlap Sections of Shinkansen, Quarterly Report of RTRI, 41 (2000) 4, pp. 159-162.

5

SUPESTRUCTURE: DESIGN, MODELLING, OPTIMIZATION, MONITORING AND CONDITION ASSESMENT

BALLAST CONDITION EVALUATION DURING TAMPING ACTIONS

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Abstract

Ballast is a key component of most railway tracks. The ballast bed must cope with high demands while fulfilling crucial tasks. Wear and contamination cause the condition of the bedding to deteriorate, which is accompanied by a loss of its proper functioning. Consequently, track alignment issues arise, which are typically corrected by tamping the affected areas. This study presents a new approach to assess the condition of the bedding. The tamping unit of a high-performance tamping machine has been equipped with an array of sensors which measure various parameters during every tamping process. A set of recorded data is analysed and compared with the prevailing ballast condition of the tamped sections, which is evaluated using proven methods. The results indicate a strong correlation between the tamping machine measurements and the condition of the bedding, which shows that tamping machines can be used to monitor the track ballast condition.

Keywords:

1 Introduction

Globally, most railway lines feature a conventional track structure [1] where the main components are rails, sleepers, and track ballast. A well-functioning ballast bed strongly contributes to track elasticity, distributes the induced forces in longitudinal and lateral direction and fosters water runoff. The bedding also secures the track grid (rails + sleepers) in its position and thereby largely defines the overall track geometry. The desired properties of the bedding lessen over time due to its contamination and wear of the stones. Consequently, load distribution within the track structure changes adversely and track geometry problems occur [2]. These, in turn, increase the dynamic forces caused by passing trains, which exacerbates the problem. This vicious circle highlights the importance of a well-maintained ballast bed, which implies the necessity of proper condition monitoring. While several methods for ballast condition monitoring exist, two are particularly well-suited for net-wide evaluations: ground penetrating radar (GPR) and fractal analyses [3]. However, both of them are indirect measurements – both assess the condition without coming into contact with the bedding. This makes the two methods susceptible for inadvertent external interferences.
This study presents a novel alternative for ballast condition assessment, using data recorded by a tamping machine while operating in the network of Austrian Federal Railways (ÖBB). Besides analysing this tamping machine data, the sections where the tamping works were conducted are also thoroughly investigated. The prevailing ballast condition on these sections is assessed via a ballast condition index, which combines GPR and fractal analyses. This ballast condition index serves as reference to which the tamping data are then compared.

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2 Methodology

This study is based on two different data sources: The data recorded by the tamping machine – hereinafter referred to as “tamping data” – are provided by Plasser & Theurer, Export von Bahnbaumaschinen, Gesellschaft m.b.H. Information of the infrastructure – hereinafter addressed as “infrastructure data” – is provided by the Institute of Railway Engineering and Transport Economy (Graz University of Technology). This chapter describes the two data sources and how they are connected.

2.1 Infrastructure data

The infrastructure database of the Institute of Railway Engineering and Transport Economy covers a large part of the ÖBB network. The data warehouse comprises general information (e.g. line speed, track load, curvature), superstructure parameters (e.g. track age, rail profile, sleeper type), and track recording car measurements over multiple years. Regarding the ballast condition, GPR evaluations and fractal analyses (derived from the longitudinal level signal) are incorporated. Within this study, all assessments of the infrastructure along the tamping sections rely on this infrastructure data base.

2.1.1 Track ballast condition

New track ballast consists of rough-edged stones which form a strong, interlocked matrix with vacant space between the stones. This structure gives the bedding its desired properties [2]. When the stones abrade or even break under the high loads of rail traffic, the voids get contaminated with fine grain (Fig. 1). Small particles can also originate from external sources (e.g. leaking coal waggons) or rise from the substructure [4]. The condition of the bedding can be assessed with different methods, two of which are applied in this study: ground penetrating radar and fractal analyses.

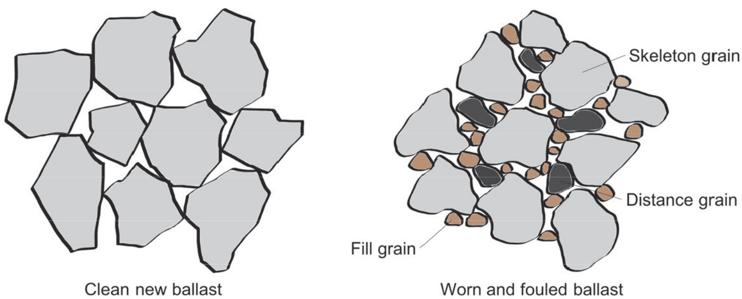

Figure 1 Illustration of the difference between clean new ballast versus worn and fouled ballast [5].

Ground penetrating radar (GPR) uses electromagnetic pulses which are emitted towards the track. Depending on the material and the condition of the evaluated structure, these pulses are absorbed and reflected to certain degrees. The discrepancies between the emitted and the received pulses enable assessments of the targeted objects – in case of track monitoring the ballast bed and the substructure [6].

Fractal analyses are a mathematical concept used for signal analysis. In the context of railways, the longitudinal level signal is dissected into three fractal dimensions. These dimensions represent short-waved, mid-waved, and long-waved track irregularities. Studies have shown that the mid-waved fractal dimension strongly correlates with the condition of the bedding; thus, using the mid-waved range, fractal analyses are well-suited to assess the ballast condition [7].

2.1.2 Ballast condition index

Landgraf [3] developed a ballast condition index which combines GPR evaluations and fractal analyses. This methodology provides a holistic assessment of the ballast condition and it serves as reference parameter in this study. Compared to the individual methodologies (GPR, fractal analysis), the ballast condition index offers two main advantages: First, it rates the ballast condition at any point in the network by a single value ranging from 0 % (worst condition) to 100 % (best condition). Otherwise, several GPR categories (such as ballast fouling, ballast humidity, or clay fouling) and multiple available fractal evaluations (one for every measurement car run; typically, four runs per year are conducted on main lines) would have to be considered separately. Secondly, the combination of ground penetrating radar and fractal analyses strengthens the validity of the results. Individually, both methodologies underly unavoidable variances caused by influences beyond control. The ballast condition index will only take on high (good condition) or low (poor condition) values if the two methods are conform with each other, i.e. if both assess the ballast condition either good or poor. These extrema are in the focus of the present study. Should the GPR and fractal analyses contradict each other, i.e. one methodology indicates good ballast condition and the other poor, the resulting condition value will be around 50 %.

2.2 Tamping data

The data recorded by the tamping machine originate from 13 tamping actions which were executed in the network of Austrian Federal Railways (ÖBB) in 2016 and 2017. During these tamping actions, approximately 10,000 tamping processes were conducted. A tamping process includes (a) positioning the tamping unit centrally above the sleeper and realigning the track grid, and (b) one to three consecutive squeezing processes. Each squeezing process consists of (i) penetrating the ballast bed, (ii) a squeezing movement, and (iii) lifting the tamping unit. During the entire process the tamping tines vibrate back and forth at a frequency of 35 Hz. [5]

The tamping data were recorded by four different types of sensors, all mounted on the same tamping arm and tamping tine (Fig. 2). Strain gauges on the tamping tine measure forces in vertical and horizontal direction, accelerators at the top of the tamping arm enable the calculation of the oscillation amplitude. A pressure measurement in the hydraulic system records any movement of the fluid and a laser rangefinder delivers the squeezing displacement [8].

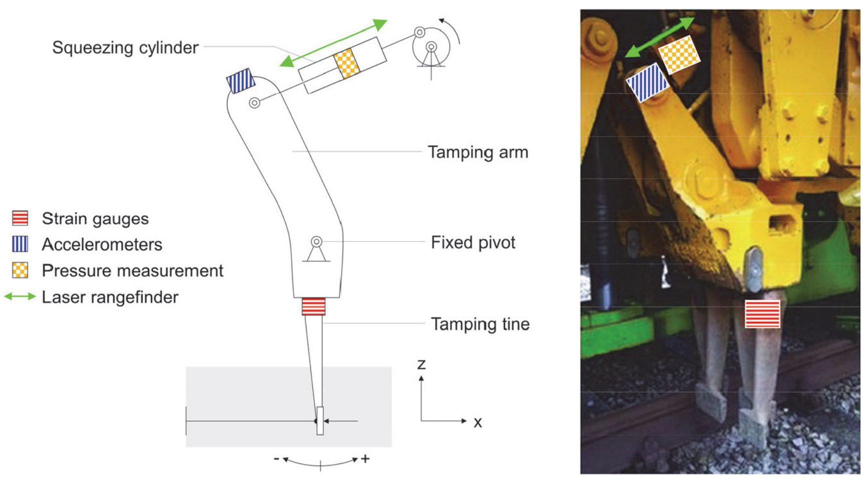

Figure 2 Positioning of the sensors on the tamping arm and tamping tine (adapted from [8]).

Six relevant parameters recorded the tamping machine are analysed in this study: penetration force, squeezing force, squeezing energy, squeezing velocity, loading response, and unloading response (see Table 1). Penetration force, squeezing force and squeezing velocity are directly measured; squeezing energy, which represents the energy that is transferred into the bedding, as well loading and unloading response, which represent the resistance of the ballast to the tamping tine movement, are calculated in retrospect. Each parameter is represented by one value per squeezing process. This value is either the maximum or the average of all tamping tine oscillations during the respective squeezing process.

Table 1 Description of the analysed tamping parameters.

Parameter	Unit	Description
Penetration force	kN	Maximum axial force (in z-direction; see Fig. 3) during ballast penetration.
Squeezing force	kN	Maximum lateral force (in x-direction; see Fig. 3) during the squeezing movement.
Squeezing energy	J/s	Standardized consumed energy per squeezing movement (average of all tine oscillations).
Squeezing velocity	mm/s	Velocity of the closing tamping tines (average of all tine oscillations).
Loading response	MN/m	Ballast response during the forward oscillation of the tamping tines (average of all oscillations).
Unloading response	MN/m	Ballast response during the backward oscillation of the tamping tines (average of all oscillations).

2.3 Data connection

Before the recordings of the tamping machine can be analysed for correlations with the ballast condition, the tamping data set needs to be linked to the infrastructure data set. This is possible via GPS coordinates, which the tamping machine recorded at every tamping process. This results in one recorded position every 2.4 m, as the machine in question is a 4-sleeper main line tamping machine and the average sleeper gap is 60 cm. In contrast, the infrastructure database provides track coordinates with intervals of 1 m. Therefore, every coordinate recorded by the tamping machine needs to be linked to its counterpart of the infrastructure data base. Fig. 3 shows an illustration of how this process is done: for every coordinate recorded by the tamping machine, the nearest coordinate of the infrastructure data base is extracted. All information of this coordinate pair – infrastructure data and tamping data – is then merged. This connected data set constitutes the basis for all further analyses.

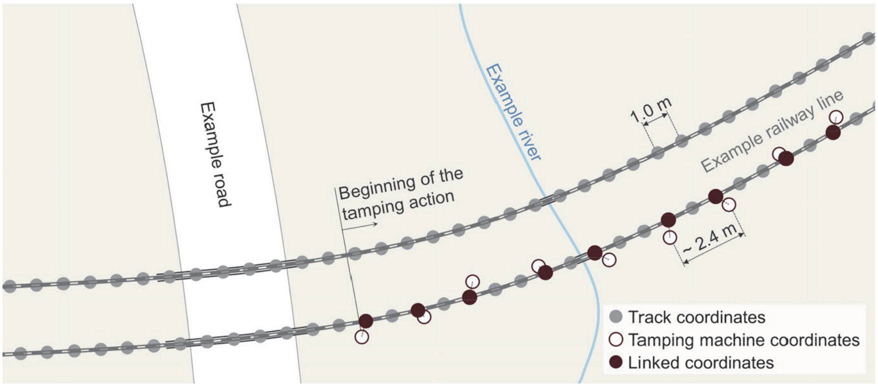

Figure 3 llustration of the connection process which links tamping machine coordinates to track coordinates.

3 Results

Having connected the two required data sets, the sections where the tamping actions took place are inspected. During the 13 recorded machine deployments approximately 6 km track was tamped. Almost 50 % of the executed sections were equipped with concrete sleepers, ~30 % with concrete sleepers with under sleeper pads (concrete USP), and ~20 % with wooden sleepers.

The tamping actions were partly performed in course of complete track renewals and partly as part of the regular track maintenance program (Fig. 4). On main lines, track renewals generally include a ballast cleaning, which invalidates any information on previous ballast condition. Thus, track renewal tamping actions are excluded from further analyses. The remaining maintenance tamping actions were largely executed on sections with concrete sleepers (~2.7 km). Therefore, they constitute the largest individual sample and will be analysed in detail.

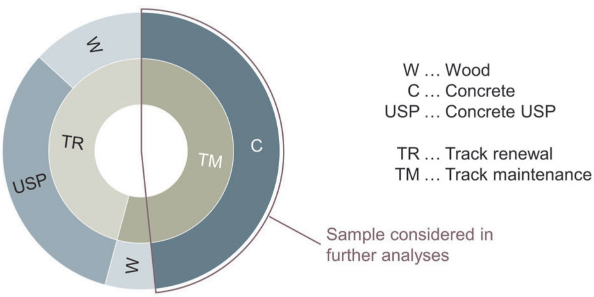

Figure 4 Share of the tamping types (track renewal and track maintenance) and sleeper types.

Fig. 5 gives an overview of the ballast condition along the tamping sections (track maintenance; concrete sleepers), expressed via the ballast condition index. The histogram bars represent the relative frequency of tamping processes (which is proportional to the track length) executed at a specific ballast condition.

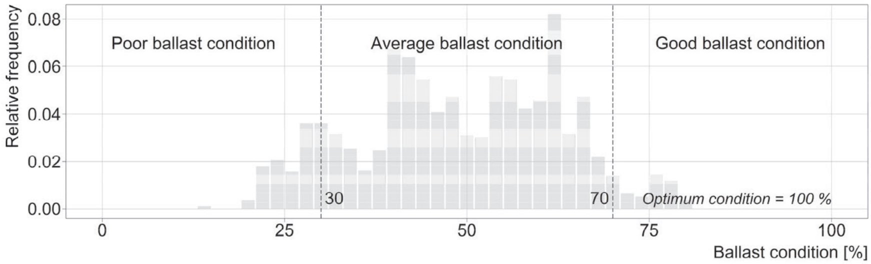

Figure 5 Histogram depicting the relative frequency of tamping processes perfomed at different ballast condition levels.

For comparison with the tamping data, the ballast condition values are clustered into three groups: poor ballast condition, average ballast condition, and good ballast condition. The threshold values separating these groups are set at 30 % and 70 % of the condition index. Thus, only if both GPR and fractal analyses rated a point either good or poor they will end up in the good or poor cluster. The majority of data is categorized as average condition; this includes any point which the two evaluation methodologies (GPR, fractal analysis) have assessed differently.

The recorded tamping data are presented in Fig. 6 in form of boxplots, clustered into the three condition groups good-average-poor. The measurement values are normalized, i.e. the lowest recorded value equals 0 % and the highest recorded value equals 100 %. The general overlapping of the boxplots between the three ballast conditions is a result of many influences, which further research needs to investigate. Presumably, variances of the underlying data and offsets between the tamped sleeper and the connected track point (see Fig. 3) affect the results. Also, the ballast condition index does not provide the precise ballast condition like wear of the stones, humidity level, contamination level, or types of contaminants. However, suchlike parameters may affect the tamping machine measurements. Despite the aforementioned uncertainties, the plot in Fig. 6 demonstrates that all tamping parameters (except unloading response which will be investigated in future research) significantly differ between the different ballast condition groups, which is marked by the non-overlapping notches of the boxplots. This proofs that the tamping machine measurements correlate with the ballast condition – or in other terms, it is possible to assess the condition of the bedding with tamping machines during track works.

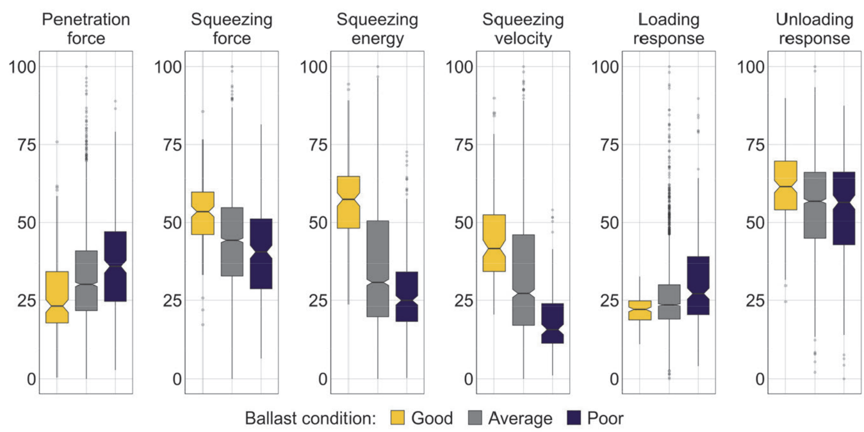

Figure 6 Boxplots of the normalized tamping parameters (lowest recorded value = 0 %, highest recorded value = 100 %), clustered into good, average, and poor ballast condition.

4 Conclusion

This study investigates whether tamping machines, upgraded with an array of sensors attached to the tamping unit, can be used to assess the condition of the bedding during tamping works. A tamping machine has been equipped with multiple sensors which measure different parameters during every tamping process. Data recorded by these sensors during 13 tamping actions in the Austrian railway network are analysed. Simultaneously, the prevailing ballast condition along the tamped sections is evaluated using ground penetrating radar and fractal analyses. The two data sets – tamping machine recordings and ballast condition information – are then linked and thoroughly analysed. The results show a clear correlation between the tamping machine measurements and the prevailing ballast condition. This provides evidence that tamping machines, upgraded with a smart sensor system, are able to evaluate the condition of the ballast bed.

References

[1] Avramovic, N.: Comparison of Ballast and Ballastless Tracks, Master´s Thesis, Graz University of Technology, 2010.

[2] Qiao, P., Davalos, J.F., Zipfel, M.G.: Modeling and optimal design of composite-reinforced wood railroad crosstie, Composite Structures, 41 (1998) 1, pp. 87–96, doi: https://doi.org/10.1016/S0263-8223(98)00051-8.

[3] Landgraf, M.: Smart data for sustainable Railway Asset Management, Verlag der Technischen Universität Graz, Graz, 2018.

[4] Bach, H.: Evaluation of attrition tests for railway ballast, dissertation, Graz University of Technology, 2013.

[5] Offenbacher, S., Antony, B., Barbir, O., Auer, F., Landgraf, M.: Evaluating the applicability of multi-sensor equipped tamping machines for ballast condition monitoring, Measurement 172 (2021), doi: https://doi.org/10.1016/j.measurement.2020.108881.

[6] De Bold, R.: Non-Destructive Evaluation of Railway Trackbed Ballast, PhD Thesis, University of Edinbrugh, 2011.

[7] Landgraf, M., Hansmann, F.: Fractal analysis as an innovative approach for evaluating the condition of railway tracks, Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit, 233 (2019) pp. 596–605, doi: https://doi.org/10.1177/0954409718795763.

[8] Barbir, O., Adam, D., Kopf, F., Pistrol, J., Auer, F., Antony, B.: Development of condition-based tamping process in railway engineering, Ce/Papers, 2 (2018) 2-3, pp. 969–974, doi: https://doi.org/10.1002/cepa.797.

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A DISCUSSION FOR THE REDUCTION IN THE LENGTH OF A PRESTRESSED CONCRETE RAILWAY TIE IN TIME

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Abstract

Increasing train speeds, contemporary requirements for reduced track maintenance costs and extended track service lives required the development and use of reinforced concrete and prestressed concrete ties. Railway engineers began to use concrete for their bi-block and monoblock railway ties heavily, following the development of an understanding for design and performance of concrete structures, production of high strength steel wires and preferable economy of prefabricated mass production for reinforced and prestressed concrete structural elements following the first half of 20th Century. Structural elements of a railway track such as reinforced or prestressed concrete ties have strict production tolerances that are not common for ordinary structural elements. Production of concrete railway ties takes place under strict dimensional control that ensures a nominal design gauge width for the railway track. Design specifications for prestressed monoblock ties frequently specify the gauge width and the shoulder width to be within 1 mm of the design width. However, prestressed concrete ties experience shortenings in length due to transfer of the prestressing force known as instant elastic shortening and shortenings due to concrete shrinkage and concrete creep in time that also relate to ambient relavite humidity.
The author conducted numerous studies on the matter, showed by calculation, and observed experimentally that if unaccounted for, such shortenings can surpass the allowed tolerances in time and result in the rejection of the produced tie for use in the railway track. This paper refers to previous studies by the author that brought international attention on the issue and presents a thorough and a practical evaluation of time related changes in tie lengths for a particular design for prestressed concrete monoblock ties under varying ambient humidity conditions.

Keywords: prestressing, concrete railway ties, shrinkage, creep, elastic shortening

1 Introduction

Railway ties align and secure rails at a fixed distance apart and transfer the forces that are imposed onto the rails to the underlying supportive materials. They are exposed to cyclic and dynamic train loads under the effects of ever-present environmental conditions. Ties of a high-speed railway line must be designed and produced to levels of dimensional precision higher than typical structural concrete elements such as beams, slabs and columns. Attainment of this precision provides for the economic, safe and reliable operation of contemporary railway services.
Train speed is heavily dependent on the railway track qualities and the loads are generated by the interactions of the wheels with the railhead. The gauge length tolerances for high-speed railways built according to the international standards can be specified to as low as

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+2 mm and -1 mm (1). However, due to gauge length variations, the interactions can take place between the flange and the inner side of the railhead as well, thereby increasing the number of possible points of contact between the wheel and the rail, provoking lateral and frictional forces, vibrations, disturbing ride comfort and ride stability, and inducing wheel and rail abrasions [2].

Contemporary freight and passenger railways today, frequently employ prestressed concrete monoblock railway ties. Design and production of these prefabricated ties involve the use of high-performance concrete and high strength steel wires and other steel attachments. Prestressing forces on these ties can reach and exceed 350 kN and the characteristic cylinder strength of the concrete used to produce the ties are typically above 50 MPa that may reach up to 70 MPa, depending on the prefabricated production method used to produce the ties. However, presence of the prestressing forces, time dependent qualities of concrete and the prestressing steel generates time dependent deformations on the railway ties that may exceed the specified dimensional tolerances of the railway ties. The initial elastic shortening of the ties following the release of the prestressing forces into the tie, followed by the shrinkage and creep of concrete causes the produced length of the tie to shorten in time. The relaxation of the prestressing steel along with the prestressing losses due to shrinkage and creep of concrete also reduce the effective prestressing force in the tie, which one must consider in the design of the ties. The relative humidity of the region within which the railway ties will serve, has a high influence on the shrinkage and creep values of concrete, which in return effects the initial prestressing force that must be transferred on to the tie to preserve a final and an effective value of prestressing force in the tie required for the mechanical needs of the tie after the losses. The effect of relative humidity on the dimensional changes of the ties also indicates the need to consider climate change during the expected service life of the railway tie.

This paper presents a summary of two previously presented papers that introduced the time dependent design dimensional variations of prestressed monoblock concrete railway ties to the international academic and engineering community of railway engineering. The sections that follow will show the measured contractions in the shoulder width of a railway tie in time and will also present the contraction estimates obtained through the empirical procedure presented in the relevant reference [3].

2 Considerations for shoulder width variations of a tie in time

Design and construction of the 212 km long Ankara - Polatlı– Konya high speed railway as a part of the project to connect the cities of Ankara – Konya was undertaken by a prominent national firm early in 2008 with national engineering and material resources. The district of Polatlı lies roughly 50 km to the west of Ankara, the connection for which to Ankara was constructed earlier. The two cities located within the central part of the Turkish Republic; known as the Central Anatolia Region, is going through a climate change. Figure 1 shows that the arid conditions of the region has shifted to values as low as 50 % within 36 years [4]. This is an on-going process and effects of climate change is visible in the region. The author, who took part early in the structural designs for the project, considered the projected effects of climate change on the dimensional tolerances of precision elements of the railway track and especially the prefabricated and prestressed railway ties.

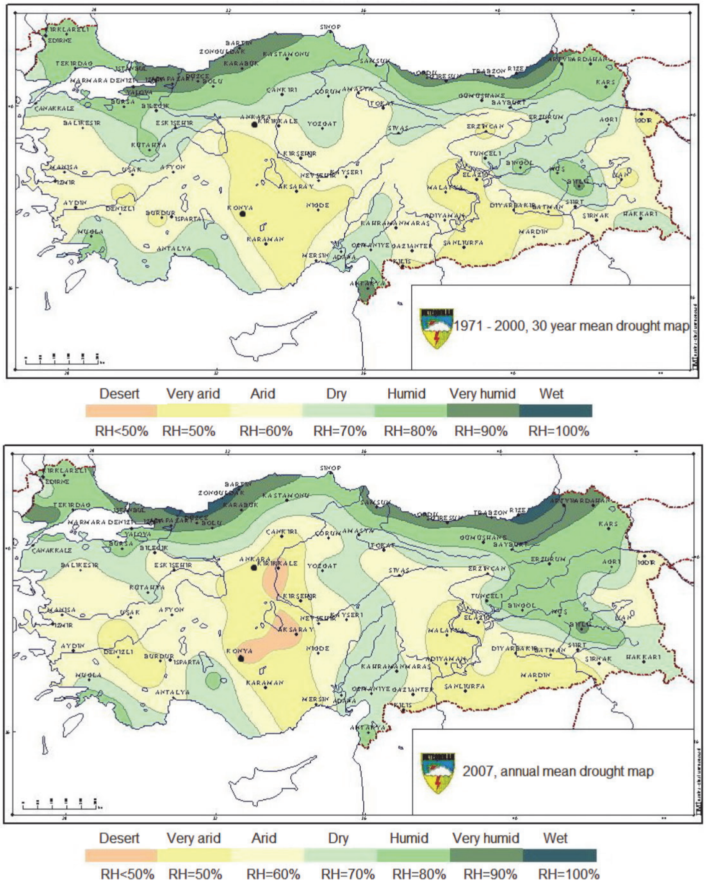


Figure 1 Variation of relative humidity in Turkey. [4]

The project involved the use of B70 type sleepers along the 212 km route between the district of Polatlı and Konya. The service speed along the route is 250 km/h and the design static axle forces for passenger trains and freight trains are 170 kN and 225 kN respectively. Design prestressing force on the ties is 350 kN, which was determined according to the design moment requirements and the projected prestressing losses within the arid climate of Central Anatolia. The ties had design tolerance requirements for their shoulder width of 1813 mm at +2 mm and -1 mm, meaning that the width of measured shoulders should not surpass 1815 mm and should not fall below 1812 mm. The lower value for the negative tolerance was because a reduced shoulder width increased the prospects of flange and railhead abrasion.

Ties produced with C65/80 class concrete had a variable cross section along its 260 cm long length that varied from roughly 530 cm^2 to 330 cm^2 under the rail seat to the centre of the tie. Prestressed design of such an element naturally involved mechanical as well as dimensional considerations for its design. Based on an estimated elastic modulus for the concrete in relation to not only the concrete strength but also the type and mechanical aspects of the aggregate used in the mix design, a preliminary evaluation for the projected longitudinal contractions were conducted based on the detailed empirical procedures presented in the relevant code [3]. Following the transfer of the initial prestressing force into the ties, the contractions were estimated for one year under varying relative humidity conditions as presented in Figure 2.

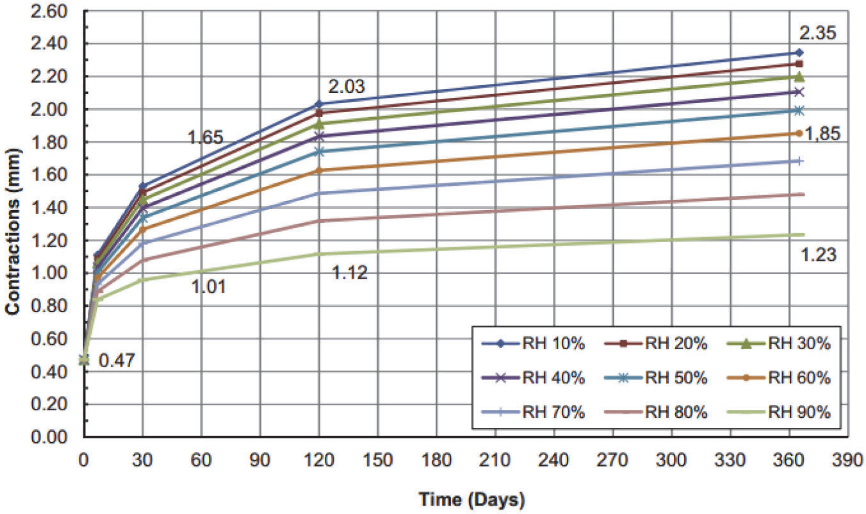


Figure 2 Estimated sleeper shoulder width contractions in one year [5, 6]

These estimates indicated that for all the relative humidity conditions considered that ranged from very wet ambient humidity conditions of RH 90 % to almost completely dry conditions of RH 10 %, within 2 months of their production, all the ties could contract more than 1 mm due to elastic shortening, shrinkage and creep and hence surpass the allowed dimensional tolerances for the sleepers. The estimates indicated that approximately 70 % and 90 % of the total shortening values expected to occur at the end of the 40-year service design lives of the ties occurred within 2 months and 1 year of the transfer of prestressing forces into the ties. These estimates naturally raise a concern with regards to the design dimensions of the moulds.

3 Shoulder width measurements of a tie in time

Following the estimates for the time dependent contractions expected along the ties, an experimental study was initiated to observe the variations in shoulder widths of count-32 ties during a two-month period in 2009. Figure 3 shows the measurement of tie shoulder width with a gauge that has a measuring precision of 0.01 mm. Ties were stored under ambient relative humidity conditions that varied between 60 % to 80 % within open air.

Figure 3 Measuring the shoulder width with a gauge within 0.01 mm. [6]

The nominal shoulder width for the test moulds was set at 1815 mm, which was produced to a tolerance of 0.5 mm. The mean of the shoulder widths as measured from the moulds was 1815.25 mm. Following the transfer of 350 kN ± 5 kN prestressing force into the ties, an initial shortening of approximately 0.5 mm was observed. The very low tensile stress relaxation class prestressing wires selected for this particular application, naturally lost about 3 to 5 % of initial prestressing due to this elastic shortening. Measurement in 1-month and 2-months indicated further shortenings of 0.3 mm and 0.2 mm with respect to the mean values respectively. In two-months, the measurements indicated a total shortening of 1 mm with respect to the shoulder width of the ties as measured within the moulds. Figure 4 shows the shoulder width variations in time for the 32-sleepers. Compared to the estimates presented in Figure 3, the estimates for the 60 % to 80 % relative humidity ranges were about 0.4 mm greater than what was measured at the mean level.

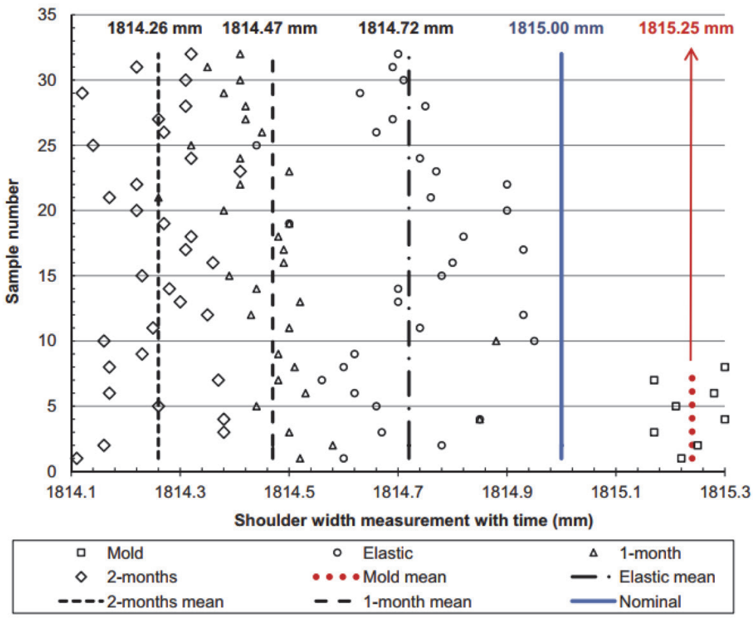
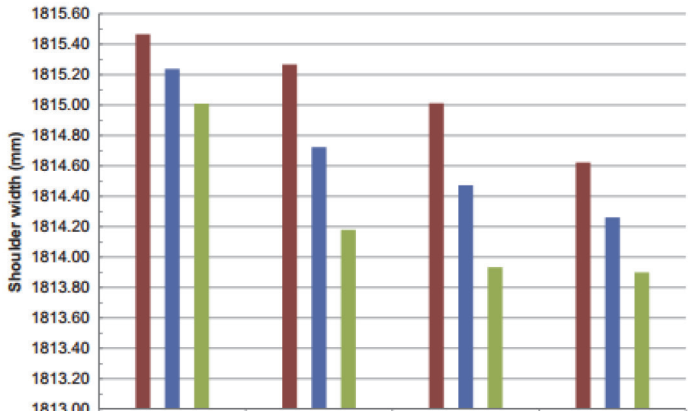


Figure 4 Shoulder width measurements for two months. [6]

However, a nominal design dimension needs to be determined based on a statistical evaluation. Composed fully of geological materials, variations in material properties reflect on to measure properties and the measured shoulder widths of the prestressed railway concrete tie vary as seen in Figure 5. The engineering importance of this variation relates to engineering needs and design tolerances. To this end, Figure 5 presents the likely shoulder width values within a 95 % confidence interval based on the estimated mean shoulder with value and its standard deviation. Such being the case, the expected shoulder width contractions vary between 0.85 mm and 1.2 mm with an average of 1 mm. An engineering design should be able to present the nominal mechanical and geometric qualities of its element to an agreed upon confidence level deemed necessary with respect to the design requirements for functionality and safety.

Finally, the importance of the concrete mix design and the geological quality and grade of aggregate used in the design must not be overlooked [7, 8]. The elastic modulus of concrete, which is a not a simple parameter to measure directly, relates to the aggregate within the strength grades of concrete used to produce the ties today. Despite the preference for volcanic aggregates such as basalt and granite in the production of the ties, geological availabilities and production economy can force designers to use select grades of sedimentary rocks such as limestone. Nevertheless, one must carefully consider the elastic modulus of a concrete along with its compressive strength. The elastic modulus values of two concrete mix designs using different aggregates but yield the same compressive strength can differ.



	Mold	Instant	1-month	2-months
Mean+4,47std	1815.47	1815.27	1815.01	1814.62
Mean	1815.24	1814.72	1814.47	1814.26
Mean-4,47std	1815.01	1814.18	1813.93	1813.90

Figure 5 Statistical variation of shoulder width measurements in two months. [6]

4 Conclusions

Railway engineering is a unique human endeavour that helped to initiate the development of certain sciences such as mechanics of solids and thermodynamics along with soil-structure interaction analysis, geotechnical engineering and metallurgy. Early railway engineers had to pursue into unknown scientific fields and produce solutions to human needs not yet clearly defined and supported by science. Therefore, the historic words, which suggests that "the history of railway engineering is written in blood" certainly has a truth to it such that many failures in the early days of railway engineering occurred due to oversights or lack of knowledge occurred during the development of railways. However, this is not the case today since we have stronger accumulation of knowledge to understand and better tools to analyse and assess railway behavior.
Railway engineering produced the ability to move larger number of people and larger amount of goods to longer distances in reliably defined shorter time intervals with respect to other means of land transportation and therefore magnified the power of human effort to unprecedented scales. This power was at the heart of the first industrial revolution. Today, contemporary railway engineering continues to preserve and extend the power of human effort with higher speeds and higher tonnages and higher service frequencies. Along this increasing demand in railway services, railway engineers need to respond with better materials and better techniques to provide for the needs of this guided means of land transportation.
Structural concrete found its place in almost every civil engineering effort since the end of the Second World War. Today, high performance concrete and other composite materials find their place for use in railway engineering. This paper evaluated a contemporary railway tie and considered the need to evaluate the effect of climate on its engineering properties. Nominal dimensions are an important aspect of engineering structures the precision of which

relates to the needs for functionality. Railway engineering structures such a railway ties, are important structural elements that require a higher order of precision compared to typical structural elements we see in reinforced concrete buildings and bridges. Prefabrication technology today, produces an array of structural elements designed for automated means of production and installation such as railway ties and tunnels segments. The dimensional precision of such elements is frequently specified within one millimetre to provide a proper fit among the structural components among which the concrete element is a part of. One does not encounter such a refined tolerance need for ordinary concrete structural elements, but contemporary civil engineering profession needs to respond to its new design requirements. Concrete structures and especially prestressed concrete structures undergo dimensional changes due to elastic shortening, creep, and shrinkage, which relate to cross section of the structural element, structural stiffness, forces imposed onto the element and the relative humidity of the climate within which the designed structure will serve. Effects of climate and most importantly the effects of climate change must not be ignored in design of contemporary civil engineering structures.

References

[1] Lichtberger, L.: Track Compendium, Track System – Substructure – Maintenance – Economics, Eurail Press, 2011.

[2] Presle, G., Hanreich, W., Paul M.: Austrian Track Testing and Recording Car EM 250 Source for Wheel-Rail Interaction Analysis In Transportation Research Record, Journal of the Transportation Research Board, Transportation Research Board of the National Academies, Washington, D.C., 2000, pp. 22 – 28

[3] Eurocode – 2 – Design of Concrete Structures, 2002.

[4] General Directorate of Turkish Meteorological Studies, MGM, Drought Maps, 1971 – 2000, 2007.

[5] Bezgin, N.O.: Design evaluations for the time dependent contractions of prestressed concrete high speed railway sleepers, 93rd Transportation Research Board Meeting, 14-0322, Washington, D.C, 2014.

[6] Bezgin, N.O.: Climate effects on the shoulder width measurements of prestressed concrete high speed railway sleepers of ballasted tracks, Measurement, 75 (2015), pp. 201–209.

[7] Bezgin, N.O.: High performance concrete requirements for prefabricated high speed railway sleepers, Construction and Building Materials, 138 (2017), pp. 340–351.

[8] Kosmatka, S.H., Kerkhoff, B., Panarese, W.C.: Design and Control of Concrete Mixtures 14th Edition, Portland Cement Association, 2008.

FROM MIXED TRAFFIC TO URBAN TRANSPORT: ACHIEVING LONGEST SERVICE LIVES AND LOWEST MAINTENANCE NEEDS BY TAILOR-MADE RAIL SOLUTIONS WITH SMART MICROSTRUCTURES

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Abstract

The high demands of the modern railway system require – either in urban tramway transport systems, or in mixed traffic transport – the use of track components with correspondingly high resistance against rail degradation to cope with high loads, more passengers and shorter train intervals. Based on the knowledge about the positive effect of fully pearlitic material concepts with C › 0.9% on the performance of rails, a new class of rail steels was introduced in tracks more than 20 years ago; the so-called hypereutectic rail steels. Based on this great deal of theoretical and practical experience, the success story of rail steels with higher carbon content has been transferred from heavy-haul application into mixed traffic and urban railways. The very positive behavior of these Super Premium Rails does not only involve rail wear minimization, but also the behavior of the rails with respect to the formation of corrugation and the development of rolling contact fatigue (RCF) is extraordinarily improved. As there is always room left for innovation, even if highest resistance is provided and despite highest improvements, all pearlitic rail materials react to cyclic loading in the contact zone between the wheel and rail in the form of material fatigue. This is the basis for the rolling contact fatigue mechanism “Head Checking”, which is typical under mixed traffic conditions. In order to minimize maintenance needs, voestalpine engineers have developed a new rail steel that prevents the initiation of head checks - the new steel grade 340 Dobain® HSH®. The 340 Dobain® HSH® rails feature a unique and complex microstructure, which prevents the initiation of head checks. The paper deals with track experiences and the most economic rail grade strategy for mixed traffic as well as urban transport.

Keywords: urban rail transport, rail steel grade, rail degradation, rolling contact fatigue, hypereutectoid rail steels

1 Introduction

Offering solutions to the railway undertakings is the key driving factor for innovative enterprises in the rail rolling business. Therefore, the elongation of service lives, the reduction of maintenance needs and the provision of highest safety is the goal to be pursued. All this is the direct result of a high resistance of the rail material against all kinds of deterioration and damage. In this context, the constant further development of existing rail steels or the development of new rail steels is much more than just courtesy to railways; it is an absolute must in order to cope with the constantly increasing demands of railway operation that all sectors of railroading are facing. The dynamics within the railway business might be an accurate characteristic to describe railroading properly - although this is not visible to the public.

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New rail steels needed at the respective time for sustainable operation have been introduced: As rolled rail steels (alloyed, naturally cooled) at the very first beginning, then premium rail steels (heat treated, sometimes also alloyed) finally culminating in the most modern and most resistant rail steel class, the so called hypereutectoid and heat treated rail steel R400HT (see table 1).

Table 1 Vignole Rail grades according to EN13671-1 [1]

Steel Grade		Chemical Composition [mass %] C	Si	Mn	Cr	Mechanical properties Rm [Mpa]	A5 [%]	Hardness [BHN]
"As rolled" rail grades	R200	0.40 - 0.60	0.15 - 0.58	0.70 - 1.20	≤ 0.15	≥ 680	≥ 14	200 - 240
	R220	0.50 - 0.60	0.20 - 0.60	1.00 - 1.25	≤ 0.15	≥ 770	≥ 12	220 - 260
	R260	0.62 - 0.80	0.15 - 0.58	0.70 - 1.20	≤ 0.15	≥ 880	≥ 10	260 - 300
	R260Mn	0.55 - 0.75	0.15 - 0.60	1.30 - 1.70	≤ 0.15	≥ 880	≥ 10	260 - 300
	R320Cr	0.60 - 0.80	0.50 - 1.10	0.80 - 1.20	0.80 - 1.20	≥ 1,080	≥ 9	320 - 360
Heat treated rail grades	R350HT	0.72 - 0.80	0.15 - 0.58	0.70 - 1.20	≤ 0.15	≥ 1,175	≥ 9	350 - 390
	R350LHT	0.72 - 0.80	0.15 - 0.58	0.70 - 1.20	≤ 0.30	≥ 1,175	≥ 9	350 - 390
	R370CrHT	0.70 - 0.82	0.40 - 1.00	0.70 - 1.10	0.40 - 0.60	≥ 1,280	≥ 9	370 - 410
	R400HT	0.90 - 1.05	0.20 - 0.60	1.00 - 1.30	≤ 0.30	≥ 1,280	≥ 8	400 - 440

What is already anchored in the standard for vignole rails is also evident in the area of grooved rails: Towards the end of the 20th century, voestalpine Rail Technology developed the heat treated grooved rail steel grades R290GHT and R340GHT with improved wear properties. These grades feature a minimum hardness of 290 BHN and 340 BHN respectively, they exhibit a long-standing history of successful use. In addition to these well-established and standardized heat treated grooved rail grades a further optimization and development was achieved, which makes maintenance, in form of gauge corner repair welding, as easy as possible with 290GHT-CL or in best case, avoids it at all with the hypereutectic grooved rail grade 400GHT® (see table 2)

Table 2 Rail grades according to EN14811 [2] and 290GHT-CL and 400GHT

Steel Grade		Chemical Composition [mass %] C	Si	Mn	Cr	Mechanical properties Rm [Mpa]	A5 [%]	Hardness [BHN]
"As rolled" rail grades	R200	0.40 - 0.60	0.15 - 0.58	0.70 - 1.20	≤ 0.15	≥ 680	≥ 14	200 - 240
	R220G1	0.50 - 0.65	0.15 - 0.58	1.00 - 1.25	≤ 0.15	≥ 780	≥ 12	220 -260
	R260	0.62 - 0.80	0.15 - 0.58	0.70 - 1.20	≤ 0.15	≥ 880	≥ 10	260 - 300
Heat treated rail grades	R290GHT	0.50 - 0.65	0.15 - 0.58	1.00 - 1.25	≤ 0.15	≥ 960	≥ 10	290 - 330
	R340GHT	0.62 - 0.80	0.15 - 0.58	0.70 - 1.20	≤ 0.15	≥ 1,175	≥ 9	340 - 390

Steel Grade		Chemical Composition [mass %] C	Si	Mn	Cr	Mechanical properties Rm [Mpa]	A5 [%]	Hardness [BHN]
Heat treated rail grades	290GHT-CL (Low Carbon for easy Gauge Corner Repair Welding)	0.40 - 0.50	0.15 - 0.58	0.70 - 1.10	max 0.15	≥ 960	≥ 11	≥ 280
	400GHT (Maximised service life)	0.90 - 1.05	0.20 - 0.60	1.00 - 1.30	max 0.30	≥ 1,280	≥ 8	400 ± 20

2 Hypereutectoid Rail Steels in track

Hypereutectoid rail steels from voestalpine nowadays have been in use in tracks with high operational demands for almost 20 years. First test installations were established in tracks of heavy haul railways with axle loads of up to 42to, which were soon followed by standard installations no longer for testing purposes, as the tests immediately showed the great potential of this type of rail steels. The very positive behaviour of Super Premium Rails does not only involve rail wear, but also the formation of corrugation as well as rolling contact fatigue. Since their first successful installations in tracks, voestalpine Rail Technology has been producing rails of the Super Premium Class under the brand 400 UHC® HSH® and 400GHT®, where HSH® stands for the world famous heat treatment process of voestalpine Rail Technology in Austria.

2.1 Track performance of 400UHC® HSH® under mixed traffic conditions

Besides various track experiences in other mixed traffic lines, the rail steel 400 UHC® HSH® has been tested in a R=300 m Radius curve in Hungary since Oct. 2015. The performance of the rails has been compared to the rail steel R260 that was implemented in this curve before and has been implemented in the opposite track of this double track section.
The service life of the rail steel R260 has been only 2.3 years (~ 56 MBGT) due to severe side wear on the high rail – see figure 1 [3]. For the 400 UHC® HSH® rails, wear was measured continuously, leading to less than 2 mm of side wear after the same experienced traffic load – see fig. 2

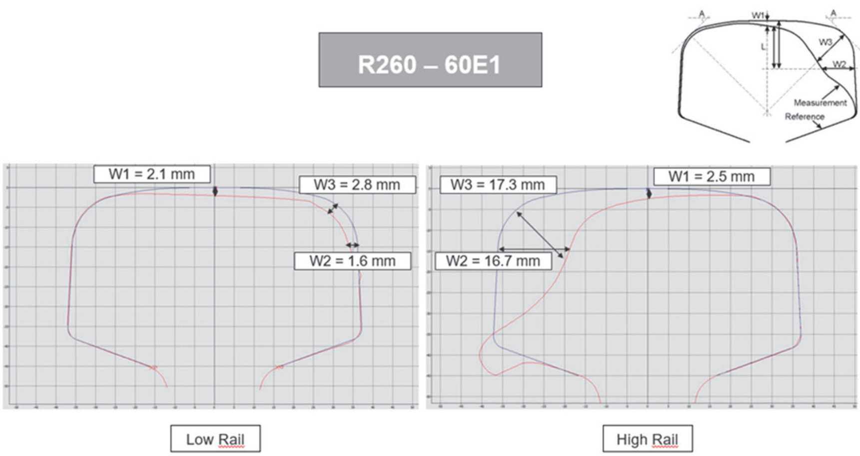

Figure 1 Wear of R260 rail steel in R=300 m curve after 2.3 years (56 MBGT)

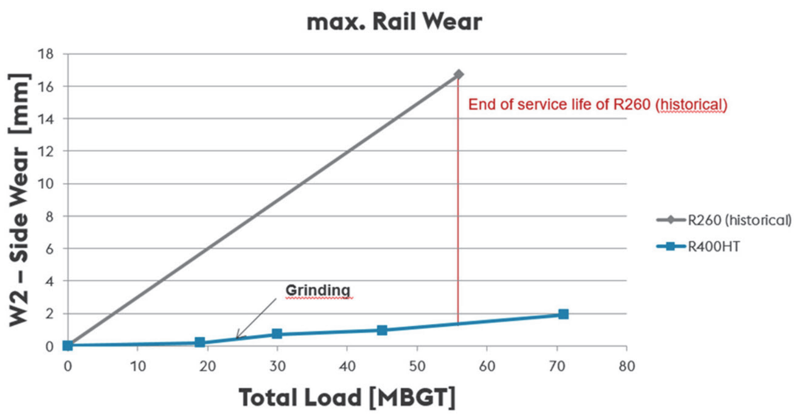

Figure 2 Comparison of rail side wear of 400 UHC® HSH® (R400HT) and R260 rails

Furthermore a significant reduction of corrugation development was observed when comparing 400 UHC® HSH® rails with R260 rails of the opposite track – see fig 3, where measurements of the rail surface at different positions are displayed. R260 already shows significant corrugation of 0.4 mm while corrugation is just beginning after 63 MBGT. For R260 rails, the corrugation has already lead to a degradation of ballast, indicating the necessity of damping.

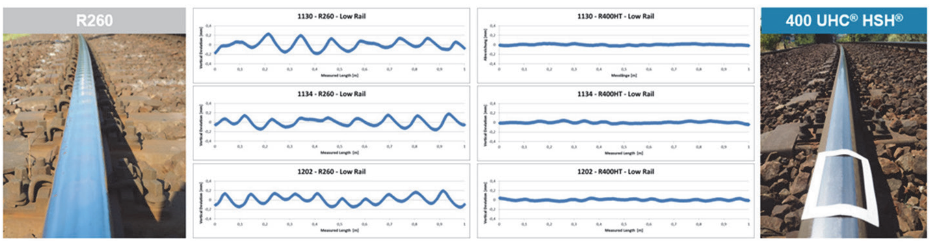

Figure 3 Depth of corrugation of R260 and 400 UHC® HSH® in comparison – reduction by factor 8

The depth of Head Checks was measured with eddy current handheld devices in serveral positions, demonstrating a reduction of the Head Check depth from max. 2.7 mm for R260 to max. 0.6 mm for 400UHC® HSH®. Continous measurement with Eddy current by measuring train showed a reduction HC depth of approximately 300 % in this tight curve.
Based on the measurements a prognosis for the service life and ideal maintenance cycle of 400 UHC® HSH® rails can be deducted by using usual maintenance intervention limits – see fig.4. As can be seen, no rail grinding was conducted for R260 as service life was limited and so short that grinding for corrugation or Head Checks would have been uneconomic.

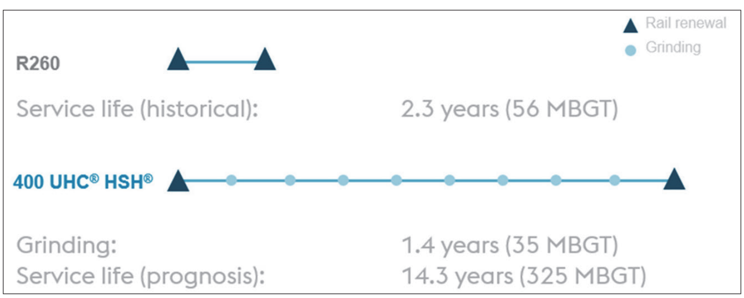

Figure 4 Modelling of service life cycles of R260 and 400 UHC® HSH®

Based on the findings several installations in mixed traffic lines in Europe have been realized. The expected good results have been obtained in all installations. E.g. in Austria die 400UHC® HSH® is used in curves below 600m radius.

2.2 Track performance of 400GHT® grooved rail in urban transport

Many tramways prolong rail service life time by build-up or repair welding (strategy “build-up welding” or “easy-to-maintain”) of the worn rail profile and re-profiling by grinding. With the rail grade R290GHT, which was developed especially for that strategy, two requirements can be fulfilled at the same time: highest resistance against wear (horizontal and vertical) and corrugation and good build-up weldability [4].
Another strategy of application is called “put-in and forget”, where the rail is not maintained during its life cycle by repair welding and remains as it was installed in track until the wear limit is reached. Therefore highest hardness and wear resistance are required. In that case the worldwide hardest grooved rail grade 400GHT® is the best solution.
Based on track experiences from Vienna, Berlin and Warsaw the material behaviour of 400GHT® grooved rail steel is demonstrated in comparison to other state of the art rail steels. Prognoses based on yearlong wear measurements suggest that the service lives of R200 and R290GHT rail steel will be exceeded in all cases, without the need for Gauge Corner Repair welding. The slower development of corrugation is a further performance attribute of 400GHT® leading ultimately to a durable silent track with lowest demands on rail maintenance.
Figure 5 shows a schematic illustration of possible maintenance strategies in urban tramway systems (3). The differences are based on track tests in various tramways, but the real number of possible build-up welding cycles depends on many boundary conditions and may differ in detail [5].

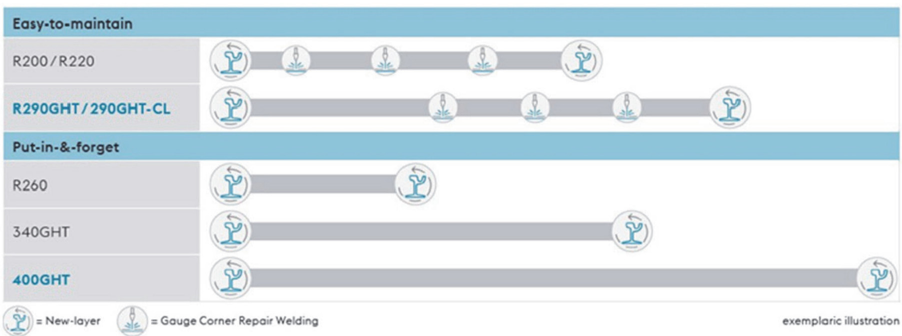

Figure 5 Exemplary illustration of expected track service lives and maintenance cycles in curves

3 New developments

In areas of increased wear and corrugation the 400 UHC® HSH® offers highest resistance against wear and corrugation, in addition it reduces Head Checking to a minimum and makes longest service lives possible but for track sections susceptible for Head Checking like wider curves, a solution which doesn´t form RCF at all is required.
While head-checks are formed during high deformation within conventional pearlitic rail steels, the 340 Dobain® HSH® reduces the deformation of the surface to a minimum. The reason for this is the unique microstructure, which differs fundamentally from that of conventional rail steel (figure 6)

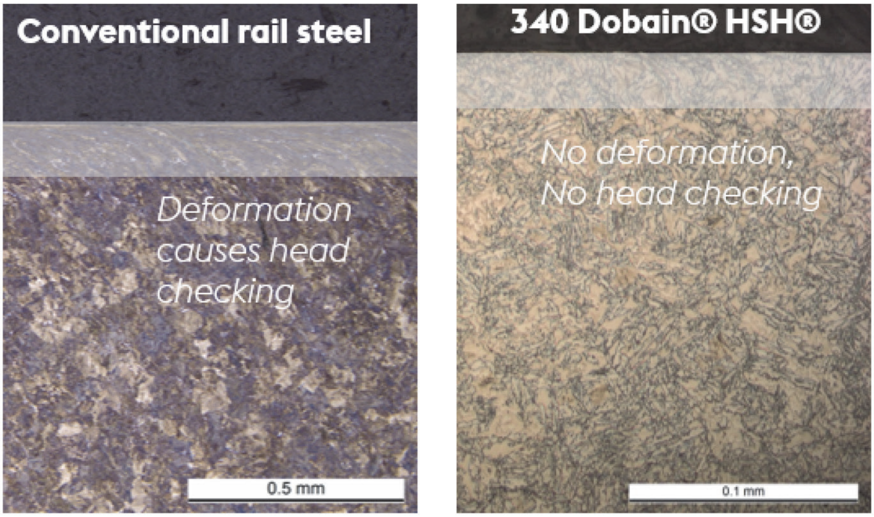

Figure 6 Microstructure of a pearlitic steel compared to 340 Dobain® HSH®.

The Dobain® material concept combined with the well-established HSH® technology creates a unique microstructure that is able to eliminate the mechanism of Head Check initiation on a physical level.
This means more safety in daily operation if grinding is not possible as well as greater economic efficiency for network operators, both due to lower maintenance requirements and longer service lifetime of the rails track.

4 Conclusion

In mixed traffic application the use of 400 UHC® HSH® rail steel reveals huge economic potential in curves up to 700 m. For wider curves which are typically prone to RCF, a future Head Check-free rail steel exhibits a fundamental reduction of rail maintenance and life-cycle-costs. Figure 7 shows the most economical and technical state of the art rail grade strategy for different curve radii.

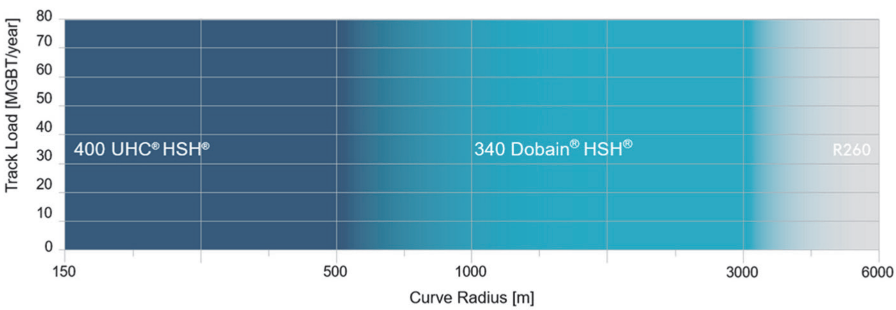

Figure 7 Economical and technical most beneficial rail grade strategy for mixed traffic application

In urban tramway networks the prolonged service life and the low maintenance concept of 400GHT® eliminates the risk of potential failures from Gauge Corner Repair welding in track. As RCF (Head Checks) plays no role in tramway networks the recommended areas of HSH® grooved rails are tight curves below 150 m and station areas, when these are prone to corrugation (figure 8).

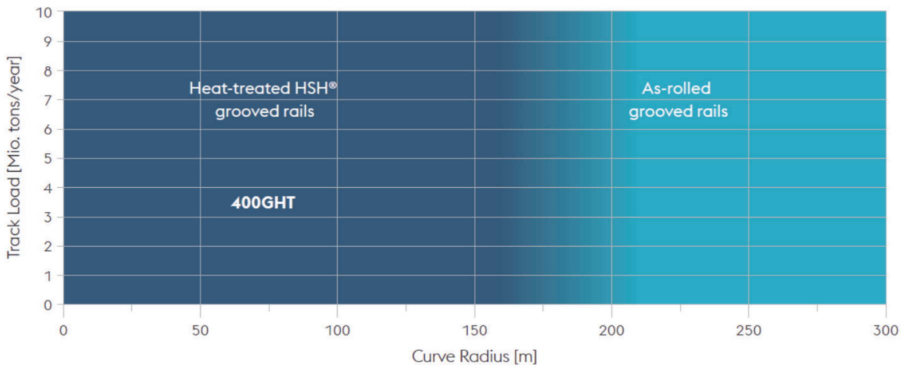

Figure 8 Recommendation according to “put-in-&-forget strategy” for tramway application.

References

[1] EN 13674-1:2017, Railway applications-Track-Rail, Part 1: Vignole railway rails 46kg/m and above.

[2] EN14811:2019, Railway applications-Track-Special purpose rail – Grooved rails and associated construction profiles

[3] Prettner, L.: Cost saving potentials by the use of new rail steels based on life cycle cost considerations, MAUT Conference 2018, Hungary

[4] Wundersamer, J.: Wirtschaftlich optimierter Einsatz von Rillenschienen-Stählen [Economically optimised use of grooved rail steels], RegioTrans, 2014.

[5] Hochfellner, R.: Latest Developments of High Strength Grooved Rail Steels, Core Conference, Melbourne, 2016.

POSSIBILITY OF APPLICATION OF CONCRETE SLEEPER WITH UNDER SLEEPERS PADS

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Abstract

One of the most sensitive segments of the railway infrastructure are certainly bridges. All steel grid bridges have wooden sleeper attached to the bridge structure and cannot change the height when maintaining the upper structure rails. It has become a practice to reconstruct or replace damaged bridges in the past by taking care that the track construction is separated from the bridge constructions. One way is that steel grid structures are replaced with reinforced concrete structures in the form of a trough or if the spans are larger with steel structures in the form of a trough. The standard classical track construction today is a gangway with concrete sleepers and a ballast of crushed stone material. Such tracks have reduced elasticity, which is particularly important at contact between the sleepers and the stone material. In order to reduce the negative impacts of vibration and thus extend the durability of railway track today, railways increasingly apply concrete sleepers with under sleepers pads. Tests have shown that with the installation of the under sleeper pads it is possible to achieve a quieter passage of the train over the substrates of different stiffness, thus reducing the possibility of damage to the track construction and the vehicle itself.

Keywords: concrete sleeper, under sleeper pads, vibration, railway bridges

1 Introduction

The railway network of the Republic of Croatia consists of slightly more than 2600 km of railways. Within the railway infrastructure there are 545 stations, 1512 railway road crossings, 109 tunnels and 538 bridges. The last newly built railways were put into operation during the 1970s, which ultimately means that most of the built facilities are ready for partial or complete reconstructions and replacements. Apart from the need to invest in the modernization of railway infrastructure, it is also necessary to adjust the instructions and regulations in order to open the way for the introduction and maintenance of new products and more modern technologies. Today's rail traffic is significantly different from when railroads were built. The biggest changes are in the axle load of locomotives and freight being transported. As the technical conditions of transport vehicles on the railways changed, so did the railway infrastructure. Perhaps one of the most sensitive segments of railway infrastructure are bridges. Most of the railway bridges of the Croatian network are steel lattice structures that have been strengthened over time to meet traffic needs. Just as the type of rail traffic has changed, so has the way it is maintained. Today, the maintenance of railway infrastructure is based on as much machine work as possible. To maintain the upper structure of the track, machines are used that maintain the level of the track in such a way that they drive the stone material under the sleepers. The result is that after each mechanical driving, the level of the track rises by a few millimeters to a few centimeters.

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All steel lattice bridges have wooden sleepers that are fixed to the bridge structure, and when maintaining the upper structure of the track they cannot change the height. As a result of the impossibility of changing the height of the level on the steel lattice bridges, we get ramps in front of and behind the bridge, which requires a reduction in train speed in the bridge zone for safety and driving comfort.

Figure 1 Individual processing of wooden sleepers on a steel bridge

The maintenance of the tracks on the structures themselves is required because they are wooden sleepers, each of which is treated separately in order to adjust the height of the bridge and the level of the railway. To change the bridge structure on a facility of about twenty meters of track, a railway closure of almost the same number of hours is required. If the closure is not in one piece for the purpose of dismantling and reassembling the tracks, this venture takes longer, which also means a higher cost.

Figure 2 Attaching wooden sleepers to the steel structure of the bridge

In recent years, it has become a practice that when renovating, reconstructing or replacing dilapidated bridges, care is taken to separate the track grid from the bridge structures. One way is to replace steel lattice structures with trough-shaped reinforced concrete structures or if the spans are larger with trough-shaped steel structures. In this way, continuity in the height of the track is obtained, which significantly affects the ease of maintaining the upper structure of the track and, more importantly, over time there is no need to reduce the speed in the bridge zone. Apart from the change in the shape of the structure, the wooden sleepers are replaced with reinforced concrete.

Figure 3 Steel structure of the bridge

Figure 4 Reinforced concrete span structure of the bridge

2 Vibrations caused by the movement of a railway vehicle

When the vehicle moves on the rails, due to the own weight of the wagon and the locomotive (static load of the track) and the dynamic forces that occur on the contact surface of the wheels and the rail, there are vertical oscillations of the rail. The higher the weight or axle load and speed, the more pronounced the intensity of vibrations that propagate from the source (track) into its environment. At high frequencies, the vibration energy propagates through the air in the form of sound waves (noise), while the vibrations of lower frequencies are transmitted in the form of mechanical waves over the rails to the lower parts of the track structure and then to the surrounding ground.

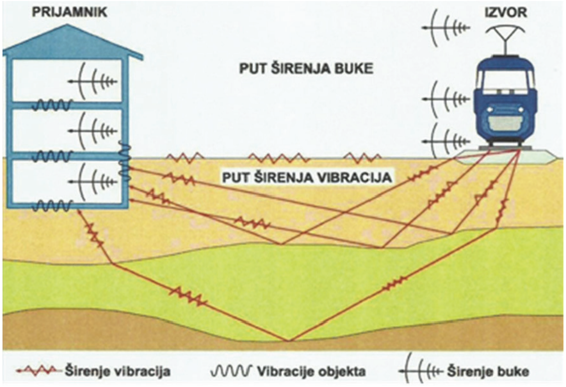

Figure 5 Vibration and noise propagation from railway traffic [1]

2.1 Effect of vibrations on the ballast

The modernization of the railway infrastructure aims to achieve a higher speed of vehicles and increase the carrying capacity of the railway. One of the most common types of classic track structures are certainly tracks with concrete sleepers and a ballast prism made of gravel material. Such tracks, in comparison with constructions made using wooden sleepers, have reduced elasticity, which is especially important at the contact between the sleeper and the ballast material.

With the construction of the railway with the use of concrete sleepers of relatively high stiffness, there were changes in the way of load distribution in the ballast prism just below the sleeper. In general, the gravel ballast prism is the weakest link in the entire track construction because during use, due to dynamic forces on the track, there are lateral dynamic displacements of gravel grains, i.e. mutual compaction and thus gravel degradation. More pronounced and faster degradation and decay of gravel grains is a consequence of the increase in the speed of rail vehicles and their loads and insufficient bending of rails and other elements due to the increased overall stiffness of the track panel.

From the aspect of track maintenance, accelerated track deterioration necessarily requires the implementation of unpopular measures to enable safe and comfortable traffic, which includes: introduction of reduction of train speed, reduction of axle load which negatively affects the transport and capacity of the track) and shortening the regular cycle of maintaining the geometry of the track, thus increasing the cost of maintaining the track.

In order to reduce the negative effects of vibration and thus prolong the durability of the track, today elastic beddings under the rails and elastic rail fastenings are commonly used, and the installation of track support elasticity under the ballast is increasingly used to reduce vibration transmission from the ballast to the lower rail structure, i.e. foundation soil. The disadvantage of such supports is the relatively high price and difficulties in installation (it is difficult to compact the ballast material). Recently, as an alternative or additional possibility to increase the elasticity of the track structure, the installation of a bedding made of elastic, soft material between the sleepers and the ballast has stood out.

2.2 Elastic beddings on the sleepers

The first application of elastic beddings attached to the lower surface of the sleeper was carried out on the Swiss Railways (SBB) in 1986. This solution proved to be better compared to the elastic beddings under the ballasts because it prevented the transition of vibrations to the structure already in the upper parts. The basic function of beddings is to reduce the effect of non-uniform track stiffness on the contact forces and to increase the area over which the sleepers transfer these loads to the ballasts.

Figure 6 Elastic beddings under the concrete sleeper [1]

The installation of beddings reduces the resonant frequency of the track panel and thus reduces the transmission of vibrations from the panel to the gravel material of the ballast and the lower structure of the track. Also, by reducing the vibrations transmitted between individual grains of gravel material, it is possible to significantly reduce its wear, which directly affects the increase in the maintenance period of the ballast.

3 Application of concrete sleepers with elastic beddings in the bridge zone

Due to the stiffness, the existing tracks are built on an inhomogeneous base. Sudden changes in vertical stiffness along the track route are in the zones:
- switch
- curves of small radii
- at road crossing points
- and in the zones of bridges or viaducts.

Significant dynamic forces (shocks) occur in these places, which manifest as an uncomfortable jerk of the train while running and result in accelerated wear of the ballast under the sleepers. In addition, the service life of structures and rail vehicles is reduced.
To prevent unwanted consequences on bridges, concrete sleepers with beddings are used to mitigate sudden changes in bedding stiffness. Tests have shown that by installing beddings on a transition stretch of sufficient length, it is possible to achieve a smoother passage of the train over surfaces of different stiffness, which reduces the possibility of damage to the track structure and the vehicle itself. Transitional stretches are 20 to 30 meters long with beddings of slightly higher stiffness than on the building itself where soft stiffness beddings are installed.

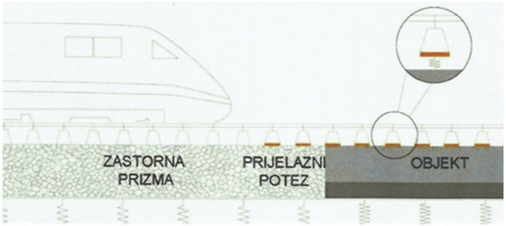

Figure 7 View of transition zones [1]

In the past ten years, at the Department of Railways at the Faculty of Civil Engineering in Zagreb, research has been conducted on the ability to dampen vibrations from individual components of track construction. For testing purposes, a testing site was made consisting of a 30 cm thick slab on which two test structures were laid, each of which consisted of two rails 60 EI, 1.20 m long, fastened with elastic rail fastenings SKL-I for two concrete sleepers PB -85-K. Vibration measurements on a concrete base with and without elastic beddings and vibration effect measurements on the ballast material under concrete sleepers with and without elastic beddings were performed at the test site.

Figure 8 Vibration measurement on a concrete base without elastic beddings [1]

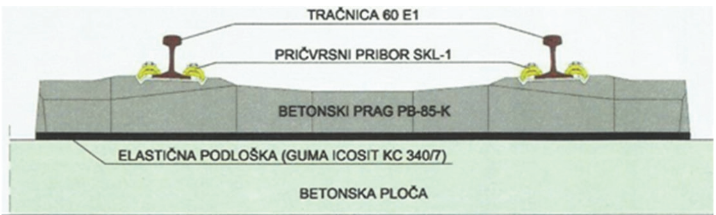

Figure 9 Vibration measurement on a concrete base with an elastic bedding [1]

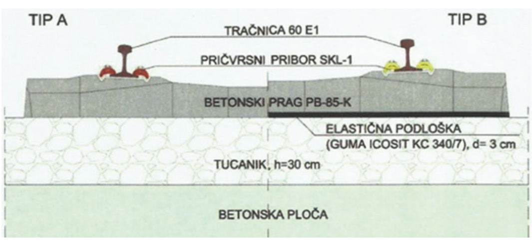

Figure 10 Measurement of the effect of vibration on the ballast material below the concrete sleepers with and without elastic beddings [1]

All the obtained measurements showed that the use of elastic beddings significantly reduces the spread of vibrations on the concrete slab, or on the ballast material. However, the results obtained were obtained on a testing site where the vibrations were caused by a weight and not a rail vehicle. The next step in the vibration propagation test should be performed on the test section where the results obtained would give a true picture of the vibration reduction effect using elastic beddings.

4 Conclusion

By changing the type of bridge construction, it is much easier to maintain the upper structure of the railway on bridges and viaducts, but with the new solution there are new problems of their maintenance. The installation of concrete sleepers with beddings in the area of bridges prolongs the life of the track structure and the facility itself, which ultimately means cheaper maintenance. Tests performed at the Faculty of Civil Engineering in Zagreb should be just the beginning of understanding the meaning of installing elastic beddings under concrete sleepers not only in areas around bridges, but also on other structures (railway crossings, switches, tunnels...). Due to environmental requirements, there are fewer and fewer wooden sleepers on the market that, due to their elasticity, meet the requirements of railways where there is no buffer layer. The results obtained at the test site are a good indicator to continue with the tests on the test section with real load and to examine the possibility of complete replacement of wooden with concrete sleepers with elastic beddings.
Since larger investments in railway infrastructure are planned, it is necessary to invest as much effort as possible in the education of designers, manufacturers, contractors... in order to keep pace with already tested products and well-established technologies for railway infrastructure maintenance.

References

[1] Lakušić, S., Ahac, M.: Betonski pragovi s elastičnim podlošcima, DANI PROMETNICA 2010 – Nove tehnologije i materijali, Zagreb, 2010, pp. 579-615

[2] Lakušić, S., Ahac, M.: Vibracije od željezničkog prometa, DANI PROMETNICA 2010 – Nove tehnologije i materijali, Zagreb, 2010.

[3] Pravilnik 314 o održavanju gornjeg stroja pruga jugoslavenskih željeznica, Beograd 1989

DURABILITY OF REINFORCED-CONCRETE TRACK SLEEPERS

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Abstract

During the regular track inspection of railway lines in various parts of the Republic of Croatia, cracks were found on prestressed reinforced concrete sleepers. These sleepers were installed between 1990 and 2003 in different parts of Croatia, on different surfaces, on railway lines with different intensity and for different types of transport. In order to determine the cause of damage and to estimate the remaining service life of the installed sleepers, the damaged and intact concrete sleepers need to be examined and analyzed. The results have to be compared with the experiences of other railway administrations that have similar problems. Based on the obtained results, it is possible to estimate the service life of concrete sleepers depending on the conditions during operation. It is also possible to estimate how long the damaged sleepers can function as part of the railway.

Keywords: track, track maintenance, concrete sleepers, service cycle concrete sleepers, damaged concrete sleepers

1 Introduction

In railway maintenance, special attention is always paid to track maintenance, which is defined as a structure made of elements of the railway superstructure [1]. It is allowed to install in the tracks wooden sleepers, reinforced concrete sleepers and in exceptional cases steel sleepers [2]. The main purpose of railway sleepers is [1]:
- to evenly carry the load caused by railway vehicles
- to ensure the support and stability of the track base and the track system
- to dampen the railway vibrations and reduce the noise caused by the contact between the wheels of the railway vehicles and the track.

In Figure 1 we can see the prescribed cross-sectional profile of the ballast shoulder of the track in the concrete sleepers [2].

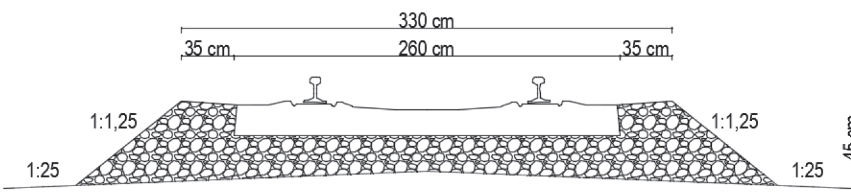

Figure 1 Cross-section profile [2]

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The most common installed type of one-piece prestressed concrete sleeper is B70 with elastic rail system. Reinforced concrete sleepers shall not be installed in the following situations/locations: unstable substructure, rail joints, 30 meters before and after bridges in open construction, on turnouts and crossings, and steel bridges. The installation of both wooden and concrete sleepers is prohibited [2]. The advantages and disadvantages of reinforced concrete sleepers are shown in Table 1.

Table 1 Advantages and disadvantages of the reinforced-concrete sleepers [1]

Advantages of the concrete sleepers:	Disadvantages oof the concrete sleepers:
- Large mass - Typical and fast production - High tenacity - Long-lasting durability - Weather resistance - Ecologically acceptable	- Less elasticity than wooden sleepers - Sensitivity to mechanical deterioration regarding the derailment of the railway vehicles - High dynamic load on the track ballast - More complex maintenance during which the track mechanization is obligatory

2 Technical characteristics of the one-piece concrete sleeper

Reinforced concrete sleepers have been installed in railway tracks since the middle of the 20th century. Currently, three types of reinforced concrete sleepers are manufactured in EU countries: One-piece concrete sleepers (HRN EN 13230-2), two-component sleepers (HRN EN 13230-3) and sleepers for turnouts and crossings (HRN EN 13230-4). Looking at the use all over the world, especially in Japan, the use of reinforced concrete sleepers should also be highlighted.

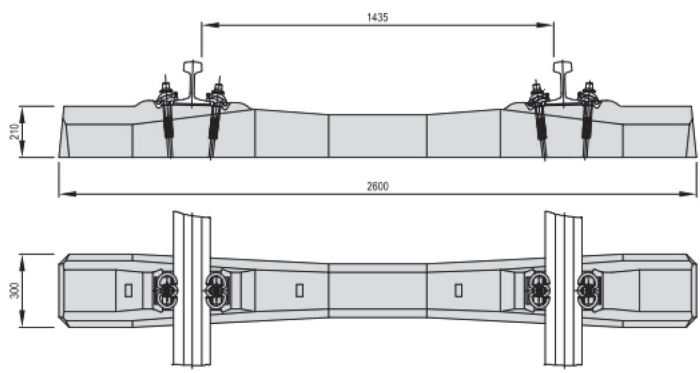

Figure 2 Concrete sleeper type B70 [3]

Elastic track fastening device for a prestressed reinforced concrete sleeper consists of: Sleeper screw with washer - 4 pieces, elastic spring clip - 4 pieces, synthetic corner tiles - 4 pieces, synthetic sub-rail pad - 2 pieces, plastic screw dowels - 4 pieces.

Figure 3 Elastic track fastening device for one rail seat section [4]

3 Analysis of different types of crackings

Longitudinal cracks on concrete sleepers due to exploitation have been found on certain railway sections. The railway sections in question have different ground conditions, they are exposed to different climatic conditions and different volumes and types of railway traffic operate on these sections. Longitudinal cracks, which can be found on concrete sleepers, are becoming increasingly common on many railway lines, both in Europe and worldwide. These cracks run parallel to the longitudinal axis of the sleeper between two fixing points. The location where the cracks occur is the position of the plastic dowel. Two types of cracks were identified during the analysis:

- Type 1: longitudinal cracks along the dowels (connection between a rail and a sleeper), maximum in length 30 cm on both sides, width ≤ 0.5 mm; the cracks are uneven and non-linear.
- Type 2: longitudinal cracks along the entire length of the sleeper, width ≥ 1 mm; the cracks are non-uniform and non-linear

Figure 4 TYPE 1 cracking

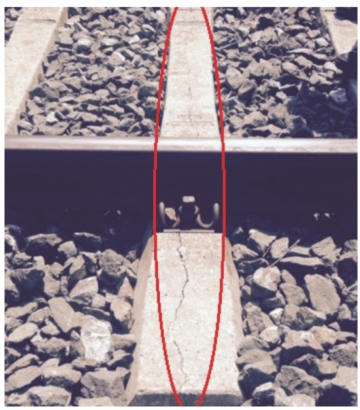

Figure 5 TYPE 1 cracking

3.1 Greek railway experience

During the reconstruction of the Korint – Tripolis – Kalamata section in 2007, longitudinal cracks were detected on concrete sleepers. As a result, an analysis was carried out and the damaged sleepers were inspected and tested. Most of the cracks had spread over the entire height of the sleeper. They started at the point of attachment and widened towards the end of the sleeper [5]. Figure 6 shows longitudinal cracks on the concrete sleeper along the entire cross-sectional plane.

Figure 6 Longitudinal cracking on the concrete sleeper [5]

After observing the problem, an analysis was performed. After analysis, it was determined that the cause of the cracks was high tightening torque of the bolts. It was proven that a tightening torque of more than 450 Nm in combination with a pre- existing pre-stress can cause the crack in the concrete sleepers and the cracks can occur. It is assumed that the cause of the cracking was the failure of the torque measuring device [5].

3.2 Iranian railway experience

After most of the prestressed concrete sleepers were replaced in 2011, the longitudinal cracks were discovered in the same year. The cracks are found where the screws are, and they widen towards the middle and end of the sleeper. Some of the cracks were noticed before the railway was opened to traffic [6]. In Figure 7, the cracking can be seen at the rail bearing point, i.e. the cracking starts in the dowels that were installed in the sleeper.

Figure 7 Longitudinalan cracking in the concrete sleeper

After the problem was discovered, a test was performed on the sleeper, parallel with numerical simulations. After the test, it was found that reinforced sleepers have a higher resistance to longitudinal cracking, as they have better properties than the non-reinforced ones when injected with a larger amount of chemical agents. It was also found that the use of transverse reinforcement bars provided adequate tensile strength to the sleepers. Thus, the longitudinal cracks could be brought under control [6].

4 Reasons for degradation of the concrete sleepers

Due to the degradation of sleepers on certain railway sections before their predicted service life, a large amount of concrete sleepers need to be replaced, which is a lengthy and expensive process [7]. It is also important to know the degree of damage of certain sleepers and the potential possibility of their further use. There are many factors that affect the durability of concrete sleepers. The most common are defects and aging of the fastening system, track ballast in poor condition, and cracking due to dynamic loading [8]. Longitudinal cracking can also be caused by the initial improper loosening of the clamp after the prestressing force is applied, resulting in some prestress in the concrete. In order to minimize high tensile stresses in concrete, compressive stresses are introduced into the concrete from outside by means of prestressing steel, so the value of prestressing force is one of the most important parameters in the design of sleepers. In manufacturing, the prestressing process is carried out before the concrete hardens [9]. Due to prestressing, it is very important to use high strength concretes for the sleepers (at least 50 N/mm2) [10].The dynamic loads to which the concrete sleepers are subjected arise from the interaction of the vehicle wheels and the rails. In practice, these loads reach large amounts when there are irregularities in the wheels, rails, attachments, and vehicle speed. Such a load is represented as a periodic impact load that varies in time [10]. Since the sleeper in the track interacts with other elements of the superstructure and substructure, significant impacts on the sleeper have a track ballast, which has its prescribed dimensions. Very often, the track ballast is contaminated, does not have a satisfactory grain size and is improperly compacted. This has a negative effect on the occurrence of vibrations in the sleeper [11]. The environment in which the sleeper is located also has a significant effect on the damage and reduced service life of the concrete sleeper. This refers to influences such as rain, sun, freezing and thawing, temperature fluctuations, various chemical substances in the soil and air that can damage the structure and reinforcement of the concrete as well as accelerate the degradation of the sleepers. These external influences also affect other parts of the track, which affect the durability of the sleepers [8].

5 Conclusion

The occurrence of longitudinal cracking is a type of sleeper degradation which is present in many railway administrations around the world. This problem has been found to be present all over the world, with various possible causes for the occurrence of longitudinal cracking being discovered. It has been found that the reasons for the occurrence of cracks can be many, some of which include errors in production and transportation, improper installation or installation on an improperly prepared subgrade, and inadequate maintenance of the track or damage caused by faulty rolling stock. To determine the cause of cracks, it is necessary to perform static and dynamic tests on damaged and undamaged sleepers, as well as tests on the material from which the sleepers are made. In addition to the planned tests, it is advisable to develop a numerical model that would serve to predict the occurrence of cracks on concrete sleepers depending on the effects of various external influences.

References

[1] Stipetić, A.: Gornji ustroj željezničkog kolosijeka, Sveučilište u Zagrebu Fakultet prometnih znanosti, 2008.

[2] Pravilnik o održavanju gornjeg ustroja pruga - HŽI 314, HŽ Infrastruktura d.o.o., Službeni glasnik ZJŽ br. 3/71, 5/71, 2/75, 5/76, 8/89 2/90 i 8-9/91; Službeni vjesnik br. 20/91, 5/04 i 8/04

[3] Leonhard moll, http://moll-betonwerke.de/en/, pristupljeno 21.07.2019.

[4] Agico Group, http://www.rail-fastener.com/, pristupljeno 21.07.2019.

[5] Tsoukantas, S. et al: Investigation on the Causes of Longitudinal Cracks on Prestressed Monoblock Railway Sleepers of Metric Gauge of the Greek Railway Network, International Conference on Case Histories in Geotehnical Engineering, Missouri, 2008.

[6] Rezaie, F., Shiri, M., Farnam, S.: Experimental and numerical studies of longitudinal crack control for pre-stressed concrete sleepers, Engineering Failure Analysis, pp. 21-30, 2012.

[7] Mikulić, I.: Analiza uzdužnih pukotina na armiranobetonskim željezničkim pragovima, Diplomski rad, mentor: Lakušić, S., Sveučilište u Zagrebu, Građevinski fakultet, 2018.

[8] Jokubaitis, A., Valivonis, J., Marciukaitis, G.: Analysis of strain state and cracking of concrete sleepers, Journal of Civil Engineering and Management, pp. 564-572, 2016.

[9] Rezaie, F., Bayat, M., Farnam, S.: Sensitivity analysis of pre-stressed concrete sleepers for longitudinal crack prorogation effective factors, Engineering Failure Analysis, pp. 385-397, 2016.

[10] Taherinezhad, M. et al: A review of Behaviour of Prestressed Concrete Sleepers, Special Issue: Electronic Journal of Structural Engineering, 2013.

[11] Damjanović, D. et al: Statičko i dinamičko ispitivanje armiranobetonskih pragova, Dani prometnica 2013. – Mjerenja, ispitivanja i monitoring na prometnicama, Sveučilište u Zagrebu, Građevinski fakultet, pp. 131-158.

CALCULATION OF THE TEMPERATURE DISTRIBUTION IN HEATED SWITCH POINTS

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Abstract

Snow and ice can accumulate between the moveable parts of a switch point during the winter season. As a result he point cannot be switched anymore. In order to prevent failures and delays of trains, switch points are heated. Electrical heating rods shall ensure the melting of snow and ice in the critical areas of a point. Practical experiences have shown that this is not always possible. A calculation model for the heating of a point has to be set up in order to investigate the effectivity of switch point heating systems. Besides that, various ambient factors (such as ambient temperature, wind, precipitation) reduce the heating of the point. However, the extent of impact of the weather conditions on the heating remains to be investigated. Therefore, it is important to study their thermal influence and implement it into the calculation model. The Thermal Network Method (TNM) is suitable in this case. Initially the single main components of a switch point will be set up in separate networks. After a verification with experimental setups, the separate networks can be connected to each other. An experimental setup of an entire model point gives the opportunity to compare calculated and measured heating results without the influence of weather conditions. Finally, the ambient conditions can be implemented into the TNM model by performing field tests. The finished model can give high-resolution temperature information for different heating powers, ambient temperatures, wind velocities, rain and snowfall. According to the practical experience of various railway companies the temperature distribution is calculated for different parameter scenarios and subsequently evaluated regarding its effectivity to prevent failures.

Keywords: switch point, snow and ice, electrical heating, failures

1 Introduction

In the winter months a malfunction of points (railroad switches/turnouts) can occur, due to an accumulation of snow and ice at the moveable components of a point. Thereby, the setting process of a point is prevented and the point cannot be passed by trains anymore. In order to avoid train cancellations and delays, point heating systems are utilised. They have the aim to melt the impeding snow and ice and, therefore, ensure a safe and faultless setting of a point. Electrical heating rods are one possibility to generate thermal energy and feed it into the point. Railway companies have been using them for many years. However, points could not be kept free of snow and ice under certain weather conditions. So, there is a demand for a closer analysis of the heat transfer and the temperature distribution in a point. The different components of a point will be individually investigated thermally and eventually merged to one calculation model. The already existing findings on the temperature distribution in the stock rail [1] constitute the basis of these investigations.

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2 Theory of heat transfer

In order to calculate the heating of a point, three thermodynamic processes for heat transfer have to be mainly considered: heat conduction, convection and radiation. Within one body or at the interface of two touching bodies, heat conduction takes place. The heat transfer is always directed from the location with the respective higher temperature to the location with the lower temperature. The heat flow Pc transferred by conduction can be calculated with the specific thermal conductivity l by Fourier's law (Eq. (1)) [2].

$$P_c = -\lambda \cdot A \cdot \text{grad}\,\vartheta \tag{1}$$

If there is no direct contact between two bodies, thermal energy can be exchanged by heat radiation and convection. In the case of heat radiation, there is no need for a transfer medium. Electromagnetic waves transfer the heat between two surfaces of different temperatures (T1 and T2) according to the Stefan-Boltzmann-law (Eq. (2)).

$$\mathrm{P_r} = \varepsilon_{1,2} \cdot \sigma \cdot A_s \left(T_1^4 - T_2^4\right) \tag{2}$$

The resulting emissivity e1,2 of both bodies depends on the surface condition, whereas s is the Stefan-Boltzmann constant. The convective heat transfer describes the heat exchange between a solid body and a fluid. This process can be devided into the heat conduction from the solid to the fluid and the flow of the fluid. Depending on the kind of the movement, a laminar flow and a turbulent flow can be distinguished. Furthermore, the cause of the flow determines whether it is free or forced convection. There is only density differences that cause the flow for free convection while an external drive like pumps or fans cause a forced convection. The thermal power transmitted by convection can only be calculated accurately for simple geometric conditions. Similarity functions are used to cover all the other cases (Eq. (3)) [1].

$$\mathrm{P}_{CD} = Nu \cdot A_s \cdot \left(\vartheta_1 - \vartheta_0\right) \cdot \frac{\lambda}{l_w} \tag{3}$$

The values for the Nusselt-number Nu and the characteristic length lw originate from experimental investigations, whereas l describes the thermal conductivity of the fluid.
Analogies between electric and thermic networks are suitable in order to calculate the heating of a body. That means electric quantities and their relations to each other can be transferred onto thermic quantities (Table 1).

Table 1 Relation between electric and thermic quantities

Field type	Electric	Thermic
current / heat flow	I	P
potential	φ	ϑ
resistance	R_{el}	R_{th}
potential difference	$\Delta\varphi = U = I \cdot R_{el}$	$\Delta\vartheta = P \cdot R_{th}$
capacity	C_{el}	C_{th}

3 Setup of the Thermal Network Method (TNM) model

The TNM uses the analogies between the electric and the thermic field in order to calculate the heating of a model. Thereby, thermal nodes divide the model into sections. Thermal elements (e.g. resistors for radiation, convection and conduction, thermal capacitors and temperature or heat sources) connect those thermal nodes. The result of a TNM calculation is an assigned temperature to each thermal node and an assigned heat flow through the connection of two thermal nodes. The TNM is capable of calculating models with a high number of thermal nodes within a short computing period. This circumstance favors this method for extensive parameter studies. An additional advantage of the TNM is the possibility to connect and combine single separate networks. This overall approach was also used for setting up the TNM model of the point. An entire point consists of various components. Because the heating of these components was not known so far, they were initially investigated separately. The general procedure is listed below:

- setup of TNM models of single components of a point
- experimental verification of the separate TNM models
- connection of the separate TNM models to one model
- experimental verification of entire point model under laboratory conditions
- implementation of weather conditions and experimental verification in open field scenarios

The main components of a point are the stock rail, the tongue rail, base plate with the slide chair and the track bed (Figure 1). The heating rod is also an important part for electrically heated points and thus has to be modelled thermally. This paper only considers the utilization of one heating rod that is fixed at the foot of the stock rail. The detailed setup of the thermal model for the stock rail and the heating rod was already topic in the in the previous publication [1].

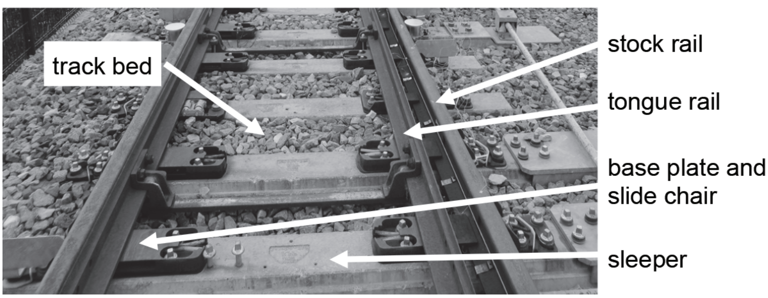


Figure 1 Main components of a point

The TNM models for the tongue rail, base plate, slide chair and the sleeper were set up analogously to the setup approach for the stock rail. Initially, the component geometry was approximated and divided by thermal nodes. While resistors for conduction connect the thermal nodes within the component, resistors for convection and radiation realize the heat transfer at the interface between the component and the ambience. Additionally, the implementation of thermal capacitors at respective nodes enables a time-dependent heating calculation. The thermal parameters of the elements were first estimated according to the literature [1] and adjusted if necessary based on the experimental measurement results subsequently.

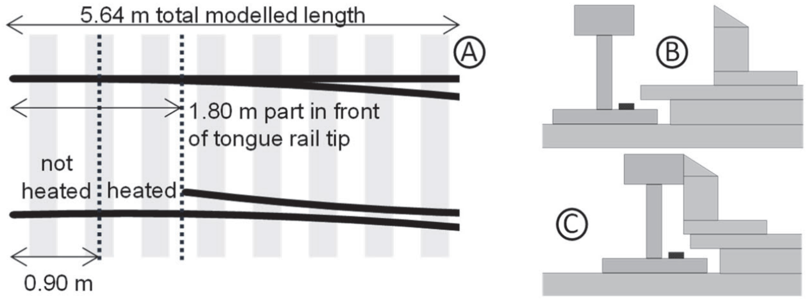
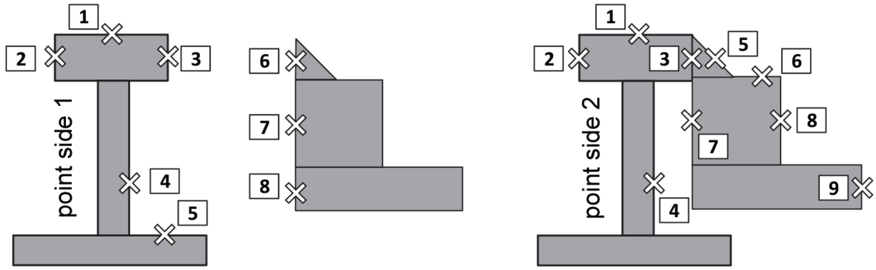


Figure 2 a) Longitudinal sections of the point in the TNM model; b) Point side 1 with detached bearing of stock and tongue rail; C) Point side 2 with touching stock and tongue rail

In contrast to the stock rail, the cross section and the circumference on the tongue rail change in longitudinal direction. In order to model the tongue rail, the TNM models of various sections were set up independently regarding their geometry and connected with each other afterwards. Experimental tests of the sleeper showed that there is no significant heat flow from one side of the point to the other via this component. The examined sleepers were made of concrete. Its low thermal conductivity of l = 0.2 W (mK)-1 [3] causes a thermal decoupling of the point side 2 where stock rail and tongue touch each other and point side 1 where they have a detached bearing from each other (Figure 2). Thus, two separate thermal networks, each for one side of the point, can model the heating of the entire point. This option reduces the number of thermal elements in each model and thereby the computing time. Another advantage of the low thermal conductivity of the sleeper material is the fact that the track bed has not to be modelled as a component. The heat flow from the sleeper into the track bed is rather low and does not affect a noticeable heating of the track bed. That is why, simple temperature sources with ambient temperature are sufficient to emulate the thermal effect of the track bed.
The both TNM models (one for every side) cover a total length of approx. 5.64 m of the point. That also contains an additional section of 1.80 m in front of the tongue rail tip. This section is important for calculating the longitudinal heat flow. The first half of this additional part is not heated. In the second half heating rods are mounted as in the remaining point (Figure 2).
A physical model point served for an experimental verification of the TNM model under laboratory conditions. Heating rods, mounted at the foot of the stock rail, with a power of P' = 300 W/m heated the point until the thermally static state. Thermocouples (type T) at eight or nine different positions respectively measured the temperatures. The comparison between calculated and measured temperatures shows only minor differences (Figure 3).
The measured temperatures confirm the accuracy of the chosen thermal parameters in the TNM model. In order to calculate the heating of a point depending on the weather conditions subsequently, affecting ambient factor have to be analysed and implemented into the thermal network.



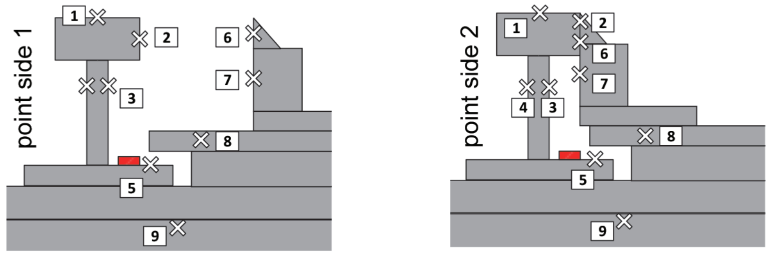
	pos.	1	2	3	4	5	6	7	8	9
point side 1	Q_{meas} in K	30.5	30.2	30.9	38.8	46.5	6.5	6.5	6.3	-
	Q_{calc} in K	32.4	32.3	32.4	37.4	45.0	8.4	8.2	8.7	-
	ΔQ in K	1.9	2.1	1.5	- 1.4	- 1.5	1.9	1.7	2.4	-
point side 2	Q_{meas} in K	34.0	33.2	30.0	44.9	21.5	20.7	22.4	20.7	19.3
	Q_{calc} in K	35.5	35.3	35.5	41.7	21.1	21.0	21.3	21.0	19.9
	ΔQ in K	- 1.5	- 2.1	- 5.5	3.2	0.4	- 0.3	1.1	- 0.3	- 0.6

Figure 3 Measured and calculated temperature rise (Q = J – Jamb) at a physical model point under laboratory conditions for a heating power of P' = 300 W/m and an ambient temperature Jamb = 24.0 °C for point side 1 and Jamb = 23.1 °C for point side 2

4 Implementation of weather conditions

The ambient temperature, wind, global radiation and precipitation can have a great influence on the heating of points in the first estimation. While adjusting the ambient temperature is a trivial process in the TNM model, the other factors need further research. The wind speed affects the convection process. In the presence of wind, the free convection changes to forced convection and the heat transfer coefficient rises significantly. Thereby, it is important to estimate the respective wind speed at the surfaces. On the one hand, the wind speed depends on the altitude and decreases by decreasing height because of increasing friction with the ground [4]. On the other hand, not all surfaces of rail components are affected by the wind to the same extent. Especially at the surfaces between tongue rail and stock rail there is only a reduced influence of the wind due to the cavity. Besides the wind speed, also the direction of wind affects the convectively emitted heating power. The heat transfer coefficient for convection is highest for a perpendicular angle between the longitudinal axis of the rails and the wind direction and reaches its lowest value for a parallel alignment [5].

Global radiation results from direct radiation and diffuse sky radiation reflected by the atmosphere. It represents an additional heat input at the point. Depending on the daytime, season and clouds various surfaces of the point components are affected to varying extents. Several thermal power sources feed the energy of global radiation into the TNM model of the point at respective surfaces.



	pos.	1	2	3	4	5	6	7	8	9
point side 1	Q_{meas} in K	17.9	17.1	19.1	21.9	27.3	3.8	3.8	11.9	0.3
	Q_{calc} in K	16.3	16.3	17.9	17.9	28.2	2.0	2.0	9.7	1.1
	ΔQ in K	1.6	0.8	1.2	4.0	-0.9	1.8	1.8	2.2	-0.8
point side 2	Q_{meas} in K	16.1	15.2	17.8	20.8	29.6	16.7	15.4	11.9	3.5
	Q_{calc} in K	15.4	14.6	19.0	19.0	31.8	14.6	13.1	12.8	1.2
	ΔQ in K	0.7	0.6	-1.2	1.8	-2.2	2.1	2.3	-0.9	2.3

Figure 4 Measured and calculated temperature rise (Q = J – Jamb) for a point under open air conditions

Precipitation can occur in forms of rain or snow. Its thermal effect on the heating of a body is highly complex because different thermal processes work at the same time. Heating experiments with a sprinkled model point were carried out under laboratory conditions. Their results could be used to implement rainfall and snowfall into the TNM model of the point.
All those modifications enabled the TNM model to calculate the heating of a point even by considering weather conditions. A comparison of measured and calculated temperatures validates the accuracy of the model (Figure 4).

5 Analysis of the heating effectivity

The functioning of the set up TNM model offers the possibility to evaluate various heating scenarios. The focus will be on point side 1, because it has the lower temperatures and is therefore the thermally more critical side.
Assuming the area on the inside of stock rail and tongue rail as well as on top of the slide chair should not give water or snow the opportunity to freeze or accumulate, the temperatures should not be less than 0 °C at those positions. Due to possible inaccuracies of the thermal network, the thermal requirement will be 2 °C at the mentioned locations for the following considerations. In order to meet this requirement one heating rod that is mounted on the foot of the stock rail must have a heating power depending on the ambient conditions (Table 2).

Table 2 Required heating power under different weather conditions to reach at least a temperature of 2 °C at the inner side of stock and tongue rail as well as on top of the slide chair

Ambient temperature	Wind	Precipitation	Heating power
0 °C	-	-	< 100 W m^{-1}
-5 °C	-	-	230 W m^{-1}
-10 °C	-	-	410 W m^{-1}
0 °C	15 km h^{-1}	-	540 W m^{-1}
-5 °C	15 km h^{-1}	-	2050 W m^{-1}
0 °C	-	strong rain (5 mm h^{-1})	> 3000 W m^{-1}
-5 °C	-	snowfall (2.5 cm h^{-1})	> 3000 W m^{-1}

This analysis shows that the influence of wind and precipitation increase the required heating power drastically. The chosen values for wind or precipitation have an amount, European railway companies absolutely take into consideration. Conventional heating rods have a nominal power up to approx. 450 W m-1. That means, a moderate wind speed of 15 km h-1 and an air temperature below 0 °C or precipitation at a temperature of 0 °C or below cause significant issues regarding the heating of a point. The analyzed heating system will not be capable to keep the point free from snow or ice. The coldest area was the inside of the tongue rail in all cases. It has the greatest distance from the heat input and so the least amount of heat reaches this area. The majority of heat will be emitted to the ambience before getting to the tongue rail (Figure 5).

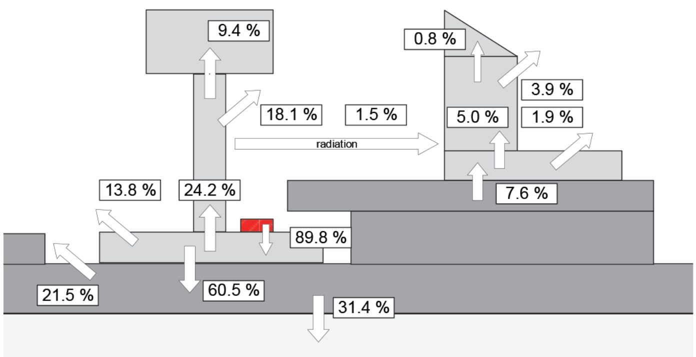


Figure 5 Proportional heat flows between point components (vertical/horizontal arrow) and to ambience (diagonal arrow) without influence of wind or precipitation

6 Conclusion

The verified TNM model of a point could show that for the using of a single heating rod at the stock rail an effective heating operation is only assured if snowfall/rain and wind do not occur at an ambient temperature of 0 °C or below. The point heating cannot prevent the area between stock rail and tongue rail from accumulating ice and snow in the presence of wind or snowfall and a malfunction of the point setting might happen.
In order to improve the heating, a consideration of a distributed heat input e.g. by using multiple heating rods should be carried out. It is difficult to reduce the heat emission to the ambience, so this method helps to transfer the thermal energy to the thermally important areas.

References

[1] Schladitz, M., Adam, R., Grossmann, S.: Set up of a thermal network method model for the calculation of temperature distribution in heated points, 5th International Conference on Road and Rail Infrastructure – CETRA 2018, Zadar, Croatia, 17.-19. May 2018.

[2] Böhme, H.A.: Mittelspannungstechnik - Schaltanlagen berechnen und entwerfen. 2. stark bearb. Aufl. Berlin, Paris, London: HUSS-MEDIEN GmbH Verlag Technik 2005.

[3] VDI-Wärmeatlas: VDI-Buch. SpringerLink: Bücher. 11., bearb. und erw. Aufl. Berlin, Heidelberg: Springer Vieweg, 2013.

[4] http://scienceblogs.de/frischer-wind/2009/10/12/wie-berechnet-man-die-hohenabhangige-windgeschwindigkeit/, 11.07.2019.

[5] Berg, I.: Untersuchungen zur Strombelastbarkeit der Geräte der Elektroenergieübertragung unter Freiluftatmosphäre

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APPLICATION OF THE BEZGIN METHOD TO ESTIMATE DYNAMIC IMPACT FORCES AND JUDGE THE CONDITIONS FOR BALLAST PULVERIZATION AND SLAB CRACKING DUE TO ABRUPT AND RAPID CHANGES IN RAILWAY TRACK PROFILE

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Abstract

Dynamic impact forces occur on railway tracks due to the presence of roughness of the track and the wheel and relate to the train speed and the rate of change of roughness. Variations in track profile and track stiffness and variations in wheel circularity are the causes of roughness. Quantification of the dynamic impact forces is not an easy task due to the complexity of the mechanics of the rolling stock interaction with the railway track. A number of experimental studies have led to an understanding of the dynamic impact forces, yielding a set of conservative and case-specific empirical equations. There are also many calculation-intensive numerical techniques, relying on iterative calculations seeking to converge to a state of temporary equilibrium for the analyzed structural domain within small-time increments. These techniques provide detailed and valuable information for the stresses that develop within the many components of the railway track. However, such numerical techniques rely on expensive computational tools that require experienced users to apply and interpret their results. The sheer amount of representative structural and material data input required to define the analyzed structural domain of the railway track properly is also an important task to accomplish in order to conduct a meaningful analysis. The second author developed a simple analytical method that can provide an accurate analysis for the dynamic impact forces on any railway track relying on track stiffness as the only mechanical railway track parameter. This paper introduces an ongoing study led by the second author and provides an insight into how a designer or a track maintainer can apply the Bezgin Method to estimate dynamic impact forces that may occur in rail-ends and within turnouts. This paper will also discuss how one can judge the conditions for ballast pulverization or slab cracking should these conditions exist.

Keywords: Bezgin Method, dynamic impact forces, rail-ends, turnouts

1 Introduction

Railway vehicles may transfer dynamic impact forces to the railway tracks that are higher than their static loads due to wheel-rail interface irregularities that occur for various reasons related to both track and/or railway vehicle itself. While track profile varies abruptly in the occurrence of insulated, bolted, welded rail joints, and singular rail surface defects [1,2], it changes rapidly along a specific length of the track in turnouts where the train must pass over discrete elements [3-5] such as a switch, crossing, and closure panels. Abrupt and rapid changes in the track profile cause significant dynamic impact forces and dynamic excitation.

The abruptness of the profile variation influences vibration levels and usually higher defects causes an increased level of vibration [6]. Especially corrugation resulting passing frequencies are likely to damage the slab in ballastless tracks and sleepers in the ballasted tracks. Therefore, structural damage related to P_2 dynamic impact forces must be minimized. American Railway Engineering and Maintenance-of-Way Association (AREMA) limits the dynamic impact factor to 200 percent and continuously reinforced concrete slab crack width to 0.3 mm in slab tracks [7].
Higher dynamic forces also damage the ballast layer in abrupt profile changes. The impact attenuation capacity may become inadequate due to excessive bearing pressure, causing ballast pulverization [6]. This damage decreases the structural resistance, drainage capability, service life and requires maintenance tasks which form a significant proportion of the lifecycle expenses. AREMA allows a maximum of 586 kPa (85 psi) ballast pressure under concrete tie for new constructions with high-quality ballast. It also recommends a 448 kPa (65 psi) limit which is more suitable for existing lines [8]. In order to satisfy these limits, dynamic forces should be estimated properly and geometrical variations should be limited accordingly.

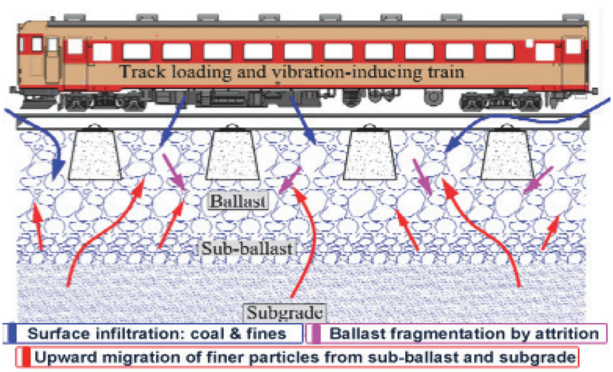

Figure 1 Ballast deterioration and sleeper damage due to dynamic loading [9]

There are some simple empirical equations that take train speed and wheel diameter as an input as well as some experiment-based case-specific relationships to quantify dynamic impact forces. In order to get more realistic results, track engineers and researchers mostly use complex numerical modeling or advanced track instrumentations that may be costly, time-consuming, and requires specialization. In this work, the previously introduced Bezgin Method [10] which is a cost-effective analytical method which takes track stiffness into consideration will be used to estimate dynamic impact forces and evaluate the conditions of railway components at turnouts and rail ends. Further advancements on the estimation of dynamic impact forces due to profile variation, stiffness change, and wheel flats using the Bezgin Method can be found in the relevant resources [11-13].

2 Dynamic impact factor estimation due to abrupt and rapid changes in track profile

This chapter focuses on the application of the Bezgin Method to estimate dynamic impact forces in special locations where profile changes rapidly such as rail-ends and turnouts. Fig. 2 illustrates the passage of a wheel over two different rails with vertical alignment difference (h). In the figure, wheel diameter is 920 mm (3 ft) and rails are type 60E1 with a depth of 172 mm (6.8 in).

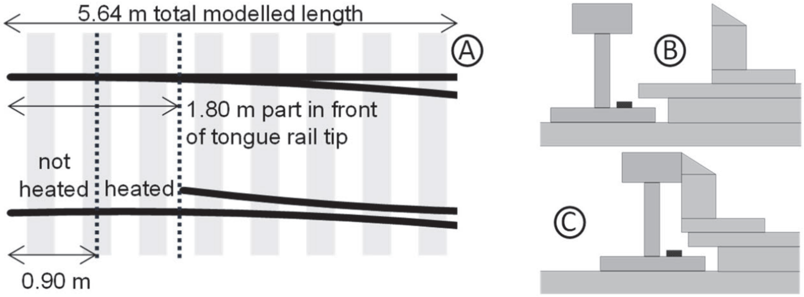

Figure 2 a) Depiction of wheels' passage over a rapidly decreasing profile and b) close-up views

While "g" denotes the horizontal distance between the rails, "h" is the vertical rail elevation and "L" denotes the contact length between the two spots where rail leaves the left rail (A) and drops onto the right rail (B). One can easily measure the track length (L) where vertical elevation (h) occurs in a turnout. However, this is not an easy task for rail-ends. Equation 1 and 2 correlates rail elevation (h) and contact length (L) with the wheel diameter (r) for rail-ends.

$$h = r\left(1 - \cos\varphi\right) \tag{1}$$

$$l = 2r \cdot \sin\frac{\varphi}{2} \tag{2}$$

One can estimate the dynamic impact forces via sets of equations provided by the Bezgin Method using equivalent stiffness of the rolling stock and railway track as an input. Bezgin Method yielded seven equations for cases of track profile and stiffness variations, and wheel flats. In this paper, "The Extended Bezgin Equation for descending track profile (K'B,d)" will be used to estimate the dynamic impact factors in abrupt changes. Equation 3 presents K'B,d where equivalent system deflection is a', impact reduction factor is f, and system damping is s. Equation 4 is the impact reduction factor which relates the free-fall time from "h" track irregularity to the time to pass the irregularity along a transition length (L).

$$K_{B,d}^{'} = 1 + \sqrt{\frac{2h}{a'}\left(1 - f - s\right)} \tag{3}$$

$$f = 1 - \frac{t_{fall}}{t_{pass}} = 1 - \frac{\sqrt{2.h/g}}{L/V} = 1 - \frac{V}{L} . \sqrt{\frac{2h}{g}} \tag{4}$$

Fig. 3 presents dynamic impact forces for abrupt changes at rail ends with h = 5 mm elevation for varying wheel diameter (D), static wheel load (F_S), train speed (V), and equivalent system stiffness (k_{eq}) that is a combination of the stiffness of rolling stock and the railway track. It is seen that the dynamic impact factor increases with decreasing wheel diameter, decreasing static wheel load, and increasing speed.

Fig. 4 presents dynamic impact forces for turnouts with h = 5 mm rail elevation where elevation occurs in limited lengths of L = 30 cm (1 ft) and L = 60 cm (2 ft). In addition to the finding of Fig. 3, it is seen that dynamic factor increases with decreasing transition length (L). The same scale is used in Fig. 3 and 4 to compare the dynamic impact factors of rail ends and turnouts with the same height of profile variation.

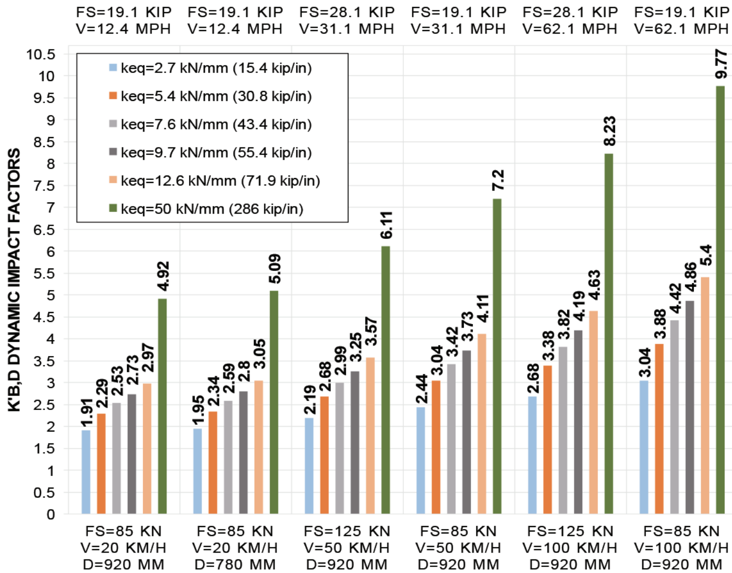

Figure 3 K'$_{B,d}$ impact factors for abrupt changes at rail ends with 5 mm rail elevation

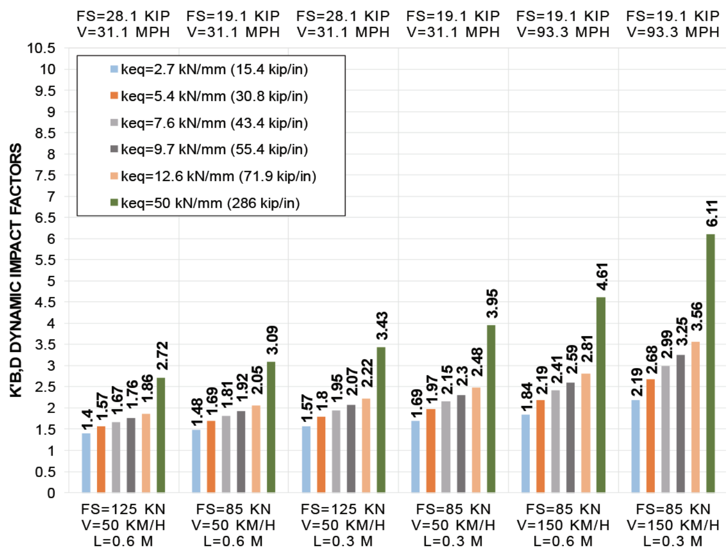

Figure 4 K'$_{B,d}$ impact factors for rapid changes at turnouts with 5 mm rail elevation

3 Evaluation of slab, tie and ballast condition under dynamic impact forces

The last chapter shows that dynamic impact forces may reach approximately up to 10-fold of the static wheel load for the assessed conditions. The quantity of the dynamic impact forces determines the average and maximum contact pressure in the wheel-rail interface, rail base (sleeper) pressure, and sleeper base (ballast) pressure. A 10-fold increase in the wheel forces means a 10-fold increase in the pressure on the bearing elements which may cause the exceedance of bearing stress limits and thus; plastification of the rail, cracking of the sleeper or slab, and pulverization of the ballast. Loss of the ballast material and/or local defects in the rail surface increases dynamic impact forces further and accelerates the deterioration of the track geometry. Therefore, track engineers must evaluate the condition of the bearing elements by comparing the stress levels of the track layers with allowable stress limits set by relevant standards. Fig. 5 and 6 shows the change of maximum sleeper bearing pressure and rail bottom pressure with dynamic impact factors for three different static wheel loads and two sleeper types (B320 and B58) with different base areas. It is assumed that the effective base area of the sleeper is %75 and %50 of the axle force is distributed to the sleeper under the wheel. 1st allowable limit in Fig. 5 refers to maximum allowable pressure on the ballast layer for newly constructed sites with high resistance ballast (0.59 MPa) and 2nd allowable limit is for existing tracks (0.45 MPa), specified by AREMA. The maximum allowable limit of Fig. 6 refers to the allowable concrete stress limit of 32 MPa [14].

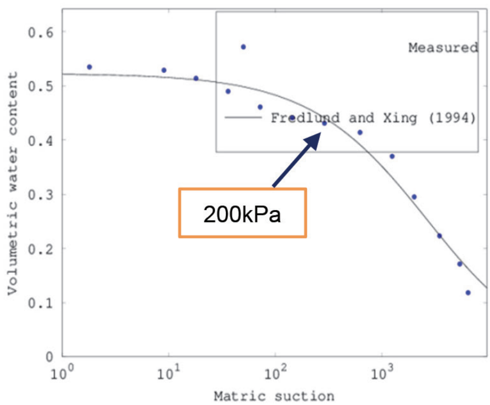

Figure 5 Variation of maximum sleeper bearing pressure with dynamic impact forces for different static axle loads and sleeper types

Studies show that deterioration of the ballast layer is directly connected to the pressure on the ballast layer, without an influence from the seating surface [15]. However, the wider base area of the B320 sleeper type (0,78 m^2) considerably reduced the pressure compared to B58 2.4 sleeper type with the narrower base area (0.61 m^2) for a constant static wheel load. The difference between the two sleepers increased with increasing dynamic impact factors. It is seen that a flawless track riding conditions without any track or wheel roughness does not generate dynamic impact forces and pressures on the track layers are far from the limits and one would not expect ballast deterioration and slab cracking. As the dynamic impact factor goes up, sleeper bearing and rail bottom pressures approach to limits causing increasing loss of friction at inter-particle contact points of ballast material and cracking of the concrete layer. Permanent settlement due to deterioration of track components/layers accelerates the development of impact forces.

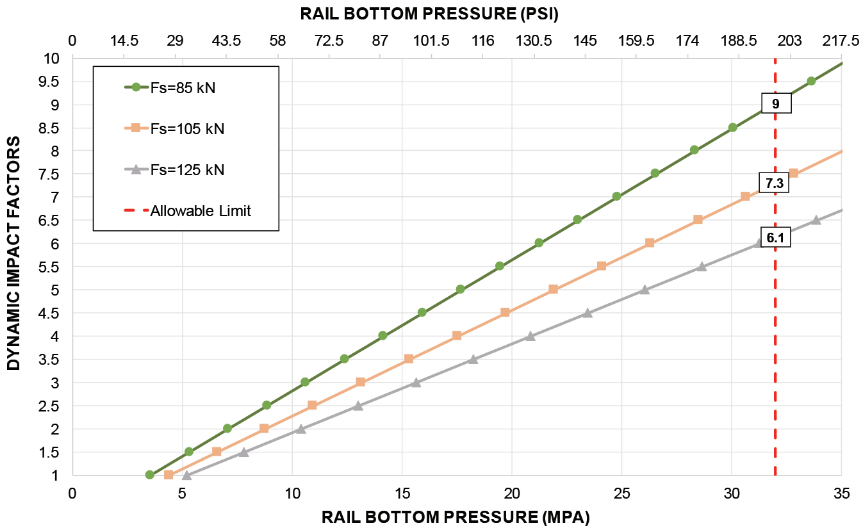

Figure 6 Variation of rail bottom pressure with dynamic impact forces for different static axle loads

Maximum sleeper bearing pressure reaches the allowable sleeper bearing pressure when dynamic impact forces are between 2.2 and 3.9 in newly constructed lines and 1.7 and 3 in existing lines. The same dynamic impact factor becomes riskier when static wheel load is heavier and sleeper base area is narrower. Pressures at the bottom of the rail, on the other hand, require higher magnitudes of dynamic impact factors to reach intolerable limits. However, it is showed in the previous chapter that dynamic impact factors may reach excessive values especially when bogie and wheel suspensions are excluded and abrupt changes occur at rail-ends.

4 Conclusion

In order to maintain track safety, one must be able to estimate dynamic impact forces efficiently so that he or she can compare the effective pressures with allowable limits and judge the conditions of track elements. This paper presented the application of the Extended Bezgin Equation for decreasing track profile developed by the Bezgin Method to estimate dynamic impact forces and judge the condition of ballast, sleeper, and slab when there are rapid and abrupt changes in track profile.

While it is relatively easy to measure the track length in which profile variation exists in turnouts, it may be difficult for rail ends. The paper presented the correlations between profile variation, contact length, and wheel diameter so that one can apply Bezgin Method to the assessment of dynamic impact forces at abrupt changes. Calculations showed that lower static wheel load, wheel diameter, and contact length, and higher train speeds increase dynamic impact forces. factors are found to be up to approximately 6-fold at turnouts and 10-fold at rail ends for given input parameters.

Dynamic impact forces acting on a track directly affects the pressure on the sleeper and ballast layer. Excessive wheel forces may lead to various damages for different track elements such as rail plastification, sleeper cracking, and ballast fouling. The second part of the paper examines the relationship between static wheel force, sleeper base area, dynamic impact factor, and resulting pressures on the track elements. The maximum dynamic impact factor to prevent ballast failure is found to be 3.9 for newly constructed lines and 3 for existing lines for given input parameters. The maximum dynamic impact factor to avoid sleeper failure is found as 9 for given input parameters.

References

[1] Zhao, X., Li, Z., Liu, J.: Wheel-Rail Impact and the Dynamic Forces at Discrete Supports of Rails in the Presence of Singular Rail Surface Defects, Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit, 226 (2011) 2, pp. 124-139

[2] Sun, Y.Q., Cole, C., Spiryagin, M.: Study on Track Dynamic Forces due do Rail Short-Wavelength Dip Defects Using Rail Vehicle-Track Dynamics Simulation, Journal of Mechanical Science and Technology, 27 (2013) 3, pp. 629-640

[3] Sun, Y.Q., Cole, C., McClanachan, M.: The Calculation of Wheel Impact Force Due to the Interaction between Vehicle and a Turnout, Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit, 224 (2010) 5, pp. 391-403

[4] Andersson, C., Dahlberg, T.: Wheel/Rail Impacts at a Railway Turnout Crossing, Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit, 212 (1998) 2, pp. 123-134

[5] Johansson, A., Palsson, B., Ekh, M., Nielsen, J.: Simulation of Wheel-Rail Contact and Damage in Switches and Crossings, Wear, pp. 472-481, 2011.

[6] Kouroussis, G., Connolly, D.P., Alexandrou, G., Vogiatzis, K.: The Effect of Railway Local Irregularities on Ground Vibration, Transportation Research Part D: Transport and Environment, 39 (2015), pp. 17-30

[7] Manual for Railway Engineering, Chapter 8, American Railway Engineering and Maintenance of Way Association, Landover, MD, USA, 2 (1999) 27

[8] Manual for Railway Engineering, American Railway Engineering and Maintenance of Way Association, 2010.

[9] Bassey, D., Ngene, B., Akinwumi, I., Akpan, V., Bamigboye, G.: Ballast Contamination Mechanicsms: A Critical Review of Characterisation and Performance Indicators, Infrastructures, 5 (2020) 11

[10] Bezgin, N.Ö.: Development of a New and an Explicit Analytical Equation that Estimates the Vertical Dynamic Impact Loads of a Moving Train, Procedia Engineering, 189 (2017), pp. 2-10

[11] Bezgin, N.Ö., Wehbi, M.: Advancement and Application of the Bezgin Method to Estimate Effects of Stiffness Variations along Railways on Wheel Forces, Transportation Research Record: Journal of the Transportation Research Board, 2673 (2019) 7, pp. 248-264

[12] Wehbi, M., Bezgin, N.Ö.: Proposal and Application of a New Technique to Forecast Railway Track Damage Because of Track Profile Variations, Transportation Research Record, Journal of the Transportation Research Board, 2673 (2019) 4, pp. 568-582

[13] Bezgin, N.Ö., Kolukırık, C.: Applications and Estimate Comparisons of Bezgin–Kolukırık Equations for Dynamic Impact Forces Because of Wheel Flats with Numerical Analysis Estimates and Instrumented Track Measurements, Transportation Research Record, Journal of the Transportation Research Board, Volume: 2674 (2020) 10, pp. 199-214

[14] Manual for Railway Engineering, American Railway Engineering and Maintenance of Way Association, 2003.

[15] Giannakos, K.: Loads on Track, Ballast Fouling, and Life Cycle under Dynamic Loading in Railways, Journal of Transportation Engineering, 136 (2010) 12, pp. 1075-1084

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THE DEVELOPMENT OF INTEGRATED ROAD CONDITION MONITORING SYSTEM FOR DEVELOPING COUNTRIES USING SMARTPHONE SENSORS AND DASHCAM IN VEHICLES

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Abstract

In developing countries like Timor-Leste, regular road condition monitoring is a significant subject not only for maintaining road quality but also for a national plan of road network construction. The sophisticated equipment for road surface inspection is so expensive that it is difficult to introduce them in developing countries, and the monitoring is usually achieved by manual operation. On the other hand, the utilization of ICT devices such as smartphones has gained much attention in recent years, especially in developing countries because the penetration rate of the smartphone is remarkably increasing even in developing countries. The smartphones equip various high precision sensors, i.e., accelerometers, gyroscopes, GPS, and so on, in the small body in low price. In this project, we are developing an integrated road condition monitoring system that consists of smartphones, dashcams, and a server. There are similar trials in advanced countries but not so many in developing countries. This system assumes to be used in developing countries. The system is very low cost and does not require trained specialists in the field side. The items that are automatically inspected in this system were carefully selected with the local ministry of public works and include paved and unpaved classification, road roughness, road width, detection and size estimation of potholes, bumps, etc., at present. All the inspected items are visualized in Google Maps, Open Street Map, or QGIS with GPS information. The survey results are collected on a server and updated to more accurate values by the repeated surveys. On the analysis, we use several state-of-the-art machine learning and deep learning techniques. In this paper, we summarize related works and introduce this project's target and framework, which especially focused on the developing countries, and achievements of each of our tasks.

Keywords: road condition monitoring, developing country, smartphone, dashcam, deep learning

1 Introduction

The road condition monitoring is an important task for public institutions of developing countries since the road is an essential part of the country's economic growth and social services development. The road condition monitoring can be done in many ways, from manual inspection to sophisticated equipment. As the sophisticated equipment for road surface inspection is too expensive for developing countries, common ICT devices like smartphones can be an alternative utility. The penetration rate of the smartphone is remarkably increasing even in developing countries, and it equips with high precision sensors at a low price. There are differences in inspection items of road maintenance between advanced and developing countries, and we need to select the inspection items carefully. In advanced countries, there are relatively many studies on road conditions, whereas little in developing countries. This study is mainly focusing on the road survey in developing countries like Timor-Leste.

Timor-Leste is the newest country and became independent in 2002, located in Southeast Asia, east of Indonesia. It is surrounded by sea waters and has many mountain areas. Most roads pass through mountains and coastal regions, and 70 % of about 1.3 million inhabitants live in rural areas, and the agricultural sector is the main contributing factor in their daily lives. Presently, Timor-Leste has more than 6,000 km of road network. It comprises 1,426 km of national roads, 869 km of district roads, 716 km of urban roads, and more than 3,000 km of rural roads that are still unpaved [1–3]. More than half of national roads have not been paved. As a tropical region, Timor-Leste road gets damaged by heavy rain. Furthermore, Timor-Leste has many unstable slopes for its steep ground and fragile geology. Frequent landslides and rocks fall during rainy seasons cover the roads and destroy all the road structures. Therefore, in developing countries like Timor-Leste, regular road condition monitoring is a significant subject not only for maintaining road quality but also for the national plan of road network construction in line with the target on a Timor-Leste strategic plan by 2015-2030 [1].

Unlike advanced countries, first of all, the survey has to be conducted following with specifying the segments of paved and unpaved road in Timor-Leste because there remains very old damaged paved road which was constructed when Timor-Leste was a part of Indonesia, and the survey has not finished after the independence. Furthermore, the road width of both paved and unpaved is also an important item to inspect. Therefore, our survey starts from the classification of road sections into ’paved’ and ’unpaved’, and then we estimate the width and roughness level as their status. The detection and size estimation of potholes and cracks are also the items which we inspect. Road facilities such as drainage, bridge, culverts, retaining wall, Gabion and landslide will also be considered in future of this projects.

As related works, Tai et al. studied to utilize smartphone fixed on a motorcycle to collect acceleration and location data, and analysed to detect road anomalies [4]. The identification of braking events and bumps on the road were achieved in [5], as frequent braking indicates congested traffic conditions and bumps characterize the type of road. In [6], the authors developed a mobile application to identify road defects. They discussed the threshold for the detection of defects depending on external factors such as vehicle types, road surface, driving style, and suspension type. The images captured by a smartphone and deep neural networks were used for road damage detection and damage type classification in [7]. In [8], a design of a system for collaborative monitoring of road roughness levels was introduced. In the study, an android smartphone software was developed, and collected data from several types of smartphones, vehicles, and drivers and the results were summarized on Google Map. Cruz J et al. introduced a low-cost road roughness survey system that consists of a smartphone and geographic information system [9]. The system used to estimate the roads roughness and visualize their status on GIS System. Investigation of measured pavement roughness by smartphone with user option is also discussed in [10]. Most of these studies mentioned above focused mainly on developing standalone system inspection using mobile sense and image based on road paved to identify the location of anomalies and estimating road Roughness, and not including road unpaved and estimation of road width.

In this project, we are developing an integrated road condition monitoring system focused on the use in developing countries. The system consists of smartphones, dashcams, and servers for analysis and data visualization. The smartphone and dashcam are set inside of a vehicle, e.g., dashboard, and by adhering to the front window of a car. To estimate the road condition continuously and detect several types of abnormalities of the road, recording data is sent to a server with global positioning system (GPS) information.

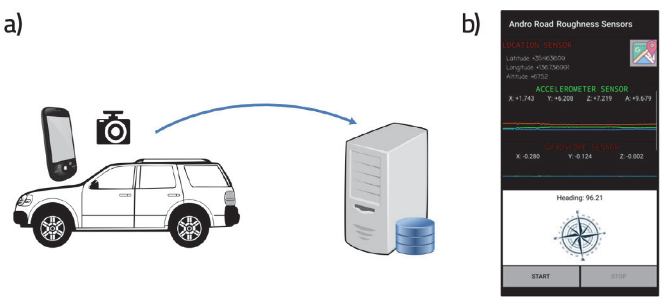
The android smartphone application which we developed will be installed in common types of smartphones, and recording will be achieved with common types of vehicles. The dash-cam is used to record video of the road simultaneously with the smartphone in the running vehicle for visual inspection and other estimation. This system is very low cost and does not require trained specialists in the field side. In this paper, first, we will introduce the whole objective and framework of this project. Then, the progress so far [7, 11, 12, 13] will be described. As machine learning techniques, we have tried several types of them on road surface estimation and compared the performances so far. The results of the comparison of each technique are also discussed in this paper. The team of this project consists of faculty of engineering of Gifu University and National University of Timor-Leste, with Department of Roads, Bridges and Flood Control (DRBFC) of the Ministry of Public Works in Timor-Leste, under the support of the Japan International Cooperation Agency (JICA).

2 Method and result

2.1 Framework of the system

This system consists of smartphones, dashcams, and servers (Fig. 1a). On each smartphone, the android application software which we developed is installed (Fig. 1b). The smartphone is fixed on the dashboard while the dashcam adheres to the front window of the vehicle. The application software monitors sensor data and records the data in 100Hz sampling frequency. Basically, the smartphone does not perform any calculation to prevent it from being hot in long recording and because the raw data size is not so big. On the server, the collected data is analysed and visualized in Google Map, Open Street Map, or QGIS with colours (Fig. 2).

Figure 1 a) Data collection is achieved by smartphones and dashcams which placed on dashboard of vehicles.



The recording data is sent to a server and analysed. b) A screenshot of the android application developed for this project.

The outline of the system is illustrated as a flowchart in Fig. 3. The red boxes indicate the procedures which we already developed for the present, and we are still improving. The orange boxes indicate the procedures which require further improvements. The black boxes indicate we have just started to develop. The system roughly consists of two subsystems. In System 1, the smartphone monitors 11-dimensional time series sensor data, including accelerometer, and we analysed paved/unpaved classification, anomaly detection, and roughness estimation for both paved and unpaved. In System 2, we analyse the video data taken by dashcam, and paved/unpaved classification, pothole detection, pothole size estimation, and road width estimation were performed. The smartphone and dashcam recording are made simultaneously, and we use each data complementary. On the server, averaging is achieved on the same road recording to decreases errors of individual recording.

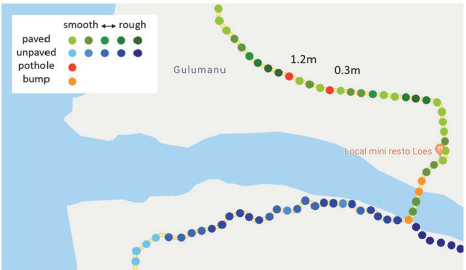


Figure 2 The collected data is visualized on map such as Google map, Open Street Map, or QGIS

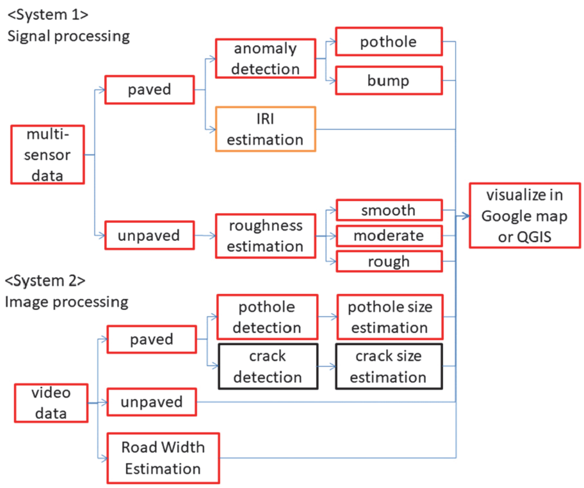


Figure 3 Flow chart of the system. The procedures with red and orange boxes indicate we have already developed. The orange boxes are procedures which require further improvements.

2.2 Data acquisition

The smartphone is fixed on the dashboard of the vehicle by curing tape. The smartphone must be placed tightly on the dashboard to get good analytical results. The android smartphone application which we developed records 11-dimensional data every 10 msec, i.e., acceleration (x, y, z), gyroscope (x, y, z), GPS (latitude, longitude, altitude), compass and timestamp. The dashcam is attached to the front window to take video in front view with 30 fps FHD format.

The data recording was conducted in the Timor-Leste road network, including national roads, district roads, and rural roads. The roads have various types of the condition such as unpaved or paved with potholes and bumps. In practical use, the application will be used by various types of vehicles and with different velocities. To develop a system that is robust for various types of vehicles and speed, we took data using six types of vehicles and with different velocities on the same road. The total length of recording is about 1,000 km of road in Timor-Leste.

2.3 Data pre-processing

First, we make the rotation for the 3D time series data to normalize the posture of the smartphone. We applied 5Hz LPF on each time series data. For classifications, we used a total of 130 features, which include mean, variance, standard deviation, mean absolute deviation, maximum, minimum, root mean square, signal magnitude area, interquartile range, correlation coefficient, energy, entropy, and skewness [11, 13], both in time and frequency domain after applying window.

2.4 Classification of paved and unpaved road

We tried two approaches to classify paved and unpaved roads, that is, signal processing of accelerometer data taken by smartphone and image processing of images clipped from dashcam video. In both of them, we used machine learning techniques. We assume that this system will be used by several types of cars derived by unspecified drivers without specialized training in the usage of the survey system. To achieve the robustness of the system in use, the machine learning techniques on the classification of road status are efficient. We compared the performance of three types of machine learning techniques on the classification of paved and unpaved using accelerometer data, i.e., SVM, HMM, and Residual Neural Network (ResNet) [11]. As a result, we got 97 % of classification precision by signal processing using ResNet. Regarding the image processing approach, we used a convolutional neural network (CNN) [12]. As a result, we could classify the paved and unpaved in various conditions of the road such as wet, muddy, dry, dusty, and shady over 96 % of precision [12]

2.5 Roughness estimation

The roughness estimation is one of the most challenging issues to solve in this type of project because the vibration of the vehicle depends on many factors such as suspension type, tire, body stiffness of car, vehicle speed, the way of driving, and so on. There are many trials to estimate International Roughness Index (IRI) using a smartphone, but usually, they require calibrations or car type manual selection. In this study, we are trying to estimate individual parameters also by machine learning. At present, we are using a regression function which manually found for each vehicle.

2.6 Anomalies detection

The existence of anomalies such as potholes and bumps on paved road sections are examined after the classification of paved unpaved road. As detection algorithms, we compared i) the combination of K-Nearest-Neighboured (KNN) and Dynamic Time Warping (DTW), ii) the combination of KNN and Euclidean Distance, and iii) SVM. As a result, we got the best performance by using i) the combination of K-Nearest-Neighboured (KNN) and Dynamic Time Warping (DTW) for detection [11].

2.7 Pothole Detection by image processing

This task is conducted after the classification of paved and unpaved roads on the road images. All the images are clipped from the dashcam video every 10m. The paved road images then feed to the model for detection of existing potholes. The amount of 13,244 training set and 3.250 validation set images were used for building the model. We used LeNet 5 [14] based model for this task. We compared our proposed system with other conventional machine learning methods such as Support Vector Machine (SVM) to evaluate the effectiveness of the proposed system,. As a result, SVM got 88.2 % of accuracy. On thesother hand, our model outperformed the SVM method by achieving 99.8 % accuracy [7].

2.8 Road width and pothole size estimation

We identified the area of width and pothole in the front view image taken by dashcam by semantic segmentation by deep learning. To estimate the real size of them, we need to transform the front view image to bird's-eye-view image. For the transformation, we have to find the vanishing point to estimate the depression angle of the dashcam. However, unlike advanced countries, we cannot use Hough transform in rural areas of developing countries. We proposed to use optical flow to find the vanishing point [15].

2.9 Visualization

The final task of the procedure is to visualize the analytical results in a map such as Google Map, Open Street Map, or QGIS. We are using each of them depending on the situation because each of them has advantages and disadvantages. Figure 3 is an example of a visualization of the analytical results in Timor-Leste.
Figure 4. (a) Visualization of the analytical results in Timor-Leste about road type and roughness in Open Street Map. (b) First, the road type is categorized into paved and unpaved and roughness is estimated in each road. Anomalies such as bump or pothole are also detected.

3 Summary and discussion

In developing countries, the utilization of the smartphone on social development has much potential. This is because the smartphone has a variety of high technology sensors and network connectivity in a small body, and the penetration ratio of the smartphone on the nation is very high even in countries with weak economic infrastructure. The main tools of this project are smartphones, dashcams, and state-of-the-art data science techniques. As the inspection items are different between advanced and developing countries, we had many discussions with the local public department in charge of road maintenance. In this paper, we introduced the whole picture of this project and achievements in Timor-Leste. The system can be used in other developing countries by fine adjustment according to each situation.

References

[1] Timor-Leste Strategic Plan 2011–2030. Available online: http://timor-leste.gov.tl/wp-content/uploads/2011/07/Timor-Leste-Strategic-Plan-2011-20301.pdf, 12.03.2020.

[2] Building a Sustainable Rural Road Network in Timor-Leste, https://dfat.gov.au/about-us/publications/Documents/roads-for-development-ilo-ausaid-concept-note.pdf, 12.03.2020.

[3] Tai, Y., Chan, C., Hsu, J.Y.: Automatic road anomaly detection using smart mobile device, Conference on Technologies and Applications of Artificial Intelligence, Hsinchu, Taiwan, November 18-20, 2010.

[4] Bhoraskar, R., Vankadhara, N., Raman, B., Kulkarni, P.: Wolverine: Traffic and road condition estimation using smartphone sensors, Communication Systems and Networks (COMSNETS), Fourth International Conference on, pp.1–6., 2012.

[5] Byrne, M., Isola, R., Parry, T., Dawson, A.: Identifying road defect information from smartphones, Road and Transport Research, 22 (2013), pp.39-50

[6] Pereira, V., Tamura, S., Hayami, S., Fukai, H.: A deep learning-based approach for road pothole detection in timor leste, Service Operations and Logistics, and Informatics (SOLI), 2018 IEEE International Conference, pp 1–6, 2018.

[7] Alessandroni, G., Klopfenstein, L.C., Delpriori, S., Dromedari, M., Luchetti, G., Paolini, B., Seraghiti, A., Lattanzi, E., Freschi, V., Carini, A., Bogliolo, B.: Smartroadsense: Collaborative road surface condition monitoring, Proc. of UBICOMM-2014. IARIA, pp. 210–215, 2014.

[8] Cruz, J., Castro, J.: Developing a Low-cost Road Roughness Survey Methodology using Smartphones and Geographic Information System (GIS), 2014

[9] Yeganeh, S.F., Mahmoudzadeh, A., Azizpour, M.A., Golroo, A.: Validation of Smartphone-Based Pavement Roughness Measures, CoRR abs/1902.10699 (2019)

[10] Cabral, F.S., Pinto, M., Mouzinho, F.A., Fukai, H., Tamura, S.: An Automatic Survey System for Paved and Unpaved Road Classification and Road Anomaly Detection using Smartphone Sensor, IEEE International Conference on Service Operations and Logistics, and Informatics (SOLI), pp. 65–70, Singapore, 31 July–2 August 2018.

[11] Pereira, V., Tamura, S., Hayami, S., Fukai, H.: Classification of paved and unpaved road image using convolutional neural network for road condition inspection system, Advanced Informatics, Concepts, Theory, and Application (ICAICTA), 2019 IEEE International Conference on, pp. 1–9, 2019.

[12] Cabral, F.S, Fukai, H, Tamura, S.: Feature Extraction Methods Proposed for Speech Recognition Are Effective on Road Condition Monitoring Using Smartphone Inertial Sensors, Sensors 19 (16) 2019

[13] LeCun, Y., Bottou, L., Bengio, Y., Haffner, P.: Gradient-Based Learning Applied to Document Recognition, Proc. of the IEEE, November 1998.

[14] Fukai, H., Mouzinho, F.A.L.N., Nagae, R., Uchida, M.: Development of automatic road width and pothole size estimation method from dashcam video for under developing countries, CETRA 2020 (in press)

GPR INVESTIGATION ON DAMAGED ROAD PAVEMENTS BUILT IN CUT AND FILL SECTIONS WITH RETAINING WALL

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Abstract

We present the GPR results dealing with flexible road pavements located on cut and fill sections with retaining wall. The aim is to evaluate the road damage (particularly the ramified cracks) taking into consideration also other parameters (cut and fill section height and traffic load). The GPR evaluation was carried out on 20 sites selected in the secondary urban road network of L'Aquila, a medium-size urban area representative of the Abruzzi Region (Central Italy). Stress induced by traffic load generally affects a road section thickness of about 1.0 m from the ground; so a monostatic GPR antenna, with a nominal frequency of 2000 MHz, was used given that its maximum inspection depth corresponds to 1.0 m from the ground. The 2000 MHz antenna has also a quite high-resolution when inspecting road damage. The GPR acquisition was carried out in damaged and adjoining undamaged road sites, to compare the GPR data of the two areas. GPR data analysis was based on the sweep-rectified power approach to evaluate the radar signal attenuation curve vs. depth, which permitted us to single out different road types of damage and to discuss the factors which caused them.

Keywords: damaged road pavements, GPR, durability performance of road pavements, inspection, maintenance strategies

1 Introduction

We report the results carried out by using GPR (Ground Penetration Radar) technique on flexible road pavements built in cut and fill sections with retaining wall. The goal was to find correlations between the types of paving deterioration, the cut and fill section height, the traffic load and the results obtained from the GPR campaign. Previous studies have always evaluated different types of paving deterioration with a reduced number of GPR scans. The results were interesting but not definite, due to their extremely small number [1].
During the current research, to increase our data we acquired five GPR measurements for each variable evaluated (cut and fill section height, traffic load). With regards to the type of deterioration inspected, the survey was conducted only on the ramified cracks (also called spider or alligator cracking) that represent one of the five types of bearing capacity structural defects regarding flexible road pavements [2].

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2 The research goals

The research goals were to carry out a survey with the GPR technique of degraded road pavements with a single type of deterioration, carried out on the cut and fill sections with retaining wall. The ramified cracks (RC) deterioration analyzed were selected to represent the worst conditions. Longer fractures measuring 5 m and wider than 5 mm were identified and, regarding the extension of degraded areas, more than 5 m^2 extended pavements were identified.
Regarding the influence of the height of the cut and fill section with retaining wall were considered pavements consisting of high cut and fill sections with retaining wall (HCF), a height of › 4 m and wall height › 3 m, and on low cut and fill sections with retaining wall (LCF), a height of ‹ 2 m and wall height ‹ 2 m.
Regarding the influence of the intensity of the traffic were considered pavements subject to heavy traffic load (HT), with average daily traffic › 4000, and low traffic load (LT), with average daily traffic ‹ 1000. Once again dealing with traffic loads, a further verification was performed which took into consideration the diversity of the flow of vehicles in transit on the roads surveyed, in order not to neglect the effect of heavy traffic. This verification was not focused on the average daily traffic load, but rather on the equivalent standard axle loads (ESALs - each of the 120 KN) per year. The evaluation confirms the results obtained in the first study and identified the following traffic load classes: heavy traffic load (HT) with › 400,000 ESALs/yr and low traffic loads (LT) with ‹ 4,000 ESALs/yr.
Therefore, the inquiry was performed on twenty sites chosen to be representative of the different combinations of the variables analysed. The same sites were identified on the secondary urban road network located into L'Aquila (Central Italy). Be informed that the studied area is placed at an altitude of 700 m above sea level and is characterized by cold winters.
The superstructures of the evaluated roads are constituted by flexible road pavements which have similar layers, with the following thicknesses: layer of foundation in granular mixture: 30 cm, base layer in bitumen mixture: 10 cm, binder layer: 6 cm and surface layer: 4 cm, both consisting of a bituminous concrete. Therefore, the superstructures on average have a total thickness of 50 cm.
The GPR surveys were carried out to perform a quantitative analysis through the examination of the GPR signal attenuation curves with the depth (the rectified power method). An antenna module with a nominal frequency of 2000 MHz was used, which is a type of evaluation that is quite reliable if directed at a depth of up to 1.0 m. This choice was made based on the effect, due to the stresses induced by traffic on cut and fill sections with retaining wall, to a maximum depth of about 1.0 m, developing, however, in the most accentuated form on an average in the first 50 cm of the superstructure [3]. In this current study, we used a 2000 MHz butterfly antenna manufactured by Systems Engineering - IDS (Pisa). It is a portable, monostatic type, non-dispersive antenna and is characterized by linear polarization, low directivity and limited bandwidth.

3 Analysis with sweep rectified power method

As it is known, the propagation of an electromagnetic field is described by the Maxwell equations, in which the constant of attenuation α appears, which expresses the amount of energy that is absorbed by the intersected layers. Please remember that the larger the void ratio of the evaluated material the greater the attenuation of the radar signal and the lower the attenuation constant is α [4]. Therefore, by determining precisely the α attenuation constant, it is possible to obtain a positive evaluation of the depth of the signal penetration itself. Since the goal of our study is to evaluate the importance of structural defects, we decided to focus the evaluation on a maximum depth of 1.0 m from the road pavement. This choice was made because the effect, due to the stresses induced by traffic on cut and fill sections with retaining wall, to a maximum depth of about 1.0 m, developing. However, in the most accentuated form on an average in the first 50 cm of the superstructure and spreading, with a still evident and easily seen result, for an additional 50 cm. Therefore, the theoretical reference road section was schematized in two portions: the first, 50 cm (S1), representing the superstructure and the second, an additional 50 cm (S2) representative of the portion of section formed by the ground which is still affected by the traffic [5]. This choice was in line with the demands that the antenna resolution of GPR had adopted (2000 MHz). Another fundamental assumption adopted in the model was that layers S1 (road pavement) and S2 (subgrade) are considered homogeneous on average. This assumption does not accurately reflect the reality, especially for the S1 layer, given the granulometric variety, specific weight and form that characterize the materials used in road pavements.

The sweep rectified power analysis, carried out in our study using software created by IDS-Gred (© 2004 IDS Ingegneria Dei Sistemi SpA, Pisa), graphically represents the average trend (straight line attenuation) energy absorbed by the ground portion of the cut and fill section with retaining wall placed between 50 and 100 cm (S2).

Through this interpretation of the rectified power diagrams, you can trace the α attenuation angle that graphically represents the angle that envelopes the R^2 regression line of power, forms with the x-axis which, in turn, indicates the depth from the road surface. Regarding the surveys carried out in our study, please note that the analysis was carried out considering two contiguous stretches in length equal to 1.5 m belonging respectively to a damaged area and an undamaged area from degradation and it not present specific abnormalities, for not vitiating the comparison. The relative diagrams for the two contiguous sections surveyed (both damaged and undamaged area) were included in the same graph to highlight their differences. The red coloured diagrams are related to the degraded road sections, while those of green coloured diagrams belong to the intact portions [6]. By making comparisons between attenuation corners of damaged areas (α_d) and undamaged (α_u), it is expected that if the difference between these values ($\Delta\alpha = \alpha_d$-α_u) tends to zero, then the probable cause that generated the deterioration of paving is attributable to phenomena of fatigue or thermal shrinkage. These phenomena are due to horizontal tensile stresses that develop in the S1 layer of the road pavement (Figure 1a). In fact, in this case, in the S2 layer the energy curves are almost coincident, which means that the subgrade terrain relating to the two examined road sections, display the same degree of densification and there are no compactions in place (Figure 2). Moreover, the breaking of the surface layers in S1 may depend on deep settlement (rotation and translation), which originate on the base of the detecting portion of the embankment on the side of the retaining wall of the cut and fill section.

If instead, $\Delta a \neq 0$ the probable cause that triggers the degradation of the road pavement is attributable to the change of soil compaction portion of the section present in the S2 layer due to the action exerted by vehicular traffic. In this circumstance, we must make a further distinction between the case in which $\Delta a > 0$ and the case in which $\Delta a < 0$. In the first, the energy curves for the damaged areas (red) are at a lower energy content than those undamaged (green). This means that the deteriorated areas are more compacted of the not deteriorated areas. More precisely, the energy curves for the damaged areas (red) are positioned beneath those undamaged (green), and this confirms the fact that a lower power consumption level corresponds to an index of lesser void ratios (Figure 3). Therefore, the degradation process is no longer in place in the deteriorated area, while the areas that are not deteriorated will tend, over time, to assume the same level of densification of damaged areas; then the entity of degradation will tend to not remain confined in the deteriorated area but to expand in the neighbouring areas (Figure 1b). In the second, the energy curves for the damaged area (red) are at a higher energy content than those which are undamaged (green), and that the deteriorated areas are less compacted than non-deteriorated areas. More precisely, the energy curves for the damaged areas (red) are positioned above those undamaged (green), and this confirms the fact that at a power level absorbed corresponds a higher void ratio greater (Figure 4). Therefore, the degradation process is still going on in the deteriorated area and continues until it reaches the level of densification of the area not deteriorated; for this, the degradation will remain confined in the area affected by deterioration (Figure 1c).

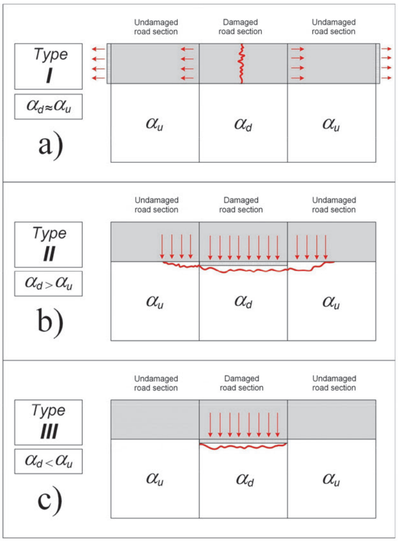

Figure 1 Diagram illustrating the three types of deterioration

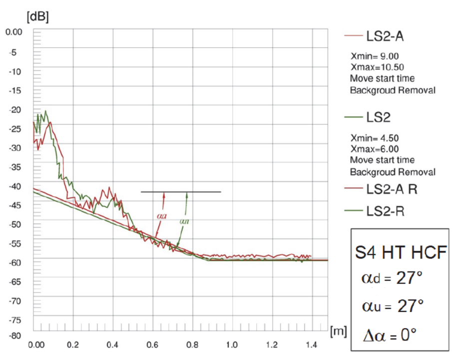

Figure 2 Example of the Δa @ 0 case

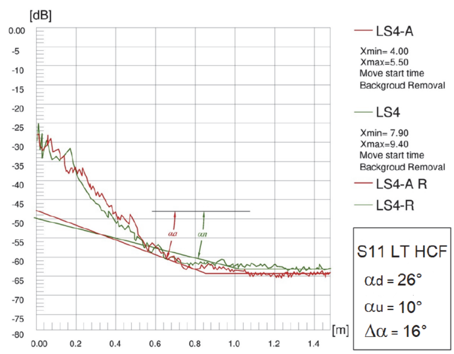

Figure 3 Example of the Δa › 0 case

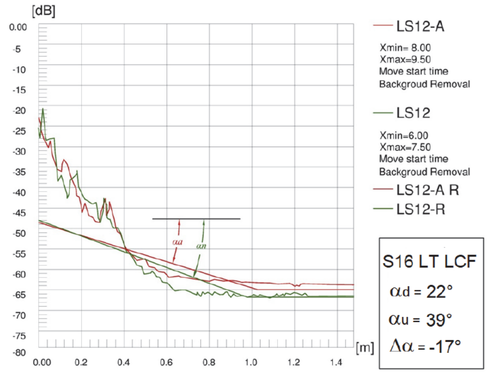

Figure 4 Example of the Δa < 0 case

4 Survey results

We analyse the results of surveys carried out with GPR in the twenty sites chosen which are representative of the various combinations of the variables studied.
In this regard, it is noted that the accuracy in the interpretation of radargrams is a function of the antenna resolution and sampling the electromagnetic signal; in our case they are respectively of 1 cm and of 1024 samples per second. Furthermore, the resolution in the power sweep diagrams is of 1 dB for the signal attenuation (y-axis), and of 4 cm from the road surface to the depth (x-axis). This approach provides a detailed resolution to be able to evaluate areas up to 1 cm^2 in a thickness of road pavement/cut and fill section up to the depth of 1.0 m (Figures 2, 3, 4).
With the assistance of the sweep rectified power analysis, we proceeded to the evaluation of signal attenuation angles is in damaged sections (α_d), and in those undamaged (α_u), to which was followed by the evaluation of the variation of the attenuation (Δa). In Table 1 we report, the various combinations tested, the values of the attenuation constant a and the index Δa calculated. Please note that any survey is characterized by a code indicating, for the type of deterioration evaluated (ramified cracks RC), the height of the cut and fill section with retaining wall (HCF, LCF) and traffic load (HT, LT), according to the preceding paragraphs.
The analysis of the results made it possible to divide the types of degradation in two categories: the first consists of the resulting deterioration due to problems inherent in the road pavement layer (layer S1), and the second consists of the interesting deterioration of the subgrade layers (layer S2).
The deterioration in the first category (breaking in S1 layer) have provided values of Δa content in a range, respectively, of $-2 < \Delta a < +2$.
In this regard, the analysis of the values reported in Table 1, it was found that for road sections characterized by high traffic (HT), both high cut and fill sections (HCF) and low cut and fill sections (LCF), we were obtained the same types of results concerning the rupture in layer S1, both with 3 sections out of 5. For the findings, we can state that, in the light of the considerations set out in the preceding paragraph, the deterioration caused in the S1 layer (Figures 1a, 2) are normally generated by horizontal tensile stresses due to fatigue or thermal shrinkage affecting the surface layers of the road pavement. Generally, the rupture of the road pavements depends from the undersize of layers respect to traffic loads. But the traffic can also generate sagging of the supporting wall (rotations and translations) due to fatigue stress caused by the repeated passage of vehicles (Figure 5).
As said, we can deduce that the soils present in the cut and fill sections (up to 1.0 m deep) are not, in this case, the cause of no damage of the cracking type. From the analysis of the results

in the first category (breaking in S1 layer), it is clear a high repeatability resulted from the values obtained from the measurements with the GPR (both 60 % with HCF and LCF sections with HT), demonstrating that GPR is reliable for evaluation of deteriorated road surfaces and provides good results. Regarding the deterioration of paving in the second category (rupture in the S2 layer), they were recorded values of Δa included in a large interval of -17< Δa <+16. As found, it can be said that the deterioration regarding the layer (S2) are generated by the sudden change in density from the subgrade terrain produced by the action exerted by the vehicle traffic (soil compaction).

Table 1 Different values of α_d and α_u measured and the index Δa calculated

Traffic load	Height	α_d	α_u	Δα	Damaged layer
HT	HCF	30	32	-2	S1
HT	HCF	28	33	-5	S2
HT	HCF	26	33	-7	S2
HT	HCF	27	27	0	S1
HT	HCF	27	27	0	S1
HT	LCF	22	31	-9	S2
HT	LCF	27	27	0	S1
HT	LCF	31	29	2	S1
HT	LCF	28	30	-2	S1
HT	LCF	23	32	-9	S2
LT	HCF	26	10	16	S2
LT	HCF	25	15	10	S2
LT	HCF	10	13	-3	S2
LT	HCF	12	15	-3	S2
LT	HCF	21	21	0	S1
LT	LCF	22	39	-17	S2
LT	LCF	27	34	-7	S2
LT	LCF	35	36	-1	S1
LT	LCF	31	37	-6	S2
LT	LCF	27	40	-13	S2

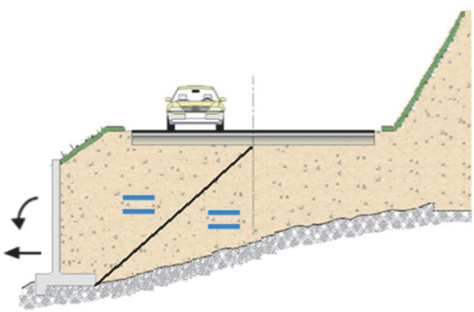

Figure 5 Cut and fill section with retaining wall and traffic

In this case please note that if $\alpha_d > \alpha_u$ the deterioration will tend to expand in neighbouring areas; if instead, $\alpha_d < \alpha_u$ the deterioration will remain confined to the already degraded area. It is noted that, in the case of low traffic (LT) and both high cut and fill sections (HCF) and low cut and fill sections (LCF), for 4 sections of 5 were recorded the highest values of Δa and 6 with a negative sign. This means that the problems inherent in the compaction are limited to the layers close to the substrate and not extend throughout in the neighbouring damaged area of the cut and fill sections. Generally, the rupture depends from landslides that interest the subgrade terrain of the roadway. Maybe this shows that the road damaged are conditioned by unpredictable external factors such as, for example, the influence of the subgrade composed of natural terrain positioned close to the road pavement.
Also, in this case the results obtained show a high repeatability with 80 % in the case of homogeneous results HCF and LCF with LT showing that it is a reliable methodology for evaluation of road pavements deteriorated even with different conditions of traffic and sections.

5 Conclusions

The results of a GPR survey conducted on degraded road pavements built on cut and fill section with retaining wall is presented. It was evaluated only one type of deterioration (ramified cracks) and was carried out a survey on a large sample (five surveys) for each of the evaluated variables (sections height and traffic load). These pavements have been identified on secondary roads of the urban area of the city of L'Aquila (central Italy). The evaluation of the attenuation curves of the radar signal detected from the road pavement, performed by the sweep rectified power method, allowed us to determine the attenuation constant a. Through the comparison of the attenuation of constant a, detected on two adjacent longitudinal sections belonging respectively to a damaged portion (α_d) and to an undamaged stretch (α_u), made it possible to trace the causes of the degradation of the analyzed pavements. The results, taken from a large sample, showed high repeatability to demonstrate that the methodology can be trusted to evaluate deteriorated road pavements with different traffic conditions and sections heights with retaining wall.

References

[1] Colagrande, S., Ranalli, D., Scozzafava, M., Tallini, M.: GPR signal attenuation vs. depth on damaged flexible road pavements, 4th International IWAGPR Workshop on Advanced Ground Penetrating Radar, Napoli, Italy, June 2007.

[2] Lahour, S., Al-Qadi, I.: Automatic detection of multiple pavement layers from GPR data, NDT & E International, 41 (2008), pp. 69-81

[3] Huang, C., Su, Y.: A new GPR calibration method for high accuracy thickness and permittivity measurement of multi-layered pavement, 10th International Conference on GPR, Delft, The Netherlands, 2004.

[4] Benedetto, A.: Theoretical approach to electromagnetic monitoring of road pavement, 10th International Conference on Ground Penetrating Radar, Delft, The Netherlands, 2004.

[5] Benedetto, A., Benedetto, F., Tosti, F.: GPR applications for geotechnical stability of transportation infrastructures, NDT & E International, 27 (2012) 3, pp. 253–262

[6] Colagrande, S., Ranalli, D., Tallini, M.: Damaged flexible road pavement inspection using GPR, World Conference on Pavement and Asset Management, Baveno, Italy, 12-16 June 2017, pp.89-95

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DEVELOPMENT OF AUTOMATIC ROAD WIDTH AND POTHOLE SIZE ESTIMATION METHOD FROM DASHCAM VIDEO FOR UNDER DEVELOPING COUNTRIES

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Abstract

Road condition monitoring usually requires extremely expensive special vehicles, equipment, or many human resources. On the other hand, with the development of ICT and data science technologies in recent years, there are several research trials in which the heavy technical tasks of road asset condition monitoring are replaced by automatic inspection systems consisting of common devices such as smartphones and dashcam videos. As the system consists of low-price devices, it also suitable for developing countries. However, there are many differences in the situation and the inspection items on road condition monitoring between advanced countries and developing countries. There are few trials to develop such a road condition monitoring system in developing countries. Our project is developing an integrated road condition monitoring system focusing on developing countries like Timor-Leste. In developing countries, many parts of the road are still unpaved, and the “road width” is an important item to be inspected. In this paper, we discuss the road width and pothole size estimation as a part of the integrated system we are developing. We survey the road width of both paved and unpaved roads. We use a common dashcam to take video along the road. The estimated values are integrated into a database with GPS information and visualized in Google Map, QGIS, or the original visualization system which we developed. To estimate the real width of the road and pothole size, we need to transform the captured forward view image of dashcam video into bird's-eye-view. For the transformation, we need to estimate the vanishing point in a captured image. However, unlike the advanced countries, it is difficult to detect the vanishing point in developing countries because there are usually no straight lines in the images in the unpaved road of the province. In this study, we propose to use the optical flow method to detect the vanishing point in the rural road. To identify the area of road and the existence of potholes in images, we apply state-of-the-art semantic segmentation using deep learning.

Keywords: developing countries, road condition monitoring, road width, pothole, dashcam, semantic segmentation, deep learning

1 Introduction

A huge budget is allocated for regular road asset condition monitoring to maintain its essential infrastructures in each country and local government. In advanced countries, there are several sophisticated ways of road surveys. However, they are too expensive to introduce in developing countries. In developing countries, road survey is usually achieved manually by humans, but the manual way has problems such as time-consuming, requires many human resources, individual variation of the results, a troublesome task of data organization, and so on.

By the way, the penetration rate of the smartphone is remarkably increasing even in under developing countries. The smartphone has many high-performance sensors, such as accelerometers, gyro sensors, GPS sensors, communication functions, touch panels, etc., in the small body at a low price. The utilization of the sensors is receiving increased attention in recent years. This is the central part of a stream of ICT framework for Developing Countries (ICT4D).

In our project, we are developing an automatic road surface condition survey system using smartphone sensors, a dashcam, and a server for calculation and database [1], [2], [3], [4], [5]. This approach is advantageous to under developing countries because the system consists of a common cheap smartphone and dashcam, and no expert knowledge or trained skills are required for inspectors on the field site. These types of trials have been conducted so far in advanced countries [6], [7], [8], [9], [10]. However, not many in developing countries as far as we know.

This study is conducted by the collaboration of faculty of engineering of National University of Timor-Leste and Gifu University in Japan, with the ministry of public works of Timor-Leste under financial support of Japan International Cooperation Agency (JICA). The survey items of the road surface are different between advanced and developing countries like Timor-Leste. For instance, more than 50 % of national road is still unpaved in Timor-Leste [11], [12], and we need to specify the unpaved sections and also road width of both paved and unpaved road for the national plan of road construction. The survey items of our whole project include paved and unpaved classification, road width estimation for both paved and unpaved, roughness estimation for both paved and unpaved, pothole detection and its size estimation, crack detection and its type and size estimation, markers on road surface detection, rutting estimation, roughness (IRI) estimation, etc. All of the inspection items are estimated by analysis of time series data acquired by smartphone sensors and dashcam video, using signal processing, image processing, and state-of-the-art deep learning techniques.

This paper focus on the road width estimation for both paved and unpaved especially in developing countries, and pothole detection and its size estimation from dashcam video.

2 Methods

2.1 Video recording of road using in-vehicle dashcam

We can use any common dashcams that can easily get in any country to take the video. However, there are several requirements for use in this study.

First, the dashcam must have functions to get GPS signal and embed the GPS information in the video file. Second, the angle of view of the lens must be specified. In this study, we used Transcend® DrivePro™ 230. The dashcam is attached to the front window as usual. We used full HD 1080P, 30 fps, and MOV (H.264) format as the specification of the video file. The viewing angle for the lens of DrivePro™ 230 is 130° (diagonal).

2.2 Extraction of GPS value from video and clipping of frames every 10 m

The information of date, time, and GPS is embedded almost every 1 second in the video file. In this study, we extract the information every 1 second by binary analysis and estimate the vehicle velocity from the GPS values. The estimated vehicle velocity is used to clip the frames of the video every about 10 m.

2.3 Estimation of the vanishing point in the frame by optical flow tracking

It is necessary to convert the captured image to the real scale for the estimation of the real road width and pothole size, that is, bird's-eye-view transformation (Fig. (1)).

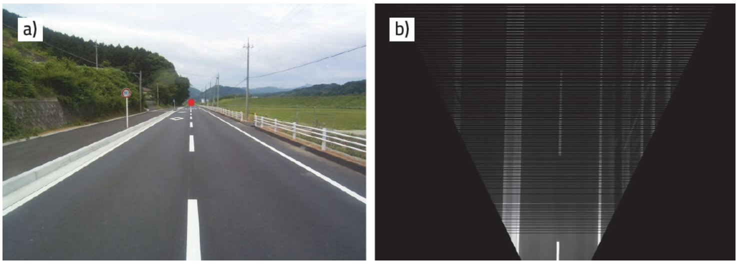

Figure 1 (a) Original dashcam image taken in Japan, (b) Transformed bird's-eye-view image

For the conversion, several parameter values are required. We can know the angles of view θ_H, θ_W from the specification of the dashcam (Fig. 2). The pixels of width and height of the video frame W, H can be set manually on the recording. We measure the height of the lens from ground level on the recording. However, we need to estimate somehow the depression angle of the optical axis of the lens of the dashcam from the videos or images taken by the dashcam. To estimate the depression angle, firstly, we need to find the vanishing point in the image.
We estimated the depression angle of dashcam using pixel sizes of the image. Let *D* be pixels from the center horizontal line in the image to the vanishing point v (Fig.2). Then, the depression angle θ_C is calculated as the following formula:

$$\theta_C \sim \sin\theta_C = \frac{2D}{H}\tan\frac{\theta_H}{2}$$

Here, we need to identify the vanishing point v. The vanishing point can be estimated as an intersection of multiple straight lines in the image (Fig. 1(a)). Usually, the straight lines in an image are detected by Hough transform. In advanced countries, there are many straight lines in the road scene, such as white center and sidelines on the road, the edges of buildings, and so on. On the other hand, it is usually challenging to find straight lines in the scene of a road in under developing countries, and we cannot use Hough transform.

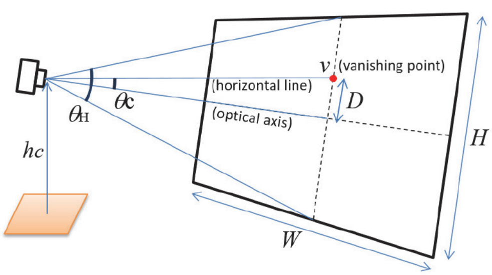

Figure 2 Schematic diagram of dashcam position and posture and its view. The vanishing point is described as a red point

In this study, we propose to apply optical flow tracking on video flames to detect the vanishing point. To reduce the influence of vibration of the vehicle on driving, we used a total of 10 seconds with 30 fps. The directions of flow of grid points are estimated using OpenCV, and we took the average of the intersections of extended lines of flow vectors as the vanishing point. The accuracy of the position of the vanishing point is evaluated by comparing it with the vanishing point estimated by Hough transform approach.

2.4 Identification of road area and pothole using semantic segmentation

To identify the road surface area and detect the pothole, we used state-of-the-art semantic segmentation by deep learning. We compare the performance of FCN [13], SegNet [14], and U-net [15] and found that SegNet and U-Net have an advantage on calculation time. On the other hand, we got a maximum IoU (Intersection over Union) value with FCN.

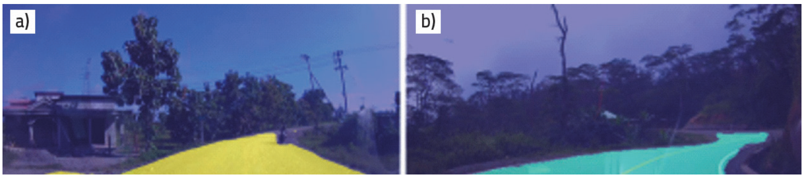

Figure 3 Results of semantic segmentation of road area using SegNet. (a) unpaved road, (b) paved road

2.5 Measurement of road width and pothole size by bird's-eye-view transformation

The estimation of road width is easier than that of pothole size. By taking extension lines with the vanishing point (red point) and each edge of road area (yellow lines), we can estimate the road width on the lower edge of flame as *L* pixels and l meter (Fig. 4).

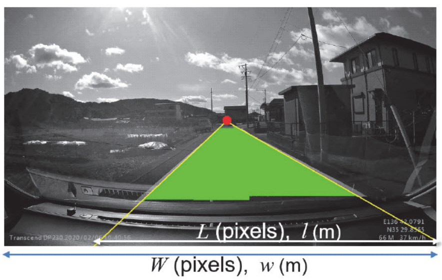

Figure 4 Estimation of the road width on the lower edge of flame

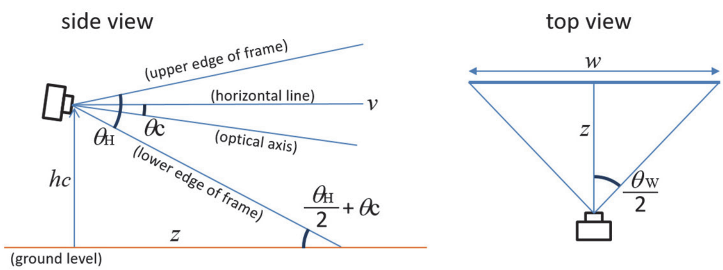

Figure 5 Calculation of the real size of lower edge of flame as meter unit

The formula to estimate the road width L (m) is as follows (Fig. 5):

$$l = w\frac{L}{W} = \frac{L}{W}2z\tan\frac{\theta_W}{2} = \frac{L}{W}2\tan\frac{\theta_W}{2} \times \frac{h_c}{\tan\left(\frac{\theta_H}{2} + \theta_c\right)}$$

To estimate the pothole size, we need to transform the front view image to the bird's-eye-view image (Fig. 1) and count pixels of the pothole area.

2.6 Storing the GPS and road width information into database

The road width information is stocked every 10m along the road with longitude and latitude values. When the pothole is detected, its size and longitude, and latitude values are also stocked. Our analyzing results are visualized on the map using Google Map, Open Street Map, or QGIS (a free and open-source cross-platform desktop geographic information system [16]).

3 Results

We took the dashcam video of more than 1,000 kilometers in Timor-Leste and a certain degree distance in Japanese for system development and comparison.
To estimate the accuracy of the optical flow method on detecting the vanishing point, we first applied the method on paved road video in urban areas in Japan (Fig. 6 (a)). Besides applying the optical flow to identify the vanishing point, we also identified the vanishing point by Hough transform using white straight lines on the road as a correct reference. As a result, the difference in the degree of the vanishing points between the optical flow method and the Hough transform method was about 0.2°. The results show that we can replace the Hough transform method with the optical flow method.

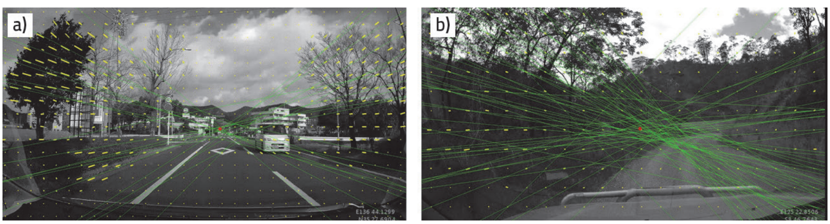

Figure 6 Optical flows of mesh points and detection of the vanishing point, a) Paved road in urban area in Japan, b) Unpaved road in rural area in Timor-Leste. The identified vanishing point is described as red point, the flows of grid points are described as yellow lines, and the extension lines of the flows are described as green lines

We applied the optical flow method on the video taken in Timor-Leste (Fig. 3 (b)). Even if there are no straight lines, we could estimate the vanishing point. The position of the estimated vanishing point does not stable in the short term because of the vibration of the vehicle on unpaved roads. We used a total of 300 flames of the video, which correspond 10 seconds with 30 fps, to remove the vibration noise.
We applied the proposed method to the dashcam video taken on a paved rural road in Japan to evaluate the road width estimation accuracy. As a result, the error of the road width estimation compare to the actual measured value was 0.25 %.

4 Conclusion

This paper proposed a method to estimate road width and pothole size from a dashcam video taken on an unpaved rural road in under developing countries. Since there are neither straight lines nor edges in the scenes of rural unpaved road, we cannot use Hough transform. We found that the optical flow method can be a good alternative. However, because of the vehicle's vibration on driving, we need to take statistics about 10 seconds driving.
Before the road width and pothole size estimation, we need to identify the area of the road surface and detect potholes. To identify the road area and detect the pothole in the image, we used state-of-the-art semantic segmentation by deep learning. Although we have evaluated the accuracy of road width estimation, the evaluation of pothole size estimation is the next task. We already have more than 1,000 km recording in Timor-Leste. We are in the process of analyzing all the data and making a database of road width and pothole size in Timor-Leste.

References

[1] Cabral, F.S., Pinto, M., Mouzinho, F.A., Fukai, H., Tamura, S.: An Automatic Survey System for Paved and Unpaved Road Classification and Road Anomaly Detection using Smartphone Sensor, IEEE International Conference on Service Operations and Logistics, and Informatics (SOLI), pp.65-70, Singapore, 31 July–2 August 2018.

[2] Pereira, V., Tamura, S., Hayamizu, S., Fukai, H.: A deep learning-based approach for road pothole detection in Timor-Leste, IEEE International Conference on Service Operations and Logistics, and Informatics (SOLI), pp. 1-6, Singapore, 31 July – 2 August 2018.

[3] Pereira, V., Tamura, S., Hayami, S., Fukai, H.: Classification of paved and unpaved road image using convolutional neural network for road condition inspection system, IEEE International Conference on Advanced Informatics, Concepts, Theory, and Application (ICAICTA), pp. 1–6, 2019.

[4] Cabral, F.S., Fukai, H, Tamura, S.: Feature Extraction Methods Proposed for Speech Recognition Are Effective on Road Condition Monitoring Using Smartphone Inertial Sensors, Sensors 19 (2019) 16, doi: https://doi.org/10.3390/s19163481

[5] Cabral, F.S., Mouzinho, F.A.L.N., Pereira, V., Fukai, H., Tamura, S.: The development of integrated road condition monitoring system for under developing countries using smartphone sensors and dashcam in vehicles, 6th International Conference on Road and Rail Infrastructure –CETRA 2020 (in press)

[6] Eriksson, J., Girod, L., Hull, B., Newton, R., Madden, S., Balakrishnan, H.: The pothole patrol: Using a mobile sensor network for road surface monitoring, 6th International Conference on Mobile Systems, Applications, and Services, pp. 29–39, Breckenridge, Colorado, 17–20 June 2008.

[7] Tai, Y., Chan, C., Hsu, J.Y.: Automatic road anomaly detection using smart mobile device. In Proceedings of the Conference on Technologies and Applications of Artificial Intelligence, Hsinchu, Taiwan, 18–20 November 2010.

[8] Seraj, F., Zwaag, B.J., Dilo, A., Luarasi, T., Havinga, P.: RoADS: A road pavement monitoring system for anomaly detection using smart phones, 1st International Workshop on Machine Learning for Urban Sensor Data Switzerland, pp. 128–146, 2014.

[9] Bhoraskar, R., Vankadhara, N., Raman, B., Kulkarni, P.: Traffic and road condition estimation using smartphone sensors, 4th International Conference on Communication Systems and Networks (COMSNETS 2012), pp.1-6, Bangalore, India, 3–7 January 2012.

[10] Tomiyama, K., Kawamura, A., Nakajima, S., Ishida, T., Jomoto, M.: A mobile profilometer for road surface monitoring by use of accelerometers, 7th Symposiumon Pavement Surface Characteristics: SURF 2012, Norfolk, VA, USA, 19–22 September 2012.

[11] Timor-Leste Strategic Plan 2011–2030, http://timor-leste.gov.tl/wp-content/uploads/2011/07/Timor-Leste-Strategic-Plan-2011-20301.pdf

[12] Building a Sustainable Rural Road Network in Timor-Leste, https://www.dfat.gov.au/sites/default/files/roads-for-development-ilo-ausaid-concept-note.pdf

[13] Long, J., Shelhamer, E., Darrell, T.: Fully convolutional networks for semantic segmentation, IEEE conference on computer vision and pattern recognition, pp. 3431-3440, 2015.

[14] Badrinarayanan, V., Kendall, Cipolla, R.: SegNet. A deep convolutional encoder-decoder architecture for image segmentation, IEEE Transactions on Pattern Analysis and Machine Intelligence, 39 (2017) 12, pp. 2481-2495, doi: 10.1109/TPAMI.2016.2644615

[15] Ronneberger, O., Fischer, P., Brox, T.: U-net: Convolutional networks for biomedical image segmentation, International Conference on Medical image computing and computer-assisted intervention, pp. 234-241, Munich, Germany, 5-9.10.2015.

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UNSATURATED CBR DESIGN APPROACH OF FLEXIBLE PAVEMENT

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Abstract

Subgrades in arid and semi-arid regions are often subjected to seasonal moisture variations that trigger volume change. To account for moisture changes in a vadose zone during pavement design, matric suction (ψ_m) is unavoidably required. In that context, ψ_m inclusion in CBR design becomes imperative. This study presents two CBR design approaches of flexible pavement, i.e. the conventional CBR design, and unsaturated CBR design methods. To compare these design approaches, subgrade soils were selected and a series of suction tests, CBR, and unsaturated CBR tests were performed to obtain the CBR design values of the subgrade materials. The results illustrate a linear relationship between suction and CBR values of the subgrades. The test results, further revealed that the unsaturated CBR design values are 2 to 2.5 times greater than the conventional CBR values. Based on the experimental results, the design analysis confirmed that the unsaturated CBR design approach is more conservative and rational compare to the conventional CBR design method.

Keywords: unsaturated CBR, suction, subgrade, design approach, pavement

1 Introduction

The theoretical network that expresses the moisture response of unsaturated subgrades concerning pavement design is demonstrated by [1-2]. The study suggested that variation in moisture content is influenced by the degree of saturation as this in turn affects matric suction. Thus, suction is the stress factor that represents the stress state of unsaturated soil response [3]. Several, attempt has been made to incorporate unsaturated soil mechanics principles in CBR testing and design [4-5] their test results indicated that CBR increases with increasing soil suction, as this lead to ultimate limit design values. Some documented studies have demonstrated the method by which hydraulic hysteresis on subgrade CBR could be evaluated using suction [6]. Whereas, vast studies in the literature have failed to account for suction in CBR testing and pavement design, despite the evidence of seasonally moisture variation with subgrades. This affects suction and in turn, reduces the bearing strength and deformation resistance of the pavement structure.
This study demonstrated the design of flexible pavement, utilizing unsaturated CBR design values. The hydromechanical behavior of the soil was determined by performing a series of unsaturated CBR tests at various dry densities. The CBR dependencies on suction at different gravimetric water content are also presented.

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2 The material and experimental program

2.1 Soils

The collected subgrade soil samples were labeled as Soil A, B, and C respectively. Dry sieving was conducted by firstly passing particles through a 9.5mm sieve and subsequently sieved using 4.75mm and 75μm sieves to separate fines, sand, and gravel. The soils that passed through 75μm sieve were used for the hydrometer test, to further differentiate percentages of silt and clay for the representative subgrade soils following ASTM D1140 and the grading curve of the soils is shown in Fig 1.

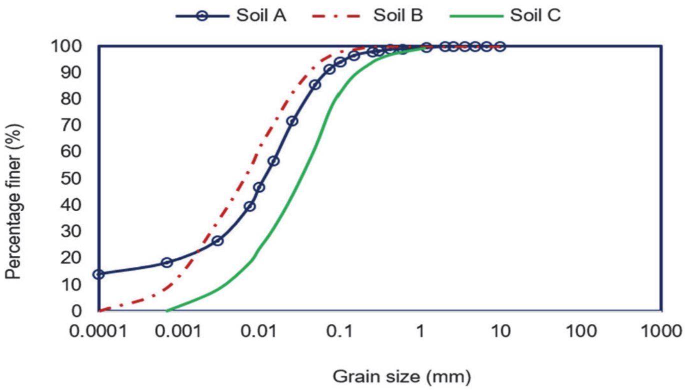

Figure 1 Grain size distribution curve

Zero swelling tests (ZST) were conducted on the respective soils in conformance to the Indian standard IS: IS 2720 test method, to evaluate the swelling potential of the subgrades. As presented in Table 1, the investigated soils are classified as CH and CL. Thus, it is qualified as an expansive subgrade based on the classification tests.

Table 1 Properties of soils

Soil designation		Soil A	Soil B	Soil C
Sampling depth (m)		0.5-1.2	0.5-1.2	0.5-1.2
Specific gravity G_s		2.67	2.70	2.71
ZST (kPa)		710	650	870
Atterberg limit (%)	LL	62.10	68.03	57.28
	PL	28.32	29.82	34.21
	PI	33.78	38.21	23.07

* GSA= Grain size analysis, **C_u= Coefficient of uniformity (D_{60}/D_{10}),
***C_c= Coefficient of curvature ($D_{30}{}^2/D_{60}$ x D_{10})

3 Sample preparations

Before sample preparation, the Proctor compaction test was carried on the subgrade soils under ASTM D-698. The soils A, B, and C rendered optimum moisture content (OMC) of 16.12 %, 22.15 % 19.11 % with corresponding maximum dry densities (MDD) 18.15kN/m^3, 20.32 kN/m^3, and 18.38kN/m^3 respectively.

4 Experimental testing procedures

Standard laboratory civil engineering testing programs routinely used to measure geotechnical properties of soil were conducted on the subgrade materials. The soil testing programs with their corresponding specifications are as follows, Proctor compaction was conducted following ASTM D-698, 2007 testing procedures, Zero swelling test (ZST) was performed under IS 2720 (Part 41: 1977) protocol, Filter paper test was performed in accordance with ASTM 5298-2018, and CBR test was conducted in line with ASTM D1883-16 testing procedures.

4.1 Unsaturated CBR test

The soaked CBR is usually used for pavement design, therefore this study only considered soaked condition. Whereas, CBR values of the subgrades were calculated according to Eq. (1).

$$CBR(\ \%) = T_L / S_L \times 100 \qquad (1)$$

where CBR is the California bearing ratio in (%), T_L is the test load, S_L is the standard load
The soil-water characteristic curve (SWCC) was established to evaluate the air-entry values (AVE) of the soils. The soaked CBR values corresponding to each matric suction were correlated and the values of the unsaturated CBR were determined using Eq. (2).

$$\frac{CBR_u}{CBR_s} = \left[\frac{\Psi_m}{u_e}\right]^n \qquad (2)$$

Where CBR_u is the unsaturated CBR, CBR_{us} is the soaked CBR obtained from the conventional CBR test, Ψ_m is the matric suction, u_e is the AVE and n is the regression parameter due to suction and dry density obtained from linear interpolation using mathematical software (NCSS 11).

5 Result and discussions

5.1 Subgrades response to suction

The variation of the total, matric, and osmotic suction at various gravimetric water content is shown in Figs. 2 and 3 for soil A, B, and C respectively. It is noted that the response of the subgrades to suction is approximately linear. Thus, suction decreases with an increase in gravimetric water content. High suction values were evaluated on the dry side of the optimum. Whereas, lower suction values were observed as the gravimetric moisture contents tend to move towards the wet-side of the optimum moisture content [7-8]

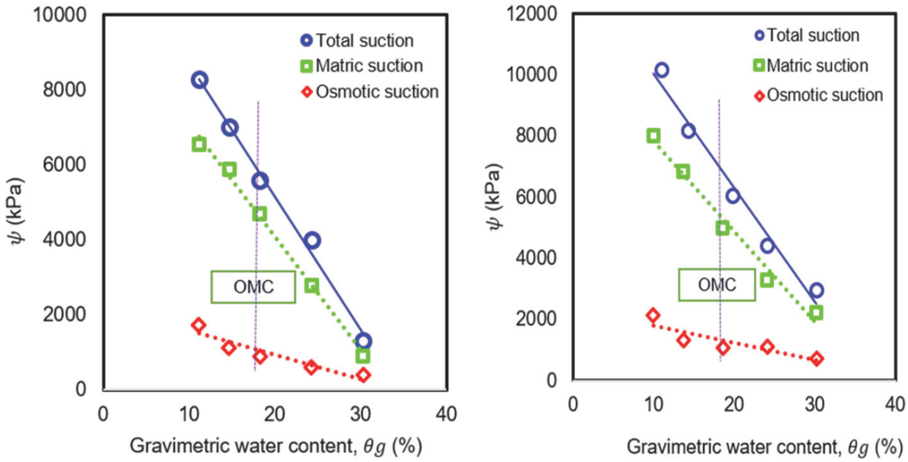

Figure 2 Suction Ψ versus gravimetric water contents curve for soil A and B

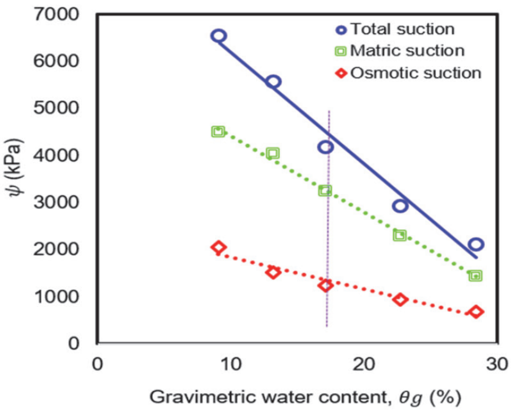

Figure 3 Suction Ψ versus gravimetric water contents curve for soil C

5.2 Soil-H_20 retention curves (SWRC)

As shown in Figs 4 and 5, the developed SWRCs were based on the data sets obtained from the filter paper test results and fitted by Fredlund and Xing model. Results show that Fredlund and Xing's [9] equation has the best-fit, and the SWRCs fitted by Fredlund and Xing's equation were used in the following description The dotted legend represents the measured SWRCs for the subgrades and the solid legend line is the Fredlund and Xing fitting curve. The subgrade yielded air-entry values (AEV) of 80 kPa, 300 kPa, and 200 kPa for soils A, B, and C respectively. The AEV for soil B is higher due to larger fine content, this indicates that the initial compaction water contents have no significant influence on the SWRC at high suction. It is noted that soil B possess more capacity to retain water compare to soil A and C. The test result clearly shows that an increase in clay content generally leads to an increase in the amount of water retained at a certain suction level and adsorption governs the high suction value of the SWRC [10]. It is also noted that suction increases as the volumetric water content of the specimens decrease.

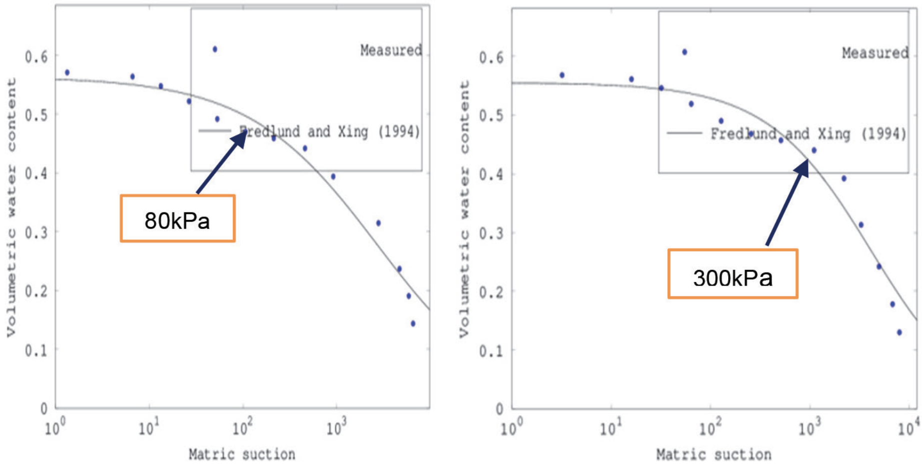

Figure 4 Soil-water retention curve for soil A and B

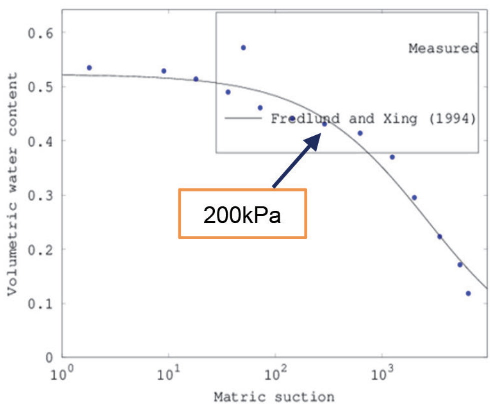

Figure 5 Soil-water retention curve for soil C

5.3 Unsaturated CBR

The values of soaked CBR_s corresponding to each volumetric moisture content and matric suction for the subgrade were determined, as the ratio of CBR_u and CBR_s as presented in Eq. (2). The test results revealed that unsaturated CBR values were 1.5 to 2 times higher compared to the soaked CBR as summarized in Table 2.

Table 2 Soaked and Unsaturated CBR values

Soil	C [%] @ 2.5 mm	C [%] @ 5 mm	C [%] @ 7.5 mm	Ψ_m [kPa]	u_e [kPa]	n	C_u [%]	P_s [kPa]	Δ [%]
Soil A	4.12	4.20	4.10	6541	80	0.20	9.94	610	7.93
	3.17	3.55	3.22	5876	80	0.22	8.16	284	6.30
	2.25	3.18	2.89	4689	80	0.24	5.88	515	3.03
	2.13	2.78	2.14	2793	80	0.26	5.36	440	2.32
	2.03	2.46	1.90	921	80	0.36	5.06	378.3	2.05
Soil B	3.53	3.93	3.75	8017	300	0.20	6.81	840	5.04
	2.10	2.72	2.73	4989	300	0.25	4.83	610	2.75
	1.91	2.51	2.63	3295	300	0.27	4.09	530	2.29
	1.81	2.24	2.33	2213	300	0.33	3.23	450	1.85
Soil C	4.33	4.58	4.44	4498	200	0.20	8.59	510	6.11
	3.43	3.55	3.68	4045	200	0.22	6.65	395	4.85
	3.05	3.36	3.34	3250	200	0.24	6.28	300	4.29
	2.21	2.70	2.66	2298	200	0.26	4.17	230	2.31
	2.16	2.48	1.56	1440	200	0.26	3.75	200	1.91

C_{soaked}: CBR P_S: swelling stress*Ψ_m: matric suction* Δ: change in C due to P_S

5.4 The unsaturated approach of flexible pavement design

Though, the basic difference between the two approaches is the incorporation of and AEV in evaluating the CBR values. Based on the CBR result presented in Table 3, Eq. (3) is employed to calculate the required thickness for the investigated pavements, to compare the aforementioned design approaches.

$$A = P(1 + r)^{n+y} \tag{3}$$

Where A is the design traffic in commercial vehicles per day (CVPD), P is average daily traffic (ADT) and it is taken to be 2500CVPD in this study, r is the annual traffic growth rate =12 %, n is the design period normally 10 year for flexible pavement and 3 years of construction period making it a total nvalue of 13 years.

$$A=P(1+r)^{n+y} = 2500(1+0.12)^{3+10} = 10909 \text{ designe value} = 10{,}100\text{CVPD}$$

According to the design calculation, the pavement falls under the design Index of 'G' as presented in Fig 6.
Based on the CBR design values in Table 3, the calculated thickness corresponding to the required pavement thicknesses without wearing course are presented in Table 4. Based on the CBR design values for CBR_s and CBR_u, it is indicated that the investigated subgrades require pavement layer thickness between 600mm to 800 mm when CBR_s design values are utilized. Whereas, CBR_u design values require pavement layer thickness within 400 mm to 530 mm according to the CBR design analysis. It is noted that the design values for unsaturated CBR require lesser thickness compared to that of the conventional CBR values. This implies that the design of flexible pavement using the conventional CBR approach leads to overdesign

of the pavement compared to the unsaturated CBR that is conservative thus requires lesser thickness for the pavement.

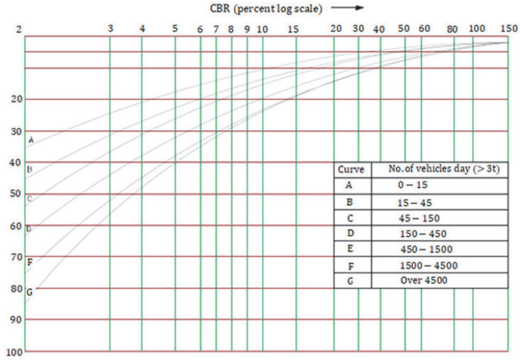

Figure 6 CBR design chart

Table 3 Pavement thicknesses

Design values	Soil A	Soil B	Soil C
CBR_s [%]	2.25	2.10	3.05
RPT [mm]	760	788	688
CBR_u [%]	6	4.83	6.30
RPT [mm]	480	530	400
ΔCBR_u due to P_s [%]	3.03	2.75	4.30
RPT [mm]	680	670	570
*RPT = required pavement thickness			

6 Conclusions

This paper aims to understand the different CBR design approaches, and three different subgrades were investigated. A series of laboratory tests such as the ZST, filter paper tests, CBR, and unsaturated CBR test method was conducted and the following conclusions are drawn:

- Suction increased as the volumetric water content of the specimens decreases, at a low suction within the range of 1 to 50 kPa. The air-entry value (AEV) of the specimens was computed to be 80kPa, 300kPa, and 200kPa for soils A, B, and C respectively. The AEV of soils revealed the degree of pore spaces, absorption capacity, and expansion degree of the soils.
- The highest swelling stress values were obtained at a lower void ratio, and this implies the subgrades possess high water absorption capacity this significantly influences the CBR_u values for pavement design. Averagely, the soaked CBR_s values were considerably 1.5 to 2 times lower than the unsaturated CBR_u values. This implies that pavement design using conventional CBR values could lead to overdesign and require high thickness asphaltic layers. Whereas, CBR_u design values are considered rational and conservative. As expected, strain-softening behavior was observed as the subgrades recorded higher resilient moduli for higher confining stresses and decreased with an increase in the deviatoric stress under identical confinement.

Acknowledgments

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References

[1] Vanapalli, S.K., Fredlund, D.G., Pufahl, D.E., Clifton, A.W.: Model for the prediction of shear strength with respect to soil suction, Canadian Geotechnical Journal, 33 (1996) 3, pp. 379-392.

[2] Fredlund, D.G., Xing, A.: Equations for the Soil-Water Characteristic Curve, Canadian Geotechnical Journal, 31 (1994) 4, pp. 521–32

[3] Zapata, C.E., Houston, W.N.: NCHRP Report 602, Project 9-23: Calibration and Validation of the Enhanced Integrated Climatic Model for Pavement Design, National Cooperative Highway Research Program, Transportation Research Board, Washington D. C, 2008.

[4] Singh, S., Sharan, A.: Strength characteristics of compacted pond ash, Geomechanics and Geoengineering: An International Journal, 9 (2014) 1, pp. 9–17. doi:10.1080/17486025.2013.772661, 2014.

[5] Aneke, F.I., Mostafa, M.H., Moubarak., A.: Behaviour of Unsaturated Soils for Road Pavement Structure under Cyclic Loading, Ph.D. Thesis, Free State, South Africa: Department Of Civil Engineering and Information Technology Central University of Technology, 2018.

[6] Mirzaii, A., Yasrobi, S.S.: Effect of Net Stress on Hydraulic Conductivity of Unsaturated Soils. Transp Porous Media, 95 (2012), 497–505, doi: https://doi.org/10.1007/s11242-012-0058-1

[7] Aneke, F. I., Mostafa, M.H., Moubarak, A.: Evaluation of subgrade resilient modulus from unsaturated CBR test, GeoMEast, International Congress and Exhibition, ISSMGE, Cairo, Egypt, 2008.

[8] Leong, E.C., Tripathy, S., Rahardjo, R.: Total suction measurement of unsaturated soils with a device using the chilled-mirror dew-point technique. Géotechnique, 53 (2003) 2, pp. 173-182

[9] Fredlund, D.G., Xing, A.: Equations for the Soil-Water Characteristic Curve, Canadian Geotechnical Journal, 31 (1994) 4, pp. 521–32.

[10] Aneke, F.I., Mostafa M.H., Moubarak A.: Swelling stress effects on shear strength resistance of subgrades, International Journal of Geotechnical Engineering, 2019, DOI: 10.1080/19386362.2019.1656445

DATA ANALYSIS APPLIED TO AIRPORT PAVEMENT DESIGN

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Abstract

Designing an airport pavement is a complex engineering task. Thus, one of the first steps is to create scenarios for the operation of the airport. In this sense, the use of data analysis techniques can extract insights for this phase. Among the various parameters that characterize a runway, the most relevant is the structural capacity of the pavement and the length. For aviation, the standard for indicating the resistance of pavement is its Pavement Classification Number (PCN). Therefore, an application was developed in Python programming language [1], having as inputs the PCN and the runway length. Outputs are the aircraft supported by the pavement and the routes served (coverage). The development of this study follows the steps: a collection of real raw data about airports and aircrafts, data processing and cleaning, model development, model testing and application, result analysis, visualization, and final report. To test the model, the Viseu Aerodrome, located in the Center of Portugal region, was used. Several combinations have been created for PCN and runway length. Of all scenarios, three of them stood out, namely: maintain current characteristics (PCN 6 and length of 1160 m); an intermediate (PCN 23 and length of 1800 m); and a more robust scenario (PCN 83 and length of 2500 m). Finally, in the first scenario, it was possible to serve mainland Portugal, Spain, and a small portion of southern France. However, the operation was limited to small aircraft of up to 20 passengers. In the intermediate scenario, it was possible to serve much of the Schengen space with aircraft of up to 70 passengers. For the robust scenario, all Schengen space was served, with aircraft of up to 200 passengers. Therefore, based on two simple parameters, such as PCN and runway length, it was possible to visualize the coverage of an airport.

Keywords: airport, data analysis, pavement design, Python programming

1 Introduction

The opening of the aviation market to private operators, as well as regional aviation, reduced costs and popularized the use of aircraft. Consequently, in recent years this demand has pushed for the creation of new routes and the construction of more infrastructure [2; 3; 4]. Likewise, in this period, Portugal had notable growth in tourism. However, the airport infrastructure is not keeping up with demand, as the two main Portuguese airports, Lisbon and Porto, operate almost to the limit. Observing a small country like Portugal, the Central and Interior zones are in a gap about air transport [5]. In this scenario, small airports and regional airports appear as alternatives to operate low-cost regional flights in places where the critical mass does not yet support large infrastructures [6]. On the other hand, at regional airports, operating costs pressure the viability of the operation, and subsidies are required [6; 7; 8]. Another relevant point in the operation of regional airports is the use of smaller

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aircraft, which end up having higher fuel consumption per seat, losing competitiveness [2]. Also, it must be assessed that air transport competes with other modes of transport, such as rail and road [3].
Therefore, there are many variables when proposing new routes. When the expansion or construction of infrastructure accompanies this analysis, this study becomes even more complex. In response to this demand, this research was developed to evaluate the construction of a new runway for the Viseu Municipal Aerodrome. This airport is interested in serving larger aircraft, which allow more extended range and carry more passengers per aircraft [5]. Requiring a runway with higher structural resistance of the pavement and with a longer length [9]. To design an airport pavement, it is necessary to know two main characteristics, namely, the Pavement Classification Number (PCN) and the runway extension. Thus, the proposed challenge is to design a pavement with capacity (PCN) and extension to serve a desired group of routes. A Python application was developed to accomplish this task [1], which processes information related to aircraft and airports and presents theoretical coverage for flights from the Viseu aerodrome.

2 Data and methods

2.1 Methods

The proposed methodology (Figure 1) for the research is based on the understanding of the problem, which is the interest of the Director of the Viseu Aerodrome to increase its number of passengers and coverage of flights. Based on this problem, the research question is to operate larger aircraft. In this sense, it is necessary to increase the resistance and the length of the runway.

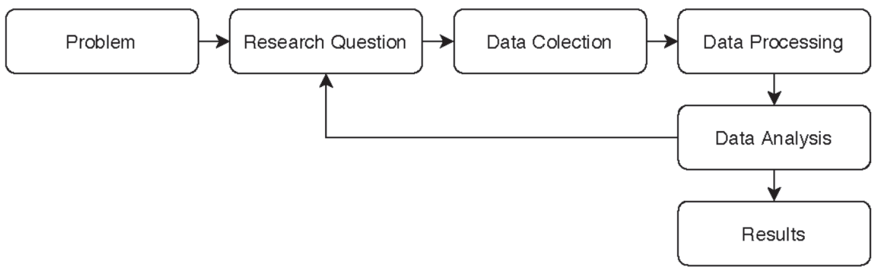

Figure 1 The methodology used in the research

After understanding the problem, the data collection and processing step begins. Finally, the last step is to analyze the data and visualize the results. In the data analysis stage, it is crucial to understand whether the problem has been answered. At this point, it may be necessary to reformulate or adjust the research question, restarting the process.

2.2 Data source

The information present in the datasets used in this research is shown in Table 1.

Table 1 Datasets used in the research

Dataset name	Fields	Source
AircraftData.csv	Aircraft identification, ACN[a], runway length required for landing and takeoff operations, autonomy, passengers[b].	[1]
AirportData.csv	Airport name, city, country, ICAO code, latitude, longitude, altitude, other fields[c].	[10]
20191227_basemap.shp	Layer file with boundaries of countries.	[11]
AirportTestExtraData.csv[d]	ICAO code, runway length, PCN.	[12]

Notes: a) Aircraft Classification Number is related to the impact of the aircraft on the pavement, and must be less than or equal to the runway PCN; b) Standard configuration; c) Other unused fields have been excluded; d) Schengen area airports only (lower airport security requirements).

2.3 Data preprocessing

The data is seldom perfect for use. Usually, they are in databases distributed in several sources, in different formats. Consequently, this information must be treated before being used. There is a pre-processing that consists of importing, formatting, and cleaning the data. Hence, in Figure 2, the pseudo-code for the data preprocessing step is presented.

Data preprocessing algorithm

```
Read input data
Merge airport data
Set ref. Airport = LPVZ (ICAO code of Viseu Aedrome)
for all airports:
        Drop unused columns
        Drop rows with PCN or Runway Length = 0
        Calculates the distance to the ref. Airport
Save results
```

Figure 2 Pseudo-code for data preprocessing step

To process the input data, the Python Pandas library is used to read, manipulate and view the data. Thus, the data are transformed into dataframes. The aircraft dataframe can be seen in Table 2.

As this base was built from scratch, it was not necessary to format and clean the data, since the dataset was created with the application in mind. On the other hand, the airport dataset had to be processed until it was usable. Initially, the file “AirportData.csv” and “AirportTestExtraData.csv” were merged. The field used as a pointer to this union was the ICAO code. Also, all unused fields were deleted. Finally, airports that had null information for runway length or PCN, were also removed. The resultant airport dataframe can be seen in Table 3.

Table 2 Aircraft dataframe

Aircraft	ACN	Required extension	Autonomy	Passengers
Dornier_228	6.4	792	1037	19
DCH-8_Q200	16.5	1000	2084	40
ATR42-600	18.6	1165	1326	48
ERJ140	21.1	1850	3058	44
ATR72-600	23.0	1175	1528	72
CRJ200	23.0	1768	2500	50
DCH-8_Q400	30.5	1425	2040	82
CRJ700	34.0	1605	2553	70
E170	38.6	1644	3982	72
E175	40.4	2244	4074	80
B737-800	79.3	2316	5436	184
B737-MAX8	82.2	2500	6570	200

Table 3 Airport dataframe (slice with the five first rows)

ICAO	Country	Latitude	Longitude	Distance to Ref[a]	RWY Length	PCN
BIKF	Iceland	63.99	-22.60	2759.7	3065	80
EBBR	Belgium	50.90	4.48	1480.6	3638	80
EBCI	Belgium	50.46	4.45	1444.6	2550	64
EDDB	Germany	52.38	13.52	2077.9	3000	140
EDDK	Germany	50.87	7.14	1617.8	3815	75

Notes: a) The Distance to ref column refers to the distance from the reference airport (Aerodrome of Viseu) to the destination airport. This field received the value zero initially. This item was calculated using the geodetic distance supported by the Python GeoPy library.

2.4 Data processing and analysis

After preparing the data for the study, the next step is to analyze the data and test hypotheses. Two datasets are formatted for processing (airports and aircrafts), and the objective is to compare the requirements for aircraft operation and the characteristics of the airports. The algorithm used to process the data can be viewed in the pseudo-code in Figure 3.
At this moment, the developed algorithm has the function of evaluating whether the aircraft registered in the database can operate at the reference airport (Viseu Aerodrome - LPVZ). In a second step, for all airports in the database, the capacity to receive the supported aircraft (which operate in the LPVZ) is analyzed. Also, it is observed whether the distance from the reference airport is within the autonomy of the aircraft.
As a result of the proposed algorithm, the airports served are presented. The list of aircraft with the possibility of operating at the reference airport is also informed.

```
Hypothesis testing algorithm
User input: ref PCN, ref Runway Length
Read input data (from data preprocessing results)
For all aircrafts:
    if aircraft ACN > ref PCN then:
        Drop aircraft
    if aircraft required extension > ref runway length then:
        Drop aircraft
    For all airports:
        if aircraft ACN > PCN then:
            Drop airport
        if aircraft autonomy < airport distance to ref then:
            Drop airport
        if aircraft required extension > runway lenght then:
            Drop airport
Results: Airports served, list of possible aircraft
```

Figure 3 Pseudo-code algorithm used for hypothesis testing

3 Results and discussion

As a result of processing the data, the user receives a list of aircraft that can operate on LPVZ, and a list of possible airports that can function as a destination is also provided. As an example, the algorithm was used with the following inputs for Viseu Aerodrome: PCN: 23; Runway length: 1800 m. The results after processing can be seen in Figure 4.

```
Aircrafts served: ['Dornier_228' 'DCH-8_Q200' 'ATR42-600' 'ATR72-600' 'CRJ200']
Routes served: ['EBBR' 'EBCI' 'EDDB' 'EDDK' 'EDDL' 'EDDM' 'EDDN' 'EDDP' 'EDDS' 'EDDT'
 'EDDV' 'EDDW' 'EDFH' 'EDLW' 'EHAM' 'EHEH' 'EHRD' 'EKCH' 'ELLX' 'ENBR'
 'EPGD' 'EPKK' 'EPKT' 'ESGG' 'GCFV' 'GCLP' 'GCRR' 'GCTS' 'GCXO' 'GEML'
 'LEAL' 'LEAM' 'LEAS' 'LEBB' 'LEBL' 'LECO' 'LEGE' 'LEGR' 'LEIB' 'LEJR'
 'LELC' 'LEMD' 'LEMG' 'LEMH' 'LEMO' 'LERS' 'LESO' 'LEST' 'LEVC' 'LEVX'
 'LEXJ' 'LEZG' 'LFBD' 'LFBO' 'LFLL' 'LFML' 'LFMN' 'LFOB' 'LFPG' 'LFPO'
 'LFRS' 'LFSB' 'LHBP' 'LIBD' 'LICA' 'LICC' 'LICJ' 'LIEE' 'LIEO' 'LIMC'
 'LIME' 'LIMF' 'LIML' 'LIPE' 'LIPX' 'LIRN' 'LIRP' 'LIRQ' 'LKPR' 'LMML'
 'LOWW' 'LPAZ' 'LPBJ' 'LPBR' 'LPCS' 'LPEV' 'LPFR' 'LPGR' 'LPHR' 'LPLA'
 'LPPD' 'LPPI' 'LPPM' 'LPPR' 'LPPS' 'LPPT' 'LPVR' 'LPVZ' 'LSGG' 'LSZH'
 'LEPA' 'LPMA' 'LPCB' 'LPSO']
```

Figure 4 Results of the data processing algorithm

The results presented show the aircraft that can operate at LPVZ. ICAO codes are also reported for all airports within the Schengen area that can serve as a route for these aircraft. The name of each airport which corresponds to each ICAO code can be seen in [10]. An essential part of data analysis is the visualization of results. At this time, the data has been processed, and there are already valid results. However, an essential part of data analysis is the visualization of results. Thus, GIS data [11; 13] were used as a base map, on which layer the possible airports are plotted, and the countries served are highlighted (Figure 5).

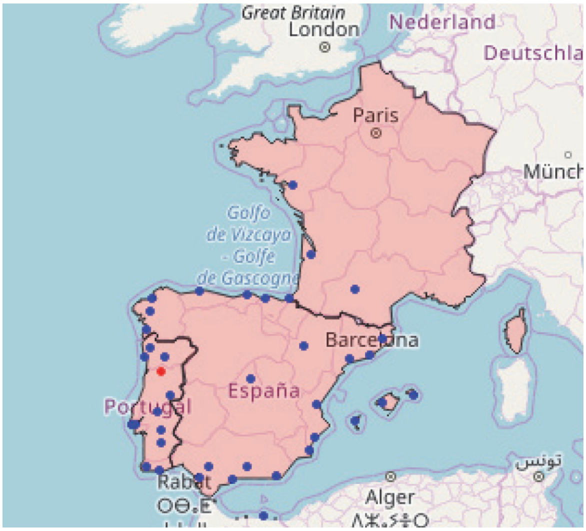

Figure 5 Results for PCN 6, and 1160 meters runway length

In this scenario, the Viseu Aerodrome (red point) maintains the pavement with PCN 6 and a runway length of 1160 m (current situation of the LPVZ infrastructure). In this situation, it was possible to serve mainland Portugal, Spain, and a portion of southern France (airports served in blue). However, the operation was limited to small aircraft of up to 20 passengers. Two other scenarios were also created, an intermediate (PCN 23 and runway length of 1800 m); and a more robust scenario (PCN 83 and runway length 2500 m). In the intermediate scenario, it was possible to serve a large part of the Schengen area with aircraft of up to 70 passengers. For the robust scenario, the entire Schengen space was served, with aircraft of up to 200 passengers. All three scenarios can be seen in Figure 6.

Therefore, based on two simple parameters, such as PCN and runway length, it was possible to visualize the coverage of an airport. This shows that the procedure developed for data analysis has the ability to provide information to decision makers.

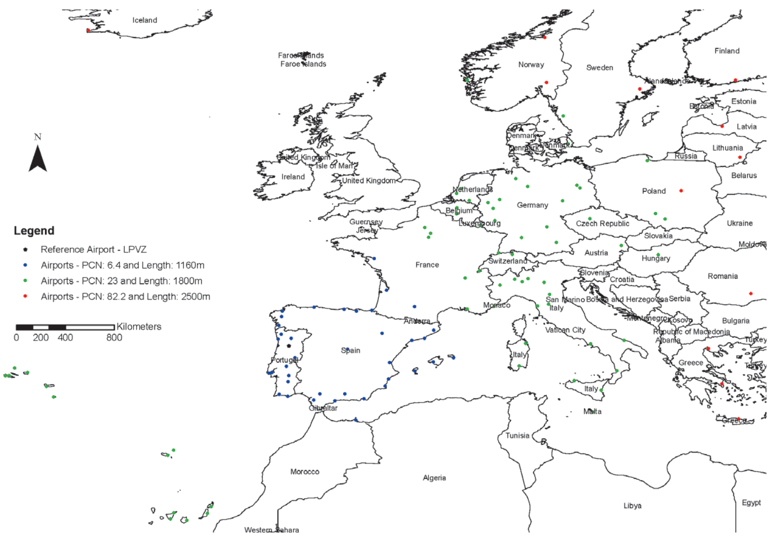

Figure 6 Results with three analysed scenarios (full resolution image)

4 Conclusions

This entire research was developed based on a problem, which is the need for the Aerodrome of Viseu to operate larger aircraft. In this sense, it is necessary to reinforce the pavement structure and increase the length of the runway. As a result, we have the following results: maintaining the current characteristics of the Viseu Aerodrome runway pavement, PCN 6 and runway length of 1160 meters, it was possible to serve mainland Portugal, Spain and southern France. Nevertheless, the aircraft in operation is limited to the use of the Dornier 228, which carries only 19 passengers. Likewise, with PCN 23 and runway length of 1800 meters, the Schengen space is almost served. However, it is limited to the use of aircrafts with higher operating costs per passenger, such as CRJ200 and ATR72-600 (these aircrafts carry between 50-70 passengers). Finally, with PCN 83 and runway length of 2500 meters, little coverage is obtained compared to the previous scenario, but larger aircraft (up to 200 passengers) can be used, with best cost/passenger ratio. For example, the low-cost airline Ryanair only operates B737-800 aircraft within these parameters. Having presented the three best scenarios, the next step is a business decision, which will be supported by the data analysis carried out in this research. Finally, in this research it was demonstrated that data analysis can be a great ally for engineers and that computational tools have evolved in this direction every day. Putting all hype aside, it is clear that data analysis can help us make better engineering decisions.

References

[1] Tamagusko, T.: Studies for the Viseu Airport Runway Project, https://github.com/tamagusko/ViseuAirportStudy, 29.12.2019.

[2] Neufville, R., Odoni, A.R.: Airport Systems: Planning, Design, and Management, 2013.

[3] Gillen, D.: The evolution of airport ownership and governance, Journal of Air Transport Management, 2011.

[4] Laurino, A., Beria, P.: Low-cost carriers and secondary airports: Three experiences from Italy, Journal of Destination Marketing and Management, 3 (2014) 3, pp. 180–191.

[5] Martins, J.: Reflexão sobre a viabilidade e localização de uma Infraestrutura Aeroportuária na região Centro de Portugal (Universidade do Porto), 2018.

[6] Kazda, A., Hromádka, M., Mrekaj, B.: Small regional airports operation: Unnecessary burdens or key to regional development, Transportation Research Procedia, 28 (2017), pp. 59–68

[7] Ferreira, A., Barros, R., Cruz, J.: The Gonçalves Lobato Municipal Airport Pavement Management System, IV International Symposium of Pavement Evaluation and Reinforcement Design Projects, Fortaleza, Brazil, 2009.

[8] Grimme, W., Maertens, S., Schröpfer, A.: Options for Traffic Growth at Smaller European Airports Under the Options for Traffic Growth at Smaller European Airports Under the European Commission's Guidelines on State Aid European Commission's Guidelines, Transportation Research Procedia, 35 (2018), pp. 130–139

[9] Tamagusko, T.: Airport Pavement Design, 2020, doi: 10.13140/RG.2.2.19628.00640.

[10] OpenFlights, Airport, airline and route data, https://openflights.org/data.html, 23.12.2019.

[11] Natural Earth, Free vector and raster map data at 1:10m, 1:50m, and 1:110m scales, http://www.naturalearthdata.com, 25.12.2019.

[12] World Aero Data, World Aeronautical Database, https://worldaerodata.com, 28.12.2019.

[13] OpenStreetMap, OpenStreetMap: Collaborative mapping with open data, https://www.openstreetmap.org, 26.12.2019.

BINDER COURSES USING COLD RECYCLED MIXTURES – A NOVEL CONCEPT IN COLD RECYCLING

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Abstract

Cold recycling with cement and bituminous emulsion is one of the most environmentally friendly techniques to maximize the reuse of reclaimed asphalt (RA) collected during demolition of roads. Cold recycled mixtures are commonly used for base courses in construction or rehabilitation of flexible and semi-rigid pavements. Current experiences demonstrated that cold recycling with appropriate requirements and technical recommendations allows long-lasting pavements to be designed. Those outcomes stimulated researchers and engineers in the new challenge of using the cold recycled mixtures in binder layers that are generally more often included in maintenance planning. This paper summarizes current San Marino and Polish experiences with cold recycled mixtures designed for binder layers. The first part of the paper presents the design phase of the cold recycled mixtures in both countries. It describes and compares the composition of the mixtures, grading curves of the mineral mixtures, binding agents and requirements. The second part of the paper shows laboratory test results of mechanical properties of cold recycled mixtures for binder courses including test results in full-scale application.

Keywords: cold in-plant recycling, binder course, mix design

1 Mix design of cold recycled mixtures

Cold recycling is one of the most ecologically friendly technology for reconstruction of old deteriorated asphalt pavements [1, 2, 3]. cold recycling can be applied following two approaches: in-plant and in-situ production. Cold in-plant recycling (CPR) allows using more than one fraction of selected reclaimed asphalt (RA) and virgin aggregates. Consequently, an optimal control of the product quality can be assured. Typical, CPR is utilized for the production of pavement sub-base and base courses [4]. However, an accurate mix design, management and production process can increase the degree of confidence and the performance expectations of the final product as required for binder courses [5].
This paper compares Polish and San Marino approaches to establish a mix design procedure for cold recycled mixture to be used as binder course.
In both countries, the mixtures presented in this study were designed according to the related requirements for cold recycled mixtures. In both countries, regulations for cold recycled mixtures were originally developed for base layers and then adapted to binder layers. Indeed, the different functionality between base and binder courses required to adequately change the requirements for mixture composition and selection criteria. Taking into account the different climatic condition and the state of practice, the procedures adopted in the Republic of San Marino and Poland differ each other as reported in table 1.

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Table 1 Qualitative comparison between San Marino and Poland mix design procedures

Issue	San Marino procedure	Poland procedure
Compaction method	gyratory compactor	Marshall compactor
Cement dosage [% by aggregate weight]	1.5 - 2.5	1.0 - 4.0
Emulsion dosage [% by aggregate weight]	4.0 - 5.0	2.0 - 7.0
Type of bituminous binder	Modified bituminous emulsion	Emulsion with unmodified bituminous or foamed bitumen
Volumetric parameter	Dry density	Dry density
Curing process	3 days at 40°C	7 and 28 days at 20±5°C
Testing temperature	25 or 20°C	5°C
Mechanical requirement	ITS at 25°C ≥ 0.40 MPa ITSR ≥ 80% ITSM at 20°C ≥ 3.000 MPa	7 days: ITS at 5°C: 0.5 – 1.0 MPa ITSM at 5°C: 1500 – 4500 MPa 28days: ITS at 5°C: 0.7 – 1.6 MPa ITSM at 5°C: 2000 – 7000 MPa ITSR ≥ 80%

1.1 Mix design procedure complying with San Marino specification

Each fraction of RA has to be designated following the EN 13108-8. Moreover the RA maximum size, shape index (SI), flakiness index (FI), gradation of RA (washed method), gradation of aggregates and bitumen content have to be determined as average and standard deviation values from 5 samples when the total amount of RA to be used in the project is less than 2,500 t or 1 sample every 500 t when the total amount of RA to be used in the project is more than 2,500 t. SI and FI have to be below 30 % and the RA maximum size does not exceed the mixture maximum size. The other characteristic values are used as reference for quality control and the standard deviation as homogeneity parameter.
The cationic modified bituminous emulsion has to respect the requirements reported in table 2. It has to be produced using a 70/100 or 50/70 paving grade bitumen and designated as C60BP10 according to the EN 13808. The cement type CEM I (Portland cement), CEM II (composite Portland cement with fly ash, slag and limestone), CEM III (blast furnace cement), CEM IV (pozzolanic cement varieties) and V (Composite cement) complying with EN 197-1 can be used with no specific restrictions. CEM II is recommended when RA has low filler content (lower than 2 %).

Table 2 Requirement for modified bituminous emulsion for cold in-plant recycling in San Marino

Modified bituminous emulsion				
Parameter	Standard	Unit	Required value	Class EN 13808
Setting at 7 days	EN 12847	[%]	≤ 10	3
Adesivity	EN 13614	[%]	≥ 90	3
Breaking value	EN 13075-1	-	> 150	5 (or above 5)
Stability with cement	EN 12848	[g]	< 2	10
Residual bitumen				
Cohesion at 10°C	EN 13589 EN 13703	J/cm²	≥ 2	6
Elastic recovery	EN 13398	[%]	> 50	3

Two gradation bands are recommended depending on the layer thickness: AC20 for 10-15 cm thick courses, AC30 for 15-20 thick courses. The volumetric and mechanical characteristics of the cold recycled mixtures have to be determined using a gyratory compactor (EN 12697-31) at the fixed energy of 100 revolutions [6]. Each sample series has to consist of at least three specimens and the coefficient of variation of results (standard deviation/average) expressed as a percentage has to be lower than 15 %. The proportion of components (granular blend, modified bituminous emulsion, cement and water) has to be established following a specific mix design method (table 3).

Table 3 Mix design procedures for cold in-plant recycled mixture in San Marino

Optimum water content – first phase			
Cement	[% by aggregate weight]	2	Maximum Dry density; Leaking water < 0.5%
Water	[% by aggregate weight]	3, 4, 5, 6	
Optimum binder content – second phase			
Water	[% by aggregate weight]	optimum	ITS ≥ 0,40 MPa; ITSR ≥ 80 %; ITSM ≥ 3000 MPa
Cement content	[% by aggregate weight]	1.5; 2.0 and 2.5	
Emulsion content	[% by aggregate weight]	4.0; 4.5 and 5.0	
Ratio between bitumen and cement contents > 1			

The first phase of the mix design requires to define the optimum water content. Specimens with different water content have to be compacted with 2 % of cement (by aggregate weight). Each specimen has to be weighed before and after the compaction to measure the leaking water. All specimens have to be dried at 40 °C till constant weight (successive weightings at least one hour apart not differeing by more than 0,1 %) and the dry bulk density has to be measured following the EN 12697-6/procedure D. The optimum water content corresponds to the highest dry density. Moreover, the water loss during the compaction process has to be less than 0.5 % by mixture weight.

The second phase of the mix design requires to define the optimum dosage of binders. Specimens (at least three specimens for each series) with different bituminous emulsion and cement contents have to be compacted with the optimum water content (considerng also water brought into the mixture by the emulsion). The combination of bituminous emulsion and cement dosages has to respect a bitumen/cement ratio upper to 1.

1.2 Mix design procedure complying with Poland specification

Cold recycled mixtures are typically used in Poland for base and sub-base courses. Nowadays, there are no specifications or regulations for designing cold recycled mixtures for binder course. For this reason, the proposed mix design procedure for binder courses follows the adopted procedure for base courses taking into consideration appropriate changes in materials and mixture properties.

The first step deals with the design of granular mix composition, which may contain of RA and virgin aggregate or only RA. In the base course the maximum particle size should not exceed 31.5 mm, but in binder course the maximum particle size is 16 mm, and the grading envelope is slightly altered (table 4). The second step determines the choice of the binding agents, which combination should be determined on the following limits for base course: cement dosage from 1 to 4 % and bituminous emulsion dosage from 2 to 6 %. Emulsion can be used in place of foamed bitumen. The alteration of grading curve leaded also to slight change of the permissible dosage of binding agents. The range was extended in the case of emulsion

up to 7 %. The last step in selecting basic composition is determination of the optimal liquid content taking into consideration additional water and water included in the mineral mix and bituminous emulsion.
After the determination of the basic composition of the mixture, specimen for performance test should be prepared. Cylindrical specimen of 100 mm diameter and 63.5±2.5 mm height should be prepared using Marshal compactor (EN 12697-30). Specimens are prepared in special perforated moulds using 75 blows per side.

Table 4 Requirement for cold recycled mixtures in Poland

Property	Required values								
	Low traffic				Medium traffic				
Voids content [%]	8 – 18				8 – 15				
ITS after 7 days, +5°C, 50 mm/min [MPa]	0.4 – 0.8				0.5 – 1.0				
ITS after 28 days, +5°C, 50 mm/min [MPa]	0.6 – 1.4				0.7 – 1.6				
ITSM after 7 days [MPa]	1000 – 3500				1500 – 4500				
ITSM after 28 days [MPa]	1500 – 5000				2000 – 7000				
ITSR after 28 days, +5°C [%]	> 70				> 80				
Mixture gradation									
Sieve size [mm]	0.063	0.125	0.5	1	2	4	8	16	22.4
Total passing [%]	0-12	2-15	8-30	13-40	21-50	32-63	52-80	85-100	100

2 Performance tests of binder course

2.1 Field experiences in Poland

The mechanical properties were evaluated in terms of the indirect tensile stiffness modulus (ITSM) according to EN 12697-26, strength (ITS) according to EN 12697-23 and Dynamic Modulus according to AASHTO TP79. In ITSM test specimens were tested in strain-controlled mode at 5 °C, and target deformation of 5 µm. The results were evaluated using a pre-defined value of the Poisson's ratio of 0.3. In ITS test specimen were tested with the rate of deformation equal to 50mm/min at 5 °C. Before the testing, specimens were conditioned in thermostatic chamber for 4 h at the testing temperature. In Dynamic Modulus test specimens were tested at three temperatures: 4 °C, 20 °C and 40 °C in strain-controlled mode (100 µstrain). Strain was measured with 3 LVDT sensors (gauge length of 70 ± 1 mm) attached to the specimen. Stiffness modulus were measured at 9 frequencies from 25 Hz to 0.1 Hz at temperatures of 4 °C and 20 °C. At the temperature of 40 °C, test was conducted at an additional frequency of 0.01 Hz. In ITS and ITSM tests, specimens were tested 7 and 28 days after compaction. In the case of Dynamic Modulus, specimens were tested only 28 days after compaction. Specimens were stored in a laboratory condition, under typical temperature and moisture conditions without any special conditioning. Master Curves of stiffness modulus were developed using procedure presented in AASHTO TP79. The used equation (1) assumed that shift factor was calculated using Arrhenius equation. In the further analysis "psi" units were converted into "MPa" units.

$$\log\left|E^*\right| = \delta + \frac{(Max-\delta)}{1+e^{\beta+\gamma\left\{\log f + \frac{\Delta E_a}{19.14714}\left[\frac{1}{T}-\frac{1}{T_R}\right]\right\}}} \tag{1}$$

where:
$|E^*|$ – dynamic modulus, psi (1 psi ≅ 0.00689 MPa);
Max – limiting maximum modulus, (treated as fitting parameter), psi;
f – frequency, Hz;
T_R – reference temperature, K;
T – test temperature, K;
β, γ, δ – fitting parameters;
ΔEa – activation energy (treated as fitting parameter).

Results of the ITSM and ITS test are presented in table 5. Results of dynamic modulus test are presented in table 6. As for Poland the mixture presents similar mechanical properties values as typical CIR used for base course for high traffic [7]. Results of SPT test for Polish mixture for binder course are compared with previous results [8] obtained for base course.
In the intermediate and higher temperatures, the obtained results for similar combination of binding agents are almost the same, with slightly higher modules for mixture for binder course. Contrary, at low temperature, mixture for binder course presents significantly lower dynamic modules. The most probable cause is using smaller maximum aggregate size and higher amount of bituminous emulsion.

Table 5 ITS and ITSM test results for C3B4.2 mixture

Property	**Value**	
	ITSM at +5 °C (MPa)	ITS at +5 °C (MPa)
7 days	4195 ± 195	0.69 ± 0.05
28 days	4921 ± 218	0.78 ± 0.09

Table 6 Dynamic modulus test results and master curve parameters

Mixture designation	Temp. [°C]	Frequency [Hz]									
		25	20	10	5	2	1	0,5	0,2	0.1	0.01
Dynamic modulus [MPa]											
C3B4.2 (PL)	4	5 531	5 357	5 017	4 684	4 263	3 954	3 668	3 311	3 071	
	20	2 957	2 855	2 569	2 305	1 983	1 777	1 594	1 378	1 248	
	40	1 675	1 582	1 365	1 182	980	869	775	676	620	508
Mixture designation	Master curve parameters										
	Max		δ		β		γ		ΔEa		
C3B4.2 (PL)	6.065		4.687		-0.215		-0.511		187 206		

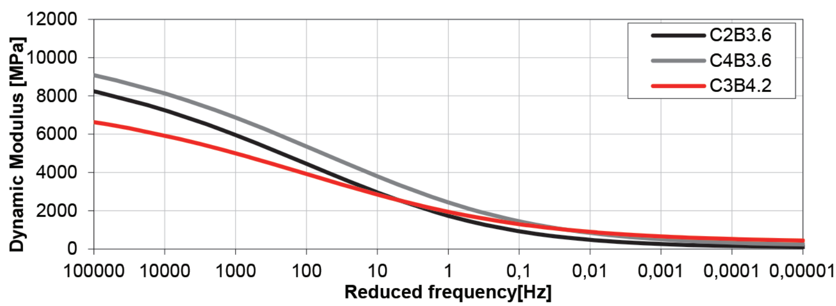

Figure 1 Master curves for CIR mixtures (PL) for binder (C3B4.2) and base (C2B3.6, C2B3.6) course with similar proportions of binding agents, T_{ref} = 20 °C

2.2 Field experiences in San Marino

Following the mix design method established in San Marino the optimum mixture consisted of: 4.5 % of bitumen emulsion (2.7 % of residual bitumen), 2 % of cement, 5 % of water by aggregate weight (including water brought in by emulsion) 88 % of 16RA0/12, 10 % of 0/1 G_F85 and 2 % of mineral filler. According to the specification of the San Marino road agencyError! Reference source not found., this recipe allowed obtaining a dry density of 2123 kg/m^3, an indirect tensile strength at 25 °C of 0.41 N/mm^2 and an indirect tensile stiffness modulus at 20 °C of 4747 MPa after a curing period of 72 hours at 40 °C [9]. Trial sections were built [5] for testing the effects of environmental factors (temperature and humidity) on the evolutions of mechanical proprieties, evaluated in terms of indirect tensile strength (ITS), of cold recycled mixtures during the curing period. Figure 2 shows the evolution of ITS values of laboratory specimens and cores taken from the trial sections as a function of the curing time. The specimens were cured in the laboratory, in a climatic chamber, in different conditions (sealed and unsealed) and temperatures (40 °C and 25 °C).
The ITS values for lab-compacted specimens cured at 25 °C show an increase from 3 to 7 days. However, the curing effects are more evident on unsealed specimens in which the ITS values increased more than 50 % (from 0,27 to 0,41 N/mm^2), while ITS values increased about 10 % for the sealed specimens (from 0,15 to 0,17 N/mm^2). In the short curing time, these results mainly depend on a slower emulsion setting in sealed condition and the reduced contribution of the hydration of cement. Whilst, the unsealed condition facilitates the water evaporation, with a faster increase of ITS values (more than 80 % respect the sealed condition) due to the bitumen setting.
Increasing the curing temperature at 40 °C, the ITS values after 3 days reached an average value of 0.41 N/mm^2, which complies with the design specifications. It should be noted that the same ITS value was obtained after 7 days when curing temperature was set at 25 °C.

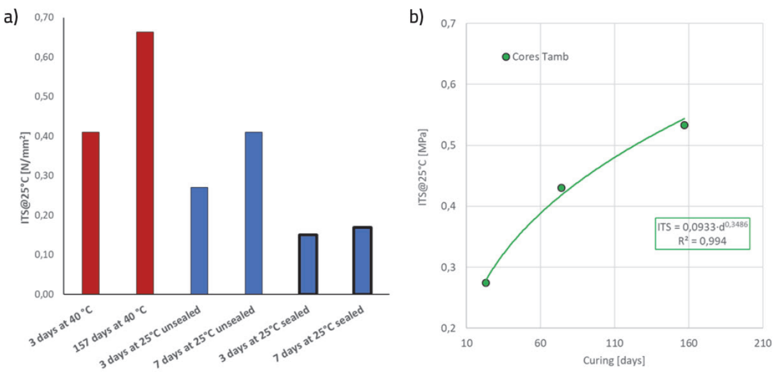

Figure 2 Evolutions of ITS values during the curing period of: a) sepcimens, b) cores

Analyzing the results for the cores extracted from the trial sections, it can be asserted that the increase of ITS values over time follows with good approximation a power law with exponent value lower than 1. The ITS values increase from 0.28 to 0.43 and finally to 0.52 N/mm^2 respectively after 23, 74 and 157 days (Figure 2b).
Using the same power law, the ITS values reached on sealed specimens and lab-curing at 25 °C for 3 and 7 days correspond to about 4 and 6 days of in-situ curing (average ambient temperature of 20 °C) showing therefore a good relationship. On the other hand, the values determined after the accelerated lab-curing at 40 °C for 3 days are reached an extremely longer time (about 70 days of in-place curing). Only after 157 days in-situ curing the core ITS tends to the maximum value obtain in the laboratory.

3 Conclusions

This paper compared two design and testing approaches for cold recycled mixtures and the following conclusions can be drawn:

- the pratical passed experiences and climatic conditions influence the selection and level of performace parameters to be established in a mix design procedure;
- selected RAP fractions and mix design procedures allow achiveing cold recycled mixture with high quality to be used for binder course;
- both contries gathered promising results from applications of cold recycling for binder courses.

Further works should be focused on evaluation of full scale sections subjected to real climatic and traffic conditions.

References

[1] Dołżycki, B.: Polish experience with cold in-place recycling, Bestinfra 2017, IOP Conference Series Materials Science and Engineering, 236 (2017)

[2] Bocci, M., Canestrari, F., Grilli, A., Pasquini, E., Lioi, D.: Recycling Techniques and Environmental Issues relating to the Widening of an High Traffic Volume Italian Motorway, International Journal of Pavement Research and Technology, Chinese Society of Pavement Engineering, 3 (2010) 4, pp. 171-177

[3] Dołżycki, B., Jaskuła, P.: Review and evaluation of cold recycling with bitumen emulsion and cement for rehabilitation of old pavements, Journal of Traffic and Transportation Engineering (English Edition), 6 (2019) 4, pp. 311-323, doi: 10.1016/j.jtte.2019.02.002.

[4] Meocci, M., Grilli, A., La Torre, F., Bocci, M.: Evaluation of mechanical performance of cement–bitumen-treated materials through laboratory and in-situ testing, Road Materials and Pavement Design, 18 (2017) 2, pp. 376-389, doi: 10.1080/14680629.2016.1213506.

[5] Ferrotti, G., Grilli, A., Mignini, C., Graziani A.: Comparing the Field and Laboratory Curing Behaviour of Cold Recycled Asphalt Mixtures for Binder Courses, Materials, Special Issue Characterization of Innovative Asphalt Materials for Use in Pavement Design and Analysis, 13 (2020) 21, doi: https://doi.org/10.3390/ma13214697.

[6] Grilli, A., Graziani, A., Bocci, E., Bocci, M.: Volumetric properties and influence of water content on the compactability of cold recycled mixtures, Materials and Structures, 49 (2016) 10, pp. 4349-4362, doi: 10.1617/s11527-016-0792-x

[7] Dołżycki, B., Szydłowski, C., Jaczewski, M.: The influence of combination of binding agents on fatigue properties of deep cold in-place recycled mixtures in Indirect Tensile Fatigue Test (ITFT), Construction and Building Materials, 239 (2020), doi: 10.1016/j.conbuildmat.2019

[8] Dołżycki, B., Jaczewski, M., Szydłowski, C.: The long-term properties of mineral-cement-emulsion mixtures, Construction and Building Materials, 156 (2017), pp. 799–808, https://doi.org/10.1016/j.conbuildmat.2017.09.032

[9] Grilli, A., Mignini, C., Graziani, A.: Field behaviour of cold-recycled asphalt mixtures for binder courses, International Conference on Sustainable Materials, Systems and Structures (SMSS 2019), New Generation of Construction Materials, pp. 100-107, Rovinj, Croatia, 20-22 March 2019.

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THE IMPLICATIONS OF CLIMATE CHANGE CONDITIONS IN THE PAVEMENT DESIGN

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Abstract

The design of pavement structure is as a set of several activities related to the design of road construction, dimension and model calculations. This includes calculations of load effects, taking into account the properties of the materials, the subgrade conditions, and the climatic conditions. The measurements of climatic conditions in Slovakia were the basis for assessing changes in average daily air temperatures in individual seasons. Since the 19th century we have seen in Slovakia an increase in the average air temperature of 1.5 ° C. Currently, there are scenarios of climate change until 2100. An increase in air temperature is assumed, with an increase in average monthly temperatures of 2.0 to 4.8 °C. In road construction, as well as in other areas of engineering, we must respond to current climate change and also to expected changes. The average annual air temperature and the frost index are the critical climatic characteristics are the main for the design (input parameter) and evaluation of pavement. From the practical side it is possible to use the design maps of average annual air temperature and frost index according to STN 73 6114 from year 1997. In cooperation with the Slovak Hydrometeorological Institute from the long-term monitoring of temperatures, different meteorological characteristics were measured in the current period. From the measurements of twelve professional meteorological stations for the period 1971 to 2020, the dependence between two variables in probability theory is derived. The average annual air temperatures used for prognoses are collected from long-term measurements (fifty years). The design of road constructions and calculations of road construction models, which are in the system design solution (comparative calculations of asphalt pavement- and cement-concrete pavement models), we have also tested road construction materials - especially asphalt mixtures. The results were used to correct the values of input data, design criteria, as well as measures to reduce the impact of changes in climate conditions.

Keywords: climate change, pavement design, cc slab, asphalt mixtures

1 Climate change in Slovakia

Analyses, which it is can point out that since the 19th century we have seen in Slovakia the average air temperature increase by 1.5 °C, regime changes and total atmospheric precipitation. Since 1988, climate change has begun to emerge more rapidly, has been globally registered since 1985. After 1987, only one year had the average air temperature below the long-term average, otherwise, the average temperature was higher. Air temperature will continue to rise and by 2075 we can expect an increase in average monthly temperatures of 2.0 to 4.8 °C - in the cold half-year of about 1.8 °C, in the warm half of the year up to 3.8 °C.

The data sources for the assessment of climatic conditions in Slovakia are the measurements of the Slovak Hydrometeorological Institute. Major experts use them and analyse the conditions for the needs of different sectors (industry, agriculture, energy, etc.). The total increase in the average summer temperature can also be observed on the basis of a comparison of temperatures in the period 1961-2019 (Fig. 1) [8]. For example, for the Bratislava locality, the average annual temperature increased to 12.6 °C in 2019 and the deviation from the normal for the period 1981-2010 reaches 2.0 °C and for the normal period 1961-1990 reaches 2.8 °C.

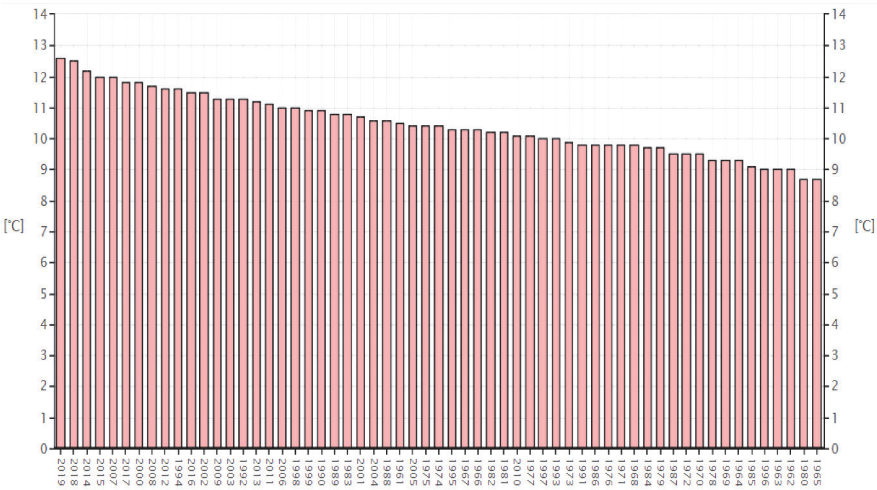

Figure 1 The average annual air temperature in the years 1961-2019 in Bratislava

The changes to the conditions are that road construction has to respond to these changes, not only with adaptation measures. The need for highway and motorway design measures, designing pavement structure, and selection of road building materials and technological processes highlights the increasing number of asphalt pavements failures and defects (track deviations), defects of CC pavements (slab shifts and faults on joints) [1].
With designing and dimensioning of pavement constructions, it is necessary to address the partial problems of improving the properties of road building materials, in particular, asphalt mixtures, the correction of the values of the input data into the calculations as well as the criteria for assessing the design of flexible pavements and rigid pavements.

2 Climatic conditions in the pavement design method

In road construction, a number of building materials are used in pavements structures whose strength and deformation properties are temperature dependent. These properties are very important as input data in analytical design methods and therefore have been the subject of measurement and evaluation. For some parts of Slovakia, were recorded up to 200 - annual series of air temperature measurements. Data are available from the expanded number of meteorological stations of 100 – annual observation period. In earlier data, it is reported that winter (average air temperature 0 ° C) lasts about 2 months and summer 132 days, with a hot summer with average daily air temperatures of more than 20 ° C (only) 49 days [5]. Very valuable are data on the daily air temperature between 1901 and 1950 at various locations in the area, [3]. The temperature regime of the waveforms under different conditions of observation (its characteristics were measured) with different altitudes. Asphalt pavements and pavements with a CC cover were built at the measuring stations.

2.1 Design of asphalt pavements

As an important characteristic of the asphalt pavement temperature regime, we considered the equivalent daily temperature of the layers (not average temperature for the season of the year), when five-sixths of the traffic load is carried by the road. The designed values of the equivalent temperature of asphalt layers and the annual dimensional periods are in Table 1. [6]. The measurements and the data on the annual course of air temperature, surface, and asphalt pavement layers were derived and designed important characteristics for asphalt pavement calculations and design. There are division of the year into three periods, average temperatures of the entire thickness of the asphalt layers, equivalent (calculation) temperatures of the entire thickness of asphalt layers.

Table 1 Dimensional periods of asphalt pavement

Season of the year	Number of days	The proportion of the year	Equivalent temperature
spring, autumn	186	0.5	11 °C
summer	104	0.3	27 °C
winter	75	0.2	0 °C

Surface temperatures and individual asphalt pavements layers are dependent on air temperature. For the average and maximum values, the general equation applies

$$T_{m,asf} = k \cdot T_{m,r} + C \; [°C] \tag{1}$$

where
$T_{m,r}$ - is average annual air temperature,
C - is a constant, altitude function.

A comparison of average temperatures of asphalt layers in changed climatic conditions in Slovakia´s localities is in Table 2.

Table 2 Average temperatures of asphalt layers

Locality	Tm,r (STN 73 6114)	Tm,r (1998 - 2020)
	Tm,asf	Tm,asf
Bratislava	33,74	35,72
Žilina	30,04	32,28
B. Bystrica	30,04	32,81
Poprad	28,46	30,04
Košice	31,36	34,00

2.2 Design of cement concrete pavements

The climatic conditions of the cement concrete (CC) pavements are characterized by the average annual temperature of CC slabs and amplitude of this temperature in the annual cycle, the average and highest values of positive and negative temperature differences between the top and bottom surface of CC slab.

These characteristics of temperature regime are important for stress and strain calculations of CC slabs. They were derived from measurements on constructions in terrain, from the sum of temperature ranges for 97 % of cases. The average annual temperature of slabs is considered as the annual average of daily air temperatures. The amplitude of the annual air temperature is calculated according to

$$Ar,h = Tm,r + 36{,}9 - 0{,}038\ hB\ [°C] \tag{2}$$

where
Tm,r - is average annual air temperature (°C),
hB - is thickness of the CC slab (mm).

The design (calculation) values of the temperature differences of the top and bottom surfaces of the CC slabs are calculated from the relationship to the average annual air temperature, considering the thickness of the slab. Empirically derived formulas have the form:

- for positive temperature difference:

$$+\Delta\ Tn = 12{,}440 - 0{,}6\ Tm,r + 0{,}028\ hB\ [°C] \tag{3}$$

- for negative temperature difference:

$$-\Delta\ Tn = 6{,}214 - 0{,}3\ Tm,r + 0{,}0113\ hB\ [°C] \tag{4}$$

A comparison of the temperature regime of CC slabs in changed climatic conditions in Slovakia´s localities is in Table 3.

Table 3 Temperatures regime of CC slabs

Lokality	$T_{m,r}$ (STN 73 6114)								
	hb = 220 (mm)			hb = 250 (mm)			hb = 300 (mm)		
	$A_{r,h}$	$+\Delta T_h$	$-\Delta T_h$	$A_{r,h}$	$+\Delta T_h$	$-\Delta T_h$	$A_{r,h}$	$+\Delta T_h$	$-\Delta T_h$
Bratislava	38,34	12,72	5,76	37,20	13,56	6,10	35,30	14,96	6,66
Žilina	35,54	14,4	6,60	34,4	15,24	6,94	32,5	16,64	7,50
B. Bystrica	35,54	14,4	6,60	34,4	15,24	6,94	32,5	16,64	7,50
Poprad	34,34	15,12	6,96	33,2	15,96	7,30	31,3	17,36	7,86
Košice	36,54	13,8	6,30	35,4	14,64	6,64	33,5	16,04	7,20
	$T_{m,r}$ (1998 - 2017)								
	hb = 220 (mm)			hb = 250 (mm)			hb = 300 (mm)		
	$A_{r,h}$	$+\Delta T_h$	$-\Delta T_h$	$A_{r,h}$	$+\Delta T_h$	$-\Delta T_h$	$A_{r,h}$	$+\Delta T_h$	$-\Delta T_h$
Bratislava	39,84	11,82	5,31	38,7	12,66	5,65	36,8	14,06	6,21
Žilina	37,24	13,38	6,09	36,10	14,22	6,43	34,20	15,62	6,99
B. Bystrica	37,64	13,14	5,97	36,5	13,98	6,31	34,6	15,38	6,87
Poprad	35,54	14,4	6,60	34,4	15,24	6,94	32,5	16,64	7,50
Košice	38,54	12,6	5,7	37,4	13,44	6,04	35,5	14,84	6,6

An important characteristic of the temperature regime of the CC covers is the temperature difference between the top and bottom surfaces of CC slabs. It is most often the temperature gradient, this is - by changing the temperature to the unit thickness of the slab °C/mm, or simply a difference in the temperature of the top and bottom surfaces. The temperature difference is usually positive during the day, negative during the night. The temperature gradient in the CC slab causes deformation, at a positive temperature gradient, the slab has a convex shape. Depending on the thickness of the slab, its dimensions (width and length) and friction on the contact of the slab and the subbase, this can lead to stresses as large as those caused by the vehicle loading [2].

3 Conclusion

After 1985, climate change began to manifest strongly, in the territory of Slovakia too. The growth of the mean annual air temperature was greater than 1.5 ° C. According to climate change scenarios in the next period (until 2100), the increase in average monthly air temperatures may be higher, e.g. in the cold half of the year about 1.8 ° C, in the warm half of the year up to 3.8 ° C

In system solution procedure for design, calculations, and assessment of flexible asphalt pavements and rigid cement concrete pavements, climatic conditions are considered. But expected climatic changes have raised demands for solutions and related issues.

References

[1] Grošek, J., Zuzulová, A., Březina, I.: Effectiveness of dowels in concrete pavement, Materials, 12 (2019), doi: 10.3390/ma12101669

[2] Leitner, B., Decky, M., Kovac, M.: Road pavement longitudinal evenness quantification as stationary stochastic process, Transport, 34 (2019) 2, pp. 195–203, doi: https://doi.org/10.3846/transport.2019.8577

[3] Decký, M., Remišová, E., Kovalčík, L., Dibdiakova, J., Hájek, M.: Climatic Characteristics of Slovakia in the Years 1971 to 2011 from the Aspect of Pavement Design, IOP Conference Series: Earth and Environmental Science, 95 (2017) 022056, doi: 10.1088/1755-1315/95/2/022056

[4] Cihlářová, D., Fencl, I., Cápayová, S., Pospíšil, P.: Use of adhesion promoters in asphalt mixtures. In Slovak Journal of Civil Engineering, 26 (2018) 1, pp. 19-24, doi: 10.2478/sjce-2018-0003

[5] Hodáková, D., Zuzulová, A., Cápayová, S., Schlosser, T.: Impact of Climate Characteristics on Cement Concrete Pavements, SGEM 2017, 17th International Multidisciplinary Scientific GeoConference, Albena, Bulgaria, 29 June - 5 July 2017, doi: 10.5593/sgem2017/41/S19.059

[6] Grošek, J., Chupík, V., Brezina, I., Janku, M.: Analysis of concrete pavement deformations on the road test section, 18 th International multidisciplinary scientific geoconference SGEM 2018, Sofia, Bulgaria, 2018, doi: 10.5593/sgem2018/2.1

[7] Samek, M.: Regionálne cesty Bratislava, a. s., Photo documentation

[8] www.shmu.sk

[9] Asphalt in figures 2016-2018, document available at www.EAPA.org

[10] STN EN 13 108-8 Bituminous mixtures - Material specifications - Part 8: Reclaimed asphalt, 2016.

[11] TP 043 Recyklácia asfaltových zmesí za horúca v obaľovacích súpravách, Hot recycling of asphalt at a central plant, 2011.

[12] TKP 6 Hutnené asfaltové zmesi, Ministry of transport and construction of the Slovak Republic, 2017.

6

TRAFFIC: MANAGEMENT, MONITORING, INTEGRATION AND MOBILITY

CHANGES IN TRAFFIC INFRASTRUCTURE WITH THE ARRIVAL OF AUTONOMOUS VEHICLES

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Abstract

Autonomous vehicles represent a significant step forward in traffic safety and efficiency. Although it will be quite some time before all vehicles on public roads are autonomous, it is certainly advisable to consider the changes that will need to be introduced to traffic and traffic infrastructure. Autonomous vehicles will significantly increase the efficiency and use of public transportation and car-sharing, which will ultimately mean fewer cars on the roads and less need for parking in urban areas, or their conversion into a type of waiting area. Also, fewer vehicles, whose software has a drastically faster reaction time and much better control of the vehicle, will also mean less traffic jams, greater intersection flow, less need to channel traffic in traffic lanes and will remove traffic lights almost entirely out of use. This paper will look at the form of transport infrastructure and its variants in the case of mixed traffic with autonomous vehicles and drivers, as well as the situation with fully autonomous traffic without people behind the wheel.

Keywords: autonomous vehicles, traffic infrastructure, drivers, AI (Artificial intelligence), car sharing

1 Introduction

Today's transport infrastructure has been developed for hundreds of years in order to adapt to vehicles and the transport of people and goods. With the growth of the economy and society, the need for greater infrastructure is also growing. With the advent of smart vehicles and the development of computer science and its aspects such as artificial intelligence, infrastructure must also take on new and modern features in order not to lag behind. However, the parameters used today in the design of transport infrastructure are oriented to the physical and mental abilities of drivers [1]. With the already mentioned arrival of smart vehicles, the human factor in traffic will be gradually eliminated, and the project parameters will take on new values. This paper will present the current state of road transport infrastructure in which the human factor is dominant, and will also present a variant of road infrastructure in which the human factor does not play a role in driving vehicles (personal and public). Proposed guidelines will be given for the transition period in which traffic will be mixed and where the role of decision-making in traffic will be played in one part by drivers, and in the other part by autonomous systems, i.e. artificial intelligence. In this paper, the focus will be only on the road part of the transport infrastructure.

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2 Overview of the elements of the existing transport infrastructure

Although urban and out-of-town transport infrastructure consists of equal elements, it is noticeable that the urban part of the infrastructure, both transport and others, is significantly more complex. Traffic in urban areas includes many more modalities and it is necessary to take into account pedestrians, people with disabilities, cyclists, cars, trucks, public transport vehicles, tram tracks and the like. It can be said that the urban infrastructure is less subject to strict compliance with the rules, and focuses more on engineering solutions that sufficiently meet the rules and satisfy all traffic participants.

2.1 Urban transport infrastructure

Given multimodality, urban infrastructure consists of many different elements that affect its use and utilization of available space. In addition to roads in urban areas, parking spaces (private, public, as well as garages) play a major role in planning and utilization of space. Existing parking spaces are becoming insufficient for the growth of the number of personal vehicles, and there is a need to increase them [2]. The current dimensions of parking spaces for their comfortable use by drivers are becoming undersized due to the dimension growth of vehicles [3] (Fig. 1), and the current regulations have not been updated with regard to the size of the personal vehicle [4]. Therefore, there is a need to increase the existing parking spaces or to find new locations for them.

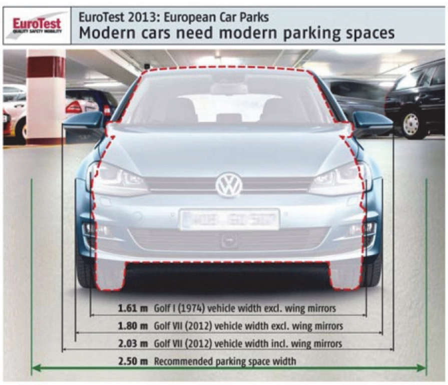

Figure 1 Display of the growth of the dimensions of the same car model over the years in relation to the standard width of the parking space [5]

Intersections in urban areas come in a variety of shapes and sizes and mainly consist of multiple traffic lanes. Such intersections are also guided by significant vertical and traffic lights. It follows from all this that intersections take up a lot of space and that with the growth in the number of personal vehicles, they will potentially require even more space. As most urban areas are extremely limited by free space, the solution for such situations is to remove the surrounding buildings in order to free up space for a larger classical intersection or for a roundabout. Otherwise, over time, there will be increasing traffic jams at intersections that are not designed for such a large number of vehicles. Signalization (horizontal and vertical) is an important factor in traffic management, but it is still there mostly because of people, because of the reduction of the possibility of human error and release from liability in the event of a traffic accident [6].
Another element of urban infrastructure and traffic is public transport. It includes vehicles of different dimensions as well as infrastructure elements such as bus and tram stations that are incorporated into the city network. Although these are additional elements of the urban

environment that are not so much in the out-of-town road network, they are there to reduce traffic congestion and the number of other vehicles on the roads and make the transport of people and goods more accessible and cheaper. Pedestrians and cyclists are also part of the traffic in urban areas and enter the traffic network with pedestrian paths and bicycle paths and lanes. All these elements need to be kept in mind, both when designing urban infrastructure and even more so when participating in traffic in the urban environment. The goal of the designer is to make all elements of the infrastructure as simple, intuitive and understandable as possible, because the people who will use it are prone to mistakes, carelessness, fatigue and similar other characteristics.

2.2 Out-of-town infrastructure

The out-of-town road network consists of motorways and state roads at a higher level and county and local roads at a lower level. At both levels, the existing road network needs to increase capacity due to the increasing number of people [7] traveling by car or intercity, which is reflected in the widening of the cross-section of the road or the use of existing pavements for additional lanes, which means lower traffic speeds. Highways and state roads require a high level of maintenance and equipment. Although they are designed for higher design speeds, their main characteristics derive from the way people use vehicles and from all the aforementioned characteristics of human drivers. Given the higher speeds of movement, it is necessary to ensure larger fields of view on out-of-town roads, both horizontally and vertically. Also, due to the human characteristic of fatigue and deconcentration, certain sections of roads are designed with curves although they could be designed as a straight line [1]. Thus, although out-of-town transport infrastructure is simpler than urban and is not so limited by space and a multitude of transport modalities, most of the design parameters still derive from the anthropocentric design system.

3 Infrastructure for fully autonomous traffic

When we talk about fully autonomous vehicles and fully autonomous traffic, locally or globally, we are talking about the end of an anthropocentric design system in which all elements of infrastructure are subordinate to human. In such traffic where human does not make decisions as a driver and artificial intelligence manages everything, it means that the infrastructure for this traffic no longer has to be understandable to human, does not have to be intuitive and adapted to the human mind to understand it faster and easier. The transport infrastructure should be adapted so that the software and hardware can interpret it as easily as possible. This implies completely new project parameters and significant simplification of the infrastructure and, ultimately, its reduction.

3.1 Urban transport infrastructure

After real estate, in many cases, a personal car is the biggest investment for many people. But despite this, people use cars on average 4 % of the time [8]. During this time, the car loses value and requires additional investment in the form of maintenance and repairs. The market already understands this fact, which has resulted in the establishment of companies with new forms of taxi services. Fleets of such taxi companies are growing, and companies are investing in the development of autonomous vehicles to make their fleet cheaper, safer and more cost-effective. With enough such companies having ever-growing fleets of autonomous vehicles on the roads, car sharing will replace owning a personal car and in the future all cars as well as other vehicles will be autonomous. When the time comes that all vehicles in traffic are autonomous, traffic should become much more efficient, faster, simpler and

cheaper. In that case, the transport infrastructure should be adapted to this and therefore take on a completely new look. Parking spaces will be almost completely eliminated from cities as vehicles will be constantly in operation [9]. Intersections will have a minimum number of lanes and traffic management in this way will no longer be necessary. Vertical signaling will be largely unnecessary, while horizontal signaling will always have to be ideal.
Thus, the transport infrastructure will experience downsizing. It will require enhanced maintenance to avoid errors in communication between the infrastructure and the vehicle. It will be necessary to introduce some new elements just for better communication between the vehicle and the infrastructure, which will mean the removal of traffic lights and traffic signs from use. But as vehicles will not only communicate with the infrastructure, but also with each other, traffic rules will change and the right of way at intersections will be almost unnecessary because every vehicle will know the trajectory and speed of all surrounding vehicles at all times.
When creating future regulations, guidelines, etc., special attention will need to be paid to the points of conflict between autonomous traffic and other participants (pedestrians, cyclists). Participants in traffic outside autonomous vehicles are the most endangered participants in traffic and it is necessary to find the best possible solution how to include them in traffic next to autonomous vehicles and ensure their mobility, which they still have today, while being protected. Artificial intelligence-driven vehicles will be able to anticipate and control conflict situations [10], but pedestrians and the like will have to go through a phase of getting used to and learning how to behave in such traffic. Because of them, traffic signals, although unnecessary for autonomous vehicles themselves, will be a necessity for other road users.

3.2 Out-of-town infrastructure

Although urban infrastructure will experience significant donwsizing, this may not necessarily be the case with the out-of-town road network. The design elements will not change significantly as the vehicles will still be subject to the same laws of physics as today. Radius, widenings, slopes and other elements will remain unchanged. What will change are the need for visibility fields as well as the need for sufficient stopping visibility given that when braking, the drivers reaction time will no longer be a variable. And with V2V (Vehicle to vehicle) communication [11], in case they have to, all vehicles can decelerate at the same time with equal intensity in the convoy. Traffic lanes and road widths will be able to be reduced, as will sidewalks, but as in the case of urban infrastructure, horizontal signage will need to be impeccably maintained and executed.

4 Transition period

As long as people in traffic participate as drivers, the infrastructure will have to be tailored to them as well, no matter how few human drivers there are. But with smart and gradual decisions, it will be possible to adapt the infrastructure to both human drivers and autonomous vehicles, just as it will be possible to motivate people to stop being drivers. If the elements of infrastructure described above are considered and placed in the context of mixed traffic, with an understanding of how autonomous vehicles function and can function, the evolution of these elements can be predicted.
With the growth of autonomous vehicles, personal vehicle ownership will be declining and car sharing on the rise. This means that people will have less need for parking spaces where they will leave their cars. Lower demand for parking spaces will result in less and less supply and in conversion of excess space. In mixed traffic, this will mean that the number of parking spaces needed by human drivers will be retained in urban areas, and if it is assumed that au-

tonomous vehicles (taxis) will be in operation almost constantly and will rarely have to park for long periods, for them there will be no need for parking spaces but certain stations where it will be possible to stay for a shorter time or slow traffic lanes where empty vehicles will be able to circle slowly in the convoy until they are called for use. In larger cities where parking is problematic and where parking buildings have been built, this will also mean relieving traffic and repurposing garage space [9]. Of course, during the night or certain days of the year when movements are declining, autonomous vehicles will also have to be parked somewhere. For such situations, certain areas outside cities can be converted into storage areas for autonomous vehicles. Although this means that the total amount of parking spaces has not decreased significantly and they have only moved from the city center to the outskirts, the occupied area for stationary vehicles will still be smaller if we take into account that autonomous vehicles do not need parking lots of the same dimensions as cars that are human operated. With time and less and less use of personal vehicles, parking spaces will disappear from urban areas [9] and this area will be available for use by pedestrians, cyclists, etc. If we look at intersections in mixed traffic and the same case of the growth of car-sharing and the decline in car ownership, it is possible to suggest steps to adjust the intersection. In the very beginnings of mixed traffic, in a multi-lane intersection, certain lanes can be adapted to be used exclusively by autonomous vehicles as is the case with special lanes for buses and fire trucks in some cities (Fig. 2). With the gradual increase in the number of autonomous vehicles, the number of dedicated lanes may also increase in order to increase the flow of intersections. In addition to lane adjustment, intersections need to be equipped with additional horizontal and vertical signaling and other equipment for I2V (Infrastructure to vehicle) and V2I (Vehicle to infrastructure) communication [11]. Some autonomous vehicles are already used today on some of the busiest roads in the world [13] (Fig. 3).

Figure 2 Example of a lane reserved for buses (taxis and cyclists), which can be used for autonomous vehicle traffic [12]

Figure 3 Example of an autonomous vehicle from Waymo LLC without a driver driving on an American road [14]

In the out-of-town part of the road network, the same approach can be used as for intersections. On motorways and other multi-lane roads, one lane can only be used for autonomous vehicles. As the number of these vehicles increase, the number of dedicated lanes may increase, but their characteristics may also change. One autonomous lane will be able to be used only for trucks and the other for smaller passenger vehicles. In another variant, one lane can be used for slow driving and the other for fast driving. In the case of multiple lanes, multiple variants are possible. What will differentiate these autonomous lanes from normal lanes is the ratio of comfort to vehicle volume. Autonomous vehicles will be able to ride in a convoy, very close to each other, like a train wagon. In the case of the human behind the wheel in such a situation, mistakes are almost inevitable and can be fatal. But in the case of autonomous interconnected vehicles in their own lane, errors are reduced to an absolute minimum. The gradual growth in the use of car sharing will gradually change the transport infrastructure, not only for drivers and vehicles, but also for pedestrians and cyclists and for other forms of personal mobility that do not include road traffic. The removal of parking spaces will mean larger pedestrian and bicycle corridors, and will even open up places to build tram tracks in city centers. Ways in which infrastructure can be gradually adapted to mixed traffic and autonomous traffic, include increased maintenance of roads and road equipment, introduction of new infrastructure elements and elimination of some existing elements, as well as redistribution and redesign of certain elements.

5 Conclusion

Although autonomous vehicles are already relevant today, it will be many more years before they fully assume a human role in traffic. This mostly depends on the change of business models of existing large car manufacturers and on the acceptance and testing of such vehicles by the market. This gives infrastructure designers a certain advantage because the future is predictable and it is possible to prepare well for major changes in traffic. However, the problem of the construction sector and infrastructure is very low agility and poor ability to progress and adapt quickly, as well as the fact that investments in infrastructure are large and demanding. While on the other hand there are car manufacturers that are much more agile in adapting to new market demands and the IT sector which is extremely agile and changing on an almost daily basis. Therefore, it is clear that autonomous vehicles will get on the roads long before the infrastructure is ready for it. In order to avoid late adjustments to such traffic and, if possible, to speed up the arrival of such traffic on public roads, it is in everyone's best interest for infrastructure designers to work closely together with the automotive industry and the IT sector. Given that autonomous vehicles are already used today on some of the busiest roads in the world, it is evident that significant infrastructural adjustments are not currently necessary for autonomous vehicles operating in traffic. But in the case of cooperation of the mentioned sectors, the infrastructure could experience rapid and economically viable changes that would significantly facilitate the work of the IT and automotive sectors to develop autonomous vehicles and put them on the market faster. Those sectors should be the ones to instruct infrastructure designers on how to design the infrastructure so that vehicles can use it in the best and safest way.

References

[1] Korlaet, Ž.: Uvod u projektiranje i građenje cesta, Građevinski fakultet Sveučilišta u Zagrebu, Zagreb, 1995.

[2] Eger, R.: Critical desing parameters for garages, Građevinar, 65 (2013) 6 pp. 547-552, doi:10.14256/JCE.806.2012

[3] Jato.com, https://www.jato.com/suv-registrations-set-record-accounting-1-4-cars-sold-europe-2016/, 20.05.2020

[4] Republic of Croatia: Pravilnik o prometnim znakovima, signalizaciji i opremi na cestama, Narodne novine, NN 92/2019.

[5] Eurotest 2013, European car parks, https://www.thelocal.es/userdata/images/1392291994_Untitled-1.jpg, 20.05.2020.

[6] Cerovac, V.: Tehnika i sigurnost prometa, Fakultet prometnih znanosti, Zagreb, 2001.

[7] Croatian Bureau of Statistics, Registered road vehicles and road traffic accidents, 2019, https://www.dzs.hr/Hrv_Eng/publication/2020/05-01-04_01_2020.htm, 16.04.2020.

[8] Bates, J., Leilbing, D.: Space Out - Perspective on parking policy, London, RAC Foundation, 2012.

[9] Millard-Ball, A.: The Autonomous Vehicle Parking Problem, Transport policy, 75 (2019), pp. 99-108

[10] Science Alert: Tesla's Autopilot Predicts a Crash Freakishly Early, https://www.sciencealert.com/watch-tesla-s-autopilot-predicts-a-crash-freakishly-early, 20.05.2020.

[11] Steyn, W.J.vdM, Maina, J.W.: Guidelines for the use of accelerated pavement testing data in autonomous vehicle infratructure research, Journal of traffic and transportation engineering, 6 (2019) 3, pp. 273-281, doi: https://doi.org/10.1016/j.jtte.2019.05.001

[12] Hawlisch, M.: Bus lane, 2006. https://simple.wikipedia.org/wiki/Bus_lane#/media/File:Busspur_und_Haltestelle_in_Mannheim_100_9128.jpg, 20.05.2020.

[13] aDigitalNomad.net: Tesla autopilot 2019.16.2 vs New York City heavy traffic, https://www.youtube.com/watch?v=hNKBfeHmGjI, 27.05.2020.

[14] dw.com: https://www.dw.com/image/39629871_403.jpg, 20.05.2020.

INTEGRATION INDEX FOR MOBILITY AS A SERVICE

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Abstract

The integration in transport informatics is facilitated by the rapid development of Information and Communication Technology. One of the realizations of the integration is Mobility as a Service (MaaS), which is proposed as a data-driven, user-centric, personalized mobility service. It integrates various forms of mobility services covering the entire travel chain. Qualitative methods have been applied in existing studies to analyse the integration of MaaS. However, a comprehensive quantitative method is still missing, which could be introduced as a supplementary tool to compare MaaS services. Therefore, we have developed a weighted elaboration method to calculate the complex integration index for MaaS systems. Three aspects are determined as variables, which are the functions of the MaaS application, involved transport modes as well as the tariff structure. Moreover, the organization as the backbone of such integration is considered as the fourth aspect. The integration phases of MaaS are introduced regarding these four aspects, then the calculation method of the complex index is developed by considering the weighted variables. Fourteen MaaS services are evaluated with the method and categorized by organization aspect. We found that public authority is proposed to be the inter-city MaaS operator, and the private company is proposed to be the MaaS operator in intra-city or national level. Our method may support decision-makers to have an abstract overview of MaaS and identify the possible development stage.

Keywords: mobility as a service, integration phases, organization, integration index

1 Introduction

Various mobility services are available, but efficient integration of them is still missing. Integration facilitates the opportunity to use the entire public transport system across a local, regional, even international area. Independently from modes, tariffs, fares, schedules, ticket systems, etc. Integration is needed to improve comfort level, the quality of information, as well as to reduce travel time, and the cost of mobility services [1]. Mobility as a Service (MaaS) is a user-centric mobility service based on smart solutions, which is proposed to facilitate the integration of mobility services. Cooperation is managed systematically by the third party MaaS operator, users experience the 'one-stop-shop' service provided by different 'service providers'. The service supplied to users through a single digital platform integrating all travel related functions, e.g. planning, booking, ticketing, payment. Users purchase bundled services from MaaS operator and pay service fare to this operator. Thus, MaaS is identified as a seamless integrated door-to-door mobility service [2].
Qualitative approaches to analysis the transport integration have been presented [3], [4], but quantitative method is missing. In order to introduce a supplementary approach, our objective is to develop a quantitative method to study the integration index of transport system.

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We take the MaaS operator, this organization into consideration. In addition, an objective weight assignment approach is introduced. The research questions are:
- What aspects are taken into consideration regarding integration?
- How is the phase of integration identified?
- How to assess MaaS in integration point of view?

The remainder of the paper is structured as follows. State of the art is summarized in Section 2. In Section 3, the variables are determined, the phases of integration are introduced. The assessment method of integration is elaborated in Section 4. Fourteen MaaS services are considered; the discussion of the assessment is presented in Section 5. The paper is completed by the concluding remarks.

2 State of the art

The integration of mobility services is to be enhanced by several aspects. Collaboration between public and private service providers increase the possibilities to integrate transport modes, and it facilitates the introduction of MaaS operator in organizational structure [5]. The MaaS is introduced to meet subscription-based, monthly mobility plan too [6]. In addition, mobility data sharing platform enhance the realization of functionalities regarding smartphone application, especially incorporation of payment function into MaaS application [2]. However, MaaS implementation still remains at a rudimentary level, governance and legislation issues have not comprehensively studied yet [7].
Typically, an index is introduced to evaluate phenomena comprehensively, e.g. smartness, sustainability, liveable city index. An index is calculated by aggregation of weights multiplying scores of relevant indicators (variables in this context) [1], [8], [9], [10]. Various index calculation methods are applied in evaluation or benchmarking of comparative studies [10], [11]. Several researchers have studied the sustainability index of transport regarding cities [1], industries [9], freight transport [10], etc. Mostly, fuzzy logic approach was applied to reveal the uncertainties of respondent's responses [9], [10].
MaaS implementation is significantly facilitated by transport informatics. We develop an assessment method to evaluate the 'backbone' informatics integration of MaaS regarding the organizational role of MaaS operator.

3 Phases of integration

3.1 Selection of variables

To describe the integration of MaaS, three aspects as variables are determined based on literatures and MaaS implementations: function, mode, tariff structure. The development tendency of the variables is presented in Fig. 1.

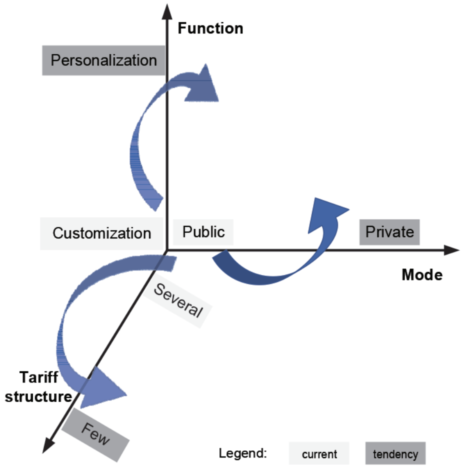


Figure 1 Development tendency of variables

Customized functions require input about users' preferences manually, e.g. preferred mode; usually predefined options are available. By contrast, personalized functions apply artificial intelligence, users' preferences are set automatically considering behaviour and habits, e.g. the recommended route with preferred mode. The functions of smartphone applications vary from basic customized functions towards the advanced personalized functions.
The boundary between public and private modes become not so clear because of the rise of shared mobility. Typically, high volume of passengers is transferred by the public transport (PT) modes. However, the taxi can be regarded as the most flexible public transit mode. Car/bike-sharing are transitional modes. Since the essence is the information platform, ridesharing and ride-sourcing are categorized into private modes. The incorporate tendency of modes into MaaS is firstly public ones and then private modes.
Mobility plan is introduced as the combination of several transport modes into a package and provided by MaaS, the complex tariff structure is packaged and delivered to users 'as a whole'. From the travellers' point of view, the journey is payed 'as a whole' as well. Travellers do not need to deal with the detailed tariff structure of each mode, in this sense, tariff structure is much more simplified.
Since MaaS operator plays a significant role, the 'organization' is also introduced as the fourth aspect. Typically, the third party MaaS operator is either public authority or private company. Determined variables are summarized in Table 1.

Table 1 Variables of integration analysis

Variables			**Description**
Integration	A_1	Function	mapping, planning, booking, ticketing, payment, reminding, etc.
	A_2	Mode	involved transport modes
	A_3	Tariff structure	pricing, fare collection methods, e.g. subscription of mobility plan, passes, pay-as-you-go
	A_4	Organization	private company or public authority

3.2 Identification of phases

Integration phases considering identified variables are summarized in Fig.2. The differences between the term ‘integrator’ and ‘operator’ in MaaS are the following: Integrator provides an established platform to collect the available mobility service websites, users purchase mobility services from service providers by clicking and turning to the corresponding websites. Operator has a higher organizational role. A unified interface for service providers and users is provided. Users purchase mobility services from MaaS operator.

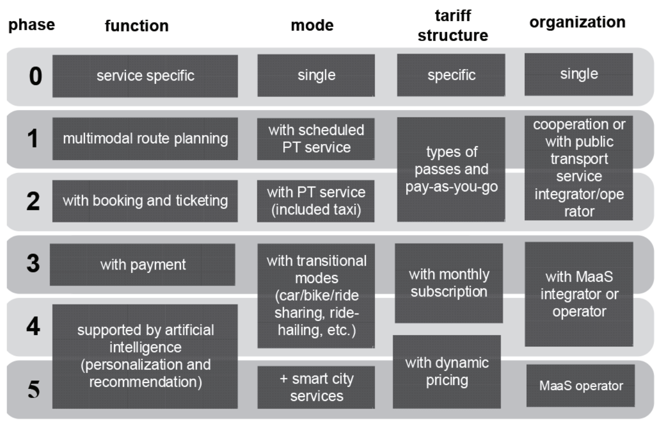


Figure 2 Identified phases of integration of MaaS

Phase 0: non integration: Integrated route planning, booking, ticketing and payment are available only for one specific service (e.g. ride-sourcing); modes or services of transport are operated separately; ticket system and fare collection are managed separately.

Phase 1: scheduled public transport service integration: Function of multimodal journey planning is available for public transport modes, e.g. combination of bus, tram, metro. Various types of public transport passes are provided; cooperation is mainly among pubic transport modes.

Phase 2: entire public transport service integration: Additional functions, such as booking and ticketing, payment, are available provided by a third party. Usually, taxi service is integrated, several public transport service providers cooperate. A common interoperable platform is available. The role of public transport service operator in organization is significant.

Phase 3: partial public-private transport service integration: Journey planning, booking, ticketing and payment functions for various transport modes are integrated in one application. Bike-sharing, car-sharing such transitional services are integrated. Advanced tariff options are provided, e.g. pay-as-you-go and monthly subscription. MaaS integrator or MaaS operator are introduced.

Phase 4: entire public-private transport service integration: Additional functions, such as reminder and PoI (Point-of-Interest) recommendation are provided. Practically almost all modes are integrated. More options for combinations are listed in monthly subscription; dynamic pricing is applied. MaaS integrator or MaaS operator manages the services.

Phase 5: fully integrated mobility service: Personalization and recommendation are introduced. All modes are integrated, even aircraft services; monthly subscription is widely available, pay-as-you-go is suggested for tourists. The organizational role of MaaS operator is significant.

4 Assessment method

The steps of integration index calculation are: Step 1: determination the weights of variables; Step 2: determination the scores of variables; Step 3: calculation the index value by aggregation of scores.

Step 1: Discrete weights W_i (from 0.1 to 0.4) are assigned to variables A_j referring to the identified phases of integration (phase 1 to phase 4), respectively. The values and scales of variables are presented in Table 2. The aspect 'organization' is not considered in index calculation, it is used for classification purpose in demonstration of the applicability method.

Table 2 Weights and scores of variables

Phase	Weight of variable	Scales assigned to variable A_j					
		A_1 Function	Score	A_2 Mode	Score	A_3 Tariff	Score
1	W_1=0.1	Mapping Journey planning	1 or 2	Scheduled intercity PT service (short distance)	1	Public passes	1
2	W_2=0.2	Booking Ticketing	1 or 2	Additional taxi service	1	Pay-as-you-go	1
3	W_3=0.3	Payment	1	Additional car/bike sharing service	1	Monthly subscription	1
4	W_4=0.4	Additional advanced functionalities	Each one count 1	Additional private service, or with scheduled intracity PT service (long distance)	Each one count 1	Dynamic pricing	1

Step 2: The availability of variables in each phase could be checked on corresponding smartphone applications, accordingly, the objective scores are assigned during assessment regarding specific service. For example, a service application is selected, when W_1=0.1, regarding A_1, if function of mapping and journey planning are both available, then scales=2 is assigned, if only one of them is available, then scales is 1. The similar checking of A_2 and A_3, then for W_2, W_3, and W_4, same checking rounds are performed. Scores of variables are obtained.

Step 3: Generally, existing approaches are applied to rank MaaS services according to identified phases; however, we assign weights to variables according to different phases. The integration index is calculated with eq (1).

$$I = \sum_{i=1}^{4}\left(W_i \cdot \left(\sum_{j=1}^{3} A_j\right)\right) \tag{1}$$

Where:
'I' - represents the integration index,
'W' and 'A' - represents the weight of variable and the variable, respectively.
'i' - indicates the phase; i = 1, 2, 3, 4.
'j' - indicates the variable; j=1, 2, 3.

Simply, first summarize the scores of variables, then multiply the summarized scores with the weights according to the phases. The result of index calculation is the aggregation of results obtained from step 2. Phase 0 and 5 should be eliminated, as integration is not available in the former, fully implemented integration is not available in the later.

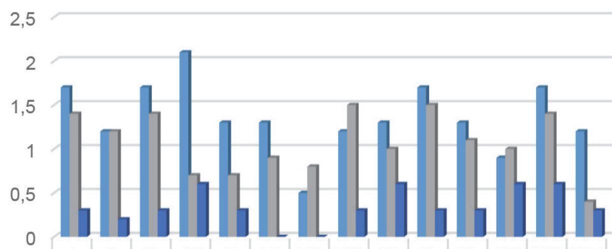
5 Result and discussion

Fourteen smartphone applications of MaaS service, which are available in northern and central Europe, have been selected. The corresponding services are assessed by the developed method. The selected services are summarized in Table 3. The MaaS operator is either public service provider (indicated by bold letter) or a third-party private company.

Table 3 The selected MaaS services

I	II	III	IV	V	VI	VII
Moovel	Qixxit	HVV	GVH Hannover-mobil	Leipeig-mobil	Moovit	Urbi
VIII	IX	X	XI	XII	XIII	XIV
Mozio	Smile	Wien-mobil	Nordest-mobil	Ubigo	Whim	Kyyti

Relevant functionalities, i.e. availability of variables, have been tested. The result regarding scores of variables is presented in Fig. 3.



	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	XIII	XIV
Function	1,7	1,2	1,7	2,1	1,3	1,3	0,5	1,2	1,3	1,7	1,3	0,9	1,7	1,2
Mode	1,4	1,2	1,4	0,7	0,7	0,9	0,8	1,5	1	1,5	1,1	1	1,4	0,4
Tariff	0,3	0,2	0,3	0,6	0,3	0	0	0,3	0,6	0,3	0,3	0,6	0,6	0,3

Figure 3 Value of variables

Function (A_1): GVH Hannovermobil (IV) application obtains the highest score as besides basic, three additional functions are incorporated in it: saving favourite locations and routes, reminder about service alterations, environmental footprint (CO_2 consumption). Mode (A_2): Mozio (VIII) and Wienmobil (X) applications obtain relatively high scores as mapping of charging points and parking facilities, long-distance bus service and, scooter service are also considered. Tariff structure (A_3): GVH Hannovermobil (IV), Smile (IX), Ubigo (XII), Whim (XIII) obtained higher scores as monthly mobility package is available. The result of aggregated index values is presented in Fig. 4.

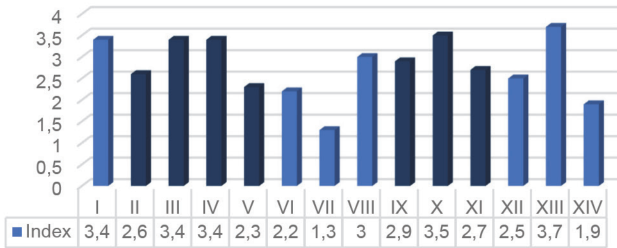


Figure 4 Aggregated value of index

The highest value of index is obtained by Whim (XIII), in which the MaaS operator is a private company. Whim is evaluated as the benchmarking in the integration studies [3], [4] as well. In addition, Whim (https://whimapp.com/) is available in Helsinki, West Midlands, Antwerp, Vienna, Tokyo, and Singapore as an application of international mobility service. Referring to the operational area, the MaaS services are operated in wider range by private companies (e.g. I. Moovel, VI. Moovit, VII. Urbi, XIII. Whim), in intra city or even in national level.

6 Conclusion

MaaS is developed under a mixture of information technology and innovative mobility services. The characteristic of transformation from vehicle ownership to service usership is expected to be met. Developing an assessment method to determine the integration index of MaaS services is main contribution of our work. Accordingly, the variables and integration phases are determined. Since MaaS operator has a key role in MaaS, the 'organization' aspect is highlighted. We have found that:

- involved functions and modes depend highly on the operational area,
- the monthly subscription of mobility package is a possible improvement direction, this function is only available in 4 applications out of 14,
- the third party MaaS operator is a public service provider in local or inter-city level, thus a private company in national and cross-border level.

We faced, as a lesson learnt, that developing an objective approach to assign weights to variables is rather complicated work. Our further research direction is to calibrate the method with the application of sensitivity analysis.

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References

[1] Errampalli, M., Patil, K., Prasad, C.: Evaluation of integration between public transportation modes by developing sustainability index for Indian cities, Case Studies on Transport Policy, 2018, doi: 10.1016/j.cstp.2018.09.005

[2] Franco, P., Johnston, R., McCormick, E.: Demand responsive transport: Generation of activity patterns from mobile phone network data to support the operation of new mobility services, Transportation Research Part A: Policy and Practice, 2019, doi: 10.1016/j.tra.2019.09.038

[3] Kamargianni, M., Li, W., Matyas, M., Schafer, A.: A critical review of new mobility services for urban transport, Transportation Research Procedia, 14 (2016), pp. 3294-3303. doi: 10.1016/j.trpro.2016.05.277

[4] Sochor, J., Arby, H., Karlsson, M., Sarasini, S., editors.: A topological approach to Mobility as a Service: A proposed tool for understanding requirements and effects, and for aiding the integration of societal goals, 1st International Conference on Mobility as a Service (ICOMaaS), Tampere, Finland, 2017, doi: 10.1016/j.trpro.2016.05.277

[5] Hrelja, R., Rye, T., Mullen, C.: Partnerships between operators and public transport authorities, Working practices in relational contracting and collaborative partnerships, Transportation Research Part A: Policy and Practice, 116 (2018), pp. 327-338. doi: 10.1016/j.tra.2018.06.032

[6] Mulley, C., Ho, C., Balbontin, C., Hensher, D., Stevens, L., Nelson, JD., et al.: Mobility as a service in community transport in Australia: Can it provide a sustainable future?, Transportation Research Part A: Policy and Practice, 2019, doi: 10.1016/j.tra.2019.04.001

[7] Hirschhorn, F., Paulsson, A., Sørensen, CH., Veeneman, W.: Public transport regimes and mobility as a service: Governance approaches in Amsterdam, Birmingham, and Helsinki, Transportation Research Part A: Policy and Practice, 130 (2019), pp.178-191. doi: 10.1016/j.tra.2019.09.016

[8] Pirannejad, A., Janssen, M., Rezaei, J.: Towards a balanced E-Participation Index: Integrating government and society perspectives, Government Information Quarterly, 101404 (2019), doi: 10.1016/j.giq.2019.101404

[9] Djekic, I., Smigic, N., Glavan, R., Miocinovic, J., Tomasevic, I.: Transportation sustainability index in dairy industry–Fuzzy logic approach, Journal of cleaner production, 180 (2018), pp. 107-115, doi: 10.1016/j.jclepro.2018.01.185

[10] Kumar, A., Anbanandam, R.: Development of social sustainability index for freight transportation system, Journal of cleaner production, 210 (2019), pp. 77-92, doi: 10.1016/j.jclepro.2018.10.353

[11] Mikusova, M.: Proposal of benchmarking methodology for the area of public passenger transport, Periodica Polytechnica Transportation Engineering, 47 (2019) 2, pp. 166-70, doi: 10.3311/PPtr.10271

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PREDICTION OF FUTURE PASSENGER INTENSITY ASSIGNING IN THE DIRECTIONS AND TIME SLOTS

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Abstract

This paper is focused on the prediction of passenger intensity on the Ostrava – Valasske Mezirici railway line in Moravian-Silesian Region, Czech Republic. The paper analyses available transport data about passenger behaviour. Data describes assigning passengers to appropriate directions and time slots in detail, emphasis is placed on daily and weekly variation of travel demand. Based on the provided travel behaviour researches, a model of assigning passenger intensity was designed. Travel surveys and available data sources from realized infrastructure studies were used. The prediction is harmonized with local specific conditions and proceeds from travel behaviour in previous years. The obtained assigning of passenger demand was exploited when designing a new operating concept for the suburban rail in the area. A model composition of the vehicle units and the frequency of connections in the respective sessions were suggested based on the obtained data.

Keywords: passenger transport, travel demand, transport supply, suburban railway transport, variation of transport demand

1 Introduction

The solved area is located in the Czech Republic, in the Moravian-Silesian Region. The task of predicting future passenger intensity was solved for the track No. 323 between cities Ostrava and Valasske Mezirici.

There is a planned modernization of the line and introduction of a new operating concept. A significant qualitative improvement of transport services is expected compared to the current situation. For the purpose of track modernization, a feasibility study was carried out, which included a framework model based on frame numbers and estimates for the future. The transport model does not provide sufficient input to prepare the operational concept, in particular the data for determining the number of routes per hour and the capacity of the trains.

As inputs for the design of the operational concept, it is necessary to know the demand for transport in individual hours, the distribution of demand into the relevant track sections and the sources and destinations of the trips. Based on these decisions, it is possible to assess the introduction of another transport segment, i.e. fast trains connecting important sessions. Finally, it is desirable to consider the variation in transport demand during the day, week and year. All calculations are essential for the design capacity and the required number of multiple units needed to ensure the operational concept.

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2 Methods

2.1 Current movement of passengers

2.1.1 Origin-destination matrix for stations

The starting point of the prediction of the future development of the number of passengers is the analysis of the current situation which will serve as an input to this prediction. For a comprehensive picture of the existing transport relations, authors drew on various publicly available sources, whose data were put into context as far as possible. It is important to take into consideration that the authors do not have a comprehensive transport model, which doesn‘t exist for this area, but only fragments. They do not have exact current data on the number of passengers on trains either, as obtaining this data is demanding.

Data from the National Population and Housing Census (2011) [1] were taken into account to determine daily commuting. All municipalities in the reasonably considered neighborhood of each station were taken into account. This allows the authors to create a origin-destination matrix. However, it is not possible to determine a modal split from the census [2]. The authors prepared a procedure for making a qualified estimate, which consists in assessing the attractiveness of the connection for the major transport modes for the purpose of determining the modal split. The attractiveness of the connection by individual car transport, the possibility of using the bus service (frequency of connection, travel time, average walking distance) and the possibility of using the railway transport (frequency of connection, travel time, average walking distance) were assessed. The result was a qualified estimate of modal split for regional rail transport for all pairs origin-destination municipality of the assessed municipalities [3].

The product of an absolute number of trips in the origin-destination matrix for each relation and the coefficient corresponding to the modal split of the regional rail transport in the given relation we get the potential in rail transport for the given relation. Subsequently, the potentials in rail transport from the origin-destination matrix were assigned to the network. A station usually includes data for more than one municipality because it provides services to several municipalities. On the other hand, there is no case on the track No. 323 where there is more than one station in one municipality. The result of this procedure is the origin-destination matrix for stations containing the potentials of relations between them.

2.1.2 Daily variation of passengers

The demand for transport is uneven at certain times of the day. The course of demand for transport within one day is expressed by a curve, which is defined as a daily demand variation. The actual course depends on many factors, the most important ones are usually the type of day (workday / weekend), the daily routine of the region and local customs and specifics [4].

The daily routine is often based on the type of municipality. Nowadays, small municipalities are not the centre of the inhabitant‘s life because of sub-urbanization. They are mostly used frequently for overnight stay. The morning trip to bigger towns and the afternoon return predominate. There are also medium-sized towns where the citizens of small municipalities commute, but at the same time the citizens of these cities commute to larger cities. Passengers from small municipalities approximately correspond to the number of passengers from these towns. The last category include the core cities of the agglomeration or centers of the metropolitan area, where dominate commuting from the surroundings in the morning and coming back in the afternoon [5].

2.1.3 Weekly variation and other influences

At present, there is a relatively significant share of trips on weekends and public holidays. The Beskydy Mountains, which is an important local touristic region, can be considered the main destination of the trips. Tourists at weekends with favorable weather use the track No. 323 mostly in the opposite direction than on working days. The number of passengers on Saturdays is often higher than on a working day, the section between Frydek-Mistek and Frydlant nad Ostravici is used tens of percent higher. Another influence on weekly variation is week-long commuting to schools in more distant destinations, which is the most apparent on Friday afternoons and Sunday afternoons, but with comparison to every day commuters it has a marginal affect on predicted demand (in this case) [6].

2.2 Future movement of passengers

2.2.1 Growth coefficient for stations

The authors utilized variations described in Section 2.1. for the determination of the number of passengers per day. Recent data due was not available at this time due to limited availability. The load values from “Beskydy“ feasibility study [7] were compared with the diploma thesis [8] and other censuses carried out in the following years and were subsequently standardized because a constant decrease in the number of passengers was identified over the years. Subsequently, the standardized number of passengers corresponds to the year 2010, from which the most valid dataset comes from.

Furthermore, the authors used relevant data for the period after the modernization of the track No. 323. These data come from the study [7] and describe passenger section load after the completion of the track modernization. The period after modernization corresponds to the year 2025. These data are available to the authors only in the form of loading of individual sections, therefore it is necessary to perform the conversion below.

On the basis of these data, an increase in the number of passengers at the relevant stations and adjacent sections was determined. The calculation was performed according to the following eqn (1).

$$k = \frac{z_{2025}}{z_{2010}} \tag{1}$$

where k is the coefficient of the passenger number growth, z_{2025} expresses the predicted passanger section load according to the transport model for 2025 after the modernization and z_{2010} expresses the standartized passenger section load according to 2010.

Using the eqn (2), the values for the estimated movements (boardings and gettings off) at all stations were received.

$$p_{2025} = p_{2010} \frac{\left(k_p + k_n\right)}{2} \tag{2}$$

where p_{2025} expresses the predicted passenger movements at station in 2025, p_{2010} expresses the passenger movements at station in 2010, k_p represents the passenger growth coefficient immediately preceding the assessed station and k_n represents the passenger growth coefficient immediately following the assessed station [9].

2.2.2 Number of passengers per hour and direction

Based on the passenger data at individual stations on the track No. 323 and data describing the section load, it is possible to determine the estimated demand for transport for the given direction and specific hour. For the allocation of passengers to the network, data on the daily variation of passengers at the station was used, divided into individual directions and their movements (boardings / gettings off) [10]. The result of this procedure is an indication of the number of passengers for each hour, their direction of travel and movement (boarding / getting off) at the station. After assigning the values of the anticipated demand for transport to the respective direction and hour, the obtained values were aggregated and tables with the expected section load in the respective direction and hour could be compiled. The validation of the section load was carried out in the vicinity of important nodes Frydek-Mistek and Frydlant nad Ostravici. The verification was carried out on data obtained during the field survey carried out in April 2019.

3 Results

By applying the procedures described in the Section 2, the authors gradually reached the expected output – the number of passengers load in each section for a given hour and direction. Partial outputs are presented below.

3.1 Current origin-destination matrix output

Table 1 shows the station potentials for the respective directions to the initial or terminal station of the track. Looking at Table 1 on the obtained slope coefficients, it is clear that boarding passengers predominate in the direction to Ostrava, the direction to Valasske Mezirici is less preferred. The endstations of the monitored section have potentials of 1 in the appropriate direction, since it is an entry into the solved area and other travel relations in the node are not considered. For the stations Frydek-Mistek and Frydlant nad Ostravici, data from the authors‘ survey was used as a validation of predicted data.

Table 1 Proportion to directions per station (section Ostrava – Frydlant n. O.)

Station	Total number of commuters [-]	proportion to direction Valasske Mezirici [-]	proportion to direction Ostrava [-]
Ostrava-Kuncice	-	1,00	0,00
Vratimov	351	0,28	0,72
Paskov	226	0,40	0,60
Liskovec u Frydku	82	0,62	0,38
Frydek-Mistek	887	0,33	0,67
Baska	266	0,15	0,85
Przno	108	0,21	0,79
Frydlant n. Ostr.	468	0,11	0,89

3.2 Comparison of daily variations

A daily variation of passenger movements at the stations Paskov and Frydek-Mistek was prepared to determine the actual course of the curve. The Paskov station represents a station near a small municipality, the Frydek-Mistek station represents a station in a medium-sized town. The available data on the 2011 and 2014 from Paskov Census and in 2019 from authors‘ survey were used ror Paskov Station. For Frydek-Mistek Station the data on the 2019 from authors‘ survey were used [6]. All surveys took place on an average workday in the middle of the week, with an emphasis on the passengers‘ boarding and getting out the trains and their assignment to directions.

The rules described in Section 2.1.2 in the above cases were applied. The suburbanization trends and the prevalence of morning commuting to larger cities are apparent from the Paskov station data and balanced boarding and gettings off confirm the rule for medium-sized towns in the case of the Frydek-Mistek station. When comparing the daily variations at the stations Paskov and Frydek-Mstek, it can be concluded that the variations are comparable.
The daily variation of passenger movements for the Paskov station was applied to stations connected with small and daily variations of passenger movements for stations Frydek-Mistek and Frydlant nad Ostravicí were applied for stations in larger settlements, where similar development of the number of passengers is expected and commuting from surrounding municipalities is noticeable.

3.3 Future movement of passengers

The method described in Section 2.2 was applied for the determination of future passenger movements. Based on the data of passengers movements at stations and data describing the load of track sections, it was possible to determine the estimated travel demand for the given direction and specific hour. For each station, passengers‘ boardings and gettings off have been computed. Furthermore, the direction in which the passenger is boarding or getting off has been taken into account. The existing division of passengers into directions according to Section 2.1.1 was used for routing.
The data collected from 2010 to 2019 were used for this purpose and daily variations of passenger movements were created. Subsequently, the daily variation of the respective station was applied to the absolute number of passenger movements, and values were obtained for specific hours of the average working day.

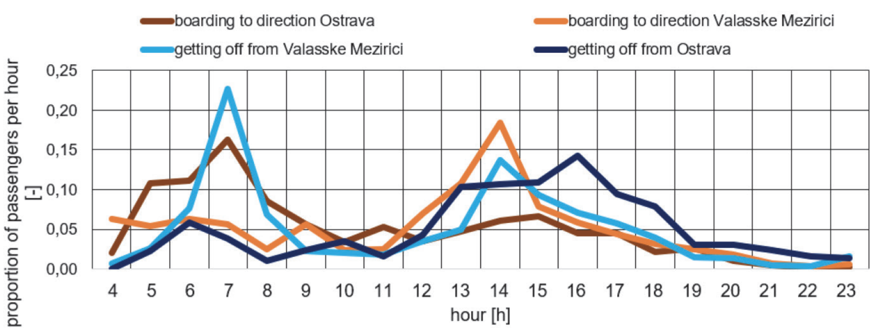

Figure 1 Daily variation at station Frydek-Mistek – proportion to directions

Tables with an estimated number of passengers in respective directions and sections per hour were created from the data analyzed so far on transport demand by their synthesis. The validation of the section load was carried out in the vicinity of transport nodes with the quarry of transport demand, it was performed on data obtained during the authors‘ survey in April 2019.
Table 2 below shows the expected section load in the direction to Ostrava per hour, values are rounded up to tens. The authors are aware of some inaccuracies stemming from the insufficient volume of data obtained and the difficulty of predicting travel behavior. However, given the design horizon and feasibility study submission, a more accurate analysis is possible only on the basis of a complete transport model with significantly larger data inputs. For the purposes of designing the operational concept, timetable and vehicle circulation, it is possible to be satisfied with the values obtained and their possible deviations.

Table 2 Expected passenger section load per hour in the direction to Ostrava

from	to \ hour	4	5	6	7	8	9	10	11	12
Valasske Mezirici	Hostasovice	20	20	20	20	10	20	10	10	30
Hostasovice	Morkov	20	20	20	20	10	20	10	10	30
Morkov	Verovice	30	30	30	30	20	20	10	10	30
Verovice	Frenstat p. Radh.	30	50	50	50	30	30	20	20	30
Frenstat p. Radh.	Kuncice p. Ondr.	30	50	130	110	20	30	50	50	50
Kuncice p. Ondr.	Celadna	40	80	160	150	40	40	50	50	60
Celadna	Frydlant n. Ostr.	50	120	210	200	60	60	60	60	70
Frydlant n. Ostr.	Przno	110	210	430	460	150	100	110	90	170
Przno	Baska	120	240	470	500	160	110	110	100	180
Baska	Frydek-Mistek	130	290	520	550	190	130	120	110	200
Frydek-Mistek	Liskovec u Frydku	80	490	660	820	410	250	200	230	180
Liskovec u Frydku	Paskov	80	500	670	850	410	250	190	230	170
Paskov	Vratimov	90	530	700	880	420	260	190	230	180
Vratimov	Ostrava-Kuncice	110	570	750	930	450	270	200	230	190

Table 2 shows the stations at which there is a significant refraction of demand for transport and a change in the frequency of the offered connections or vehicle capacity can be expected.

3.4 Application in operational concept designing

Table 2 shows the passenger load on each track section at the appropriate hour. This output makes it possible to determine a suitable operational concept for track No. 323. It is obvious that the utilization of the sections is variable, in the part of the track between Frenstat pod Radhostem and Valasske Mezirici the number of passengers is usually up to 50 passengers per hour and the differences between the peak and saddle hours are minimal. In this section it is about ensuring a minimal transport service, which corresponds to a one train connection hourly.

On the other hand, in the section between Frydek-Mistek and Ostrava, the estimated number of passengers is almost 1,000 passengers per hour. It is necessary to consider the capacity strengthening of the trains, as well as shortening the interval. It is also apparent from the outputs that there is a significant change in the number of passengers in Frydek-Mistek and Frydlant nad Ostravici [11]. These passengers are heading to Frydek-Mistek or Ostrava. This implies the need to introduce fast suburban trains for this large group of passengers [12]. Conversely, there is no need for such a number of trains during the saddle period. In relation to the number of passengers, it is also possible to determine the optimal train capacity, capacity coverage during the morning rush hour is crucial.

In addition, the outputs allow to determine the range of periods of the rush hour and transport saddle, the extent of periods with shorter intervals or additional train lines could be determined accordingly.

4 Conclusion

A prediction of future passenger intensity assigning in the directions and time slots was made for the design of the new operational concept. Passenger movements (boarding and getting off) at stations were determined from available surveys. These movements were assigned to the respective track direction using the origin-destination matrix. In accordance with the procedure described in Section 2, a daily variation was determined for each type of station.

Subsequently, the data from the current period were converted into future passenger load of track sections and stations. Demand growth coefficients for the period after the modernization were used. The decomposition was carried out in the respective directions and with respect to the daily variation of the passenger movements. Based on this decomposition, the data was aggregated and output describing the number of passengers in the directions and time slots was made.

It is possible to determine a suitable operational concept for track No. 323 on the basis of output. There are sections where a minimum range of transport service is sufficient - the hourly interval of trains. On the contrary, in the section between Frydek-Mistek and Ostrava, it is essential to carry out the capacity strengthening of trains, as well as shortening the interval. This also implies the need to introduce fast suburban trains for a large group of passengers commuting between these two cities. In relation to the number of passengers, it is also possible to determine the optimal train capacity. Operational concepts for rush hour periods and transport saddle periods can be designed.

In conclusion, the authors would like to state that there are no sophisticated considerations in Czech conditions. A design of the operational concept for the modernization or reconstruction of railway tracks is usually accomplished without a detailed analysis. As a result of this procedure, the track is often oversized for the intended operational concept or, conversely, the infrastructure is inadequate and it is not possible to introduce a sufficient number of trains.

References

[1] Sčítání lidu, domů a bytů 2011 (ČSÚ), https://www.czso.cz/csu/sldb, 10.04.2020

[2] Kraft, S., Marada, M.: Delimitation of functional transport regions: Understanding the transport flows patterns at the micro-regional level, Geografiska Ann. Ser. B Human Geograph, 99 (2017) 1, pp. 79-93, doi: 10.1080/04353684.2017.1291741

[3] Janoš, V., Kříž, M.: Pragmatic approach in regional rail transport planning, Zeszyty Naukowe Politechniki Śląskiej, 100 (2014), pp. 35-43, doi: 10.20858/sjsutst.2018.100.4

[4] Drábek, M., Pospíšil, J.: Fluctuations in Passenger Railway Service Period, Young Transportation Engineers Conference 2018, Praha, Czech Republic, 2018, pp. 1-8

[5] Wardman, M., Lythgoe, W., Whelan, G.: Rail Passenger Demand Forecasting: Cross-Sectional Models Revisited, Research in Transportation Economics, 20 (2007), pp. 119-152, doi:10.1016/S0739-8859(07)20005-8

[6] Fridrisek, P.: New Operating Concept on the Track Ostrava - Valašské Meziříčí, master thesis, Praha, Czech Republic, 2019.

[7] Composite authors: Studie proveditelnosti Beskydy, SUDOP Brno, AF CITYPLAN, Brno, 2015.

[8] Solansky, S.: Assessment of the extent of infrastructure and its capacity on track Valašské Meziříčí - Ostrava for current and prospective operation, master thesis, Univerzita Pardubice, 2011.

[9] Pekel, E., Kara, S.S.: Passenger flow prediction based on newly adopted algorithms, Applied Artificial Intelligence, 31 (2017) 1, pp. 64-79, doi: 10.1080/08839514.2017.1296682

[10] Wang, D., Yao, E., Yang, Y., Zhang, Y.: Modeling Passenger Flow Distribution Based on Disaggregate Model for Urban Rail Transit, ed. Sun F., Hu D., Liu H.: Foundations and Practical Applications of Cognitive Systems and Information Processing, Advances in Intelligent Systems and Computing, 215 (2014)

[11] Zhu, Y.T., Wang, F.Z., Shan, X.H., Lv, X.Y.: A Model for Seat Allocation of Railway Passenger Transport Based on OD's Priority, Advanced Materials Research 2014, 989–994 (2014), doi:10.4028/www.scientific.net/amr.989-994.2344.

[12] Janoš, V., Vávra, R.: Comparison of Different Conceptions of Suburban Railway Transport, 2019 Smart City Symposium Prague, IEEE Press, New York, 2019, doi: 10.1109/SCSP.2019.8805699

BICYCLE PARKING FOR OFFICE BUILDINGS IN FRANKFURT MAIN/ GERMANY

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Abstract

Until the end of 20th century new big office buildings in Germany – even in the central business district CBD – didn't provide any or much too less facilities for bicycle parking. Since then the already existing state and municipal regulations have been enforced and new ones exist which define the mandatory number of bicycle parking lots for new office buildings. But now it can be seen that too many lots are required, the real demand is in many cases – and will be in the foreseeable future – much lower. With data from EU and German sources the situation in the Frankfurt am Main CBD is described: political goals for sustainable traffic by enforcing bicycle usage; municipal regulations for the number of bicycle parking facilities; comparison between (politically defined) requirements and the expected real demand. Today's and future modal-split data for bike usage at office buildings from empirical traffic surveys and assumptions in Frankfurt are presented. Examples of bicycle parking facilities for already existing and planned office buildings in Frankfurt are provided, showing as result a compromise between city's regulation and the expected demand for bicycle lots.

Keywords: bicycle traffic, parking facilities, urban traffic, modal choice, trip generation

1 Introduction

The Central Business District of Frankfurt am Main is situated just in the inner circle of the ancient merchant town (Fig. 1), dating back till the year 794. Because of many banks and skyscrapers there; Frankfurt is sometimes also called "Bankfurt" or "Mainhattan". Frankfurt municipality has approximately 745.000 inhabitants (2018, it is expected to be 810.000 in 2030) and 700.000 employees (2018), the wider metropolitan region (radius 30 km) has about 3 million inhabitants. Due to the very high number of workplaces in Frankfurt and their concentration in the inner city, there is a very high number of commuters entering the city every day (375.000). Only about 1/3 of employees are living in Frankfurt, which is muss less than in other towns in the Rhein-Main metropolitan area (e.g. Wiesbaden and Darmstadt over 50 %). As the goals of reducing CO_2 force the municipalities everywhere to reduce car-traffic, it is necessary to get as much commuters as possible to use bikes and feet on their way to and from work. The official goal for "2030+" therefore is to be a "City of short distances" with higher percentage of bike and pedestrian traffic [1]. To enforce bike use it is therefore necessary to provide a sufficient – but not overcalculated - number and a good quality of bicycle parking lots for commuters at or near their workplace. As this article concentrates on office bicycle parking in the CBD, the other necessary provisions to improve bike traffic are not discussed (e.g. bike lanes, signalization ...).

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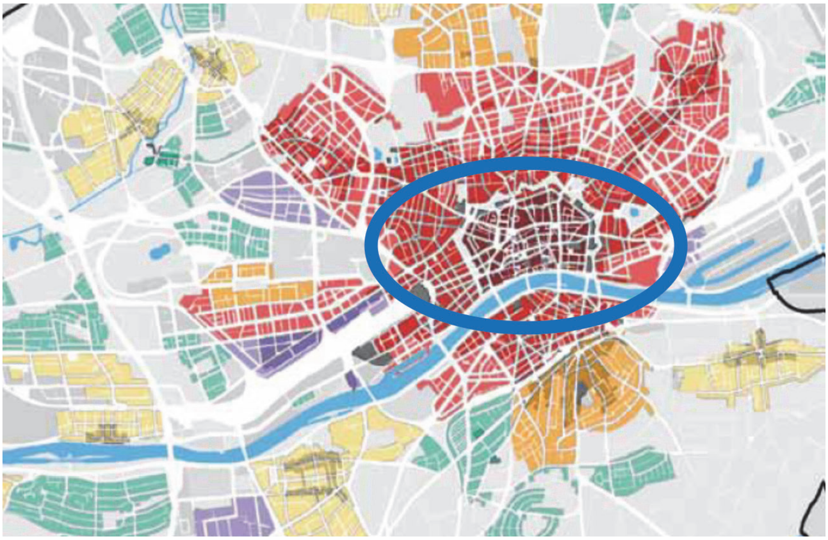

Figure 1 Inner City of Frankfurt Main and CBD [2]

2 Regulations for the number of bicyle parking lots at office buildings

The following does not mention lots of available recommendations made by NGOs or bicycle clubs but – with the exception source [4] in chapter 2.1 - only "official" recommendations, guidelines or legal requirements.

2.1 Regulations on EU level

There could be found only one EU-paper which mentions the recommended number for bicycle parking at office-buildings [3], based on Swiss recommendations: 1 bicycle lot per 5 employees.
A compendium of regulations in certain European countries as drawn in Table 1 [4] shows a wide range for the required number of bicycle parking stands at office buildings and differs greatly regarding the referred basic, e.g. number of employees or office space (assumed: m² GFA).

Table 1 Some regulations in the EU [4]

Land/ City	1 bike lot/ … m² GFA	Other regulation 1 bike per …
Bulgaria, Hungary, Croatia	100	
London	90 (long stay, inner city)	
Malmö / S	55	
Copenhagen	25	
France		1,5 % GFA or 15 % of empl.
Styria/ A		20 employees

2.2 Regulations in the Federal Republic of Germany

On federal level there are no mandatory regulations but recommendations from the German FGSV [5]: 0,3 bicycle lots per 1 employee (= 1 lot per 3,3 employees).
On state level (16 states) for most of the states there are no state-wide regulations regarding the required or mandatory number of bicycle lots at office buildings [6]. Instead the state laws empower the municipalities to define such regulations.

2.3 Regulations in German cities

There is a wide range, mostly based on the useful floor space [m²] [7]: 1 bicycle lot per 40 m² … 100 m² useful floor space.
The city of Frankfurt Main followed those numbers till 2016, which in most cases led to 1 bicycle lot per 80 m² useful floor space. The new regulations since 2016/2020 [8] defines the mandatory number of bicycle parking lots at office buildings based on the Gross Floor Area (GFA) [m²]: 1 bicycle lot per 125 m² GFA
The GFA is usually much easier to define in an early phase of planning as the useful floor area. Most of the city regulations allow to reduce or increase the resulting number if it can be shown by a traffic expertise that the really required number (demand) for such lots differs from the regulation. With the typical space of approximately 30 m² GFA per office employee in the CBD, the rule of 1 bicycle lot per 125 m² GFA would mean that about ¼ of all employees would need a bicycle parking lot, i.e. the bicycle mode choice would be 25 %. Hence today's and future bicycle mode choice have to be realistically assessed.

3 Bicycle parking at office buildings in Frankfurt Main

3.1 Bicycle mode choice of employees

To calculate the demand for bicycle parking at an office building in the inner city / CBD of Frankfurt the mode choice of the employees has to be assessed: What percentage of employees on a workday will use their own bike as mode of transport with trip end / begin at the office? Rent-a-bike trips are not included, because rental bikes usually do not have parking lots on the building's property.
While there is usually data available regarding the mode choice of a city's resident population, there is only a very limited number of data available which tells about the mode choice of employees, a great number of whom are commuters and are therefore not included in the data for the resident population. In very bike-friendly cities, like in the Netherlands, usually around 25 % of all employee-trips are done with the bicycle as the main mode of transport with an additional percentage of commuters combining public transport and bicycle [9].
The bicycle mode choice for Frankfurt resident population on a workday has risen from 6 % in 1998 to 13 % of all trips in 2013 while the private car mode choice decreased from 40 % to 35 % 2013. Of trips up to 1 km 12 % were done by bikes; trips 1 to 3 km were 27 % bikes; 3 to 5 km were 12 % bikes; 5 to 10 km were 9 % bikes, over 10 km only 1 % were bike trips (all figures: last available data 2013 [11]). As mentioned in Section 1 only about 1/3 of employees are living in Frankfurt. The main residential areas are between 3 and 5 km distant from the CBD therefore one can assess a current bicycle mode choice of 12 % for trips to and from the city center.
As there is no "official" published data available for the mode choice of employees in the city of Frankfurt, especially employees in office buildings, some data has been collected during the last 10 years in the framework of student's diploma thesis for the office skyscrapers shown in Fig. 2. The results regarding the percentage of bicycle as the main mode of transport for employees are shown in Table 2.

Table 2 Empirical data for employees in Frankfurt office buildings using bicycle as mode of transport to come/ go to work

Building	Number in Fig. 2	Bicycle as mode of choice [% of all trips from/ to work]	Median trip distance [km] or origin	Year of survey	Source [--]
Kastor	1	7 %	--	2009	12
Skyper	2	7 %	--	2009	12
Taunusturm	3	6 %	88 % loc	2016	13
Opernturm	4	6 %	--	2016	14
Junghofstrasse	5	6 %	3,0	2016	14

Figure 2 Skyscrapers No. 1 ... 5 referred to in Table 2 and construction site of skyscaper „Omniturm“ No. 6 (picture source [10])

3.2 Demand of bicycle parking at skyscraper “Taunusturm”

The office skyscraper “Taunusturm” is shown in Fig. 2 as No. 3. It has been finished in 2014. The Gross Floor Area GFA for the offices is appr. 81.000 m².
At the time of the construction permit (2010) the number of mandatory parking lots for this office building had to be calculated with the ratio of 1 bicycle lot per 80 m² useful floor space (see above Section 2.3). With the new regulations since 2016 the ratio is 1 bicycle lot per 125 m² GFA: 81.000 m² GFA/ 125 m² per lot =~ 650 bicycle lots required, which is occasionally the same result as with the former regulation due to the ratio GFA/ useful floor sp. To get a permit to build much less bicycle lots than the figure following the city’s regulation, a traffic expertise had to be done. In short the calculation is:

- Estimated number of employees · daily attendance rate · trip per day · percentage of bike usage / turn around rate of one bike lot = demand number of bicycle lots

In this example it leads to the following result:

- Estimated number of employees: 3,5 empl./ 100 m² GFA
- 3,5 empl./100 m² * 81.000 m² /100 =~ 2.800 employees
- Attendance rate: 0,8 at a workday; trips: 1,1 daily trips per direction

Mode choice for bicycle: today 6 %, estimated in future 10 %, considering that only 1/3 of employees come from Frankfurt, a lot of them using public transport, rent a bike or feet.
Turn around rate: 1,1 daily per lot 2.800 · 0,8 · 1,1 · 0,1 / 1,1 =~ 225 bicycle lots as future demand.
As a compromise between the figure resulting from the city's regulation and the expected demand some 300 bicycle lots for the office employees have been built in the underground levels of the building (Fig. 3), still leaving space for future increased demand. A traffic survey [7] done on a workday in June 2016 has counted 92 occupied office bicycle stands (occupancy rate ~23 % of the ~300 lots). The access to the garage level is made easy for bicycles by providing a 1,20 m wide walkway along the garage ramp, thus users who do not want to use the ramp can push the bike up along.

Figure 3 Bicycle parking in underground Garage "Taunusturm" [7]

3.3 Demand of bicycle parking skyscraper "Omniturm"

The multi-purpose skyscraper "Omniturm" is shown in Fig. 2 as No. 6. It has been finished in 2019. The Gross Floor Area GFA for the offices is appr. 51.000 m².
At the time of the construction permit (2016) the number of mandatory parking lots for this office building had to be calculated with the ratio of 1 bicycle lot per 125 m² GFA, following the new regulation (see above Section 2.3).
51.000 m² GFA/ 125 m² per lot =~ 410 bicycle lots required.
To get a permit to build much less bicycle lots than the figure following the city's regulation, again a traffic expertise had to be done. Following the same calculation as with "Taunusturm", but with some other assumptions, the result was:
- Estimated number of employees: 3,0 empl./ 100 m² GFA
- 3,0 empl./100 m² * 51.000 m² /100 =~ 1.530 employees

Attendance rate: 0,8 at a workday; trips: 1,1 daily trips per direction
Mode choice for bicycle: today 6 %, estimated in future 10 %, same as Taunusturm
Turn around rate: 1,1 daily per lot
- 1.530 · 0,8 · 1,1 · 0,1 / 1,1 =~ 125 bicycle lots as future demand.

As a compromise between the figure resulting from the city's regulation and the expected demand and to have additional space for increasing future demand, some 180 bicycle lots are provided on the underground levels of the building. The access to and from the garage levels is made easy for bicycles by a direct and wide walkway from street level to a big elevator, leading to the garage levels (Fig. 4).

Figure 4 Ground floor plan with bicycle access to elevator (picture [10])

4 Conclusion

The number of employees working in an office building in the CBD using a bike and thus demanding a parking lot can vary widely depending on density (employees per 100 m² GFA), kind of office work, mode choice ... The overall regulations (e.g. in Frankfurt 1 bicycle lot per 125 m² GFA) can only give a rough estimation and therefore have to be adapted to the local situation.

References

[1] Stadtplanungsamt Frankfurt, FRANKFURT 2030+, p. 53, 2019. www.frankfurt.de

[2] Stadtplanungsamt Frankfurt, PLAN.WERK.17., p. 33, 2018. www.frankfurt.de

[3] Bicycle parking made easy. BICY project (2011). Publisher: BICY – Cities and Regions for cycling, http://www.bicy.it/docs/64/20110706_BICY_bicycle_parking_EN_final.pdf

[4] Küster, F., Peters, M.: Making buildings fit for sustainable mobility, Comparing regulations for off-street bicycle and car parking in Europe, European Cyclist's Federation, Brussels, 2018.

[5] Empfehlungen für Anlagen des ruhenden Verkehrs EAR 05, table B-2. Köln 2005/2012

[6] https://www.ziegler-metall.de/media/pdf/29/31/d5/Stellplatzverordnungen-fuer-Fahrradabstell-plaetze.pdf

[7] Tos, I.: The demand of bicycle parking at office buildings in a metropolitan city center – example Taunusturm Frankfurt am Main, Master thesis at University of Applied Sciences Wiesbaden, Faculty of Architecture and Civil Engineering and University of Zagreb, Faculty of Transport and Traffic Sciences, Wiesbaden/ Zagreb, 2016.

[8] Stellplatzsatzung der Stadt Frankfurt am Main 2016 and 2020. https://www.bauaufsicht-frankfurt.de/fileadmin/Downloads__alle/Rechtsgrundlagen_und_Satzungen/2017_02_01_Stellplatzsatzung_und_Stellplatzzahlen.pdf

[9] Heinen, E., Maat, K., van Wee, B.: The effect of work-related factors on the bicycle commute mode choice in the Netherlands. Published online 11 April 2012, Springerlink.com, doi: 10.1007/s11116-012-9399-4

[10] Peric, Z.: The demand of bicycle parking at office buildings in a metropolitan city center – examples Opernturm and Junghofstrasse, Fankfurt am Main, Master thesis, University of Applied Sciences Wiesbaden, Faculty of Architecture and Civil Engineering and University of Zagreb, Faculty of Transport and Traffic Sciences, Wiesbaden/ Zagreb, 2016.

[11] https://www.n-tv.de/regionales/hessen/Mehr-Menschen-pendeln-nach-Frankfurt-article21587974.html

INCREASING CYCLIST MOBILITY BY IMPROVING CYCLING INFRASTRUCTURE: CASE STUDY KOPRIVNICA

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Abstract

The use of a bicycle as a form of transport is an essential factor within a sustainable transport system. The increased number of cyclists is changing their need for better and better infrastructure. Koprivnica is traditional cycling city, with one of the longest cycling infrastructure in the Republic of Croatia. However, parts are disconnected and partly inconsistent with the Bicycle Infrastructure Regulations (OG 26/2016). This results with reduced mobility and safety for all road users, especially pedestrians and cyclists who often share a common surface. The paper presents a method of mapping bicycle infrastructure in the city. As a reference point for comparing the state of cycling infrastructure, data were taken from the 2015 Sustainable Urban Mobility Plan of Koprivnica. In addition to personal bicycles in the city, public bicycles are also proposed to optimize this system. After the analysis, suggestions were made for improvements and connecting parts of existing network, into a united network that would meet the highest standards. Particular attention should be paid to intermodality, ie connection with railway and bus stations, and planned parking areas around the city. This model can be applied in all cities.

Keywords: urban mobility, bicycle infrastructure, COVID 19

1 Introduction

Due to its relatively cheap construction and availability, bicycles are nowadays the most widespread means of transportation, so it is estimated that there are over one billion bicycles in the world today. The use of bicycle as a means of transport is becoming more and more common in cities of developed countries around the world. In the Republic of Croatia, bicycle traffic and use of bicycles are not keeping pace with European trends, primarily due to inadequate cycling infrastructure. Developed European countries such as Sweden, Denmark, the Netherlands or Germany, which in the past have also faced similar problems as the Republic of Croatia (increasing popularity of cars, increasing congestion and air pollution) have recognized the problem much earlier and taken appropriate measures, including systematic encouragement of cycling traffic as early as the 1970s.

The development of bicycle traffic is achieved through tried and tested strategies such as: introduction of a public bicycle system, construction of quality and safe bicycle lanes, introduction of safe bicycle parking spaces, information and education of cyclists and other road users [1]. Koprivnica, as a “city of bicycles”, has a long tradition in the development of bicycle traffic, and as such is a good example for the analysis of cycling infrastructure.

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By increasing the number of passenger cars in cities, and using them to go to work and carry out daily tasks, congestion on city roads is becoming an increasing problem in urban traffic. Recent statistics show that in the most congested cities, cars are moving at an average speed of 7.5 kmh (New York [2]) or 8.2 kmh (London [3]), which is dangerously approaching pedestrian speed. On the other hand, speed in Copenhagen's busiest bike city is twice as fast. [4]
Traffic jams and lack of parking space can make driving a downtown car very impractical. A bicycle is a good alternative for moving and avoiding problems faced by passenger car users. Cycling can greatly contribute to a more efficient, sustainable and healthy transport system. Good cycling infrastructure and daily bike use are closely linked. The design of the cycling infrastructure should be adapted to improve traffic safety and quality. The infrastructure should allow cyclists to do direct, comfortable cycling in an attractive and safe traffic environment. Only then is it possible to compete with the car as a means of transport.

2 Cycling Infrastructure

Towards the end of the 19th century, cycling became a common mode of transportation in cities, especially on shorter distances. Already at that time, the problem of road use was shared by cyclists, horse-drawn carriages and pedestrians. In recent decades, developed European countries have paid particular attention to the development of sustainable urban mobility and to planning cycling to reduce traffic congestion, increase safety and make cities more liveable.
Promoting daily cycling is an ongoing process that needs more than just well thought out investments in cycling infrastructure. Each city has a different approach to cycling - some implement a stand-alone policy, while others integrate cycling policy into other planning documents, eg general development plans, transport and transport policies, etc.
The strong cycling culture of a city requires well-developed infrastructure and extensive facilities that support the large amount of everyday cyclists in an urban environment. Modern trends in mobility support the idea of living without noise and in the context of sustainable development, which implies the revival of walking, cycling and public transport. Cities should maintain and improve their cycling infrastructure not only to retain cyclists but also to attract new ones [5].
According to Sindik [6], in some cities in Europe such as Copenhagen, Amsterdam, Bremen and Antwerp the proportion of bicycle traffic ranges from 20 % - 30 %. An interesting estimate is that more than 30 % of car trips in Europe are shorter than 3 kilometers and 50 % shorter than 5 kilometers. These distances can be covered by children and the elderly by comfortable biking. When considering the length of trails in terms of population as an indicator of the size of cycling infrastructure, Denmark is the leading city in Copenhagen (454 km / 770,000 inhabitants) and the Dutch cities Amsterdam (400 km / 1,300,000) and Utrecht [7].

3 Case study Koprivnica

According to the Sustainable Urban Mobility Plan of the City of Koprivnica [8], there are around 70 km of cycling trails and more than 15 km of cyclotouristic routes in the city, leading Koprivnica even ahead of European cities in terms of length of tracks in terms of population. However, motor vehicle traffic in the City of Koprivnica is still far ahead in the overall modal split of travel. Considering that Koprivnica is an industrial hub affecting a large number of daily migration trips from the surrounding settlements and municipalities, one of the main reasons for the high intensity of motor vehicle traffic in the very center of Koprivnica is the lack of adequate transport alternatives to private vehicles to reach the town of Koprivnica from the surrounding settlements. and the municipality.

3.1 Methodology of data collection

Prior to the process of mapping bicycle infrastructure, it was necessary to define which roads would be covered by the mapping. The analysis covers only a narrower area of the city, without suburban settlements. The mapping includes only roads that fall into the category of cycling paths, bicycle lanes and cycling and walking trails and are marked with vertical and horizontal signage. The map thus obtained does not include interruptions in traffic routes due to unsettled bicycle crossings. Improperly marked crossings are therefore not considered as interruptions, so any street with a broken track or lane is shown on the map as continuous. Unfortunately, there are more than 80 % of them in Koprivnica [9].
If two-way lanes are on either side of the road, then their length is counted on each side individually, but if it is a two-way track on one side of the road only, then the length is counted only once and not for each direction separately.

3.2 Field mapping

Road mapping was performed between July 10 and July 19, 2019. For mapping, an Android-based mobile phone device was used. The software used for the mapping was OsmAnd, while map creation and analysis of the cycling trail data were made using the OpenStreetMap and QGIS computer applications. The reason for choosing these road mapping applications is that they are free, easy to use and offer complete freedom for users to create and analyze roads.
Entire cycling infrastructure was passed and recorded by bicycle. Each track is stored as a gpx file. All such files were later imported into the OpenStreetMap online map. After the bicycle roads were imported and loaded, any defects and errors were manually corrected. After that, each of the paths has been assigned attributes to further define:
• the type of road ('Generic path' is entered here to separate it from other roads in OSM-u
• street name
• type (cycling or cycling-walking)
• direction (one-way or two-way, ie track on one side of the road or on both sides).

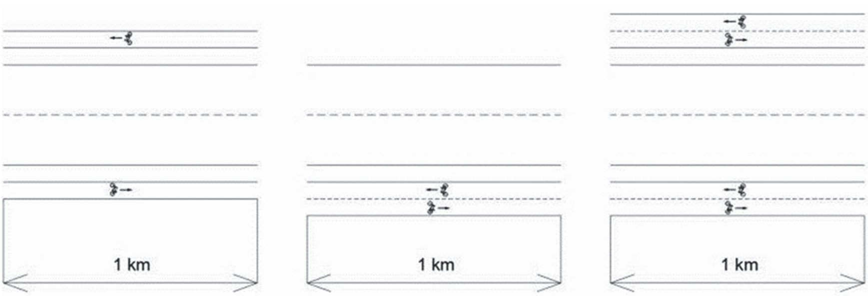

Figure 1 Methodology for calculating the length of bicycle infrastructure (created by the author)

The attributes mentioned are important because of the further processing and analysis of the data that is made in the QGIS application. QGIS allows users to create maps with multiple layers that use different map projections. The maps thus created consist of raster or vector layers. Vector data can be stored as points, lines, or polygons, and various types of raster images are supported.
After importing the vector data, it was first necessary to filter out any data that was not needed for analysis. OSM distinguishes three main types of spatial data and organizes them into these categories: points, lines, and polygons. Since the bike lanes and paths are line ob-

jects, the export of line objects is selected. Furthermore, if all the line objects (roads, paths, lanes, etc.) are to be singled out for cycling lanes and paths, it is necessary to define fields for which certain attributes will take on value [9].

3.3 Data analysis

The paper defines the questions to be answered after the map has been created:
- Where are the bike lanes and paths located in Koprivnica?
- Can the principles for planning and designing cycling infrastructure as set out in the Cycling Infrastructure Regulations (safety, economy, integrity, directness and attractiveness) be applied to the existing network?
- What is the total length of bike lanes and paths in Koprivnica? [9]

Comparing with the primary and secondary networks presented in Figure 1, it can be concluded that the cycling infrastructure is being built as planned. Except for the part in the western part of the city where most of the infrastructure has not yet been built.

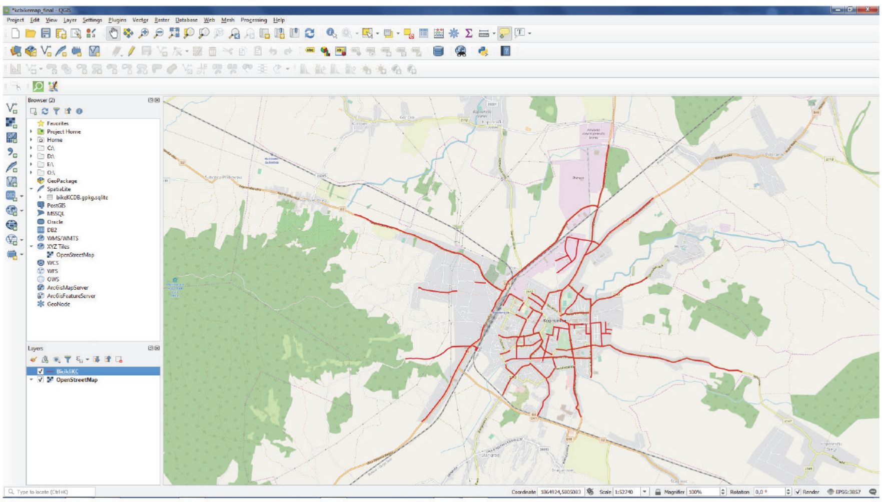

Figure 2 The layout of the resulting map after merging the vector file and the geospatial file section from OSM (created by the author)

According to the results, unmarked and undeveloped crossings of bicycle infrastructure over pavements, bridges or rail crossings in the city represent the biggest problem in the safety of bicycle traffic (Figure 4). As a consequence, the cycling network is disconnected, which is why cyclists are often forced to violate regulations and continue to drive on the roadway or sidewalk, endangering themselves or pedestrians.
This situation is largely due to the lack of regulations for the design of cycling infrastructure at national level, which was adopted in 2016 [10], so it is difficult to expect that existing pedestrian-cycling paths will be harmonized with the basic needs of sustainable forms of traffic, which is ultimately the cause of reduced safety. pedestrian and bicycle traffic.
A total of 61 streets were processed through field mapping. The obtained statistics are classified in two ways: in the first column of the table, the length of the road corresponds to the length if it counts only in one direction, and in the other if it counts in both directions, ie those bicycle roads which are constructed as two lanes on either side of the road. The total length of bicycle lanes in Koprivnica is 53.5 km, counting the roads with two bicycle lanes on either side of the road.

According to the available data on the length of bicycle roads, Koprivnica should in total have about 70 km of bicycle infrastructure [8]. The assumption is that this data also includes suburban settlements that were not included in the analysis and as well as the possibility that this length is increased by the value of the paths marked on one side in both directions.

Figure 3 Interruption of the cycle path on Starogradska and Miklinovec Street (created by the author)

3.4 Survey research

In addition to field mapping, a survey was conducted among the city residents about the state of cycling infrastructure and cycling habits when participating in traffic. According to the survey, one third of users (33 %) use bike daily or almost daily, and only 12 % said they did not use it at all. 42 % of those polled go to work, college or school with a bicycle, while as many as 49 % of respondents said that they use the bike most for recreational purposes or for going to sports. Respondents were also asked what would contribute to greater use of the bicycle as a means of transport. Most of them (more than 70 %) are not satisfied with the safety on bikes and want more safety at intersections and separate bike paths. [11]

4 What to do?

There are many parameters that affect the development of cycling in a city. Certainly good infrastructure and safety are among the most important. However, in recent days, public bicycle system as well as events organized for the benefit of cycling, must be considered.
In terms of population, Koprivnica has an above-average infrastructure, especially within the Republic of Croatia. One part of the infrastructure is not in compliance with the Rule book [10] and should be affected as soon as possible. This would allow, where possible, the separation of cycling from other infrastructure, thereby affecting the safety of cyclists and thus other road users.
In addition, most of the crossings of bicycle paths across the intersection are not properly marked, which means that the cyclist should get off the bike and push it over the pedestrian crossing. There are more than 80 % of crossroads like these in the city [9] , and it does not take much to improve this situation.
Regarding the public bicycle system, there is a completely free system in Koprivnica consisting of 60 bicycles, 7 docking stations with IT surveillance of the system and publicly available GIS tickets for tourists. The number of borrowings varies from 15,000-25,000 per year with

over 1,000 registered users. It can be concluded that such a system is sufficient for the current and future development of cycling in Koprivnica. [11]
Furthermore, this research needs to be extended to the surrounding settlements, since there are many people who communicate with the city center on a daily basis, and if they had secure and sufficient infrastructure, they would probably use bicycles more for daily communication.

5 Bicycle infrastructure in COVID 19 crisis

During the pandemic, the possibilities and needs for mobility, primarily the use of passenger cars, were partially reduced, thus freeing up more space for other modes of individual mobility - walking and cycling. To provide city dwellers with the recommended distance, many cities began “taking” from cars and “giving” to pedestrians and cyclists.
This was done by temporarily rearranging parts of the pavement into bicycle paths (so called “pop-up bike lanes”), thus partially relieving the common areas of pedestrians and cyclists and thus increasing the space for pedestrians. This satisfies the condition of the prescribed distance, thus relieving the already overcrowded public transport, and increasing the safety of all traffic participants. This has led to increased use of bicycles in cities that have implemented this measure of sustainable mobility.

6 Conclusion

Within modern sustainable transport systems, bicycles play an increasing role as a means of transportation. Koprivnica, as a “smart” city that strives for sustainability at all levels, realized that only investments in sustainable transport projects would reach other European and world cities that have already made progress in this regard. Good cycling infrastructure is the first and foremost prerequisite for this. The results obtained show good integrity and directness, but poor safety and attractiveness of the infrastructure. The major problems are caused by the inconsistency of old infrastructure with the new Bicycle Infrastructure Regulations, which was adopted in March 2016. Many of the temporary measures introduced in pandemic will remain after returning to “normal” and thus contribute to sustainable mobility. Now is the opportunity for our cities to expand their cycling infrastructure and help return to a healthier and more sustainable lifestyle.

References

[1] Anđelković, Z., Jokanović, I.: Biciklistički saobraćaj u urbanim sredinama, Građevinski fakultet Univerziteta u Beogadu, Beograd, 2015.

[2] Agrawal, N.: The average speed of traffic in Midtown Manhattan is 4.7 mph. New York thinks it’s found a solution, Los Angeles Times, 2018.

[3] Middleton, N.: Average driving speeds plummet in UK’s major cities, Fleetworld, 2017.

[4] Annual Report 2018, Cycling Embassy of Denmark, Copenhagen, 2018.

[5] Handbook on Cycling inclusive planning and promotion, Institute for Social-Ecological Research, Frankfurt am Main, 2012.

[6] Sindik, J.: Komunikacijski čimbenici biciklističkog prometa u Zagrebu i okolici, Media, culture and public relations, vol. Vol 6 (2015), no. No 1, pp. 17-31

[7] Velosock, https://velosock.com/blogs/blog/whats-the-worlds-most-bike-friendly-city

[8] Plan održive urbane mobilnosti Grada Koprivnice, Fakultet prometnih znanosti, Zagreb, 2015.

[9] Gadanec, R.: Analiza biciklističke infrastrukture u Gradu Koprivnici, Koprivnica, Sveučilište Sjever, 2019.

[10] Pravilnik o biciklističkoj infrastrukturi, NN 28/2016, Ministarstvo pomorstva, prometa i infrastrukture RH, Zagreb, 2016.

[11] Brlek, P., Krpan, Lj., Petar, S.: Sustavi javnih bicikala u hrvatskim gradovima, Zbornik radova 43. međunarodni stručni seminar o prometnoj infrastrukturi i sigurnosti u prometu, CESTE 2019, Rovinj, 2019.

ALTERATION IN MODAL SHARE DUE TO AUTONOMOUS VEHICLE-BASED MOBILITY SERVICES

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Abstract

Alteration in road-based mobility services in cities is expected due to introduction of autonomous vehicles (AVs). On-demand and shared services based on small capacity AVs emerge, which influence the modal share. The alteration has been estimated by simulation of scenarios; the travellers' willingness-to-shift to an AV-based mobility service has been considered as a random variable in studies. In our developed modal share estimation method, the travellers' current mobility habits and willingness-to-shift are considered. To determine the value of variables, a questionnaire survey was elaborated. The method was applied to calculate the modal shift in Budapest, Hungary. According to the results, willingness-to-shift is the highest among car users and the lowest among bikers. Based on the stated preferences, individual car use can be reduced by shared, on-demand, AV-based mobility services. Our method is applicable to determine the impacts of AVs.

Keywords: autonomous vehicle, autonomous vehicle-based mobility service, modal share, modal shift, willingness-to-shift

1 Introduction

The impacts generated by the introduction of autonomous vehicles (AVs) are the increased performance of the transport system, safer transport, individual travel options for people without driving license, increased energy efficiency, improved land use [1], [2]. The impacts can be estimated by qualitative or quantitative methods. In the case of quantitative methods, traffic simulations are used [3], [4], or the travellers' expectations are analysed [5], [6]. Some of the impacts are caused by vehicle characteristics (e.g. shortening head-up distance); moreover, some of the impacts can be influenced by mobility management (e.g. modal-shift).

The modal share (or modal split), i.e. the percentages of the use of transport modes, is expected to be changed. Modal share alteration is analysed by the introduction of scenarios in current studies [3], [7]; the rate of individual car use is compared. However, scenarios are based on estimations; traveller expectations and willingness-to-shift are considered indirectly without measuring them.

The research question is how the alteration in modal share could be estimated based on the travellers' mobility habits and willingness-to-shift. Questionnaire survey-based data collection and analysis methods were elaborated. The AV-based mobility service types were determined according to our previous researches and the literature. The elaborated estimation method could be used for traffic modelling and estimating other impacts (e.g. alteration in land use).

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The paper is structured as follows: the results of the literature are reviewed in Section 2. The alteration in mobility services is summarized in Section 3. The elaboration method is detailed in Section 4. Section 5 presents its application. The paper is completed by the concluding remarks, including future research directions.

2 Literature review

The modal share can be calculated in several ways; however, the most proper way is to calculate it based on the covered travel distance. Studies analysing the alteration in modal share can be grouped: based on data estimated by traffic modelling or based on user expectations. The rate of AVs in the whole fleet was expected at 7-60 % in 2050 by transport experts [3]. Scenarios were compered in small-town Brunswick, too [7]. Individual and shared AV use were considered. The shared AVs replace only current car use; however, its modal share is low (2-3 %). The reason for the low value may be that the current modal share of public transport (PT) is low; thus, the modal share of cycling is high. Simulating the alteration in Singapore [8], it was expected that a shared AV-based mobility service could replace 10 % of the current feeder bus services. However, only the alteration in PT fleet was modelled, the willingness-to-shift from car use was neglected.
The willingness-to-shift is influenced by the expected gains. In terms of travel time, travellers consider AV-based mobility services less attractive, which can be characterized as a ride-sharing service, than services which can be characterized as a car-sharing service [9]. Similar to our research objective, the model share alteration was examined in [10]. However, in that research, everybody shifts from individual car use to AV-based taxi or shared taxi service. The willingness-to-shift was estimated for travel groups formed according to census data instead of individuals. It was assumed that AV-based taxi service is used by current car users without PT pass. Walking and high-capacity PT use remain a significant transport mode. The influence of travellers' socio-demographic characteristics on the use of AV-based services in Paris was examined [11]. Only AV-based car-sharing service was considered, the travellers' willingness-to-shift was calculated according to group characteristics. The model share of AV-based mobility service was estimated at 3.8-5.3 %; the shift from individual car use is typical; the modal share of PT increases as feeder AV-based services are used. The willingness-to-shift to shared AV-based service was estimated to Munich, too [12]. Only the shift from individual car use was considered; the modal share of other modes was deemed to be invariable. The result showed that the modal share of shared AVs is 5-13 %.
We conclude that current studies consider the willingness-to-shift at a superficial level. Assumptions are used regarding travel groups instead of revealing the individuals' willingness. Current studies focus mostly on the alteration in car use.

3 Alteration in mobility services

The AV-based mobility services could replace individual car use and the use of 'transitional' transport modes, such as car-sharing, taxi. The characteristics of transitional modes take place between the characteristics of the individual car and PT. The new AV-based mobility service provides mostly on-demand, shared, informatics-based service in which pre-ordering via mobile application is mandatory. A small capacity autonomous car (max 4 passengers) and the so-called pod (5-15 passengers) are considered. The service types are the following:

- taxi: door-to-door service between any departure and arrival points without sharing.
- shared taxi: like the taxi service type but with sharing.
- feeder pod: feeder service from any departure points in a zone to the stop of a high capacity line.
- fixed route pod: mostly feeder service on fix route with fix stops.

It is operated according to a timetable, but additional departures may be inserted according to demand. [13]
Since large one-directional travel demands can be served efficiently by high-capacity, arterial PT lines (e.g. subway), their role remains significant. Moreover, the automation of PT vehicles is also expected. The role of soft mobility forms, such as walking, cycling, using micromobility forms, remains essential. Transport modes in the future are as follows (i) individual modes: non-motorized (walking, bicycle), motorized (individual car, motorcycle, micromobility); (ii) PT: small capacity (non-motorized: bike-sharing; motorized: shared AV, shared micromobility), high capacity (autonomous - e.g. bus, automated - e.g. subway).

4 Methods: estimation of modal share alteration

4.1 Questionnaire survey

The stated preferences about not known or barely known facts can be collected by a questionnaire survey; the respondents' opinions about an imagined situation are measured. The risk of measuring stated preferences is high as the respondent may act in a different way in fact. As AV-based services barely exist, we conducted a survey measuring the stated preferences about the willingness-to-shift. The questions were assigned to the following groups:
- socio-demographic characteristics for filtering, for correlation analyses;
- current mobility habits for the calculation of current modal share;
- using AV-based mobility services for the calculation of future modal share.

The structure of the questionnaire is presented in Fig. 1. One multiple-choice question (signed by box) describes one characteristic of a person or mobility habits. Different values are assigned to one character as a variable according to the answers. Sub-questions are used for different motivations, such as working, shopping, recreational (signed by dark blue). The respondents provide data about mobility habits according to motivations:

II.1 frequently used transport mode. Options: cycling, car use (as a driver or as a passenger), PT use, combined transport use (individual car + PT).

II.2 covered distance. Options: ‹1 km, 1-3 km, 3-5 km, 5-10 km, ›10 km.

II.3 frequency of traveling. Options: daily (5-6 times/week), several times in a week (3-4 times/week), weekly (1-2 times/week), rarely.

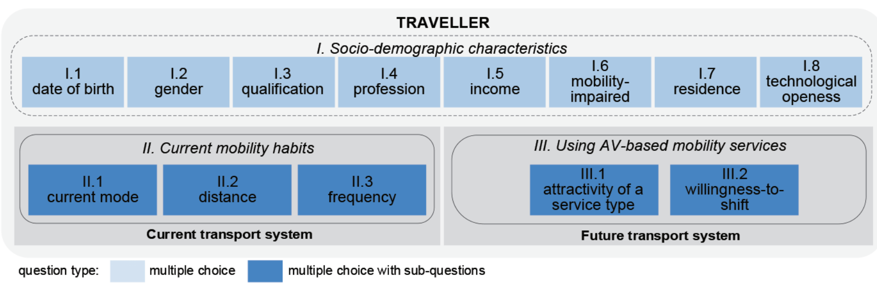

Figure 1 Structure of the questionnaire

As AV-based mobility services have not been operated in Hungary yet, the survey contains a description of the service types. The respondents provide data regarding the use of AV-based mobility services according to motivations: (III.1) preferred service type instead of current transport mode; (III.2) frequency of willingness-to-shift to AV-based service types - options: never, every second time, every time.

4.2 Calculation method

The used indexes are k respondent k∈N, i current transport mode i=1..4 (1: walking, 2: cycling, 3: individual car use, 4: PT use), j AV-based mobility service type: j=1..4 (1: taxi, 2: shared taxi, 3: feeder pod, 4: fixed route pod), m motivation m=1..3 (1: working, 2: shopping, 3: recreational).
The following variables can be determined from the survey. The respondents chose the appropriate options; the options were transformed into values, respectively. The set of values may alter according to the characteristics of the application field. The listed values are typical for Hungarian urban traveling.

- ${}_k^i l^m$ - travel distance of k respondent with i mode according to m motivation [km]. Options: ‹1 km: 1, 1-3 km: 2, 3-5 km, 5-10 km: 8, › 10 km: 12.
- ${}_k^i f^m$ - travel frequency of k respondent with i mode according to m motivation [travels/month]. Options: daily: 20, several times: 15, weekly: 10, rarely: 5.
- ${}_k^i a_j^m$ - willingness-to-shift of k respondent from i mode to j type according to m motivation [%]. Options: every time: 1, every second time: 0.5, never: 0.

Step 1: current modal share: The current modal share is calculated based on the survey. ${}^iM$ signs the current modal share of i transport mode, Eq. (1).

$$ {}^iM = \frac{{}^iL}{\sum_i {}^iL} \tag{1}$$

${}^iL$ signs the total travel distance with i mode [km], Eq. (2). The distances of each respondent and each motivation are considered. The modal share can be calculated according to motivations (${}^iM^m$); the summarization according to motivation is not needed in Eq. (2).

$$ {}^iL = \sum_m \sum_k {}_k^i l \cdot {}_k^i f^m \tag{2}$$

Step 2: future modal share: ${}^iM^*$ signs the future modal share of i mode, Eq. (3), and M^*_j signs the future modal share of j AV-based service type, Eq. (4).

$$ {}^iM^* = \frac{{}^iL^*}{\sum_i {}^iL^* + \sum_j L_j^*} \tag{3}$$

$$ M_j^* = \frac{L_j^*}{\sum_i {}^iL^* + \sum_j L_j^*} \tag{4}$$

${}^iL^*$ signs the total future travel distance with i mode, Eq. (5). It represents the remaining travel distance with i mode after the shifting to all the j types.

$$ {}^iL^* = {}^iL - \sum_j {}^iL_j^* \tag{5}$$

L^*_j signs the total future travel distance with j AV-based service type, Eq. (6).

$$ L_j^* = \sum_i {}^iL_j^* \tag{6}$$

${}^iL^*_j$ summarizes the total future travel distance with j type instead of i current mode according to m motivation, Eq. (7).

$$ {}^{i}L_{j}^{*} = \sum_{m}\sum_{k} {}_{k}^{i}l^{m} \cdot {}_{k}^{i}f^{m} \cdot {}_{k}^{i}a_{j}^{m} \cdot {}^{i}c \quad (7) $$

${}^{i}c$ is a correction factor describing the proportion of the ${}^{i}M_{real}$ real and the ${}^{i}M$ calculated modal shares, Eq. (8). Its application is needed if the survey is not representative for the real modal share to manage the under- or overrepresentation of the modes. If the real modal share is available according to motivations, the ${}^{i}c^{m}$ correction factor according to motivation can be involved in Eq. (7).

$$ {}^{i}c = \frac{{}^{i}M_{real}}{{}^{i}M} \quad (8) $$

If the current i mode is shifted to feeder pod (j = 3) or fixed route pod (j = 4), the travel chain contains a feeder distance with a shared AV and a PT distance with a high capacity vehicle if the length of travel is long enough. The feeder distance is indicated by l_j. Its value is constant in the whole study area and depends on the area. The distance covered by PT is calculated as ${}_{k}^{i}l^{m} - l_j$. Thus, the additional total future travel distance with PT (${}^{i}L^{*}_{j=3,4}$) is calculated by Eq. (9). This additional PT distance is added to the future total travel distance with PT (${}^{i}L^{*}$).

$$ {}^{i}L_{j=3,4}^{*} = \sum_{m}\sum_{k} ({}_{k}^{i}l^{m} - l_{j=3,4}) \cdot {}_{k}^{i}f^{m} \cdot {}_{k}^{i}a_{j=3,4}^{m} \cdot {}^{i}c \quad (9) $$

The modal share can be calculated for different m motivations; in these cases, summarization according to m motivation is not needed in Eq. (2), (7) and (9).
The limitation of the method is that only the willingness-to-shift from current mode to AV-based service type is considered. Other impacts (e.g. promotion of cycling) are neglected. In the case of combined transport, the travel distance is divided by car and PT use in a proportion of fifty-fifty. Moreover, the feeder distance l_j is a constant value independently of the real network and the current distance covered by car use. However, even with the limitations, tendencies can be determined.

5 Case study

The method was applied to estimate the modal shift, the alteration in the use of modes, in Budapest, Hungary. An online survey was conducted in February 2018. 510 responses have been received. Statistical or random sampling could not have been executed; thus, the sample is not representative. However, relevant consequences can be drawn as the number of respondents is relatively high. The question regarding the residence filtering the citizens of Budapest was used.
The latest reliable official distance-based modal share data from 2017 provided by Centre for Budapest Transport (BKK) were considered. As we conducted the survey at the beginning of 2018, the respondents considered mostly rides travelled in 2017. Official modal share data are 11 % walking, 2 % cycling, 40 % car use, 47 % PT use. As the modal share data according to motivations were not available as reference data, the surveyed data were summarized according to motivations.
Step 1, current modal share: according to 304 responses from Budapest, the total travel distances in km are walking: 2605, cycling: 3850, individual car use: 20 205, PT use: 42 845. Additionally, the calculated current modal share in percentage [%] are walking: 4, cycling: 5.5, car use: 29, PT use: 61.5.
Step 2, future modal share: as the official and the calculated modal share are not the same, the application of ${}^{i}c$ was needed: walking: 2.93, cycling: 0.36, car use: 1.38, PT use: 0.76. Furthermore, $l_{j=3,4}$ = 2 was considered; we assumed that 2 km feeder distance is adequate considering the dense PT network operating in Budapest. Table 1 presents the alteration in travel distance according to modes. What percentage of traveled kilometers is covered by

the current mode or by the AV-based service types, as well as, what is the increment in the modal share of PT as the consequence of the use of feeder types? (Cells regarding the not considered modal shift between current transport modes are empty.) The future modal share is depicted in Fig. 2 presenting the official modal share as a reference.

Table 1 Alteration in travel distance according to transport modes [%]

		Transport mode (i)				AV-based service type (j)				i
		walking	cycling	car use	PT use	taxi	shared taxi	feeder pod	fixed route pod	PT use incre-ment
i	modes	1	2	3	4	1	2	3	4	4
1	walking	39	-	-	-	9	19	9	14	10
2	cycling	-	59	-	-	7	6	4	6.5	17.5
3	car use	-	-	31	-	22	19	4	4.5	19.5
4	PT use	-	-	-	43	7	11.5	7	7.5	24

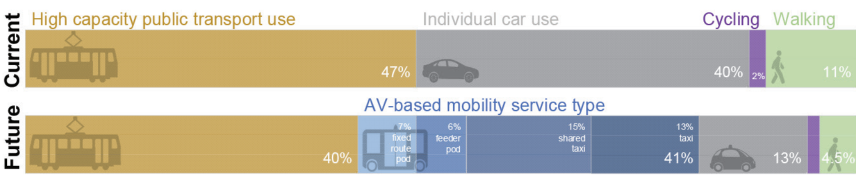

Figure 2 Alteration in modal share (source of current modal share: BKK, 2017)

The individual car use can be significantly reduced by the introduction of flexible AV-based service types; similar consequences were drawn in [10]. Current car users' willingness to shift is the highest; its modal share is reduced from 40 % to 13 %; only 31 % of the current travel distance covered by car use remains. The travellers' willingness-to-shift covering big distances is significant as the increment of PT use is significant in all cases. However, as a constant feeder distance was considered, the percentage of feeder types is low; thus, the increment in PT is high. The willingness-to-shift from walking is popular both in short and long travels as the percentage of shared taxi and the increment of PT use are high. Bikers' willingness-to-shift is the lowest; the willingness to use feeder service types is also high in the case of long travels. The modal share of traditional PT is reduced, but if AV-based feeder service types are considered as PT, as they have similar characteristics as a traditional public bus service, the modal share of PT increases significantly. Small capacity bus lines are expected to be replaced by AV-based feeder services.

6 Conclusion

The alteration in modal share is expected after the introduction of AV-based mobility services. The main contribution of our research is the modal share estimation method considering the travellers' current mobility habits and willingness-to-shift. The method was applied in Budapest, Hungary, as a case study. We found that the current individual car users' willingness-to-shift is high. Individual car use can be reduced from 40 % to 13 % with the introduction of flexible AV-based service types, such as taxi or shared taxi. The willingness-to-shift is the lowest among bikers and PT users. However, these travellers are also willing to shift in the case of long travel; furthermore, less flexible AV-based feeder service types, such as feeder pod or fixed route pod, are rather popular. The method could be improved by removing limitations (e.g. use of different feeder distances), or with the application of specific values instead of categories. The research potential in the field examined in this paper is significant; our future research focuses on removing limitations and elaboration of additional estimation methods regarding impacts of AVs considering more user expectations (e.g. alteration in land use).

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References

[1] Alessandrini, A., Campagna, A., Site, P.D., Filippi, F., Persia, L.: Automated Vehicles and the Rethinking of Mobility and Cities, Transportation Research Procedia, 5 (2015), pp. 145-160, doi: 10.1016/j.trpro.2015.01.002

[2] Fagnant, D.J., Kockelman, K.: Preparing a nation for autonomous vehicles: opportunities, barriers and policy recommendations, Transportation Research Part A: Policy and Practice, 77 (2015), pp. 167-181, doi: 10.1016/j.tra.2015.04.003

[3] Milakis, D., Snelder, M., van Arem B., van Wee G.P., Correia H.A.G.: Development of automated vehicles in the Netherlands: scenarios for 2030 and 2050, European Journal of Transport and Infrastructure Research, 17 (2017) 1, pp. 63-85, doi: 10.18757/ejtir.2017.17.1.3180

[4] Gruel, W., Stanford, J.M.: Assessing the long-term effects of autonomous vehicles: a speculative approach, Transportation Research Procedia, 13 (2016), pp. 18-29, doi: 10.1016/j.trpro.2016.05.003

[5] Nordhoff, S., de Winter, J., Madiga, R., Merat, N., van Arem, B., Happee, R.: User acceptance of automated shuttles in Berlin-Schöneberg: A questionnaire study, Transportation Research Part F: Traffic Psychology and Behaviour, 58 (2017), pp. 843-854, doi: 10.1016/j.trf.2018.06.024

[6] Bansal, P., Kockelman, K.M., Singh, A.: Assessing public opinions of and interest in new vehicle technologies: An Austin perspective, Transportation Research Part C: Emerging Technologies, 67 (2016), pp. 1–14, doi: 10.1016/j.trc.2016.01.019

[7] Cyganski, R., Heinrichs M., von Schmidt A., Krajzewicz D.: Simulation of automated transport offers for the city of Brunswick, Procedia Computer Science, 130 (2018), pp. 872-879, doi: 10.1016/j.procs.2018.04.083

[8] Shen, Y., Zhang, H., Zhao, J.: Integrating shared autonomous vehicle in public transportation system: A supply-side simulation of the first-mile service in Singapore, Transportation Research Part A: Policy and Practice, 113 (2018), pp. 125-136, doi: 10.1016/j.tra.2018.04.004

[9] Kolarova, V., Steck, F., Bahamonde-Birke, F.J.: Assessing the effect of autonomous driving on value of travel time savings: A comparison between current and future preferences, Transportation Research Part A: Policy and Practice, 129 (2019), pp. 155-169, doi: 10.1016/j.tra.2019.08.011

[10] Martinez, L.M., Viegas, J.M.: Assessing the impacts of deploying a shared self-driving urban mobility system: An agent-based model applied to the city of Lisbon, Portugal, International Journal of Transportation Science and Technology, 6 (2017) 1, pp. 13-27, doi: 10.1016/j.ijtst.2017.05.005

[11] Kamel, J., Vosooghi, R., Puchinger, J., Ksontini, F., Sirin G.: Exploring the Impact of User Preferences on Shared Autonomous Vehicle Modal Split: A Multi-Agent Simulation Approach, Transportation Research Procedia, 37 (2019), pp. 115-122, doi: 10.1016/j.trpro.2018.12.173

[12] Moreno, A.T., Michalski, A., Llorca, C., Moeckel, R.: Shared Autonomous Vehicles Effect on Vehicle-Km Traveled and Average Trip Duration, Journal of Advanced Transportation, Article ID 8969353 (2018), doi: 10.1155/2018/8969353

[13] Földes, D., Csiszár, Cs.: Framework for planning the mobility service based on autonomous vehicles, Smart Cities Symposium Prague 2018, pp.

IMPACT ASSESSMENT OF COOPERATIVE INTELLIGENT SERVICES ON THE TEN-T ROAD NETWORK OF HUNGARY

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Abstract

Cooperative intelligent transport systems and services (C-ITS) are based on periodical data sharing between cooperative V2X communication units (in-vehicle and roadside units, ITS stations) via a short-range ad-hoc network. All participants are able to acquire information several times per second from others such as position, speed and driving direction as well as intentions and event triggered messages like incidents and emergency braking. Roadside infrastructure can share traffic information like road works or speed limits.
As such, C-ITS improves road safety and effectiveness of the transportation system while reduces harmful environmental effects. Transportation authorities and road network operators use cost-benefit analysis to decide on necessary developments. Taking into account only the momentary statistical renewal rate of the vehicle fleet (disregarding all handheld devices), a fully capable operation of such a system can only be predicted in more than 10 years.
An overview of existing C-ITS use cases throughout Europe and a simplified comparative analysis of estimated costs and quantifiable benefits of such a system in Hungary is presented in this article. Our research assigned the first developments (i.e. technologies and use cases) in the next 1-3 year period to prepare certain parts of the public road network for CAV testing.

Keywords: C-ITS, Day1 services, C-ITS benefits, C-ITS costs, C-ITS development

1 Methodology

To conduct the assessment described above, a collection of available and documented use cases is needed to understand and study the basic goals and parameters of any measures, including all the experience on deployment and operation [1]. This information helped us to define and analyse costs and benefits of possible development scenarios in Hungary, and to decide which C-ITS use cases worth to apply throughout the country based on its cost-benefit ratio. After calculating these indicators, it is possible to propose a strategy to be followed in the first years of C-ITS development.

2 C-ITS service as groups of use cases

Certain groups of use cases are feasible using the same infrastructure and hardware/software tools, so they are much more efficient than standalone use cases [3]. A step-by-step method was defined in the EU C-ITS strategy introducing Day1, Day1.5, Day2, Day3 and Day4 groups of C-ITS use cases (see Fig. 1). The simultaneous development, standardization (both

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in-vehicle and roadside), pilot projects and implementation of use case bundles can keep up pace with the development and uptake of the technologies used.
Day1 services focus on exchanging information enhancing foresighted driving. Day2 services improve the service quality and share perception and awareness information. Day3+ adds further sophisticated services like sharing intentions, supporting negotiation and cooperation that paves the way towards cooperative accident-free automated driving. The deployment of the cooperative V2X services proceeds in different innovation phases, starting with Day1, a basic set of information and warning services support low penetration rates of C-ITS capable road users during the market introduction. A few Day1 services are already available in cooperative V2X vehicles on certain European roads. Services related to Day2 and Day3+ phases are investigated in R&D projects that are generating the knowledge for developing related customised functions and standards (see Fig.1).

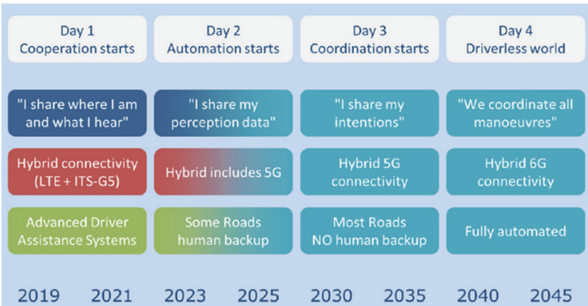

Figure 1 C-ITS services as suggested in the EU C-ITS strategy

Now we are focusing on Day1 and Day1.5 services listed in Table 1. Day1.5 services are feasible where Day1 services are using the same technology, networks and interfaces. The difference is the sphere where the benefits are realized since Day1 services generates social benefits while Day1.5 services mostly add value for individual users (parking information, charging stations, navigation services) or extend the user group to the vulnerable road users. Day1 and Day1.5 services present their warnings, instructions and information on a screen inside the target vehicle. Using these services, no automation level over SAE2 assumed since they do not trigger automatic longitudinal or lateral manoeuvres.

Table 1 List of Day1 and Day1.5 C-ITS services

	Day1 services	Type	Focus area	Bundle
1	Emergency brake light	V2V	Safety	1
2	Emergency vehicle approaching	V2V	Safety	1
3	Emergency vehicle approaching	V2V	Safety	1
4	Traffic jam ahead warning	V2V	Safety	1
5	Hazardous location notification	V2I	Motorway	2
6	Road works warning	V2I	Motorway	2
7	Weather conditions	V2I	Motorway	2
8	In-vehicle signage	V2I	Motorway	2
9	In-vehicle speed limits	V2I	Motorway	2
10	Probe vehicle data: CAM aggregation	V2I	Motorway	2
11	Shockwave Damping	V2I	Motorway	2
12	Green Light Optimal Speed Advisory (GLOSA)	V2I	Urban	3
13	Signal violation	V2I	Urban	3
14	Traffic signal priority request by designated vehicles	V2I	Urban	3
	Day1.5 services	Type	Focus area	Bundle
15	Off street parking information	V2I	Parking	4
16	On street parking information and management	V2I	Parking	4
17	Park & Ride information	V2I	Parking	4
18	Information on AFV fuelling & charging stations	V2I	Routing	5
19	Traffic information and smart routing	V2I	Routing	5
20	Zone access control for urban areas	V2I	Routing	5

3 Impact assessment and results

In the course of the impact assessment, expected costs and expected social benefits of the introduction of C-ITS Day1 services were quantified based on the methodology of RICARDO-TFT-TEPR 2018 (R-T-T-2018) [4].
In our calculations, we took into account the regulatory environment of Policy Option 2 (PO2). PO2 specifies C-ITS services, common service profiles and compliance with C-ITS policies in a legally binding delegated regulation. This policy has a strong emphasis on coordination and standardization but does not include an obligation to deploy Day1 V2V services, and does not create legal bodies to perform security and compliance assessment tasks.

3.1 List of considered services and road network (scope) to be implemented

As an initial step in the impact assessment, the range of services and network elements to be taken into consideration were defined. In the calculations only Bundle1 and Bundle2 measures of Day1 services were examined, and the effects of the three sequential scenarios built from them:
- In the Baseline scenario (case without the project), the measures of Bundle 1 will work with a modest penetration, and the already established V2I services will be maintained in the future but will not be expanded.

- In Option A, V2V measures of Bundle 1 will work with more ambitious penetration rates, and services of Bundle 2 will be built on the entire TEN-T core network.
- In Option B, the measures of Bundle 1 and Bundle 2 will be available on both TEN-T core and comprehensive network (gradually, but with full development over a few years).

3.2 Socio-economic conditions, traffic forecasts

Socio-economic conditions (general assumptions) primarily determine the number of vehicles and the uptake rate of in-vehicle equipment. The number of vehicles equipped with C-ITS unit on the national road network depends on:
- the long-term trend of the country's GDP forecasts;
- the country's population forecast;
- motorization trends (based on the relation of GDP to the degree of motorization);
- the penetration rate of C-ITS equipment, which is strongly influenced by the policy option considered (PO2).

Social, economic and traffic development data were taken from the National Traffic Survey [OCF-2016] [6]. The change in the number of passenger cars was calculated as the product of the population and the expected degree of motorization. In the case of heavy goods vehicles, the OCF-2016 study points to the peculiarity that the rearrangement of the transport market among vehicle categories predicts different fleet developments for vehicles smaller and larger than 3.5t. It is estimated that the number of vans is growing dynamically, while the number of heavy goods vehicles is expected to stagnate and fall slightly - in line with international trends. The characteristic feature of the traffic development is that the projected passenger and heavy truck traffic on the motorways is constantly increasing, but on the main road network it decreases significantly, while the traffic performance of vans increases on both types of roads. Further general assumptions:
- the calculation of social costs and benefits does not take into account changes in GDP and inflation,
- costs incurred and social benefits are expressed in real terms, at 2015 price level, in Euros,
- the evaluation period runs from 2020 to 2030.

3.3 In-vehicle and road-side C-ITS equipment

The baseline scenario is a case in which "no further EU action" takes place beyond the ongoing EU C-ITS activities. Only developments that have already been initiated by national or regional authorities are expected - these will be continued throughout the evaluation period (until 2030).
In the case of vehicles, new factory-fitted vehicles reach 100 % uptake in 4 facelift (mid-model) cycles. (Model facelift cycle in the PO2 environment is 4 years for cars and 5 years for trucks). Retrofitting is only possible through smartphones via downloaded application, so ITS devices are unable to deliver the safety V2V services of Bundle 1, but V2I services are available, and a maximum penetration of 95 % can be assumed. Figure 2 illustrates the development of passenger car and heavy goods vehicle equipment.

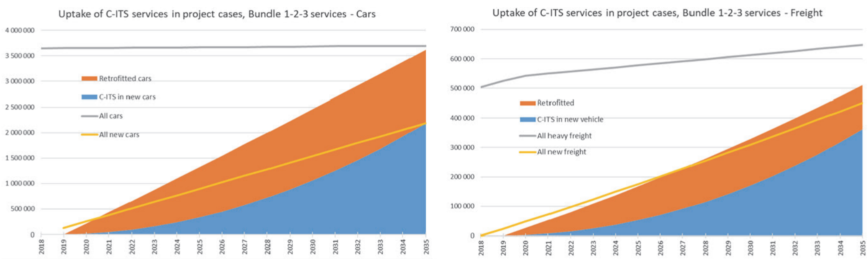

Figure 2 Uptake of C-ITS sub-systems in the Hungarian fleet

In project scenario, the pace of the deployment of road-side units (RSU) on the TEN-T core network starts from the 2020 deployment level and follows the trend of European front runner countries (Figure 3). On other motorways a 4-year lag was considered and 50 % of the growth of TEN-T core network was applied, on other rural roads 25 % of the growth of these TEN-T core network elements were taken into account.

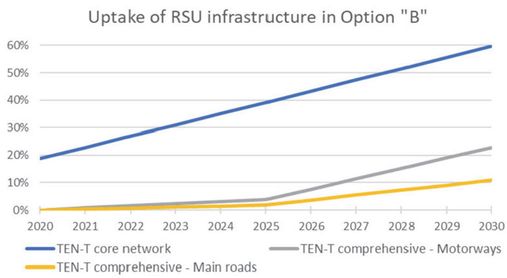

Figure 3 Uptake of road-side unit infrastructure

3.4 Investment and operation costs, social benefits

The following costs are incurred in setting up and operating C-ITS systems:

- Central ITS sub-systems (installation, operation);
- Personal ITS sub-systems, i.e. smartphones, that can be used for V2I communication and in the future also for V2V communication (app development, software update, operation). They are currently unable to communicate with low latency through 3G/4G, so this study does not count on personal Day1 V2V services.
- In-vehicle ITS subsystems – the study does not consider a retrofitted in-vehicle ITS sub-system (installation, operation, maintenance, software update);
- Roadside ITS subsystems– beacons on gantries, columns, smart traffic lights (installation, operation, maintenance).

Various forms of costs were accounted for: upfront costs, ongoing costs and replacement costs – when it was necessary. The costs were borne by various actors. Figure 4. shows the distribution of the costs among the mentioned objects.

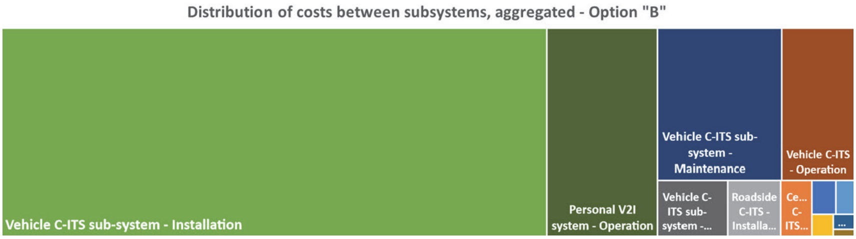

Figure 4 Distribution of costs among subsystems

Summarizing the costs, it can be stated (Figure 4.) that:

- the largest part of the costs (64 %) is for the purchase and then maintenance of in-vehicle systems.
- the operation of communication equipment is also a significant item (~ 20 %, mainly data flow);
- the deployment of TEN-T comprehensive network elements involves relatively low additional costs in case of an already the existing TEN-T core network C-ITS system.

The external social impacts of C-ITS systems may include:

- increase in transport efficiency (increase in average speed, %),
- change in fuel consumption (%),
- change in pollutant emissions (NOX, CO, VOC, PM %)
- change in accident risk (%).

The measures in Bundle1 and Bundle2 of Day1 services have a predominant impact on road safety, so accident savings were quantified. According to the R-T-T 2018 study, each measure helps to reduce the number of accidents. As the individual influence of each factor is difficult to separate, there are some overlapping impacts. To avoid multiple counting, the cumulated effect on traffic accidents is reduced by 10 % at the end of the calculation.
The number of injuries was taken into account on the basis of specific injury indexes calculated from accidents on the examined road network in 2016-2018. The decrease in the number of injuries compared to the baseline case in the core years is shown in Table 2.

Table 2 Impacts of Bundle 1. and 2. services on traffic safety

	Project A: TEN-T core network			**Project B: TEN-T core+compl**		
Decrease in the number of injured	2020	2025	2030	2020	2025	2030
Fatality	0,09	1,92	6,84	0,09	1,99	7,53
Severe injury	0,39	8,55	30,33	0,39	8,79	32,94
Slight injury	1,11	24,46	87,97	1,11	24,74	91,46

3.5 Results

The order of magnitude of the costs and benefits is the same, but the totally aggregated costs are about three times over the calculated benefits. Both the costs and benefits increase with the deployment of the network and the penetration of in-vehicle devices. Compared to Option “A”, the cost of Option “B” is slightly higher, but the benefits of Option “B” are significantly higher. In-vehicle costs are, of course, inseparable from the introduction of C-ITS services, but the last pair of figures illustrates the cash-flow diagram of the implementation of the C-ITS system with a free (“swallowed”) on-board device. Note that the cost of on-board units is borne by each vehicle owner, so it is shared among a large number of users, while installation of the infrastructure of each service is entirely service provider or road operator responsibility.

Figure 5 Cash-flow diagrams of feasible options

4 Conclusions

Assessing the calculation of costs and benefits of Day1 C-ITS services above, our conclusions are as follows:

1. A strong prerequisite to start any C-ITS service is an OBU in all possible vehicles;
2. The up-front cost of OBUs is huge – but beyond this threshold, additional services and social benefits can be offered “cheaply”.
3. A first wave of easily deployable services will induce benefits in the field of traffic safety.
4. It is clear that the service area and the number of services can also be extended relatively cheap after the first areas are covered with the first group of services.

There are further considerations with effect on the costs and benefits:

- Based on traffic counts done in 2019, the official traffic prediction for Hungary (made in 2016 and only upscaled in 2018) seems to be underestimating road traffic on motorways and first-class main roads. Greater traffic performance with the same costs of C-ITS development means greater benefits for the society.
- Statistical Value of Life (SVL) used to calculate benefits from the accidents that do not happen in the future is a thin ice to walk on. We kept the values from the R-T-T 2018 study applied for the whole EU. However, there is a newly published “Handbook on the external costs of transport, 2019” since, with significantly (30-50 %) higher SVL values. Our estimation seems to be too conservative in this respect.
- We see an „early market” phenomenon regarding the OBUs. When implementing C-ITS systems, the early adopters pay most of the price of technical development and starting series

production. Most probably, after the first years of investments into such systems, prices of on-board and roadside units will drop significantly (at least by 50 %).

- This is a forward-looking, preventive development. There are further developments with significant benefits that can be realized on the same communication infrastructure with the same interfaces where only application development ("software") is necessary.
- The only benefits in our calculation were the accidents that will not happen thanks to the new C-ITS systems. No other benefits like less congested hours (time savings multiplied by time value) or smaller environmental impacts were calculated – just because that would be another study. But we can assume that only the time spared without the congestions after the accidents withheld has a time value of the same order of magnitude as the benefit from the lower number of accidents themselves.

These considerations suggest that the real benefits of such a C-ITS project described above are significantly higher while the costs can be much lower than calculated.

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References

[1] C-ROADS: Annual pilot overview report 2018, v1.0, 2019.

[2] Communication from the Commission to the European Parliament, the Council, the European Economic and Social Committee and the Committee of the Regions: A European strategy on Cooperative Intelligent Transport Systems, a milestone towards cooperative, connected and automated mobility, COM (2016), 766, http://eur-lex.europa.eu/legal-content/EN/AUTO/?uri=CELEX:52016DC0766

[3] Study on the Deployment of C-ITS in Europe: Final Report, WG4 - ANNEX1 - Analysis of Data Protection and Privacy in the context of C-ITS, Report for DG MOVE, https://ec.europa.eu/transport/sites/transport/files/themes/its/doc/c-its-platform-final-report-january-2016.pdf, 2016.

[4] Ricardo,TFT,TEPR: Support study for Impact Assessment of Cooperative Intelligent Transport Systems, Final Report – on behalf of the European Commission, Directorate-General for Mobility and Transport, 2018.

[5] European Commission: Study on the Deployment of C-ITS in Europe, Final Report Framework Contract on Impact Assessment and Evaluation Studies in the Field of Transport, 2013.

[6] KTI Közlekedéstudományi Intézet Nonprofit Kft. : Országos célforgalmi felvétel és mátrixok kidolgozása (OCF-2016), TEN-T elemzések - Egyeztetett záró dokumentum.

[7] Mobility Research Centre of the Institute of Transportation Sciences: Development of national traffic survey and matrices (OCF-2016), TEN-T analyzes - Agreed final document, 2017.

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THE ACCESSIBILITY OF RAIL TRANSPORT TO PEOPLE WITH REDUCED MOBILITY – CASE STUDY

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Abstract

The means of rail transport as well as other modes of transport primarily enable people to easily overcome certain distances over a period of time. This implies the need for it to be adapted to such specificities, together with other elements in places where high concentrations of passengers are exchanged. This paper deals with a common problem of accessibility of rail transport to people with reduced mobility (PRM). PRM refers to people whose mobility is reduced due to physical incapacity, an intellectual deficiency, age, illness or any other cause of disability when using transport. Therefore, a thorough analysis of accessibility of rail transport to this group of passengers will be conducted for the city of Zagreb according to certain indicators. The rail station is considered to be accessible if a PRM passenger can enter passenger terminal, consume services available there and if is able to get to the platform from where the train will depart. The rolling stock is accessible if a PRM passenger can buy a ticket, enter the train and have predicted and available space in the train. By determining the real state of accessibility to PRM passengers of all railway stations and used rolling stock in suburban rail transit in the city of Zagreb, measures will be proposed to increase accessibility to these passengers and thus facilitate their integration into society in this segment and increase their quality of life.

Keywords: people with reduced mobility, suburban rail transport, accessibility, case study

1 Introduction

In the literature, there are various explanations of accessibility. Generally, it has been defined as some measure of spatial separation of human activities, representing the ease with which activities may be reached from a given location using a particular transportation system [1], [2]. Moreover, Litman interprets that when people say “location, location, location,” they really mean “accessibility, accessibility, accessibility” and that it refers to people’s ability to reach goods, services and activities, which is the ultimate goal of most transport activity [3]. According to him, accessibility can be defined in terms of potential (opportunities that could be reached) or in terms of activity (opportunities that are reached). Rietveld defined accessibility as the potential of opportunities for interaction based on both network features and the spatial distribution of activities. Since for each activity the relevant types of destinations are different, one may have to identify different accessibility indicators [4]. Mamun and Lownes described trip, spatial and temporal coverage as three primary components of accessibility, to which can be added an aspect that reflects comfort as a sufficient space available at the public transport at the time one wants to travel [5].

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Once the accessibility is defined, the real question is in measurement of it [6]. One of the propositions for measuring public transport (PT) accessibility for European cities includes population distribution and frequency of the service [7]. Authors defined five groups of accessibility, based on the access and departure frequency, from no access if people cannot easily walk and reach PT stop within 5 minutes (bus or tram) or 10 minutes (metro or rail) to very high access that provides more than 10 departures/h. The accessibility of public transport in Croatia, shown in Figure 1, was examined in a similar way. As can be seen, a good level of public transport availability is a key generator of mobility. This mostly applies to the metropolitan areas of Zagreb, Osijek, Rijeka and Split, where the highest number of daily migrations takes place. Compared to other considered cities, public transport in the wider area of the City of Zagreb is dominantly more developed. Given that its main backbones are the urban and suburban railways, the research of accessibility in this paper is focused on the urban railway of the City of Zagreb.

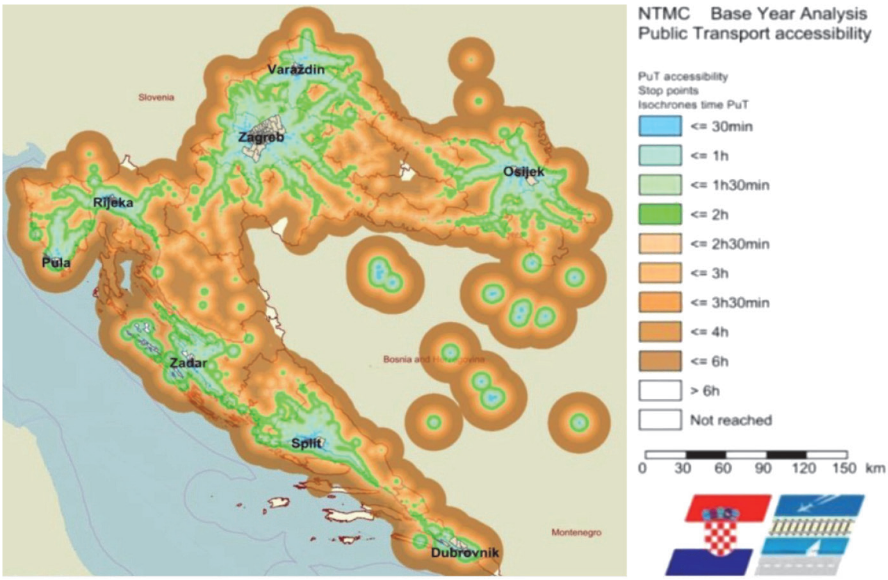

Figure 1 Public transport accessibility in Croatia, [8]

Railways, like any other PT, primarily integrate different social and economic needs, making themselves available to all potential users. However, shortcomings in accessibility to key integration points have the greatest impact on more vulnerable user groups such as people with reduced mobility (PRM). PRM refers to people whose mobility is temporary or permanent reduced due to physical incapacity, an intellectual deficiency, age, illness or any other cause of disability when using transport. Therefore, the following analysis of accessibility of rail transport, in terms of the current state of the infrastructure elements, rolling stock and organizational aspect, is considered from the aspect of PRM.

After this short introduction, the second chapter explains the chosen approach in data processing. This is followed by an analysis of the accessibility of urban rail transport for PRM in the city of Zagreb in the third chapter. The fourth chapter summarizes the proposals for its improvement, and the last one gives a final review of the obtained results and guidelines for future work.

2 Methodology

The area of the urban railway in this analysis covers a total of 6 stations and 12 stops. Among them, the largest is Zagreb Main Station with more than 2,6 million of passengers in 2018, while the following are Sesvete with 615 000 of passengers and Vrapče with 351 000 [9]. In order to facilitate the monitoring of the further course of analysis, these stations and stops are grouped according to the directions and main lines on which they are located. The borders of the selected routes are the last stations or stops that are within the administrative area of the city of Zagreb. According to the above, the analysis includes:

- direction towards Zaprešić: Zagreb Main Station, Zagreb West Station, Kustošija, Vrapče, Gajnice, Podsused;
- direction towards Dugo Selo: Maksimir, Trnava, Čulinec, Sesvete, Sesvetski Kraljevec;
- direction towards Karlovac: Remetinec, Hrvatski Leskovac, Horvati, Mavračići;
- direction towards Velika Gorica: Klara, Buzin and Odra.

Accessibility to rail transport in these selected points is analysed from several perspectives. On the one hand it is the accessibility to the station or stop from which the journey by rail begins or ends, and on the other the accessibility to the railway vehicle that allows the distance to be covered. In order to cover both of these aspects, the set of necessary elements has been determined. Their existence and condition will contribute to the formation of a detailed overview of the current availability of rail transport in Croatia to PRM. Those important for the availability of railway buildings are tactile tapes, wheelchair ramp on station entrance, necessary infrastructure elements that would enable PRM to reach all platforms, elevators, escalators, toilets for PRM, step-free access to the services at the station and voice and video information systems. Selected elements for accessibility in railway vehicles are floor height in relation to the platform, way of opening the door, wheelchair ramp at the train, dedicated place for wheelchair and toilets for PRM. Another important aspect is the way in which the service is adapted to PRM. Therefore, the ratio of low-floor trains to the total number of trains, possibility of assisting when passengers enter and exit the train and the existence of other benefits like more favorable transport prices were selected for the indicators. Once these elements and indicators were identified, field research was conducted. The collected data were then compiled for a synthesized review of the accessibility of railway service to people with reduced mobility in the city of Zagreb.

3 Analysis of accessibility of rail transport to PRM - case study Zagreb

The overall state of equipment of stations and stops with the necessary elements which ensure the accessibility of railway transport to PRM is shown in Table 1. Depending on the observed direction, the maximum number of a particular element of accessibility corresponds to the largest number of stations and stops located on a given route. It can be seen that the equipment of the stations and stops on route I, the one in the direction of Zaprešić, is significantly better than at the stations and stops on other routes, although the condition of equipment is not in a completely satisfactory in all passenger buildings.

Table 1 Overview of the elements of accessibility at passenger buildings

Element	Presence of elements in stations/stops per maximum in each direction			
	I (max. 6)	II (max. 5)	III (max. 4)	IV (max. 3)
Tactile tapes	3	0	0	1
Wheelchair ramp on station entrance	4	1	0	2
Step-free access to the services at the station	6	5	0	0
Infrastructure elements to reach all platforms	4	3	1	1
Elevator	3	0	0	1
Escalator	0	0	0	0
Toilets for PRM	1	0	0	1
Video information system about timetable and delays	1	0	0	0
Voice information system about timetable and delays	4	1	1	1

A wheelchair ramp is one of the most important elements that would enable PRM to access the station/stop. On the line towards Zaprešić, only Podsused and Zagreb West Station do not have ramp. Other stops have it and their overall condition is good. On the line towards Dugo Selo, ramp is placed only in stop Sesvetski Kraljevec, while on the line towards Karlovac there is none. Odra and Buzin on the line towards Velika Gorica both have ramps which are in excellent condition. Buzin also has an elevator in excellent condition. This is a good substitute for the lack of a ramp, as is the case with elevator in Podsused, which is in the same condition as previous. However, two other elevators on the same line, in Gajnice and Vrapče, are not in the function. Also, no escalator is applied anywhere in the area under consideration. Once the user is in the terminal, it is necessary to enable step-free access to all services that terminal provides (shops, ticket, etc.). This condition is best met by passenger buildings on the route to Zaprešić, and otherwise to Dugo Selo. It is not surprising since this is where the busiest line of urban and suburban rail transport runs. However, if the infrastructural elements for accessing the platforms are observed, the situation is worse. Zagreb Main Station and Zagreb West Station on I. direction, Maksimir and Čulinec on the II., Horvati, Remetinec and Hrvatski Leskovac on the III. and Odra and Klara on the IV. direction do not meet this condition. In terms of existence of toilets for PRM, they are only in Zagreb Main Station and Buzin, but the one in Buzin is not functional. Tactile straps significantly facilitate the orientation and movement of passengers. Unfortunately, guidance with their help is possible only in Buzin, Gajnice, Vrapče and Podsused. Informing passengers about stable and changing data can generally be done visually (video) or by voice. Stable data refer to, for example, timetables and travel directions, while variable data are often delays. Most of the observed passenger buildings do not have any of these systems, or if do, they are in poor condition or not working like those in Vrapče, Gajnice and Podsused. On the contrary, voice information for passengers is excellent in Sesvete, Hrvatski Leskovac and Buzin. Video informing is available only in Zagreb Main Station.

Urban and suburban rail transport of the city of Zagreb uses different electric multiple units (EMU). There are two basic types, the older trains HŽ 6111 and the newer HŽ 6112. From the point of view of accessibility, the main difference between them is in the height of the floor in relation to the platforms and in the interior of the train. Older EMU trains have stairs at the entrance and are not adapted to PRM, while newer ones are low-floor and equipped with wheelchair ramps. In addition to the above, the newer trains have a wider front door, a dedicated wheelchair space and a custom toilet. In older versions, the doors open and close automatically, while in newer versions they open by pressing a visually and tactilely highlighted button._Besides the above, classic train compositions with locomotives and wagons, powered by both electric and diesel, as well as diesel multiple units (DMU) intended for regional and international needs, also participate in the provision of railway services in selected area. While different versions of the DMU are similar to the previously described EMUs in terms of accessibility, the difference in the types of passenger wagons used is significant in the classic compositions. They are usually not equipped with special ramps, have a narrow and stepped entrance and narrow toilets and passages between the seats without a special place for wheelchairs. To open the door, you need to turn the lever manually or press a button, which also varies.

If observing the proportion of low-floor trains, with respect to all passenger trains (excluding fast and IC trains) that operate on the considered sections, most newer EMUs and DMUs are in use on the section towards Zaprešić (79 %), then towards Velika Gorica (74.3 %) and Dugo Selo (63.6 %), while the least is towards Karlovac (26.5 %), [10]. Moreover, regarding the organizational aspect of PRM transportation by rail, it is necessary to announce the planned journey at latest 48 hours in advance. In that case, the employees of the infrastructure manager and the operator assist while entering/exiting the train free of charge. However, this solution is not the most convenient because it restricts the user's freedom in choosing travel time. On the other side, national passenger operator HŽ Putnički prijevoz offers two types of benefits in the form of smart cards for users in PRM category, [11]. Firstly, all Croatian pensioners and persons over the age of 60 are entitled to an annual 50 % discount on domestic rail travel. Secondly, persons with disabilities are allowed four journeys a year with a 75 % discount from the regular transport price in the 1st or 2nd class of passenger and fast train, while their companions receive a 100 % discount. A special category consists of military and civilian war invalids who have one free journey per year, in 1st and 2nd class passenger or fast train, to visit the grave of the deceased or four journeys per year with 75 % discount from regular transport fares in 1st or 2nd class passenger or fast train. Their companions also receive a 100 % discount. Furthermore, members of the Croatian Association of the Blind and the Croatian Association of Deafblind People DODIR and one accompanying person receive a discount of 50 % on all regular trains in domestic traffic. Members of the Association of Disabled Workers of Zagreb are entitled to a discount of 50 % if they are retired and unemployed, or 30 % if they are employed, but at distances greater than 25 km.

4 Increase of accessibility of rail system

At the level of the European Union (EU), access to platforms, stations, rolling stocks and other facilities that railway undertakers and station managers should provide to PRM is regulated by regulation No 1371/2007 of the European Parliament and the Council (article 21) and accompanying Commission Regulation (EU) No 1300/2014 Technical specifications for interoperability (TSI) for PRM [12], [13]. Furthermore, regulation of this issue at the national level relies directly on this TSI, as Croatia adopted the PRM National Implementation Plan (NIP) in 2017.

The point 4.2.1 in the PRM TSI describes functional and technical specifications of the infrastructure subsystem related to accessibility for persons with disabilities and PRM. Table 2 shows the results of the analysis of the condition of stations and stops in Croatia according to the mentioned requirements. As can be seen only 8 % of stations and stops in Croatia met at least 50 % of the PRM accessibility requirements. The plan is to modernize 109 stops and stations in the next 10 to 15 years in order to meet all the requirements of point 4.2.1 of the PRM TSI. Apart from that, it is planned to meet some requirements on additional 58 stops and stations.

Table 2 Condition of stops and stations based on the 4.2.1 point of PRM TSI, [14]

State of stations and stops	All requirements fulfilled	More than 80 % requirements fulfilled	50 % requirements fulfilled	Requirements slightly met
No. of stops and stations	4	13	18	391

Similarly, the following section 4.2.2 in the PRM TSI defines the requirements for rolling stocks. The results for Croatia are presented in Table 3 where is seen that 73 % of multiple units are not compliant with the TSI for PRM and that will not change in the near future.

Table 3 Passenger rolling stock condition based on the 4.2.2 point of PRM TSI, [14]

Rolling stock	TSI PRM requirements met/planned condition for the 2020	TSI PRM partially met/ planned condition for the 2020	Without PRM TSI requirements/ planned condition for the 2020
Multiple units	24/47	6/6	82/82
Wagons	2/2	/	209/108

Plans for the improvement of the existing infrastructure in the PRM NIP are within reconstruction and modernization projects of Croatian railway network in general, and without a systematic approach in targeted removal of accessibility barriers. Since the directions discussed here are currently part of the projects in the stages of preparation of study documentation and design, and due to the nature of the following stages, their implementation cannot be expected in the next few years. The authors of this paper believe that more urgent actions are necessary in order to improve access for the PRM. The improvement of the basic elements on the infrastructure subsystem should not be linked to the projects to be funded in the next 10 to 15 years. Since 30 % of all transported passengers by rail are in City of Zagreb [9], infrastructure manager together with the city government should pay more attention to the real needs of users and consider numerous recommendations from a European and national basis to increase access to the rail transport for all groups of users, including the PRM. This would set a high scale for the quality of public transport in the Croatian capital.

At the same time as increasing the accessibility of infrastructure, railway operator should continue to work on renewal of the available rolling stock and introduction of low-floor trains structurally adapted to all user groups. This refers not only to the fleet used in the urban area discussed in this article, but also to the rest of the network where the classic train compositions are still primarily used, which makes the access conditions to PRM even more difficult. Therefore, it is important to introduce at least one pair of low-floor train on regional and local lines. On the lines where that will not be possible, due to the number of available trains, operator and station manager must ensure access for the PRM using other resources like mobile ramps.

5 Conclusion

The analysis of the accessibility of urban railway transport to the PRM showed the poor condition of the identified elements of accessibility of stations and stops in the area of the Croatian capital. A large number of them are not adequately equipped at all, or if certain elements do exist, it is likely that they are not in function, making them direct obstacles to the accessibility of rail to the PRM. An additional problem in the organization and realization of the transport process is the unfit interior of a large number of trains, and for access to which

PRMs must be announced 2 days in advance if want help. This despite financial benefits significantly limits their more frequent use. The recommendation is the joint implementation of existing national plans to increase accessibility for people with reduced mobility in terms of infrastructure and vehicles so that the entire rail transport service is barrier-free for this and all other categories of users. This analysis covered a small proportion of total rail network in Croatia, and future work should consider the accessibility of rail transport on the entire network in order to obtain a complete state and make more effective recommendations.

References

[1] Gutierrez, J., Monzon, A., Pinero, J.M.: Accessibility, network efficiency, and transport infrastructure planning, Environment and Planning, 30 (1998), pp. 1337-1350

[2] Morris, J.P., Dumble, P.L., Wigan, M.R.: Accessibility indicatoras for Transport Planning, Transportation Research Part A, 13A (1979), pp. 91-109

[3] Litman, T.: Accessibility for Transportation Planning - Measuring People's Ability to Reach Desired Goods and Activities, Victoria Transport Policy Institute, 2017.

[4] Rietveld, P., Bruinsma, F.: Is Transport Infrastructure Effective?, Springer, 1998.

[5] Mamun, S.A., Lownes, N.E.: A composit index of public transit accessibility, Journal of Public Transportation, 14 (2011) 2, pp. 69 – 87

[6] Gould, P. R.: Spatial diffusion, Resource paper no. 4, Commission on College Geography, Association of American Geographers, Washington, DC, 1969.

[7] Poelman, H., Dijkstra, L.: Measuring access to public transport in European cities, EC, 2015.

[8] Republika Hrvatska: Strategija prometnog razvoja Republike Hrvatske (2017. -2030.), Ministarstvo mora, prometa i infrastrukture, 2017.

[9] HŽ Putnički prijevoz d.o.o.: Statistika za 2018. godinu, Zagreb, 2019.

[10] HŽ Infrastruktura d.o.o.: Vozni red 2019/2020, Zagreb, 2020.

[11] HŽ Putnički prijevoz d.o.o., http://www.hzpp.hr/pogodnosti?m=453&mp=321, 25.05.2020.

[12] EU: Uredba 1371/2007 Europskog parlamenta i Vijeća od 23. listopada 2007. o pravima i obvezama putnika u željezničkom prometu, 2007.

[13] EU: Uredba Komisije 1300/2014 od 18. studenoga 2014. o Tehničkoj specifikaciji za interoperabilnost u vezi s pristupačnošću željezničkog sustava Unije osobama s invaliditetom i osobama s ograničenom pokretljivošću, 2014.

[14] Republika Hrvatska: Nacionalni provedbeni plan za Uredbu Komisije 1300/2014 od 18. studenoga 2014. o tehničkoj specifikaciji za interoperabilnost u vezi s pristupačnošću željezničkog sustava Unije osobama s invaliditetom i osobama s ograničenom pokretljivošću, Zagreb, 2017.

VEHICLE WHEEL LOAD ESTIMATION WITH FIBER OPTICAL CONTACT PATCH ELONGATION MEASUREMENT

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Abstract

Load estimation of wheels, especially for heavy vehicles, is of importance for several reasons. First safety imposes to respect loading limits for a given tire, but the variety of road infrastructures or bridges passed by a vehicle are defining constraints of larger scales as structure resistance or pavement durability. Moreover, multiple-wheels load estimation may be an efficient verification mean of the loading uniformity of goods inside a heavy vehicle.
All these reasons are justifying the interest for a continuous estimation of load for each wheel. In this context, this work aims at contributing to the development of an intelligent tire solution, able to estimate the loading applied on a wheel from the elongation measurement of the tire-to-road contact patch. As a first step of proof of concept, without regarding durability, this measurement has been done with a tire instrumented with a longitudinal, circumferential optical fiber. Measurement on a static test wheel has shown the relevance of the method to detect slight elongation of the contact patch, surrounded by compression of nearby tire areas. The Distributed Optic Fiber (DOF) measurement, widely used in the structural health monitoring domain (SHM), has been related to the force applied to the wheel, by a near linear relation, on the experienced domain of 70 mm to 110 mm for the contact length and 1.1 to 2.6 kN for the vertically applied force. As a result, demonstration is done that an intelligent tire could provide a relevant information on a given wheel load of a vehicle. The optimization of the experimental setup should lead to a robust system, usable continuously on heavy vehicles, to detect harmful loading displacements or to qualify adequacy between vehicle load and road infrastructure capacity.

Keywords: load estimation, force estimation, fiber optic, intelligent tire

1 Introduction

Knowledge of a vehicle loading is of importance for both road managers and vehicle users. Vehicle weighting and even weigh-in-motion techniques are therefore widely used and studied in this double aim. According to the cases, the loading evaluation could involve vehicle or infrastructure-based systems.
First of all, at the loading phase, the vehicle user may want to evaluate the load in order to be sure that the vehicle loading limit is respected or evaluate the quantity of raw goods sold or bought. Complementary, the infrastructure managers may want to evaluate the vehicle loads for a given road section or structure, for durability consideration or structural resistance.

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Infrastructure-based vehicle weighting can be achieved by large scale systems, as instrumented bridges [1], with the use of concrete embedded strain transducers. In this case the B-WIM (bridge weigh-in-motion) was worked out successfully in order to evaluate ballast instability, since the bridge was hosting a railway, but the same approach is usable to monitor heavy vehicle traffic.
Weighting is useful to anticipate long-term fatigue of roads too. Rutting induced by loads applied to flexible pavement has been quantified with accelerated pavement testing facility (APTF) in [2]. Repeated loads on regular flexible pavement are then prone to generate rutting, and lead to structural rupture or vehicle aquaplaning in adverse weather conditions. Thus, vehicle loading control is an important parameter facing to the structure resistance.
Enforcement of overloaded commercial vehicles is an everyday application for weigh-in-motion. A sensitivity study has been realized in this aim [3], with piezo-ceramic and piezo-quartz sensors embedded in the pavement structure of a circular test track. Motivations were infrastructure premature deterioration, road safety and unfair competition between transport operators or modes. A precision of 10 % on axle loads has been reached, linked to metrological and trajectory parameters (dynamical loading effects are of importance too).
Road wear is another motivation to study loads of freight vehicles. In [4] it is suggested to set up infrastructure pricing to compensate the marginal cost of the road wear. Transport companies may accept higher road pricing for higher allowed axle loads and reduced transport cost, justified by higher road maintenance costs.
Load distribution at the contact patch can be achieved with a surface sensor. In [5] a multi-digit sensor has been used to measure the load and its spatial distribution (Fig. 1a and 1b). The advantage over embedded sensors is to avoid variation of road structural behavior, but there is a high constraint on wheel trajectory. Another infrastructure-related mean to measure dynamical loading has been developed and compared to a vehicle dynamometrical wheel. The Mevi system [6] provided high speed measurements, of a comparable precision with the Kistler wheel, but with the advantage to concern a large variety of vehicles. 3D forces are giving both vertical loading and rolling/braking forces (Fig. 2).

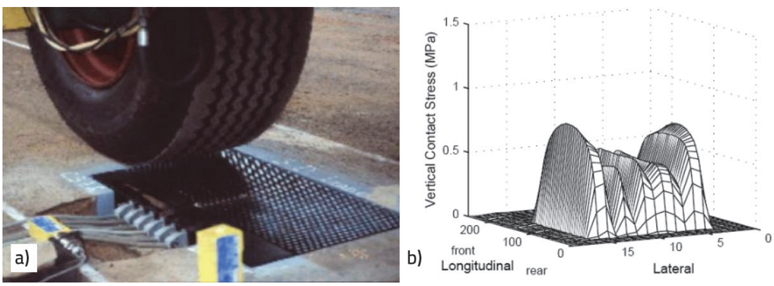


Figure 1 a) Set of instrumented tips; b) Distribution of contact pressures measured under a tire by this experimental assembly [5]

Figure 2 The Kistler wheel on the MEVI system [6]

Driving path and dynamical variations of forces are justifying the investigation of vehicle-based loading measurements. Such measurements could be deployed willingly by vehicle users for goods quantification or vehicle resistance, or be imposed by the infrastructure managers for pricing settlement.
Load estimation can be worked out by instrumented dampers and identification methodologies, as in [7] where low-cost sensor signals (two accelerometers for sprung/unsprung masses) are inputs for identification purpose with real-time observers, or even by direct wheel bearing instrumentation ([8], Fig. 3). Nevertheless by these methods the load estimation is subject to attenuation and vibratory shifts by joints and tire, and ideally measurement should be done as possibly close to the contact patch to avoid damping and resonance effects of intermediate system elements.

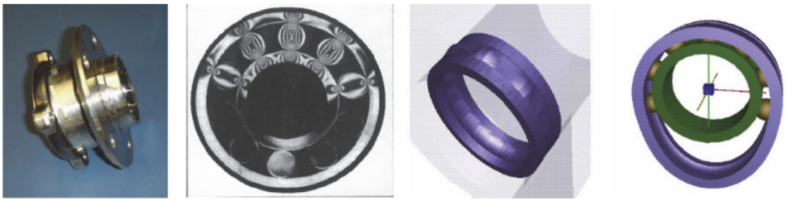


Figure 3 Dynamometer wheel bearing (SKF-TNO) [8]

At the immediate proximity of the contact patch, intelligent tire measurement solutions are involving embedded magnet and Hall effect [9] or saw sensors for tread element flexion monitoring [10] (respectively Fig. 4a and 4b).
In this context, the variation in length of the contact patch is studied in this work, as an indicator of the applied load on the wheel. Indeed, tires are composed of rubber, fibers, steel wires, and the tire belt is often modeled by an equivalent cord model [11]. Individual materials are leading to various elongation to tensile force ratios, with hysteresis loops [12] but due to a large steel wire volume fraction, the equivalent cord can be assimilated to a solid steel wire [12]. A linear fit of such a solid steel wire is given in Fig. 6. Outside the contact patch, the tire cord is subject to compression, and therefore inside the contact patch, the cord model is subject to elongation. Then, despite the fact that loading has been often evaluated with the tire vertical deformation [13], this work is centered on the elongation evaluation according to loading variations of the tire (fig 5a and 5b). The instrumentation is expected to be simpler and less dependent to the tire pressure. The estimation principle and experimental setup are described in the following.

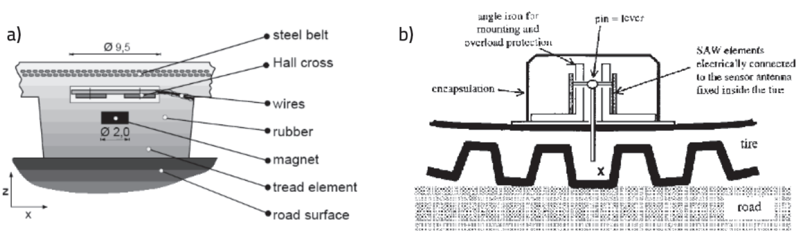


Figure 4 Instrumented tread block: a) Hall effect [9], b) SAW sensors [10]

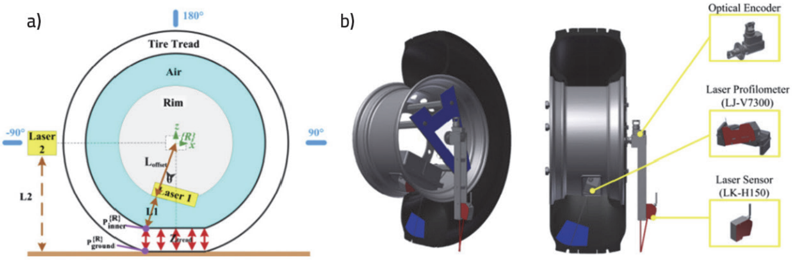
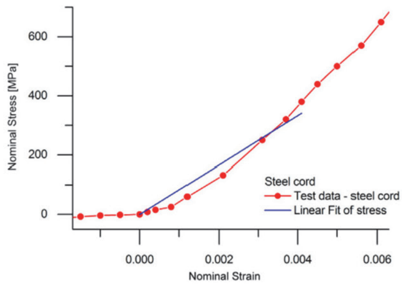


Figure 5 a) Tire deformation system; b) Prototype [13]



Linear parameters (y=ax+b) [MPa] and goodness of fit	
a	0
b	83209.40171
adjusted R^2	0.97543

Figure 6 Stress/strain linear fit (through 0/0); tire belt as a steel element [12]

2 Experimental setup

In the aim of wheel load evaluation, various loadings have been applied and the elongation of the belt in contact with the soil has been evaluated (Fig. 7). Indeed, as schematized on Fig. 8a and 8b, and while considering preceding works [11, 12], elongation of the equivalent steel cord should be in relation to the loading. This parameter has been chosen particularly for its presumable relevance, the measurement being very close to the tire-to-road contact patch. To do so, an optical fiber has been fixed inside the tire, with the help of a non-rigid epoxy-based glue, in the center of the tire belt (Fig. 7b and 7c). The loading is performed by means of a vertical press (Fig. 7a). The applied step-wise increasing force is recorded by a weighting sensor placed under the tire.

Stress is recorded all along the tire belt by means of a fiber optical method, described in the next section. Fiber sensor data are sampled at the frequency of 10 kHz. The tire loading is continuously measured and the steadiness of the temperature is monitored. The reference Rayleigh scatter frequency profile, used by the method, is established with an unloaded tire inflated at 2 bars.

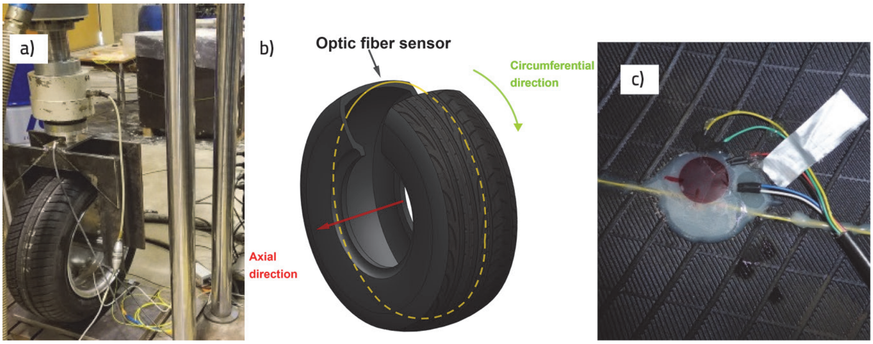


Figure 7 Instrumented tire placed in the testing frame (hydraulic loading system); b. Placement of the optical fiber in the tire; c. Detail of the rosette strain gauge beside the glued optical fiber (yellow).

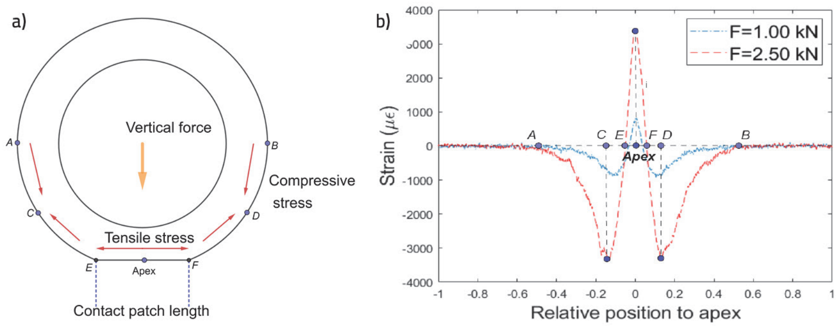


Figure 8 a. Compression and tensile stresses; b. Example of strain measured over the full belt, two loading cases (same reference points)

3 Fiber optical measurement principle

Depth of focus (DOF) Rayleigh sensing is widely used for structural health monitoring [14] and it is involving swept-wavelength interferometry [15]. The fiber index of refraction is locally altered by surrounding strain and temperature variations. Fluctuation of this index induce signal losses by backscattering, namely, the Rayleigh scattering phenomenon. Practically the local strain or temperature variations are determined by comparing the stressful scatter frequency profile to the stressless profile, or the reference profile. The spectrum frequency shift can be expressed in function of temperature and strain variations:

$$\Delta v / v = KT \cdot \Delta \mathrm{T} + K\varepsilon \cdot \Delta\varepsilon \tag{1}$$

with $\Delta v/v$ the frequency shift to the frequency v (Hz), ΔT and $\Delta \varepsilon$ respectively the temperature (°K) and the strain variations, KT and Kε proportional constants. At a constant temperature the strain can be directly computed.

The DOF method allows to locally evaluate the strain at each spatial pitch of the optical fiber, down to pitches of 0.65mm. Up to 1500 strain values can be recorded all along a 1 meter long fiber.

4 Results

Loadings have been applied to the center of the instrumented wheel with an electro-hydrostatic actuator. Four steps of increasing loading have been applied, each with a duration of 50 seconds (Fig. 9). The high loading speed of 0,14 kN/s generates peaking and relaxation values of the contact length at the establishment of each step (Fig. 9). This behavior is due to the loading speed and to the visco-elastic properties of the tire. It results in a small bounce followed by a steady section. In real-world situations, wheel loading could show rapid variations and identification of steady measurement samples is necessary.
Moreover, it can be seen on the third and fourth loading steps that these steps are divided into two “steady” sections for the applied force (ranges of 160-180-200 s for the third loading step and 210-220-270 s for the fourth loading step). This is an actuator issue, which is following a force target and occasionally switched between two close targets. New ongoing experiments would withdraw this issue by using another loading device.
Nonetheless, by using steady parts of inner contact length in the rolling direction and vertical force measurements presented in Fig. 9, representatives mean values can be established (Table 1.). The applied force can be expressed in function of the inner contact length by the mean of a simple linear regression:

$$F_v = a + b \cdot L_c \tag{2}$$

with F_v the applied vertical force (kN), L_c the length of contact (mm).
Computed regression coefficients are: a = 0.040 ; b = – 1.827.
The value of the associated coefficient of determination, r^2, is of 0.998.

There is a strong linear correlation between the applied force and the contatc patch length, measured by an optical fiber, as it has been already verified with a simplified model (Fig. 6, [12]). This correlation is obtained over a large variation of the applied force of 1:2.29 and a large variation of the contact length of 1:1.48. However new experiments are planned with a more progressive loading.

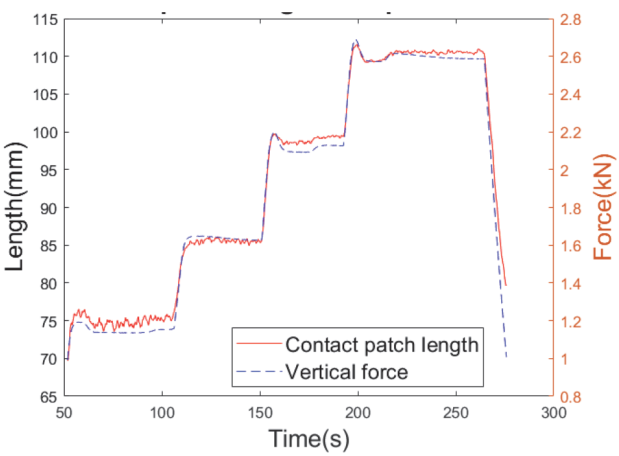


Figure 9 Correlation of the contact patch length to the wheel load

Table 1 Contact length and applied force (mean values).

Loading case	Mean value: Contact length [mm]	Mean value: Applied force [kN]
1	74.6	1.1367
2	85.6	1.6374
3	99.3	2.1200
4	110.5	2.5977

5 Conclusion

Measurement of wheel loading is of importance, to ensure the safe usage of tires or infrastructures. After having reviewed the existing methods both infrastructure-based and vehicle-embedded, this work demonstrates the possibility to use Distributed Optic Fiber (DOF) measurement for wheel loading evaluation.

This measurement has been done with a longitudinal, circumferential optical fiber, glued on the inner side of a tire belt. Measurements on a non-rotating test wheel have shown the relevance of the method to detect slight elongation of the contact patch, surrounded by compression of nearby tire areas. The measured elongation has been found to be related to the force applied to the wheel by a near linear relation, on the experienced domain of: 70mm to 110mm for the contact length and 1.1 to 2.6 kN for the vertically applied force.

Intelligent tire could then provide relevant information on a wheel load, and at the closest location to the tire-to-road patch, which minimizes intermediate materials as dampers or even tire sidewalls. The durability of the optical fiber inside a rolling tire is an issue, but a short fiber, of about 1cm, could be used without material fatigue and it is an ongoing work.

References

[1] Karoumi, R., Wiberg, J., Liljencrantz, A.: Monitoring traffic loads and dynamic effects using an instrumented railway bridge, Engineering structures, 27 (2005) 12, pp. 1813-1819

[2] Khan, S., Nagabhushana, M.N., Tiwari, D., Jain, P.K.: Comparison of Uni and Bi-directional Load Induced Rutting in Flexible Pavement with Accelerated Pavement Testing Facility (APTF), Transportation Research Procedia, 17 (2016), pp. 521-528

[3] Jacob, B., Cottineau, L.M.: Weigh-in-motion for Direct Enforcement of Overloaded Commercial Vehicles, Transportation Research Procedia, 14 (2016), pp. 1413-1422

[4] Martin, T.C., Thoresen, T.R.: Estimation of the marginal cost of road wear as a basis for charging freight vehicles, Research in Transportation Economics, 49 (2015), pp. 55-64, doi: https://doi.org/10.1016/j.retrec.2015.04.006

[5] De Beer, M., Fisher, C., Jooste, F.J.: Evaluation of non-uniform tyre contact stresses on thin asphalt pavements, ISAP2002, Copenhagen, Denmark, 2002.

[6] Coiret, A., Orfila, O.: Concurrent evaluation of the tire/pavement contact torsor by means of a dynamometer wheel and a road system, BLPC, 273 (2008), pp. 133-145

[7] Pham P.T, Sename, O., Dugard, L.: Comparative study of three robust observers for automotive damper force estimation, 8th International Conference on Mechatronics and Control Engineering, Paris, France, 23-25.7.2019, doi: 10.1088/1757-899X/707/1/012014

[8] Zuurbier, J., Van Leeuwen, B.: Vehicle dynamics control based on force-sensing wheel bearings, Vehicle dynamics 2007, Stuttgart, 8-10.5.2007.

[9] Yilmazoglu, O., Brandt, M., Sigmund, J., Genc, E.: Integrated InAs/GaSb 3D magnetic field sensors, Sensors and Actuators A Physical, 94 (2001) 1-2, pp. 59-63, doi: 10.1016/S0924-4247(01)00700-2

[10] Pohl, A., Steindl, R., Reindl, L.: The “intelligent tire” utilizing passive SAW sensors-Measurement of tire friction, IEEE Transactions on Instrumentation and Measurement, 48 (1999) 6, pp. 1041–1046, doi: 10.1109/19.816111

[11] Cho, J., Lee, S., Jeong, H.Y.: Finite element analysis of a tire using an equivalent cord model, Finite Elements in Analysis and Design, 105 (2015), pp. 26-32, doi: 10.1016/j.finel.2015.06.009

[12] Korunović, N., Fragassa, C., Marinković, D., Vitković, N., Trajanović, M.: Performance evaluation of cord material models applied to structural analysis of tires, Composite Structures, 224 (2019) 15, doi: https://doi.org/10.1016/j.compstruct.2019.111006

[13] Xiong, Y., Tuononen, A.: Rolling deformation of truck tires: Measurement and analysis using a tire sensing approach, Journal of Terramechanics, 61 (2015), pp. 33-42, doi: https://doi.org/10.1016/j.jterra.2015.07.004

[14] Bassil, A., Wang, X., Chapeleau, X., Niederleithinger, E., Abraham, O., Leduc, D.: Distributed Fiber Optics Sensing and Coda Wave Interferometry Techniques for Damage Monitoring in Concrete Structures, Sensors, 19 (2019) 2, doi: 10.3390/s19020356

[15] Froggatt, M., Moore, J.: High-spatial-resolution distributed strain measurement in optical fiber with Rayleigh scatter, Applied Optics, 37 (1998) 10, pp. 1735-1740, doi: https://doi.org/10.1364/AO.37.001735

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HIGHER AUTOMATION - METHODS TO INCREASE ENERGY EFFICIENCY IN RAILWAY OPERATION

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Abstract

Automation is already present in many areas of the railway sector (e.g. computer-aided dispatching or electronic interlockings). In order to achieve climate goals and offer an attractive transport service, it is essential to advance automation and higher grades of automation (GoA). The four levels of automation range from supporting systems (GoA1) to automotive trains (GoA4). This paper summarises a study which outlines the impacts, requirements and potentials of higher GoA within different segments: passenger transport, freight and mixed traffic on mainlines and branch lines. The findings show that energy-efficiency and capacity can already be increased with the first two GoA for both, passenger and mixed traffic. Enhancements have an influence on costs, not to mention the customer satisfaction. The potential in freight transport, e.g. in shunting, can be exploited with intelligent freight trains (GoA4). This leads to improved safety and reduced costs. Within this study a tool to calculate energy consumption is established. It enables the depiction of various scenarios for different trains and driving behaviours. The simulation tool is validated by real measured data. The outcome of the calculation tool underpins the benefits of driver advisory systems (DAS) and automatic train operation (ATO). It can be stated that higher automation, especially on a dispositive level is essential if energy and capacity improvement are to be achieved, regardless of the type of network (electrified or non-electrified). However, operational optimisation has its limits. For non-electrified lines, alternative drives offer the opportunity to further mitigate environmental impacts.

Keywords: automatic train operation, energy efficiency, alternative drives, sustainable railways

1 Introduction

European passenger transport has been constantly increasing for the last decade [1]. By contrast, rail freight is stagnating and has come under pressure due to increasing road freight traffic [1]. To achieve climate goals, remain attractive and competitive, the railway sector needs to focus on a higher capacity throughput, cost reduction and also without a doubt on environmental sustainability. Thus, it is essential to systematically apply higher grades of automation (GoA). Moreover, the implementation of alternative propulsion technologies (considering the significant amount of diesel-powered rolling stock worldwide) can help to further reduce the environmental impact and increase energy-efficiency in railway operation. The aim of this paper is to sharpen our understanding of higher automation in railway operation. It investigates legal, operational and technical requirements and analyses the potentials of higher automation within different systems (cf. Section 2). The importance and

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potentials of energy efficient solutions is mirrored by the results of a calculation tool for energy consumption. The paper presents various scenarios for different trains and driving behaviour in Section 3 and gives an insight into alternative propulsion technology in railway operation (cf. Section 4).

2 Higher automation levels in railway operation

2.1 Definition and status quo of higher automation

According to UITP [2] higher automation in railway operation is classified according to four grades of automation (GoA). ATP (automatic train protection) together with DAS (driver advisory systems) are classified as GoA1 and are state-of-theart in railways. ATP is widely used, especially in cases of higher top speeds, and it ensures basic safety (e.g. braking in the event of an emergency). DAS provides the driver with a speed profile in order to arrive on time or to save energy. Automatic train operation (ATO) is considered a subsystem with different functions depending on the GoA and must be combined with ATP to ensure safety. GoA2 combines ATP and ATO, where ATO executes traction and brake commands. Much effort is currently put in field trials for GoA2, albeit existing examples of GoA2 can also be found, such as the Thameslink project in London [3]. In GoA3 the train runs automatically, whereas there is still a train attendant on board to respond in case of a disruptive event. GoA4 corresponds to fully automatically run vehicles without a human railway employee on board. Until now GoA4 has only been applied in urban metro lines, with the exception of Rio Tinto heavy haul freight trains in Australia [4].

2.2 Requirements for higher grades of automation

Railways can be divided into three basic components: infrastructure, vehicle and operation. Norms, rules and regulations ensure secure interaction. It follows that higher automation not only requires a legal and normative framework (safety, security, certification etc.) but also has operational and technical boundary conditions (e.g. specific trackside and trainborne equipment). Since ATP is a safety requirement of GoA2 and to ensure interoperability, many institutions and suppliers support the idea of ATO over ETCS. Efforts are currently being made to incorporate new specifications for GoA1 and 2 in the TSI [5]. Furthermore, adaptions in national legislation, liability issues (of trial runs), certification issues and harmonised authorization processes all need to be considered and solved. To ensure the safe guidance of a train a continuous ATP must be implemented and continuous information, usually known to the driver, needs to be submitted to the ATO. In Europe ETCS Level 2 is regarded as the basis for ATO. However, the current infrastructure and slow migration process of ETCS makes the use of a harmonised, sophisticated ATP unrealistic. ATP solutions based on satellites should thus be examined together with migration concepts in case of ATP other than ETCS [6]. In order to increase energy efficiency and punctuality ATO must be combined with DAS providing an optimised speed profile for one train. To optimise train movements throughout an entire network, ATO must be connected to a cross-network traffic management system (TMS). This implies adapting trajectories continuously to the current traffic to avoid unnecessary stops, reactionary delays or conflicts. One approach is known as dynamic capacity optimisation: it is based on an automatically computed timetable in real-time combined with ATO and can reduce headways (90-100 sec.) [7]. Technical equipment at wayside and trainborne level will need to be adjusted depending on the GoA. As of GoA3, the train must take over the driver’s visual functions. For wayside obstacle detection, solutions stem from drone-based cameras to fibre optic sensing [8]. The installation of laser or radar sensors combined with image processing at level crossings or fences at platforms are conceivable solutions [9]. As for onboard obstacle detection the combined installation of radar, infrared, laser or cameras is suggested because of different characteristics in reach and also dependence on the weather [10].

2.3 Benefits of higher grades of automation

Different systems can benefit from increasing automation according to their boundary conditions. Capacity problems are particularly prevalent in passenger transport, especially on mainlines. Solutions as of GoA2 in connection with TMS show great potential in passenger transport and mixed traffic for coping with peak demand in hubs [7]. The need for additional infrastructure (as of GoA3) could therefore be replaced by means of a dispositive level. Comfort can already be achieved as of GoA2, since ATO can balance e.g. aggressive styles of driving. The use of TMS reduces waiting time, increases reliability and punctuality, which has added value for both, freight and passenger transport. In a first step this can already be achieved to a certain degree with DAS. Introducing TMS plus fully automatically run vehicles on branch lines could bring about a cost-effective and demand-based transport service [11]. There is a common understanding that safety increases by taking out the human factor. However, as of GoA3, risks caused by new tech-nologies in terms of cyber security, failures of providers, manufacturers or systems must all in sum be of a lesser character than the human-risk factor. Recently developed “intelligent” vehicles (equipped with a centre buffer coupling and able to perform an automatic brake test) could replace the remaining manual work of coupling processes [12]. Safety in shunting could thus be increased, in particular in the context of the high risk of accidents in this area. A useful way to save energy is to exploit the acceleration, cruising, coasting and braking phase in a more energy efficient manner. In order to show the potential of energy savings due to different driving behaviour and scenarios, a calculation tool was established (refer to Section 3). Energy efficient driving could reduce energy costs by 10 % on average for one train in one year [13]. This could in particular increase the competitiveness of freight traffic. While cost cuts by replacing drivers is a double-sided issue, the economic benefit in shunting is certain (decrease in manual labour) [11].

3 Energy consumption simulation tool

The original calculation tool used in this study (developed in Microsoft Excel by Messner [14], elaborated by [11]) was further improved. Energy consumption can be computed for different driving behaviours and scenarios for various types of rolling stock. The model is based on the total train resistance which occurs in the course of a train journey on a random route and can be expressed in energy needed for that section (cf. Fig. 1). Energy consumed by auxiliary functions is also considered. In order to validate the simulation tool, energy consumption of specific trains and scenarios is compared to real measured data (average deviation of 7 %).

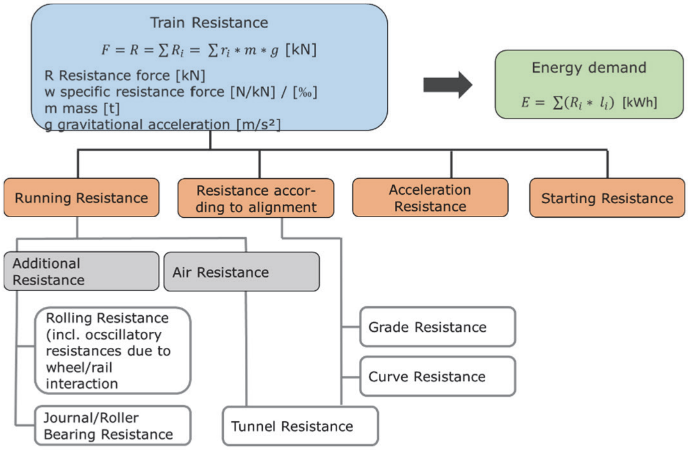

Figure 1 Mathematical model to calculate energy consumption based on [14]

3.1 Results of different driving scenarios for passenger and freight trains

Different driving scenarios and behaviours are computed for a section on an Austrian main-line (approx. 50 km) considering five train types: a long-distance, a regional, a suburban, a light and a heavy (5 and 22 tonnes axle load) freight train. Due to a homogeneous topography (gentle incline) in the chosen section energy recovery remains unconsidered. Fig. 2 and Fig. 3 depict an excerpt of the results.
Case 1 (maximum top speed) demonstrates a tight speed profile which might counteract a reactionary delay, in the event of accumulated delays originating from the point of departure. Stopping points are chosen according to the train type (e.g. the fact that freight trains are often being put aside in railway operation to ensure capacity throughput of passenger transport is also taken into account). Case 2 (no stops) eliminates the stops along the route. Although non-realistic, it serves as a pure comparison between the different train types. Apart from minor differences (e.g. other possible speed limits due to vehicle characteristics), they share the same conditions (i.e. no stops along the route). Case 3 (top speed reduction) shows energy efficient driving for passenger trains by limiting the top speed and yet arriving on time. Buffer time (10 % for long-distance and 5 % for the other two passenger trains) is not reduced. Stopping points are the same as in case 1.

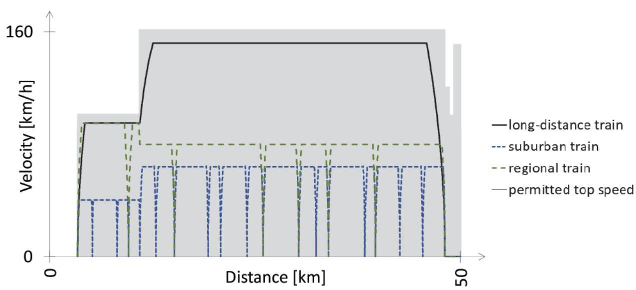

Figure 2 Path-time diagram for passenger trains case 3 (top speed reduction)

The ratio of the train resistances (see Fig. 3) shows that most of the energy con-sumed is related to acceleration resistance. A significant top speed reduction can save up to around 50 % of energy compared to a tight speed profile. The decrease in energy consumption gen-

erates a 35 % higher running time which highlights that the degree of energy reduction is not wedded to the degree of increase in travel time. The results must be treated with caution to a certain extent, since in reality a train will not accelerate up to the permitted top speed every time before a stop. It can be assumed that the energy savings will be lower and correspond more fully to those that can be found in the literature, e.g. [15]. Furthermore, the energy efficient driving profiles only concern the optimisation of one train. It must be assured that the optimisation of one train does not affect other trains negatively. An appropriate TMS is therefore necessary to save energy on a network-wide level. Results of freight trains confirm the need of an improved traffic management. Around 35 % of energy could be saved on the investigated route without putting freight trains aside. Running resistance increases with higher velocities (derived from the fact that air resistance is part of running resistance) compared to other resistances. The long-distance train confirms this fact. Whilst the other two passenger trains have a lot of retardations due to stopping more often, the long-distance train remains in a cruising phase (with a constant, high velocity) for much longer. Train mass influences all resistance forces. The long-distance train is around three times heavier than the regional and suburban train. This is why resistance according to alignment and running resistance (apart from the air and oscillatory resistance which are both dependent on speed) are proportionally higher in the long-distance train. Case 2 highlights the influence of the train mass. The results also show that auxiliary functions consume one sixth of the total energy requirement implying the need for improvements in vehicle technology.
A detailed analysis was performed for long-distance trains. Additional use cases were added, namely a best-case scenario (with coasting phase) and a worst-case scenario (two unexpected stops). The results show the significantly lower energy consumption in the best-case scenario with maximum coasting. By contrast, twice the energy is required in the worst-case scenario (on the route studied). This underlines the effect of conflict management and energy-efficient speed profiles on the energy consumption.

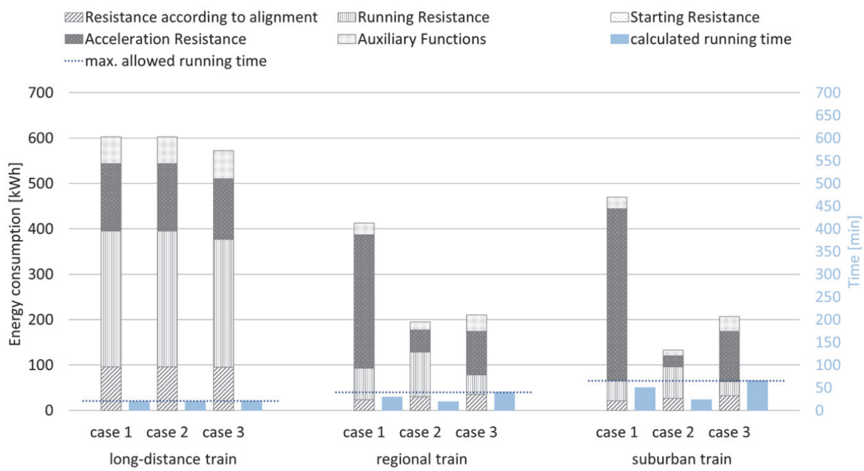

Figure 3 Energy consumption (due to train resistance and auxiliary functions) for different passenger trains and scenarios

4 Alternative drive solutions

It is commonly known that traveling by rail is more eco-friendly compared to other modes of transport. In Austria, a passenger train emits 7,7 grams of CO_2 per passenger kilometre [16]. The European average is 28 gCO_2/pkm [17]. Reasons for this comparatively low value are as follows: the Austrian Federal Railway (OeBB) network has a high electrification rate of 70 % [18] and OeBB uses 100 % green electricity for railway operation [19]. On the global scale, railway operation is not as sustainable as it is in Austria. More than 60 % of the world's railway network is not electrified [18] and around 70 % of the world's locomotive fleet operates on non-electrified lines [20]. Although higher automation and optimisation in railway operation allows for energy reduction, non-electrified lines need to focus on addi-tional solutions in order to cut emissions. Electrification is not always viable (n)or technically feasible. Thus, alternative drives are considered a sustainable solution, e.g. less direct emissions, less noise pollution and higher efficiency. This concerns segments which tend to be operated with diesel, like branch lines, shunting and freight corridors. Rail traffic on the American continent, in Africa and Australia still relies heavily on diesel traction, which are regions with relevant freight activities. Alternative propulsion technologies in railways are at an early stage of technology development. They currently account "for less than 2 % of all orders" [21]. Available technology, the lack of infrastructure (e.g. recharging, refuelling) and higher costs due to scalability are barriers for the market uptake of alternative drives. Experts expect a "stable market for the next 10-15 years" due to electrification projects, cleaner diesel technologies (considering stage V engine technology being offered on the market) and the lack of incentives in various countries [21]. However, emission targets in rail could change tender conditions in the future. Some European governments or railways have explicit decarbonisation strategies [21],[22],[23]. Most of these aim at the substitution of diesel traction within the next 15-20 years. On-going activities also show a trend towards alternative solutions in the aforementioned segments. The multiple unit segment proves to be the most mature segment for alternative drives. Battery electric multiple units or hydrogen fuel cell trains are currently being implemented and tested on branch lines, especially in Europe [21]. Gas-powered solutions (LNG, CNG) are under consideration and going through trials in North America [21] and in Eastern European countries, notably Russia [24]. It is noteworthy, that the environmental impact of battery and hydrogen applications depends on the electricity mix, the production process and end-of-life of their components. If used with green electricity, they are considered to have a high potential to mitigate environmental impacts for non-electrified lines.

5 Conclusion

The results of the calculation tool for different scenarios highlight the importance of energy efficient driving and conflict management. In reality, DAS (GoA1) can already achieve energy savings by providing energy efficient speed profiles. In a next step, speed profiles could be executed by ATO in GoA2 more precisely. ATO in combination with a traffic management system could prevent conflicts on a network-wide level. Energy savings, along with capacity improvement, increased punctuality and cost-effective offers bring added value to the customer and boost the railway sector. In cases of passenger and mixed traffic operations, energy savings and capacity increases can already be achieved with GoA1 and 2. Nevertheless, technical requirements (e.g. continuous signalling systems, obstacle detection) as well as the lack of regulations and standards are possible barriers for higher automation. Furthermore, there are limits to operational optimisation. Addi-tional measures are required for reducing environmental impacts. This involves improved vehicle technology to reduce energy consumption of auxiliary functions or the introduction of alternative propulsion technologies on non-electrified lines.

References

[1] Passenger Transport Statistics, Freight Transport Statistics – modal split (Eurostat), https://ec.europa.eu/eurostat/statistics-explained/index.php/Main_Page, 15.02.2021.

[2] Press Kit Metro Automation Facts, Figures and Trends (UITP), https://docplayer.net/26241614-What-is-metro-automation.html, 15.02.2021.

[3] Thameslink first with ATO over ETCS. (N.N), http://www.railwaygazette.com/news/traction-rolling-stock/single-view/view/thameslink-first-with-ato-over-etcs.html, 15.02.2021.

[4] Smith, K.: Rise of the machines. Rio Tinto breaks new ground with AutoHaul, International Railway Journal (IRJ), pp. 14-18, 2019.

[5] Treydel, R.: Development of the harmonised European specifications for mainline ATO, Railtech, Vienna, Austria, 28 November 2017.

[6] Trinckauf, J.: Der Bahnbetrieb auf dem Weg zur Digitalisierung und Automatisierung, Deine Bahn, 09 (2017), pp. 7–9

[7] Weidmann, U., Laumanns, M., Montigel, M., Rao, X.: Dynamische Kapazitätsoptimierung durch Automatisierung des Bahnbetriebs, Eisenbahn-Revue, 12 (2014), pp. 606-611

[8] Vidovic, I.: Railway Infrastructure Condition Monitoring and Asset Management –The Case of Fibre Optic Sensing, Railway Research Volume 5, TU Graz, 2020.

[9] Pachl, J.: Betriebliche Randbedingungen für autonomes Fahren auf der Schiene, Deine Bahn, 9 (2017), pp. 11–19

[10] Gebauer, O., Pree, W., Stadlmann, B.: Autonomously driving trains on open tracks - concepts, system architecture and implementation aspects, IT - Information Technology, 6 (2012), pp. 266 279, DOI: 10.1524/itit.2012.0689.

[11] Zeiner, M.: ATO (Automatic Train Operation) - Optimierung durch erhöhte Automatisierung auf der Vollbahn, Master's thesis, TU Graz, Austria, 2018.

[12] Müller-Hildebrand, B.: Automatisierung und Digitalisierung im Schienengüterverkehr, ZEVrail, 08 (2017), pp. 301–303

[13] Winter, J., Lindemann, M., Schlegel, S., Kloos, H.: Fahrerassistenz-System, Signal + Draht, 10 (2009), pp. 6–14

[14] Messner, M.: Berechnung des Energieverbrauchs für Triebfahrzeuge, Master's thesis, TU Graz, Austria, 2014.

[15] Lehnert, M., Ricci, S.: Rail Operations and Energy Management (Chapter 2), Sustainable Rail Transport, Lecture Notes in Mobility, ed. Marinov, pp. 7-30, 2018.

[16] Emissionskennzahlen (Umweltbundesamt) https://www.umweltbundesamt.at/fileadmin/site/themen/mobilitaet/daten/ekz_pkm_tkm_verkehrsmittel.pdf, 15.02.2021.

[17] Energy efficiency and specific CO2 emissions (European Environmental Agency), https://www.eea.europa.eu/data-and-maps/indicators/energy-efficiency-and-specific-co2-emissions/energy-efficiency-and-specific-co2-9, 15.02.2021.

[18] Railway Statistics – Synopsis (UIC), https://uic.org/IMG/pdf/uic-statistics-synopsis-2020.pdf, 15.02.2021.

[19] Nachhaltigkeitsbericht 2019 (ÖBB Holding AG), https://presse.oebb.at/dam/jcr:68768d3c-28f1-4b60-9df0-a5b12e9896a9/Nachhaltigkeitsbericht-2019.pdf., 15.02.2021.

[20] Hydrogen scaling up (Study Task Force of the Hydrogen Council), www.hydrogencouncil.com, 15.02.2021.

[21] Smith, K.: Do Hydrogen and Battery Trains Mean the End for Diesel Traction? International Railway Journal (IRJ), 4 (2020), pp. 18–21

[22] Prießnitz, M., Gerstenmayer, T.: The End of Fossil Fuels ÖBB Personenverkehr AG, The End of Fossil Fuels UIC Workshop Zürich, Zürich, Switzerland, 13 November 2019.

[23] Two Dutch Provinces Want to Shift to Zero-Emission Trains by 2035 (Zasiadko, M.), https://www.railtech.com/rolling-stock/2018/11/06/two-dutch-provinces-want-to-shift-to-zero-emission-trains-by-2035/, 15.02.2021.

[24] Ivanov, B.: About experience history biofuels in Russian Railways and the time of the diesel locomotives, The End of Fossil Fuels, UIC Workshop Zürich, Switzerland, 13 November 2019.

OVERVIEW OF EMERGING ROAD TRAFFIC DATA COLLECTION METHODS

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Abstract

It is a well-known fact that the data on road traffic flow characteristics is essential for sustainable road network management. First road traffic volume counts date back to the 1950s when short-term periodic road traffic counts were carried out in cities worldwide. Manual traffic counting is one of the oldest and most technologically simple methods to obtain data on road traffic volume and its composition. Today, because of the ever-growing road transport demand, it has become clear that the development of Intelligent Transport Systems (ITS) is vital to increase safety and tackle increasing emission and congestion problems. The introduction of ITS highly depends on the quality and quantity of traffic data. Under the growing requirement of long-term traffic flow information, various traffic data collection methods have evolved. They allow systematic recording of the traffic flow volume and composition but also vehicle speed, total gross weight, number of axles, axle load and travel destination. This data, which is collected continuously over longer periods, enables a detailed analysis of traffic flows, and represents the basis for decision making in planning, designing, construction and maintenance of road infrastructure. This paper gives an overview of traditional and emerging traffic data collection methods - both fixed and mobile - and the analysis of the current road traffic data collection methods used on the Croatian road network, in terms of their potential and limitations.

Keywords: road network, traffic data, collection methods

1 Introduction

The need for the new infrastructure planning while maintaining the existing road network originates from the day-to-day population movements that affect the traffic flow. The level of success of the transport system operation depends on the organization and quality of the road network management, which includes collecting historical data on traffic load and traffic flow [1, 2]. There are two principal reasons for road managers to have accurate estimates of road traffic characteristics – to support the funding, and to optimise decision making in terms of directing resources. Increased population migration leads to higher traffic flow densities, and therefore frequent traffic accidents, shorter maintenance intervals, and lower levels of service. To mitigate these events, it is necessary to continuously collect traffic data, at least in specific locations, such as network nodes and identified black spots [3]. Following data can be collected by monitoring the traffic: the number of vehicles, the composition of traffic flow, the direction of movements and vehicle speed, and the distance between the vehicles. By analysing the collected data, it is possible to manage the road infrastructure more rationally and to plan the construction of new roads and their maintenance more efficiently.

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In general, traffic data collection methods can be divided into in-situ and on-board techniques [4]. In-situ techniques are used to obtain traffic data measured using sensors located along the road and are divided into intrusive methods and non-intrusive methods. Intrusive methods include pneumatic road tubes, piezoelectric sensors, inductive loop detectors and magnetic sensors. Non-intrusive methods obtain data by remote sensing using video cameras, manual counts, infrared sensors, microwave radar, laser radar, acoustic tracking systems and surveys [5].
The first film camera recordings of traffic flows were made in the 1930s. This led to a significant development of traffic data collecting methods, and in the 1950s the first continuous traffic monitoring was performed by automatic devices [6, 7, 8, 9]. From the 1970s, due to ever-increasing traffic volumes and the advancement of technology, traffic monitoring has shifted to more modern methods. In the 1990s, under the growing requirement of long-term traffic flow information, the rapid development of Intelligent Transport Systems (ITS) began [10]. In 2010, the framework of the deployment of ITS was defined by the EU directive 2010/40/EU [11]. ITS application improved environmental efficiency and enabled better planning, maintenance, and management of transport systems, efficient transport of passengers and goods, traffic safety, and protection of passengers and cargo, as they provide users accurate and fast information on traffic flow and road conditions. A prerequisite for ITS services is the timely collection of accurate and reliable traffic data [11, 12]. Today this process is assisted by different data collection and processing methods and devices. In ITS, data is collected on-board by using test vehicles, mobile phones, Global Positioning System (GPS) or other sensors in real-time [4].

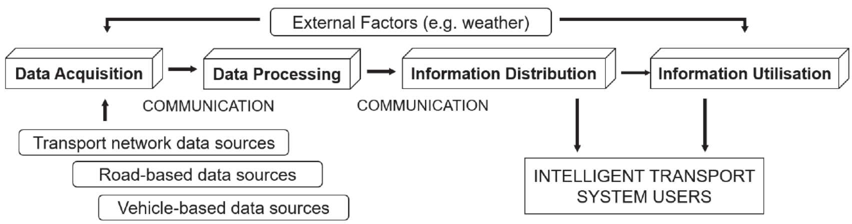

Figure 1 Intelligent Transport Systems information chain (made by Author based on [12])

The choice of appropriate road traffic data collection method depends on the type and the amount of data needed, the time interval in which the data needs to be collected, and the financial means at the disposal. ITS have received a lot of attention in the last decade, especially the issue of vehicle classification [13], which is very important information for numerous road infrastructure design and management activities. For example, the most important information in pavement structures design and maintenance management, besides the traffic volume, is the flow composition (vehicle classification), because the traffic load on the pavement is expressed by the average daily number of equivalent axles [14].
This paper provides an overview of road traffic data collection methods and lists their advantages and disadvantages. An overview of methods in use on the Croatian road network is presented, together with the different vehicle classifications they provide. Here, to present a problem in the determination of the vehicle type for the purpose of road pavement management, a systematic classification of "heavy" vehicles is given, considering their axle load.

2 Review of traffic data collection methods

In this section, six traffic data collection methods - from the oldest manual to the most recent Floating Car Data method - are presented in brief. Based on the performed literature review, the main advantages and disadvantages of each data collection method are identified (Table 1). Manual traffic data collection, performed by traffic meters, is the oldest and simplest method of collecting traffic data [15]. This method provides precise traffic data during peak hours, and it is very appropriate for counting traffic at intersections and for estimations of the average daily and annual values. The measurement itself can be performed on-site or from video, and the vehicle classification is performed based on visual observation of the counter [16].

Automatic traffic data collection is performed to determine the temporal and spatial distribution of traffic characteristics. It can be performed occasionally or continuously. Automatic devices installed on or next to the road allow obtaining data on traffic volume, vehicle speed, wheelbase, weight, and headways, and flow structure [15]. According to the mode of operation and construction, they can be divided into stationary and portable [15,17]: inductive loop, magnetic meter, microwave radar (RTMS), pneumatic meter, active infrared meter, passive infrared meter, ultrasonic meter, acoustic counter, and a video image processor (VIP).

Toll traffic data collection is based on the records of the passage of vehicles using an information card with data on the time of use and passage through the toll station, type of vehicle and location of its entry and exit. A major advantage of this method is the simplicity of determining the number of vehicles at a particular location and the composition of the traffic flow [18].

Computer image processing is becoming the go-to method for defining traffic flow characteristics from video recordings, primarily at multilane intersections and interchanges, as they allow real-time monitoring of speed, volume, queue lengths, and headways. A great advantage of such systems is adaptability to new conditions at the intersection such as the addition of traffic lanes or temporary road closure. Also, it can be used to detect accidents, which allows a shorter time for emergency reactions and provides better traffic flow [19].

Satellite traffic data collection is a method that is still in the experimental phase. New algorithms for vehicle detection are constantly being developed and perfected. To analyse the traffic, a high-resolution satellite is needed. The main limitation of this method is time discrimination because the traffic flow is constantly changing. Here, the advantage is the ability to monitor remote or difficult to access roads [20]. For automated traffic flow tracking, a complex algorithm first needs to segment the satellite image by automatically inserting road edges using vector data. Only then can the algorithm classify objects according to the maximum probability of affiliation. This method still does not replace continuous traffic counting but serves as an additional source of data [21].

Floating Car Data (FCD) method is used to collect real-time traffic data via mobile phones or GPS as sensors [12, 22]. Data such as vehicle location, speed and driving direction are sent to a central information processing centre. Practically every vehicle today has at least one mobile device on-board by which it is possible to collect traffic data or monitor the speed and traffic density. Floating Car Data is key to the further development of ITS [12].

Table 1 Advantages and disadvantages of traffic data collection methods

Method	Advantages	Disadvantages
Manual count	+ noticing traffic anomalies + good monitoring traffic at intersections + easy use of forms for further data processing + low counting costs + counting errors are less than 1% + stopping the survey to record data + a smaller number of counters	- required numerator training - high initial costs (camera acquisition) - infrastructure required to set up the camera - necessary favourable weather conditions - counter fatigue affects data accuracy - transferring data to digital format - classification errors between 4-5%
Automatic count	+ continuous recording of data over a long period + accuracy of data + does not depend on weather conditions + easy installation and removal	- high initial costs (purchase, installation) - required time for assembly and disassembly of the counter - necessary construction works on roads - an inability to monitor traffic at intersections - requires regular maintenance
Toll count	+ high data accuracy + obtaining data through the billing system + low costs of data collection and processing + weather resistance + simple determination of traffic load and traffic flow composition	- limited to infrastructure with tolls - inability to collect additional traffic parameters
Computer vision	+ availability of multiple traffic parameters data collections + the possibility of tracking the vehicle through the license plate + automatic incident detection + traffic violations detection	- high initial costs (camera acquisition) - infrastructure required to set up the camera - ideal weather conditions required - complex image processing algorithms required - large computing and memory resources required
Satellite	+ high geographical coverage	- high initial costs - necessary vehicle contrast - the object´s shadow may prevent the vehicle from being recognized - depends on favourable weather conditions
FCD	+ lower installation and maintenance cost compared to sensor or camera + high geographical coverage + fast setup	- possible bigger time delays of data transfer in case of large traffic volumes

3 The methods used on the Croatian road network

Public roads in Croatia are classified into four groups: highways, state, county, and local roads [23]. Traffic is monitored by three methods: automatic continuous, automatic occasional and toll collection method. From the data given in Table 2, which presents number of cross sections according to the traffic data collection method on the entire classified network, it can be concluded that the data collection requirements are fulfilled with a traffic monitoring system placed, on average, on every 32 km of the network. However, in each method, vehicles are classified differently. Portable meters (PM) used for automatic occasional data collection classify vehicles into five classes according to their length. Stationary meters (SM) used for automatic continuous data collection classify vehicles into nine descriptive groups. In the toll collection (TC) method, vehicles are classified into five groups according to their height, axle number and maximum permissible weight [24].

Table 2 Structure of the classified Croatian road network and number of cross sections according to the traffic data collection method

Road class	Length [10^3 km]	No. of counting cross sections by method		
		Automatic continuous	Automatic occasional	Toll
Highways & State	8.7	264	164	99
County	9.4	205	41	0
Local	8.8	27	45	0
Total	26.9	496	250	99

These differences in vehicle classification can be considered as a hindrance for more efficient management of pavement structures, as they require an extra step (vehicle load re-calculation) in the prediction of pavement maintenance needs. Table 3 shows an overview of heavy commercial vehicles considered as load in pavement structures management, created to further emphasize this issue. Their systematization was made according to the vehicles monitoring method classification, number of axles, maximum length between the axles, maximum weight, and axle load. During the systematization process, a wide range of these values has been observed even for vehicles classified in the same group by a particular monitoring method. The question being raised here is how these disparities affect the tolerable margin of error in the assessment of cumulative pavement structure load during the planning of new, and maintenance of existing road infrastructure.

Table 3 Vehicles systematization

PM	SM	TC	Vehicle description	No. of axles	Max. length [m]	Max. weight [t]	Axle load [t]
3	B3	III.	truck without trailer	2	10.8	32	16
5	C1		bus		18.0	28	14
2	B2		medium trucks		6.6	26	13
5	B5	IV.	semi-trailer truck	3	18.4	40	
3	B3	III.	heavy goods vehicles		10.6	32	11
			truck without trailer		10.8		
5	C1		bus		18.0	28	10
	B5	IV.	semi-trailer truck	4	18.4	40	
	B4		truck with 2-axle trailer		20.5		
1	B1	III.	small trucks, vans	2	5.3	18	9
2	B2		medium trucks	3	6.6	26	
3	B3		heavy goods vehicles	4	10.6	32	8
		IV.	truck without trailer		10.8		
5	B5		semi-trailer truck	5	18.4	40	
	B4		truck with 2-axle trailer		20.5		
	B5		semi-trailer truck + trailer		22.9		
	B4		truck with 3-axle trailer		30.8		
	B5		semi-trailer truck + trailer	6	22.9		7
	B4		truck with 3-axle trailer		30.8		

4 Conclusion

This paper provides an overview of six methods for traffic data collection that are used to determine the traffic volume, vehicle type identification and classification. Based on the performed literature review, the main advantages and disadvantages of each method were identified. The overview showed that each method has certain shortcomings that can affect the tolerable margin of error when determining traffic volume and composition.

The choice of a particular method depends mainly on the financial possibilities of its implementation as well as on the type of data road managers want to obtain. In Croatia, three traffic data collection methods are used, and each method classifies the vehicles differently, depending on vehicle size, weight, purpose and number of axles. The more uniformed systematization of heavy vehicles could optimize the process of management of pavement structures as this process requires the determination of cumulative traffic load expressed by the number of equivalent standard axles.

Modern traffic monitoring methods that could enable more unambiguous vehicle identification should significantly contribute to the optimization of processes that use the composition and volume of traffic flow as their basic input, such as pavement maintenance planning, road traffic noise management, road network compositions and infrastructures upgrades.

References

[1] Habuš, M.: Priručnik za brojanje prometa – terensko istraživanje i anketiranje, Savez za željeznicu, Zagreb, 2016.

[2] Novačić, T.: Aplikacija za višekorisničko brojanje prometa, Diplomski rad, Sveučilište u Zagrebu Fakultet prometnih znanosti, Zagreb, 2017.

[3] Rodwell, M.: Guidelines for Traffic Counting, Road Information Management Systems, Hastings District Council, May 2013.

[4] Perez Martinez, A.: Traffic data collection framework: Freight traffic data in the city of Eindhoven, Technische Universitet Eindhoven University of Technology, 2015.

[5] Leduc, G.: Road Traffic Data: Collection Methods and Applications, Office for Official Publications of the European Communities, 2008.

[6] Dadić, I., Kos, G., Ševrović, M.: Teorija prometnog toka, Sveučilište u Zagrebu Fakultet prometnih znanosti, Zagreb, 2014.

[7] Greenshields, B.D.: The Photographic Method of studying Traffic Behaviour, 13th Annual Meeting of the Highway Research Board, 1933.

[8] Greenshields, B.D.; A study of highway capacity, Proceedings Highway Research Record, Washington, 14 (1935), pp. 448-477

[9] Kuhne, R. D.: Foundations of Traffic Flow Theory I: Greenshields´Legacy – Highway Traffic, Symposium on the Fundamental Diagram: 75 Years (Greenshields 75 Symposium), Woods Hole Massachusetts, United States, 8-10 July 2008.

[10] Auer, A., Feese, S., Lockwood, S.: History of Intelligent Transportation Systems, U. S. Department of Transportation, Washington, 2016.

[11] Directive 2010/40/EU of the European Parliament and of the Council, Official Journal of the European Union, L207/1, 2010.

[12] Jarašūniene, A.: Research into intelligent transport systems (ITS) technologies and efficiency, Transport, 22 (2007) 2, pp. 61-67

[13] Kamkar, S., Safabakhsk, R.: Vehicle detection, counting and classification in various conditions, Intelligent Transport Systems, 10 (2016) 6, pp. 406-413

[14] Babić, B.: Projektiranje kolničkih konstrukcija, Htvatsko društvo građevinskih inženjera, Zagreb, 1997.

[15] Botswana_Guideline 9: Traffic Data Collection and Analysis, 2003.

[16] Pengjun, Z., McDonad, M.: An Investigation on the Manual Traffic Count Accuracy, Procedia – Social and Behavioral Sciences, 43 (2012), pp. 226-231

[17] Slavuj, M.: Brojanje prometa, Fakultet prometnih znanosti, Zagreb, 2010.

[18] Rožić, P.: Primjena teorije prometnoga toka u brojenju prometa na cestama, GRAĐEVINAR 58 (2006), 1, pp. 25-34

[19] Ishak, S., Codjoe, J., Mousa, S., Jenkins, S., Bonnette, J.: Traffic Counting Using Existing Video Detection Cameras, Lousiana Department of Transportation and Development Louisiana Transportation Research Center, 2016.

[20] Kaack, L.H., Chen, G.H., Morgan, M.G.: Truck traffic monitoring with satellite images, ACM Compass, pp. 155-164, Accra Ghana, July 2019.

[21] Larsen Oyen, S., Koren, H., Solberg, R.: Traffic Monitoring using Very High Resolution Satellite Imagery, Photogrammetric Engineering and Remote Sensing, 75 (2009) 7, pp. 859-869

[22] Prabha, R., Kabadi, M.G.: Overview of Data Collection Methods for Intelligent Transportation Systems, The International Journal of Engineering and Science (IJES), 5 (2016) 3, pp. 16-20

[23] Zakon o cestama, NN 84/11

[24] Božić, M., Kopić, D., Mihoci, F., Marold, N., Gršetić, J.: Brojenje prometa na cestama Republike Hrvatske godine 2018., Prometis d.o.o., Zagreb, 2019.

DEPENDENCE OF DESIGN HOURLY VOLUME ON THE FUNCTION AND NATURE OF TRAFFIC DEMAND OF RURAL ROADS

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Abstract

In the first phases of study and design documentation of rural roads, one of the key parameters to determine in the analyses is the Design Hourly Volume (DHV). The required level of service and the feasibility of the project depend to a large extent on a properly established DHV. Essentially, the problem is to determine the value of the K-factor for a certain nth highest hour of the year. This paper points to the need for additional analysis of existing databases of long-term automatic traffic counting, from which the necessary guidance for planners and designers can be derived, enabling them to understand and apply the K-factors in a clearer and more detailed way. Using specific data examples, characteristic sections of rural roads with different functions and types (seasonal variations) of traffic demand were selected to show significant differences in the values of the K-factors for the same selected nth highest hour of the year. Several guidelines (BiH, Slovenia, Croatia, Italy, Serbia) were analysed beforehand to get a better understanding of how the K-factor or DHV is explained and used in different countries. The main objective of the article is to show that, on the basis of the existing databases of continuous automatic counting in these countries, with additional analyses presented in this paper or in a similar form, significant regularities in determining the DHV can be achieved, eliminating difficulties of application in engineering practice. As all guidelines practically recommend the use of HCM in capacity analyses, specific examples are selected to show the difference between the definition of HCM for a route with dominant recreational traffic and our route with dominant tourist traffic (recreational versus tourist).

Keywords: K-factor, DHV, measure of seasonal variation

1 Introduction

In the capacity analysis of rural road sections, the first step is to determine the Design Hourly Volume (DHV). This analysis is also an integral part of the study documentation up to the level of the main project. An insufficient determination of the DHV can lead to considerable errors in choosing the most optimal solution. In the absence of more detailed guidelines, quite different approaches, as well as the lack of importance attached to this problem, can be observed in engineering practice.

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1.1 Design Hourly Volume

Design Hourly Volume is defined as a proportion of AADT using the K-factor. It is determined at the n^{th} highest peak hour at the end of the planning period.

$$DHV = K\ AADT$$

DHV – design hourly volume (veh/h),
K – factor representing the proportion of annual average daily traffic occurring in the n^{th} highest hour,

AADT - the annual average daily traffic volume, corresponding to the average daily (24-hour) traffic volume on a given cross-section over the whole year. It is obtained by dividing the total number of vehicles that have passed through the cross-section during the year by 365 days. In cases where continuous automatic counting (Croatian for "permanent counting" is NAB) is available for 365 days or 8760 hours, the problem is determining the n^{th} highest hour. Figure 1 shows the descending curve of hourly volume as a percentage of the AADT in the year (8760 hours) from maximum to minimum. The shape of the curve shows that there are only very few (n) hours of the year when the road is extremely congested, and in the remaining hours of the year the volume is much lower. There is the "knee" on the curve, which is the boundary between the strongly descending and the slightly descending part of the curve. It is not possible to plan the road for a few hours and for a volume that on tourist roads can be up to 50 % higher than during the rest of the year. The n^{th} highest hour is defined differently in different countries (30^{th}, 50^{th}, 60^{th}, 100^{th}). Practically, the K-factor is the percentage ratio of the hourly volume in the n^{th} highest hour to the AADT.
Figure 1a shows the percentage ratio of all hourly volumes of the year to the AADT in the order from highest to lowest for two different roads (left and right). In this case, the range is from a maximum of 15.21 % (left) or 21.98 % (right) to values close to zero. It can also be seen that there is the above-mentioned "knee", which is more clearly recognizable with a lower number of peak hours (Figure 1 b, c, d). All countries carry out automatic traffic censuses to a greater or lesser extent, but the usual situation is that permanent censuses are carried out on about 50 % of the total number of sections. The remaining sections are usually subject to a temporary count, from which the AADT and other required parameters are estimated based on permanent count data on sections with similar traffic characteristics. Various methods for estimating AADT have been proposed, based on a combination of temporary and permanent counts [1], while research on K-factors has yielded significantly fewer results [2]. These are mostly manuals [3], [4], [5], [6]. The key question in this article is the estimation of the K-factor or DHV.

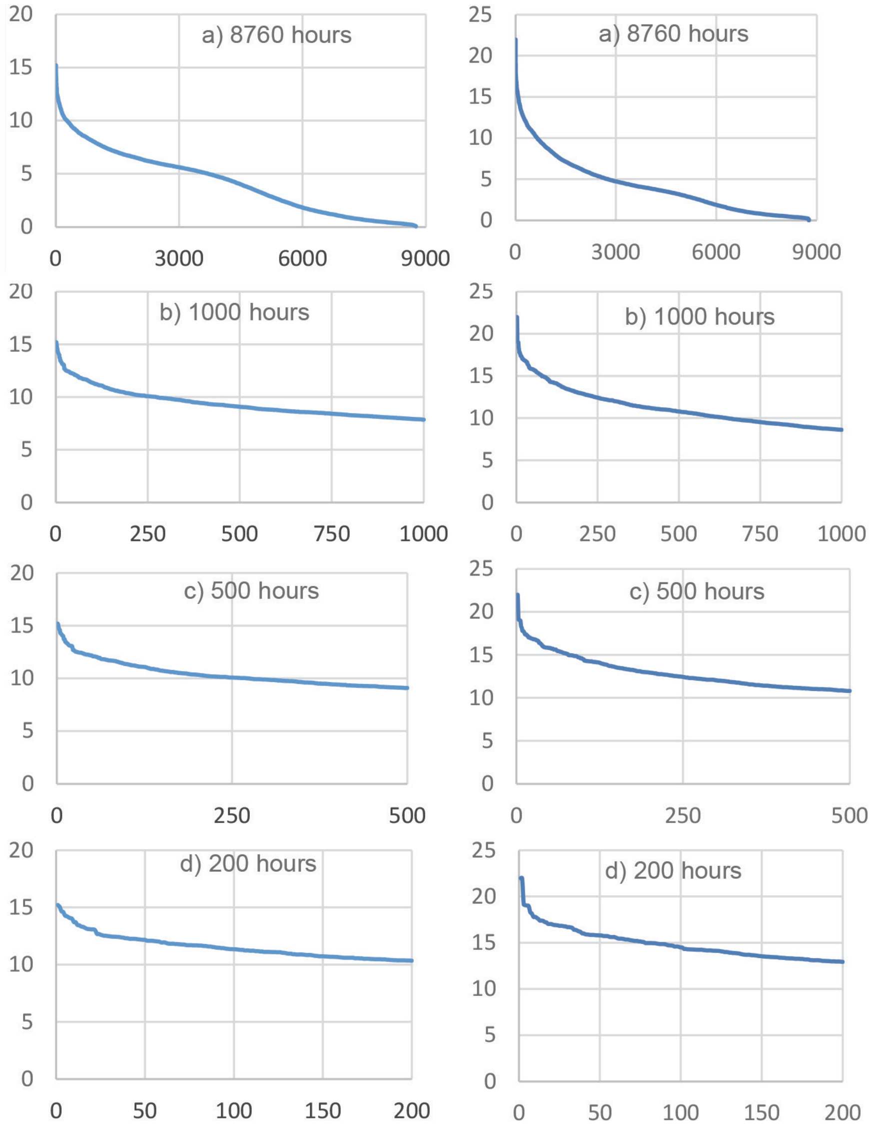


Figure 1 The percentage ratio of hourly volume to AADT for two different roads

1.2 Determination of DHV according to different guidelines

On road sections where there is no permanent counting, the K-factor must be estimated based on additional analyses that are not sufficiently specified in the guidelines. The following is a brief overview of how the guidelines in Bosnia and Herzegovina, Croatia, Slovenia, Serbia and Italy define it.
The guidelines in BiH [7] do not specify the n^{th} highest hour at all, but only give approximate values of the K-factor (Table 1).

Table 1 Indicative values of K-factor [7]

Type of road	K-factor [% AADT]
Roads for long-distance connection	12 - 16
Intercity roads (rural)	10 - 14
Suburban roads (and long - distance)	9 - 11
Urban roads (except local)	8 - 10

In addition, it is stated: “On roads with particularly heavy seasonal traffic (when seasonal traffic exceeds the average by more than 50 %), it is recommended that traffic data and flow calculation be provided separately for seasonal and non-seasonal traffic. In such a case, for reasons of construction rationality, it is recommended to consider a lower service level or a 10-20 km/h lower average speed than planned on a specific road of a certain category for the season. The above recommendation generally cannot be applied to multi-lane roads with separate lanes” [7]. The guidelines in Croatia [8] only specify that the 100th highest hour (Q100) of a year is relevant for the capacity analysis. The guidelines in Slovenia [9] specify that traffic loads should be determined on the basis of HCM methodology. In the absence of traffic load forecasts for peak hours (DHV), the following AADT percentages are used to calculate the level of service – LOS (Table 2).

Table 2 Indicative values of K-factor from Slovenian guidelines [9]

Type of road	K-factor [% AADT]
Roads for long-distance connection	12
Connecting road	10
Collector road	9
Access road	8

The guidelines in Serbia [10], take a closer look at this issue. First, they introduce specific classifications in the functional classification of roads, including the type of transport demand. On rural roads, relatively independent of the functional classification, there are different characteristics of the dominant traffic flows in terms of frequency of occurrence. There are three types of roads (Table 3).

Table 3 Road types in terms of traffic demand character [10]

Nature of traffic flow	Movement frequency	Characteristic day	b_s	K-factor (%)
Urban - suburban	every day	weekday	< 1.2	10 - 14
Intercity	temporary	weekday, weekend	$1.2 < b_s < 1.4$	13 - 17
Intercity - tourist	seasonal	weekend, season	> 1.4	15 - 30

It is further illustrated by the approximate coefficient of annual unevenness b_s, which is defined as the ratio between the average daily traffic volume (ADT) in the peak month (July, August) and the ADT in the average month (April, May, October, November). The months mentioned above refer to road sections that were counted temporarily.

As usual the DHV is defined by the K-factor for a given n^{th} highest hour. The Q_{30} is assumed for the highest categories of rural roads and the Q_{60} for the other roads. For the dominant character of traffic flow (Table 3), the approximate values of the K-factor are given for road sections on which no continuous automatic counting takes place.
The guidelines in Italy [11], are limited to providing only a few definitions and implement the contents of the HCM (Highway Capacity Manual published by the TRB, 1994) as regards the definition of Level of Service (LOS). In general, the roads are classified and divided as in the following table.

Table 4 Italian Road classification [11]

Network	Scope	Extra-urban area	Urban area
Primary	transit and sliding	extra-urban highway main suburban roads	urban highway extra-urban roads
Principal	distribution	main suburban roads	extra-urban roads
Secondary	penetration	secondary suburban roads	urban neighbourhood streets
Local	access/entry	urban local roads	extra-urban local roads

In the Italian guidelines, the service flow rate is the maximum value of the traffic flow that can be accommodated by the road at the assigned service level. It depends on the transverse characteristics of the section and plano-altimetric characteristics of the axis.
The Italian rules do not provide any K-value but there is a good practice of adopting some American values. In fact, in agreement with Florida Department of Transportation FDOT, K-values are considered to be between 0.095 and 0.10, for rural developed and rural undeveloped roads, respectively [12].
The two manuals most frequently used worldwide for capacity analysis to determine DHV use the Q_{30} (American HCM [3]) and the Q_{50} (German HBS [4]).
All countries carry out traffic counts on their road networks. In places where continuous automatic counting takes place in the year (8760 hours), it is possible to determine the change in hourly volume, expressed as a percentage of the AADT, in descending order from the highest to the lowest value (Figure 1a). Normally only 200 peak hours aredisplayed. On the other hand, road sections with temporary counting points require additional analyses.
From the above it is clear that the most frequently mentioned guidelines do not sufficiently define how the DHV is to be determined. Planners and designers are required to recognise this in the early design stages with additional analyses. Moreover, as will be shown below, the nature of traffic demand varies considerably from country to country, suggesting the need to derive additional specific characteristics from existing automatic counting databases.

1.3 Recreational versus tourist traffic

Due to the different nature of traffic demand on the one hand and the function of roads on the other hand, changes in the characteristics of traffic flows can vary significantly by seasons, months and weeks. Typically, a daily volume (veh/day) is used and the following characteristic quantities are available:

- Average annual daily traffic (AADT), as defined above
- Average summer daily traffic (ASDT) represents the average daily (24 hours) traffic volume on a given cross-section during the summer (here July and August). It is obtained by dividing by 62 days the total number of vehicles that have passed through the cross-section in July and August.

• Average daily traffic (ADT) represents the average daily (24 hours) traffic volume for a given cross-section over a period of less than one year (normally used for months). It is obtained by dividing the total number of vehicles that have passed through the cross-section in the given time interval by the number of days in that interval.
• Average annual weekday traffic (AAWDT) represents the average daily (24 hours) traffic volume on a given cross-section during the year, counted only on weekdays (Monday - Friday). It is obtained by dividing the total number of vehicles that have passed through the cross-section in weekdays during the year by the number of weekdays (usually 260 days).
• Average weekday daily traffic (AWDT) represents the average daily (24-hour) traffic volume on a given cross-section in weekdays over a period of less than one year (usually used for months). It is obtained by dividing the total number of vehicles that have passed through the cross-section in the given time interval by the number of weekdays in that interval.

AADT, ASDT and ADT are used in our prevailing traffic conditions. AAWDT and AWDT represent the average daily traffic from Monday to Friday and are not of great interest to us as the seasonal fluctuations in traffic demand are much more dominant.
Figure 2 shows a typical US classification of roads in terms of the type of traffic demand on roads with dominant recreational traffic and roads with dominant intercity traffic. The variation of monthly ADT and AWDT in the year is shown. “Monthly volume variations for routes with recreational traffic show much higher seasonal peaking than for routes with predominantly intercity traffic” [3]. Weekend traffic values range from about 100 % to 200 % of the AADT. For roads with dominant intercity traffic, this difference is insignificant (Figure 2b).

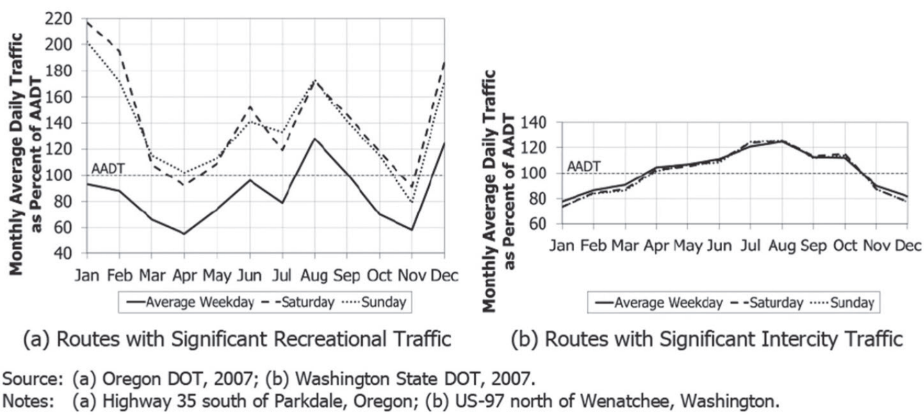


Figure 2 Examples of monthly ADT variation on highway [3]

Figure 3 shows a comparison of the annual variation of monthly ADT and AWDT as percent of AADT for:
• Road with dominant recreational traffic [5]
• Road M2 - section - Neum - Zaton Doli in BiH [13]
• Road M6.1 - section – Mostar - Široki Brijeg in BiH [13]

In contrast to the pattern of traffic demand on recreational roads in the US, which are used by drivers on weekends throughout the year and where the AADT is significantly higher than the AAWDT, it is obvious that the M2 road with the pattern of “tourist” traffic demand does not show this difference at all (Figure 3). In contrast, a significant difference in seasonal variation can be observed.

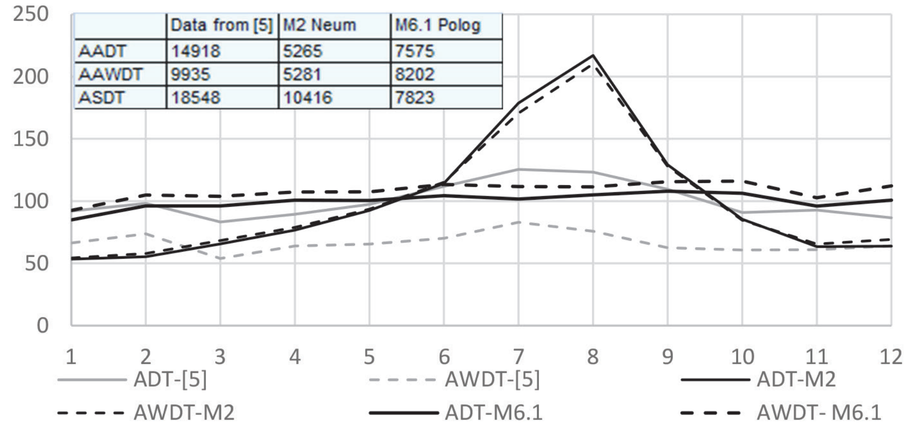


Figure 3 Variation of Monthly ADT and AWDT as Percent of AADT for USA road with recreational traffic and two types of roads in BiH

Although there is also a seasonal (summer) increase in traffic under the conditions of traffic demand in America, it is negligible in relation to the weekly changes, while under our conditions of traffic demand from “tourist” roads, the seasonal (in this case summer) increase has a dominant influence. For example, it is obvious that the average daily traffic during the summer peak exceeds 200 % of AADT, i.e. the ASDT/AADT ratio is almost twice as high. In contrast to the M2 section, the M6.1 road section is characterized by dominant intercity traffic. A slight difference between ADT and AWDT is also noticeable. If the function of the house-work connection is considered, it is logical in this case to obtain slightly higher AWDT values. There are no seasonal fluctuations.
Given the significant differences in the conditions of traffic demand in the USA and here described above, it is necessary to distinguish between a road with recreational (HCM) and a road with “tourist” traffic (here). A road with significant “recreational” traffic is a road that has a higher volume of traffic on weekends (Saturday and Sunday) than on weekdays throughout the year. The characteristic traffic parameters are AADT, AAWDT, ADT and AWDT. A road with significant “tourist” traffic is a road with dominant seasonal variations (summer and/or winter). The characteristic traffic parameters are AADT, ASDT and ADT.

2 K-factor in function of the character of traffic demands

It is clear from the above that the choice of the n^{th} highest hour of the year varies from one guideline to another between the 30^{th} and 100^{th} highest hour. Essentially, this question should be answered by the institutions responsible for overall road management, and it must be based on the results of multi-year traffic counting and research. The choice of the n^{th} highest hour and the determination of the K-factor is always a compromise between the satisfaction of traffic demand (desired level of service) on the one hand and investment possibilities on the other. Regardless of the choice of the n^{th} highest hour, the value of the K-factor depends primarily on the character of traffic demand, on its different occurrence. The four characteristic sections of BiH’s main roads on which permanent traffic counting is carried out over several years were selected [13]:

a) Road M17, section: Jablanica - Mostar, counting station Salakovac, example of a rural section
b) Road M17, section: Mostar - Buna, counting station Ortiješ, example of a suburban section

c) Road M6.1, section: Mostar - Široki Brijeg, counting station Polog, example of a rural section
d) Road M2, section: the border of the Republic of Croatia - Neum, counting station Neum, example of a rural section

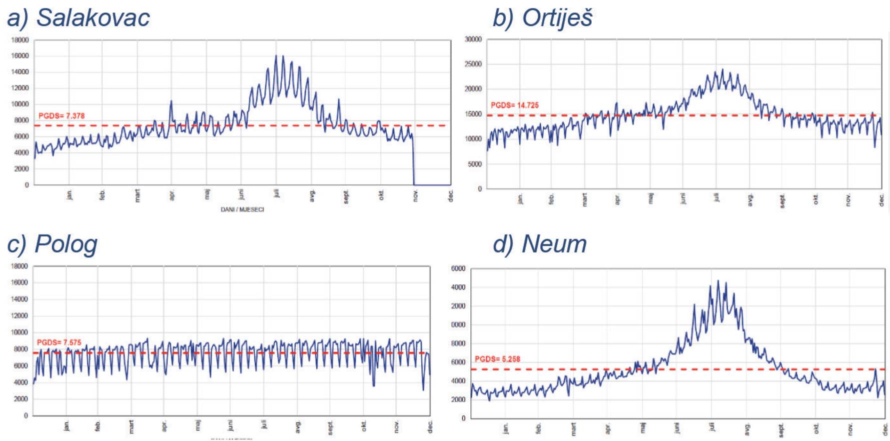
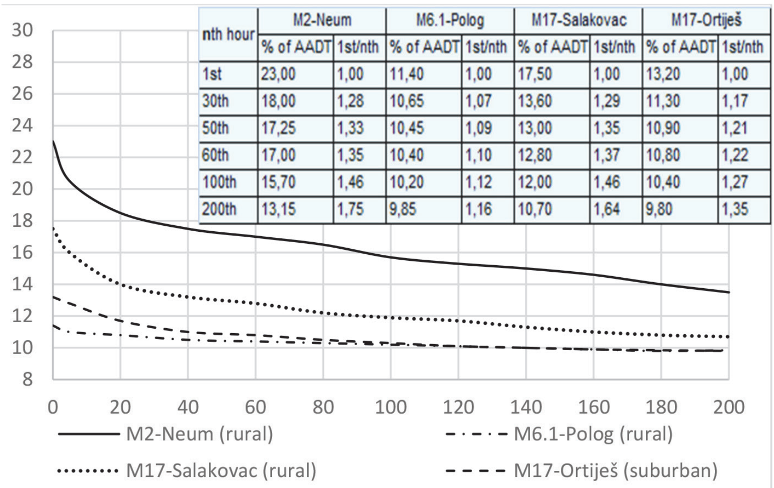


Figure 4 Variation of daily volume for 4 above selected roads in BiH

Annual variation of daily volume for these road sections is shown in Figure 4. The M17 (E-73) is a main road connecting the north and south of BiH. It is located along the Corridor Vc and, because the motorway is not built yet, it is still one of the main roads with the function of a long-distance connection. Two sections (rural and suburban areas) were selected to identify differences. Figures 4a and 4b also show considerable seasonal variations. The M6.1 (Figure 4c) represents the rural section of the main road without seasonal variations. The M2 (Figure 4d) represents the section of the main road (along the Adriatic Sea) in BiH, which has a dominant seasonal variation.
The graphs in Figure 5 show the decreases of the 200 highest hours of the year as a percentage of AADT for the listed sections based on actual permanent counting data. The following points can be highlighted:

- It is expected that a significant difference in the K-factor values will be observed, although all roads have the same classes and almost the dominant connection function. The value of the K-factor thus depends on the nature of the traffic flows in terms of their seasonal variation.
- Roads without seasonal variations (M6.1) have the lowest values of the K-factor as well as the smallest difference in percentage ratio in the 1st and the indicated nth highest hour. Roads with dominant seasonal variation (M2) have the highest K-factor values and the greatest difference in the characteristic nth hour.
- The K-factor for rural roads with a certain degree of seasonal variation takes values between the lowest and the highest (M17-Salakovac). In the case of a suburban road section (M17 - Ortiješ), the value of the K-factor decreases and approaches the values of the sections without seasonal fluctuations due to the high proportion of local traffic in the AADT.
- The specific values of the K-factor for different nth highest hours are shown in the table in Figure 5. They are not representative but are presented with the aim of showing significant differences.

It is obvious that it is necessary to introduce a measure of seasonal variations following a similar logic as the Serbian guideline.

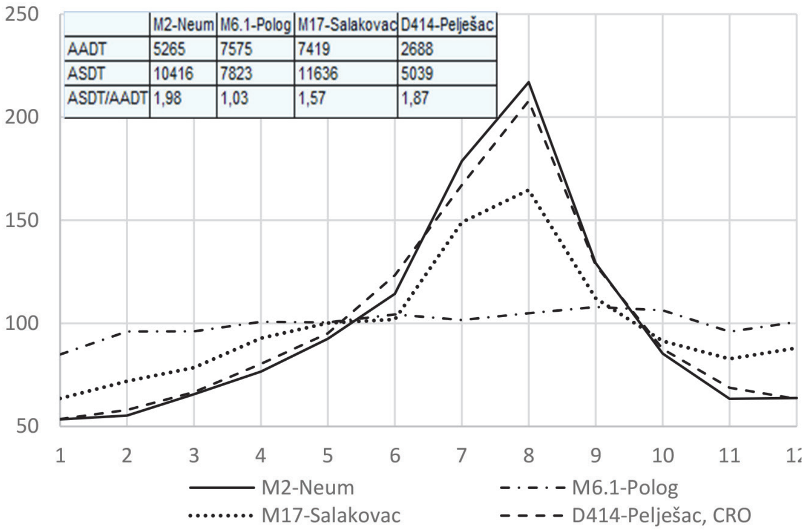


nth hour	M2-Neum		M6.1-Polog		M17-Salakovac		M17-Ortiješ	
	% of AADT	1st/nth	% of AADT	1st/nth	% of AADT	1st/nth	% of AADT	1st/nth
1st	23,00	1,00	11,40	1,00	17,50	1,00	13,20	1,00
30th	18,00	1,28	10,65	1,07	13,60	1,29	11,30	1,17
50th	17,25	1,33	10,45	1,09	13,00	1,35	10,90	1,21
60th	17,00	1,35	10,40	1,10	12,80	1,37	10,80	1,22
100th	15,70	1,46	10,20	1,12	12,00	1,46	10,40	1,27
200th	13,15	1,75	9,85	1,16	10,70	1,64	9,80	1,35

Figure 5 Volume of 200 highest hours as a percent of AADT for road sections with different seasonal variation

3 A measure of seasonal variation

From the above examples of seasonal variation, it is clear that the relationship between ASDT and AADT can be one measure. Unlike AADT, ASDT is defined differently in different countries. In this article, ASDT represents the average daily summer traffic in July and August.
Figure 6 shows the changes in monthly ADT over the year, the values of AADT and ASDT and their ratio for the listed sections. Instead of the suburban section (M17 Ortiješ), a section of the main road D414 in the Republic of Croatia (Peljesac) was inserted, the Zamaslina counting station [14], which, like the section M2 Neum, has a dominant seasonal variation.



	M2-Neum	M6.1-Polog	M17-Salakovac	D414-Pelješac
AADT	5265	7575	7419	2688
ASDT	10416	7823	11636	5039
ASDT/AADT	1,98	1,03	1,57	1,87

Figure 6 Annual variation of monthly ADT as percent of AADT for roads with different character of traffic demand

Seasonal variation (changes in monthly ADT and the ratio of ASDT/AADT values) shows three characteristic road types in relation to the type (character) of traffic demand:

- Roads without seasonal variations are those where the ASDT/AADT ratio is close to 1.0. In Figure 6 it is section M6.1.
- Roads with a dominant tourist function (seasonal variations) are those where the ASDT/AADT ratio is extremely high. In Figure 6 these are sections of M2 Neum in BiH and D414 in Peljesac, Croatia.
- Roads in the middle range of the ASDT/AADT ratio are those where there is significant but not dominant seasonal variation. An example is the section of the M17 road, data from the Salakovac counting station.

It can be observed that the differences in the values of the K-factor in Figure 5 and the differences in the ASDT/AADT ratio in Figure 6 show the same analogy in terms of the seasonal nature of traffic demand. The logical question is whether there is a functional relationship between this seasonal measure and the K-factor.

4 K-factor dependence on ASDT/AADT ratio as a measure of seasonal variation

To answer the above question, an example of two two-lane rural roads was taken. The first road is the D8 in the Republic of Croatia and the second is the M17 in BiH. Only data from permanent counting stations are analysed [13], [14]. As the Croatian guideline defines the 100th highest hour as relevant for the determination of the DHV, and the BiH guideline does not define the n^{th} highest hour at all, in this article the 100th highest hour was used for the determination of the K-factor.
There are 39 counting stations on the D8 road from Slovenia to Montenegro, and permanent counting is carried out at 27. The data from 2 counting stations (Solin and Stobrec) were left out, as these are suburban and four-lane sections. Figure 7 shows a very strong correlation between the measure of seasonality of ASDT/AADT and the value of the K-factor. This allows a more accurate determination of the K-factor on a road section with temporary counting.

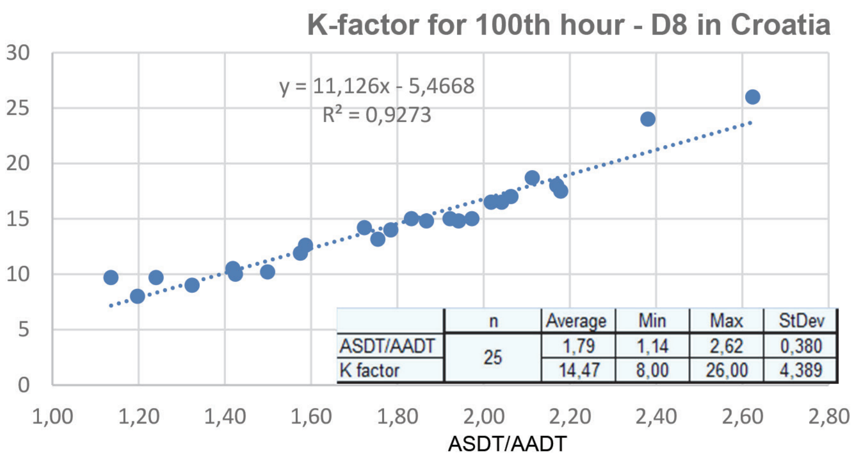
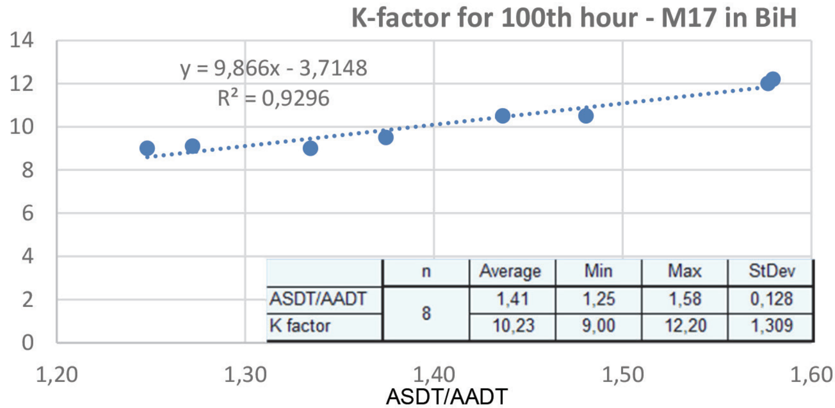


Figure 7 Dependence of K-factor on ratio ASDT/AADT for two-lane state road D8 in Croatia

On the road M17 (E-73), there are a total of 18 counting stations, 9 of which are with permanent counting. At the Buna counting station, the data since 2010 are incomplete, so they are not taken into account.

As in the previous case, a high collinearity was achieved between the ASDT/AADT measure and the K-factor. It is important to note that the ASDT is not taken from the original publication, in which it was calculated as an average of the calendar summer days, but it was calculated as an average of the daily volumes in July and August, as in the case of the D8 road.



	n	Average	Min	Max	StDev
ASDT/AADT	8	1,41	1,25	1,58	0,128
K factor		10,23	9,00	12,20	1,309

Figure 8 Dependence of K-factor on ratio ASDT/AADT for 2-lane main road M17 in BiH

One must also note the difference in the value ranges of the K-factors of one road and the other, which is also shown in Figures 7 and 8. Differences are expected, but only by introducing a measure of seasonality can this difference be quantified. Apart from the fact that BiH and Croatia generally have different seasonal effects, the main difference lies in the function of these two roads. For most of the D8 road, there is a parallel A1 motorway, which significantly reduced its long-distance function, and its seasonal character increased, while the M17 still has a long-distance function due to the lack of construction of the Vc motorway. In order to obtain more accurate results and verify them, it is certainly necessary to extend this research to a much larger number of roads.

5 Conclusions

The results presented in the paper show the following:

- It is possible and necessary to introduce a measure of seasonality that defines more clearly the character (nature) of traffic demands. This paper shows that this can be an ASDT/AADT ratio.
- Examples of two roads with different functions in different states, which are significantly covered by permanent counting, show that there is a high collinearity between the seasonality measure of ASDT/AADT and K-factors. There are no precise guidelines on how to determine the K-factor and DHV on road sections with temporary counting. This approach would enable engineers in daily practice to identify more accurately and clearly the DHV, which is one of the key parameters in capacity analyses.

As most of the guidelines refer to HCM, the difference between the “recreational” and “tourist” character of traffic demand is explained. All countries have an “endless” data source (decades of automatic traffic counting systems) from which these and a number of other regularities can be derived, which contribute significantly to achieving a more sustainable road system.

References

[1] Gastaldi, M., Rossi, R., Gecchele, G., Della Lucia, L.: Annual Average Daily Traffic Estimation from Seasonal Traffic Counts, Procedia – Social and Behavioral Science 87 (2013) pp. 279 – 291, doi:10.1016/j.sbspro.2013.10.610

[2] Sabry, M., Abd-El-Latif, H., Yousef, S., Badra, N.: Determination of K – Factor for Intercity Roads in Egypt, Journal of Applied Sciences Research, 3 (2007) 7, pp. 628-634

[3] Transportation Research Board, Highway Capacity Manual, Sixth Edition, TRB, National Academies of Sciences, Engineering and Medicine, Washington, D. C., 2016.

[4] FGSV: Handbuch fuer die Bemessung von Strassen (German Highway Capacity Manual), HBS 2015, FGSV, Cologne, edition 2015.

[5] Roess, R.P., Prassas, E.S., McShane, W.R.: Traffic Engineering, Third edition, Pearson Education Inc., Pearson Prentice Hall, New Jersey, 2004.

[6] FDOT, Project Traffic Forecasting Handbook, 2014, https://fdotwww.blob.core.windows.net/sitefinity/docs/default-source/content/planning/systems/programs/sm/ptf/docs/ptf_handbook_2014.pdf

[7] Službeni glasnik Bosne i Hercegovine 13/2007: Pravilnik o temeljnim uvjetima koje javne ceste, njihovi elementi i objekti na njima moraju ispunjavati s aspekta sigurnosti prometa (BH Guidelines for the Design of Rural Roads), Sarajevo (in Croatian)

[8] Narodne novine Republike Hrvatske NN 110/01, 2001. Pravilnik o osnovnim uvjetima kojima javne ceste izvan naselja i njihovi elementi moraju udovoljavati sa stajališta sigurnosti prometa (Croatian Guidelines for the Design of Rural Roads), Zagreb (in Croatian)

[9] Uradni list Republike Slovenije RS 91/2005. Pravilnik o projektiranju cest (Slovenian Guidelines for the Design of Rural Roads), Ljubljana (in Slovenian)

[10] Službeni glasnik Republike Srbije 50/2011: Pravilnik o uslovima koje sa aspekta bezbednosti saobraćaja moraju da ispunjavaju putni objekti i drugi elementi javnog puta (Serbian Guidelines for the Design of Rural Roads), Beograd (in Serbian)

[11] Decreto Ministeriale 5 novembre 2001, n. 6792 (S.O. n.5 alla G.U. n.3. del 4.1.02) Norme funzionali e geometriche per la costruzione delle strade

[12] Guide, F.R.: Florida Department of Transportation, Tallahassee, March 1996.

[13] JP Ceste Federacije BiH, Brojanje prometa na magistralnim cestama FBiH (in Croatian) https://jpdcfbh.ba/bs/aktivnosti/brojanje-saobracaja/22

[14] Hrvatske ceste, Brojenje prometa na cestama Republike Hrvatske (in Croatian), https://hrvatske-ceste.hr/hr/stranice/promet-i-sigurnost/dokumenti/14-brojenje-prometa

7

ROAD SUPERSTRUCTURE: INNOVATION AND SUSTAINABILITY

CETRA2020*
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PERFORMANCE OF CONCRETE MIXTURES CONTAINING MSWI BOTTOM ASH

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Abstract

In the European Union, each inhabitant annually generates about 500 kg of municipal waste. About 30 % of this are incinerated in waste-to-energy plants. It results in approximately 20 million tonnes of residues known as municipal solid waste incinerator (MSWI) bottom ash, which is typically landfilled. To address the continuous growth of landfills and to implement zero waste and circular economy policies, researchers are focusing on possibilities to use MSWI bottom ash in civil engineering instead of landfilling. One of them is to replace natural aggregates in concrete mixtures applicable for roads with MSWI bottom ash. Therefore, the subject of this research is the performance of concrete mixtures containing different amount (0–100%) and fraction (0/5–0/16) of MSWI bottom ash. Four specimens with similar aggregate gradations were designed. Each of them was mixed with two different amount (340 kg/m3 and 300 kg/m3) of cement (CEM I 42.5 R). In total eight different concrete mixtures were tested and analysed. The performance of designed concrete mixtures containing different amount of MSWI bottom ash was evaluated according to density and compressive strength after 28 days. The results showed good MSWI bottom ash performance as a substitute for natural aggregates. The compressive strength after 28 days varied from 21 MPa to 29 MPa depending on the aggregate type and amount of MSWI bottom ash and cement. For concrete mixtures made only of MSWI bottom ash at least 340 kg/m3 of cement is required to achieve compressive strength higher than 20 MPa.

Keywords: bottom ash, municipal solid waste, municipal solid waste incinerator (MSWI), concrete mixture, compressive strength

1 Introduction

According to Eurostat each inhabitant annually generates about 500 kg of municipal waste in the European Union. About 30 % of this waste are incinerated in waste-to-energy plants. It results in approximately 20 million tonnes of large agglomerated residues, known as municipal solid waste incinerator (MSWI) bottom ash, which is typically landfilled. To address the continuous growth of landfills and to implement zero waste and circular economy policies, researchers are focusing on possibilities to use MSWI bottom ash somewhere else instead of landfilling.

Conducted studies confirm MSWI bottom ash suitability for road construction, i.e. construction of embankment and subgrade as well as unbound and cement bound sub-base courses and in some cases unbound base courses [1]–[6]. It is also used as binder to improve or stabilize soil and as substitute for aggregates in concrete and asphalt mixtures production [5–15]. From all possible application areas, MSWI bottom ash utilization as aggregate for concrete and asphalt mixtures production is preferable since it minimizes the release of toxic elements (e.g. alkaline element and heavy metals), which presence in the combustion by-product [16]. Environmental protection is a key factor considering bottom ash utilization. In concrete MSWI bottom ash is used to replace coarse aggregate, fine aggregate or both of them. Pera et al. [7] one of the earliest researchers, who analysed MSWI bottom ash as alternative aggregate for concrete, replaced 50 % of coarse aggregate with MSWI bottom ash. It gave a compressive strength after 28 days of 25 MPa. Rübner et al. [8] fully replaced coarse aggregate with additionally treated MSWI bottom ash. In this case, concrete with a compressive strength class of C20/25 has been produced. Zhang and Zhao [11] analysed concrete with partially (30 %, 50 % and 70 %) replaced coarse aggregate. The authors concluded that the maximum amount of MSWI bottom ash should not exceed 50 % and residues should be treated by a wet grinding process. Otherwise, compressive strength after 28 days is lower than 25 MPa. Kim et al. [13] replaced 10 %, 20 %,30 % and 50 % of fine aggregate with MSWI bottom ash. Compressive strength after 28 days reduced by 4–57 % depending on the amount of MSWI bottom ash in comparison with reference concrete. However, even with the highest amount of MSWI bottom ash compressive strength was higher than 20 MPa. Minane et al. [15] replaced all fine aggregate (sand) with 0/2 fraction of MSWI bottom ash. The compressive strength after 14 days was more than twice lower compared with control concrete of purely natural aggregate, but it increased depending on superplasticizer content. Sorlini et al. [10] fully replaced natural aggregate with washed MSWI bottom ash and mixed it with high early strength cement (CEM I 42.5 R). The compressive strength after 28 days was 27.25 MPa and concrete was classified as C25/30. Abbà et al. [12] used 27 % of MSWI bottom ash (0/10 fraction) to produce concretes with two high early strength Portland-limestone cements (CEM II B-LL 32.5 R and CEM II A-LL 42.5 R) at the same water/cement ratio (0.75). In both cases, compressive strength after 28 days was similar to the reference specimens. The difference of compressive strength between concretes with different cements was about 6 MPa. Concrete with the higher cement class had compressive strength higher than 20 MPa. From zero waste and circular economy policies point of view is desirable to use MSWI bottom ash in concrete as much as possible. However, it is not clear which aggregate (coarse, fine or both) should be replaced with MSWI bottom ash. Besides, it is not possible to directly apply other countries practice since MSWI bottom ash characteristics strongly depend on waste composition, which is directly influenced by people's habits and economic policy in the country or region. The main aim of this paper is, therefore, to design concrete mixtures for roads by replacing coarse aggregate, fine aggregate and both of them with MSWI bottom ash generated in the waste-to-energy plant in Klaipėda (Lithuania) and determine performance of those mixtures.

2 Materials and methods

2.1 Materials

Portland cement, conforming to European standard EN 197-1, of strength class 42.5 with high early strength (CEM I 42.5 R) was used in this experimental research. As natural fine and coarse aggregates were used sand (0/4 faction) and dolomite (5/16 fraction) respectively. MSWI bottom ash was produced in a waste-to-energy plant in Klaipėda (Lithuania). Ferrous and non-ferrous metals from MSWI bottom ash were recovered after more than 3 months of its ageing (weathering). During ageing MSWI bottom ash was stored in uncovered stockpiles with direct access to water. After the recovery of ferrous and non-ferrous metals two fractions of bottom ash were produced: 0/5 and 0/16 (Fig. 1). In this research, 0/16 fraction of MSWI bottom ash is assumed as coarse aggregate since more than 80 % of particles are coarser than 4 mm. Particle size distributions of all aggregates used to design concrete mixtures are given in Table 1.

Figure 1 MSWI bottom ash after recovery of ferrous and non-ferrous metals

Table 1 Particle size distributions of aggregates used to design concrete mixtures

Sieve size [mm]	Passing [%]			
	Sand	dolomite	MSWI bottom ash	
	0–4 mm	5–16 mm	0–5 mm	0–16 mm
22	100.0	100.0	100.0	100.0
16	100.0	97.9	100.0	90.8
8	100.0	27.0	99.9	48.1
4	99.5	0.4	92.8	16.3
2	88.3	0.4	75.2	12.2
1	67.8	0.4	55.8	11.0
0.5	40.3	0.3	39.1	10.0
0.25	11.0	0.3	25.1	8.4
0.125	1.0	0.3	16.9	6.3

2.2 Concrete mixture proportions

Four particle size distributions similar to each other have been designed for concrete by changing the type and amount of coarse aggregate, fine aggregate or both of them (Fig. 2). Aggregate proportions are listed in Table 2. Each aggregate mixture was mixed with 340 kg/m^3 and 300 kg/m^3 of Portland cement (CEM I 42.5 R). Water and cement ratio for concrete made purely of natural aggregates (reference mixture) was 0.40. Meanwhile, water content for concrete mixtures with MSWI bottom ash was gradually increased in order to improve workability since MSWI bottom ash absorbs much more water compared to natural aggregates [5].

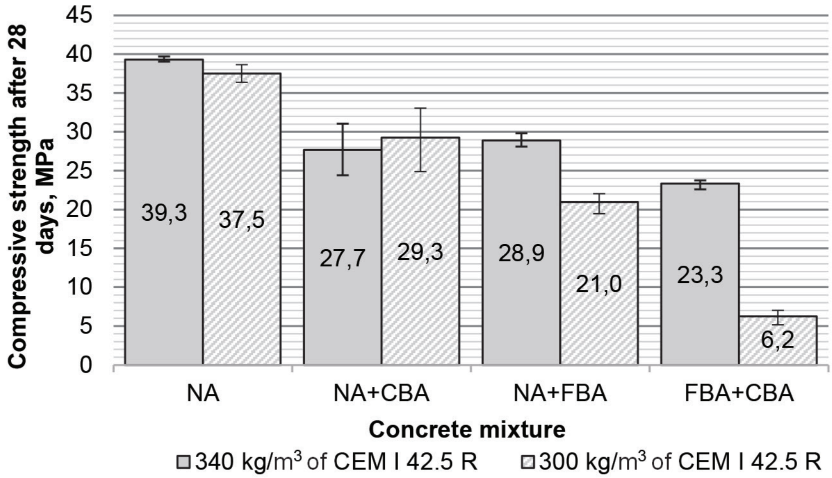

Figure 2 Particle size distributions

Table 2 Aggregate proportions in concrete

Concrete mixture	Aggregate [%]			
	Sand	dolomite	MSWI bottom ash	
	0–4 mm	5–16 mm	0–5 mm	0–16 mm
NA	50	50	–	–
NA+CBA	40	–	–	60
NA+FBA	–	50	50	–
FBA+CBA	–	–	30	70

2.3 Test methods

Physical and mechanical performance of designed concrete mixtures were evaluated by measuring their density and compressive strength after 28 days. These characteristics were determined in accordance with European standards EN 12390-7 and EN 12390-3. All specimens were compacted with a vibrating table and stored in the laboratory at an ambient temperature of 20 °C and relative humidity of 95 %. 1 day after casting, they were removed from the moulds and remaining 27 days were stored in the water at temperature of 20±2 °C.

3 Results

3.1 Density after 28 days

The density of concrete mixtures containing different fraction and amount of MSWI bottom ash as well as different cement content is given in Fig. 3. Replacement of natural aggregates with MSWI bottom ash results in 4–26 % lower density. The density decreases as the amount of MSWI bottom ash increases. It is caused by the higher porosity of MSWI bottom ash in comparison with natural aggregates. When all natural aggregates are replaced with MSWI bottom ash and mixed with 340 kg/m^3 and 300 kg/m^3 of cement, the density is 2.0 g/cm^3 and 1.7 g/cm^3, respectively. In all cases except concrete mixtures, in which only coarse aggregate was replaced with MSWI bottom ash (NA+CBA), lower amount of cement lead to slightly lower density. The highest (0.3 g/cm^3) difference in density between different cement content was determined for concrete mixtures with pure MSWI bottom ash.

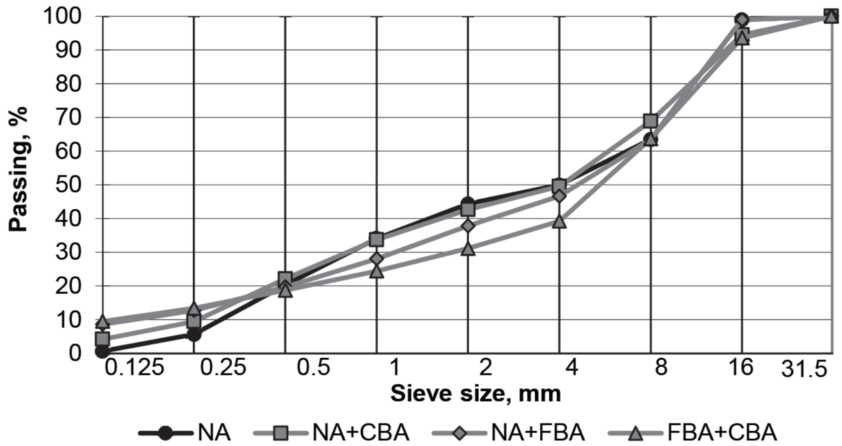

Figure 3 Density after 28 days

3.2 Compressive strength after 28 days

The compressive strength after 28 days of concrete mixtures containing different fraction and amount of MSWI bottom ash as well as different cement content is given in Fig. 4. In all cases replacement of natural aggregates with MSWI bottom ash reduced the compressive strength and the difference in strength increased with the increment in the amount of MSWI bottom ash. Concrete mixtures with partially replaced natural aggregate (NA+CBA and NA+F-BA) except mixture with cement of 300 kg/m^3 performed similarly and had the compressive strength of 27.7–29.3 MPa independent of cement content and MSWI bottom ash fraction. It is about 22–30 % lower compared to the reference specimens. While compressive strength of concrete mixture with fine aggregate of MSWI bottom ash (NA+FBA) and cement of 300 kg/m^3 was almost twice lower in comparison with reference specimens.

Concrete mixtures with pure MSWI bottom ash (FBA+CBA) revealed the worst performance since the difference in strength increased more than twice. Nevertheless, concrete mixture with cement of 340 kg/m^3 had compressive strength of 23.3 MPa. Meanwhile, lower cement content gave compressive strength of only 6.2 MPa. From the road construction point of view, concrete with compressive strength lower than 20–25 MPa is not suitable. It is worth highlighting that reduction in strength is related not only to replacement level of natural aggregate with MSWI bottom ash, but also to significantly increased water and cement ratio. Seeking to reduce the water content, superplasticizers have to be used.

As seen from the Fig. 4, concrete with a compressive strength class of C20/25 is successfully produced by partially (50–60 %) replacing natural aggregate with MSWI bottom ash. However, the cement content has to be at least 340 kg/m^3.

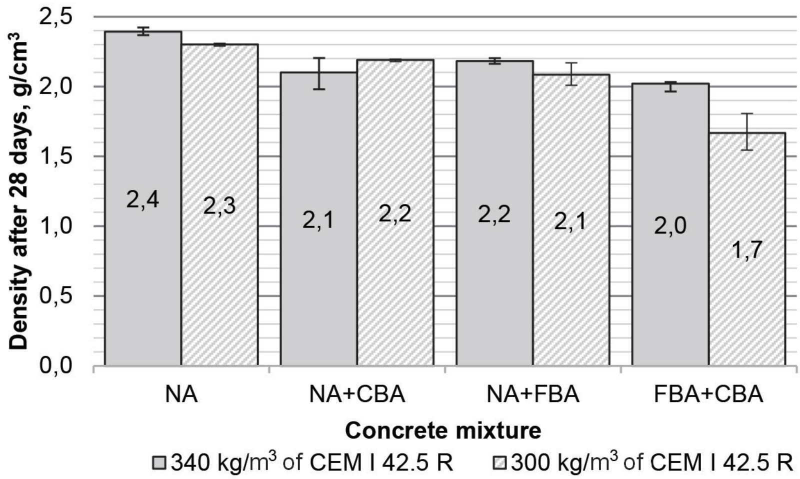

Figure 4 Compressive strength after 28 days

4 Conclusions

This research presents the performance of concrete mixtures containing different amount (0–100 %) and fraction (0/5–0/16) of MSWI bottom ash generated in the waste-to-energy plant in Klaipėda (Lithuania) and mixed with two different amounts (340 kg/m^3 and 300 kg/m^3) of cement (CEM I 42.5 R). Based upon the results obtained in this research the following conclusions are drawn:

- The use of MSWI bottom ash decreases concrete density since MSWI bottom ash has lower density in comparison with natural aggregates.
- MSWI bottom ash can be successfully used as a substitute for natural aggregates to produce concrete for low volume roads as well as for bicycle and pedestrian paths with a compressive strength class of C20/25 by fully replacing coarse or fine aggregate with the total amount of 50–60 %. Concrete of pure MSWI bottom ash is not acceptable since the compressive strength after 28 days is lower than 20 MPa.
- Reduction in compressive strength after 28 days is related not only to replacement level of natural aggregate with MSWI bottom ash, but also to significantly increased water and cement ratio. Seeking to reduce the water content and have workable concrete mixture, superplasticizers have to be used. Their optimal content have to be determined by trial mixing or by gained experience.
- Cement content has effect on concrete compressive strength except when coarse aggregate is replaced with MSWI bottom ash. To reach a compressive strength class of C20/25 at least 340 kg/m^3 of cement (CEM I 42.5 R) has to be used.

Acknowledgments

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References

[1] Paine, K.A., Dhir, R.K., Doran, V.P.A.: Incinerator Bottom Ash: Engineering and Environmental Properties as a Cement Bound Paving Material, International Journal of Pavement Engineering, 3 (2002) 1, pp. 43–52, doi: 10.1080/10298430290023458.

[2] Forteza, R., Far, M., Seguı, C., Cerdá, V.: Characterization of bottom ash in municipal solid waste incinerators for its use in road base, Waste Management, 24 (2004) 9, pp. 899–909, doi: 10.1016/j.wasman.2004.07.004.

[3] Hjelmar, O., Holm, J., Crillesen, K.: Utilisation of MSWI bottom ash as sub-base in road construction: First results from a large-scale test site, Journal of Hazardous Materials, 139 (2007) 3, pp. 471–480, doi: 10.1016/j.jhazmat.2006.02.059.

[4] Becquart, F., Bernard, F., Abriak, N.E., Zentar, R.: Monotonic aspects of the mechanical behaviour of bottom ash from municipal solid waste incineration and its potential use for road construction, Waste Management, 29 (2009) 4, pp. 1320–1329, doi: 10.1016/j.wasman.2008.08.019.

[5] Vaitkus, A., Gražulytė, J., Vorobjovas, V., Šernas, O., Kleizienė, R.: Potential of MSWI bottom ash to be used as aggregate in road building materials, Baltic Journal of Road and Bridge Engineering, 13 (2018) 1, pp. 77–86, doi: 10.3846/bjrbe.2018.401.

[6] Vaitkus, A., Gražulytė, J., Šernas, O., Vorobjovas, V., Kleizienė, R.: An algorithm for the use of MSWI bottom ash as a building material in road pavement structural layers, Construction Building Materials, 212 (2019), pp. 456–466, doi: 10.1016/j.conbuildmat.2019.04.014.

[7] Pera, J., Coutaz, L., Ambroise, J., Chababbet, M.: Use of incinerator bottom ash in concrete, Cement and Concrete Research, 27 (1997) 1, pp. 1–5, 1997, doi: 10.1016/S0008-8846(96)00193-7.

[8] Rübner, K., Haamkens, F., Linde, O.: Use of municipal solid waste incinerator bottom ash as aggregate in concrete, Quarterly Journal of Engineering Geology and Hydrogeology, 41 (2008) 4, pp. 459–464, doi: 10.1144/1470-9236/07-036.

[9] Hassan, H.F., Al-Shamsi, K.: Characterisation of asphalt mixes containing MSW ash using the dynamic modulus | E *| test, International Journal of Pavement Engineering, 11 (2010) 6, pp. 575–582, doi: 10.1080/10298436.2010.501865.

[10] Sorlini, S., Abbà, A., Collivignarelli, C.: Recovery of MSWI and soil washing residues as concrete aggregates, Waste Management, 31 (2011) 2, pp. 289–297, doi: 10.1016/j.wasman.2010.04.019.

[11] Zhang, T., Zhao, Z.: Optimal Use of MSWI Bottom Ash in Concrete, International Journal of Concrete Structures and Materials, 8 (2014) 2, pp. 173–182, doi: 10.1007/s40069-014-0073-4.

[12] Abbà, A., Collivignarelli, M.C., Sorlini, S., Bruggi, M.: On the reliability of reusing bottom ash from municipal solid waste incineration as aggregate in concrete, Composites Part B: Engineering, 58 (2014), pp. 502–509, doi: 10.1016/j.compositesb.2013.11.008.

[13] Kim, J., Nam, B. H., Al Muhit, B. A., Tasneem, K.M., An, J.: Effect of chemical treatment of MSWI bottom ash for its use in concrete, Magazine of Concrete Research, 67 (2015) 4, pp. 179–186, doi: 10.1680/macr.14.00170.

[14] Lynn, C.J., Dhir OBE, R.K., Ghataora, G. S.: Municipal incinerated bottom ash characteristics and potential for use as aggregate in concrete, Construction and Building Materials, 127 (2016), pp. 504–517, doi: 10.1016/j.conbuildmat.2016.09.132.

[15] Minane, J.R., Becquart, F., Abriak, N.E., Deboffe, C.: Upgraded Mineral Sand Fraction from MSWI Bottom Ash: An Alternative Solution for the Substitution of Natural Aggregates in Concrete Applications, Procedia Engineering, 180 (2017), pp. 1213–1220, doi: 10.1016/j.proeng.2017.04.282.

[16] Tasneem, K.M., Eun, J., Nam, B.: Leaching behaviour of municipal solid waste incineration bottom ash mixed with Hot-Mix Asphalt and Portland cement concrete used as road construction materials, Road Materials and Pavement Design, 18 (2017) 3, pp. 687–712, 2017, doi: 10.1080/14680629.2016.1186108.

UTILIZATION OF GLASS WASTE IN VEHICLE RESTRAINT SYSTEMS

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Abstract

The growing demand of society requires that engineers should concentrate more and more on recycling of broken road materials and various by-products in order to ensure environmental sustainability for future generations. Concrete technology has evolved to such an extent that it has become an important role in waste/secondary material management by now. There are several benefits of using glass waste in concrete mixtures. The experi-ments carried out at the Hungarian firm Ferrobeton Ltd. concentrated on exploring the possibility of using glass waste in the cement concrete recipe of vehicle restraint systems in order to release reflection in its material and thereby to increase road safety. In the concrete recipe, the maximization of both usability and reflectivity were aimed to. However, it was also necessary to make sure that the resistance to mechanical and environmental loads could not be worse than in the case of reference (basic) mixture recipe. The quality and type of concrete surface finishing is a financial and also lifetime design issue. The surface treatment method actually applied basically influence the light properties (gleam, reflectivity) of concrete surface as a significant traffic safety parameter.

Keywords: glass waste, vehicle restraint system, road restraint system, traffic safety, compressive strength, reflectivity

1 Introduction

One of the main challenges of our days is the providing environmental sustainability for future generations. It is well-known that there are many advantages of applying glass waste in concrete mixtures. A Hungarian test series was aimed at scutinizing the possibility of partially replacing natural concrete aggregate by broken glass in vehicle restraint systems in order to improve glitter and reflectivity of the material, and thereby to increase road safety. At the same time, it was also compared the compressive strength values of the mixture of new recipe with those of the reference mixture.
First a short international review is presented on the experiences obtained with the use of glass waste in cement concrete surface, then the main features of vehicle (road) restraint systems are shown briefly. Finally, the research aim, the methodology followed, the laboratory test results and the conclusions drawn are introduced.

2 Glass in concrete

Glass aggregate can replace part or all of the sand and gravel in concrete, for effects that range from colourful terrazzo, to granite- or marble-like finishes, to concrete that reflects light like a mirror. Glass aggregate can even be used to produce concrete that literally glows [1]. Glass aggregate almost always comes from recycled glass, saving landfill space and requiring no mining. Glass aggregate is typically graded by colour and size. Sizes can range from six-inch (some 190mm) rocks to gravel-sized pieces to a fine talc-like powder. Polishing, grinding or other exposed aggregate techniques are employed to reveal the glass. Also glass can be seeded on the surface and then exposed.
Coloured glass can be coordinated with the matrix of integrally coloured concrete. In general, lighter colours of glass are used in darker matrixes, and vice versa. A dark brown glass in a dark brown matrix can have an appeal all its own. Mixing light and dark colours of glass will give you a terrazzo effect. If you use clear glass aggregate, it will take on the colour of the matrix, and it will add the most depth. Since glass is acid resistant, acid staining will colour the surrounding matrix without affecting the colour of the aggregate.
Finely ground glass can add background colours to the matrix. Using finely ground clear glass in place of sand can make for purer colours of concrete. Finely ground glass also lends itself to highly polished finishes. A marble or granite look can be attained by putting a high polish on concrete made with finely ground, earth-toned glass aggregates.
As for strength, glass aggregate can match, exceed or fall short of traditional aggregates, depending on size. Studies have found that very finely ground glass aggregate used in place of sand actually increases the strength of the concrete, whereas gravel-sized glass aggregate decreases strength. Mixing fine and coarse glass aggregates can have a net effect of zero, rendering concrete no stronger or weaker than that mixed with traditional sand and gravel.
Glass aggregate can be obtained from a variety of sources. Locally, recycling centers may have cullet — crushed bottles and other glass — cleaned and sorted by size and colour. Nationally, specialty glass manufacturers melt down bottles and window glass to produce glass aggregate for terrazzo floor contractors, landscapers and decorative concrete artisans. Specialty glass aggregates made from recycled glass that is melted down and re-formed give you a different look than plain old crushed glass. Crushed bottles and window glass tend to be flat, with parallel sides, whereas specialty glass aggregates can have fuller, more irregular shapes, like crushed gravel.
When using glass aggregate outdoors, or anywhere else that the concrete will be exposed to moisture, beware of the dreaded alkali-silica reaction, an unhappy phenomenon. The reaction may happen right away or it may take 20 years, but in either case it can cause cracking. Any source of moisture can set it off, including mopping, using excess water in the concrete mix, and so forth.
But the alkali-silica reaction can be prevented since the reaction can be avoided if the glass is ground finely enough to pass through 50-mesh or smaller screen. They also found that the mineral admixture metakaolin will suppress the reaction — an effective but expensive solution. It was also found that green glass does not cause the alkali-silica reaction, due to the chromium oxide used to get the green color.
As part of a comprehensive scientific research project, HERING [2], together with the University of Siegen, over a period of several months attempted to discover a method of manufacturing permanent and stable concrete mixtures that included real glass. The optimum mixture ratio was discovered, with eight of the tested glass types being suitable for use and meeting the requirements of the DAfStb Alkali Guideline. HERING's specially-developed „concrete-glass recipe“ also fulfils the defined criteria of DIN 1045 for structural concrete components. The tested glass types are naturally also suitable for various types of surface treatment, including acidification, washing, blasting and grinding.

The recycling of waste glass poses a major problem for municipalities nationwide. New York City alone collects more than 100,000 tons annually and pays Material Recycling Facilities (MRF's) up to $45 per ton for the disposal of the glass, commingled with metals and plastics. While the MRF's have little difficulty with profitably disposing of the metals and plastics, markets for recycled glass are limited to nonexistent. The use of crushed waste glass as aggregate in concrete is problematic because of the chemical reaction between the alkali in the cement and the silica in the glass. This alkali-silica reaction (ASR) creates a gel, which swells in the presence of moisture, causing cracks and unacceptable damage of the concrete. It can also occur in regular concrete, if the natural aggregate contains certain reactive (typically amorphous) silica. ASR in uranyl acetate treated concrete, visualized under UV-light This phenomenon is particularly vexing, because it is a long-term problem, and the detrimental consequences may not show for years. Predictions of the susceptibility of naturally occurring aggregates are uncertain, as they require accelerated laboratory tests, which are of limited reliability.
There is not much uncertainty with regard to ASR if waste glass is used as aggregate in concrete. Research at Columbia University [3] has focussed on a basic understanding of the ASR phenomenon and on searching for ways to avoid it or to mitigate its detrimental consequences. Some of the techniques developed so far or under investigation are: grinding the glass fine enough; replacing part of the cement by metakaolin; applying protective coatings to the glass particles; modifying the chemical formulas for the glass; use of lithium in glass powder. Old glass may find use in new, better concrete although glass is thought of as being relatively eco-friendly because it‘s recyclable, the fact is that a lot of it doesn‘t get recycled – this is particularly true of small fragments, that are too fiddly to sort. Australian researchers started with various pieces of non-recyclable glass, then ground them up into a coarse powder [4]. They then utilized that powder as an aggregate in polymer concrete, in place of the sand that‘s normally used. Polymer concrete itself substitutes polymer resin for cement as a binding agent, and is typically used in applications such as waterproof flooring. It was found to be significantly stronger than its traditional sand-based counterpart. Additionally, because sand has to be mined, washed and graded, it was determined that use of the ground glass resulted in lower concrete production costs. While a shortage of appropriate sand has been predicted, there are stockpiles of old glass that are just sitting around unprocessed.

3 Road (Vehicle) Restraint Systems

Road Restraint Systems are an essential component of a modern road infrastructure and constitute one of the most important life-saving devices available to public authorities and road operators [5]. They represent an immediately available solution that can, in addition to saving lives, significantly reduce the accident related health care cost.
Road restraint systems can be also considered as the most “flexible safety device” possible: they are designed to withstand a crash from different kind of vehicles in different conditions. According to their containment level, they are tested both for a small city car or a large family car; small to heavy truck or coach, with the possibility to equip it with a motorcyclist protection system (MPS) to further extend this protection to a particularly affected class of vulnerable road users.
An example of the effectiveness of those solutions is the analysis carried out by the German Land of Hessen. The erection of a median and a road side barrier in two identified ‘black spots’ in the German road network resulted in a decrease in accidents with injuries by 65 % and 91 % respectively, while, at the same time, reducing the annual accident costs by 70 % and 88 %, thus leading to a global yearly saving of € 1.214.000.

The existence of protective barriers on road can reduce fatalities up to a factor of 4 when compared to collisions against other road obstacles. Actually, the presence of a road restraint system appears to offer the highest level of protection compared to accidents against obstacles in non-urban environments [6].
The European Norm 1317 for Road Restraint Systems was created in 1998 and lays down common requirements for the testing and certification of road restraint systems in all countries of the CEN. The introduction of EN 1317 represents a significant change in terms of safety and quality for European drivers insofar that it establishes an EU market based on performance, replacing previous 'prescriptive based systems based on empirical experience'. In practical terms, this means first, that new barriers placed on European roads can offer guaranteed levels of safety and secondly, that the level of guarantee is the same across the whole of the EU, i.e. a single market for safety barriers.
While the EN 1317 for Road Restraint Systems guarantees common testing methods for road restraint systems across EU Member States, it is up to national governments to decide the level of protection on their road network. As a result, European drivers are confronted with varying levels of road restraint systems protection on the European motorway network despite the fact that speed limits and driving conditions are very similar.
As for cars, roadside obstacles represent a high danger for motorcycles as well: an impact against a tree, or a fall from a cliff, is dangerous for the 4-wheel vehicle as well as for the 2-wheel ones. Additionally, standard road restraint systems are designed to redirect cars and trucks and thus, are not designed to prevent the impact of motorcyclists against obstacles. On the contrary, they represent an obstacle in themselves.
For more than 20 years, road restraint systems manufacturers have invested and carried out research and development on dedicated products in order to increase the safety also of motorcyclists. Since 2008, CEN (European Committee for Standardization) and its members have been working on the development of a European standard for the testing of those products, which has been approved as a Technical Specification (TS 1317) and published..
While motorcycle riders often advocate the removal of standard safety barriers, the fact is that such a decision would increase the risk of serious collisions for all users, given that their drivers would be unprotected against roadside obstacles. In the view of the ERF, the use of high protection (HIC ‹650) TS 1317 - part 8 [7] tested products would be the best solution to guarantee a higher motorcyclist safety, and to maintain the existing safety level for 4 wheel vehicles.
While placing better performing barriers on Europe's motorways can undoubtedly improve driver safety, the potential safety gains by acting on Europe's rural roads can be said to be substantial given that 56 % of Europe's fatalities occur on rural roads compared to only 6 % on motorways, which can be attributed also to the existence of guard rails.
As the previous examples of 'black spot management' have demonstrated, placing barriers on secondary/rural roads can have impressive results at a relatively low cost. These findings are also supported by the European Road Assessment Programme, which found that a median barrier on a rural road can help reduce the kinetic energy of a run-off crash, thus decreasing the risk factor by approximately a factor of 310.
The ERF believes that, at a time of economic constraint, acting on passive safety solutions that are already available can represent one of the most cost-effective solutions for public authorities and citizens alike. In this respect, it welcomes the European Parliament's Transport Committee's Report on European Road Safety Programme 2011-2020 and the paragraph 26 [8]. It'calls on the Member States to take prompt action (including replacing the existing guard rails) to refit dangerous stretc-hes of road with rails with upper and lower elements as well as with other alternative road barrier systems, in accordance with Standard EN 1317, in order to lessen the repercussions of accidents for all road users.

Some of the road restraint systems can be customised to provide optimum solutions and are designed and tested according to European standards [9]. The main function of crash cushions is to prevent lethal damage to car passengers when crashing into static objects. The modular structure of the parapets contributes to quick installation and low cost.

4 Research aim

The aim of the experiments carried out at Ferrrobeton Ltd. (Hungary) was to identify the amount (share) of glass aggregate in the concrete vehicle restraint systems that ensures favourable visual appearance („lightness") to the elements, without influencing negatively the mechanical properties of concrete structure.

5 Methodology

The steps of research methodology chosen and carried out were as follows:
- determination of basic concrete recipe,
- identification of the aggregate fractions to be replaced eventually, partly by crushed glass,
- choosing the shares of above aggregate fractions for possible glass waste replacement,
- identification of the recipes creating favourable visual appearance of glass concrete surface,
- producing specimens of concrete mixtures with promising recipes,
- measuring compressive strength of specimens after 10 and 28 days,
- producing and testing various surface treatment methods on various samples,
- identification of time need and surface gleam of various surface treatment methods,

6 Laboratory test results

Table 1 shows the cement concrete mixture recipe that was considered as a basic (reference) one for the research on the optimum use of glass aggregate in vehicle restraint systems.

Table 1 "Basic" cement concrete mixture recipe

Constituent	Unit weight [kg/m^3]
Aggregate	1950
Cement I 42.5 R	400
Water	195
Additive (Glenium 51)	4

Based on the analysis of relevant international literature, two – 0/4 mm and 4/8 mm – glass fractions were tested for the eventual replacement of natural aggregate ones. The first version of „mixing plan" consisted of the replacement of 5 % - 10 % - 25 % - 50 % of the fractions mentioned by broken glass. However, based on the results of first concrete mixture variant, it became apparent that the use of 0/4 mm glass aggregate cannot meet the expected, favourable visual appearance of cement concrete surface. That is why susequently the experiment concentrated on the replacement options of 4/8 mm fraction by glass aggregate.
Further laboratory tests proved that the entire (100 %) replacement of 4/8 mm fraction by broken glass could ensure the expected positive changing in the surface reflective appearance. From that pont on, the experiment carried out was driven towards the grain size distribution of the cement concrete mixture. Thus concrete specimens with poorly graded grain

(particle size) distribution of concrete aggregate were produced in which various fractions and shares were replaced by glass. Table 2 presents the 10-day and 28-day laboratory compressive strength test results of the samples with various glass aggregate contents compared to that of reference value (basic recipe).

Table 2 Compressive strength values of concrete samples with various broken glass aggregate shares

Replaced fraction share(s) by glass	Compressive strength [N/mm²]	
	10 days	28 days
(reference)		69.0
5% 0/5 mm	55.1	64.8
10% 0/5 mm	59.0	66.5
25% 0/5 mm	62.1	65.5
50% 0/5 mm	57.2	65.1
5% 4/8 mm	63.2	69.5
10% 4/8 mm	61.3	70.0
25% 4/8 mm	61.1	65.5
50% 4/8 mm	62.7	67.8
100% 4/8 mm	62.1	66.8
100% 4/8 mm + 50% 8/16	60.8	66.1

The laboratory test values shown in Table 2 have proved that the increasing addition of broken glass as cement concrete aggregate does not have a negative effect on the compressive strength of the mixture.
After the positive results of water tightness (6 mm) and freeze-thaw scaling tests (XF4) of concrete specimens, trial factory production had started not showing any technological difficulties in casting, application and formworks.
In such a way, it was clearly demonstrated that the partial replacement of the cement concrete mixture aggregate of vehicle restraint systems does not worsen the mechanical properties while offering an environmental-friendly solution by the use of a waste material instead of natural material. The next issue to be tested was the analysis of the abilities of various surface treatment procedures on the reflectivity of cement concrete surface as an importont traffic safety feature. The following surface treatment methods were covered: grinding, sandblasting and pressurewashing. Table 3 introduces not only the measure of reflectivity (gleam) in each case but also the time need of various surface treatment methodologies. (The reflectivity of concrete surface was characterized by the number of gleaming aggregate grains per unit area).

Table 3 Time needs (working hours) and reflectivity (gleam) of the unit area of sample surfaces treatad by various procedures

Surface treatment method	Working [hours/m²]	Reflectivity [gleaming grains/m²]
Grinding	1.5	4
Sandblasting	1.0	6
Pressurewashing	1.0	4

Of course the reflectivity attained is not comparable to glass beaded or prismatic solutions, but it had never targeted. However, it has been clearly demonstrated that utilization of glass aggregate in the recipe of restraint systems can achieve a glitter level that makes it detectable for the drivers even without light effect coriginated by road vehicles.

7 Conclusion

Laboratory and early site experiments on the partial replacement of aggregate of cement concrete vehicle restraint system by broken glass were performed by Ferrrobeton Ltd. (Hungary). The results proved that the mechanical properties (actually compressive strength) of concrete structure were not negatively influenced by the addition of glass as aggregate. The use of this kind of waste material is obviously an environmental friendly solution by reducing deposited by-product, and decreasing the need of the use of primary aggregates with less greenhouse gas emiss-ion. As an additional advatage, it was also shown that the glass aggregate grains on the concrete surface of vehicle restraint elements ensure a favourable visual appearance, a kind of glitter after appropriate surface treatment increasing the traffic safety effect of vehicle restraint systems. .

References

[1] David, T.: How to use Glass as Aggregate in Concrete, Concrete Decor, 5 (2005) 3. June/July 2005.

[2] Glass agg-regate in architectural concrete. Hering Archiectural Concrete, https://www.heringinternational.com/en/products-services/architectural-concrete/reference-projects/

[3] Concrete Materials Research at Columbia University, http://www.columbia.edu/cu/civileng/meyer/download/glass.pdf

[4] Zheng, K.: Recycled glass concrete, Eco-Efficient Concrete, 2013, pp. 241-270.

[5] Coxworth, B.: Old glass may find use in new, better concrete, New Atlas, 2019

[6] Road Safety and Road Restraint Systems. A flexible and cost-effective solution, European Union Road Federation, 2019.

[7] The European Technical Specification CEN TS 1317-8 Motorcyclist Protection Systems, 2012.

[8] Report on European road safety 2011-2020, Committee on Transport and Tourism of European Parliament, Brussels

[9] https://www.saferoad.com/businessareas/road-restraint-systems/

USAGE OF NEW MATERIALS DURING REHABILITATION OF ROAD STRUCTURES USING COLD RECYCLING TECHNOLOGY

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Abstract

Until recently, the most common way for recovering of damaged and worn asphalt pavements on Ukrainian roads remains the provision of additional reinforced layers over the old pavement with patching. However, such measures give only a short-term effect because after one or two years, the existing deformations and fractures beneath reinforced layers occur, especially in conditions of insufficient strength of the foundations. But nowadays, recycling technology of different variations became the main method of existing pavement renovation. The economic attractiveness of cold recycling technology is primarily in the reuse of existing road material for arrangement of new pavement layers, so there is no need to arrange special areas for storage and disposal of old asphalt. In addition, the use of such technology helps to minimize the harmful impacts on the environment during road repair works. The essence of cold recycling technology, which is the most widely used in Ukraine for the arrangement of a road foundation layer, is in the fact that the defective and destroyed pavement layers are strengthened directly by complex admixtures of organic (hot bitumen, bituminous emulsion, foamed bitumen) and mineral suspensions, lime) binders. Cold recycling, according to the complications of the work, is divided into several types, depending of the depth of cutting. The choice of a particular type of recovery depends mainly on the condition of the entire pavement structure, which is determined prior to the start of repair works. Optimal mis design of the organic and mineral mixture for the arrangement of the road foundation layer by cold recycling technology is also executed before the beginning of the works. Actually, the main direction of cold recycling technology research in Ukraine is the usage of new materials such as fiber - basalt or polymer, stabilizing additives (ionic or polymeric), industrial waste - slags of various types of production or other by-products. Performed studies have shown that the use of organic and mineral mixtures of optimal design with the insertion of basalt fiber increases crack resistance and durability of the arranged road foundation layer.

Keywords: road consrtructiony, cold resycling, basalt fiber

1 Introduction

In Ukraine, as well as throughout the world, asphalt concrete is the main material for the construction of road pavement. Asphalt concrete pavements during the entire service life must provide regulatory riding qualities. However, during the operation of road pavement under conditions of constantly increasing loads, wear and aging of all constituent materials occurs, which leads to the accumulation and increase of deformations, defects and destruction of road pavements.

Road repair and construction organizations in Ukraine annually perform big scopes of work to eliminate defects and deterioration of highways. Rehabilitation of pavement is carried out by different ways, methods and materials which jointly determine the lifetime, the cost and the quality, i.e. the efficiency of the repairs performed. The main purpose of these works is to ensure safe and continuous traffic of motor vehicles with the given speeds [1].

During the recent years in Ukraine, for arranging the non-rigid pavements, the cold recycling method was used on the basis of machines of Wirtgen Group (FRD) - a recycler WR2500, a concrete –water suspension preparation plant WM1000 and a milling cutter [2]. Usage of the milling cutters of Wirtgen company production allows milling the existing asphalt pavement resulting in the formation of so called asphalt crumbs.

The reclaimed mixture designed by the authors is used on the roads of Ukraine and includes asphalt crumb, cement, bituminous emulsion and basalt fiber in the volume of up to 5 % of total mass of mixture. The result is an organo-mineral dispersion-reinforced mixture based on the reuse of milled asphalt concrete.

The three-dimensional chaotic distribution of basalt fiber in the mixture improves the physical and mechanical properties of the material providing high crack resistance of the pavement, increased resistance to shock and dynamic loads, wear resistance, that is, increase the service life. The tensile strength at bending limit, corrosion and weather resistance is increased. The bearing capacity of the pavement is increased by 1.2-1.5 times, and the operation durability by 40-50 %.

The source material for basalt fiber is basalt rock which is a fine-grained, effusive raw material of volcanic origin. Basalt fiber is obtained by melting of basalt stone and drawing fiber from the resulting melt and subsequent splitting into small fibrous elements. The obtained fibrillated fibers that had a circular transverse section, are cut into pieces of various lengths - mono- and multifilament fibers - fiber. A large number of types of fibers are known: mineral, basalt, diabase, metal, cellulose, synthetic. All types of fibers differ in nature, size, application and impact on material properties. The main technical characteristics of various types of fibers are shown in Table 1. As can be seen from the data in Table 1, basalt fiber, in comparison with fibers made from synthetic materials, is characterized with high tensile strength and low elongation at break, which will allow creating a spatial grid of dispersed reinforcement in the binder when mixing, improving structural and mechanical, technological and operational properties of organic and mineral mixture with basalt fiber.

Table 1 Characteristics of the fibers type that are used

Fiber	Density [g/m³]	Elasticity modulus [MPa]	Tensile strength [MPa]	Elongation at break [%]
Polypropylene	0,9	3500-8000	400-700	10-25
Polyamide	0,9	1900-2000	720-750	24-25
Polyethylene	0,95	1400-4200	600-720	10-12
Acrylic	1,1	2100-2150	210-420	25-45
Nylon	1,1	4200-4500	770-840	16-20
Viscose extra strong	1,2	5600-5800	660-700	14-16
Polyester	1,4	8400-8600	730-780	11-13
Cotton	1,5	4900-5100	420-700	3-10
Carbon	1,63	280000-380000	1200-4000	2,0-2,2
Carbonic	2,0	200000-250000	2000-3500	1,0-1,6
Glass	2,6	7000-8000	1800-3850	1,5-3,5
Asbestos	2,6	68000-70000	910-3100	0,6-,07
Basalt	2,6-2,7	7000-11000	1600-3200	1,4-3,6
Steel	7,8	190000-210000	600-3150	3-4

2 The main part

For research, basalt fiber obtained at the PJSC NVP "Teploizolyatsiya" (Ukraine) with a diameter of 14 microns, a strength of 1900 MPa, and a density of 2.7 g / cm^3 was used. Due to the chemical composition, basalt fiber is characterized by high acid and alkali resistance, low wettability, which ensures high adhesion to the molecules of the mineral and organic binder, and, accordingly, to the grains of stone material, as well as evenly distribution in the mixture, that is, absence of clumping during mixing (hedgehog form formation). The general view of fiber is shown (Figure 1).

Figure 1 Basalt fiber, the fiber length is 20-25 mm and the diameter is 13-20 mm

Organic and mineral mixture was prepared according to the following procedure: PC 400 cement, bitumen emulsion (cationic, of average decomposition) and basalt fiber were added to the mixture of milled asphalt crumb, followed by thorough mixing. After preparing the samples at laboratory, the compaction load within the range from 20 MPa to 40 MPa was applied for approximately 3 minutes. Schematic representation of the organic-mineral mixture with basalt fiber is shown (Figure 2).

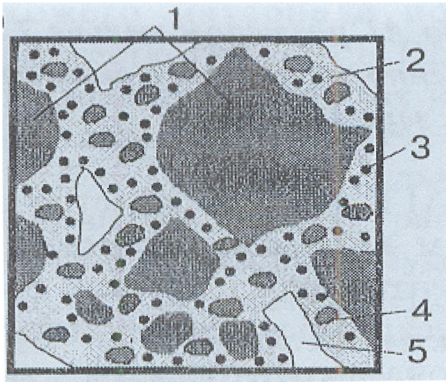

Figure 2 Structure of dispersed-reinforced organic-mineral mixture (1 - Asphalt crumb; 2 - film of cement paste with the inclusion of droplets of emulsion bitumen; 3 - basalt fiber; 4 - grains of asphalt crumb with size less than 2 mm; 5- air voids)

As a result of the studies, the authors found that the introduction of basalt fiber has a positive impact on the structure formation of the organic and mineral mixture, the number of bonds between the particles (grains) of asphalt crumb is increased, which increasing the deformation-strength characteristics and the density of the composite material.
The strong spatial structure of the mixture, which is occurred by the introduction of fiber due to the interaction of the dispersed-reinforced binder with particles of the aggregate, allows evenly distributing the stresses from the moving load and increasing the shear stress resistance of the arranged pavement layer. The results shown an increase in the ability of the dispersion-reinforced mixture with basalt fiber to resist rutting and cracking were obtained. Comparative tests of organic and mineral mixtures with the addition of basalt fiber and without it on rutting on the "wheel" device are shown in Figure 3.

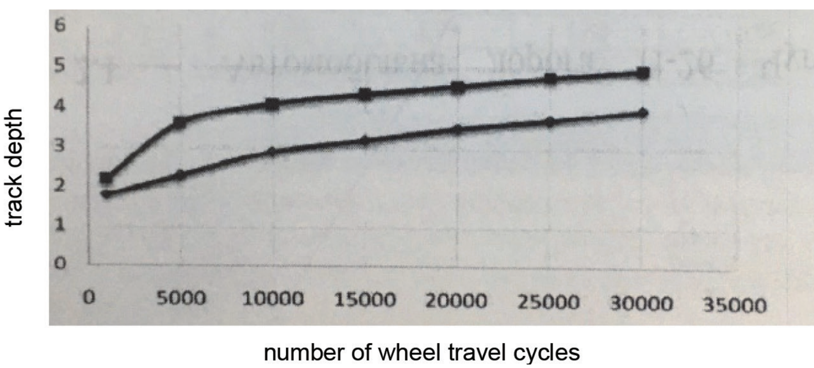

Figure 3 Number of wheel travel cycles (1 - polymer-bitumen binder mixture, 2 - mixture on bitumen with additive Forta)

It is known that the structure of the recycled mixture with the addition of a bitumen emulsion (cationic medium decomposition) is formed for a rather long time. The introduction of basalt fiber accelerates the process of structure formation of the mixture due to chemical interaction with bitumen droplets during the decomposition of the emulsion, which accelerates the layer formation and allows opening the movement of vehicles on the regenerated pavement almost immediately after the compaction.
The length of the basalt fiber of 20-25 mm is typical for discrete offcut of polymer or carbonic materials, which are used for dispersed reinforcement.
The use of basalt fiber with a length of 20-25 mm leads to an increase in the number of micro-particles of a binder connected by one offcut of fiber with the formation of a spatial grid, the nodes of which are the connected structures of sufficient strength, which is due to the strength of the basalt fiber and expressed by high values.
As a result, the strength of the dispersed-reinforced organic and mineral mixture is increased not only in compaction, but also in tension at bending.
Physical and mechanical characteristics of the designed organic-mineral mixture with basalt fiber, depending on different degrees of compaction load, are given in Table 2.

Table 2 Physical and mechanical parameters of the organic-mineral mixture with basalt fiber

Fiber	Density [g/m^3]	Elasticity modulus [MPa]	Tensile strength [MPa]	Elongation at break [%]
Polypropylene	0,9	3500-8000	400-700	10-25
Polyamide	0,9	1900-2000	720-750	24-25
Polyethylene	0,95	1400-4200	600-720	10-12
Acrylic	1,1	2100-2150	210-420	25-45
Nylon	1,1	4200-4500	770-840	16-20
Viscose extra strong	1,2	5600-5800	660-700	14-16
Polyester	1,4	8400-8600	730-780	11-13
Cotton	1,5	4900-5100	420-700	3-10
Carbon	1,63	280000-380000	1200-4000	2,0-2,2
Carbonic	2,0	200000-250000	2000-3500	1,0-1,6
Glass	2,6	7000-8000	1800-3850	1,5-3,5
Asbestos	2,6	68000-70000	910-3100	0,6-,07
Basalt	2,6-2,7	7000-11000	1600-3200	1,4-3,6
Steel	7,8	190000-210000	600-3150	3-4

Analysis of the data indicates that the increase of the compaction load decreases the level of organic and mineral mixture water saturation, increase the average density, and increase the compressive strength and the water resistance coefficient (both, short-term and long-term).
The optimal choice of the binder's quantity of cement, emulsion, as well as basalt fiber was confirmed by determining the dependence of frost resistance coefficient on different quantity of binder, as shown (Figure 4).

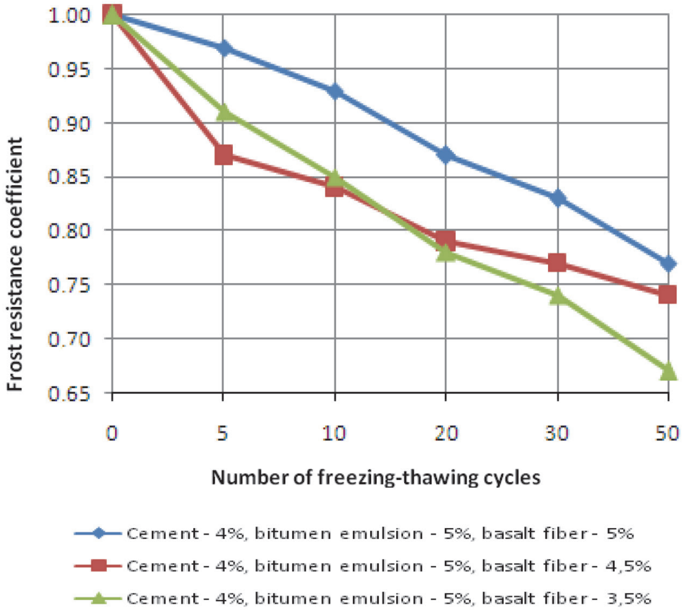

Figure 4 Dependence of frost resistance coefficient on different quantity of binder

As can be seen from the data shown in Figure 4, the organic and mineral mixture of 4 % cement, 5 % cationic bitumen emulsion and 5 % basalt fiber has the highest values of the frost resistance coefficient.

3 Conclusions

The use of organic and mineral mixtures, consisting of milled asphalt concrete with the addition of cement, cationic bitumen emulsion and basalt fiber for the arrangement of the top layer of the pavement during works by the cold recycling method, allows obtaining a durable, crack-resistant, water- and frost-resistant strength structural layer of non-rigid pavement which is meet the modern requirements of road traffic.

The use of an organic and mineral mixture based on milled dispersed-reinforced asphalt concrete will increase the volume of repair work with a significant decrease of their material consumption due to the reuse of existing asphalt concrete will provide high performance of the arranged pavement layer.

The introduction of basalt fiber into the organic and mineral mixture promotes the formation of a durable homogeneous structure of the material, accelerates the process of layer formation and does not complicate the technological process of preparation, which is a positive factor in its use.

References

[1] Ilchenko, V.V.: Restoration of pavement using cold milling technology, Collection of scientific works, branch mechanical engineering, construction, 18 (2006), Poltava: PoltNTU

[2] Draft Guidelines: Road pavement recycling, PIARC Version, August, 2002.

[3] Sasko, M.F.: Cold Recycling, Its Benefits and Prospects, Avtoshlyachovyk Road Worker of Ukraine, 2 (2004), pp. 37- 40

[4] Golovko, S.K.: Cold Recycling - Effective Pavement Rehabilitation Technology, Avtoshlyachovyk (Road Worker) of Ukraine, 6 (2003), pp. 34-35

[5] Kostelev, M.P.: Cold Recycling Technology, Road technique, 3 (2004), http://library.stroit.ru.

[6] Dolgilevich, Yu.P.: Experience in applying road pavement cold recycling technology in USA, Road technique, 1 (2005), http://library.stroit.ru.

[7] Malyutin, A.: Technique for cold recycling, 7 (2008), http://www.os1.ru.

[8] Livitina, V.V.: Cold recycling - an effective technology for the rehabilitation of asphalt pavement of airfields and highways, Bulletin of construction machinery, 9 (2007), http://stroy.dbases.ru.

[9] Turenko, F.P., Filatov, S.F., Shipicin, V.V.: Improving the efficiency and environmental friendliness of road asphalt concrete pavement repair by cold regeneration using slow hardening mineral and organic additives, Omsk Scientific Herald, 3 (2004) 36, pp. 89-91

[10] Barackh, G.S.: The influence of the structure of asphalt granulobeton on its properties, 2001.

[11] Prokopets, V.S., Filatov, S.F., Ivanova, T.L., Tarasova, M.V., Pomorova, L.V.: Restoration of asphalt concrete pavements by cold recycling and chemical additives, Bashkir Chemical Journal, 13 (2006) 5, pp. 61-65

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RECYCLING CONSTRUCTION AND DEMOLITION WASTES WITHIN HYDRAULICALLY BOUND MIXTURES FOR ROAD PAVEMENTS

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Abstract

The recycling of Construction and Demolition Waste (CDW) is an actual challenge regarding the construction industry because of the increasing volumes worldwide produced and the related environmental impacts. In this regard, a suitable application can be developed in the road construction field, in particular for the production of hydraulically bound mixtures for road subbase and foundation layers. In this sense, the reuse of CDW can strongly enhance the environmental sustainability of road construction thanks to the achievable savings of natural resources such as the mineral aggregates obtained from quarry operations. Indeed, the utilization of a CDW as aggregate must ensure the production of mixtures with adequate mechanical and environmental characteristics. Therefore, the herein paper presents the results of an experimental characterization aimed at assessing the suitability of CDW for the production of hydraulically bound mixtures for road pavements. In particular, the main mechanical properties of some mixtures including different percentages and gradations of CDW were analysed and compared with the main technical prescriptions and classification criteria indicated by the reference European standards. Basic properties and production processes of the CDW materials were also studied to determine their effects on the optimum binder and water contents of the mixtures. The research clearly demonstrated that the use of a preliminary-washed CDW coarse aggregate was able to enhance the overall structural properties and the water resistance of mixtures.

Keywords: construction and demolition waste, cement treated mix, road materials, European standard classification, laboratory characterization

1 Introduction and research objective

The reuse of Construction and Demolition Waste (CDW) is an actual challenge concerning the construction filed and the building industry because of the increasing volumes produced worldwide. Concerning this point, Europe generated more than 800 million tonnes of CDW in 2016, overtaking the annual production of the United States in the same year (700 million tonnes). Today, the largest and populated countries are able to create more than 10 billion tonnes of CDW by year [1]. Statistical projections estimated also that the production trend is increasing with a non-negligible rate [2]. Generally, CDW consists in a debris that can comprise steel, wood, drywall and plaster, brick and clay tile, asphalt shingles, concrete and asphalt concrete as by-products resulting from construction, renovation and demolition activities [3]. Several CDW-related environmental impacts (land and water pollution, space occupation, greenhouse gas emissions, energy consumptions, etc.) have been widely documented in literature [4], [5]. Actually, CDW is recycled as secondary aggregate in many construction fields (buildings, roads and bridges and other sectors). A suitable application can be developed in road constructions, in particular within hydraulically bound mixtures for road subbase and foundation layers [6], [7]. Such solution can strongly enhance the environmental sustainability of roads, thanks to the achievable savings of natural resources such as the mineral aggregates obtained from quarry operations that are requested for the above-mentioned layers. However, given the extreme heterogeneity of CDW in terms of nature, origin, composition and gradation [8], [9], its recycling in pavements must be adequately designed and executed considering the characteristics requested by the final mixtures for road layers (mechanical performance, environmental properties, etc.). Given this introduction, the present study aimed at evaluating the possible use of different CDW aggregate fractions to produce suitable cement bound mixes for road subbases and foundations. To accomplish this goal, a laboratory plan was arranged to test different mixtures including various CDW stockpiles and evaluate their structural performance in comparison to the main technical prescriptions and classification criteria indicated by European standards. EN 14227-1 and EN 14227-15 were assumed as references for the mechanical properties and the water susceptibility of the mixes, respectively. Fine and coarse fractions were used to estimate the role of aggregate size and the influence of granulometric distribution. Moreover, the use of stockpiles subjected to different preliminary treatments allowed establishing the best practices to reuse the CDW coming from common construction sites in order to maximize the materials performance for road pavements deep layers applications.

2 Materials, sample preparation and test methods

Four CDW aggregate stockpiles were selected for the experimentation. In particular, the gradation of two stockpiles was between 0 and 6.3 mm (they were named “fine aggregates”), whereas other two had gradations between 0 and 31.5 mm (they were identified as “coarse aggregates”). All CDW derived from an Italian construction site. The two fine aggregates had the same origin and were initially subject to a preliminary treatment process with the objective to remove metallic, glass and organic residues. Then, the first one was utilized as it was (hereafter coded F); the second one (hereafter named F_w) was additionally washed with water in order to dispose further impurities (mainly, cohesive or granular particles with size ‹ 0.100 mm were eliminated). In the case of coarse stockpiles, the same procedure was followed: after the initial selection, a non-washed coarse aggregates (hereafter coded C) and a washed one owing the same origin (hereafter named C_w) were obtained. Even no additional information about stockpiles nature was available (e.g., fine part plasticity index), clear effects due to water-washing were noticed in their final gradations. In this perspective, Figure 1 illustrates some images of such aggregates, whereas Figure 2 presents their granulometric distributions. Combining the aggregate portions, four cement bound mixtures were produced: their composition was chosen in order to produce different granulometric combinations complying with specifications proposed by EN 14227-1: in particular, the grading envelope for cement bound granular mixtures of type 1 (0/31.5) and category G1 was considered. The codes of the produced mixes and their proportions are given in Table 1, whereas Figure 3 presents the resulting mixture lithic matrixes (plots are split for the sake of clarity). A Portland cement type II/B-LL 32.5 R (EN 197-1) was utilized as binder. Its dosage, together with the water content, was optimized according the mix design procedure described below.

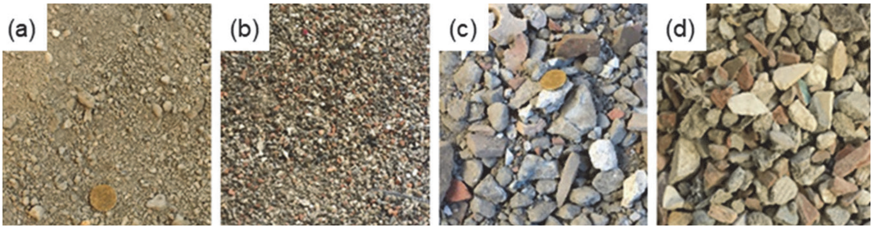

Figure 1 Images of the selected CDW aggregates: a) F, b) F_w, c) C, d) C_w

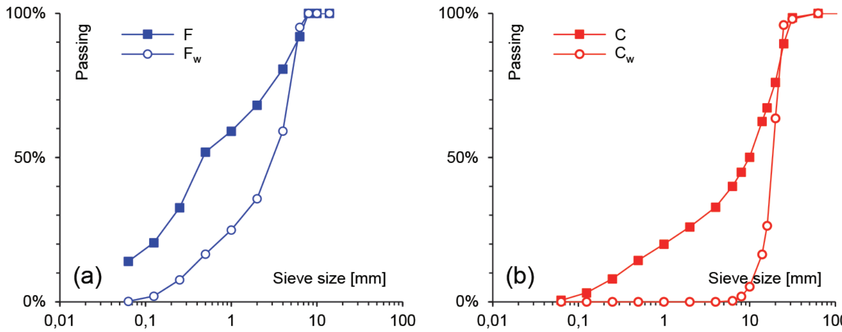

Figure 2 Gradations of selected stockpiles: a) fine aggregates F and F_w, b) coarse aggregates C and C_w

Table 1 Mixture compositions and proportions

Mix code	**Constituents**			
	F	**F_w**	**C**	**C_w**
FC	30%	-	70%	-
FCC_w	20%	-	70%	10 %
FF_wC_w	20%	30%	-	50 %
FF_wCC_w	15%	20%	30%	35 %

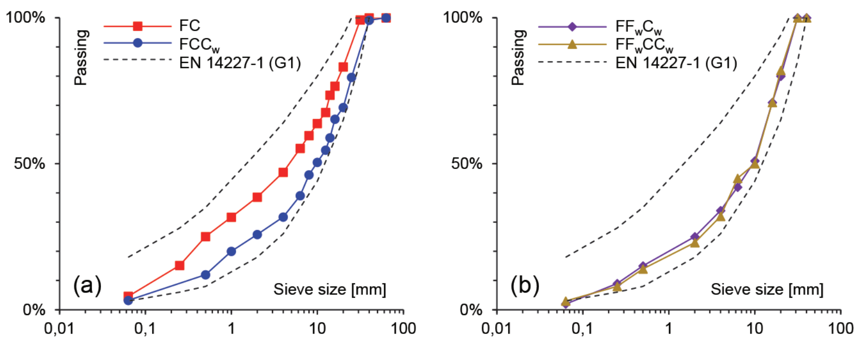

Figure 3 Final gradations of mixtures:a) FC, FCC_w, b) FFwCw, FF_wCC_w

Cylindrical 150 mm-diameter specimens were produced in the laboratory for each mixture through the Proctor modified procedure (reference standard EN 13286-50). In the early experimental stage, samples were replicated using different cement (c) and water (w) contents to optimize the mixtures. Three replicates for each cement-water dosages were compacted. The optimization was then based on the mechanical properties exhibited by specimens (compressive and tensile strengths at 14 days). Once obtained the optimum c/w contents, further samples were reproduced for the selected mechanical characterization. First, indirect tensile strength tests at 7 days were performed determining R_{ti} parameters and the direct tensile strengths were estimated using the standardized relationship $R_t=0.8\cdot R_{ti}$ (reference standard EN 13286-42). Then, compression tests were executed to determine strength R_c (reference standard EN 13286-41). Such R_c was coupled with compressive stiffness tests and was used to calculate the modulus of elasticity E_c of the mixtures, then to estimate the modulus at 7 days using the standard relation $E=E_c$ (reference standard EN 13286-43). Overall results were utilized to classify the cement bound mixes according to the specifications given by EN 14227-1. The final step of the study concerned the study of the water influence in the mechanical properties of the materials. To this purpose, laboratory specimens (produced at c/w optimum content) were classified using EN 14227-15 standard: after four days of water immersion, they were analysed in terms of compressive strength ratio (R_i/R between cases with and without immersion, according to EN 13286-41), linear swelling LS (reference standard EN 13286-47) and CBR (reference standard EN 13286-47). EN 14227-15 applies to soils rather than to aggregates, but it was used in order to have further criteria suitable for characterizing and classifying the materials.

3 Results and discussion

3.1 Mix Design

The mix design of each mix consisted in determining the optimum Portland cement dosage and the optimum water content: the results are presented in Table 2 whereas the following Figure 4 illustrates an example of the procedure applied to FF_wCC_w mixture (compressive and tensile strength optimizations presented in Figures 4a and 4b respectively are considered to determine the final average adopted water content w). Main findings indicated that the maximum water content was requested in the case of FC mix (non-washed fine and coarse aggregates). At an equal c dosage (3.5 %), FCC_w (mix with washed coarse aggregate) had a lower optimum w (7.3 %). Supposing a greater presence of fine plastic particles (not eliminated by the washing) for FC, this seemed to be slightly in contrast with existing literature stating that higher clay-like particles can reduce the water requested for the optimum mix design [10]. However, such a finding should be read considering that the possible presence of porous lateritious constituents in the coarse CDW can alter the natural w equilibrium, thus also the cement hydration [11]. When washed fines F_w were included in the lithic skeleton, no evident variations in the optimum contents were detected, despite the overall different percentages of fine and coarse fractions (3.0 % of c for both mixtures, 8.2 % and 8.3 % of w for FF_wC_w and FF_wCC_w, respectively). In general, the preliminary washing of the stockpiles led to a reduction of the requested water or cement; thus, it could be read as a positive factor in the overall economy of mixtures. Otherwise, based on the proposed results, it was not possible to better identify the role of the washed fine or coarse fractions. Furthermore, no additional indications could be drawn at this stage of the study: probably, future experimentations should be addressed to compare non-washed vs. washed aggregate establishing the global proportion of mixtures, as well as to strictly determine the influence of possible lateritious parts in the stockpiles.

Table 2 Results of the mix design

Mix code	Cement c (by agg. weight)	Water w (by agg.+c weight)
FC	3.5 %	10.8 %
FCC_w	3.5 %	7.3 %
FF_wC_w	3.0 %	8.2 %
FF_wCC_w	3.0 %	8.3 %

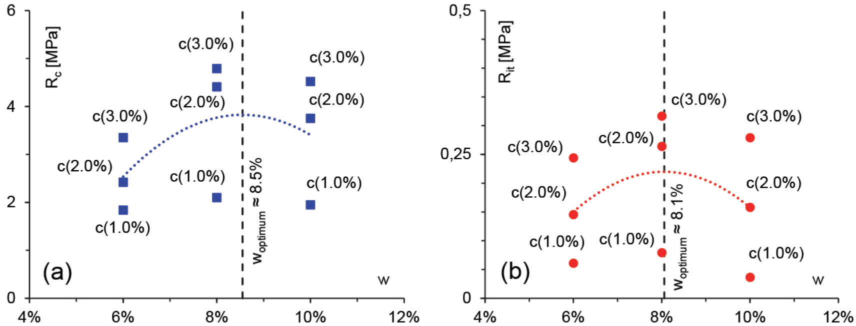

Figure 4 Mix design of FF_wCC_w mixture: R_c (a) and R_{it} (b) vs. c and w

3.2 Mechanical properties and mix classification according to EN 14227-1

The results of the mechanical characterization concerning the optimized samples (optimum c and w) are here summarized. Table 3 reports compressive strength values R_c, intended as the average of three test replicates: strength categories according to EN 14227-1 are also reported. Further information on materials can be collected also from Figure 5, in which mixes are described in terms of elastic modulus and tensile strength and are classified according to the EN 14227-1 (category based on E and R_t). As far as the compressive strength concerns, the non-washed mixture FC reasonably presented the lower resistance. Progressively including the washed portions, R_c values increased: in particular, it seemed that the addition of C_w enhanced the resistance of the corresponding mixture containing C (see the comparison between FC and FCC_w, or FF_wC_w vs. FF_wCC_w). As a result, the resistance classes (EN 14227-1) related to C_w-containing mixes were always greater than the others (FCC_w and FF_wCC_w owned $C_{2.3/3}$ labels, FC and FF_wC_w had $C_{1.5/2}$). On the contrary, the only addition of F_w (e.g., see FC vs. FF_wC_w) did not allow to improve the compressive strength class (even if R_c slightly increased passing from 2.79 to 2.83). With respect to E and R_t values, similar indications could be collected. Otherwise, analysing Figure 5, it was found that E-R_t class did not vary (T1 label for all the mixtures), regardless the increments of the values. Overall, it was finally stated that the use of washed stockpiles rather than non-washed ones was able to improve the characteristics of the mixes, but the inclusion of washed coarse aggregates (C_w) had the greatest impact (it led to the increments of compressive resistance class).

Table 3 Compressive strength results and related EN 14227-1 classes

Mix code	FC	FCC_w	FF_wC_w	FF_wCC_w
R_c [MPa]	2.79	3.45	2.83	3.02
R_c class [EN 14227-1]	$C_{1.5/2}$	$C_{2.3/3}$	$C_{1.5/2}$	$C_{2.3/3}$

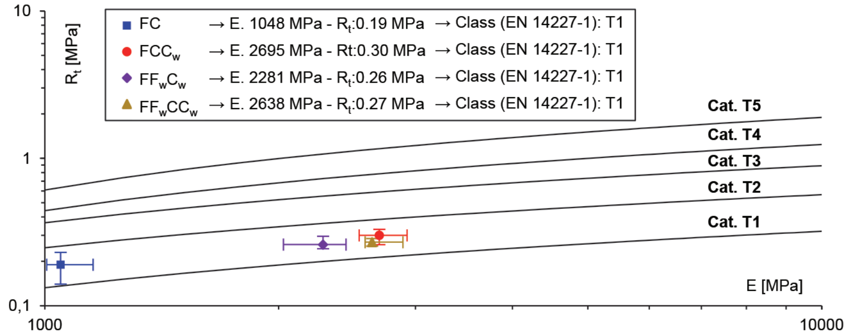

Figure 5 Figure 5. E and Rt experimental results and classification (EN 14227-1)

3.3 Water sensitivity and mix classification according to EN 14227-15

Based on the results presented in Table 4, the compressive strength ratio between dry and water-conditioned samples indicated good water resistance for all cases (R_i/R always greater than 0.8). The use of F_w (washed fine) did not seem to influence the results, whereas C_w (washed coarse) produced a significant increase of the observed parameter, regardless the coupling with washed or non-washed fines. Analysing the linear swelling behaviours, despite the variability of the values, LS parameter could be considered negligible in all mixes (always included in EN14227-15 LS_1 class). Thus, the selected construction and demolition waste can be adopted for the construction of road subbases and foundations without particular expansion issues in presence of water (no specific cares are needed for the presence of humidity in the construction site or for possible moist environments) [12]. Based on CBR results collected after 4 days of soaking in water, the role of C_w aggregate was again detected (with respect to the corresponding mixes, the C_w-containing materials had always the greater CBR). Overall, considering the water susceptibility of the produced mixes, laboratory findings indicated the possibility of producing water-resistant cement bound materials including in the gradations the washed coarse aggregate fraction.

Table 4 Water influence: classification according to EN 14227-15

Mix code	FC	FCC_w	FF_wC_w	FF_wCC_w
R_i/R [-]	0.8	1.0	0.8	1.0
R_i/R class [EN 14227-15]	$I_{0.8}$	$I_{1.0}$	$I_{0.8}$	$I_{1.0}$
LS [%]	0.055	0.060	0.030	0.025
LS class [EN 14227-15]	LS_1	LS_1	LS_1	LS_1
CBR	255	525	250	310
CBR class [EN 14227-15]	CBR_{255}	CBR_{525}	CBR_{250}	CBR_{310}

4 Conclusions and further studies

The study proposed an experimental characterization aimed at evaluating the possibility of using only CDW aggregate for the construction of road subbases and foundations. Some concerns about the preliminary treatment of CDW were also analysed. Based on the obtained results, the main findings indicated a good suitability of the selected coarse aggregate when subjected to a preliminary washing. In this case, the conventional removal of metallic, glass and organic residues was integrated with a cleaning treatment performed with water, which partially alters the original CDW grading and was reasonably able to eliminate further impurities (principally, fine clay-like particles with cohesive characteristics). Therefore, including such a stockpiles in cement bound mixes having a gradation complying with envelop given by European standards, a reduction of the optimum water content (beneficial for the mixture economy), an improvement of the compressive resistance category and some clear increases in the values of tensile strength and elastic modulus can be ensured, whatever the treatment process applied to the CDW fine. In turn, this addition provided water-resistant mixtures that did not need particular care when adopted in humid and moist construction environments. Further improvements of the research could include the modification of the mix-design in order to keep constant some constituent proportions and definitely account the contribution of the single fractions. Some ongoing studies are also trying to characterize the CDW source materials with the target to understand how the typical heterogeneity of such a waste will affect the final properties of cement bound mixes for road pavements.

References

[1] Wu, H., Zuo, J., Yuan, H., Zillante, G., Wang, J.: A review of performance assessment methods for construction and demolition waste management, Resources, Conservation and Recycling, 150 (2019), pp. 1-15

[2] Environmental Protection Agency: Advancing sustainable materials management: facts and figures report, United States, 2019.

[3] Ruiz, L.A.L., Ramon, X.R., Domingo, S.G.: The circular economy in the construction and demolition waste sector - A review and an integrative model approach, Journal of Cleaner Production, 248 (2020), pp. 1-15

[4] Poon, C.S., Yu, A.T.W., Wong, A., Yip. R.: Quantifying the impact of construction waste charging scheme on construction waste management in Hong Kong, Journal of Construction Engineering and Management, 139 (2013) 5, pp. 466-479

[5] Wang, J., Wu, H., Duan, H., Zillante, G., Zuo, J., Yuan, H.: Combining life cycle assessment and building information modelling to account for carbon emission of building demolition waste: a case study, Journal of Cleaner Production, 172 (2018), pp. 3154-3166

[6] Sangiorgi, C., Lantieri, C., Dondi, G.: Construction and demolition waste recycling: an application for road construction, International Journal of Pavement Engineering, 16 (2015) 6, pp. 530-537

[7] Kien, T.T., Thanh, L.T., Lu, P.V.: Utilisation of construction demolition waste as stabilised materials for road base applications, The International Conference on Sustainable Built Environment for Now and the Future, pp. 285-293, Hanoi, Vietnam, 26-27 March 2013.

[8] Angulo, S.C., Mueller, A.: Determination of construction and demolition recycled aggregates composition, in considering their heterogeneity, Materials and Structures, 42 (2009) 6, pp. 739-748

[9] Pasetto, M., Baldo N.: Recycling of waste aggregate in cement bound mixtures for road pavement bases and sub-bases, Construction and Building Materials, 108 (2016), pp.112-118.

[10] Hazelton, P., Murphy, B.: Understanding soils in urban environments, CSIRO Publishing, 2011.

[11] Wu, P.: Cement-bound road base materials, Delft University of Technology, 2011, pp. 1-115

[12] Francisco, J.T.M., De Souza, A.E., Teixeira, S.R.: Construction and demolition waste in concrete: property of pre-molded parts for paving, Cerâmica, 65 (2019) 1, pp. 22-26

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THE USE OF FIBERS IN CEMENT-STABILIZED BASE COURSE OF PAVEMENT

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Abstract

Nowadays, various materials are being analyzed as a possible component of pavement structure with the goal of using sustainable building materials and protecting the environment. Waste and recycled materials are added to pavement layers in order to improve it. Also, the possibility of using natural, renewable materials by incorporating them into existing standard materials is been examined. Cement-stabilized base course increases load-carrying capacity of the pavement but is prone to cracking which causes reflection cracks in an asphalt surface. Reinforcement of cement-stabilized base course can be achieved by the addition of fibers. Fibers added to the cement stabilization tend to prevent or delay the crack initiation and propagation by redistributing the resulting stresses. Considering the research conducted to-date and the need to use sustainable materials in combination with cement stabilization, some attempts are being made to achieve improvements of this pavement layer. Natural fibers are locally available, economical, renewable and degradable, and can be used as reinforcement. In the Mediterranean area, a possible source of cellulose fibers is found in the wild plant named Spanish Broom (Spartium junceum L). This paper offers an overview of research studies about fiber reinforcement of cement-stabilized base course. It also presents current research on Spanish Broom fibers in cement composites, as well as possible ways of obtaining and treating fibers. Based on the results of this research, a method for obtaining the fibers can be selected which might improve the mechanical properties of cement-stabilized course.

Keywords: cement-stabilized course, fibers, Spanish Broom

1 Introduction

Cement-stabilized base (or subbase) course (CSBC) is the structural part of pavements. It is a good base for upper pavement layers due to its high strength and low sensitivity to water, as well as a good protection for underlying layers because it significantly reduces stresses caused by traffic load. The main problem of CSBC is its sensitivity to cracking, which eventually reflects through the upper bituminous layers to the surface. Over time, cracks can widen and the pavement can become damaged, which seriously reduces its structural strength, thus affecting the pavement service performance. The use of fibers may help control the problem with cracks, especially in the light of the findings of previous studies on cementitious materials. Adding fibers to the cement mixture might affect the mechanical properties and control crack initiation, propagation rate and width. Reinforcement of CSBC can be achieved by the addition of fibers to improve the toughness, ductility and cracking resistance of the cement matrix. There are different types of fibers that can be used as reinforcement, and given their origin, they can be classified into four basic groups: steel, glass, synthetic and natural fibers. The influence of fiber addition on crack resistance and mechanical properties of cement stabilization have been analyzed in numerous research studies, some of which are presented in this paper. Nowadays, more attention is paid to the possibility of using waste, recycled and natural materials with the aim of protecting the environment and using renewable materials. Natural fibers have not yet been tested as a possible reinforcement in CSBC; hence, the basic properties of cellulose fibers, with an emphasis on Spanish Broom fibers, are presented in this paper.

2 Reinforcement of the cement-stabilized base course by various fibers

CSBC contains aggregate of various sizes, small quantities of cement and water. This layer has high strength, rigidity and water stability. However, this material also presents some disadvantages, such as high shrinkage rate, poor resistance to deformation and high brittleness [1]. Because of that, and due to changes in temperature and humidity, cracks appear in this layer of pavement. Initial cracking occurs in the form of transverse cracks caused by thermal contraction and shrinkage of the layer. Secondary longitudinal cracking is created due to the traffic load. Those cracks cause most damage in the pavement and they occur owing to the tensile failure of a CSBC. They are controlled by improving the aggregate gradation, adding various additives or applying other pre-cracking techniques. These measures are less effective in preventing CSBC from cracking and do not solve the cracking problem effectively [1][2]. Fiber reinforcement in CSBC is intended to overcome the problems which result from cracking. Adding fibers to CSBC strives to prevent or delay generation and propagation of cracks by transferring the resulting stresses to adjacent sections. This can be achieved because fibers act like bridges between two cracked sides of the matrix, i.e. like micro reinforcement. The fiber reinforcement of CSBC is examined in various research studies. Shahid et al. [3] reinforce CSBC with 1 % (by volume) steel fibers. They perform tests on cube specimens (100 mm sides) and cylinder specimens (150 mm diameter and 150 or 300 mm height) and compare the results with and without fibers. They analyzed direct and indirect tensile strength, compressive strength, elastic stiffness, load versus deformation characteristics and post-cracking behavior of the mixtures. The resulting cracks in the specimens have had a very narrow width and a high load transfer. The tensile strength increased by 33 %, whereas the compressive strength did not change significantly. Coni and Pani [4] presented results of the experimental indirect tensile tests of reinforced CSBC with steel fibers. Also, they analyzed two semi-rigid pavement sections to evaluate the effects of fibers during the pavement service life. The number of load cycles prior to collapse increased by more than 60 times on the section with 3.5 % of cement and 1.5 kg/m^3 of steel fibers.
Farhan at el. [5] used recycled steel fibers extracted from old tires (0.5 % by volume of aggregate) to reinforce the CSBC matrix. Cement content varies up to 7 % by weight of aggregate and fibers. As expected, the results indicate better tensile strength in the case of reinforcement with higher cement content. Also, the use of steel fibers (extracted from waste tires) reduces the crack propagation rate at all cement contents, with the greatest reduction occurring at high cement contents. They recommended that this reinforcement should be used with a cement content not lower than 5 %. Zheng et al. [2] used basalt fibers for mixtures of CSBC with the constant length of fibers (25 mm) and their different content. The tests included the analysis of compressive and flexural strength of specimens, the analysis of anti-shrinkage properties and the analysis of cracking. The results indicated higher flexural strength without significantly affecting the compressive strength of specimens. The anti-dry shrinkage properties and cracking ability under the temperature cycle of cement-stabilized macadam are improved by basalt fibers in quantities exceeding 6 kg/m^3.

Basalt fibers can help the CSBC to transfer and share the load stress, thus improving the cracking resistance capacity of CSBC.

Zhang et al. and Peng et al. [1][6][7] analyzed the effects of polypropylene fibers on the properties of CSBC. Mechanical and shrinkage properties of CSBC were improved. Polypropylene fibers can decrease the average dry shrinkage coefficient and average thermal shrinkage coefficient with the growth of fiber content (up to 0.1 % of fiber volume), so the fracture behavior is improved. With the increase of fiber volume, the compressive modulus of resilience and flexural modulus of elasticity also decrease. By reinforcing CSBC with polypropylene fibers, Ma et al. [8][9] investigated the fatigue performance and the freeze-thaw performance of specimens. They concluded that the mixing of polypropylene fibers of a certain length and volume content into the cement-stabilized aggregate can significantly improve its bending fatigue resistance. The amount of fibers they recommend is 0.7 kg/m3. The fibers improve the density of the internal micro-structure of the cement-stabilized aggregate, which helps to improve the strength and fatigue performance of the specimens. The freeze-thaw compressive strength and freeze-thaw splitting strength in reinforced specimens increased, while the freeze-thaw mass-loss rate decreased.

Liu [10] investigated the effects of CSBC reinforced by polyester fibers. His conclusion based on the conducted test is that the addition of those fibers can effectively decrease the shrinkage coefficient in CSBC, following the fiber content increase. At the optimum fiber content (approximately 0.7 ‰), the cleavage strength and the compressive strength of mixed specimens increase, by 7.6 % and 7 % respectively. Also, the effects of polyester fibers are compared with those of polypropylene fibers. According to his results, polyester reinforcement has a greater effect on the performance of CSBC specimens than polypropylene reinforcement. Cavey et al. [11] conducted a coordinated laboratory and field study to assess the feasibility of producing an economically suitable pavement base course material by reinforcing cement-stabilized recycled concrete aggregate with strips of reclaimed plastic or tire wires and tire chunks from recycled scrap tires. Based on the results, they concluded that the waste fibers not only afforded little or no improvement in material behavior, but also adversely affected both the strength and toughness of the composite material.

Nowadays, a special emphasis is placed on the use of environmentally friendly materials and in this regard, extensive research is being conducted on the possibility of using renewable natural fibers in composite materials. The use of cellulose fibers and their effect on the CSBC has not been investigated so far.

3 Reinforcement by natural fibers

Natural cellulosic fibers have several advantages, such as low cost, bio-renewability and biodegradability, low density, good toughness and strength [12]. Different properties of fibers, including their length, diameter, density, surface roughness and structure, stem from different natural sources. The quality of fibers also depends on their required processing. All of those characteristics will result in the quality of adhesion between fibers and the matrix, and further impact the properties of composite materials. The use of cellulose fibers results in reduced plastic shrinkage and better thermal and sound insulation. The reinforcement capacity of specimens with fibers also depends on the amount of fibers used, their length/thickness ratio and dispersion in the matrix [13]. Fiber-matrix bonding can be well-balanced, allowing stress transfer between the matrix and the fibers. On the other hand, if fibers are not well spread in the matrix because of proportioning limitations (they are too long or overdosed), it can cause a grouping of fibers which results in a non-existent or poor bonding quality. Fiber modification with different treatments can improve the durability of the fibers themselves and the fiber–cement matrix adhesion. Due to the need to obtain fibers from the shoots, some chemical elements of the structure need to be decomposed, which can be

achieved by soaking those shoots in appropriate solutions. This process is called maceration.
Ardanuy et al. [13] presented a review of recent research studies on cement-based composites reinforced with cellulosic fibers. The reinforcement based on cellulose fibers can be classified by the function of their form (strands, staple fibers or pulp). Some of the natural fiber sources that are used in cement-based composites are hemp, jute, sisal, agave, eucalyptus, coir, banana and pinus [13]. According to [13], the use of cellulose fibers can improve the mechanical properties of cement-based composites if the fibers are adequately dispersed in the matrix. One of the disadvantages of using natural fibers is that they have great variability in mechanical properties. The quality of natural fibers depends on geographical and climate conditions, soil quality, weathering conditions, extraction methods, time of harvesting and plant maturity [14].

4 Properties of Spanish Broom fibers

In the Mediterranean area, a possible source of cellulose fibers is found in the wild plant named Spanish Broom (Spartium junceum L). Spanish Broom usually grows as a bush varying from 1 to 1.5 m in height and it is widely known for its yellow flowers characterized by a rather intense scent. Branches are very tough and constitute the most important part of the plant. The fibers can be found inside the shoots and the process of obtaining fibers includes harvesting and maceration of shoots. Spanish Broom bushes and fibers obtained after the maceration of shoots are presented in Figure 1. The chemical composition of Spanish Broom fibers (SBF) is 91,7 % cellulose, 3,2 % lignin, 4,1 % pentosane and some ashes [14]. SBFs have the lowest specific weight and similar tensile strength compared to other natural fibers. Angelini et al. [15] established that the elastic modulus for Spanish broom is approximately 21.5 GPa, which is in the range of other fibers such as cotton, jute and sisal [16].

Figure 1 Spanish Broom bush and fibers

Several studies have been conducted on the possibility of using SBF as a reinforcement for composite materials. Nekkaa et al. [17][18] used SBF and polypropylene matrix. The tests indicated that the quantity of absorbed water increased with an increase of SBF in the composite. Good adhesion between the fibers and the matrix can be improved if the fibers are treated with silane. The surface treatments give better tensile and impact strength as well as fiber dispersion. Kovačević et al. [19] reinforced polylactic acid matrix with SBF. Avella at al. [20] used SBF as reinforcement for the polypropylene matrix. In both studies, better mechanical properties were obtained compared to non-reinforced specimens.
Juradin et al. [14][16] examined the possibility of cement mortar reinforcement with SBF. They treated the shoots in the solution of 5 % NaOH, in seawater and a combination of alkali (5 %

NaOH) and seawater. After obtaining the fibers, they cut them into three lengths: 10, 20 and 30 mm. The quantities of fibers in cement mortar were 0.5 % and 1 % of the total volume. Fibers with 10 and 30 mm in length have achieved better results than fibers of 20 mm in length. The samples with 30 mm-long fibers showed the highest flexural strength values, but also a slight decrease in compressive strength. Furthermore, the authors compared referent mortar specimens (label E – specimens without fibers) with those whose fibers were obtained by a 28-day maceration with seawater, followed by 7 days in 5 % NaOH solution (label MN). They concluded that SBF specimens can take over the load, while the etalon specimens are currently fractured, which is ultimately the purpose of micro-reinforcement (Fig. 2a). Fig. 2b shows fiber reinforcement of Spanish Broom in cement mortar specimens.

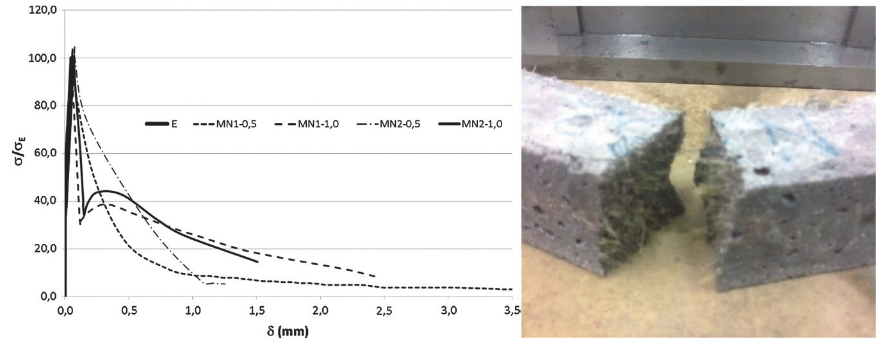

Figure 2 a) σ / σE – δ diagram [23] and b) SBF reinforcement of cement mortar

In [21] the authors compared the results obtained for Spanish Broom and hemp fiber-reinforced mortars and concluded that natural fibers do significantly increase mortar ductility. Treatment of Spanish Broom and hemp fibers in different solutions (2.5, 5, 6, 8, 10 and 15 % NaOH solution, 2.5 % NaOH + 2 % Na_2SO_3 and 5 % NaOH + 2 % Na_2SO_3 mixed solution, seawater, and a combination of 5 % NaOH and seawater) showed that pH values of the treatment medium influence the crystallinity of fibers. It was observed that the Spanish Broom fibers showed potential for reinforcing cement-based composites.

5 Conclusion

Cement-stabilized base course increases load-carrying capacity of the pavement and serves as a good protection of underlying layers because it redistributes stresses over a wide area. The main disadvantage of this layer is its sensitivity to cracking which causes reflective cracking in the asphalt surface. Results of the presented studies indicate that reinforcement of CSBC with steel or synthetic fibers can improve certain mechanical characteristics of the material. The improvements depend on type, characteristics and amount of fibers, amount of cement and the quality of bonding between fibers and the cement matrix. With regard to using sustainable materials, natural fibers are renewable, biodegradable and environmentally friendly. Research conducted with Spanish Broom fibers has indicated the potential for reinforcing cement-based composites with these fibers. Due to the fact that the Spanish Broom plant is locally available and its fibers contribute to the reinforcement of different materials, the aim of the future study is to use SBF in the CSBC matrix. Therefore, future research will be based on examining the possibility of CSBC reinforcement with differently treated (NaOH and seawater), different lengths and different amounts of SBF.

Acknowledgment

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References

[1] Zhang, P., Li, Q.: Experimental study on shrinkage properties of cement-stabilized macadam reinforced with polypropylene fiber, Journal of Reinforced Plastics and Composites, 29 (2010) 12, pp. 1851–1860, doi: 10.1177/0731684409337336

[2] Zheng, Y., Zhang, P., Cai, Y., Jin, Z., Moshtagh, E.: Cracking resistance and mechanical properties of basalt fibers reinforced cement-stabilized macadam, Composites Part B: Engineering, 165 (2019), p.p. 312–334, doi: 10.1016/j.compositesb.2018.11.115

[3] Shahid, M.A., Thom, N.H.: Steel fibre reinforcement in cement bound bases, Proceedings of the Institution of Civil Engineers: Transport, 129 (1998) 1, pp. 34–43, doi: 10.1680/itran.1998.30091

[4] Coni, M., Pani, S.: Fatigue analysis of fiber-reinforced cement treated bases. In Proc. SIIV Congress., 2007.

[5] Farhan, A.H., Dawson, A.R., Thom, N.H.: Damage propagation rate and mechanical properties of recycled steel fiber-reinforced and cement-bound granular materials used in pavement structure, Construction and Building Materials, 172 (2018) , pp. 112–124, doi: 10.1016/j.conbuildmat.2018.03.239

[6] Zhang, P., Liu, C.H., Li, Q.F., Zhang, T.H.: Effect of polypropylene fiber on fracture properties of cement treated crushed rock, Composites Part B: Engineering, 55 (2013), pp. 48–54, doi: 10.1016/j.compositesb.2013.06.005

[7] Peng, Z., Qingfu, L.: Effect of polypropylene fibre on mechanical and shrinkage properties of cement-stabilised macadam, International Journal of Pavement Engineering, 10 (2009) 6, pp. 435–445, doi: 10.1080/10298430802363985

[8] Ma, Y., Gu, J., Li, Y., Li, Y.: The bending fatigue performance of cement-stabilized aggregate reinforced with polypropylene filament fiber, Construction and Building Materials, 83 (2015), pp. 230–236, doi: 10.1016/j.conbuildmat.2015.02.073

[9] Ma, Y., Gu, J.: Study on freeze-thaw performance of polypropylene fiber reinforced cement-stabilized aggregate, Advanced Materials Research, 446–449 (2012), pp. 2595–2598, doi: 10.4028/www.scientific.net/AMR.446-449.2595

[10] Liu, Z.: Experimental research on the engineering characteristics of polyester fiber-reinforced cement-stabilized macadam, Journal of Materials in Civil Engineering, 27 (2015) 10, pp. 1–10, doi: 10.1061/(ASCE)MT.1943-5533.0001251

[11] Cavey, J.K., Krizek, R. J., Sobhan, K., Baker, W. H.: Waste fibers in cement-stabilized recycled aggregate base course material, Transportation Research Record, 1486, pp. 97–106, 1995.

[12] Satyanarayana, K.G., Arizaga, G.G.C., Wypych, F.: Biodegradable composites based on lignocellulosic fibers-An overview, Progress in Polymer Science (Oxford), 34 (2009) 9, pp. 982–1021, doi: 10.1016/j.progpolymsci.2008.12.002

[13] Ardanuy, M., Claramunt, J., Toledo Filho, R.D.: Cellulosic fiber reinforced cement-based composites: A review of recent research, Construction and Building Materials, 79 (2015), pp. 115–128, doi: 10.1016/j.conbuildmat.2015.01.035

[14] Juradin, S., Boko, I., Netinger Grubeša, I., Jozić, D., Mrakovčić, S.: Influence of harvesting time and maceration method of Spanish Broom (Spartium junceum L.) fibers on mechanical properties of reinforced cement mortar, Construction and Building Materials, 225 (2019), pp. 243–255, doi: 10.1016/j.conbuildmat.2019.07.207

[15] Angelini, L.G., Lazzeri, A., Levita, G., Fontanelli, D., Bozzi, C.: Ramie (Boehmeria nivea (L.) Gaud.) and Spanish Broom (Spartium junceum L.) fibres for composite materials: Agronomical aspects, morphology and mechanical properties, Industrial Crops and Products, 11 (2000) 2–3, pp. 145–161, doi: 10.1016/S0926-6690(99)00059-X

[16] Juradin, S., Boko, I.: Possibility of cement composite strengthening by weaver's broom fibres, GRADEVINAR, 70 (2018) 6, pp. 487–495, doi: 10.14256/jce.2293.2017

[17] Nekkaa, S., Haddaoui, N., Grillet, A. C., Merle, G.: Thermal, thermomechanical, and morphological properties of spartium junceum fiber reinforced polypropylene composites, International Journal of Polymeric Materials and Polymeric Biomaterials, 55 (2006) 11, pp. 837–853, doi: 10.1080/00914030500504008

[18] Nekkaa, S., Guessoum, M., Haddaoui, N.: Water absorption behavior and impact properties of spartium junceum fiber composites, International Journal of Polymeric Materials and Polymeric Biomaterials, 58 (2009) 9, pp. 468–481, doi: 10.1080/00914030902936535

[19] Kovačević, Z., Bischof, S., Vujasinović, E., Fan, M.: The influence of pre-treatment of Spartium junceum L. fibres on the structure and mechanical properties of PLA biocomposites, Arabian Journal of Chemistry, 12 (2019) 4, pp. 449–463, 2019, doi: 10.1016/j.arabjc.2016.08.004

[20] Avella, M., Casale, L., Dell'erba, R., Focher, B., Martuscelli, E., Marzetti, A.: Broom fibers as reinforcing materials for polypropylene-based composites, Journal of Applied Polymer Science, 68 7, pp. 1077-1089, 1998.

[21] Juradin, S., Boko, Netinger Grubeša, I., Jozić, D., Mrakovčić, S.: „Influence of different treatment and amount of Spanish broom and hemp fibres on the mechanical properties of reinforced cement mortars“, Construction and building materials, 273 (2021) 121702, doi:10.1016/j.conbuildmat.2020.121702

WASTE RUBBER - SUSTAINABLE PAVEMENTS SOLUTION?

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Abstract

Cement bound base courses increase pavement bearing capacity and resistance to detrimental effects of frost along with being a good base for surface courses. Due to its stiffness, cement bound base courses reduce vertical deflections and tensile stress in asphalt layers thus reducing fatigue and appearance of alligator cracks in asphalt. But, in order to generate required layer strength and stiffness of cement bound base, appropriate amount of cement is necessary. This in turn can cause shrinkage induced cracks which spread towards surface courses and cause accelerated deterioration of pavements. To prevent rapid deterioration of pavement surface layers and occurrence of reflective cracking, incorporation of recycled rubber from waste tire in mixtures for cement bound base courses is currently being considered. In this paper a short review of waste tire application and its behaviour in pavement structures will be shown. In addition, planned methodology and activities that are going to be conducted with in a research project are going to be discussed.

Keywords: sustainable pavements, base course, waste tire

1 Introduction

The Transport Development Strategy of the Republic of Croatia 2017-2030 [1] states that the main road network in the Republic of Croatia has been established, and it mainly consists of highways. Given the economic situation and the high quality of these roads, most of the developed countries, including the Republic of Croatia, are reducing investment in the construction of highways and turning their investments into the construction, reconstruction and modernization of lower category roads and maintenance of the existing network to extend their life span. Thus, according to the Public Roads Construction and Maintenance Program for the period 2017-2020 [2] 55 % of all funds, were planned to be invested in state roads network and it is expected that trend to be continued within the next planning period. At the same time, the Strategy [1] among other things cites the need to reduce the impact of transportation system on climate change and the environment, as the basic goal of further transportation system development.

State roads pavement, due to the traffic load, is built with a cement-stabilized bearing course (CBC). CBC are installed in a pavement structure to increase its bearing capacity, they are good base for surface courses and they increase the pavement resistance to the detrimental effect of frost. The installation of these layers results in reduced fatigue in asphalt layers and appearance of alligator cracks as their stiffness reduces vertical deflection and deformations of asphalt layers caused by tensile stress. On the other hand, using cement in base and subbase courses increase their stiffness leading to appearance of cracks. Cracks occur due to the shrinkage of material and are reflected on the surface of asphalt layers. So, here lies

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the potential for exploring new materials and techniques for installation in CBC with the goal of reducing roadway maintenance, extending its life span, reducing reflective cracks occurrence and being environmentally friendly.
With the aim to prevent deterioration of pavement surface and reduce usage of natural materials, other materials have been researched to implement in pavement construction. Material with great potential is waste rubber and its potential usage in CBC is the objective of a new scientific project funded by Croatian Science Foundation which will be presented within this paper.

2 Waste rubber application in road construction

2.1 Waste rubber in asphalt mixtures

Crumb rubber is added to the asphalt mixtures as an additive in order to improve some of the mixture properties, such as the resistance to rutting, reduction of the fatigue cracking and low-temperature cracking or the reduction of the noise emission [3]. According to the many researches performed in order to investigate the effect of the crumb rubber in the asphalt mixtures, the general conclusion is that the addition of the crumb rubber extends the life span of the flexible pavement construction. In comparison to standard industrial additives such as SBS, SBR or EVA polymers, it is more ecological and less costly [4].
The rubber asphalt is produced by wet or dry process. In the wet process, the rubber waste from vehicle tires is a part of the asphalt binder, either dissolved in the liquid asphalt binder before mixing (as a part of the binder) or by substituting a portion of fine aggregate with ground rubber which does not completely react with the binder. In the dry process, the crumb rubber is substituting a percentage of aggregate particles in the asphalt mixture and it is not a part of the binder [4].
When the crumb rubber is added as a part of the binder in the wet process, it influences the rheological properties of the mixture such as viscosity, softening point, penetration, temperature susceptibility, strength and durability [5]. Viscosity and the temperature susceptibility tend to increase with the crumb rubber addition. Also, the homogeneity of the mixture is disrupted which causes the reduction in the penetration and the ductility and consequently makes the mixture stiffer [6]. The strain capacity of the mixture is also increased, which means that the rubber-modified mixture is more flexible and tough [5]. The Marshall stability is higher for low percentage of crumb rubber [6], but when the amount of crumb rubber is higher than 10 % from the bitumen weight the Marshal stability decreases which means that the strength and the quality of the mixture is reduced. The investigation of dry process-produced crumb rubber asphalt mixture properties show that the addition of the rubber could improve the resistance to permanent deformation and low-temperature cracking [7]. Due to the less amount of the binder content needed in the crumb rubber mixture, the amount of air voids in the mixture and the permeability increases, which can cause a reduction of the mixture durability [6]. The performed investigations of asphalt mixture properties modified with the crumb rubber lead to the conclusion that the amount of the rubber and the production process of the modified asphalt mixture significantly influences the final properties.

2.2 Waste rubber in subgrade, subase and base layers

Available literature also provides numerous researches on the applicability of waste crumb rubber in lower layers of road construction and in stabilization of local soil / subgrades. Research into application of rubber on its own or in combination with binders, such as cement, for clayey soil stabilization show different results, which largely depends on the amount and type (shape and size) of the used rubber. Yadav and Tiwari [8] state that the presence of the rubber can lead to reduced strength due to weaker bonds and less friction between crumb rubber and clay particles or due to lower stiffness in cement-stabilized mixtures. However, in their research, there was a slight increase in compressive strength and increase of California Bearing Ratio (CBR) by addition of 2.5 % rubber to clay soil, while further increase in tire volume resulted in a decrease of measured values. In the same study, in cement stabilization, the rubber reduced compressive and indirect tensile strength, but resulted in ductile behaviour of these mixtures.

Li et al. [9] investigated the applicability of crushed rubber in unbound layers from recycled aggregate and crushed stone. Measurement of CBR showed that small tire fractions act as filler, and CBR increased with increased content of fine rubber fraction from 0.5 % to 2 %. For coarser rubber particles, the optimal proportion is 1 %, followed by a significant drop in CBR. In addition, the crumb rubber increases failure strain in relation to the control samples. Arulrajah et al. [10] also found a reduction in the stiffness upon addition of crumb rubber to the mixture of waste crushed rock. Research has shown that the optimum replacement ratio of aggregate with crumb rubber is 2 % by which the engineering properties are equivalent to those of a natural aggregate. The CBR of all the blends has met the conditions set for application in base/subbase layers, whereas application in the base layers is not possible only when the 3 % coarse fraction of crumb rubber is applied.

2.3 Waste rubber in concrete pavement

The addition of Portland cement to unbound grained material improves its mechanical properties, but it certainly increases stiffness, and this layer becomes more susceptible to cracking and fatigue failure [11]. Considering the dynamic nature of traffic loads, research has also been carried out over the last couple of years to explore the application of rubber in a conventional or roller compacted concrete for pavements. The application of rubber in concrete presents the addition of elastic material to a rigid concrete matrix that changes its properties [12]. Numerous studies of the use of crumb rubber in concrete suggest that the addition of rubber and increase of its amounts in concrete usually has a negative effect on mechanical properties such as compressive, indirect tensile and flexural strength, modulus of elasticity and density [13,9,14]. Strength reduction is described as a result of poor bonds between cement paste and rubber particles and also because rubber with low modulus of elasticity that is imbedded in concrete of high strengths, acts like a void [13,12]. But the rubber addition also increases the capacity of concrete for energy absorption and ductility and reduces the possibility of brittle fracture. The addition of rubber to concrete increases the number of load cycles that will lead to fracture or fatigue of the material [15]. Kardos and Durham [16] investigated the possibility of applying fine aggregates of crushed rubber as a volumetric sand replacement (10, 20, 30, 40 and 50 %) in concrete pavements. The results of this research have shown that the replacement of sand with crumb rubber shows satisfactory properties of fresh and hardened concrete with a rubber share of up to 30 %. With the addition and increase in rubber share, the drop in compressive and indirect tensile strength was noted, but the samples were ductile, could take higher deformations, and had residual load capacity after failure.

Pacheco-Torres et al. [12] have established a methodology for measuring the deformations in concrete mixtures with crumb rubber, created to evaluate mixtures for pavements under cyclic loads. They found that 20 % of the rubber aggregate significantly reduces the mechanical properties but that up to 10 % of the rubber of different granulations achieves better deformability and performance of the material under cyclic load with an acceptable reduction in mechanical properties. Similar conclusions are presented also in [17].

3 Project RubSuPave

Despite the widely researched application in concrete and asphalt, crumb rubber as an alternative material for construction of cement stabilized layers is sparsely researched in available literature. In the greatest extent, Farhan and his associates [11, 18, 19] dealt with the application of crumb rubber in cement stabilized layers. Although researches in the area of waste rubber application in cement stabilized base layers of pavement structures are limited, there are indications that its application could result in a reduction of shrinkage cracking. However, the optimum mix composition (optimal amount of cement, optimum tire aggregate content), and in particular the effect on behaviour of asphalt courses in this kind of pavement structure, is not yet fully explored. This is where innovation is seen in the field of application of automotive waste tires for pavement structures.
Research project called Cement stabilized base courses with waste rubber for sustainable pavements is based on aspiration to examine application of waste rubber in cement stabilized base course (CBC). The project gathers scientists from different fields of civil engineering from the Faculty of Civil Engineering in Rijeka, GIC Gradnje d.o.o. Rogaška Slatina, Slovenia and from the Faculty of Civil Engineering and Architecture Osijek as a project leader. The possibilities for improving behaviour of pavement incorporating CBC will be researched through an analysis of the possibility of recycled rubber replacement for fine aggregate fractions. Firstly, extensive laboratory research will be conducted within which material mechanical properties will be defined and upon that, optimal mix composition. For the purpose of defining the optimal CBC mix of satisfactory strengths and improved elastic properties, the first subgroup of laboratory research will be conducted. Within mix composition, binder ratio and waste rubber granulation (Fig.1.) will be varied.

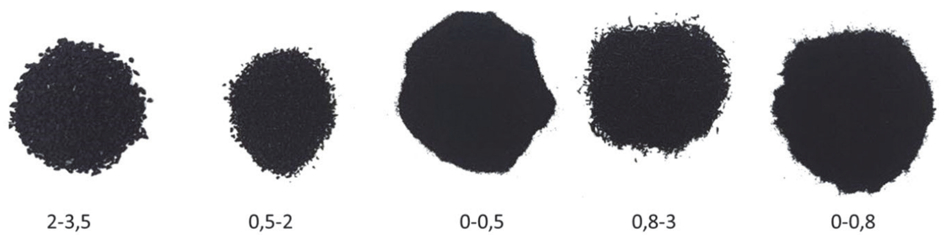

Figure 1 Different waste rubber granulation (mm)

After defining material mechanical properties, statistically analyses of the results and comparison to the valid technical requirements will be done and the mixtures will be ranked according to the required properties. After the material characterization of CBC with waste rubber at one level, behaviour and impact of this layer on the asphalt wearing course will be examined. Samples of asphalt wearing course (two different types) will be installed on the CBC samples and the behaviour of the CBC – wearing course system will be monitored by 3D-DIC method under cyclic loading as traffic load simulation. During the test, pavement course deflections and crack development will be measured in order to develop pavement behavioural model. Based on the laboratory research results, pavement spatial numerical models will be developed using the finite element method. Results gained by laboratory tests, as well as those gained by the numerical modelling, will be used as input parameters for the optimization model which will provide optimal or near optimal design of cement-stabilized bearing layer with the addition of recycled tires.
Numerical and optimization models verification and validation will be performed through constructing and testing a test section with the defined optimal mix of CBC. The test section will be built with CBC without waste rubber (reference section) and with CBC containing

waste rubber, in equal dimensions. Within the last year of project, pavement condition will be monitored by means of measuring cracks and deformations development. Weather conditions will be also monitored (air temperature, pavement warming and rainfall) as well as traffic loads. Summary of research activities is shown on Fig. 2.

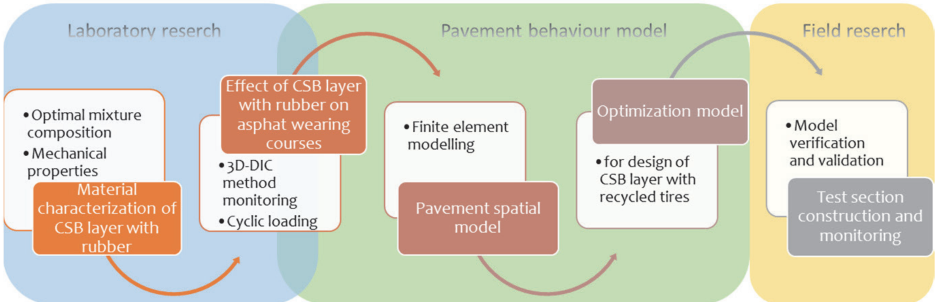

Figure 2 Diagram of project activities

The importance of exploring the proposed subject is emphasized in two aspects: the reduction of the application of natural materials to which road construction traditionally relies on and the extension of pavement life time. The use of waste rubber in CBC has the potential to reduce cracks occurrence in this layer, which results in reflective cracking on asphalt layers. This will reduce the need for road maintenance and extend its lifespan, which will result in savings of financial resources and reduced energy consumption.

4 Conclusion

Waste rubber, as is shown in various literature, can be a beneficial material for construction of different pavement layers, from wearing courses to subgrade. Its application can especially be beneficial for elastic properties and deformability, reduction of fatigue cracking and performance of material under cyclic loading. Despite this benefits, application in cement bound courses, the most rigid layer of flexible asphalt pavements, has been sparsely investigated and just on material level. This CBC layer can highly effect properties of wearing courses by causing shrinkage induced cracks which spread towards surface and thus accelerating deterioration of pavements. Therefore, in a research project Cement stabilized base courses with waste rubber for sustainable pavements, along with material properties of CBC with waste rubber, its effect on asphalt courses and overall pavement should be determined. To better understand the effect waste rubber in CBC has on pavement laboratory reaserch, field research, pavement modelling and model optimization are planned.
Results of this scientific project will contribute to the creation of new knowledge on the practical application of waste tires in cement stabilized base layers. Also, the project activities are aimed at solving and improving the practical goal: creating a sustainable roadway through the application of waste material and extending the life of the road construction.

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References

[1] Ministarstvo mora, prometa i infrastructure: Strategija prometnog razvitka Republike Hrvatske 2017-2030, 2017, https://vlada.gov.hr/UserDocsImages/ZPPI/Strategije/MMPI%202017-2030%20STRAT%20PROM%20RZV%20RH%2025-8_17.pdf, 3.4.2019

[2] Vlada Republike Hrvatske: Program građenja i održavanja javnih cesta za razdoblje 2017-2020, 2017, NN 47/2017

[3] Issa, Y.: Effect of Adding Waste Tires Rubber to Asphalt Mix, International Journal of Scientific Research and Innovative Technology, 3 (2016) 5, pp. 61-68

[4] Mashaan, N., Ali, A., Karim, M., Abdelaziz, M.: A review on using crumb rubber in reinforcement of asphalt pavement, The Scientific World Journal, 2014, doi: http://dx.doi.org/10.1155/2014/214612

[5] Tuncan, M., Tuncan, A., Cetin, A.: The use of waste materials in asphalt concrete mixtures, Waste Management&Research, 21 (2003), pp. 83–92, doi: https://doi.org/10.1177/0734242X0302100202

[6] Wulandari, P., Tjandra, D.: Use of Crumb Rubber as an Additive in Asphalt Concrete Mixture, Procedia Engineering, 171 (2017), pp. 1384-1389, doi: https://doi.org/10.1016/j.proeng.2017.01.451

[7] Cao, W.: Study on properties of recycled tire rubber modified asphalt mixtures using dry process, Construction and Building Materials, 21 (2007) 5, pp. 1011-1015, doi: https://doi.org/10.1016/j.conbuildmat.2006.02.004

[8] Yadav, J. S., Tiwari, S. K.: Effect of waste rubber fibres on the geotechnical properties of clay stabilized with cement, Applied Clay Science, 149 (2017), pp. 97–110, doi: https://doi.org/10.1016/j.clay.2017.07.037

[9] Li, J., Saberian, M., Nguyen, B.T.: Effect of crumb rubber on mechanical properties of crushed recycled pavement materials, Environmental Science and Engineering, 218 (2019), pp. 291–299, doi: https://doi.org/10.1016/j.jenvman.2018.04.062

[10] Arulrajah, A., Mohammadinia, A., Maghool, F., Horpibulsuk, S.: Tyre derived aggregates and waste rock blends: Resilient moduli characteristics, Construction and Building Materials, 201 (2019), pp. 207–217, doi: https://doi.org/10.1016/j.conbuildmat.2018.12.189

[11] Farhan, A.H., Dawson, A.R., Thom, N.H.: Characterization of rubberized cement bound aggregate mixtures using indirect tensile testing and fractal analysis, Construction and Building Materials, 105 (2016), pp. 94–102, doi: http://dx.doi.org/10.1016/j.conbuildmat.2015.12.018

[12] Pacheco-Torres, R., Cerro-Prada, E., Escolano, E., Varela, E.: Fatigue performance of waste rubber concrete for rigid road pavements, Construction and Building Materials, 176 (2018), pp. 539–548, doi: https://doi.org/10.1016/j.conbuildmat.2018.05.030

[13] Shu, X., Huang, B.: Recycling of waste tire rubber in asphalt and portland cement concrete: An overview, Construction and Building Materials, 64 (2014) B, pp. 217-224, doi: https://doi.org/10.1016/j.conbuildmat.2013.11.02

[14] Sofi, A.: Effect of waste tyre rubber on mechanical and durability properties of concrete – A review, Ain Shams Engineering Journal, 9 (2018) 4, pp. 2691–2700, doi: https://doi.org/10.1016/j.asej.2017.08.007

[15] Zhang, Y.M., Zhao, Z.: Internal Stress Development and Fatigue Performance of Normal and Crumb Rubber Concrete, Journal of Materials in Civil Engineering, 27 (2014) 2, pp. 1-9, doi: https://doi.org/10.1061/(ASCE)MT.1943-5533.0001000

[16] Kardos, A.J., Durham, S.A.: Strength, durability, and environmental properties of concrete utilizing recycled tire particles for pavement applications, Construction and Building Materials, 98 (2015), pp. 832–845, doi: https://doi.org/10.1016/j.conbuildmat.2015.08.065

[17] Strukar, K., Kalman Šipoš, T., Dokšanović, T., Rodrigues, H.: Experimental Study of Rubberized Concrete Stress-Strain Behavior for Improving Constitutive Models, Materials, 11 (2018), 2245, doi: https://doi.org/10.3390/ma11112245

[18] Farhan, A.H. et.al.: Flexural characteristics of rubberized cement-stabilized crushed aggregate for pavement structure, Materials and Design, 88 (2015), pp. 897–905, doi: https://doi.org/10.1016/j.matdes.2015.09.071

[19] Farhan, A.H., Dawson, A.R., Thom, N.H.: Effect of cementation level on performance of rubberized cement-stabilized aggregate mixtures, Materials and Design, 97 (2016), pp. 98–107, doi: https://doi.org/10.1016/j.matdes.2016.02.059

THE STUDY OF MOISTURE SUSCEPTIBILITY FOR ASPHALT MIXTURES CONTAINING BLAST FURNACE SLAGS

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Abstract

Nowadays, in the context of increasing traffic, extending road network, and environmental protection, an important target is to develop sustainable roads through the use of by-products derived from various manufacturing processes that can lead to the reduction of environmental degradation. Blast furnace slag is the resulting material during the casting processes of the iron. This is a non-metallic process that is obtained by melting the chemical compounds from the sterile, ash coxe, and the founders. As a result of global research, it has been found that an ecological asphalt mixture (with slags as aggregate) can be used as a pavement layer. However, there are inconveniences related to poor moisture susceptibility. In this context, this paper presents the study of moisture susceptibility of asphalt mixtures with blast furnace slags starting from a basic recipe of AC 31.5 type with natural aggregate and replacing the natural aggregate with slag in different percentages. The paper presents the tests carried out on 15 asphalt mixture recipes with partial and total replacement of natural aggregates with slag artificial aggregates and compared to a classic recipe where 100 % natural aggregates were used. The used tests were chosen to characterize the water-related behavior: water absorption and indirect tensile strength test. The results indicate that asphalt mixture with slag aggregates can be a valuable resource in designing asphalt mixtures and satisfactory performance has been achieved.

Keywords: asphalt mixtures, blast furnace slag, water absorption test, indirect tensile strength test

1 Introduction

Damages from moisture action is a common problem in bituminous pavements, encountered in the temperate continental climate and can be considered as a degradation of the mechanical properties of the asphalt due to the action of moisture or water, causing serious distresses. Moisture susceptibility is usually associated with the loss of adhesion between bitumen and aggregate and/or loss of cohesion within the asphalt mixture mainly due to the presence of water [1]. This leads to a loss of aggregate, which in the medium and long term, and combined with other negative factors (traffic loads, ice formation, binder aging, etc.) finally cause most road pathologies and the eventual failure of the road surface course [2]. For the bituminous mixture to be used in the layers of a road structure, it has to carry out a series of laboratory testing that highlight its performance for the different types of stresses to which will be subjected during its operational life [3].

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Furthermore, pavements could be subjected to moisture during rainy seasons. Any moisture remaining in or on the aggregates would affect the aggregate coating and exacerbate the loss of bond between bituminous binder and aggregates, causing asphalt stripping and premature pavement failure. Typically, the loss of bond begins at the bottom of the pavement layer and progresses upward [4].
From manufacturing operations, service industries, and mining, many residual materials are emerging as a problem for environmental pollution. Many of the waste occupy large areas of land around factories in many countries and they are not biodegradable materials. Thus, in recent years, in many countries, legislation has attempted to impose the use of waste in various sectors of interest.
Road materials such as quarry or river aggregates, hydrocarbon, and hydraulic binders are used for road construction. It is known that natural materials are exhaustible, their quantity gradually decreasing. Also, the cost of extracting good quality materials is increasing.
The lack of traditional road materials and environmental protection has led to the use of waste in road construction with a wide range of possibilities. Types of waste that can be used in road construction are blast furnace slag, thermo-central ash, rubber, fibers.
Waste can bring some benefits from a technical point of view. For example, slags can successfully replace certain natural aggregates, and when used bituminous mixtures ensure safe and comfortable operating life due to texture that gives roughness and high skid resistance. The use of waste provides an economic advantage for both the beneficiary and the contractor by reducing construction costs. If the use of these materials is planned from the outset, the total cost of the project may be low, allowing the beneficiary to carry out a larger volume of work with the same budget. Materials from industrial waste are often less expensive than natural materials they replace, and recycling or reuse of materials in situ can reduce transport costs.
The ecological benefits result from the disposal of landfills by using them as substitutes for non-renewable virgin materials to be exploited and processed. The use of waste preserves natural resources and reduces the energy used and the pollution associated with these activities ensuring environmental protection.
At the present moment, there is an increasing concern regarding the construction of sustainable roads by making bituminous mixtures with artificial aggregates derived from various production processes that lead to a decrease in environmental degradation.
Blast furnace slag is a non-metallic product and results as a secondary material during cast iron metallurgical processes, it is obtained from the melting of chemical compounds in the ore tailings, coke ash, and fondant.
Blast furnace slag is a homogeneous product that is produced at temperatures above 1480°C and the fact that it is a liquid is an important advantage because by controlling the cooling process different structures can be obtained (vitreous or crystalline and can be embedded in different proportions glassy phases and various sizes and shapes can be obtained.
Oxides in the slag are divided into three groups (acids - SiO_2, P_2O_5, bases - CaO, MgO, FeO, MnO, amphotries - Al_2O_3). Therefore, according to the chemical nature, the slags are classified into: acid slags (where predominantly acid oxides),basicslags (predominantly basic oxides), and neutral slags.The classification of the slag in one of these groups is appreciated through the basicity index, defined as the ratio of the percentage quantity of the basic and acidic components. Most blast furnace slags are characterized by CaO: SiO_2 ratios ranging from 1.0-1.3 and (CaO + MgO):(SiO_2 + Al_2O_3) between 0.85-1.20.
Slags can be considered complex oxide melts formed mainly from CaO, SiO_2, Al_2O_3, MgO and FeO oxides. Besides these, slags can contain Mn, Ba, Cr, P, Ti, V, B oxides. Besides FeO, slags can also contain iron oxides, Fe_3O_4, and Fe_2O_3. Thesulf in slags is in the form of sulphides and sulphates of Ca, Mn, and Fe. Blast furnace slag can be analyzed by the CaO-SiO_2-Al_2O_3-MgO system.

The amount of blast furnace slag produced is 200-300 kg / t of cast iron and now has a recovery yield of about 100 %. In Romania, at ArcelorMittal Galați we get approx. 360 kg slag/ton of cast iron. Depending on the cooling rate and the solidification mode, there are three types of solid slag:slowly cooled in the air with crystalline structure;expanded slag, cooled with a controlled amount of water to accelerate the solidification process, with a glassy cellular or vesicular structure in the form of a product with reduced specific weight and granular slag obtained by rapid cooling (the highest cooling rate compared to other types of slag) with a glass structure.
Worldwide, there is a multitude of research on the use of slag aggregates both as a base layer, grounding foundation and roads, and aggregates used in bituminous mixtures.
Huang Yi and al.in a study in 2016 note that countries such as the USA, Japan, Germany, and France, the rate of use of these products are about 100 % compared to China where the use rate of slag is 22 %. The amount of deposited steel slag (mainly BOFS) in China has been accumulated to more than 300 million tons, which leads to the occupation of farmland and pollution of groundwater and soil. In Germany annually, 400.000 tons of slag are used to replace natural aggregates in the foundation layers of the road'sconstruction [5].
In Romania, slag is also obtained at the Arcelor Mittal Galați Iron and Steel Works, where the processing capacity of the blast furnace slag plant produces 3000 tons/day.
The natural aggregates and the gravel are natural resources of the environment that cannot be regenerated and considering the extension of the road network, it becomes imperative to find alternative solutions for the production of high-performance bituminous mixtures that meet the requirements imposed by the technical standards and regulations in force.
Thus, at the international level, there are several studies on the use of aggregates obtained from steel slags or furnace slags where conventional aggregates have been replaced with slag and the results have been satisfactory [6-11].
Fernando da C. G. M. in doctoral thesis [12], 2014 concluded that the slags show a high particle porosity and this is the reasonthat absorption of water by the slag is very high, the porosity is greater than 10 %. If the surface is too rough, then the adhesion to the bitumen is good, but this leads to using higher percentages of bitumen when a coarser fraction is used. Regarding sensitivity at moisture, it was found that, in general, the recycled aggregates with high microtexture and absorption provide good results. This research refers to slag obtained from the natural cooling of the fluid slag in contact with the environment temperature used in an bituminous mixture for the base course.
In Romania, researches in this field were made at the Technical University of Constructions Bucharest [13-14].
In Romania, authors experience [15] show that for better results of an ecological bituminous mixture it is recommended to use a combination of natural aggregate and blast furnace slag. Better results were obtained by replacing one limestone granularity class 16/31.5 with slag and by replacing 50 % limestone with a 50 % slag for each aggregate granularity class.
In 2015 at UTCB an bituminous mix was studied for the base layer AC 31.5 where three attempts were made on bituminous mixes (bituminous mix with 100 natural aggregates, bituminous mixture with 50 % aggregate of slag and one where they were replaced 100 % of the natural aggregates with the slag. The study's conclusions were satisfactory about the rigidity module, the recipe with the replacement of 50 % aggregate with slag gave a much better module than the conventional recipe and unsatisfactory results were given for the absorption of water that exceeded the limits imposed by the rules in force on slag aggregates [14].
This study presents laboratory research to study the possibility of using blast furnace slag in bituminous mixes in the base layer taking into account the effect of water on the behavior of the resulting composite material.
The paper has studied this topic in more detail by making 15 asphalt mix recipes and analyzing the performance of the 15 mixes compared with the requirements of the currently applicable European and national norms: EN 13108 and AND 605.

2 Experimental study

2.1 Used materials and asphalt mixture recipe

The study was carried out on the same asphalt mix recipe in the AC 31.5 D50/70 base layer, according to EN 13108-1, designed by the Marshall method, and the 15 different types of mixed recipes with partial and total replacement (integral granularity class) of natural aggregates are those in Table 1.

The natural aggregates used (grades 0-4, 4-8, 8-16, 16-31.5 mm) were aggregates of granite. Granite is a magmatic rock, gray color with homogeneous equigranular structure, massive and compact texture. The mineralogical composition consists of feldspar, quartz, and biotite. Crushed aggregates (sort 0-4, 4-8, 8-16, 16-31,5 mm) from the blast furnace slag from Arcelor Mittal Galati and filler and bitumen D50/70 pen.

The determination of the AC 31.5 base D50/70 asphalt mix recipe dosing is set out in Table 1 and the asphalt mix recipes with the partial and total replacement of the aggregates with the aggregates are listed in Table 2.

Table 1 Recipe AC 31.5 basis D50/70

The material used, granularity class	Percentage [%]
Size 16-31.5 mm	20
Size 8-16 mm	18
Size 4-8 mm	17
Size 0-4 mm	40
Filler	5
Bitumen D50/70 pen	4,4

Table 2 Variants of asphalt mix recipes

Recipe type	Natural aggregates, granularity class	Blast furnace slag, granularity class
A (the reference recipe)	0-4, 4-8, 8-16, 16-31,5 mm	-
B	4-8, 8-16, 16-31,5 mm	0-4 mm
C	0-4, 8-16, 16-31,5 mm	4-8 mm
D	0-4, 4-8, 16-31,5 mm	8-16 mm
E	0-4, 4-8, 8-16 mm	16-31,5 mm
F	8-16, 16-31,5 mm	0-4, 4-8 mm
G	0-4, 4-8 mm	8-16, 16-31,5 mm
H	4-8, 8-16 mm	0-4, 16-31.5 mm
I	0-4, 16-31,5 mm	4-8, 8-16 mm
J	0-4, 8-16 mm	4-8, 16-31,5
K	4-8, 16-31,5	0-4, 8-16 mm
L	0-4 mm	4-8, 8-16, 16-31,5 mm
M	4-8 mm	0-4, 8-16, 16-31,5 mm
N	8-16 mm	0-4, 4-8, 16-31,5 mm
O	16-31,5 mm	0-4, 4-8, 8-16 mm
P (100% slag)	-	0-4, 4-8, 8-16, 16-31,5 mm

2.1 Performed tests

Aggregates used for bituminous mixtures must fulfill some conditions to be used for this purpose and the characteristics of these aggregates are those set out in EN 13043 and the AND 605 normative. EN 13043:2003-AC/ 2004 is the standard setting the characteristics of aggregates and fillers obtained from natural and artificial materials to be incorporated into bituminous mixtures and surface finishes used in road, airport, and of other traffic areas.
Physical and geometric characteristics are given by the resistance to fragmentation with Los Angeles machine according to EN 1097-2, Micro-Deval wear resistance according to EN 1097-1, actual bulk density and water absorption according to EN 1097-5, fine parts content according to EN 933-1, resistance at magnesium sulfate test according to the standard EN1367-2, aggregate shape according to the standard EN 933-4 and bituminous binder affinity.
According to producer and EN 13043 requirements, the used natural and artificial aggregate show characteristics presented in Table 3.

Table 3 Characteristics of natural aggregates and blastnfurnance slag

Characteristic	Natural granitic aggregates				Blast furnace slag				Tehnical conditions according to:	
	0-4 mm	4-8 mm	8-16 mm	16-31,5 mm	0-4 mm	4-8 mm	8-16 mm	16-31,5 mm	EN 13043	AND 605
Fine parts content, %	1,5	1,0	0,5	0,5	0,01	0,05	0,08	-	Sand f_3- f_{22} Gavel$f_{0,5}$-f_4	Max. 3
Shape index, %	-	9,5	4,5	5,0	-	1,17	2,5	2,1	SI_{15}-SI_{50}	Max. 25
Bulk density, Mg/m^3	2,56	2,91	2,90	2,89	2,36	2,37	2,36	2,42	Declared value	Declared value
Resistance to fragmentation Los Angeles %	-	-	13,6	-	-	-	35	-	LA_{15}-LA_{50}	Max. 16
Wear resistance (Micro Deval)%	-	-	9.5	-	-	-	30	-	M_{DE15}- M_{DE50}	Max. 20
Resistance to magnesium sulfate test%	-	1.0	0.5	0.5	-	-	2,0	3,5	MS_{18}-MS_{25}	Max. 6

The slag used in this study, where the basic oxides with Ib = 1,2-1,3 (CaO / SiO_2) predominate, is a basic slag providing superior desulphurization but, compared to acid slag, is lower in density and is more fluid. Slowly chilled slags in the air tend to spontaneously disintegrate. This trend is due to the behavior of the dicalcium silicate, which at 1240°C passes from form γ into form β, and at 675 ° C passes into α-shape with monocrystalline structure, the transformation is accompanied by a volume increase of ~ 12 %, which causes the crystals to disintegrate.
According to the producer, the chemical analysis of slag shows the following percent for the main components: 31.92–36.76 % SiO_2, 33.51–41.49 % CaO, 7.62–9.10 % Al_2O_3, 6.24–6.82 % MgO, the difference up to 100 % being given by Fe, MnO, Mn_2O_3, Na_2O, K_2O.
In the design of AC 31.5 base D50/70 Asphalt Mixing Recipe References D50/70 in addition to quarrying and slag aggregates, filler was used which fulfilled the requirements of EN 13043 for granularity, the content of fine particles, water content, bulk density, stiffness characteristics (porosity of the Rigden compacted dry filler), but also the calcium carbonate content (table 4). The used bitumen characteristics are presented in table 5.

Table 4 Filler characteristics

No.	Filler characteristics	Results	Test method	Technical conditions according to EN 13043
1	Calcium carbonate content, %	91.4	EN 1744-1	Min. 90
2	Water content, %	0.5	EN 1097-5	Max. 1
3	Grading of filler aggregates: Passing through the:		EN 933-10	
	2 mm	100		100
	0.125 mm	94		85....100
	0.063 mm	92		70.....100
4	The apparent (bulk) density of filler in kerosene, Mg/m³	0.37	EN 1097-3 Annex A	Declared value
5	The porosity of the Rigden compacted dry filler, %	27	EN 1097-4	$V_{28/38}$
6	Assessment of fines. Methylene blue test(MB_f)	6.7	EN 933-9+A1:2013, Annex A	VB_f10
7	Determination of the particle density of filler. Pyknometer methodMg/m³	2.70	EN 1097-7	Declared value

Table 5 Bitumen characteristics

No.	Characteristic	Bitumen type D50/70	Test method
1	Penetration at 25°C (0.1 mm)	57	EN 1426:2007
2	Softening point (IB) (°C)	50	EN 1427:2007
3	Ductility: at 25°C	>100	SR 61:1997
4	Fraass breaking point (°C)	-12	EN 12523:2007
5	Determination of the resistance to hardening under the influence of heat and air–TFOT method		EN 12607-2:2015
	mass loss (%)	0.02	
	residual penetration at 25°C (%) of initial	61.0	
	residual ductility at 25°C, (cm)	>100	
	increasing the softening point (°C)	4	

Table 6 presents the results performed to have the affinity of bitumen to aggregate. An additive ATICA ABR-1 has been used in the recipe to determine if water absorption decreases but the value of water absorption was not influenced but the only affinity of bitumen to aggregate has increased.

Table 6 The affinity of bitumen to aggregate

Bitumen	Aggregate	Additive	Results
50/70	Granite	-	93.0 %
		0.4 %	98.5 %
	Slag	-	87.0 %
		0.4 %	96.7 %

The grading curve for asphalt mixture meets the European norm requirements EN 13108 as well as the Romanian standard requirements AND 605/2016.

2.2 Samples preparation

The specimens were compacted in the laboratory depending on performed tests. Thus, for Marshall Test and water absorption test, were manufactured cylindrical samples with 100.6 mm diameter and 63.5 mm high at Marshall Hammer for 50 blows compacting energy per side. To test the mixtures to water sensitivity in the laboratory were produced cylindrical specimens with Ø=100.6 mm and h≈63 mm at Marshall hammer for 35 blows/side. Samples preparation and mixing were according to EN 12697-35. During compaction, it observed a partial crushing of slag which may be explained by slag poor resistance characteristics.
To determine the influence of artificial aggregate on AC 31.5 base D50/70 asphalt mixture was conducted the tests: Bulk density, Water absorption, Water sensitivity, and Marshall test.

2.2.1 Bulk density

The apparent bulk density of the bituminous specimen is determined according to EN 12697-6, from the mass of the specimen and its volume. The mass of the specimen is obtained by weighing it in a dry state in the air. The volume of the specimen is obtained from the weight weighed in the air and the weight weighed in the water.

2.2.2 Water absorption (according to AND 605)

Water absorption according to AND 605 is the amount of water retained in the pores until saturation by the dry mass of the asphalt mix.

2.2.3 Water sensitivity (according to EN 12697-12)

Moisture sensitivity is expressed through ITSR - indirect tensile strength ratio which represents the ratio between the mean tensile strength of the wet samples and the mean tensile strength of the dry samples.

2.2.4 Marshall Test

The Marshall test (stability and index flow) according to EN 12697-34 is applied to verify the quality of different types of bituminous mixtures in terms of high-temperature behavior. The principle of the Marshall Stability is to determine the resistance to plastic flow of cylindrical specimens of a bituminous mixture loaded on the lateral surface. It is the load-carrying capacity of the mix at 60ºC and is measured in kN and the Marshall flow is the vertical deformation of the compacted specimen. Loading conditions followed the European norm EN 13108-20 and Romanian Norm.

2.3 Results

Following the above laboratory tests on cylindrical samples on the 16 types of AC 31.5 base 50/70 bituminous mixtures (A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P) the experimental results represented in figures 1-5 (bulk density, water absorption, Marshall's stability and flow, ITSR value) were obtained to highlight the influence of the aggregate on the physical-mechanical characteristics of the asphalt mixture. Figures 1 to 5 show the results obtained from the tests presented above according to the standard EN 13108-1 as well as within the limits imposed by the AND 605-2016 norm:

• Bulk density represented for the 16 mixtures - Figure 1;
• The water absorption represented for the 16 mixtures - figure 2;
• The Marshall stability and creep represented for the 16 mixtures - Figures 3 and 4;
• Water sensitivity represented for the 16 mixtures - Figure 5.

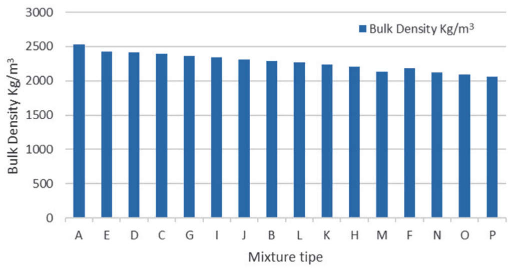

Figure 1 Bulk density of asphalt mix

It appears from figure 1 that the values of bulk density obtained for mixture with natural aggregate is higher than that obtained for the blast furnace slag mixture. The result is explained by the fact that the density of the slag aggregates is lower than that of the granite aggregates.

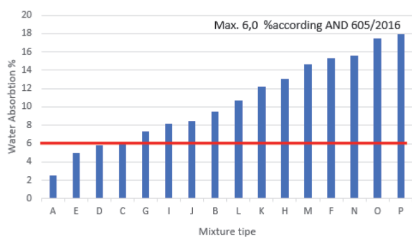

Figure 2 Water absorption of asphalt mix

The water absorption is 2.5 % at the reference mix compared to 17.9 % when mixed with 100 % slag (this very high value is explained by the very high porosity of the blast furnace slag). Good water absorption values were given in recipes C (4-8 mm replacement), D (8-16 mm replacement), E (16-31.5 replacement types) where only one type of natural aggregate was replaced with the blast furnace slag.
Lower Marshall stability values gave to the recipes where 3 types of slag were replaced and the replacement percentage exceeded 60 % of the blast furnace slag but also the recipe B where the 0-4 granite aggregate was replaced with the slag where the percentage was 40 %.

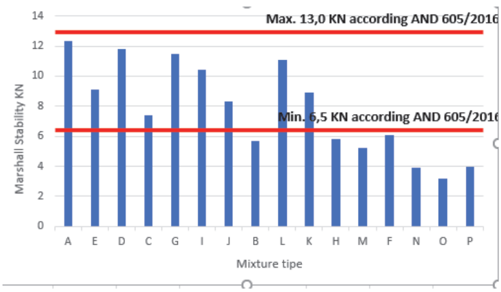

Figure 3 Marshall stability of asphalt mix

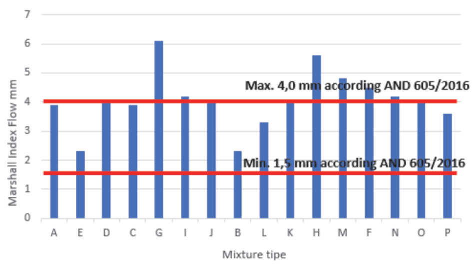

Figure 4 Marshall index flow of asphalt mix

The results indicate that unsatisfactory results of the Marshall index flow met in recipes where 2 granularity class were replaced (the recipes F, G, H, I where the percentage of granite aggregates replaced with blast furnace slag varied between 35-60 %) but also in the recipes where 3 types of slag were replaced (the recipes M, N, O were replaced percentage of granite aggregates with slag was over 75 %).

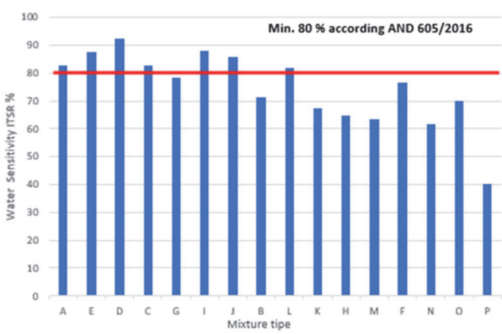

Figure 5 Water sensitivity ITSR of asphalt mix

The results indicate that good results of ITSR in receips C, D, E, I, J, L were obtained, where the percentage of granite aggregates with blast furnace slags varied between 17-65 %.

3 Conclusions

The use of slag asphalt mixes has several important advantages for environmental protection, thus reducing existing slag deposits in our country and saving local resources. In this study, we can see how aggregates that have a resistance to Los Angeles fragmentation and good Micro Deval wear influence positively the performance of the asphalt mix, especially the Marshall stability. Lower Marshall stability values gave to the recipes where 3 types of slag were replaced because the Los Angeles resistance and Micro Deval wear have high values on slag aggregates. During compaction, it observed a partial crushing of slag which may be explained by slag poor resistance characteristics. It was observed that furnace slag used as a coarse aggregate improved the mechanical properties of bituminous mixtures and thus resulting as suitable for use in the construction of the road. On the other hand, the use of 0-4 mm of slag type, the water absorption value increased greatly even by over 350 % (9.5 % compared to 2.5 %) compared to the reference mix and the value of ITSR decreased by more than 12 % compared with the reference mix A.

The results indicate that bituminous mixture with slag aggregates can be a valuable resource in designing bituminous mixtures and satisfactory performance has been achieved.

In the future, it will be necessary to study in the laboratory the modulus of stiffness but also the behavior of fatigue and permanent deformation of the bituminous mixture with furnace slag. Also, studying the bituminous mixture with other slag percentages and partial and not a total replacement of the 0-4 mm range may lead to a better conclusion about bituminous mixture with furnace slag behavior.

Another problem identified is to study in the future, the influence of additives on moisture susceptibility of bituminous mixtures with blast furnace slag.

References

[1] Brown, E.R., Kandhal, P.S., Zhang, J.: Performance Testing for Hot Mix Asphalt, National Center for Asphalt Technology (NCAT), Report 2001-05, Alabama, 2001.

[2] Barksdale, R.D.: The Aggregate Handbook, National Stone Association, Washington, D. C., 1991.

[3] Majidzadeh, K., Brovild, F.N.: Effect of water on bitumen- Aggregate Mixtures –State of the art, Special HRB Report, No. 98, Highway Research Board, 1968.

[4] Mallick, R.B., El-Korchi, T.: Pavement Engineering: Principles and Practice, New York: Taylor & Francis Group, 2009.

[5] Huang, Y., Guoping, X., Huigao, C., Junshi, W., Yinfeng, W., Hui, C.: An overview of utilization of steel slag, The 7th International Conference on Waste Management and Technology, 2012.

[6] Mohd, R.H., Nur, I.M.Y., Mohd, F.M.S., Mohd, A.A.A., Mohd, A.S.H., Wasid, F.R.: Steel Slag as an Aggregate Replacement in Malaysian Hot Mix Asphalt, International Scholary Research Network, ISRN Civil Engineering, Malaysia, 2012, doi:10.5402/2012/459016

[7] Ludfi, D., Hendi, B., Yulvi, Z.: Evaluation of the Use of Steel Slag and Pyrophyllite Materials in the Porous Asphaltmix, International Journal of Civil & Structural Engineering 2 (2013) 1, pp. 147-151

[8] Sofilić, T., Rastovčan-Mioč, A., Šosiš, M., Merle, V., Mioč, B., Sofiliš, U.: EAF Steel Slag Application in Croatian Asphalt Mixture Production, Sustainable Development, 2-4 June 2010, Rovinj, Croatia

[9] Praveen, M., Leni, S., Jaleen, G.: Steel Slag ingredient for concrete pavement, International Journal of Innovative Research in Science, Engineering and Technology, 2 (2013) 3

[10] Ziauddin, A.K., Rezqallah, H. M., Khalaf, A.A., Nafisullah, K.: Review of steel slag utilization in Saudi Arabia, The 6th Saudi Engineering Conference, KFUPM, Dhahran, December 2002.

[11] Deneele, D., De Larrard, F., Rayssac, E., Reynard, J.: Control of basic oxygen steel slag swelling by mixing with inert material, EUROSLAG, Oulu, Finland, June 2005, doi: 10.13140/2.1.2695.6162

[12] Da Conceição, G.M.F.: Misturas Betuminos as Temperadas com Incorporação de Subprodutos Industriais – Caracterização e Validação Tecnológica – Tese, Universida de Lisboa, 2014.

[13] Diaconu, E., Lazăr, M.S.: Recovery of steelworks slag in hot cylindering of bituminous asphalt mixture, Scientific Research Symposium, Road Administration "CAR", Bucharest, Romania, 2009.

[14] Lixandru, C., Dicu, M., Răcănel, C., Burlacu, A.: Ecological Solutions Applied to Hot Mix Asphalt, Romanian journal of transport infrastructure, 4 (2015) 1, doi: 10.1515/rjti-2015-0030

[15] Răcănel, C., Burlacu, A.: Ecological asphalt mixtures with blast furnace slag, SGEM 2016

[16] SR 61:1997- Bitumen, Determination of ductility;

[17] 605 Normative: hot expected asphalt mixtures technical conditions regarding design, preparation i implementation

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TRAFFIC:
SUSTAINABILITY AND INTERMODALITY

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INCLUSIVE MOBILITY – HOW TO TACKLE NEEDS AND CHALLENGES OF PERSONS WITH REDUCED MOBILITY

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Abstract

Mobility is an essential component of all European societies and is at the heart of the European Integration project. It is widely recognized that all citizens should be able to participate in economic, social and cultural life. European Union addresses investment in multimodal, environment-friendly, green, safe transport and mobility, to name some of the objectives, it seeks to achieve by mobilizing different funds. The idea of accessible transport is also high on the EU agenda. Accessibility is a multi-faceted objective, it can include the availability of information, the connection of metropolitan areas with rural or remote areas and also other aspects. However, the basic idea of accessibility in an integrated area should primarily mean the barrier-free mobility of people with disabilities and people with reduced mobility. This problem deserves to receive much more attention than is currently the case, considering that many recent studies estimate that the number of senior citizens and people with disabilities will double in the next 20 years.

Keywords: mobility and accessibility, inclusion, persons with reduced mobility

1 Introduction

A disability is an impairment that may be cognitive, developmental, intellectual, mental, physical, sensory, or a combination thereof. It significantly affects a person's life activities and may be present from birth or occur during a person's lifetime." Elderly People known as +65-year-olds, have the majority of the population of disabled people with physical impairments and changing mental states. "...Aging is rhetorically - sometimes ominously -invoked as a pressing reason why disability should be of vital interest to all of us (we are all getting older), thereby inadvertently reinforcing the damaging and dominant stereotype of aging as exclusively an experience of decline and deterioration. But little attention has been paid to the interconnectedness of aging and disability," argues [1]. Globally, there are several organizations such as WHO [2], Eurostat [3, 4], European Federation of Retired and Elderly People (FERPA) and American with Disabilities Act, Public Health Agency of Canada, Independent Living, and Rehabilitation Research [5] that publish valuable statistics. The EU-28 population is estimated to be 510.3 million, with 19.2 % of the population aged 65 and older [3]. Furthermore, the population aged 65+ increased by 2.4 % over the last decade. Wright [6] has provided an overview of the average age and percentage of people aged +65 in the UK and associated countries. It also shows how the UK and EU countries have aged between 1985 and 2010 and are projected to age by older and disabled people every year until 2035.

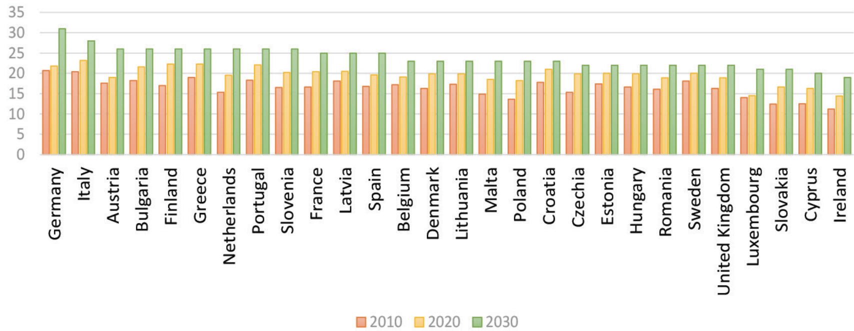

Figure 1 Percentage of persons aged 65 and over across EU by a country for years 1985 and2010 and projection for 2035; adapted from "Office for National Statistics" [3]

There has been significant progress in the implementation and also adoption of ITS technologies to remove barriers in the physical mobility of PRMs through the provision of e-government, e-health services, e-housing or even the ease of use of video conferencing, teleshopping etc. The penetration of e-government in the population varies and, as expected, decreases with age, so it is expected to have a very limited impact on older people or people with disabilities, who often and in many parts of Europe still do not have the same opportunities for education and access to technology.

Therefore, physical mobility is one of the most important features that people need for almost all activities, e.g. social contacts, visiting friends, work, leisure, shopping - physical mobility is a basic requirement. Although today some physical trips can be replaced by virtualization (e.g. teleworking, video conferencing), physical movements will never be completely replaceable by technological inventions. Therefore, physical transport systems must be as user-friendly as possible to meet the needs of all people, including mobility-impaired people.

The UN Convention on the Rights of Persons with Disabilities, adopted in 2006 at UN Headquarters in New York, is one of the most important documents defining, promoting and protecting the fundamental rights of persons with disabilities. Accordingly, the EU member states have adopted the law on ratification of the UN Convention on the Rights of Persons with Disabilities, which makes the Convention an integral part of national legislation.

2 Mobility and accessibility for all

In recent decades, great progress has been made in Europe in improving the accessibility of public transport for people with reduced mobility. However, this progress has not been substantial in all areas and varies considerably across Europe. While people with reduced mobility are passively protected by a number of protected rights enshrined in EU and national laws, the situation in practice varies from country to country and especially from region to region in the EU, despite clear objectives in the document „Transforming our world: the 2030 Agenda for Sustainable Development“, namely to:

- reduce inequality within and between countries (Goal 10);
- make cities and human settlements inclusive, safe, resilient and sustainable (Goal 11).

Based on the Convention on Road Traffic, commonly known as Vienna Convention on Road Traffic, every driver of a motor vehicle must hold appropriate documentation; a driving license (also known as a driving permit) can only be issued after passing a theoretical and practical test, which are regulated by each country or jurisdiction. A driving license issued by an EU member state is recognized and can be used throughout the EU as long as it is

valid, the driver is old enough to drive a vehicle of the relevant category, and the license is not suspended or restricted and has not been revoked in the issuing country. People with disabilities (PwD) and reduced mobility (PRM) are entitled to obtain a driving license if they pass tests, but they may have some restrictions determined by an expert. The EU has created uniform codes for restrictions due to health conditions and general codes for car adoption needed to enable PwD and PRM drive safely. The codes are added on the driving license.
According to the legislation, people with various disabilities are also entitled to have a driver‘s license (can be with some restrictions), but national standpoints and willingness to help and support this group of people differ. From countries where people with disabilities are reimbursed for car purchase and adoption and the cost of obtaining a driver‘s license, to countries where people with disabilities are discouraged and disprivileged to drive. Accessibility to public places and services, as well as to public transport, correlates strongly with the understanding and acceptance of persons with disabilities as equal members of society.

2.1 Needs and challenges in the field of disabled person’s mobility

At the European level and in terms of legislation, EU member states have ratified the UN Convention on the Rights of Persons with Disabilities which obliges parties (states) to make their transport systems accessible “on an equal basis with others”. There is some improvement in transport accessibility at EU level, but there is a lack of mainstreaming of accessibility requirements, national accessibility legislation is not harmonized and is characterized by significant fragmentation.
The accessibility of transport, barrier-free mobility, varies widely across EU countries. Small member states tend to perform better, while other countries still have much potential for improvement. Improvements are needed in every area, e.g. legislation, harmonization, funding, training.
Even in countries like Austria, which can be considered as a good performing country, accessibility varies a lot in different provinces/regions. Vienna, the capital of Austria, for example, has often been described as one of the most accessible cities by public transport, not only in Europe but in the world. Progress over the last 40 years in terms of public transport accessibility in Vienna has been significant, as Figure 2 shows. The web-based route planner in Vienna, which has been online since 2009, offers people with disabilities and persons with reduced mobility the possibility to choose journeys only on busses that are suitable for them (e.g. low-floor busses).

Figure 2 Vienna Tramways – adoptions for (disabled) people [10]

When it comes to long-distance transport, as mentioned above, the situation varies from province to province, but it is worth mentioning one of the good practices found in Austria. Namely, there is an information platform for barrier-free holidays in Austria on the website https://euregio-barrierefrei.eu/en, which provides holiday-related information for wheelchair users and all other mobility-impaired travelers.

Figure 3 Barrier free ticket machines (ticket machines for everyone) installed in Vienna [10]

According to the clustering of EU countries on the basis of transport accessibility in the Research for TRAN committee, countries can be divided into groups: 1) Low-achievers (e.g. Hungary), where both the legal framework and implementation require improvements, 2) Late-achievers (Slovenia, Croatia), lacking serious implementation, but also countries with general accessibility plans that include all modes of transport, 3) Countries with good or adequate legal frameworks (Bulgaria, Romania) with limited resources leading to low implementation, and 4) Countries with better opportunities for long-distance transport and room for improvement in local transport accessibility.
The general problems of persons with disabilities and reduced mobility in Southern Europe range from dealing with a bad legacy from past times with prejudices and stereotypes, through scares attempt to build policies of inclusion and non-discrimination, to more adequate measures and policies - which are still not fully developed - to enable barrier-free life for persons with disabilities and reduced mobility.
To improve this situation, a wide range of requirements must be addressed, starting with building a positive image of PWDs and PRMs, accessibility of information, accessible public and private transport, including barrier-free access to indoor and outdoor spaces. In terms of safety and independence, it is very important to create hazard-free streets and buildings, safe roadways and signage for drivers and pedestrians, and safe, accessible and affordable public transport.
Limited accessibility, lack of information, and other problems with public transportation force people with disabilities and those with limited mobility to choose private transport - usually cars - to gain or maintain their independence. Even when there is good accessibility to public transport, the last mile is traveled by car, as is the case for the general public who do not face similar barriers. It is worth noting that disabled people and people with reduced mobility have a desire and need to reduce their dependency as much as possible. This in turn leads to economic and social benefits for society as a whole. For this reason, transport policy should aim to meet the aspirations of non-disabled and disabled people alike. A particular part of the mobility of people with disabilities and RM relates to the opportunities to move around by public transport, i.e. busses, trains, taxis. These opportunities directly depend on the adaptability of the transport infrastructure at the stops for people with disabilities, on the accessibility of the access points for these people and, above all, on the adaptability of

the vehicles for their boarding and safe travel. The practice of accessibility and adaptation of public transport for people with disabilities in a state varies, depending on the cities, and often with the common problem of not meeting all the elements necessary for the smooth movement of these people, for example, not all public transport infrastructure (vehicles, platforms, etc.) are adapted for the boarding of people with disabilities.

The ability to drive and be independent is not the privilege of all PwD and but where this potential exists, improving the social, economic and cultural integration of people with disabilities and persons with disabilities into local and wider society leads to a higher quality of life of PwD and PRM. Countries and governments should devote all possible attention to empowering PwD and PRM to be mobile and independent by driving themselves and obtaining their own car. To realize the full potential of self-driving of PwD and PRM, a number of conditions need to be met from the perspective of national and local governments:

- availability of funding and support for car adaptations and car ownership,
- of procedures for PwD and PRM to obtain/retain a driving license,
- the availability of (accessible) parking spaces for people with disabilities.
- a higher driving culture in terms of respecting parking places for disabled persons (not occupying the parking space and leaving space necessary for people with disabilities and reduced mobility to get into the car),
- harmonized physical and mental examinations of disabled persons to determine their ability to drive and the conditions (e.g. type of car adoption) under which PwD and PRM are allowed to self-drives,
- accessible roadways and roadsides.

Given the attention that PwD and PRM have received, at least declaratively, one might assume that in most European countries the simplest step to achieving/maintaining independence through self-driving, where possible, is regulated and largely guaranteed. However, the situation in the EU varies from the attitude of “anyone that wants can drive” (e.g. Sweden) to the attitude of “any disability is a barrier for driving” (e.g. Western Balkan countries).

The study from 2000 conducted in 15 EU Member States, shows that even in countries with well-developed public transport systems, people, both non-disabled and disabled, prefer to use cars; trips by private cars are about eight times more preferred than trips by public transport.

At this point, it should be noted that enabling independent self- driving where appropriate would have a positive impact on the overall quality of life of PwD and PRM but would also economically create a whole new, previously untapped economy of accessible tourism. In the study for the TRAN Committee, it is estimated that in the EU Member States alone, demand from tourists with disabilities and older people is estimated at 780 million travels, resulting in €400 billion revenues per year, and is expected to grow by 1 % per year in the coming years. However, it is estimated that only 9 % of tourism services in the EU28 provide accessible services (and even these have different levels of accessibility due to the lack of harmonised accessibility standards). Furthermore, estimates show a potential increase in demand for accessible travel and tourism of 44 % per year, which could be achieved if appropriate accessible offers were created.

2.2 Status quo in Slovenia

The total population of Slovenia is about 2.067 million people and the average life expectancy of Slovenian men and women is 78.2 and 84.3 years, respectively. The share of people over 65 in the total Slovenian population is about 19.2 % and is estimated to reach 26 % by 2035. More than 15 % of the total EU population are PwDs and PRMs. In Slovenia, the proportion is around 12-13 % of the total population, which is not significantly different from the

estimates for the EU. There are no official data on the number of PwD and PRM in Slovenia; the number is estimated on the basis of registers for each category of disability.
One of the fundamental rights of all people, including PwD and PRM, is accessibility to public and private sector services and the physical environment. This includes the ability to use public transport. In addition to the EU Regulation on the rights of passengers when using public transport (all means of transport, i.e. land services, airplane, or waterborne mean of transport), which imposes certain obligations on public transport providers, the national Equalization of Opportunities for Persons with Disabilities Act aims to ensure that disabled persons in Slovenia can use public transport on the same terms as others. The main measures envisaged in the national legislation, which represent a step forward in ensuring mobility and accessibility of public transport for all, are the following: adaptation of busses and long-distance busses and trains, availability of information on the possibility of using public transport, bus and train stations must have barrier-free entry and exit, information must be accessible in the techniques adapted for PwD and PRM, if a disabled person uses a wheelchair, guide dog, etc., he/she must not be charged extra costs, to name a few.
lovenia has committed itself to implementing the Convention, but similar to the other EU countries, the process of implementation is long and takes place mainly in terms of the country's financial possibilities. Nevertheless, it can be said that the situation is visibly improving. As far as public transport passengers are concerned, a wheelchair user can travel by train between all major Slovenian cities, as the stations allow boarding and alighting, but not all trains are fully adapted to PwD and PRM. People in wheelchairs can use public transport, such as busses, in Ljubljana and Maribor. In intercity bus transport, the possibility to transport them is limited for the time being, as the vehicles are not adapted for the transport of persons with disabilities in wheelchairs. Specially adapted taxis are available in Ljubljana for transporting wheelchair users and other disabled people. In order to provide a free (or minimal cost) transport service, many disability organizations in Ljubljana, Maribor and some other larger Slovenian cities have specially adapted minibusses to transport their members. At Jože Pučnik Airport they provide exemplary access for PwD and PRM to the plane. However, in the area of information and ticket sales, the current situation is not so promising. Apart from some newer vehicle information systems (e.g. voice announcement of stops on busses and trains), rare Braille signs and marked corridors for access to vehicles, there is hardly any other assistance for PwD and PRM.
In Slovenia a disabled person who can drive the vehicle on his own, only if the vehicle is properly adopted, can claim the cost of adapting the vehicle once every six years. The disabled person who does not operate the vehicle by himself/herself is also entitled to the adaptation if the adaptation is necessary for the disabled person to get into the vehicle and to ensure a safety aspect of driving. In this case, the disabled person can also claim the costs for devices and accessories to bridge the height difference when getting into the vehicle and for the relevant systems to attach the wheelchair to the vehicle once every six years. The type of adaptation of the vehicle, the conditions for the adaptation, the durability, the quality standards for the adaptation and the type of maintenance are specified in the Regulations on technical aids and adaptation of vehicles. The disabled person is entitled to the reimbursement of 85 % of the value of each adaptation, for a disabled person with poor economic status, the reimbursement can also be up to 100 %. No funds are available for co-financing the purchase of a (new) vehicle.

3 Efficient improvements towards equal mobility

Even the most advanced counties in terms of accessibility for all still have room for improvement in many areas that would greatly enhance the mobility of people with disabilities and limited mobility, such as:

- Provide passenger information at bus stops that is easy to understand both visually and audibly.
- Provide bus/train platform edges with visual and tactile strips.
- Make barrier-free parking spaces along the road as possible so that people do not have to get on the road when getting out of the car.
- Provide controls on parking ticket machines at appropriate heights and with tactile design.
- Design stairs with high-contrast step marking, handrails, and no hazards.
- Provide some counters at wheelchair accessible heights and accessible to wheelchair users.
- Remove barriers from otherwise accessible routes and make staff aware of accessibility issues.
- Make information about the facility's accessibility available on the Internet.
- Develop solutions for the different needs of the individual groups of disabled persons together with those who needs this kind of solutions.
- Ensure a continuous barrier-free mobility chain and take into account the design principle "Design for all" and the "two senses principle" (always two senses so that in each case one can act).
- Develop appropriate standards, regulations and recommendations on a regular basis and implement them consistently.

References

[1] Adams, R., Reiss, B., Serlin, D.: Keywords for Disability Studies, First. ed, New York University Press, New York University Press, New York, 2015.

[2] World Health Organization, Better health for people with disabilities: Infographic, 2016.

[3] Eurostat, People in the EU – statistics on an ageing society - Statistics Explained 13, pp. 1–20., 2015.

[4] Eurostat, EU-Commonwealth of Independent States (CIS) - statistics on active ageing - Statistics Explained, European Commission, http://ec.europa.eu/eurostat/statistics-explained/index.php/EU-Commonwealth_of_Independent_States_(CIS)_-_statistics_on_active_ageing, 26.11.2017.

[5] Salzman, C., Matter, R., Ganapathiraju, A.: A Planning Guide for Making Temporary Events Accessible to People with Disabilities, ADA National Network. Washington DC, 2015.

[6] Wright, E.: Population Ageing in the United Kingdom, its Constituent Countries and the European Union, Population Trends 139 (2010), pp. 91–114. https://doi.org/10.1057/pt.2010.7

[7] European Commission, European Urban Mobility, Policy Context, European Union, Brussels, 2017.

[8] European Commission, Innovative and Sustainable Urban Mobility Solutions, CIVITAS, Brussels, EU, 2017.

[9] European Commission, A Concept for Sustainable Urban Mobility Plans, Brussels, EU, 2013.

[10] 13th World Conference on transport research, http://www.wctrs-society.com/wp-content/uploads/abstracts/rio/selected/2913.pdf

CARSHARING AS AN INTEGRATED MEAN OF TRANSPORTATION - A COHESIVE PLANNING APPROACH FROM THE CITY OF WIESBADEN, GERMANY

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Abstract

In Germany and beyond there is a widespread social and political discussion on a "mobility turnaround", which refers to the technological and behavioural change of the entire transport sector towards sustainability goals. Within that approach of a "mobility turnaround" the concept of car-sharing is universally seen as a central component. This attribution is mainly based on the presumed combination of the advantages of rationally using a car on one hand without baring the negative social effects of a private-car ownership.
Until recently, however, it has not been legally possible in Germany to reserve exclusive parking spaces for carsharing vehicles in public spaces. Carsharing stations could therefore only be located in private spaces, which in turn was a strong limiting factor in the expansion of the service. With the introduction of the German "Carsharing Law" in 2017, municipalities were given the possibility to identify and assign exclusive parking spaces in public areas, which now serves as an instrument for targeted planning of carsharing services.
The following contribution presents an overview of the typical organizational and operational forms, use-cases and user groups of casharing services. The positive and negative effects are identified, classified and discussed. Further, the paper proposes how an ideal carsharing service should be designed from a municipal and transport planning perspective.
Based on this general findings, the contribution presents the exemplary approach of the city of Wiesbaden. The municipality allocates public spaces to private-sector providers based on a defined comprehensive network concept. Within that concept public spaces are only tendered to providers in accordance with clearly defined targets and operational standards. With that approach Wiesbaden is proactively fostering a city-wide carsharing network as an integrated mobility service.

Keywords: carsharing, urban mobility, multimodality, sustainable mobility, traffic planning

1 Introduction

For a good 30 years, carsharing has become increasingly widespread as a new transport service in many countries. Carsharing refers to mostly commercial offers in which a car can be used independently by different people. In contrast to classic car rental, carsharing cars can also be booked for short periods (usually hourly) and are available close to the journey's starting point.
In the sense of sustainability oriented transport planning, carsharing is an attractive mode of transport as it can help to tackle the dominance of private cars by reducing its numbers to a reasonable and city-sensitive level.

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One of the main arguments in favour of carsharing is its potential to gradually reduce private car ownership, which results in a decreasing need for parking space in favour of qualitatively upgraded public space. Furthermore, carsharing decreases car usage in general as each single trip is preceded by a more rational consideration of alternatives and a conscious decision rather than an automatic choice of transport based on routine and comfort.
Transportation planning is faced with the question of how to design a reasonable and attractive carsharing service. The answer to this question should not be left to commercial carsharing providers alone, but must be part of a cohesive and integrated transport planning approach on a municipal or regional level, so that the best possible integration into existing transport systems can succeed.
In the following, we will first show which forms of carsharing services do exist and which effects those different forms have on the individual mobility behaviour.
Then, using the example of the city of Wiesbaden, possible approaches are discussed as to how a reasonable and attractive carsharing service can be developed within existing planning structures. Finally, potential tools and instruments that a municipality can use to implement an integrated service are elaborated and proposed.

2 Basics of Carsharing

2.1 Types of service

There are two fundamental types of different carsharing services [1]: Free-floating and station-based carsharing. Station-based carsharing represents the classic form of carsharing in which the vehicles are assigned to a fixed station. Pick-up and return of the vehicles must take place at the same station, i.e. only round trips with return to the starting point are possible. The stations are usually marked by signs on a specially designated zone and hold several carsharing vehicles. Each carsharing car must be booked in advance for a fixed period of time. The rates are based on the duration of the booking period and the distance travelled.
Free floating carsharing was first introduced in Germany in 2009. The providers offer a large number of vehicles for flexible use in defined business areas, usually areas in and around central parts of major cities. The vehicles are not assigned to fixed stations, but can be parked in public parking spaces within the specified business area. This allows one-way trips between any points in the business area. Booking and reservation in advance is not possible. The rates refer solely to the actual period of use.
In addition, there are also hybrid forms combining the two main concepts (called “station-flexible” or “combined carsharing”): As with station-based carsharing, the vehicles are only available at defined stations, but can be returned at other stations as well, so that one-way trips are possible.

2.2 Dissemination in Germany

At the beginning of 2021 a total of 26,220 carsharing vehicles are available in Germany [2]. Despite a growth of 400 % within the last ten years in the number of carsharing vehicles, this corresponds to only 0.05 % of all passenger cars in Germany. In contrast, the number of people registered with a car-sharing provider results add up to 2.87 million, which corresponds to 3.54 % of the German population and roughly 5,0 % of all driver’s licence holders in Germany. Of these, however, only one third regularly use carsharing [3].
Around half of the vehicles are offered within station-based systems and the other half with free floating systems. As free floating systems are concentrated in 15 major cities, station-based systems are offered in a total of 855 municipalities across Germany. 83 % of German cities with a population over 50,000 inhabitants hold a carsharing service.

Four large commercial providers operate in Germany. Outside the metropolitan areas, however, carsharing is often offered and operated by voluntary associations on a non-profit basis [2].

3 Carsharing as an integrated mean of sustainable transport

3.1 Effects of carsharing services

In recent years, a large number of national and international studies have analysed the effects of carsharing on overall car use, on the mobility behaviour of users, and on car ownership (e.g. [4], [5], [6], [7], [8]). Even if the results vary depending on the research question, design and methodology, they overall show a significant potential of carsharing towards a more sustainable mobility.
This thesis is exemplified by the results of the EU-funded STARS project [9]. As a key finding the study shows that carsharing customers of station-based systems have reduced the number of privately owned cars by almost two-thirds overall compared to the period before registering as customers. This clearly confirms the potential of carsharing to reduce the number of privately owned cars in general as well as affecting the common mode choice of its users, 91.4 % of whom use a car less than weekly.
The situation is considerably different with customers of free floating systems: here, the decrease in private car ownership is less than 5 %. Free floating carsharing is probably mainly used as a substitute for public transport or cab when the private car is not available or situationally not suitable. 61 % of free floating carsharing customers use a car at least once a week, however, only one third of these trips are made by carsharing.
A good 67 % of station-based carsharing customers state they use a car less frequently after signing up for carsharing, while 38 % of free-floating customers show reduced general car use.
Various studies have used different empirical methods to determine how many private cars a carsharing vehicle practically replaces. The results vary widely, but show predominantly high values from 1 : 7 (one carsharing vehicle replaces 7 private cars) up to 1 : 20, especially for station-based systems.

3.2 Municipal goals and requirements for a carsharing service

For years, the predominant transport policy goal of many municipalities has been to reduce the burden of car traffic and to enhance the quality of public spaces more, while at the same time ensuring mobility. Carsharing can significantly contribute to this by reducing vehicle trips and easing the demand for parking space, as described above. Station-based systems in particular appear effective, while the benefits of free-floating systems are not as apparent.
In order to achieve the primary goals, from the perspective of municipal transport planning a carsharing service must meet various requirements:

- wide and dense spatial service coverage to enable efficient system access for as many people as possible,
- different types of vehicles for different purposes (e.g., small cars, vans),
- environmentally friendly vehicles,
- integration into the overall transport system, e.g. through integrated information, booking and payment platforms for carsharing, public transport, bike sharing and other mobility services.

3.3 Challenges in the expansion of carsharing

The growth of station-based carsharing, which seems particularly promising from a sustainable transport policy perspective, is hampered by limited space available for stations within the cities. In the past, carsharing companies had to rent private parking spaces because the designation of exclusive station areas on public ground was not legally permissible. Additionally, in neighbourhoods with high parking demand, where carsharing could potentially relieve parking pressure significantly, private parking spaces are especially scarce and expensive. Moreover, stations on private ground are often publicly not well visible, so that the service is only known and used to a limited extent.
Since 2017, the federal Carsharing Act [11] has in principle created the legal framework to provide carsharing on public ground. Since carsharing providers are usually private companies, but the intended road space is publicly owned, many questions must be addressed before stations can actually be set up. In particular, municipalities need to consider how the carsharing service should be designed to meet their transportation policy goals. While the projected locations of the stations should also be attractive for the carsharing providers to meet their economic expecations. In this area of sometimes conflicting priorities, completely new types of planning considerations must be initiated.

4 Municipal planning strategies - the example of Wiesbaden

4.1 Project framework

The city of Wiesbaden has politically and strategically set the goal of proactively shaping mobility and urban transport. To this end, attractive services for multimodal transport are to be introduced. Mobility stations are to be rolled out throughout the city as points of connection between different multimodal modes of transport, a central component of which will include the service of carsharing. Within this planning framework, the city of Wiesbaden plans to introduce a cohesive, visible and attractive carsharing service as an integrated mode of public transport for the entire city area.
Central elements of the presented project consist of a city-wide potential analysis with regard to the various capabilities and use-cases of carsharing, the search for locations on a macro level, the consideration of the defined macro locations as mobility stations, and the preparation of a transparent and legally compliant implementation procedure. A strategic benchmark of 100 stations on public ground was politically predetermined, so that a multi-stage process needed to be developed for the distribution and location of these 100 public stations.

4.2 Site selection procedure

The site selection contains a quantitative and qualitative approach. The quantitative location selection defines where in the city area stations are to be located, the qualitative location analysis then defines the specific service quality of each carsharing station. In the quantitative location selection, a distribution system is developed that initially distributes the 100 stations proportionally to the respective city districts based on the factors population and population density. It was strategically determined beforehand that even the smallest districts should obtain at least one station in order to theoretically provide access to a carsharing service to every resident.
In the next step of the site selection, a GIS-based accessibility analysis of the existing public transport system was then carried out to identify both central nodes and gaps in the system. Both aspects can be important in the strategic design of an integrated carsharing service.

Where there is no sufficient public transport service, carsharing as part of public transport can close specific service gaps and thus reduce the dependency on private cars; where, on the other hand, there are central transport nodes in the public transport network, carsharing can function as an important part in an intermodal travel chain integrated in mobility stations.
Then, urban characteristics were identified for each district at the macro level in order to ensure good accessibility and a large catchment area for each station, as well as to enable high visibility through a clear allocation in the area of existing infrastructures (e.g. local bank, supermarket). Based on these spatial criteria, the number of stations determined in the first step were then located for each of the 26 districts of Wiesbaden.

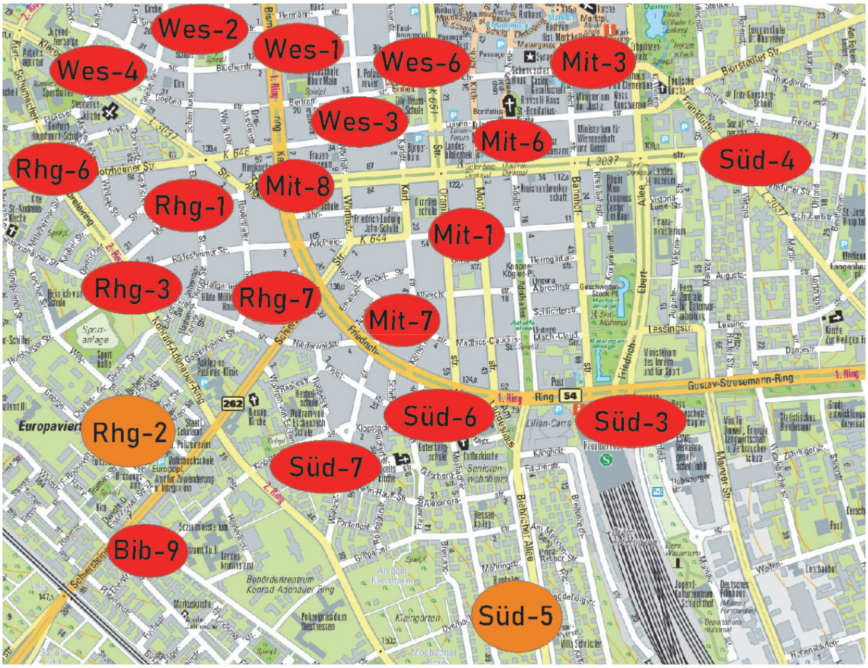

Figure 1 Exemplary section of the defined car sharing stations at macro level in Wiesbaden city centre with clear coding for later specific allocation (red are L-stations, M-stations are orange).

4.3 Different service levels

In order to achieve the best possible transport benefits, it is not only important to define where the stations are located, but also what (minimum) services they should provide. Therefore, three different station categories were initially developed: Small stations (“S-stations”) each have 1 to 2 vehicles that universally cover the widest possible range of uses (e.g. small family car). The service is solely station-based.
Medium-sized stations (“M-stations”) provide 2 to 5 vehicles per station in a station-based concept. The range of vehicles is mixed, from small cars to vans, in order to meet different usage requirements.
“L-stations” have 4 or more vehicles in a combined system of station-based and semi-flexible services, so that one-way journeys from one “L-station” to another are possible.
In the next step, the defined station types are assigned depending on the population density at the district level. Rural and peripheral districts with a population density of less than 1.000 people per km² are predominantly assigned “S-stations”, residential and mixed-use areas

close to the city are predominantly assigned “M-stations” and urban districts with a population density of more than 3.000 people per km² are predominantly assigned “L-stations” with semi-flexible services.
The proposed systematic station distribution at macro level is a data-based transport planning recommendation. The individual locations must all be examined again in detail at the small-scale level in accordance with urban planning criteria and, if necessary, coordinated with local stakeholders.

4.4 Roll-out strategy

After quantitatively and qualitatively locating 100 carsharing stations throughout the city, in the last project step a roll-out strategy was developed. In an immediate action programme, 35 stations will initially be implemented in all larger city districts (> 7.000 inhabitants) with theoretically high carsharing potential. To analyse districts with a high carsharing potential, the factors affinity to environmental transport (derived from modal split), car availability per household, purchasing power and parking pressure in public spaces were considered.
The aim of this immediate action programme is the prompt introduction of a large-scale carsharing service to enable low-threshold entry and to generate visibility of stations and vehicles on the road. This immediate action programme should be accompanied by professional marketing and scientific evaluation. After the successful implementation of the first 35 stations and the succeeding awareness of the carsharing service, the next 65 stations will be implemented gradually expanding to all districts of the city as well as further densifying the service in the high potential areas.
In addition to these 100 stations, the city of Wiesbaden is aiming to integrate local businesses and public administrations into the city-wide carsharing service as a further important element. By using carsharing companies and public administrations can reduce their own vehicle fleet when, in return, they have fixed contingents from the carsharing pool at their disposal throughout their business hours. Outside business hours and at weekends, these carsharing vehicles are then available to all public users. The potential is estimated at around 30 additional stations for commercial cooperation partners citywide, all of which must be proactively acquired through targeted marketing. The cooperation with local companies and administrations has the opportunity to further expand the carsharing service with private engagement and at the same time to gain stable anchor-customers.

4.5 Allocation procedure for stations

For the implementation of the developed network, an effective and transparent procedure for the allocation of stations to specific carsharing providers needs to be established. In theory, the city of Wiesbaden determines requirements on the service quality and carsharing providers then agree to those requirements by applying for tendered stations. In practice, however, this enters uncharted territory in terms of planning and public law. Therefore, a step-by-step approach is necessary in which potential service providers and municipalities discuss and negotiate realistic parameters of the service design in joint exchange.
Questions to be addressed include how to distribute the stations fairly among the providers, what requirements for vehicle availability are reasonable and what data the city needs from the providers to monitor the quality of service and as a basis for transport planning? The proceedings in Wiesbaden had not yet been completed when this article was written.

5 Conclusion and outlook

Carsharing can have an important impact on more sustainable mobility as it has the potential to make private cars redundant, decrease car use, and reduce parking spaces in cities. The carsharing service must be well integrated into the overall transport system to be successful. With carsharing municipalities mainly aim to achieve transport planning goals. The carsharing providers, in turn, primarily pursue economic goals. Municipalities and carsharing providers must therefore jointly design the service in order to create the best possible solution for both parties. The planning procedures and the rules for cooperation have yet to emerge in practice. The carsharing concept for Wiesbaden can be a promising approach for this new type of public and private co-creation in transport services.
The unique charaterictic of services such as carsharing compared to rather more conventional transport infrastructure measures is their potential for dynamic adaptation to new developments, innovations and changes in the framework conditions during their lifetime. They therefore can perform as potential learning systems. To enable this learning potential, constant monitoring of success is vital; digitization provides the necessary technical basis for this. The effective utilisation of digital possibilities by service providers and municipalities alike is a key success factor for carsharing and other new mobility services.

References

[1] Blees, V., Zerban, M.: Neue Mobilitätsangebote: Ersatz oder Ergänzung des ÖPNV? Eine Analyse am Beispiel des stationsflexiblen Carsharings, ed. Mertens A., Ahrend, K.M., Kopsch, A., Stork, W., Smart Region - Die digitale Transformation einer Region nachhaltig gestalten, Wiesbaden, pp. 375 – 399, 2021.

[2] Bundesverband Carsharing: Aktuelle Zahlen und Fakten zum CarSharing in Deutschland, https://carsharing.de/alles-ueber-carsharing/carsharing-zahlen/aktuelle-zahlen-daten-zum-carsharing-deutschland, 20.02.2021

[3] Nobis, C., Kuhnimhoff, T.: Mobilität in Deutschland – MiD-Ergebnisbericht (im Auftrag des BMVI), Bonn, Berlin, 2019.

[4] Rid, W.: Carsharing in Deutschland: Potenziale und Herausforderungen, Geschäftsmodelle und Elektromobilität, Springer Vieweg, Wiesbaden, 2018.

[5] Bogenberger, K., et al.: Entwicklung und Nutzungsstruktur von Carsharing-Systemen in Deutschland, ed. Jacoby, C., Wappelhorst, S.: Potenziale neuer Mobilitätsformen und –technologien für eine nachhaltige Raumentwicklung, Arbeitsberichte der ARL 18., Hannover, pp. 157 – 174, 2016.

[6] Schick, P., Herffs, G.: Flächenhaftes Carsharing-Stellplatzkonzept in der Stadt Freiburg, Straßenverkehrstechnik 3, pp. 189 – 197, 2017.

[7] Schreier, H., Becker, U., Heller J.: Evaluation CarSharing (EVA-CS) Landeshauptstadt München, Endbericht, Berlin, 2015.

[8] Hülsmann, F. et al.: share – Wissenschaftliche Begleitforschung zu car2go mit batterieelektrischen und konventionellen Fahrzeugen, Freiburg, Frankfurt am Main, 2018.

[9] Nehrke, G., Loose, W.: Nutzer und Mobilitätsverhalten in verschiedenen CarSharing-Varianten, Projektbericht zum Projekt STARS - Shared mobility opporTunities And challenges foR European citieS, Berlin, 2020.

[10] Bundesverband CarSharing: Wirkung verschiedener CarSharing-Varianten auf Verkehr und Mobilitätsverhalten, CarSharing Fact Sheet Nr. 3., Berlin, 2016.

[11] Gesetz zur Bevorrechtigung des Carsharing (Carsharinggesetz - CsgG) vom 5. Juli 2017 (BGBl. I S. 2230), zuletzt geändert durch Artikel 328 der Verordnung vom 19. Juni 2020 (BGBl. I S. 1328)

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PLANNING AND DESIGNING INFRASTRUCTURE AND SERVICES FOR SUSTAINABLE BICYCLE TOURISM ALONG THE EUROVELO ROUTES IN THE DANUBE REGION

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Abstract

Before the Covid-19 pandemic air travel was growing steeply and “flight shame” became one of the catch phrases of the climate crisis. Too often urban citizens undermined eco-friendly workday mobility with long-distance holiday air travel. The arrival of the Covid-19 pandemic posed new challenges. The recreational needs had to be satisfied by domestic tourism closer to home. There is a need for innovative sustainable tourism products and concepts to respond to both of these trends. Bicycle tourism combined with public transport for destination access and egress is a promising candidate for low carbon and regional tourism concepts. The Danube region is among the most important European cycle tourism destinations. A long stretch of the EuroVelo route 6 follows the river Danube, other EuroVelo routes cross the region. The European Interreg-project EcoVeloTour combines three main elements to support new sustainable tourism concepts in this corridor: ecotourism, use of public transport to access the destination or starting and final points of cycle tours and cycling within the destination region. Key elements of the EcoVeloTour approach are sustainable mobility and ecotourism guidelines. The guidelines utilize synergies between sustainable multimodal mobility planning, including cycling infrastructure, and cyclist ecotourism related services and ecotourism development (e.g. destination management, marketing, product development) along the EuroVelo routes. The guidelines for sustainable bicycle tourism provide a comprehensive basis for planning and improving all relevant mobility-related infrastructure and services. The chapter “Infrastructure for high-level bicycle tourism” deals with relevant road infrastructure elements like different types of tracks, intersections and roundabouts, route signposting, bicycle parking, shelters for cyclists, lighting and maintenance. The chapter “Transport services and intermodality” addresses public transport use for transfers to origin and from final destination of bicycle tours. It describes infrastructure, information and services needed at intermodal nodes. Regional bicycle tourism strategies and pilot projects are elaborated based on the EcoVeloTour guidelines. An interactive self-assessment tool to support strategy development and pilot actions of the regions was developed and tested in transnational learning interactions.

Keywords: bicycle tourism, ecotourism, intermodality, public transport, infrastructure

1 Introduction

Before the Covid-19 pandemic air travel was growing steeply. In the decade 2009-2019, the number of air passengers worldwide increased from 2.6 to 4.5 billion [1]. This corresponds to a growth of 80 per cent. The annual growth rate varied between 3.6 per cent (2018) and 8.7 per cent (2009). In the same period, the number of passenger kilometres travelled increased

from 4,565 billion to 8,686 billion. This corresponds to a growth of 90 per cent. The average length of a flight increased by around six per cent. As air travel is the source of large amounts of greenhouse gas emissions, “flight shame” became one of the catch phrases of the climate crisis and the Fridays for Future movement [2]. In the past urban citizens have undermined their eco-friendly workday mobility too often with long-distance holiday air travel. This trend calls for innovative products and concepts to mitigate the climate crisis and create a sustainable future tourism sector. Bicycle tourism combined with public transport for access and egress of holiday destinations is a promising candidate for future low carbon tourism concepts. Arrival and departure of the average cycling tourists is much more climate-friendly than that of the average summer tourists. Arrival and departure of the average German cycling tourist produces 56 kilogram CO_2 emissions while the average German tourists produces 248 kilogram CO_2 emissions [3]. While cycling tourism is still a niche market, its popularity was continuously rising in recent years. In 2018 about four per cent of the summer tourists in Austria stated cycling as the main motive for their summer holiday [4]. In Germany the number of cycling tourists increased from 4.0 million in 2014 to 5.4 million in 2019 [5]. This corresponds to an increase of 35 per cent in five years or an average yearly growth rate of six per cent.
The arrival of the Covid-19 pandemic in Europe posed new challenges for our society in general and the tourism industry in particular. Travel restrictions and lockdowns had a severe impact on tourism. In the year 2020 tourism suffered its deepest crisis. International arrivals dropped by 74 per cent worldwide and by 71 per cent in Europe [6]. Under these conditions the basic human need for recreation had to be satisfied by domestic tourism and travel closer to home. Open-air activities, nature-based products and rural tourism have emerged as a popular travel choice [6]. This trend was accompanied by a rising interest in ‘slow travel’ and community-based tourism, linked to a more sustainable, authentic and responsible experience [7]. During summer 2020 cycling tourism has proven its resilience and its capability to satisfy basic recreational needs of a population in lockdown. In such times touristic cycling infrastructure helps to meet the need for physical outdoor activities and hence supports the resilience of cities [8]. During the first Austrian lockdown starting on the 15th of March 2020 mobility in Vienna collapsed by between minus 46 per cent (workplace) to minus 86 per cent (retail and recreation) [9]. Between calendar week eleven and twelve the number of bicycle trips measured by the twelve Viennese automatic counting points decreased by 22 percent [10]. Contrary to this bicycle traffic increased between four and 32 per cent at three locations on routes along riverbeds which have a high share of recreational use. This observation reflects the importance of adequate cycling infrastructure for the local and regional population in times of crisis. Providing appropriate infrastructure for cycling improves the resilience of a society.
The Danube region is among the most important European cycle tourism destinations. A long stretch of the EuroVelo route 6 follows the river Danube and other EuroVelo routes cross the region. On the EuroVelo route 6 between Passau and Vienna a total of 774,000 cyclists was counted in 2019 [11]. About 23 per cent were multi-day tourists, 33 per cent day tourists and 41 per cent everyday cyclists. All these trends and facts form the starting point and background of the Interreg-project EcoVeloTour (Fostering enhanced ecotourism planning along the Eurovelo cycle route network in the Danube region) [12].

2 The project EcoVeloTour

2.1 Study area

The Danube region was chosen as the case study area of the project EcoVeloTour for two reasons. First, as mentioned above parts of the region are already well developed destinations for cycling tourism. Second, the region has a unique natural ecosystem with a highly diverse biological and cultural heritage. Nevertheless, the region faces significant challenges. Some sections of the EuroVelo cycle route network run through less developed regions. As a result, the quality of cycling and tourism infrastructure also varies significantly along the Danube. Apart from the section Passau – Vienna many stretches of the EuroVelo network fail to attract tourists due to their poor infrastructure quality. Insufficient infrastructure and lack of workplaces, especially in rural areas, induce migration within the EU, leading to brain-drain and losses of social and cultural knowledge. This increases economic disparities in the Danube region. The unique ecosystem is negatively affected by increasing anthropogenic interventions. There is a need to change the mind-set towards services provided by nature. The ecosystem services concept provides a promising framework to simultaneously protect and utilise nature in the Danube region. The project team is led by the Municipality of Zuglo in Budapest and includes partners from Austria, Bulgaria, Germany, Hungary, Romania, Serbia and Slovakia. Hence, the study areas include seven of the eight countries along the river Danube.

2.2 Objectives

The main objective of the project EcoVeloTour is to develop coherent regional ecotourism offers along the EuroVelo cycle routes 4, 6, 9, 11 and 13 in the Danube region. The challenges mentioned in section 2.1 are tackled by combining the following three elements: ecotourism, use of public transport to access the destination or starting and final points of cycle tours and cycling within the destination region. The results of this combination are new sustainable tourism strategies and concepts, which can also be applied in similar regions elsewhere. Further sub-objectives of the project EcoVeloTour are:

- Create an environmentally sound management framework, and policy and planning recommendations to integrate ecosystem services into tourism.
- Utilize synergies between Eurovelo cycle route infrastructure and ecotourism development including coordinated inter- and intraregional sustainable mobility planning, ecotourism planning and product development.
- Facilitate the promotion of cyclist ecotourism destinations in the Danube region in line with ecotourism strategies including the implementation of small-scale pilot actions and targeted communication and awareness-raising activities.

The following sections describe the elements that are employed by the project EcoVeloTour to achieve the abovementioned objectives.

2.3 Market research and stakeholder analysis

The starting point of the project activities was a transnational market research about the actual situation and trends of cyclists' ecotourism across the Danube region. The aim of the transnational market research was to get an overview about regional specifics and structures of the tourism sector in the participating countries along the EuroVelo network, focusing on bicycle and ecotourism [13]. The sample consisted of the project partners and relevant stakeholders (e.g. tourism boards, administrative districts) from all seven partner countries.

In total 118 persons took part in an online survey carried out between February and April 2019. The results of the market research illustrate the heterogeneity of the different regions along the Danube. On average 48 per cent of the respondents stated that cycle tourism is a main focus point of their regions strategic positioning in tourism. The variation by country ranges from seven per cent in Romania to 97 per cent in Germany. The respondents from Austria, Germany and Slovakia see the age group 40 to 59 years as their main target group. In Bulgaria, Hungary, Romania and Serbia the main target group is the age cohort 20 to 39 years.

The market research was accompanied by comprehensive stakeholder analysis in the Danube region. In depth interviews with 18 experts from all involved countries have been carried out [14]. The interviews revealed that the stakeholders have positive attitudes towards ecotourism in the Danube Region and see the development of cycling as an important way to achieve the objective of sustainable development. Concerning a common Danube regional marketing strategy, the majority of respondents showed a positive attitude, although some doubts concerning the complexity of this task were raised.

2.4 Tools and guidelines for planning support

The second key element of the EcoVeloTour approach was the development of sustainable mobility and ecotourism guidelines [15], [16]. The aim of the guidelines is to facilitate the development of regional strategies and planning processes. The guidelines utilize synergies between sustainable multimodal mobility planning, including cycling infrastructure, and cyclist ecotourism related services and ecotourism development (e.g. destination management, marketing, product development) along the EuroVelo routes. The guidelines for sustainable bicycle tourism provide a comprehensive basis for planning and improving all relevant mobility-related infrastructure and services. The chapter “Infrastructure for high-level bicycle tourism” deals with relevant road infrastructure elements like different types of tracks, intersections and roundabouts, route signposting, bicycle parking, shelters for cyclists, lighting and maintenance. The chapter “Transport services and intermodality” addresses the topic public transport use for transfers to origin and from final destination of bicycle tours and holiday destinations. It describes the infrastructure, information and services needed at intermodal nodes.

An interactive self-assessment tool was developed on the basis of these guidelines. A mock-up of the tool was tested in transnational learning interactions. During the project EcoVeloTour the tool is used as an instrument to collect feedback from project partners and external stakeholders. The final version of the tool will be part of the EcoVeloTour e-learning platform. The main aim of the tool is to support stakeholders in the early phases the development of strategies and projects. The tool supports the users by asking concrete, practical and detailed questions, e.g.: How easily can tourists travel to your region in an eco-friendly way? How do you assess the maintenance of the bike routes? Is there enough information about bike rental services? Help texts direct users to specific sections of the guidelines if they are in need of support. Nevertheless, this strategic tool is not a substitute for detailed expert planning and assessment.

A more detailed planning and assessment tool for bicycle infrastructure has been developed at the Institute for Transport Studies. This tool enables a detailed quality-assessment of bicycle facilities for sections and longer routes of bicycle infrastructure, as well as whole regions. The tool can be applied to everyday cycling as well as to leisure or touristic routes. The core assessment starts with homogeneous stretches of infrastructure. Characteristic criteria are collected to rank each section according to internationally approved quality criteria. In a second step all intersections along the route are assessed for quality criteria, which are a different set. In a compilation, the average quality of the route or region can be assessed. In a second step, also signposting and touristic offers like accommodation, gastronomy, resting places, cultural assets etc. can be assessed. The quality of a route cannot be simply defined as a mean value of all criteria values of all individual sections. If some criteria fall below certain thresholds, this cannot be compensated by good grading for other criteria. E.g. a section of the route running on a road with high levels of heavy goods traffic cannot be compensated by good resting places. Therefore, two categories of criteria were defined: strictly mandatory criteria ("must have") and non-mandatory criteria ("nice to have"). The criteria and the complete assessment have been tested in real life conditions [17]–[19]. Currently the project team is working on harnessing synergies between these two tools.

2.5 Strategies and small scale investments

Six project partners elaborated regional ecotourism strategies based on the developed framework and guidelines (section 2.4), good practices, potential ecosystem service mapping of pilot areas and learning interactions at regional and transnational level. The strategies cover the most typical geographic regions within the Danube region. The guidelines and local knowledge have been put in practice in the development of these strategies. As an example, the strategy for the region Ruse in Bulgaria makes a proposal for a 185 kilometre long circular bicycle route. The purpose of the route is to combine unique natural and cultural features into a single touristic route using a brand, which summarises the characteristics of the area into one recognizable and distinct product. Based on a previous project the brand "Rivers of time", referring to the history along the rivers Danube, Yantra and Rusenski Lom, was chosen. The strategy suggests a "Greenways" concept, which combines cycling with hiking, horse riding and other sustainable forms of tourism. The strategy furthermore developed a detailed plan for the construction of quality cycling infrastructure, promotion of cycling tourism, data collection and impact assessment.

Lessons learnt from the market research (section 2.3), the guidelines for planning support (section 2.4) and the strategy development were incorporated into seven concrete pilot projects covering the topics information and signposting, innovative accommodation and rest areas and automatic counting stations.

The Serbian project partner Danube Competence Center developed a concept for smart bicycle rest places [20]. Smart bicycle rest places are objects for temporary residence of hikers or cyclists along routes through the Republic of Serbia. It should be possible to locate the rest places directly at routes including environmentally sensitive areas. Therefore, the construction has to be minimally invasive and the supply with water and energy has to be self-sufficient. This results in a modular design based on standard shipping containers, solar panels and rainwater collection. In a pre-survey with a sample of 88 Serbian respondents about 58 per cent state that the concept makes perfect sense. Another 40 per cent state that the concept makes sense, but they are not certain how these would function in real life. Figure 1 shows a rendering of a smart bicycle rest place directly located at a bicycle route [20]. A bachelor student of the Institute of Transport Studies of the University of Natural Resources and Life Sciences Vienna used such renderings to test the attractiveness of the concept among young Austrian people [21]. About 41 per cent of the 173 respondents in the age group 14 to 30 years see the concept as very attractive (Figure 1, right side), another 43 per cent see it as attractive. Nearly 85 percent rate the concept as attractive or very attractive.

Figure 1 EcoVeloTour smart bicycle rest place concept [20], [21]

Some of the small scale investments have been implemented and are operational. A bicycle rest place including a watch tower was built in the Subotica-Palić region in Serbia. The opening ceremony took place on the 30th of October 2020. Automated bicycle counter have been installed along the EuroVelo route 6 in Serbia. Work on bicycle rest places, signposting and automated counters in the Kosice region is partially finished and partially ongoing.

3 Conclusions

Tourism is currently facing its worst crisis in history. Past and ongoing trends call for innovative, more sustainable and resilient tourism products. The combination of ecotourism and cycling tourism is a promising candidate for such new products. Especially when these are combined with sustainable, intermodal mobility offers for arrival and departure. A market research and stakeholder analysis carried out in the Danube region identified the high potential of this approach. On this basis, the project EcoVeloTour developed a range of planning tools. Guidelines for ecotourism and sustainable mobility are the backbone of these tools. The guidelines have been used and tested in the development process of ecotourism strategies for six regions. Feedback from this process will be incorporated into the final version of the guidelines, which will be published at the end of the project. An interactive self-assessment tool was developed as a spin-off of the two guidelines. This tool facilitates an easy assessment of the status quo ahead of strategy development and planning processes. It is possible to identify strengths and weaknesses of a route or region and thus, helps to focus resources in the most prominent areas. The tool can be easily combined with other planning tools on a more detailed level. Strategies and small scale investments have been developed and partly implemented on this basis. First surveys demonstrate the high acceptance of the EcoVeloTour concepts and products.

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References

[1] ICAO, Presentation of 2019 Air Transport Statistical, www.icao.int/annual-report-2019/Documents/ARC_2019_Air Transport Statistics.pdf., 2019.

[2] Gössling, S., Humpe, A., Bausch, T.: Does 'flight shame' affect social norms? Changing perspectives on the desirability of air travel in Germany, Jorunal of cleaner production, 2020, doi: https://doi.org/10.1016/j.jclepro.2020.122015.

[3] VCÖ (Hg.): Klimafaktor Reisen, VCÖ-Schrif, Wien, VCÖ - Verkehrsclub Österreich, 2020.

[4] Österreich Werbung, Radurlauber in Österreich (Sommer 2018), Wien, www.austriatourism.com/tourismusforschung/studien-und-berichte/radurlauber-in-oesterreich-sommer-2018/, 2018.

[5] ADFC: ADFC Bicycle Travel Analysis, 2020.

[6] UNWTO: Tourism suffers its deepest crisis with a drop of 74 % in 2020 in international arrivals, World Tourism Barometar, 19 (2021) 1

[7] Kaernten.ORF.at, 2020 bringt Tourismus Minus von 17 Prozent, hkaernten.orf.at/stories/3086979/, 17.02.2021

[8] Pfaffenbichler, P.: Touristic cycling infrastructure supports the resilience of cities in times of crisis, EcoVeloTour Discussion paper, Vienna, 2020.

[9] Google, COVID-19 Community Mobility Report - Austria, 11.04.2020.

[10] Rottenberg, T.: Corona in Zahlen: Abwärts, Falter, Vienna, pp. 45–46, 15.04. 2020.

[11] ARGE Donau Österreich: Positive Bilanz 2019 - Erstmals mehr als 10 Millionen Ankünf¬te, Linz, 2020.

[12] EcoVeloTour, Fostering enhanced ecotourism planning along the Eurovelo cycle route network in the Danube region, http://www.interreg-danube.eu/approved-projects/ecovelotour/news 29.04.2020

[13] Biedersberger, A., Harasser, J., Waldmann, D., Mang, S.: EcoVeloTour WP3 Activity 3.2A - Transnational Market Research final results - benchmark, Passau, 2019.

[14] Kolos, K., Kiss, K., Kovács, E., Hegedüs, S.: EcoVeloTour - Danube Regional Stakeholder Analysis, Budapest, 2019.

[15] Hegedűs, S., Kasza-Kelemen, K., Nagy, R.I., Patak, G., Pinke-Sziva, I., Varga, Á.: WP3 Ecotourism planning - Guidelines for ESS-based ecotourism strategy, Budapest, 2019.

[16] Gauster, J., Hartwig, L., Meschik, M., Pfaffenbichler, P.: EcoVeloTour WP3 Ecotourism Planning - Guidelines for Sustainable Bicycle Tourism, Vienna, 2019.

[17] Ferstner, T.: Qualitätskriterien im Radverkehr - Bewertung des Radverkehrsangebots für den Alltagsradverkehr bzw. Radtourismus, Universtität für Bodenkultur Wien, 2020.

[18] Hacker, D., Kiener, C.: Radfahren in der Südstadt Maria Enzersdorf, Universität für Bodenkultur Wien, 2019.

[19] Matz, S.: Neugestaltung der verkehrlichen Situation am Campus Türkenschanze, Master Universität für Bodenkultur Wien, 2017.

[20] Petrovic, B.: Study on SMART bike rest places concept development, Belgrade, 2020.

[21] Graus, K.: Attraktivitätssteigerung von fahrrad-basiertem Tourismus für Jugendliche, Universität für Bodenkultur Wien, 2021.

ELECTRO MOBILITY ACCEPTANCE: THE INFLUENCE OF POLITICAL BONUS AND MALUS FACTORS AND PREFERENCES FOR CHARGING STATIONS

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Abstract

As a considerable amount of greenhouse gas emissions are caused by the transport sector with road traffic being the biggest polluter, the German government has initiated programs to promote electric vehicles (EVs). Currently, the main activity is to install charging infrastructure and to provide a financial bonus for the purchase of EVs. As part of the project "Electric City Russelsheim", CAPI interviews have been conducted to determine the acceptance of EVs among the population. The survey aims to investigate the effects of possible bonus and malus factors to promote EVs. Moreover, it analyses people's preferences for the configuration of charging stations in a discrete choice experiment. In choice tasks, respondents indicate their preferences by choosing a charging station configuration between two alternatives. Preliminary results from a Multinomial Logit Model on a sample of 462 respondents are presented in this paper. As configuration, respondents mostly prefer Plug & Charge as authentification method, card-based payment method, billing according to the amount of charged electricity, and a higher share of energy from renewable sources.

Keywords: stated preferences, discrete choice experiment, electric vehicles, electric mobility, charging stations, charging infrastructure

1 Introduction

With a share of almost 20 %, the transport sector is one of the largest sources of greenhouse gas emissions in Germany, whereby road traffic is the biggest polluter within this sector [1]. One method to reduce the emission, is the promotion of alternative fuel vehicles [2]. Therefore, the German government initiated a program "Sofortprogramm Saubere Luft 2017-2020", which provides a financial bonus for the purchase of electric vehicles (EVs) and develops charging infrastructure. Within this program, a large-scale project "Electric City Russelsheim" has been initialized to equip the city Russelsheim with charging infrastructure all over the place. The installation of the charging stations for EVs is accompanied by a social research study, which aims to get insights about the acceptance of EVs and to derive recommendations for future practices to promote EVs in the population. This paper is part of the social research program and focuses on insights about people's preferences for charging infrastructure.
Currently, the charging stations are sparsely distributed across Germany, whereby this distribution shows spatial clusters with the highest density in few metropolitan cities [3]. However, for the spread of electric cars, it is not only necessary to install a nationwide charging infrastructure [4], but also to increase the user-friendliness of the charging stations. At present, the configurations of charging stations are very heterogeneous in Germany [5], making a

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simple and user-friendly handling difficult. At the same time, findings show, that consumers are not only sensitive to the mere presence or density of charging stations, but are more concerned with attributes such as costs, location, duration and waiting times [3]. However, yet, preferences for charging stations have not been fully studied by mainly focussing on willingness-to-pay [3] or on preferences for the infrastructure [2] rather than on user-friendly configuration. To provide a customer-friendly charging, other attributes such as registration, authentication, and payment methods are important to investigate, since previous studies have shown that these attributes seem to have the largest discrepancy between desired configuration and current situation [6]. Therefore, the research question covered in this paper, is: Which preferences do potential users have for the design and configuration of charging stations? To answer this question a survey study including a stated preferences experiment to assess the preferences for the configuration of charging stations was conducted. Preliminary results from this survey effort are presented in the following.

2 Methodology

2.1 Study area, field work and sample

The survey area is Russelsheim am Main, surrounded municipalities and the city of Wiesbaden. Adults of 18 years and older have been recruited from a sample of 6,107 addresses. Potential respondents were contacted with an introduction letter and a follow-up recruitment phone call. An incentive of 20 Euros was offered to the respondents. Data were collected in computer-assisted personal interviews (CAPI), programmed as a Java application. The fieldwork started in January 2020 as Faceto-Face interviews in respondents´ household. Due to COVID-19, this procedure had to be stopped in March to be adopted to web-based CAPI-interviews conducted via an online video-based communication tool. The new field work period started in May to be finished in December 2020. A total sample of n = 462 respondents has been achieved after data cleaning.

2.2 Stated preference: Study design

To provide recommendations for the configuration of charging stations, this study aims to assess people's preferences by applying a stated preferences (SP) approach [7], where respondents are faced with pre-defined configurations of charging stations and can choose between pre-defined alternatives. To define the alternatives of charging station configurations, relevant attributes together with possible levels have to be specified [8]. This was done with reference to previous literature on possible configurations in Germany [5], [6] and preferences for configurations [3], [9]. In addition, a qualitative study was in the field in Russelsheim and collected peoples´ views on charging stations. Finally, the focus lies on the attributes authentification method, payment opportunities, billing method and share of electricity from renewable energy sources in the energy mix which is offered at the charging station. In this study, respondents need to choose between two alternatives of configurations. The alternatives are defined by experimental design, where for each attribute one of the possible levels is assigned. So, attribute levels are varied in the experiment. A brief description of the attributes, their functionality and the levels are presented in Table 1.

Table 1 Stated Preferences experiment: attributes, levels and description

Attribute	Level	Description of the level
Authentication	Plug & Charge	automatically by connecting the charging cable to the charging station
	RFID	by using a card from the provider with a Radio-frequency identification (RFID) chip
	App	via an app installed on a smartphone
Payment	web-based	via web-based services (e.g. PayPal)
	card-based	via card-based services (e.g. credit card)
	debit transfer	automatically via direct debit transfer (contract with the provider is necessary)
Billing	by electricity	price is based on the actual amount of electricity charged
	by time	price is based on the time (the longer you charge, the more expensive)
	fixed fee	fixed price for a charging process
	flat rate	unlimited charging at a fixed price (e.g. monthly)
Share of electricity from renewables	0%	0 % from renewable energy sources
	50%	50 % from renewable energy sources
	100%	100 % from renewable energy sources

An efficient experimental design [8], which uses only a subset of all possible choice tasks (combinations of attributes and levels), has been created with the software Ngene [10]. It resulted in 72 choice tasks split into six blocks with 12 choice situations in each block. Finally, every respondent was randomly assigned to one block and asked to answer the according choice situations. Hereby, as presented in the example in Table 2, respondents stated their preferences as discrete choices [8] between two unlabeled alternatives of charging stations "Configuration 1", "Configuration 2" and a labelled alternative "I do not choose configuration 1 or 2" (for more details on the study design please refer to previous work [11]).

Table 2 Stated Preference experiment: example of a choice task

Which configuration would you prefer for a charging station?			
Authentication	RFID	Plug & Charge	
Payment	web-based	debit transfer	
Billing	fixed fee	by electricity	
% of renewable energy	100%	50%	
	○ Configuration 1	○ Configuration 2	○ I do not choose configuration 1 or 2

In addition, the survey instrument collects information on the household, living situation, socio-economic and demographic characteristics, but also socio-psychological factors (e.g. intention to buy an EV, environmental awareness), since they are expected to have an impact on preferences for charging infrastructure [2].

2.3 Model specification

Discrete choice data are commonly analysed by applying Random Utility Maximization theory. It assumes that when completing a choice task as in Figure 1, respondents associate an utility with each alternative and are assumed to choose the alternative with the highest utility [12], [13]. In more detail: an individual n is confronted with j alternatives in t choice tasks. Hereby, an individual n associates an indirect utility U_{njt} for an alternative j in a choice task t and chooses the alternative with the highest utility. The utility of an alternative j is decomposed as

$$U_{njt} = V_{njt} + \varepsilon_{njt} = x'_{njt}\beta + \varepsilon_{njt} \tag{1}$$

Where U_{njt} is not observed, V_{njt} is the deterministic utility (known) of alternative j and ε_{njt} is a random error (cannot be measured directly). The deterministic utility V_{njt} can be specified by $x'_{njt}\beta$. Hereby x, is a vector of explanatory variables (e.g. attribute levels, socio-demographics), and β are the coefficients to be estimated, which indicate the utility associated with the explanatory variables in x.

In our experiment, respondents choose between three utilities associated with the J = 3 alternatives (configuration 1, configuration 2, neither configuration 1 nor 2). All alternatives are described by K = 4 attributes (authentification, payment, billing, share of renewables). Consequently, three utility functions (V_{njt}) need to be specified for each alternative to measure the utilities associated with the attribute levels. Hereby, the equations for the unlabelled alternatives are identical [14], since the options "Configuration 1" and "Configuration 2" themselves are generic and do not have a meaning and thus have the same utility for the respondents [8], [14]. For the labelled alternative "neither 1 nor 2" a constant β_0 (ASC) will be estimated.

The levels of the categorical attributes authentification, payment, and billing have been transformed to dummy variables (0 = not applicable, 1 = applicable). Hereby, for every attribute one level is omitted from analysis, which serves as reference category and whose utility is fixed to zero. Therefore, the parameter estimates for the remaining levels of the attribute capture the utility differences to the reference category [13], [14]. The attribute "share of renewable energy" has shown an approximately linear behaviour in the analysis and thus has been included as a continuous variable into the utility functions [13].

To assess differences in preferences of men and women, this study follows the segmentation approach used by previous studies [3], [15]. This aproach suggests to estimate separate models for males and females. Further, interactions with age have been tested for all dummies of the categorical attributes (authentification, payment, billing). Since the interaction parameters were not significant on the 95 % level and did not contribute to model improvement, the reduced model has been chosen as the best. As the attribute „share of renewables" is included as a continuous variable, a continuous interaction with age will be included into analysis following a technique applied by previous work [15]. Hereby, an interaction parameter $\lambda_{age,renewables}$ will be estimated to assess the sensitivity towards the attribute „share of renewable energy" in dependence to age and which answers the question: With increasing age, do people become more or less sensitive towards the share of renewables in the energy mix at charging stations?

3 Results

A multinominal logit model (MLM) [16] has been applied on the total sample (n = 462) which results in 5,603 observations (choice tasks), but also separately for males (n = 327 individuals, n = 3,972 observations) and for females (n = 135 individuals, n = 1,631 observations).

All analyses have been done with R using the package Apollo. The estimation results for the MLM models are presented in Table 2 and the preferences are visualized in Figure 2.
As mentioned previously, the parameter estimates for dummy-coded levels of each attribute capture the utility differences to the reference category [13], [14]. Thus, for each attribute only differences in utilities (preferences) between the attribute levels can be interpreted.
For Authentication, Plug & Charge has the highest positive estimate (β = 0.288) and thus the highest utility. The estimate of RFID card is lower (β = 0.139) and thus is less preferred than Pug & Charge, but it is positive, which means that it is more preferred than the reference category of an authentification via App. The difference in utilities for Plug & Charge to App is higher for males than for females.
Concerning payment, respondents prefer a card-based method most: When automatic debit transfer is used as reference category, the utility for card-based method is positive (β = 0.196) and thus has a higher utility. Web-based payment shows the least utility for respondents as the parameter is negative (β = -0.103) and is thus less preferred than automatic debit transfer. Especially women associate highest utility with card-based method, since the parameter estimate shows a larger difference to the reference category than for males.
As billing method, respondents mostly prefer to pay according to the amount of electricity they charged, since its parameter estimate has a higher utility (β = 0.876) in comparison to the reference category flat rate. Moreover, in comparison to the flat rate option the billing method by time (β = -0.312) or as a fixed fee (β = -0.401) show negative parameter estimates and thus lower associated utility. Especially females dislike the billing by time, since the difference of its estimate to the reference flat rate is larger for females than for males.
Finally, respondents prefer charging stations with higher share of electricity from regenerative sources as for each increase by 1 % of renewables, the associated utility increases by 1.533 points. The positive utility is true for males and females. With a negative value for $\lambda_{age,renewables}$ (λ = -0.842) the sensitivity to the share of green energy sources decreases when age increases [15]. Thus, when people become older, they become less sensitive towards the share of energy from renewable energy sources in the energy mix they would charge. This is true for both, males and females, since the estimate is negative for both subsamples.
The specified model for the total sample shows an adjusted Rho-squared (ρ^2) of 0.107. It can be considered as an adequate model fit, since values between 0.2 to 0.4 are recognized to be indicators of very good models [14, p. 54].

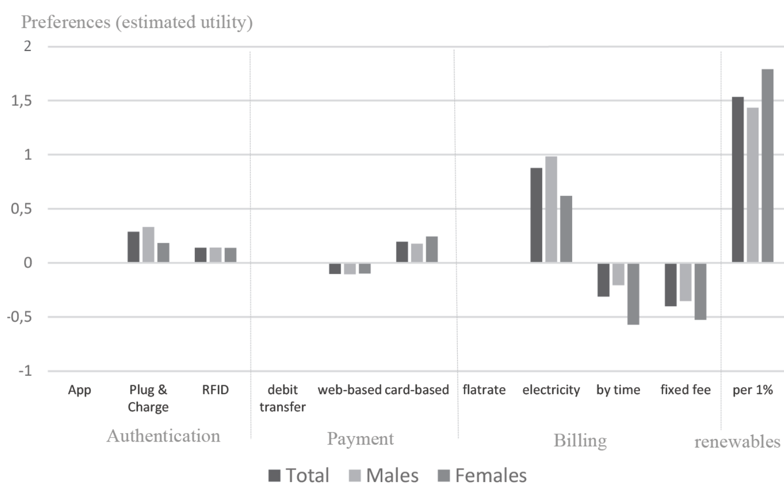

Figure 1 MNL results: Estimated preferences for the total sample, males and females

Table 3 Estimation results for MLM model for the total sample, males and females

	Total sample			Males		Females	
	est.		t-ratio	est.	t-ratio	est.	t-ratio
ASC (None)	0.685		9.015	0.714	7.865	0.614	4.411
Authentification (Reference level: App)							
Plug & Charge	0.288		4.736	0.332	-0.106	0.183	1.614
RFID	0.139		3.561	0.141	0.177	0.138	1.905
Payment (Reference level: debit transfer)							
web-based	-0.103		-1.946	-0.106	-1.682	-0.099	-1.012
card-based	0.196		3.838	0.177	2.921	0.243	2.554
Billing (Reference level: flat rate)							
by electricity	0.876		12.829	0.983	12.037	0.620	4.934
by time	-0.312		-4.488	-0.207	-2.504	-0.572	-4.419
fixed fee	-0.401		-5.736	-0.353	-4.228	-0.527	-4.083
renewable energy [%]	1.533		21.171	1.434	16.908	1.789	12.934
$\lambda_{age,renewables}$	-0.842		-8.423	-0.686	-4.983	-0.998	-6.643
Individuals:		462		327		135	
Observations:		5,603		3,972		1,631	
log-likelihood(Null):		-6155.525		-4363.688		-1791.837	
log-likelihood(Final):		-5484.732		-3891.389		-1582.708	
Adjusted ρ^2:		0.107		0.106		0.111	

4 Discussion

The survey study aims to perform analysis and provide insights on political bonus and malus factors and their impact on the promotion of EVs as well as preferences for configuration of charging stations. This paper presents preliminary results of the analyses of respondent's discrete choices between different charging stations. The initial results provide deep impressions of respondent's preferences with respect to configuration of charging stations: For authentication, Plug & Charge is the mostly preferred method. Respondents prefer to pay with a card-based method or via an automatic debit transfer (second preferred) and do not want to use web-based procedures. A billing according to the charged amount of electricity is the most preferred option. In addition, a higher share of electricity from regenerative sources is preferred, whereby people become less sensitive when getting older.

In future steps, interactions with additional socio-demographic variables (e.g. education, employment) will be tested. Moreover, since attitudinal constructs have been shown to explain preferences and evaluation of charging infrastructure [2], the surveyed socio-psychological constructs will be included in the analysis to achieve a deeper understanding and to provide recommendations.

Acknowledgement

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References

[1] Salb, C., Gül, S., Cuntz, C., Monschauer, Y., Weishäupl, J.: Klimaschutz in Zahlen (2018) – Fakten, Trends und Impulse deutscher Klimapolitik, Bundesministerium für Umwelt, Naturschutz und nukleare Sicherheit, (BMU), 2018.

[2] Globisch, J., Plötz, P., Dütschke, E., Wietschel, M.: Consumer evaluation of public charging infrastructure for electric vehicles, Working Paper Sustainability and Innovation, 2018.

[3] Wolff, S., Madlener, R.: Willing to Pay? Spatial Heterogeneity of e-Vehicle Charging Preferences in Germany, Social Science Research Network, , SSRN Scholarly Paper, ID 3676410, 2020, Rochester, NY , doi: 10.2139/ssrn.3676410.

[4] Liao, F., Molin, E., van Wee, B.: Consumer preferences for electric vehicles: a literature review, Transport Reviews, 37 (2017) 3, pp. 252–275, doi: 10.1080/01441647.2016.1230794.

[5] Kistner, R. , Kowald, M.: Ladelust oder Ladefrust – Ausgestaltungsmöglichkeiten bei der Interaktion zwischen Mensch und Ladesäule, Straßenverkehrstechnik, 7 (2019), pp. 478–484

[6] Vogt, M., Fels, K.: Bedarfsorientierte Ladeinfrastruktur aus Kundensich, ATZelektronik, 12 (2017) 3, pp. 56–61, doi: 10.1007/s35658-017-0036-z.

[7] Lee-Gosselin, M.E.H. : Scope and potential of interactive stated response data collection method, Conference on Household Travel Surveys, New Concepts and Research Needs, 1996, pp. 12–15

[8] Rose, J.M., Bliemer, M.C.J.: Stated choice experimental design theory: the who, the what and the why, Handbook of Choice Modelling, Edward Elgar Publishing, 2014, pp. 152–177

[9] Nienhueser, I.A., Qiu, Y. : Economic and environmental impacts of providing renewable energy for electric vehicle charging–A choice experiment study, Applied Energy, 180 (2016), pp. 256–268

[10] ChoiceMetrics, http://www.choice-metrics.com/

[11] Gutjar, M., Kowald, M.: Electro mobility acceptance: The influence of political bonus and malus factors, Symposium of the European Association for Research in Transportation, Lyon, France, 2021.

[12] Adamowicz, W., Louviere, J., Williams, M.: Combining revealed and stated preference methods for valuing environmental amenities, Journal of Environmental Economics and Management, 26 (1994) 3, pp. 271–292

[13] Mariel, P., Hoyos, D., Meyerhoff, J., Czajkowski, M., Dekker, T., Glenk, K., Jacobsen, J.B., Liebe, U., Olsen, S.B., Sagebiel, J., Thiene, M.: Environmental Valuation with Discrete Choice Experiments: Guidance on Design, Implementation and Data Analysis, 2021.

[14] Louviere, J.J., Hensher, D.A., Swait, J.D.: Stated choice methods: analysis and applications, Cambridge university press, 2000.

[15] Hess, S., Adler, T., Polak, J.W.: Modelling airport and airline choice behaviour with the use of stated preference survey data, Transportation Research Part E, Logistics and Transportation Review, 43 (2007) 3, pp. 221–233

[16] Rose, J.M., Bliemer, M.C.J.: Sample size requirements for stated choice experiments, Transportation, 40 (2013) 5, pp. 1021–1041, doi: 10.1007/s11116-013-9451-z.

SIMULATION OF ROAD SPEED-SECTIONING BY ASSESSING THE IMPACT OF TRAFFIC AND ROAD INFRASTRUCTURE

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Abstract

In a context of climate change, lowering road vehicles consumption is a key point to meet CO2 reduction requirements. In addition to car technological advances, eco-driving is part of the solution but the road infrastructure should ensure its development. In a previous study, a gain of 5% in the spent energy was estimated on specific route by slightly moving some speed signs, but under the assumptions that drivers practice eco-driving and the traffic is free-flow. This paper deepens and widens these first results. The base of this research is to provide a simulation model to study the impact of traffic and speed-sectioning on the environment. Inside this model, the impact of different approach speeds to a speed-sectioning is assessed. The simulation is conducted within the Trafficware Synchro environment where parameters according to road infrastructure, vehicle and driver are based on real traffic data. Moving a speed limitation sign can contribute to a reduction of fuel consumption up to 8% depending on driver structure. This new methodology improves the accuracy of our first results and detects adverse effects as the possible emergence of congestion due to the modification of speed sectioning. In perspective this methodology represents a significant argument in road managers strategy. In addition it also represents an orienting point to investigate different action scenarios and a first step to a global optimization policy in managing road infrastructure.

Keywords: road traffic, climate change, environment, eco-friendly road

1 Introduction

Road traffic is inducing the consumption of large amount of fossil fuels, both for goods transportation and traveling for work, leisure, necessity. It is therefore accompanied with an important production of particle pollution and greenhouse gases, responsible of heath and climate change issues. Transportation is responsible for 24 % of direct CO_2 emissions from fuel combustion [1]. Response required to keep warming below 1.5°C and enhance the capacity to adapt to climate risks would require large increases of investments in low-emission infrastructure (IPCC 2018 report [2]). For the purpose of resources preservations and climate change mitigation, a solution on rural roads could be to provide more eco-friendly road infrastructure to the drivers. Traffic phenomena are complex phenomena that have been studied for many years to maintain the transport performance at the level of the -ever growing- mobility demand, and to ensure a high road safety level. The traffic can be observed experimentally for an existing road, to verify the associated mobility and safety performances, but simulations are a tool to evaluated traffic of projected roads or projected exploitation or new equipments of roads. Simulations

can be either macroscopic or microscopic. Traffic simulations for road exploitation strategy or ecodriving-friendly road design are difficult because various types of human behaviour have to be taken into account in addition to varying environmental conditions of roads as lightning and weather.
The approach developed by the “Trafficware Synchro 9”[3] model consists in considering that the phenomena of traffic originates from the actions and interactions of the various participants in the “road system” (road designers, managers, users, etc.), each actor having his own knowledge, goals and strategies. The modelling of the behaviour of the drivers is an essential point to obtain in simulation individual and collective behaviours “realistic”, in that sense certain experimentations are needed to apprehend the large variety of the psychological driving inputs and reactions, within a full traffic system.
Then, for our research of environmental evaluation of roads, we have chose this simulation model in order to take into account different type of drivers, like eco- drivers, conservative or aggressive drivers, types that will be detailed in the following.

2 Simulation

Traffic simulation models are designed to predict system performance based on representations of the temporal and/or spatial interactions between system components (normally vehicles, events, control devices), often characterizing the stochastic nature of traffic flow. The traffic can be studied according to an “integrated” approach in which the identification, modelling and simulation of the actual practices of the drivers are at the heart of the modelled traffic [4,5]. Traffic simulation models may be classified according to the level of detail with which they represent the transportation performance, as well as flow representation, namely:

- in microscopic models, traffic is represented discretely (single vehicles); individual trajectories can be explicitly traced. Disaggregate performanc e measures are calculated based on explicit modelling of driver behaviour,
- in mesoscopic models, traffic is represented discretely (vehicles or group of vehicles); individual trajectories can be explicitly traced as for microscopic models. However, aggregate performance measures are calculated as for macroscopic models,
- in macroscopic models, traffic is represented continuously following the fluid approximation.

2.1 Calibration of a simulation model

The aim of this work is to assess the eco-friendly level that can be associated to infrastructure management options (speed sign positions), while fully taking into account the driver behaviour variety. For that reason simulations with a microscopic traffic model have been worked out, since macroscopic models are not able to describe the driver scale. Trafficware Synchro model has been chosen for the methodology validation, because of its capacity to represent both infrastructure and driver/vehicle particularities. This model has been developed in the United States of America, and it has default features of those roads, so basic calibration was needed to adapt the simulation to European standards. For example, the width of the roads is not the same as in Europe. Another parameters to tune are the length of the vehicles, driver types etc. These calibration possibilities are presented on Table 1 and Table 2.
Vehicles are distinguished by size, acceleration profile, and even occupancy (not relevant here). Drivers have reaction times related to traffic lights (green and yellow reacts).
Yellow reaction times are used here for the reaction time facing a limit speed sign and yellow deceleration rate are used in the same purpose. The diversity in driver behaviour are based on a combination of “yellow decel”, “courtesy decel”, and speed factor, which is the adaptation factor of drivers to the enforcement speed limit (table 2).

Table 1 Configuration of vehicles types

Vehicles Types	1	2	3	4	5	6	7
Vehicle Name	Car1	Car2	Truck SU	SemiTrk1	SemiTrk2	Truck DB	Bus
Vehicle Occurence (%)	0.64	0.16	0.60	0.10	0.05	0.05	0.20
Acceleration	File	File	File	File	File	File	File
Vehicle Length (m)	4.88	4.26	10.67	16.16	16.16	19.50	12.20
Vehicle Width (m)	1.80	1.80	2.40	2.40	2.40	2.40	2.40
Vehicle Fleet	Car	Car	Trk	Trk	Trk	Trk	Bus
Occupancy (# people)	1.3	1.3	1.2	1.2	1.2	1.2	20.0
Graphics Shape	Car	Car	Truck	SemiTrk	SemiTrk	DBTruck	Bus
Table Index (1 to 7)	1	2	3	4	5	6	7

Table 2 Configuration of driver types

Driver Types	1	2	3	4	5	6	7
Yellow Decel (m/s^2)	3.60	3.60	3.60	3.60	3.60	3.30	3.00
Speed Factor (%)	0.85	0.88	0.92	0.95	0.98	1.02	1.05
Courtesy Decel (m/s^2)	3.00	2.70	2.40	2.10	1.80	1.50	1.20
Yellow React (s)	0.7	0.9	1.0	1.0	1.2	1.3	1.3
Green React (s)	0.8	0.7	0.6	0.6	0.5	0.5	0.5
Headway @ 0 km/h (s)	0.65	0.63	0.60	0.58	0.55	0.45	0.42
Headway @ 30 km/h (s)	1.80	1.70	1.60	1.50	1.40	1.20	1.10
Headway @ 80 km/h (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40
Headway @ 130 km/h (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40

As schematized on Fig. 1, the simulation worked out under Trafficware Synchro relies on the infrastructure decomposed on nodes, lanes, links between current/destination lanes, vehicles referenced by id numbers and vehicle types, drivers of different behaviors assigned to these vehicles.

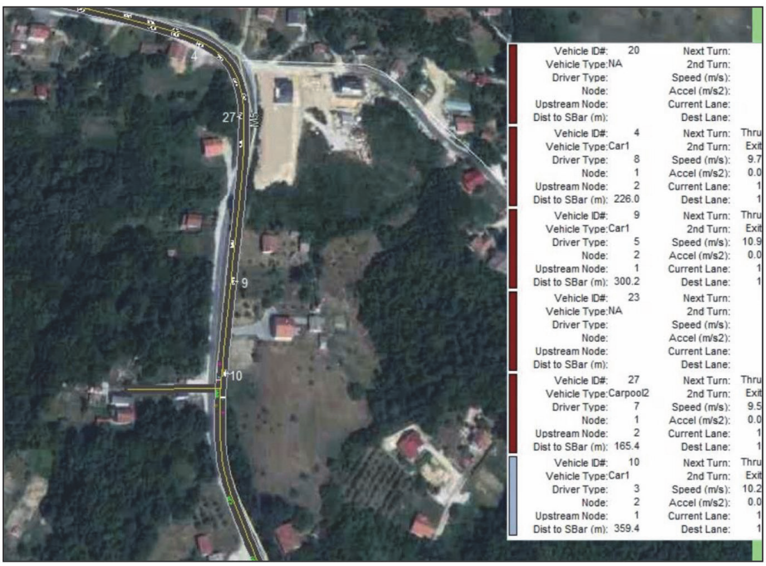

Figure 1 Simulation framework on Trafficware Synchro 9 by identifing each vehicle parameters

3D simulations have been worked out and executed in the Trafficware Synchro simulation framework, with various sets of vehicles/drivers/road management. The Fig. 2. gives an example of simulated traffic.

Figure 2 3D simulation in Trafficware Synchro 9

3 Results

Simulations that have been realized are targeting three baseline scenarios: Initial speed sectioning, Optimized speed sectioning and Pre-signalisation speed sectioning), for the evaluation of the road exploitation and its eco-friendly associated level. The better positioning of a speed sign could then be evaluated facing to an initial bad adequacy of the speed-sectioning to the longitudinal road profile. Pre-signalisation is another enhancement mean, although based on driver volunteer choice.
To emphasis the variety of driver behaviour, and the traffic speed spread around fixed limits, various approaching speed have been used for simulations, even at speeds higher than allowed speed.
All of the three scenarios and approach speed hypothesis gave us results and conclusions on how driver type can influence fuel consumption depending on driver's behaviour.
The length of the simulation area is of 1189 m. Traffic flows and composition are built upon road data recorded from JP Road of Federation of BiH.
Table 3. represents obtained data by the simulation software Trafficware Synchro 9 by defining fuel consumption and data on kpl (kilometers per 1 litre of fuel) in dependence on driver structure. Each column corresponds to one scenario. Each line corresponds to a combination of one type of driver and one approaching speed (70 km/h, 80 km/h, 90 km/h, 100 km/h).
We have classified the types of drivers into 4 types depending on their acceleration and deceleration: type 1 (aggressive, color red in the table 3) where the deceleration rate is 1.4-3 m/s^2, type 2 (defensive drivers, no color in thetable
3) where the deceleration rate is 0.7-1 m/s^2, type 3 (eco-drivers, color green) where the deceleration rate is 0.3-0.4 m/s^2, type 4 (combination of all types (40 % of aggressive, 30 % defensive, 30 % eco-drivers, color blue) in accordance to their visibility distance. In the simulation we have used the speed factor (SF in table 3) for each type of driver.
All realized simulations have "real time" durations of 15 minutes. Each simulation is repeated 10 times in order to extract simulation results with a satisfying repeatability.
Simulation results of fuel consumption are given in Fig.3, depending on driver behaviour and speed-sectioning hypothesis, varying with different approaching speeds and speed limitation sign placement.
Fig. 3a, 3b, 3c and 3d are respectively dedicated to the cases of aggressive driver, defensive driver, eco-driver, and a combination of previous drivers.

These results in Fig. 3 show the important impact of speed sign management on vehicle fuel consumption. As expected, the initial speed sectioning (purple bars) is leading to the higher fuel consumption, for any approaching speed or driver behaviour combination. The optimized speed-sectioning (green bars) leads always to better fuel economy than the intermediate solution of pre-signalisation speed sectioning (additional speed limitation sign of 60 km/h to amortize a speed variation), except for the very high approaching speed of 100 km/h and (a), (c), (d) drivers types. Indeed, in that specific case of very high speed, the pre- signalisation is encouraging drivers to decelerated very early, at the exception of the defensive driver which has the primary goal to maintain his speed, and don't benefit of pre-signalisation.
Eco-drivers are the driver category which takes the highest advantage in fuel economy of the optimized and pre-signalized scenarios. In the case of an approach speed of 90km/h, its initial fuel economy of 12 kpl (km/l) is raise for the both optimized and pre-signalized cases to 14 kpl (km/l). This is an improvement of 16,7 %. The methodology was targeting eco-drivers in order to offer them an infrastructure more eco-friendly, but results show that other types of drivers are benefiting of the speed management.

Table 3 Configuration of driver types and fuel consumption data

Speed/Type	Optimized (L)/(kpL)	Initial (L)/(kpL)	Presignalisation (L)/(kpL)
70-comb (SF-0.8,1,1.2)	22.5/13	23.4/12.5	22.5/13
80-comb	21.9/13.3	23.7/12.4	22/13.3
90-comb	21.7/13.5	24.4/12	21.9/13.4
100-comb	21.9/13.3	24.7/11.8	21.7/13.5
70-agg (SF-1.2)	21.7/13.5	24.7/11.9	21.7/13.5
80-agg	21.3/13.7	25.6/11.4	21.2/13.8
90-agg	21.3/13.7	25.8/11.3	21.5/13.6
100-agg	22.7/12.9	26.0/11.2	21.8/13.4
70-def (SF-0.8)	23/12.7	23.5/12.5	23/12.7
80-def	21.2/13.8	22.1/13.2	21.6/13.5
90-def	21.2/13.8	22.8/12.8	21.7/13.5
100-def	21.1/13.9	23.8/12.3	21.6/13.6
70-eco (SF-1)	20.5/14.3	21.5/13.6	20.6/14.2
80-eco	20.7/14.1	23.1/12.7	20.7/14.1
90-eco	20.4/14.3	24.3/12	20.6/14.3
100-eco	20.5/14.3	24.4/12.1	20.4/14.4

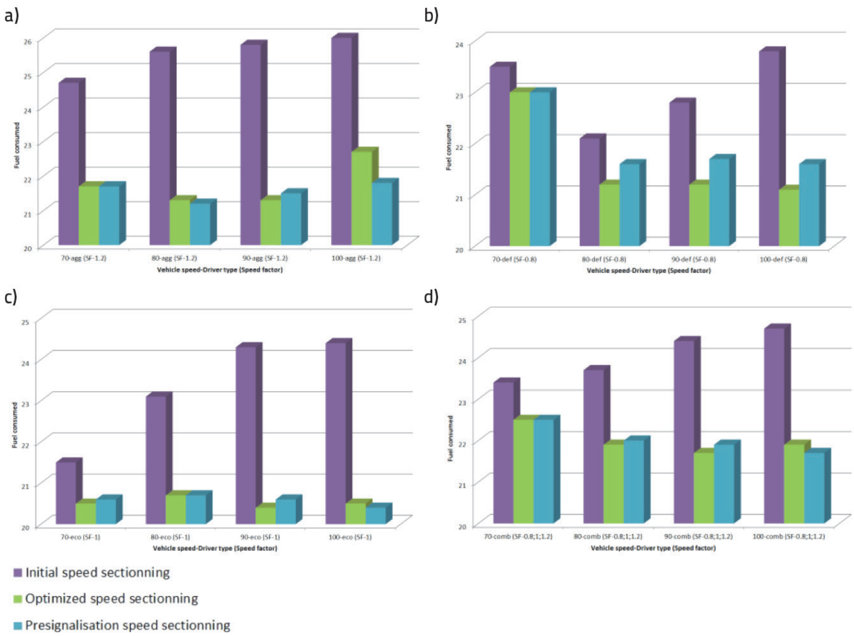

Figure 3 Simulation results of fuel consumption depending on driver behaviour, speed-sectioning with different approaching speeds: a) aggressive driver, b) defensive driver, c) eco-driver, d) combined (40 %agg, 30 %def, 30 %eco)

4 Limits and hypotheses

- The simulation model has potential to assess the traffic locally on a defined route, however it has also some disadvantages. Driving behaviour is modelled by continuous acceleration phases (speed adaptation factor), without the complexity of human driving sequences, often discontinuous and affected by factors as fatigue, stress, visibility altered by weather conditions.
- Nevertheless simulations show that fuel consumption gains are significant (11.5 %, from 23.1l to 20.7l) for eco-drivers while being inexpensive to implement for the infrastructure manager, because it is enough just to displace a speed limiting sign. A risk for the manager is that moving a speed sign may cause congestion. This can increase greenhouse gas emissions, which is contrary to initial methodology goal. Traffic simulations over full networks are then recommanded for assessing this risk in particular cases.
- Simulation results may underestimate actual fuel savings. Indeed simulation's route length can not be shorter than one kilometer in simulation framework of Trafficware Synchro as for physical restrictions, albeit in real world there can be two or three points of speed-sectioning to be optimized in such a one kilometer length. So, in dense areas, optimizing speed-sectioning could lead to somewhat twice the previously estimated fuel savings.
- Lastly, based on the research of Wilco [6] where a model of number of replications is explained and needed in a micro-traffic simulation to obtain reliable results, it is checked that the number of replications was sufficient: obtained results are very similar every time, conducted for each type of driver, because of the environment of Trafficware Synchro model already does certain number of MoE (measure of effectiveness) for each simulation as we calibrated, and validated the speed factor of each type of driver, the starting point of each vehicle and the measuring period.

5 Conclusion

Using this simulation model we have been able to see a eco-driving approach, and have realistic data. The simulated traffic conditions that are seen in the simulation model are similar as the one we have encountered during experimentation phases [7, 8]. The simulation was important to have a global view of eco-potentiality of the infrastructure in correlation to emission and energy parameters. Gathered data helped evaluate different situations in dependence to drivers structure.

References

[1] IEA, Tracking Transport, Paris https://www.iea.org/reports/tracking- transport-2019, 2019.

[2] de Coninck, H., et al.: Strengthening and Implementing the Global Response, Global warming of 1.5°C, IPCC Special Report on the impacts of global warming of 1.5°C, ed. Masson-Delmotte, V., et al., Press, chap. 4, 2018, pp. 313–443

[3] Synchro Studio 9, Warrants 9, Trafficware, 2017.

[4] Golubev, K., et al.: A framework for a multi-agent traffic simulation using combined behavioural models, Procedia Computer Science, 136 (2018), pp. 443-452

[5] Stolfi, D.H., Alba, E.: Generating realistic urban traffic flows with evolutionary techniques. Engineering Applications of Artificial Intelligence, 75 (2018), pp. 36-47.

[6] Wilco, B.: A note on the number of replication runs in s tochastic traffic simulation models, Technical report, Technical Report CTR2004:01, Stockholm, Sweden, Centre for Traffic Simulation Research (CTR), Royal Institute of Technology, 2004, pp. 1–6

[7] Deljanin, E., et al.: Ecodriving potential of roads according to their speed sectioning: Bosnia and France cases, English, Cetra2018 - 5th International Conference on Road and Rail Infrastructure, 2018, pp. 987–992

[8] Coiret, A., Vandanjon, P.-O., Cuervo-Tuero, A.: Ecodriving potentiality assessment of road infrastructures according to the adequacy between infrastructure slopes and speeds limits, 4th International Conference on Road and Rail Infrastructure (CETRA 2016), Šibenik, Croatia, 23-25 May 2016, pp 589-595

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DISRUPTIVE CHANGES IN THE TRANSPORT SYSTEM - FROM CAR-REGIME TO SUSTAINABILITY

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Abstract

Achieving the climate goals depends to a large extent on the reorganization of the transport sector at all levels. The initiation of a paradigm shift in transport policies must be understood and take place primarily as a result of a change in structures. Besides the transport infrastructure, this also includes the financial and legal system. It is necessary to include the procedures and processes of transport policy decision-making in a comprehensive transition management. The implementation of specific national objectives in the respective administrative levels across the federal states to the municipalities would be a first necessary step. However, initiating necessary radical changes in the transport sector will require disruptive changes to the established structures. The paper discusses such changes in the transport system by introducing theories of sustainability transition and political economy. The role, dependencies and influence of actors in planning processes are prototypically analysed, drawing on the situation in Austria. We further examine how structural barriers suppress or delay measures supporting environmental sustainability. The limits of bottom-up and classic top-down processes are shown and their effectiveness as a contribution to the mobility transition is critically questioned.

Keywords: systemic change, transformation, policy, sustainable infrastructure, multi-level perspective

1 Introduction

“Sustainability and transport” is an increasingly obvious contradiction when looking at current developments and transport policy decisions worldwide. Particularly regarding the climate crisis, change is overdue. Global CO_2 emissions in the transport sector have risen by 250 % between1970 and 2010, where road transport accounts for the majority [1]. Emissions are still rising in many European countries. In Austria, for example, transport emissions have continuously risen again since 2014. Between 1990 and 2019 they increased by 74.4 %, cancelling out all reductions in other sectors [2].
More and more analyses and simulations on resource use and planetary boundaries (climate change, biodiversity, availability of resources) conclude that sustainability goals can only be reached if there is radical change in the transport sector. Purely technically oriented solutions and trends that continue to rely on motorized individual transport, such as e-mobility, car sharing or autonomous driving, can only make a moderate contribution towards achieving the climate goals.
de Blas, et al. [3], for example, analysed worldwide scenarios of a shift to electric mobility. They conclude that it is not possible to decarbonize the transport system through electrification of vehicles under the current growth paradigm. Their simulations show only one sce-

nario where climate targets are met and it includes not only a shift to lighter electric vehicles and non-motorized transport modes but also a strong decline in transportation demand. Millward-Hopkins, et al. [4] showed in a global scenario that it is possible to provide decent living for all people with minimum energy, which makes it possible to live within planetary boundaries. Concerning transportation, this entails an "(ambitious) combination of non-motorised transport, public transport, and limited private vehicle use and air travel". Especially for the Global North, this means that passenger km travelled with private motorized vehicles have to decrease significantly.

For Austria, Heinfellner, et al. [5] assessed how national climate targets in the transport sector can be met. Their simulations show that the goals cannot be achieved solely with technological changes. In the study, they also identified the most effective measures for decarbonisation. They include higher fuel taxes, lower speed limits, cordon charges, better quality infrastructure for walking and cycling, spatial planning measures such as more compact settlement structures and expansion of public transport services. Even though these measures are known, they have not been implemented and, following the political discourse, it does not seem likely that they will be implemented anytime soon. Banister and Hickman [6] call this observation an "implementation gap" and Gössling and Cohen [7] forecast that EU climate policy will fail due to "transport taboos" – "barriers to the design, acceptance and implementation of such transport policies that remain unaddressed as they constitute political risk".

Measures that are effective in reducing CO_2-emissions require or produce changes in the legal, financial and built structures of the current system. Their implementation is the result of planning and political decision-making processes. It is therefore necessary to include the analysis of such processes in a comprehensive transition management. This is where engineering research focused on transportation infrastructure and modelling of environmental impact has to face its limitations, acknowledge that the challenges cannot be met with existing tools and multidisciplinary research is needed to find out how it is possible to implement the known measures in a manner that makes them socially and politically acceptable.

In this paper, we introduce theories of sustainability transitions to the field of transport infrastructure research. We analyse why a multitude of known problem diagnoses, suggested strategies and calls to action have shown little impact so far and why a transformation in the sense of a paradigm shift did not happen yet in the transport sector. We show the roles, dependencies and influence of actors in planning processes in a prototypical way, drawing on the situation in Austria. We further examine how structural barriers suppress or delay the diffusion of measures supporting environmental sustainability and show where possible points to intervene lie.

The rest of the article is structured as follows. In Section 2, we describe theories of systems, regimes and sustainability transition and link them to transport planning and policy. Section 3, discusses the role of structures and agency in planning processes, referring to the Austrian transport system in particular. In Section 4, we draw our conclusions.

2 Theory of systems, regimes and sustainability transition

2.1 Human needs and systems of provision

Mattioli [8] proposed a framework connecting human needs theory and systems of provision, which was extended by Brand-Correa, et al. [9] to show possible places to intervene in the currently non-sustainable transport system. Basis for their assessment is the order of need satisfiers, with the private car as an example. Humans have basic needs that they satisfy with need satisfiers. While car use is not a need itself, it serves as a need satisfier of higher order. We can look at the basic need for subsistence as an example. People need to earn money

to make a living. For this, they have to get from their home to their workplace. First order satisfiers are socio-technical systems of provision such as infrastructure (e.g. a road that connects home to workplace). Second order are activities, third services and fourth specific products (such as the car).
While first order need satisfiers are the most effective places to intervene, they are also the hardest to change. This can be illustrated with a cog-metaphor (see [9]) or as leverage points (see Figure 1). A first order intervention to move away from a car-oriented system would include "a shift in the provision of non-automobile infrastructure, improved and integrated public transport systems and changes to urban planning and design, including a relocation of workplaces to more easily accessible areas" [9] – and therefore a change in the system structure. While a fourth order intervention could look like a change to cars running on biofuel (change of only one parameter), which presents a much easier task but is not nearly as effective in terms of climate mitigation.
However, socio-technical provisioning systems not only entail physically built infrastructure but also institutions and economic and political logics - ultimately mindsets and paradigms. Addressing these might seem like an unsurmountable challenge and actors in transport might take this as an excuse for inaction. While it is out of scope of this study to analyse fundamental economic and political logics that determine conditions for transport in detail, it is still necessary to address all aspects of the socio-technical provisioning systems. In this work, we focus on the infrastructural part as well as the legal and administrative bases that enable or inhibit decisions in infrastructure planning. It is true that these aspects alone cannot trigger disruptive change in economics and politics, but on the other hand, radical change in these fields of practice is not possible without changes in the transport system and its infrastructure.

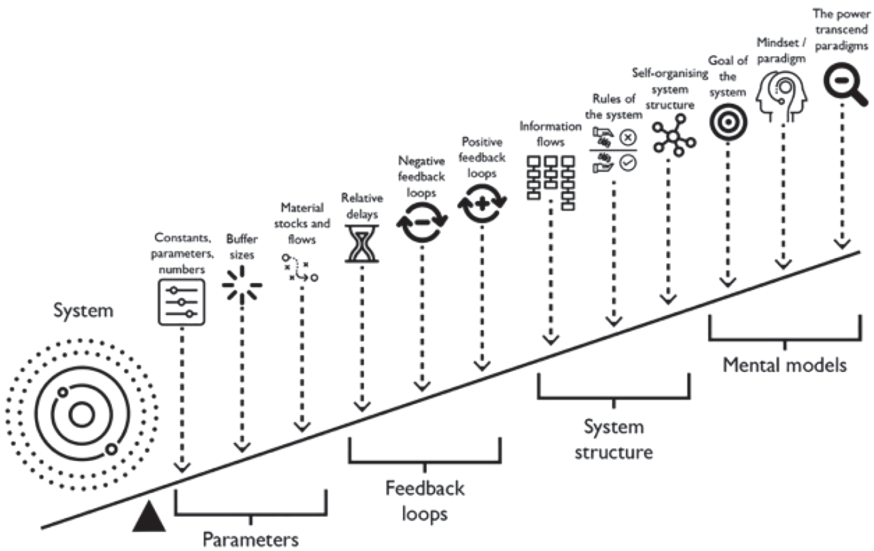

Figure 1 Places to intervene in a system with increasing leverage to the right, figure from [10] based on [11]

2.2 Theories of sustainability transition: Multi-level perspective

Markard, et al. [12] describe the emergence of sustainability transition research and its importance. In many sectors such as energy, agriculture and the transport sector, ecological, social and economic problems are imminent. In the transport sector, these problems are apparent in the form of local air pollution, depletion of fossil fuels, CO_2-emissions and traffic accidents. Due to past developments (path dependencies) and lock-in effects, well-established systems only change incrementally and not radically. However, such incremental changes are not enough to rise to the sustainability challenges in due time. This is why transition research deals with the question how radical change of these well-established systems can be supported and steered.
The multi-level perspective (MLP) developed by Geels and Schot [13] is one of the theories that describe transitions of socio-technical systems. It has emerged as being practical to show barriers in the transition of transport systems.

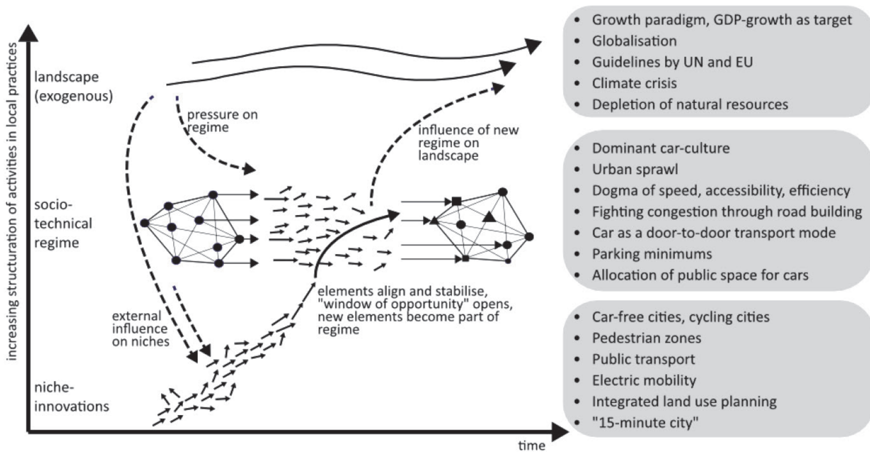

Figure 2 Multi-level perspective, adapted illustration from [14], based on [13]

There are three levels in the MLP: landscape, regime and niches (see Figure 2). The central regime includes the dynamically stable, established and hegemonial practices, discourses, institutions and artefacts [15]. Rip and Kemp [16] define technological regimes as the "rule-set or grammar embedded in a complex of engineering practices, production process technologies, product characteristics, skills and procedures, ways of handling relevant artefacts and persons, ways of defining problems". Within the regime, there are three different dimensions of entities: (1) tangible technologies (e.g. road infrastructure, cars), (2) actors and social groups and (3) rules (formal and informal) such as laws, planning guidelines, etc. [17]. Inside the regime, institutional structures connect the artefacts, rules and actors. A transition is defined as a shift from one regime to a different regime. Niches and landscape are defined in relation to the regime [18].
The landscape represents exogenous factors that affect the regime but are not directly part of the regime. The distinction between landscape and regime is discussed in literature, but it is not clearly defined [19, 15]. The distinction depends on the system that is being analysed and the view of the analyst. In the classic approach by Geels and Schot [13], socio-technical systems are in focus and economic conditions are seen as exogenous. Vandeventer, et al. [20] extended the theory and showed that it is also useful to describe changes in socio-economic systems in which economic parameters such as the capitalistic growth paradigm are part of the regime. According to the MLP theory, the landscape can enact pressure on the regime,

which can lead to breaking connections within and the order of the regime. This destabilized state marks a "window of opportunity" during which it is possible to create a new regime order that incorporates innovations or niche developments.
Niches are defined as technologies or practices that deviate substantially from the existing regime. They can support the regime or be seen as opponents. On the regime level, there are multiple individuals and groups that act independently and in an uncoordinated way. They can form networks and align their actions to create a dominant stable form, which makes it more likely that they make the leap to the regime in times of an opportunity.
The MLP can be used as a tool to define regime components and discern landscape and niche components as well as to describe path dependencies. MLP has been used multiple times for examining aspects of sustainability transition in transportation. Vogel [15] used it for analysing sustainable urban mobility, Zijlstra and Avelino [21] for a socio-spatial analysis of mobility and Sheller [22] for describing the cultural dimension of mobility. In this work, we apply a transport infrastructure and policy perspective on a national scale, with examples drawn upon the situation in Austria.

2.3 The "car-regime" and barriers for change

In most countries in the Global North, the established transport regime can be described as a "car-regime" [21, 23]. The private car with combustion engine is favoured as a transport mode and (legal, financial and built) structures are oriented towards its use. This inhibits the implementation of measures that are effective in reducing CO_2-emissions and therefore in mitigating climate change. The current regime puts conditions in place that still enable the planning, financing and building of infrastructures that have been demonstrated to lead to drastic and continuing increase of CO_2-emissions. Mattioli, et al. [24] describe the current system of car dependency based on six systems of provision: the automotive industry, car infrastructure, car-dependent land use patterns (urban sprawl), (undermining of) public transport and cultures of car consumption. They are interconnected and work in positive feedback loops as a self-reinforcing system.
The phenomenon of this regime not having changed in the direction of sustainability (despite such efforts in the past), has been described in literature by several different terms. In the field of System Dynamics, it is described as "policy resistance" [25, 26]. Driscoll [27] writes about "carbon lock-in", Mattioli, et al. [24] about "political economy of car dependence", Blühdorn, et al. [28] about "sustainable non-sustainability" and Marletto [23], Zijlstra and Avelino [21] use the term "car-regime" to describe a system of interrelated and self-reinforcing entities that make it impossible to change by standalone policies or reformative approaches.
Ultimately, they all describe similar problems that can be illustrated using the MLP. Over time we have created a complex system that is made up of self-reinforcing elements that generate a growing dependency on cars ("political economy of car dependence" [24]). Individual measures do not show the desired effect but are weakened by the system's response ("policy resistance", [25, 26]). Often we have to deal with short-term false solutions that provoke a long-term effect that reinforces the problem. One example for this is the reaction to congestion with increased capacity such as building more roads, adding lanes or implementing ITS (intelligent transport systems). In the beginning there might be some level of traffic relief but in the longer term more traffic is induced and the traffic load is increased compared to the initial situation [29].
Decisions in transport policy are mostly based on theories and principles that have not been critically challenged. They are the basis for defending the status quo and maintaining a hierarchy of values that is in some cases even formally defined in legal regulations and industry standards. Dogmas such as "accessibility", "design speed" or the mostly monomodally

discussed "elimination of capacity bottlenecks" are defended at all cost for stabilising the regime. It is pretended that there exists technical objectivity, even though the underlying assumptions are not explicitly voiced and are often not public. Different assumptions that challenge the status quo are referred to as unrealistic [30].
In this way, it is made impossible to create an ecologically and socially sustainable transport system beyond individual niche solutions. Some scholars speak of an "implementation gap" [6]. Research constantly creates new findings that show how the system should change to enable sustainable mobility. But this knowledge is not adopted by the regime, at most, it is tested in niches. Necessary changes in the system structure however are "transport taboos" [7] – measures that are unthinkable for the majority of actors. Therefore, they are not addressed by the established regime and are being disqualified for being too radical or politically not feasible.

3 Structures and agency in planning processes

(Transport) planning can be described as the mental anticipation of actions that seem necessary to reach a goal. This is a process that results in an abstract (simplified) illustration or model of the expected reality [31]. The basis for a change of behaviour (in transport systems) is the change of structures. Following the structuration theory of Giddens [32], on the one hand, structures determine behaviour and on the other hand, structures are the outcome of social actions. Structures are to be seen as all elements that determine or influence behaviour [33]. They can be physically built elements as well as legal and financial regulations, information, social or economic conditions. Particularly in the built environment, they are the result of planning processes. Kloss [34] defines the stakeholders in a planning process on the basis of transport planning in the city of Salzburg as follows:
- Planning authority and administration (city, province, federal government)
- Representatives of public authorities
- External planners (companies and research institutions)
- Citizens and advocacy groups
- Media representatives

Politicians have a central role. They are ultimately responsible for implementing measures that decrease the target/actual difference. Reality is made up of countless interrelated feedback loops and systems. The task of planners is to identify significant feedback loops and to choose system variables and indicators to describe the system in question. Here, it is essential to note that perceived and actual reality often differ and therefore decisions that follow "wrong" goals are made [34]. Long time delays between action and impact reduce the willingness to initiate transformative processes.
Since individuals are embedded in the structures, there is only a limited degree of freedom for them to act. Actions can reproduce and intensify the given structure or work in changing the structure in a different direction. The decisions of individuals are not only influenced by the system structure itself but also by their personal background. Their perceived reality is influenced by individual and biased interpretation, values, education, legal preconditions, technical codes and standards and chosen indicators [33]. This defines what exactly is seen as a problem and in practice this often leads to not addressing actual human needs but perceived needs such as the need for fast travel with a private car. The persistent pursuit of "wrong" goals (e.g. increasing speed or increasing capacity) leads to increasing dependence and lock-in. This is the opposite of what Heinz von Foerster described as his Ethical Imperative "Act always so as to increase the number of choices." [35].

The step from the perception of a problem to the solution of the problem is not trivial. Even when a problem is recognized, it is not always easy to identify the cause, since problems often appear as symptoms or syndromes and are in many cases treated on that level. We cannot make absolute statements about reality in its entirety because we only ever perceive a certain part of the unknown reality [33]. This perceived reality is influenced by the personal background in education, system knowledge and expertise [36]. Fasching [37] describes this as an objective illusion, based on an intersubjective reality, caused by the scientific methods.

4 Conclusion

The current transport system favours the private car over other means of transport; its structures are oriented towards private cars. This “car-regime” inhibits the implementation of effective climate mitigation measures and ensures that even today, decision makers plan and finance infrastructures that lead to a continuous increase in CO_2-emissions. Niche innovations are only implemented in a regime if they do not change the system behaviour fundamentally. Actors in the car-regime and their values are highly anthropocentric and do not evaluate ecological criteria in their actual relevance.

In the political discourse, there is a lack of realistic assessment of measure intensity to come even close to achieving the climate targets in the transport sector. The measures that have already been quantified to effectively reduce CO_2-emissions (such as in [5]) should be implemented quickly. In addition, further measures focused on process structures must be realized. This could include linking fiscal transfers to CO_2-saving goals, reviewing the spatial and settlement policies and paying fines for climate-damaging infrastructures and surface sealing. Such measures would be systemically relevant and exert a leverage effect since they lead to a change in behaviour of the relevant institutions. However, to initiate the necessary radical changes in the transport sector, the established structures have to be disrupted. This also requires external impulses that question the existing “system”, for example by bottom-up initiatives.

A new paradigm sees an abandonment of the current car-regime in favour of planning oriented towards sustainable transport modes (walking, cycling and public transport). The spatial preconditions, the availability and the attractiveness of different transport modes determine the mobility behaviour of people. The built structures as well as legal, financial and organizational structures have to be changed in order to achieve effective changes towards sustainable transport. This is not only about shifting trips from cars to public transport and making cars electric but a fundamental transformation of the transport system under consideration of socio-economic, cultural and spatial dynamics. To achieve this, it is necessary that such a paradigm shift arrives in people’s minds, especially in those of politicians, planners, administrators and researchers, since they create or influence conditions, decision-making tools and have to define and implement measures.

References

[1] Sims, R., et al.: Transport, in Climate Change 2014: Mitigation of Climate Change, Contribution of Working Group III to the Fifth Assessment Report of the Intergovernmental Panel on climate Change-Cambridge, United Kingdom and New York, NY, USA, Cambridge University Press, 2014.

[2] Umweltbundesamt, Treibhausgas-Bilanz 2017., Daten, Trends & Ausblick, https://www.umweltbundesamt.at/fileadmin/site/aktuelles/2019/treibhausgas-bilanz_2017.pdf, 2019.

[3] de Blas, I., Mediavilla, M., Capellán-Pérez, I., Duce, C.: The limits of transport decarbonization under the current growth paradigm, Energy Strategy Reviews, 32 (2020), doi: 10.1016/j.esr.2020.100543

[4] Millward-Hopkins, J., Steinberger, J.K., Rao, N.D., Oswald, Y.: Providing decent living with minimum energy: A global scenario, Global Environmental Change, 65 (2020), doi: 10.1016/j.gloenvcha.2020.102168

[5] Heinfellner, H. , et al.: Sachstandsbericht Mobilität und mögliche Zielpfade zur Erreichung der Klimaziele 2050 mit dem Zwischenziel 2030, Wien, 2019.

[6] Banister, D., Hickman, R.:Transport futures: Thinking the unthinkable, Transport Policy, 29 (2013), pp. 283-293, doi: 10.1016/j.tranpol.2012.07.005

[7] Gössling, S., Cohen, S.: Why sustainable transport policies will fail: EU climate policy in the light of transport taboos, Journal of Transport Geography, 39 (2014), pp. 197-207, doi: 10.1016/j.jtrangeo.2014.07.010

[8] Mattioli, G.: Transport needs in a climate-constrained world, A novel framework to reconcile social and environmental sustainability in transport, Energy Research & Social Science, 18 (2016), pp. 118-128, doi: 10.1016/j.erss.2016.03.025

[9] Brand-Correa, L.I., Mattioli, G., Lamb, W.F., Steinberger, J.K.: Understanding (and tackling) need satisfier escalation, Sustainability: Science, Practice and Policy, 16 (2020) 1, pp. 309-325, doi: 10.1080/15487733.2020.1816026

[10] Angheloiu, C., Tennant, M.: Urban futures: Systemic or system changing interventions? A literature review using Meadows' leverage points as analytical framework, Cities, 104 (2020), doi: 10.1016/j.cities.2020.102808

[11] Meadows, D.H.: Leverage Points - Places to Intervene in a System, The Sustainability Institute, 1999.

[12] Markard, J., Raven, R., Truffer, B.: Sustainability transitions: An emerging field of research and its prospects, Research Policy, 1 (2012) 6, pp. 955-967, doi: 10.1016/j.respol.2012.02.013

[13] Geels, F.W., Schot, J.: Typology of sociotechnical transition pathways, Research Policy, 36 (2007) 3, pp. 399-417, doi: 10.1016/j.respol.2007.01.003

[14] Haselsteiner, E., et al.: Change! Mobilitätswende in den Köpfen –Transitionsprozesse nutzerorientiert managen lernen!, Programm "Mobilität der Zukunft" , BMK, Vienna, 2020.

[15] Vogel, N.: Transition in the making: A critical dispute on urban transition processes toward sustainable mobility, Department of Development and Planning, Aalborg University, 2015.

[16] Rip, A., Kemp, R.: Technological change, Human choice and climate change, 2 (1998) 2, pp. 327-399

[17] Geels, F.W.: Transformations of Large Technical Systems, A Multilevel Analysis of the Dutch Highway System (1950-2000), Science, Technology, & Human Values, 32 (2007) 2, pp. 123-149, doi: 10.1177/0162243906293883

[18] Geels, F.W.: The multi-level perspective on sustainability transitions: Responses to seven criticisms, Environmental Innovation and Societal Transitions, (2011) 1, pp. 24-40, doi: 10.1016/j.eist.2011.02.002

[19] Koretskaya, O., Scholl, C.: Towards a Framework for Understanding Discursive Regime Destabilisation: A Case Study of a Social Movement Organisation "Economy for the Common Good", Sustainability, 11 (2019) 16, doi: 10.3390/su11164385

[20] Vandeventer, J.S., Cattaneo, C., Zografos, C.: A Degrowth Transition: Pathways for the Degrowth Niche to Replace the Capitalist-Growth Regime, Ecological Economics, 156 (2019), pp. 272-286, doi: 10.1016/j.ecolecon.2018.10.002

[21] Zijlstra, T., Avelino, F.: Socio-spatial perspective on the car regime, in Automobility in Transition? A Socio-Technical Analysis of Sustainable Transport, New York, London, Taylor & Francis, 2012.

[22] Sheller, M.: The Emergence of New Cultures of Mobility: Stability, Openings and Prospects, in Automobility in Transition? A Socio-Technical Analysis of Sustainable Transport, New York, London, Taylor & Francis, 2012.

[23] Marletto, G.: Structure, agency and change in the car regime: A review of the literature, European Transportation, 47 (2011), pp. 71-88

[24] Mattioli, G., Roberts, C., Steinberger, J.K., Brown, A.: The political economy of car dependence: A systems of provision approach, Energy Research & Social Science, 66 (2020), doi: 10.1016/j.erss.2020.101486

[25] Meadows, D.H.: Whole earth models and systems, CoEvolution Quarterly, 1982, pp. 68-80

[26] Sterman, J.: Business dynamics, Irwin/McGraw-Hill, 2000.

[27] Driscoll, P.A.: Breaking Carbon Lock-In: Path Dependencies in Large-Scale Transportation Infrastructure Projects, Planning Practice & Research, 29 (2014) 3, pp. 317-330, doi: 10.1080/02697459.2014.929847

[28] Blühdorn, I., Butzlaff, F., Deflorian, M., Hausknost, D., Mock, M.: Nachhaltige Nicht-Nachhaltigkeit: Warum die ökologische Transformation der Gesellschaft nicht stattfindet, transcript Verlag, 2020.

[29] Frey, H., Leth, U., Mayerthaler, A., Brezina, T.: Predicted congestions never occur, On the gap between transport modelling and human behaviour, Transport Problems, 6 (2011) 1, pp. 73-85

[30] Macoun, T.: Bewertungsverfahren in komplexem Umfeld, Habilitationsschrift, Institut für Verkehrsplanung und Verkehrstechnik, TU Wien, 2000.

[31] Frey, H.: Wer plant die Planung? – Widersprüche in Theorie und Praxis, REAL CORP 2014–Plan it smart! Clever Solutions for Smart Cities, Proceedings of 19th International Conference on Urban Planning, Regional Development and Information Society, January 2014.

[32] Giddens, A.: The Constitution of Society: Outline of the Theory of Structuration, Cambridge, Polity Press, 1984.

[33] Knoflacher, H.: Grundlagen der Verkehrs- und Siedlungsplanung, Verkehrsplanung, 2007.

[34] Kloss, H.P.: Wirkungsanalysen von Planungsprinzipien in der Verkehrsplanung gezeigt am Beispiel der Stadt Salzburg, Dissertation, Institut für Verkehrsplanung und Verkehrstechnik, TU Wien, Wien, 2009.

[35] Von Foerster, H.: Understanding understanding: Essays on cybernetics and cognition, Springer Science & Business Media, 2002.

[36] Riedl, R.: Die Spaltung des Weltbildes. Biologische Grundlagen des Erklärens und Verstehens, Berlin/Hamburg: Paul Parey, 1985.

[37] Fasching, G.: Objektive Illusionen: Ein Essay über das Wesen der naturwissenschaftlichen Wirklichkeit, Peter Lang Edition, 2005.

OPTIMIZATION OF ROAD SPEED-SECTIONING BY ASSESSING THE IMPACT OF A ROAD SPEED LIMITATION SIGN

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Abstract

Energy consumed by road vehicles has a high impact on climate changes; indeed this energy use accounts for 23 % of total energy-related Green House Gases (GHG) emissions of 2014 global GHG emissions. GHG emissions are growing constantly year after year, in spite of global objectives (COP) and researches on vehicle efficiency and modal shift. The contribution of the infrastructure to lower this energy is less studied, since it is often seen as immuable or too costly. This paper aims to demonstrate that simple and low-cost solutions exist for that purpose. Particularly a methodology has been developed, based on an optimization of the speed layout over an itinerary in order to improve the eco- driving potential of a given road infrastructure. The key point of this work is that inconsistency often exists between vehicle dynamics, road longitudinal profile and changes in regulation speeds. These changes in speed are defining the speed- sectioning of a route, and an optimization of this speed-sectioning can be easily carried out while displacing or modifying speed signs. The objective of this study is to build an optimized speed sectioning which minimizes the fuel consumption for realistic traffic and various driver behaviors, while maintaining the required safety levels. A progressive optimization loop has been worked out with a Python script including an embedded microscopic road traffic simulator. As a result, an optimized speed-sectioning is leading to a gain of 227 ml for 60 minutes of simulated flow of 100 veh/h/lane, for a modification of a single speed changing point. The overall benefits are reduced energy consumption, air pollution and noise which otherwise would have been produced by braking. This work brings an effective optimization tool for road managers and its practical application is passive and inexpensive. This methodology is suitable for rural and urbanized territories and easily adaptable to any type of traffic in various countries. In perspectives, the optimization process could be extended to a full road route and to a wide range of different speed-sectioning layouts.

Keywords: energy savings, road design, road exploitation, eco-driving, speed optimisation

1 Introduction

Road transportation has a large impact on greenhouse gas emissions (GHG) and climate change. Expert are considering that a roughly 15 % reduction in transport sector final energy use by 2050 compared to 2015 is needed to fulfill the “best” scenario of global warming below 1.5°C (IPCC 2018 report [1]). On the other hand, this IPCC report states that emissions from the transportation sector increased by 2.5 % annually between 2010 and 2015 and that transport accounted in 2014 for 28 % of global final energy demand and 23 % of global energy-related to CO_2 emissions.

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At this point, it can be concluded that our current transport system is not sustainable. The key point is road system since road vehicles account for nearly three-quarters of transport CO2 emissions (IEA, 2018, [2]). To comply with this environmental emergency, road vehicle emissions can be lowered by several means: limiting the car use, enhancing vehicle efficiency and developing the use of vehicles relying on non-fossil energies, reducing the infrastructure-linked energy demand.

This study contributes to this reduction of infrastructure-linked energy demand, by proposing a more convenient succession of speed limitations along a route, speed, which could increase eco-driving potential of the road. Indeed, this succession of speed limitations, called speed-sectioning could impede or favour driver ecodriving, depending on its adequacy between vehicle dynamics and longitudinal road profile.

The surrounding methodology of this study is detailed in [3], with experimental and simulations steps validated [4] before this present optimisation phase, but, out of other application example, a speed sign placed in a downgrade can impede ecodriving by forcing drivers to brake mechanically instead of to simply decelerated; a beforehand situation of the sign, in a uphill or plane section, could otherwise allow ecodriving.

Speed signs or other speed-sectioning points (roundabouts, pedestrian crossings, should then be considered both with consideration of vehicle dynamics and longitudinal road profile. Turns are another road parameter which can impede eco-driving with a non-optimal speed-sectioning, by limiting driver sight distance.

The objective of this research is to determine an optimization process of a route speed sectioning which minimizes the fuel consumption while meeting the safety constrains.

Another research field of lowering the infrastructure energy demand is to optimise the design of new roads, or to rebuild old roads, in the aim to lower the use phase energy demand (vehicles) without impacting too much its building and maintenance phases. This complementary research could lead to different longitudinal profiles or curves [5].

In the next section the methodology and its numerical implementation are described. A case study is used to support the proof of concept of the methodology. The last section concludes this paper.

2 Optimization methodology

2.1 Iterative road speed-sectioning optimization

Fig. 1 displays the proposed methodology. Starting from an initial speed sectioning, visibility distances of speed signs associated with this initial speed sectioning are determined (visibility distances can also be directly given by the user) and the associated maneuvers of vehicles allow to compute the fuel consumptions of vehicles by the mean of a traffic simulation for given vehicle data and driver behaviors. The output of this simulation is the sum of each vehicle fuel consumption. The algorithm optimizes iteratively the speed sectioning among of admissible speed sectioning according to road safety constraints and to road geometry. The optimization criterion is the minimization of the fuel consumption.

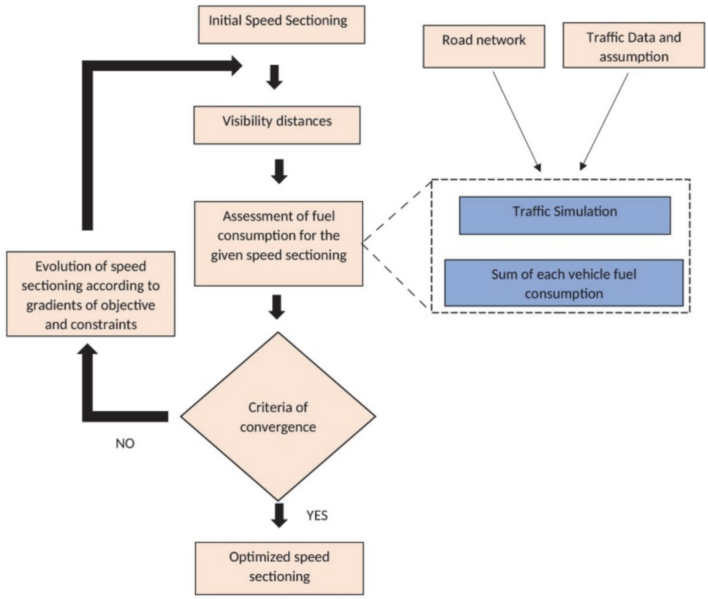

Figure 1 Optimization methodology

2.2 Optimization implementation with SUMO/Python

The previous methodology implies that the traffic simulation is carried out inside a loop. Opensource software Sumo was used on the ongoing research. Its crucial feature for this study is that Sumo can be launched and controlled from Python by using the Python library Traci. This feature is illustrated by fig. 2. this figure displays the implementation of the traffic simulation (blue blocks of fig. 1) by using the interaction between Python and Sumo. The blocks in green (resp. blue) are the blocks of instruction in Python(resp. Sumo). At first, the Traci library is loaded in the Python environment. Speed sectioning and visibility distance are given by the optimization algorithm. Then Sumo is launched. The first step in Sumo is to load xml files including information on road network and traffic data. Then Sumo enters in a loop. It computes one time step of the simulation. At each time step, Python's script changes the speed computed by Sumo of the vehicles which are inside the visibility distance of speed sectioning as well as those which are inside this section. The simulation is over when the number of iterations is reached which is equivalent to the simulation time when reached. If it is not the case, Sumo simulates traffic for a new time step. If it is the case, Sumo delivers an xml file including the fuel consumption of all vehicle. By processing this xml file, Python's script computes the fuel consumption associated with the given speed sectioning. By using this feature of the couple Python/Sumo to control directly the speed of each vehicle inside speed sectioning, it means that this speed sectioning can be modified from Python. This feature is the key of opening the door to the optimization algorithm. Moreover, by using this feature to control directly the speed of each vehicle approaching speed sectioning, drivers' modeling is accurate. With this interaction between Sumo and Python, the cast of each trajectory according to the type of driver can be done. In fact, the speed is proposed by the Python's script to Sumo which takes into account other factors: maximal deceleration of the vehicle and of the driver, tracking model between vehicles in order to compute the vehicles speed used in the simulation.

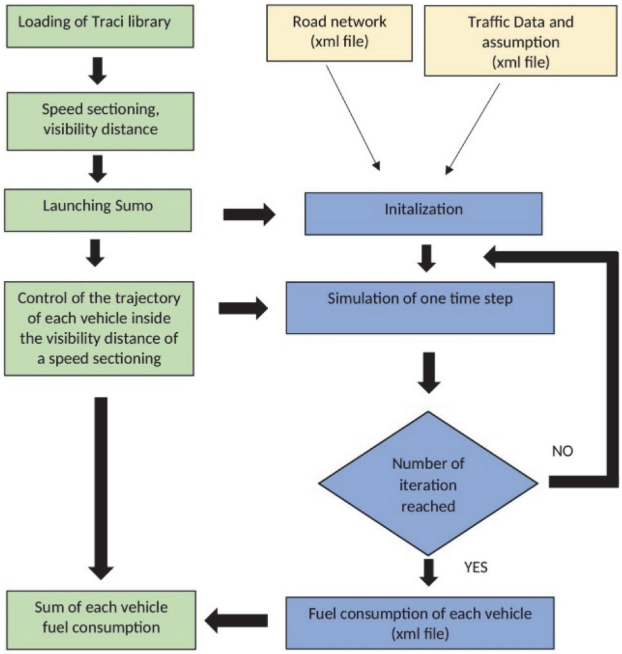

Figure 2 Interaction between Python (green color) and Sumo (blue color) to control trajectory of vehicle approaching speed sectioning

3 Case Study

3.1 Geographical and traffic data

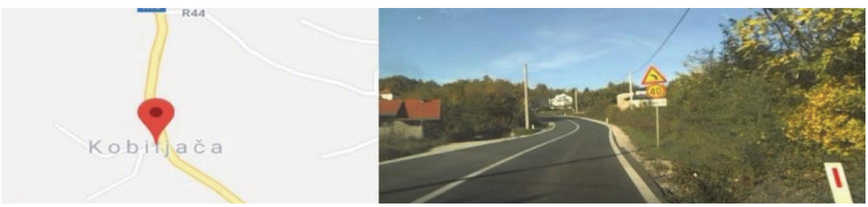

Figure 3 Situation (google maps) and view of the considered speed sign

Fig. 3 represents the misplaced speed limitation sign in Bosnia-Herzegovina at its location. This ecologically black spot was documented in a previous study [5]. It consists in a speed section of 40 km/h on a road mainly limited to 80 km/h. This speed section of 40 km/h ensures safety conditions of nearby villages. The limiting speed sign of 40 km/h is located after a sharp turn. Thus, drivers are surprised, they have no other choice than to brake mechanically if they want to respect the speed limitation sign. The objective of the optimization algorithm is to find the best place to implement the speed limitation sign and by minimizing fuel consumption of the traffic while ensuring safety conditions to the nearby villages. The degree of freedom of the algorithm is the position of the speed limiting sign between the initial position and an maximum upstream position (300 m upstream).

The geographical position of the sign is located in mountainous rural area. It is on the path of two nearby villages. The length of the analyzed section is 7.6 km. Based on real traffic data the number of vehicle on this route is approximately 200-500 veh/h.The visibility distance for the actual panel is 50m. The next table presents for each position of a virtual panel located upstream of the current panel, the visibility distance. The time period of the simulation is 3600 sec. where 100 vehicles/lane are simulated.

Table 1 Visibility distance according to the position of the virtual panel

Pos (m)	0	20	30	50	60	80	100	130	150	260	300
Vis (m)	50	65	80	85	75	60	45	125	130	170	160

3.2 Sumo Setup

The model of environment is imported in Sumo by using open-street map.The simulation is conducted primarily on diesel vehicles with EURO 4 standards and HBEFA 3.2 protocols. Two types of vehicle are simulated: cars and trucks with three different types of drivers: aggressive, defensive, and eco-driver. It means that 6 (3x2) flows are simulated. The main characteristics are displayed as follows (C stands for Car, T for Truck)

Table 2 Drivers characteristics for the Sumo simulation

Type	C-eco	C-agg	C-def	T-eco	T-agg	T-def
Max dec. ($m.s^{-2}$)	0.40	1.00	1.00	0.3	0.9	0.9
Max acc ($m.s^{-2}$)	0.60	1.00	0.60	0.50	0.90	0.50
Speedfactor	1.03	1.10	0.90	1.03	1.10	0.90
Max Speed ($m.s^{-1}$)	30.0	38.0	30.0	22.2	27.0	22.2
Percentage	55	15	15	8	5	2

3.3 Drivers Modeling

- Drivers Model depends on the position of the vehicle:
- when a vehicle is approaching the village section or is in the village, the script Python proposes a speed computed according the type of drivers in order to cast the trajectory to a trajectory type. Sumo manages this speed by taking into account the maximum deceleration, speed factor and interaction between vehicles;
- outside this part of the road network, Sumo manages by itself vehicles speed.

Firstly, the interaction between Sumo and Python are presented, then the trajectory types omputed by our Python algorithm are described.

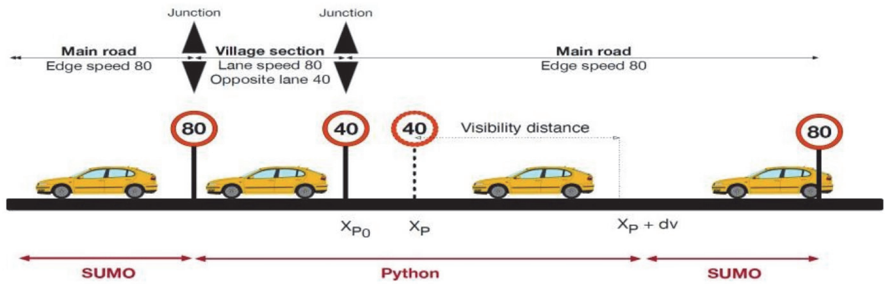

Figure 4 Simulation schema of a optimized speed-sectioning

Fig. 4 displays the interaction between Sumo and Python when a vehicle is approaching the village section. The model of the considered road network inside Sumo is a graph composed of edges and junctions. An edge includes two lanes. XP0 is the actual speed limiting sign position along the road. XP is the virtual position, dv is the visibility distance. Before entering in the distance of visibility of the virtual panel, the trajectory is controlled by Sumo. Afterward, Python's script proposes a speed to Sumo in order to control the trajectory until the end of the village section. For Sumo, there is no speed sectioning on this lane, the limiting speed is 80 km/h along all the lane. The speed sectioning is managed by the Python's script. This feature is critical to include the simulation in the optimization loop.

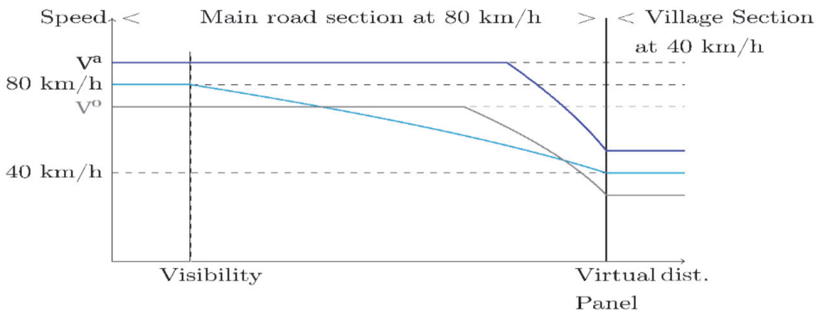

Figure 5 Three types of trajectory when approaching the virtual panel

The drivers trajectories approaching the virtual panel were cast according to the three types of drivers described above (aggressive, defensive and eco-driver).

Fig. 5 illustrates the behavior of two types of non eco-driver in comparison with the trajectory of an idealized eco-driver while approaching a speed sectioning. The blue curve is the trajectory of a vehicle driven by a aggressive driver. The driver speed, Va is slightly above the authorized speed 80 km/h. After seeing the speed limiting sign, the driver will brake mechanically with its deceleration of the driver type until reaching his new speed target. The grey curve represents the simplified trajectory of the defensive driver who will drive slower, at the speed Vo, than the regulatory speed and with a longer reaction delay. The deceleration is still important because she/he wants to comply with the speed limit although its longer reaction delay. The cyan curve represents the idealized eco-driver, who will release the gas pedal as soon as she/he sees the speed limiting sign. Its deceleration is the smallest of the driver types. The speed of the eco-driver is consistent with the regulatory speed.

Fig. 5 is an example of idealized trajectories when the visibility distance is long enough for each driver to brake according to its desired deceleration. If the visibility distance is too short, drivers have to decelerate more strongly to reach their desired speed after the virtual panel. Our Python algorithms adapts these idealized trajectories to the actual visibility distance.

4 Results

- The algorithm proposes to re-position the speed limitation sign 192 m upstream.
- The optimized fuel consumption is 72.3 l to compare with 72.5 l when the speed limitation sign is at its actual position.
- The gain is 227 ml for 60 minutes of simulated traffic flow of 100 veh/h/lane.

If these results are presented daily it would represent a gain of 5.5 l of fuel/day. On a yearly base a gain of 1988 l of fuel/year would not be spent in the atmosphere.
These results can be transferred in CO_2 emissions. Considering that the combustion of 1 liter of diesel ouputs 2.6 kg of CO_2.which is a simplified computation of emission factor given by [7]. One optimized single speed limitation sign could save up to 5250 kg of CO_2 not emitted in the atmosphere by year which is quite significant considering the low marginal cost to displace a speed sign.
The optimization is monocriteria on the fuel consumption. This can lead to unfeasible solution as to limit the speed along all the network to the minimum speed. The proposed methodology can avoid this drawback by monetizing the fuel and the driving time.

5 Conclusion

A new methodology to optimize speed sectioning has been presented. As a proof of concept, the optimization process was applied on a real single misplaced speed sign. Results are significant by saving 5t of CO_2.
The next step is to apply the optimization code on a larger road network. At short term, assessments of misplaced speed sectioning will be enriched by other criteria: air pollution and braking noise. At mid term, these academic results will be transferred to road managers by delivering them suitable tools to assess and to optimize speed sectioning of their network.

References

[1] De Coninck, H..: Strengthening and Implementing the Global Response, Global warming of 1.5°C. An IPCC Special Report on the impacts of global warming of 1.5°C, In Press, pp. 313–443, 2018.

[2] IEA (2019), Tracking Transport, IEA, Paris https://www.iea.org/reports/tracking- transport-2019

[3] Deljanin, E.: Eco-driving potentiality of road infrastructures according to the adequacy between infrastructure geometrical characteristics and vehicles speed, University of Nantes; University of Sarajevo, PhD Thesis, 2019.

[4] Coiret, A., Vandanjon, P.-O., Deljanin, E., Ortiz, M., Lortino, T.: Management of road speed sectioning to lower vehicle energy consumption, AIIT 2nd International Congress on Transp. Infrastructure Systems in a changing world, Rome, Italy, 23-25.9. 2019.

[5] Vandanjon, P.O., Vinot, E., Cerezo, V., Coiret, A., Dauvergne, M., Bouteldja, M.: Longitudinal profile optimization for roads within an eco-design framework, Transportation Research Part D, 67, pp. 642-658, 2019.

[6] Deljanin, E., Vandanjon, P.-O., Coiret, A., Causevic, S.: Ecodriving potential of roads according to their speed sectioning : Bosnia and France cases, 5th International Conference on Road and Rail Infrastructure, Zadar, Croatia, May 2018, pp. 987–992

[7] Emission factor database (EFDB) IPCC website, https://www.ipcc- nggip.iges.or.jp/EFDB/downloads.php

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COMPARATIVE ANALYSIS OF TAC ON RAILWAY FREIGHT CORRIDORS BETWEEN NORTH ADRIATIC PORTS AND ŽILINA

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Abstract

In recent years, European ports have, due to the increasing traffic between the Far East and Europe, become increasingly important. Large volumes of freight come to ships and it is important to have a link to the mainland via railway, which should have a functional overall system. This research compares three variants of possible routes for railway freight transport from the North Adriatic ports of Rijeka, Koper, and Trieste to Žilina, in the north of Slovakia. The methodology for calculating the minimum package of train access charges for the countries covered by these routes is presented. A comparative analysis of train access charges (TAC) for the minimum access package for the corridors between North Adriatic ports and City of Žilina has also been conducted. The result entailed in this research is the most favourable railway route for freight transport from the North Adriatic ports to the City of Žilina.

Keywords: North Adriatic ports, freight transport, railway, intermodal transport, train access charges

1 Introduction

Globalisation development has led to a substantial increase in trade and sea transport between the Far East and Europe. The increase has placed higher geographical value on the North Adriatic which ensures the development of ports in that area. The ports of Rijeka, Koper, and Trieste are seaports on the north shores of the Adriatic Sea [1], which ensure direct access to the European mainland. Such favourable location is what makes these ports the shortest link between the Central and Central Eastern Europe and Asia, Africa, and Mediterranean countries. The importance of the transport position of the three ports lies in the fact that they are located along the EU core transport network, which is comprised of nine TEN-T network corridors [2]. The Port of Rijeka is the main port of the Mediterranean corridor, while the Port of Koper and the Port of Trieste are main ports of the Mediterranean and Baltic Adriatic corridor, respectively.
The City of Rijeka is located in the west of the Republic of Croatia. It is the third-largest Croatian city and the seat of the Primorje-Gorski Kotar County. The Port of Rijeka is the largest and the most significant in the country. Given its location, the port of Rijeka advantage over the North Sea and Baltic ports lies in the fact that it is the shortest route between Europe and the Far East. For Central and Central Eastern European countries without access to the sea, a faster transport route ensures a faster freight movement and thereby also a reduction in transport costs.

Koper is a city in south-western Slovenia, the seat of the Coastal–Karst Region and the only coastal city along the 47-km coastline which includes one of the busiest North Adriatic ports – the Port of Koper. Because it is situated along two TEN-T corridors – the Mediterranean and the Baltic-Adriatic Corridor – the port has a great connection to the mainland.

The City of Trieste is the regional capital of Friuli Venezia Giulia situated in the north-eastern part of Italy. The Port of Trieste is situated in the Gulf of Trieste on the North Adriatic coastal belt. Trieste is also where the main longitudinal transport routes intersect with the mainland routes from Central Europe – the Baltic-Adriatic Corridor and the Mediterranean Corridor – which ensures an adequate connection between the port and European cities along the TEN-T European Network. The 18-meter draught, easy access, and outstanding road and rail routes have all turned the port of Trieste into an efficient and competitive port. Good rail connectivity between Trieste and Europe has placed the port among the twenty most significant ports of Europe.

Žilina is the fourth largest city in the Republic of Slovakia. It is situated in the north, near the Czech-Polish border, 200 kilometres from the capital of the country, Bratislava. The city of Žilina is the main industrial centre of the northern confluence of the river Váh, with a fast-growing economy. Strong industry helps develop an area and improve the life standard of residents. Therefore, railway transport is essential for the economy of a country and a region which is why substantial research is made for the role of the international railway transport and regional economy [3]. From a geographical standpoint, Žilina has a quite favourable position. It is located along the firth TEN-T corridor that connects the Baltic ports in Poland to Adriatic ports, and numerous other strong economic centres such as Warsaw, Katowice, Ostrava, Brno and Vienna along the ninth Rhine–Danube corridor [4]. Additionally, Žilina is also situated along the newly planned Amber corridor.

This research will analyse the benefits of the different variants of railway freight transport along the North Adriatic ports – Žilina routes whereby three variants will be shown for each port. The aim of this is to establish the advantages of the variants between the North Adriatic ports and Žilina.

2 Train access charges calculations

Train access charges (TAC) is a model of charging railway operators for the use of railway infrastructure. The fundamental principles of such a model must include: 1) simplicity, 2) transparency, 3) neutrality, and 4) cost dependence [5].

The simplicity indicates that there are no additional, hidden, or ambiguous expressions in the practical application of the model and that the calculation is clear and logical. Transparency means that, regardless of the obligations, the costs will be consistent and fair. Neutrality refers to a railway undertaking having equal approach and relationship to every services user. Since the charging model includes various services, the model itself must be founded upon real generated costs for these services. This way, the model directly adheres to the principles of transparency and neutrality [6].

Every country has its calculation methodology which is why the methodologies of charge calculations must be conducted for every country through which freight will be transported from North Adriatic ports to Žilina [7]. At the same time, only the minimum access package calculation which represents the basic service package provided by the railway undertaking to the operators will be taken into account. The minimum access package charge in Croatia [8][9] is calculated according to the following formula:

$$C = \left[(T + d_m + d_n) \cdot \sum (L \cdot I) \cdot C_{vlkm} + (I_{el} \cdot C_{el})\right] \cdot S \qquad (1)$$

whereby C is the total charge amount, T is the train path equivalent, d_m is the charge for the use of tilting technique, L is the line parameter, l is the train path length, C_{vlkm} is the basic price (HRK/trainkm), l_{el} is the length of train path with electric traction, Cel is the additional charge on trainkm price for the train path with electric traction (HRK/trainkm), and S is the coefficient for the single wagon load train. In Slovenia, the minimum access package charge [10] is calculated according to the following formula:

$$\mathbf{U} = \sum_{i=l}^{vv}\sum_{vv,\,i} \mathbf{Q}_{vlkm(vv,\,i)} \cdot \mathbf{F}_{vv} \cdot \mathbf{P}_i \cdot \mathbf{C}_{vlkm} \cdot \mathbf{C}_{vp} \tag{2}$$

$Q_{vlkm(vv,i)}$ refers to the number of train kilometres performed on certain line categories (i) and by the same power car (vv), F_{vv} is the coefficient of the power car category (vv), P_i is weighting of the line category (i), C_{vlkm} is the cost per train kilometre, and C_{vp} the cost of supplement or deduction for the type of transport (depends on the type of the train).
The minimum access package in Italy [11] is calculated using the following formula:

$$\mathbf{A}_c = \mathbf{A} + \mathbf{B} \tag{3}$$

$$\mathbf{A} = \mathbf{A}_{weight} + \mathbf{A}_{speed} + \mathbf{A}_{contact\ line} = \left(\mathbf{T}_{A1} + \mathbf{T}_{A2} + \mathbf{T}_{A3}\right) \cdot \mathbf{l} \tag{4}$$

$$\mathbf{B} = \mathbf{T}_B \cdot \mathbf{l} \tag{5}$$

A_{weight} relates the wear and tear of the track due to the weight of the train, T_{A1} is the train weight parameter, A_{speed} refers to the relates the wear and tear of the track due to the operating speed classes of the train, T_{A2} is the train speed parameter, A_{weight} refers to the wear and tear of the overhead contact line, T_{A3} refers to the use of contact network depending on the traction type, and T_B refers to the service cost, and l refers to the distance travelled.
The Austrian railway undertaking calculates the minimum access charge [12] based on the following formula:

$$\mathbf{T}_{AC} = \mathbf{Trainkm} \cdot \mathbf{z} + \mathbf{Gtkm} \cdot \mathbf{g}_{tk} \pm \ \mathbf{reductions / supplements} \tag{6}$$

Trainkm is the train-kilometre component, z is the train-kilometre coefficient. G_{tkm} is the multiplication of the gross-tonne and kilometres, and g_{tk} is the gross-tonne-kilometres coefficient. Reductions or supplements are infrastructure congestions, delays in minutes, traction unit coefficient, noise bonus, and the number of axles.
The Hungarian minimum access package [13] is calculated using the following formula:

$$A_c = A_{TV} + A_{KV} + A_E \tag{7}$$

$$A_{TV} = a_{TV} \cdot train\ km \tag{8}$$

$$A_{KV} = a_{tkm} \cdot trainkm + a_{gtk} \cdot brtt \cdot trainkm \tag{9}$$

$$A_E = a_E \cdot trainkm \tag{10}$$

where A_c refers to the total charge, A_{TV} is the train route charge, a_{TV} refers to the train route insurance charge per train kilometre, A_{KV} is the train movement charge, a_{tkm} is the train kilometre charge (depending on the track category), gross-tonne-kilometre charge is a_{gtk}, the charge for using electric traction is A_E, and a_E is the charge charged for the use of electric traction per train kilometre.

In Slovakia [14], the minimum access package is calculated as follows:

$$\boldsymbol{U} = \sum_{i=1}^{U_i} \boldsymbol{U}_i \tag{11}$$

$$\boldsymbol{U}_1 = \boldsymbol{u}_1 \cdot \boldsymbol{l} \tag{12}$$

$$\boldsymbol{U}_2 = \boldsymbol{u}_2 \cdot \boldsymbol{l} \tag{13}$$

$$\boldsymbol{U}_3 = \frac{\boldsymbol{u}_3 \,.\, \boldsymbol{brtt}/_{\boldsymbol{vl}} \cdot \boldsymbol{l}}{1000} \tag{14}$$

$$\boldsymbol{brtt}/_{\boldsymbol{vl}} = \boldsymbol{m}_L + \boldsymbol{n}_{vag} * \left(\boldsymbol{m}_{vag} + \boldsymbol{tara}_{vag}\right) \tag{15}$$

$$\boldsymbol{U}_4 = \frac{\boldsymbol{u}_4 \,.\, \boldsymbol{brtt}/_{\boldsymbol{vl}} \cdot \boldsymbol{l}}{1000} \tag{16}$$

U_1 is the maximum charge for requesting and assigning capacity, U_2 is the traffic management and organization charge, U_3 is the infrastructure service use charge, U_4 refers to the use of electric traction, l represents the distance, m_L is mass of the locomotive, n_{vag} indicates the number of carriages, $tara_{vag}$ is the carriage mass, $brtt_{vl}$ is the overall train weight, and u_1, u_2, u_3, and u_4 are fees charged for a component on a route.

3 Case study: A comparative analysis of TAC on railway freight corridors between North Adriatic ports and Žilina

The comparative analysis for all three ports was carried out in three variants for each port. The following corridors were analyzed for Rijeka: 1) Port of Rijeka–Hungary–Žilina Teplička, 2) Port of Rijeka–Slovenia–Austria–Žilina Teplička, and 3) Port of Rijeka–Slovenia–Hungary–Žilina Teplička. The corridors from the Port of Koper included: 1) Port of Koper–Austria–Žilina Teplička, 2) port of Koper–Hungary–Žilina Teplička, and 3) port of Koper–Croatia–Hungary–Žilina Teplička. The corridors from the port of Trieste included: 1) Port of Trieste–Austria–Žilina Teplička, 2) Port of Trieste–Slovenia–Hungary–Žilina Teplička, and 3) Port of Trieste–Slovenia–Croatia–Hungary–Žilina Teplička [15].

The first 929.30-km corridor originating in Rijeka continues through Hungary to Žilina in the north of Slovakia. The entire track of the corridor is electrified by an AC 25 kV, 50 Hz system, apart from the final section, Púchov–Žilina, which uses a 3 kV electrification. The number of tracks depends on the section. The maximum axle load is 22.5 tons and the longitudinal load varies across sections – the minimum being 6.4 tons per metre. The second section passes through Slovenia and Austria where there is the issue of non-interoperability. Each of these countries has its voltage system which requires a locomotive change at border crossings. Alternatively, multi-system locomotives can be used. The corridor is 822.23 kilometres long. The third corridor is the longest, measuring 988,76 kilometres. The shortest section thereof passes through Croatia (31 kilometres). Electrification systems along the corridors vary - Croatian section uses a 25 kV 50 Hz system, Slovenia electrifies the track with 3 kV, and Hungary and partly in Slovakia, the track again uses the 25 kV 50 Hz electrification system. The final section of the corridor is again electrified using 3 kV.

The first corridor from the port of Trieste goes through Austria to Žilina and is 798.62 kilometres long. It uses three different electrification systems. The Italian section and a part of the Slovak track are electrified with the direct 3 kV system, the Austrian section uses alternating 15 kV 16 ⅔ Hz current system, and the part between the Austrian-Slovakian border and Puchov uses the 25 kV and 50 Hz system. The Austrian and Slovakian section has a maximum axle is 22.5 tons per axle and the longitudinal load is 8 tons per meter. The second corridor – 988.90 km – stretches from Trieste to Žilina via Slovenia and Hungary. The corridor is mainly double-track with either 3 kV direct or 25 kV 50 Hz alternating current system. The third corridor, Trieste – Žilina, crosses Slovenia, Croatia and Hungary and is the longest – 1086.30 km. The maximum loads and number of tracks differ between sections, as does the track electrification systems. The Italian-Slovenian part of the corridor and the Puchov – Žilina section use a direct 3 kV electrification, while the remaining part of the track uses the 25 kV 50 Hz system. For the comparison to be fair, the same train composition was used on all routes - a single locomotive and 21 Sgs carriages, approximately 500 meters in length and 1281 tons in weight, not counting the locomotive weight (Table 1).

Table 1 An overview of the routes, access charge and journey time of the corridors

Route	Variant	Transport length [km]	Access charge [€]	Journey time [h]		
				v = 30 km/h	v = 50 km/h	v = 70 km/h
Port of Rijeka - Žilina Teplička	1.	929.30	2704.20	31.00	18.60	13.30
	2.	822.23	3176.34	27.40	16.40	11.70
	3.	988.76	2758.84	33.00	19.80	14.10
Port of Koper - Žilina Teplička	1.	836.17	3209.90	27.90	16.70	12.00
	2.	1002.40	2907.93	33.40	20.00	14.30
	3.	1000.30	2816.19	33.30	20.00	14.30
Port of Trieste - Žilina Teplička	1.	798.62	3938.13	26.60	16.00	11.40
	2.	988.90	2797.68	32.90	19.80	14.10
	3.	1086.30	2822.73	36.21	21.70	15.50

The freight corridor with the shortest journey time, not taking the charges into consideration, is the Trieste–Žilina Teplička via Austria – 798.62 kilometres. The journey time of an express train that combines multimodal transport and does not stop along the way is 11.4 with an average speed of 70 km/h (all journey times do not include stops to change traction systems). Other routes have approximately the same journey time – the Rijeka–Slovenia–Austria–Žilina Teplička corridor measures 11.7 hours for 822.23 kilometres, and the Port of Koper – Austria – Žilina Teplička route of 836.17 kilometres has a 12 hour journey time. If time is not a factor and the objective is to reduce transport costs as much as possible, the best corridor is the Port of Rijeka to Žilina via Hungary. The length is 929.3 kilometres and is covered in 13.3 hours at an average speed of 70 km/h - not taking into account the stops for traction change. Variant 3 (Rijeka–Žilina Teplička) and variants 2 and 3 (Port of Koper–Žilina Teplička) and variant 3 (Port of Trieste–Žilina Teplička) are 988–1003 kilometres long with an average journey time of 14 hours and 15 minutes and access charge 2750–2910 euros.

4 Conclusion

Geographically, North Adriatic ports have an exceptionally favourable position compared to other European ports. It is the significantly shorter journey time that contributes most to their importance - it is 6 times shorter compared to North Sea ports. The Port of Trieste has the largest volume due to its long tradition in freight transport and continuous investments in maritime and mainland infrastructure. The Port of Koper consistently invests in its infrastructure, particularly the railway, which ensures its competitiveness in the market. The Port of Rijeka has been experiencing issues because for years there have been no infrastructural improvements. This has led to a situation where the port's railway transport is less competitive than road transport. In container transport, the port of Koper dominates with 988,501 TEU units in 2018. The Port of Trieste is keeping up with Koper, increasing its annual volume – 725,426 TEU units in 2018. The Port of Rijeka trails behind with 260,375 TEU units in 2018. However, it should be noted that this is the port's record which indicates an upward trend of TEU units every year.

This research has compared three variants of possible corridors between North Adriatic ports – the ports of Rijeka, Koper and Trieste – and the north of Slovakia, i.e. Žilina. Ultimately, the analysis has examined the length of the corridors, journey times, access charges – taking into account only the minimum access package. The analysis has also concluded that that the shortest corridor (shortest journey time) is the Trieste–Austria–Žilina Teplička corridor. The only issue that arises here is the high charge. Given that most of this corridor passes through Austria, where charges are highest, which makes the corridor more expensive than the others. The corridor which is more favourable for the operators for whom the cost is not a more decisive factor than the journey time. For others, the most favourable corridor is the Rijeka–Hungary–Žilina Teplička. Compared to the shortest one, this corridor is 130.70 km longer, which amounts to approximately 2 hours. However, access charges are 1233.93 euros lower. The two corridors tower over the others at approximately the same range in terms of transport route and access charge.

Railway operators demand various services and make their own respective decisions as to how much they will pay for the service. Similarly, every port and every terminal provide various additional services, reception capacities, freight storage and different levels of connectedness to the mainland. What ultimately determines the corridor choice is up to the transport user, the one that dispatches freight. They can opt for a North Adriatic port that can provide the best services at a given cost. The corridors they choose will be based on time and financial conditions.

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References

[1] Ports of Napa, http://www.portsofnapa.com/, 2019.

[2] Ližbetin, J., Ponický, J., Zitrický, V.: The throughput capacity of rail freight corridors on the particular railways network, NAŠE MORE: znanstveno-stručni časopis za more i pomorstvo, 63 (2016) 3, pp. 161-9

[3] Abramović, B., Zitricky, V., Biškup, V.: Organisation of railway freight transport: case study CIM/SMGS between Slovakia and Ukraine, European Transportation Research Review, 8 (2016) 4

[4] Dolinayova, A., Cerna, L., Zitricky, V.: The Role of Railway Transport in East-West Traffic Flow Conditions of the Slovak Republic, In Transport Systems and Delivery of Cargo on East–West Routes, pp. 121-169, 2018.

[5] Abramović, B.: Infrastructure access charges, Sustainable Rail Transport 2018, Cham, pp. 45-58, 2018.

[6] Široký, J.: Price for the Allocation of Railway Infrastructure Capacity as a Tool for the Improvement of Train Transport Planning, MATEC Web of Conferences 2017.

[7] Dolinayova, A., Camaj, J., Kamaj, J.: Charging railway infrastructure models and their impact to competitiveness of railway transport, Transport Problems, 2017.

[8] Network Statement 2019, HŽ Infrastruktura, Zagreb, 2019.

[9] Abramović, B., Šipuš, D.: The Comparative Analysis of Transport Service Quality in Regional Rail and Bus Traffic, Horizons of Railway Transport, 2016.

[10] Network Statement 2019, SŽ-Infrastruktura. Ljubljana, 2019.

[11] Network Statement 2019, Rete Ferroviaria Italiana S.p.A., Roma, 2019.

[12] Network Statement 2019, ÖBB-Infrastruktur AG, Wien, 2019.

[13] Network Statement 2019, VPE - Rail Capacity Allocation, Budapest, 2019.

[14] Network Statement 2019, ŽSR, Bratislava, 2019.

[15] Ribarić, M.: Comparative Analysis of Railway Freight Corridors Between North Adriatic Port and Žilina, Master Thesis, University of Zagreb, Faculty of Transport and Traffic Sciences, 2019.

A BPMN MODEL FOR RAIL PORT OPERATIONS TO EVALUATE POTENTIAL CAPACITY INCREASE

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Abstract

The rapid growth of container movements deriving from the advent of globalization has caused a relevant pressure on ports. In this context, the port efficiency performances requested by the market have been met by an increased adoption of railway transport. Despite its environmental and economic sustainability, this intermodal transport solution is characterized by a high degree of complexity given by the execution of different activities and the involvement of several stakeholders. As such, the need of deeply analyzing logistics business processes has led to the embracement of modeling techniques, which allow practitioners to identify possible criticalities along the chain. In this regard, the selection of the appropriate modeling method and notation definitely influences the results of the process examination. In the present paper, the train departure process in the Port of Trieste, Italy, has been investigated in terms of information flows: to that end, the Business Process Modeling Notation (BPMN) standard has been used. On one hand, this task has enabled to clarify the documentary procedure occurring in diverse operational scenarios and, on the other hand, it has allowed also to recognize the process bottlenecks which hinder an increase in the railway traffic volume. Furthermore, a quantitative analysis concerning the terminal and gateway capacity, before and after the implementation of a technological intervention, has demonstrated the possible growth of train flows by modifying the process only at the organizational level. These accomplishments have proved to port managers the usefulness of applying a theoretical graphical representation to a practical transport process.

Keywords: intermodal freight transport, rail operations, capacity, modelling standard, BPMN

1 Introduction

The phenomenon of globalization and the extensive adoption of containers occurred since the 1960s have substantially affected the change of the market environment which ports and shipping lines belong to [1]. Indeed, containerization has simplified freight transport and, at the same time, the size of containerships has significantly risen over the last decades to exploit the advantages of the economies of scale. The increase in freight standardization has allowed the implementation of intermodal transport systems, which permit the transfer of containerized materials by rail, truck or sea [2]. In order to sustain the growing freight demand, the effective management and execution of port operations has turned out to be fundamental as much as the presence of an adequate intermodal accessibility to roadways and railways [3]. A solution to drain the higher volumes of freight arriving to ports has consisted in a greater employment of trains, which has led to an enhancement of the competitiveness

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of ports and, therefore, of their throughput [4]. In this regard, due to the expected raise in the railway share in the near future, capacity usage has become an even more central issue. Indeed, focusing on railway nodes, the identification of critical infrastructural elements in their configuration and the determination of effective measures to fully exploit their potential use constitute two common problems of railway engineering [5].

The need of conveniently managing intermodal systems can be analogized to the one of running industries, independently from the activity field, since in both circumstances an efficient organization of business processes represents a key feature for gaining competitiveness within the marketplace. A solution to assess possible productivity advancements can be provided by Business Process Management (BPM), which is defined as "a systematic, structured approach to analyze, improve, control and manage processes with the aim of improving the quality of products and services" [6]. The activity of representing the processes of a company is called Business Process Modeling (BPMo) and is usually carried out by business analysts and managers with the aim of analyzing the current situation ("as is") and planning an improved one ("to be") [7]. Numerous BPMo techniques, and their corresponding tools, have been developed to capture the various aspects of business processes but, as suggested in [8], the determination of process modeling goals should be the initial task to choose the appropriate BPMo method. In the present paper, the main objective of process modeling consists in analyzing the departure procedure of freight trains in the commercial port of Trieste, Italy, in order to identify possible bottlenecks that impede a capacity increase. To this end, rail operations have been modeled taking into account also the documentary flow which occurs among the numerous private and administrative actors involved in the examined process. More in detail, the BPMN standard has been used to represent the activities performed by the different stakeholders and the execution time of the principal tasks has been estimated in order to compare the capacity values before and after the potential implementation of a technological intervention.

The article is structured as follows. The second chapter reports a literature review on both the most used graphical standards, and the application of BPMN to freight intermodal transport case studies. The third chapter explains the methodological approach adopted for BPMo, while the fourth one illustrates the key features of the port of Trieste and of the rail operations included in the BPMN process diagram. The fifth chapter contains the capacity analysis results, which are discussed in the sixth one, along with the description of future developments in the use of BPMo for the analyzed problem. Finally, the last chapter draws conclusions on the usefulness of applying the theoretical methodology of process modeling to the practical context of intermodal goods transfer.

2 Literature review

In line with the increasingly common principle of organizing businesses according to value-adding processes rather than to functional hierarchies, the deployment of modeling techniques has diffused among practitioners and academics, facilitating also the development of supporting software [9]. Limiting the scope of the present study to the design stage of the business process life cycle, the focus of the literature review has been restricted to graphical standards. Among the various existing techniques, the following methods are the most widespread when modeling business processes in a graphical manner: Unified Modelling Language Activity Diagrams (UML AD), BPMN, Event-driven Process Chains (EPC), Role-Activity Diagrams (RADs) and flow charts. Actually, these latter three methods can be considered only as tools to graphically display the chronological implementation of activities, since a standardization process is missing. On the contrary, BPMN and UML AD are two proper standards used to model business processes which result to be very similar, as they supply analogous symbols and control flow patterns [10]. UML AD is one of the thirteen

diagrams which are provided by the Object Management Group (OMG) mainly for modeling object-oriented software. Despite its maturity in designing single processes, there are some difficulties in modeling sub-processes and resource-related or organizational aspects, like the interaction with the operational environment [10]. BPMN is a standard originally developed by the Business Process Management Initiative (BPMI), which then joined the Object Management Group (OMG) for developing a new BPMN Specification document [11]. BPMN rapidly became the de facto standard, since it offers a simple and expressive look to business analysts and provides the foundation for process implementation. It uses an expressive flowchart-based graphical representation to model the business process flow. In particular, the main graphical elements are activities, gateways, events, sequence flows, pools and lanes. Since each of graphical element is translated into an XML element, BPMN models are used to communicate and interchange the business requirements of a business process, as well as to execute them on enterprise engines.
The literature review concerning the application of BPMo techniques to the intermodal transport field has been limited to those using BPMN, especially for its capability of representing processes at different levels of granularity and for the opportunity of mapping BPMN models to execution code [10]. A few previous researchers have investigated the intermodal transport topic stressing the relevance of information exchanges between the actors engaged in the operations. For instance, in [12], the authors adopted BPMN to model business processes and data flows related to the incoming container traffic in the Port of Hamburg focusing on the elements standardization and integration, in order to identify the significant junctures of the transport chain where these two features would enable a more efficient utilization of the current infrastructure. In [13], a BPMN model is proposed to examine the Port Community System of Salerno, Italy, not only in terms of the organizational procedure of each involved actor, but also of the inter-organizational routines between them. To this end, the as-is, to-be and gap analysis regarding the administrative activities of an export process were carried out, proving the relevance of creating an integrated information and communication platform for intelligent logistics services. The case study reported in the present paper differs from the previous ones for two main reasons. The first one consists in the fact that the standardization and integration of information flows are given for granted in the Port of Trieste, as it has been undertaking a radical and innovative computerization process of data exchanges for years, definitely improving the communication among stakeholders. The second difference comes from the higher degree of complexity of the examined logistics context, which is given mostly by the engagement of a variety of actors. More specifically, the suggested BPMN model has been created with the intention of making explicit the documentary procedure of the rail operations related to an outgoing train. In this regard, a detailed status quo analysis of the selected business process has enabled the determination of its criticalities, which constitute hindering factors for an increase in the railway node capacity.

3 Method - BPMN

Following the framework for selecting BPMo methods proposed in [8], the objective of process modeling has consisted in the analysis of the current situation of rail operations in the Port of Trieste, which has permitted the identification of possible improvements in the involved activities. Consequently, the established objective has influenced the perspective of the modeling method and its features. On one hand, the activity perspective has been adopted, enabling the representation of both the performed actions and of the relationships between them. On the other hand, characteristics like scalability and enactability have been sought during the selection of the appropriate modeling method to ensure its capability of, respectively, dealing with large processes and offering automated tools for process simulation. In light of these three constructs, that are objective, perspective and characteristics, the BPMN technique has been selected as the most appropriate modeling approach to address the appraisal of potential capacity increase in the Port of Trieste. To create the BPMN model of the problem considered in this study, much effort has been put to reach a good level of understandability, which depends mostly on graphical readership and pattern recognition. Attention has been paid in particular to the three categories of features, namely structure, layout and labeling, since, according to the investigation results reported in [14], they can frequently implicate some quality issues. Some recommendations concerning style and method suggested in [15] have been applied and, besides, the peculiar characteristics of the analyzed case study have been taken into account with the aim of building a context-sensitive model. Finally, in line with [16], stakeholder participation, information resources, and modeler's expertise have represented important factors to process modeling success. The relevance of engaging key actors in the evaluation of transport interventions has been corroborated in [17].

4 Case study

The Port of Trieste is located in the North-East of Italy, in a strategic position at the center of Europe, and represents the crossroads of different maritime routes and transport corridors. It constitutes an important international hub for the land-sea flows related to the marketplaces of Central and Eastern Europe, and more lately of Far East, also thanks to a great water depth. Besides, the Port of Trieste possesses an internal railway network, which is effectively integrated with the national and international ones. In this regard, it is the Italian port with the highest traffic volume of freight trains. The Port of Trieste is considered a Free Port, which means that customers can take advantage of special regulations with respect to customs procedures and the fiscal regime. In the recent past, the Port Authority has introduced a technological innovation consisting of an IT platform, named Sinfomar, in order to computerize the running process of port system operations. Through the creation of a section dedicated specifically to trains, Sinfomar enables not only the management of the incoming and outgoing railway flows in the Port of Trieste, but also the dematerialization of the document called CH30 and the interoperability with the information platforms of both railway and logistics third parties. CH30 is a digital customs document that contains detailed information on both the freight and the physical composition of trains and it certifies the arrival/departure of goods by train to/from the Port of Trieste, evolving through different statuses.
Bearing in mind the goal of identifying possible enhancements to increase the port railway capacity, in the present study it was decided to describe only the train outgoing process (Fig. 1) [18], since it allows to clearly visualize the potential criticalities limiting a growth of the traffic volume. The start event of the process consists in the reception by the customs of a provisional version of CH30, which is then athorized and verified also by the financial police if no mistakes are observed in the document and thus no modifications by the Multimodal Trasport Operator (MTO) are needed. Once the train is loaded, it is subjected to a pre-check by the competent railway company: in case of irregularities, cargo units have to be controlled, which can implicate their potential removal or addition and, consequently, a variation of CH30 by the MTO. If the outcome of the pre-check turns out to be positive, the MTO generates a definitive version of CH30, which is successively confirmed by the customs, and the train departure is validated. During the outgoing shunting, the train has to transit through a gateway which delimits the Free Port zone from the remaining port areas. In correspondence to this passage, the financial police perform a verification of both the wagons and loading units: if any irregularity is encountered, the execution of an additional shunting and a few modifications in the content and status of CH30 are necessary. In case of an affirmative result of this check, the financial police validate the final version of CH30 and the train is allowed to leave the port. Subsequently, after the detachment of the diesel locomotive used to perform

the exit shunting, the operation manager makes the train available to the railway company at the main port railway station, which is directly linked to the national network. Finally, prior to the opening of the departure signal by the national railway infrastructure manager, called Rete Ferroviaria Italiana (RFI), the railway company carries out a brake test on the train.

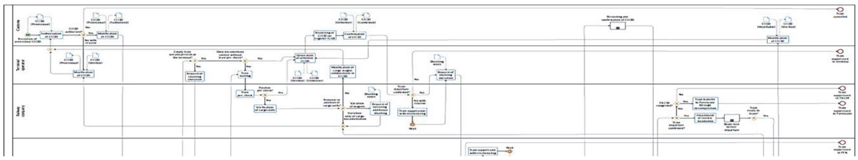

Figure 1 Part of the BPMN model of the examined railway process

5 Results

Using BPMN to model the train departure process, it was noticed that, at terminals, the loading procedure is followed by a train pre-check, which is currently performed manually by an operator of the railway company. According to the data provided by the Port Authority, that activity, along with the verification of cargo units, lasts about 35 minutes. This duration turns out to be quite substantial with respect to the one needed for loading operations, which are usually carried out in 3 hours. Considering that an additional hour elapses between two successive train loadings, the long duration of the pre-check primarily entails some traffic issues at terminals, rather than to the whole railway node. Indeed, taking into account a 22-hour daily operability and 288 working days for the terminals, the capacity of the three analyzed piers is limited to 9677 trains per year. The BPMN representation of the examined process has enabled the identification of another significant task for port capacity, i.e., the train check at the gateway separating the Free Port zones from the surrounding port areas. The detection of irregularities during that control implicates the addition of further 40 minutes to the normal execution time of the procedure. This delay is registered for 2 trains a day and entails the train stop right in correspondence of the gateway or, at times, on a buffer track in the proximity. The quite high frequency of check failures definitely has severe consequences on port railway traffic because, in that part of the infrastructure, a single track is present. In these circumstances, considering the time interval in which a departing train travels along the Free Port zone railway network until the gateway and the time necessary to exit its tail, the annual capacity on that specific infrastructure component equals to 11192 trains. Reasoning on the results of the "as-is" analysis, a procedural strategy to enhance the current operability has been developed and its impacts on the train flows have been estimated. The solution consists in improving the technique in which the train pre-check at terminals is carried out. As a matter of fact, the installation of optical reading portals able to automatically read the identification code of freights would certainly eliminate possible human mistakes. The spare of the time which is currently needed to perform the manual control, would allow an increase in the capacity of each terminal of 15 %, leading to an overall annual value of terminal capacity that is around 11088 trains. However, still considering a rate of 2 irregular trains per day, the growth of the total terminal capacity could potentially generate critical operational conditions, since it is near to the actual limit gateway capacity. Supposing that, thanks to the technological intervention at terminals, no irregularities are recorded during the control at the gateway, the annual capacity of this last infrastructure element would increase of 6.8 %, which means up to 11955 trains, obtaining a higher capacity margin.

6 Discussion

The evidences resulting from the BPMN modeling of a train departure process and from the quantitative capacity analysis have proved that the introduction of optical reading portals at terminals would not only enhance their capacity, but it would also have a beneficial impact on the control at the gateway, which represents the critical component of the port railway infrastructure. Indeed, the positive effects of the technological intervention proposed in the "to-be" scenario, i.e., the shorter duration of the execution time of the train pre-check and its greater quality reliability, would allow a growth in the annual port railway traffic of about 1000 trains. Thus, the suggested capacity increase would be accomplished just by modifying the current procedure at the organizational level, without realizing any additional track.
With respect to future advancements of the present study, going beyond the analyzed documentary flow, the departure process of a train within the Port of Trieste is planned to be investigated in depth also at the operational level, taking into account each specific shunting that is necessary to be performed. Besides, the interaction of different trains circulating in the port railway network is considered to be examined, in order to capture possible interference problems. Finally, the proposed static model is intended to be simulated using a tool called Business Process Simulation (BPSim), a standard by Workflow Management Coalition (WfMC) defining a specification for the parameterization and interchange of process analysis data, which is complementary also to BPMN. BPSim allows structural and capacity analysis of process models and supports both pre-execution and post-execution optimization of process models [19].

7 Conclusions

The remarkable increase in freight demand due to globalization and containerization has brought out the importance of an appropriate intermodal accessibility to inland transport modes and, consequently, the need of efficiently managing transfer processes at port facilities. To this end, business process modeling techniques have been adopted to deeply understand transport-related procedures at the organizational level, in order to identify potential criticalities of the current situation and to suggest an improved scenario. In the present study, the BPMN standard has been used to display the departure process of a train in the Port of Trieste, enabling to capture the bottlenecks that hinder a growth of the port rail capacity. The graphical representation of the examined process has constituted the basis for a quantitative analysis to compare the different time durations of the main involved activities before and after the implementation of some organizational interventions. This integrated evaluation has proved the possibility of a quite significant rise in the train traffic flows, respectively of 15 % at terminals and of 6.8 % at the critical gateway, without any modification to the existing railway infrastructure layout. Future developments of the research consist in representing via BPMN also the activities performed at operational level of the same railway procedure, considering the interactions among trains. Furthermore, the proposed model is intended to be animated by means of a simulation tool, in order to optimize the considered process. The accomplishments obtained in the present study confirm that BPM is a multi-disciplinary approach which demonstrates the usefulness of applying theoretical concepts to operational issues.

References

[1] Notteboom, T. E.: Container Shipping and Ports: An Overview, Review of Network Economics, 3 (2004), pp. 86-106

[2] Parola, F., Sciomachen, A.: Intermodal container flows in a port system network: Analysis of possible growths via simulation models, International Journal of Production Economics, 97 (2005), pp. 75-88

[3] Bassan, S.: Evaluating seaport operation and capacity analysis - preliminary methodology, Maritime Policy & Management, 34 (2017), pp. 3-19

[4] Caballini, C., Sacone, S., Siri, S.: The port as a system of systems: A System Dynamics simulation approach, 7th International Conference on System of Systems Engineering (SoSE), pp. 191-196, Genova, Italy, 16-19 July 2012.

[5] Malavasi, G., Molkovà, T., Ricci, S., Rotoli, F.: A synthetic approach to the evaluation of the carrying capacity of complex railway nodes, Journal of Rail Transport Planning & Management, 4 (2014), pp. 28-42

[6] Elzinga, D.J., Horak, T., Lee, C.Y., Bruner, C.: Business Process Management: Survey and Methodology, IEEE Transactions on Engineering Management, 42 (1995), pp. 119-128

[7] Chinosi, M., Trombetta, A.: BPMN: An introduction to the standard, Computer Standards & Interfaces, 34 (2012), pp. 124-134

[8] Luo, W., Tung, Y.A.: A framework for selecting business process modeling methods, Industrial Management & Data System, 99 (1999), pp. 312-319

[9] Aguilar-Savén, R.S.: Business process modelling: Review and framework, International Journal of Production Economics, 90 (2004), pp. 129-149

[10] Ko, R.K.L., Lee, S.S.G., Lee, E.W.: Business process management (BPM) standards: a survey, Business Process Management Journal, 15 (2009), pp. 744-791

[11] Object Management Group, https://www.omg.org/spec/BPMN/2.0, 16.11.2019.

[12] Elbert, R., Pontow, H.: Standardization and integration of information flow along the maritime transport chain, Darmstadt Technical University, Department of Business Administration, Economics and Law, Institute for Business Studies, 2013.

[13] Bisogno, M., Nota, G., Saccomanno, A., Tommasetti, A.: Improving the efficiency of Port Community Systems through integrated information flows of logistics processes, The International Journal of Digital Accounting Research, 15 (2015), pp. 1-31

[14] Leopold, H., Mendling, J., Guenther, O.: Learning from Quality Issues of BPMN Models from Industry, IEEE Software, 33 (2015), pp. 26-33

[15] Silver, B.: BPMN Method and Style, with BPMN Implementer's Guide, Cody-Cassidy press, Aptos, 2011.

[16] Bandara, W., Gable, G.G., Rosemann, M.: Factors and measures of business process modelling: model building through a multiple case study, European Journal of Information Systems, 14 (2005), pp. 347-360

[17] Caramuta, C., Giacomini, C., Longo, G., Padoano, E., Zornada, M.: Integrated evaluation methodology and its application to freight transport policies in the port of Trieste, Transportation Research Procedia, 30 (2018), pp. 119-126

[18] https://drive.google.com/drive/folders/1MqKO15xGi7OcwXd5muVzQjcXDqlT7rDi?usp=sharing

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CAR-FREE TRAVEL TO HOLIDAY REGIONS - MEASURES TO STRENGTHEN THE RAILWAY

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Abstract

There are two main reasons why people often choose to travel by car rather than by train. On the one hand, there is the transport of luggage, which many find too difficult. This is especially true when changing trains or travelling with children. Secondly, concern about the lack of mobility options at the holiday destination without a vehicle of one's own is a reason to prefer to go on holiday in one's own car. The paper refers to the easytravel project, an extensive research project in the Ötztal valley, a typical holiday region in Austria with a pronounced winter and summer tourism. The paper addresses the needs and concrete requirements of holidaymakers with regard to luggage transport and mobility offers in the holiday region and shows concrete measures how the willingness to travel by train can be improved.

Keywords: sustainable tourism, luggage transport, sustainable mobility, easy booking

1 Introduction

Tourism in Austria is largely characterised by guests in rural (alpine) areas who travel to the destination predominantly by their own cars. In the sense of sustainable tourism, an environmentally friendly arrival of guests is being promoted today. In order to encourage as many holidaymakers as possible to travel by train, it must be possible to replace their own car as far as possible at all stages of a journey. Studies show that the transport of luggage as well as concerns about lack of local mobility are the main criteria for choosing the means of transport. The aim of the "easytravel" (already completed) and ULTIMOB (currently ongoing) projects, which are funded by the Ministry for Climate Action in Austria in the Mobility of the Future programme, was to develop an "all-round carefree package" for non-car travellers, with special consideration of rail travel. This concerns in particular suitable luggage transport, ensuring the most flexible local mobility offers, and simple booking of all components and a seamless mobility chain in one process.

2 Methods

In addition to intensive research on the internet and in scientific literature, one focus of the project was a comprehensive survey of expectations, wishes, requirements and willingness to provide various services in the field of local mobility, baggage transport and information and booking options. This was achieved by interviewing approximately 11,000 people in trains, ski resorts or summer attractions and at Innsbruck Airport in winter, summer and in the off-season.

3 Measures to increase the attractiveness of rail travel in holiday regions

3.1 Baggage transport

A car offers the comfort of loading luggage into the car boot right at the starting point of the journey. In addition, in many cases it is not necessary to pay attention to the compact packing method, several loose pieces of luggage can be stowed. Before the luggage is unloaded at its immediate destination, the entire intermediate journey luggage does not essentially affect the comfort of the journey. For example, it has been shown that for 82 % of winter holidaymakers in Austria travelling by car, luggage transport is a major reason for choosing the car, whereas for only 55 % the cost and for 40 % the travel time have a decisive influence (see Figure 1).

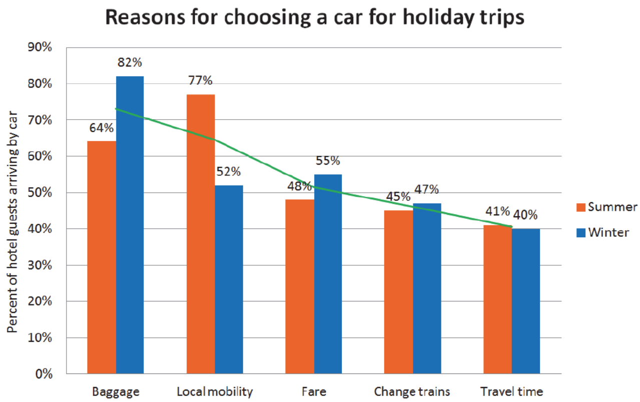

Figure 1 Reasons for choosing a car for holiday trips, Source: Rüger B.

The investigations in the above-mentioned projects show that there is an interest in baggage services, especially door-to-door delivery, and that good services would also create a concrete incentive to choose rail for travel to the holiday destination. Door-to-door baggage services are nothing new per se and are offered in many European countries for rail travel. In general, however, the weakness of these services is that they are not considered sufficiently flexible for travellers. In particular, the large time slots when collecting luggage from home or vice versa after returning home when delivering the luggage means that people have to wait an indefinite amount of time at home for the luggage service and sometimes need up to two days' holiday for it. In the holiday regions, hotel staff at the reception usually take care of the luggage, but if you are staying in apartments, there is seldom someone on site who can take care of the luggage for the guest who has not yet arrived. The projects show that there is a corresponding need and demand for different baggage handling services, which is particularly (but by no means exclusively) in the following areas:

- Holiday travel [Interest: approx. 40 %].
- In general when taking along larger pieces of luggage [approx. 50 %].
- Elderly people [approx. 40 %].
- Persons with reduced mobility [approx. 50 %].
- People who travel with children [approx. 50 %].

An attractive service from the traveller's point of view comprises two essential measures, the absence of which are also the main points of criticism of existing services: These are "(subjective) security" and the "flexibility" already mentioned above. The perception of security includes the virtual observation of the luggage and the certainty that it is on the right track or has reached its destination and must be achieved by means of real-time tracking information. Flexibility covers several points. One is flexibility at home:

- The time slot for both collection and delivery at the home address should ideally not exceed two, maximum three hours.
- A collection or delivery must also take place in the morning and in the evening so that you do not have to take a day off.
- On the day of collection or delivery there should be as exact and timely information as possible about the exact time of collection or delivery.
- Alternative collection or delivery options, such as partner shops or automated terminals similar to lockers, are desired.

Furthermore, flexibility in booking is desired. It is important to be able to make changes to the booking at short notice, for example, to the number of pieces of luggage.
The third important point is flexibility at the holiday destination: not all people stay in hotels where luggage can be delivered to or collected from. For holidaymakers in second homes or apartments, alternative collection and delivery options are also important.
In the project, a concrete concept was developed with the involvement of local companies, especially a regionally specialized logistics company, and tourism managers, which enables flexible collection and delivery possibilities in the model region Ötztal. If there is nobody at the destination address who can accept the luggage, a central address in the holiday region will be given as delivery address. These can be partner shops, concrete possibilities in the model region Ötztal are currently being coordinated. The logistics company responsible for the luggage transport then transports the luggage to the specified partner shop. Travellers, who usually move into their apartment the next day, can order the delivery of their luggage after arrival by phone or mobile phone application. Since passenger and goods transport is constantly taking place in the model region, it is expected that within two hours the luggage can be taken along and delivered to the guest within the framework of a journey that has been carried out anyway. On the return journey, luggage can also be picked up by a local company after pre-ordering and taken to the partner shop and stored temporarily where the logistics company can pick it up.

3.2 Local mobility

For 52 % of winter holidaymakers in Austria, concern about insufficient mobility at their destination is a major reason for travelling by car. In summer this applies to 77 % of holidaymakers (see Figure 1), which is due to the fact that in winter the focus is on typical winter sports such as skiing, while the required range of movement is smaller, as many hotels are located close to the ski lifts and guests can walk there or use the ski buses. In summer, holidaymakers also want to go on excursions, which increases the need for mobility. People from cities in particular are used to being able to choose their mobility flexibly. In cities, there are usually well-developed public transport systems with a dense timetable. Many people who live in cities therefore often do not own a car at all. Car sharing is a popular alternative. This flexibility at all times is therefore also required on holiday. Guests do not want to be tied too closely to timetables or even to means of transport that only run once a day. In many holiday regions there is therefore, at least in the high season, a well-developed public transport system. Guests expect the offer of public transport means at least every 30 minutes, and it should tend to be more frequent. For 75 % of holiday maker this is a must to imagine car-free holiday (see Figure 2).

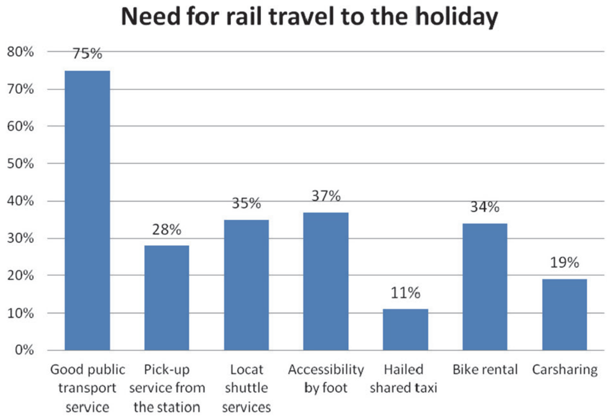

Figure 2 Needs from the perspective of holiday makers to be willing to travel by train, Source: Wanner A.

As an additional service, local car rental or car sharing services are considered important. These increase above all the subjective security of being able to have a vehicle available at your destination at any time, even without your own car. In most cases, these offers are then not even taken up, but the knowledge of being able to borrow a car at any time increases the willingness to arrive without an own car. If such offers exist, they must be easy and uncomplicated to use. Car sharing offers are also expected to be able to use the same systems as at home. For 19 % a car sharing offer is required (see Figure 2).

3.3 One-stop-shop

The car has the decisive advantage that people at the home address load their luggage, get into the car, are guided to their destination using modern navigation systems and unload their luggage there. Travellers do not need to worry about anything else, even necessary steps such as refuelling or paying tolls are internationally standardised.

If you are travelling by train, there are many more steps to be taken to obtain information and booking. Travellers have to obtain information about the travel options, then have to obtain the tickets and possibly reservations. Often several means of transport or operators have to be used, which means that separate ticket purchases are necessary. Taxis are often available for the last mile, which often must also be reserved in advance. This apparent complexity is another reason why many holidaymakers prefer to use their own car for the journey, which is why simple information and booking systems are required, where all information and travel documents can be obtained from one source.

However, the easytravel project has made it clear that it is practically impossible to develop an application or a website where all the information for a holiday trip and all the relevant booking options can be obtained “from a single source”. The reason is that the interests of the various players are too diverse and too complex for the necessary data interfaces. It has become clear, however, that in sum most of the information is already available on the web, but it is often difficult to find. In any case, it makes sense to at least strive for good cross linking, e.g. the homepage of a tourism region should provide all relevant information on travel options including bookings and on local mobility. This requires a good and clear bundling of information. Despite conflicting interests of various stakeholders, it makes sense to actively integrate platforms such as Google-Maps or Booking.com, as the majority of holidaymakers are used to and use these systems. If the local public transport offer is not or only to a very limited extent shown in a timetable query via Google Maps, the impression is created that there is a corresponding undersupply of public transport offers, which entails clear disadvantages for the location.

4 Conclusion

For global and local environmental protection reasons, the aim is to shift as much mobility as possible to sustainable transport systems such as rail. This also applies to tourist transport. However, especially when travelling on holiday, there are many challenges for travellers which make it particularly difficult to travel by train. These challenges are intensified by the fact that families often travel with children and have a correspondingly higher amount of luggage with them for the holiday. Despite the advantages of a train journey compared to a car journey, such as relaxed travelling, especially with children, or the usability of the travel time, the subjectively perceived disadvantages outweigh the disadvantages, which arise in particular due to the taking along of the sometimes extensive holiday luggage combined with necessary transfer processes.

Particularly in summer, the concern about insufficient mobility at the holiday destination is an important reason for choosing not to travel by train but to use your own car. Nevertheless, many people are willing to use the railways for holiday trips. Younger people from urban regions with very good public transport systems in particular often do not own a car themselves. However, this group is also "spoiled" by the good offers they are used to at home. The flexibility offered at home by very dense intervals on public transport combined with various car-sharing offers is also expected at holiday destinations. Uncomplicated travel by rail and the best possible flexible mobility at the holiday destination are thus increasingly important location factors for holiday regions. In order to increase the willingness to travel by rail in holiday regions, the following points are of particular importance:

- Uncomplicate procurement of information and simple ticketing: For this purpose, all relevant information for travel and local mobility must be available centrally and must also be displayed in full. Even if many operators do not want to give up their data, it seems to make sense to provide systems like Google Maps with all relevant information, as these systems are used by many people. If timetables are not available there, the impression is created that there is a lack of public transport in the holiday destination, which discourages people who want to travel by train from travelling to this region. In parallel with the provision of information, it must be possible to book the entire journey without complications, including taxis for the first and last mile and any luggage transport.
- Sufficient local mobility: An important reason for not taking the train on holiday is the concern about insufficient mobility at the holiday destination. Especially in summer, holidaymakers also want to go on excursions, which is why the radius of action is greater than in winter. Well developed public transport systems with at least one continuous 30-minute interval and operating times even at off-peak times are seen as a minimum requirement. In addition, rental cars or car sharing are a good complementary offer, as people who are willing not to travel by car know that they can always fall back on a car locally. As a rule, rental cars are then not or rearly used locally, but the knowledge that they can fall back on a car at any time is an important incentive for travelling by train.
- Luggage transport: A major reason for choosing a car is the need to carry luggage on holiday trips. To free travellers from bulky luggage, it is important to offer door-to-door luggage services. To ensure that these are ultimately accepted and contribute to the desired use of the railway, price and flexibility are the main factors to consider. Currently there are large time slots for the collection and delivery of luggage. Important for the attractiveness of baggage services are short time slots for collection and delivery of a maximum of three hours as well as a service also in the morning and evening. In general, drop-off and pick-up terminals or partner shops are also considered interesting, to which travellers can bring their luggage at any time, for example the day before, and are therefore flexible in terms of timing.

The results presented in this paper originate from the research project easytravel, which is funded by the Ministry for Climate Protection in Austria within the framework of the call for proposals “Mobility of the Future”.

References

[1] Final project report project, Easytravel, 2018

[2] Seltenhammer, K.: KundInnenanforderungen an Gepäckserviceleistungen im Urlaubsreiseverkehr, (engl. Customer requirements for baggage services in holiday travel), Diploma thesis, TU-Vienna, Institute of Transportation, 2017.

[3] Dietrich, B.: Bahnanreise im Urlaubsverkehr - Anforderungen an die Mobilitätskette aus Sicht von Familien (eng. Rail travel in holiday traffic - demands on the mobility chain from the perspective of families), Diploma thesis, TU-Vienna, Institute of Transportation, 2017.

[4] Coudek, E.: One Stop Shop Buchungen im Reiseverkehr (engl: One stop shop bookings in travel), Diploma thesis, FH-St.Pölten, Department of Rail Technology and Mobility, 2017.

[5] Feiel, S.: Anforderungen an den Gepäcktransport entlang der Reisekette (eng. Requirements for luggage transport along the travel chain), Diploma thesis, FH-St.Pölten, Department of Rail Technology and Mobility, 2017.

[6] Rüger, B.: Reisegepäck im Eisenbahnverkehr (eng. Baggage in rail transport), PhD-thesis, TU-Vienna, 2004.

[7] Wanner, A.: Sanfte Mobilität - Vor Ort Mobilität für einen autofreien Urlaub (eng. Local mobility promoting a car-free holiday), master thesis, Vienna University, 2017.

IMPACT OF LIBERALIZATION ON THE TAXI MARKET IN THE REPUBLIC OF CROATIA

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Abstract

Market liberalization is an important objective of the European Union. It is a process of removing government restrictions and opening up markets for private companies. Road Transport Act which entered into force on 12 May 2018, enabled a completely free market for taxi services on the territory of the Republic of Croatia. Local self-government units, by this Law, are obliged to enable the provision of taxi services in their administrative area to all business entities that are eligible for taxi transportation. The paper will present a cross-section of the impact of liberalization on the taxi market in the cities of the European Union, as well as analyse specific features in the legal regulations of individual EU member states. The analysis will be made in order to better understand the new situation in the Republic of Croatia. The purpose of this paper is to determine the effects of liberalization of the taxi service market, and to what extent auto-taxi transportation contributes to the congestion of urban roads in Croatian cities. The aim of the paper is to analyse the structure of traffic flow, the share of taxi vehicles in traffic flow, and to draw guidelines and conclusions based on the analysis.

Keywords: liberalization, taxi transport, congestion, urban area, traffic flow

1 Introduction

Urban mobility is a global problem for cities and is one of the key themes of the transport sector in the 21st century. In its transport strategy, the European Union prioritizes urban mobility in the context of a sustainable transport system. Cities in the Republic of Croatia also face the issue of transport system sustainability. This is especially pronounced in Adriatic cities, where the growing number of tourists and seasonal visits during the year result in a high traffic loads on the transport network. The harmonization of the Law on Road Transport with EU regulations has resulted, among other provisions, in the liberalization of taxi transport [1]. The monograph "Uber-Brave New Service or Unfair Competition" analyses the issue of market liberalization in the Republic of Croatia. The authors state that "the legislator justified the introduction of full liberalization with several grounds, such as inadequate regulation of certain services (namely, taxi service and rent-a-car services), appearance of new business model in the market, and, issues with regard the supply-demand nexus during the tourist season (particularly in the coastal areas)" [2] .
Gwilliam in his paper "Regulation of Taxi Markets in Developing Countries" presents the three most common approaches to the regulation of the taxi market, and they are: quantity of supply (specified in terms of the number of operators or number of vehicles); quality of supply (including the quality of the vehicle, the financial capability of the operator, the competence and trustworthiness of the driver, and sometimes the efficiency of the dispatching arrangements); and fares (either in terms of fixed or maximum tariff schedules) [3].

Rainstra et al., in the paper “International comparison of taxi regulations and Uber” presents the Dutch approach to the liberalization of the taxi market. Liberalization has allowed taxi drivers to operate freely in the whole country. However, as a regulatory measure, local administrative units have been enabled to implement separate models that ensure the desired quality of service [4].
Barrett in his paper “Regulatory capture, property rights and taxi deregulation: A case study” compares the process of gradual liberalization and deregulation of the market It was concluded that gradual liberalization schemes are much less radical in terms of new market entry, compared to deregulation. However, according to the experience in Ireland, they state that there should be full and immediate deregulation rather than a measure of liberalization of taxi markets. Large reductions in passenger waiting times have made deregulation popular among the public. The authors also find that the new regulation did not result in a reduction in either driver or vehicle standards [5].

2 An overview of the impact of taxi market liberalization in EU members

This chapter is based on the data presented in the “Study on passenger transport by taxi, hire car with driver and ridesharing in the EU” [6], presented in 2016 in Brussels, by the European Commission. The study provides a detailed overview of regulatory policies implemented in EU member states, following the introduction of liberalization of the taxi services market.
In all EU Member States, access to taxi services is subject to licensing. The term “license” also includes the terms “authorizations”, “concessions” and “permits” used in different Member States. All these conditions refer to the administrative approval for performing the activities of a taxi service. The conditions for obtaining such licenses vary from country to country. In most EU Member States, license requirements are set at national level while local authorities set their own requirements and control access to their local markets.
Restrictions on market entry are usually motivated by the oversupply of relatively unskilled workers available to the taxi industry, especially in times of economic coercion and the need to maintain public order by limiting the number of vehicles circling and / or parking on the streets. With the exception of Austria, the Czech Republic, Estonia, Hungary, Latvia, Lithuania, the Netherlands, Poland, Slovenia, Slovakia and Sweden (and the cities of Berlin, Hamburg, Sofia and London), other EU Member States have introduced quantitative restrictions based on socio-economic criteria such as population, number of tourists and business travellers. The taxi market is also geographically fragmented because licenses are usually valid only for the area of the municipalities that issue them, while in the Netherlands, Sweden, Slovakia and Luxembourg it is the practice that a valid license allows taxi services to be provided throughout the country [6]. Regarding the approach to the restriction of taxi activity, two groups have been created in the Member States [6]:

- Member States with quantitative restrictions;
- Member States without quantitative restrictions.

In the first group, the number of taxi licenses has remained stable or decreased over the last few years, with the exception of Germany, where the number of licenses decreased between 2008 and 2012, but the number of taxi vehicles increased. In Belgium, the total number of licenses decreased between 2014 and 2016, as well as in Spain during the period 2010-2015, while in Cyprus, Italy, Luxembourg and Malta the total number remained stable [6].
A small increase was recorded in France between 2003 and 2015, where the number of taxi licenses increased by 2065. In England, the number of taxi licenses increased by 1.5 % between 2013 and 2015, but the number of taxi drivers decreased by 0.9 % [6].

In the second group, Austria and Poland recorded a constant annual increase. In Sweden, indirect barriers (i.e. a particularly high level of financial standing) could contribute to maintaining supply at a certain level without too much change. According to available information, the number of permits has increased since the introduction of liberalization but has remained constant over the years. In the Netherlands, the number of taxi companies (business licenses) decreased in 2015, while the number of drivers (individual licenses) increased from 2014 to 2015. In Ireland, liberalization has resulted in a reduction in vehicles and drivers, and remains second only to Sweden in the ratio of taxi drivers to population [6]:

- 75 drivers per 10,000 inhabitants in Sweden;
- 59 drivers per 10,000 inhabitants in Ireland;
- 40 drivers per 10,000 inhabitants in Romania.

While this ratio is quite low in [6]:

- 4.7 drivers per 10,000 inhabitants in Italy;
- 2.3 drivers per 10,000 inhabitants in France;
- 2.0 drivers per 10,000 inhabitants in Hungary.

Most EU Member States give local authorities the power to regulate the number of licenses issued, except for countries such as Slovakia, Sweden and Luxembourg (after July 2016) where the Ministry of Transport issues taxi licenses at the national level. Local authorities limit the number of licenses to various formulas according to the effective needs of cities / municipalities. In general, various criteria are taken into account, such as the number of inhabitants, the existence of an airport and the number of passengers to / from railway stations. In some Member States, a person may have more than one license. In Belgium, each of the three autonomous regions sets the maximum number of taxis at regional level [6]. Among the Member States that limit the number of permits, some capitals or major cities (Sofia, Berlin, London) do not impose a maximum ceiling [6]. Quantitative restrictions have not been set in Austria, Hungary, Ireland, Lithuania, the Netherlands, Poland, Slovenia, Slovakia and Sweden [6].

3 Research on the impact of liberalization in urban areas of the Republic of Croatia

Two cities, Split and Dubrovnik, were selected for the purpose of researching the impact of taxi market liberalization on the territory of the Republic of Croatia. Both cities are tourist centres, and they are considered to have a much more pronounced impact of liberalization, with the exception of the capital Zagreb.

3.1 Traffic flow analysis

In the study of traffic flow structure, the method of counting was used. Counting is a methodological procedure that determines the number of elements or members of a set. In this case the elements or members of a set are defined groups of vehicles. [1]
Consequently, the traffic flow structure is divided into seven groups of vehicles, namely: City bus – public transport; Tourist bus; Passenger car; Taxi; Motorcycle / moped; Van (cargo and passenger); Heavy vehicle.. [1] [7] [8].
The counting was done manually at the cross sections of selected roads. In total, ten counting locations were selected, six in the city of Split, and four locations in Dubrovnik. Counting was done in the morning and afternoon peak periods. Complete data can be accessed in studies [1] [7] [8]. Traffic counting was conducted in two periods of the day, the morning and afternoon peak periods. Figure 1 and Figure 2 show the share of each vehicle type in the total traffic flow for the city of Dubrovnik and Split during the observation period.

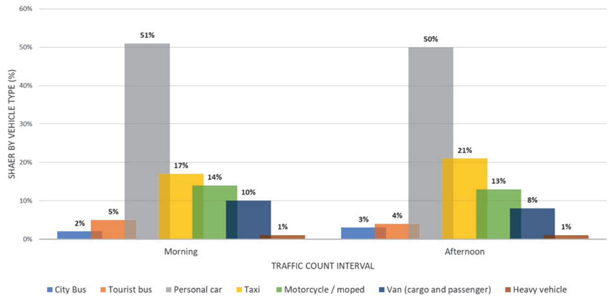

Figure 1 Share of vehicles by type, City of Dubrovnik [1] [7]

According to data obtained from a field research in the City of Dubrovnik (Figure 1), passenger cars account for the largest share of vehicles in traffic, with a share of 51 % in the morning and 50 % in the afternoon periods. This is followed by the share of taxi vehicles with 17 % in the morning, and 21 % in the afternoon periods. Of other vehicles, motorcycles / mopeds have a higher share in the total traffic flow with a share of 14 % in the morning and 13 % in the afternoon, and vans with a share of 10 % of the total traffic flow in the morning and 8 % in the afternoon periods.

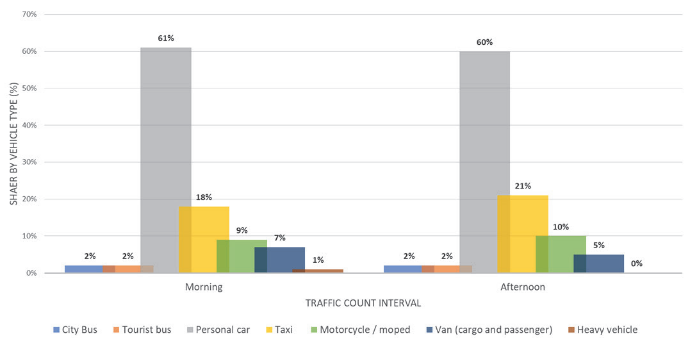

Figure 2 Share of vehicles by type, City of Split [1] [8]

According to data obtained from a field research in the City of Split (Figure 2), passenger cars account for the largest share of vehicles in traffic, with a share of 61 % in the morning and 60 % in the afternoon periods. This is followed by the share of taxi vehicles with 18 % in the morning, and 21 % in the afternoon periods. Of other vehicles, motorcycles / mopeds have a higher share in the total traffic flow with a share of 9 % in the morning and 10 % in the afternoon, and vans with a share of 7 % of the total traffic flow in the morning and 5 % in the afternoon periods.

The data presented in this paper show the average results of all observed locations, and there are considerable discrepancies for certain locations. There are locations where the traffic flow with large differences in proportions when it comes to taxis is recorded, thus highlighting the location of “Ulica Domovinskog rata” in Split, where at peak time the share of taxi vehicles was as high as 40 %, thus becoming the primary form of traffic in the city. There are also similar locations in Dubrovnik, an example of which is the “Pile” location, where the share of taxi vehicles in the peak hour reaches 34 % of the total traffic flow.

3.2 Analysis of auto-taxi operators

According to the National Register of Road Carriers [9] on December 6, 2019, there were a total of 4858 registered taxi drivers performing the activity of taxi transport in the territory of the Republic of Croatia. Out of that, 795 taxi carriers were registered in the Dubrovačko-neretvanska County, which represents 16.4 % of the total number of carriers. In the area of Splitsko-dalmatinska County, 867 taxi carriers are registered, which represents 17.8 % of the total number of carriers. In the City of Dubrovnik, the activity is performed by 349 carriers, which in relation to the county amounts to 43.9 %, and at the state level represents 7.2 %. In the City of Split, the activity is performed by 379 carriers, which in relation to the county amounts to 43.7 %, while at the state level it represents 7.8 %.
There are 211 taxi drivers operating in the City of Dubrovnik who obtained a license before the entry into force of the Road Transport Act [10] on 12 May 2018, which is 60.5 % of the total number of carriers in the City of Dubrovnik. These carriers have a total of 283 licensed vehicles, which is 58.5 % of the total number of licensed vehicles.
After the enactment of the cited Act [10] in the area of the City of Dubrovnik, by November 29, 2019, another 138 taxi carriers obtained a license, which is 39.5 % of the total number of carriers in the area of the City of Dubrovnik. These carriers have a total of 201 licensed vehicles, which is 41.5 % of the total number of licensed vehicles. Prior to the entry into force of the new Act [10], 169 taxi companies operated in the City of Split, which is 44.6 % of the total number of carriers in the City of Split. These carriers have a total of 309 licensed vehicles, which represents 51.3 % of the total number of licensed vehicles.
After the adoption of the cited Act [10] in the area of the City of Split, until November 29, 2019, another 210 taxi drivers obtained a license, which is 55.4 % of the total number of carriers in the area of the City of Split. These carriers have a total of 293 licensed vehicles, which represents 48.7 % of the total number of licensed vehicles.
According to the data obtained from the competent office, a total of 637 permits were issued (until 31.10.2019) in the City of Dubrovnik, and 759 in the City of Split. According to the collected data, it is possible to estimate the number of vehicles quite accurately, because according to the data of the competent ministry at the national level, the number of vehicles per carrier is 1.57. Approximately a similar ratio was obtained for both the City of Dubrovnik and the City of Split. In Dubrovnik, the ratio is 1.4, while in Split it is almost identical to the national average and is 1.6. It follows from the above that the estimated number of taxis in Dubrovnik is equal to 1000, while in Split 1192 vehicles.

4 Discussion

Guidelines for improving the management of transport demand in the area of local self-government units is a key factor in reducing the negative effects of the transport system. Reducing the negative effects together with increasing the benefits of the transport system of a area is the basis for achieving sustainable mobility [1]. The legislative and regulatory framework differs from one EU Member State to another due to different legal traditions and constitutional frameworks. In various Member States, taxi transport is considered to provide a service of public interest and is therefore part of integrated urban mobility. Traditionally, local governments have decided to regulate taxi services to ensure safe and predictable transportation services.
In terms of the overall legal approach, there are two different groups of Member States: on the one hand, those that have introduced entry restrictions, especially in terms of quantitative regulation, and on the other hand, those that have not introduced restrictions on the number of permits [6].

Analyses have demonstrated the need to reduce the use of personal motor vehicles and increase the use of sustainable modes of travel (walking, cycling and public transport). Taxi transport is considered one of the forms of public transport. Increasing the use of taxis reduces the number of passenger car journeys and reduces the required number of parking spaces. Taxi transport can have a positive impact on the transport system if it is an alternative to the use of personal vehicles, but it has a negative impact if it is an alternative to mass public passenger transport.
Transport policy in tourist centres can include various specific strategies to improve transport opportunities, integrate alternative transport for the purpose of tourist activities, and promote alternative modes of transport. These may include: improving transportation; organized tourist transport; taxi service improvement; improving the non-motorized way of traveling (walking and cycling); public bicycle system; bicycle parking infrastructure; parking policy management; traffic calming measures; measures to reduce car use in the city centre; promotion with a view to encouraging the use of alternative modes of transport; heavy cargo management; air traffic management [11].

5 Conclusion

In EU Member States, the legislative and regulatory framework differs due to different legal traditions and constitutional frameworks. Most Member States consider that taxi transport provides a service of public interest and is therefore part of integrated urban mobility.
In terms of the overall legal approach, there are two different groups of Member States: on the one hand, those that have introduced entry restrictions, especially in terms of quantitative regulation (license restrictions), and on the other hand, those that have not introduced a license limit. Analysing the approach of EU member states to the administrative regulation of taxi transport, it can be concluded that 12 out of 28 EU member states do not have quantitative limits on the number of taxi licenses.
The Road Transport Act, which was declared valid on 3 May 2018, enabled a completely free market for taxi transport in the Republic of Croatia. By this Act, local self-government units are obliged to enable the performance of taxi transport in their administrative area to all business entities that meet the conditions for performing taxi transport and have requested a permit.
Providing all eligible businesses that have requested a taxi license has created a demand-driven taxi system. Such a situation has led to an increase in business entities performing taxi transport, an increase in registered M1 category motor vehicles, and an increase in employees as motor vehicle drivers for taxi transport. The summer season has a significant impact when the demand for transport is high, taxi operators send their vehicles to other destinations (cities), in order to respond to increased demand, i.e. as soon as the demand for the service decreases, operators move vehicles to continental cities where demand is higher.
It can be stated that the average share of taxis in the traffic flow is significant, especially considering that the city centres of the analysed cities have previously had an unfavourable volume / capacity ratio, which contributes to traffic congestion in the centres of Split and Dubrovnik.
Future research should annually monitor the structure of traffic flow and the volume / capacity of the road, as one of the most important indicators of the traffic load on the city's transport network. Only on the basis of an in-depth analysis of the traffic network of a certain city can one conclude about the condition of the existing network, and about improvement measures, i.e. define strategies for the future traffic volume and the desired condition of the transport network.

References

[1] Studijska analiza gradskog prometa u gradovima Splitu i Dubrovniku s osvrtom na taksi prijevoz, University of Zagreb, Faculty of Transport and Traffic Sciences, Zagreb, Croatia, 2019.

[2] Marin, J., Petrović, S., Mudrić, M., Lisičar, H.: Uber-Brave New Service or Unfair Competition, Ius Gentium: Comparative Perspectives on Law and Justice, 76 (2019), Springer, Zagreb, Croatia

[3] Gwilliam, K.M.: Regulation of Taxi Markets in Developing Countries: Issues and Options, Transport Notes: Urban Transport Thematic Group, The World Bank, Washington, DC, 2005.

[4] Rienstra, S., Bakker, P., Visser J.: International comparison of taxi regulations and Uber, KiM Netherlands Institute for Transport Policy Analysis, Ministry of Infrastructure and the Environment, The Hague, Netherlands, 2015.

[5] Barrett, S.D.: Regulatory capture, property rights and taxi deregulation: A case study, Institute of Economic Affairs, Blackwell Publishing, Oxford, England, 2003.

[6] CERTeT Università Luigi Bocconi: Study on passenger transport by taxi, hire car with driver and ridesharing in the EU, European Commision, Brussels, Belgium, 2016.

[7] Istraživanje gradskog prometa u Gradu Dubrovniku, University of Zagreb, Faculty of Transport and Traffic Sciences, Zagreb, Croatia, 2019.

[8] Istraživanje gradskog prometa u Gradu Splitu, University of Zagreb, Faculty of Transport and Traffic Sciences, Zagreb, Croatia, 2019.

[9] Ministry of the Sea, Transport and Infrastructure: https://nrcp.mmpi.hr/apex-/f?p=100:LOGIN:7196384065312, 16.12.2019.

[10] Zakon o prijevozu u cestovnom prometu, NN 41/18, 98/19, Narodne Novine, Official Gazette of the Republic of Croatia, Croatia, 2019.

[11] Victoria Transport Policy Institute, Tourist Transport Management, https://www.vtpi.org/tdm/tdm46.htm, 10.04.2020.

9
ROAD SUPERSTRUCTURE: TESTING AND MODELLING

RESILIENT PERFORMANCE OF EXPANSIVE SUBGRADES STABILIZED WITH NANOSIZED AND ACTIVATED FLY ASH

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Abstract

Subgrades across arid and semi-arid region are known for its random swelling, with high plasticity due to moisture infiltration of the pavement structures. Subgrades materials are significantly influenced by the cahnges in degree of saturation, which is unavoidable. Studies in the past, have reported several positive results on the stabilization of expansive soils with additives like lime, cement, fly ash, etc. In this study, resilient performance of expansive subgrades treated with 0.5 %, 1.0 %, 1.5 % and 2.0 % of nanosized and activated fly ash (NFA and AFA) is presented. Series of cation exchange capacity tests, zero swelling tests (ZST) and resilient modulus tests were performed to study the effects of NFA and AFA on resilient modulus () and swelling index of the subgrades material respectively. Scanning electron microscopy (SEM) tests was conducted to evaluate the morphological changes in the subgrades, and compounds responsible for resilient strength development. The result showed that, NFA and AFA inclusions in the treatment of expansive subgrades caused an increase in resilient strength and decrease in swelling stress to a limiting stabilizer content of 0.5 % and 1.0 % beyond which, the resilient modulus values increased triggering a significant decrease in swelling stress. The test result revealed that the reduction was caused by the pozzolanic reaction between the stabilizers and available moisture required for full completion of pozzolanic process. Based on the test result, nano-fly ash exhibite high potential in improving resilient strength and reducing swelling stress to 58.7 % and 63 % respectively on the average compared to activated fly ash. This study suggest a feasible solution to improve the quality and performance of expansive subgrades.

Keywords: nano fly ash, activated fly ash, subgrades, resilient modulus, pavements

1 Introduction

Expansive subgrades causes frequent fatigue to the pavement, due to volumetric changes. Moreover, in arid/semi-arid regions, moderate swelling could induce major damages to pavement structures (Aneke, Mostafa, and Moubarak 2018).
The ratio of cyclic deviator stress to the resilient strain is known as and it is one of the important parameters in flexible pavement design (Sun et al. 2016; Banerjee 2017). In general, subgrade tends to swell or shrink for a given change in moisture content (Jones and Jefferson 2012). A review of the available literature indicated that increases with increase in density, despite the fact is function of stiffness (Titi et al 2015). Though, is affected by many factors, i.e. moisture, the amount and type of the clay-size particles and the minerals contained in the subgrade soils, density, matric suction and applied stress level (Azam et al. 2012). To improve the performance of expansive subgrades, variety of treatment methods have been

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developed in the recent past. Essentially, traditional chemical stabilizers such as lime, cement and fly ash are utilized to control the swelling and enhance the soil stiffness (Pei et. al 2015, Zhang 2018). The utilization of nano-technology to improve is still very sketchy (Ng et al 2014).Thus, nano-particles may generate lower total cost in practical engineering (Ren and hu 2014). There are very few attempts in using nano-particle additive to treat subgrades (Tabarsa 2018). This study rationaly investigated the resiliennt permance of subgrades using NFA and AFA with respect to their morphological changes.

2 Material and experimental program

2.1 Soils

The subgrade samples were collected from three sites in South Africa. Specifically, Soils 1, 2 and 3 were selected from Bloemfontein, Winburg and Welkom, respectively. All soil samples were disturbed, as they were collected at a depth of 0.5 to 1.2 m below the ground level. Fig. 1 shows the grain size fractions.

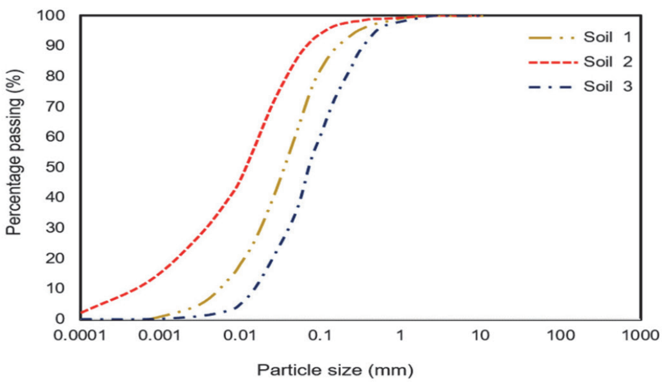

Figure 1 Soils grain-size curve

The collected subgrade materials are categorized as fat clay (CH) soils according to the Unified Soil Classification System(USCS). The cumulative percentages of the soils passing through ASTM sieve size of #200 vary between 50 % and 100 %. Table 1 present the mechanical properties of the investigated subgrade soils.

Table 1 Properties of soils

Number	Measurements results					Rail-to-earth conductance [S/km]			
						Based on standard		Based on Ohm's law	
	I1 [A]	I3 [A]	Ure1 [V]	Ure2 [V]	Ure3 [V]	G're 1	G're 2	G're 3	G're
1	256	291	3.5	5.4	8.9	0.077	0.418	0.561	4.424
2	248	286	4.0	5.9	9.3	0.082	0.375	0.564	4.474
3	458	536	6.6	10.2	16.5	0.075	0.377	0.540	5.173
4	323	369	3.5	6.0	10.3	0.080	0.376	0.547	5.136
5	349	398	3.9	6.6	11.3	0.077	0.368	0.554	5.021
6	251	294	4.4	6.3	9.8	0.075	0.407	0.540	4.684
7	461	528	7.8	11.5	18.0	0.063	0.398	0.535	4.062
8	439	507	4.3	7.7	13.7	0.063	0.390	0.542	5.903
9	459	531	4.6	8.1	14.3	0.067	0.385	0.547	5.925
10	570	666	6.1	10.6	18.5	0.059	0.393	0.542	6.100

*USCS: Unified Soil Classification System * S1, 2, and 3: Soil 1, 2 and 3 * P_s: Swelling stress

2.2 Fly ash and lime

The fly ash used herein are sampled from Lethabo power station in South Africa. The classification of this materials, were achieved in accordance with the standard specification for coal fly ash (ASTM C6 2018) .The classes of fly ash are class "C" and "F". fly ash. The chemical compositions of the soil, fly ash and lime used in this study are summarised in Table 2, as obtained from XRF test..

Table 2 Materials chemical composition

Chemical oxides	S 1 mass [%]	S 2 Mass [%]	S 3 Mass [%]	FA "C" mass [%]	FA "F" mass [%]	Lime Mass [%]
SiO_2	58.16	62.40	59.65	41.20	56.34	10.69
Al_2O_3	21.41	13.36	14.11	16	37.1	0.33
Fe_2O_3	12.09	4.32	12.34	6	1.95	0.39
CaO	1.75	0.67	2.11	26	3.69	78.88
LOI (%)	0.23	1.23	2.54	2.33	0.38	0.67
pH	7.12	6.78	6.35	10.13	9.67	11.54

3 Material preparations

3.1 Nano fly ash (NFA) and AFA preparations

The NFA was prepared using top down method, the bulk fly ash material was reduced into nanostructures by the means of mechanical processes i.e. mechanical ball-milling and grinding. The milling process was done using planetary ball milling and measured quantity of class "C' fly ash was subjected to milling for 18hrs. Subsequently, the surface area of the nanosized fly ash increased from 0.228 m^2/gm 28.40 m^2/gm. Whereas, the activated fly ash (AFA) was prepared in accordance with activated analytical procedure by (Aneke et. Al 2019).

3.2 Specimens preparation

The maximum dry densities and optimum moisture contents of subgrades were initially determined, by conducting standard Proctor compaction test according to (ASTM D698 2007). Subsequently, the prescribed NFA and AFA contents for the specimens were determined by the percentage dry mass of soil, given by Eq. (1). in a mould, having a volume of 23.05 x 10-4 m^3.

$$M_{stab} = M_s \cdot \beta \ (\%) \tag{1}$$

where is mass of stabilizers in kg, and is mass of the soil in kg and the is the percentage of stabilizers. The stabilizer dosages used herein were 0.5 %, 1 %, 1.5 % and 2 %, this percentages were selected to ensure easy mixing of soil with the stabilizer and also to provide full reactivity for pozzolanic reaction based on the pH results.

4 Experimental testing procedures

The experimental procedures followed to directly pursue the objective of this investigation are listed Table 3.

Table 3 Tests and specifications

Tests	Specifications
Cation exchange capacity	Indian standard 2720 XL
Zero swelling	Indian standard part 41
Repeated load triaxial	AASHTO T 307

5 Result and discussions

5.1 Effects of stabilizer inclusion on CEC

The variation of CEC with stabilizers inclusions at optimum moisture content is shown in Fig. 2. It is clearly noted that, CEC values of the soil increased as the percentage of NFA and AFA increases. The specimens stabilized with NFA, increased from initial CEC values of 118.12,120.32 and 42.51 meq/100 g to 286.24, 341 and 290 meq/100 g for soil 1, 2 and 3 respectively.

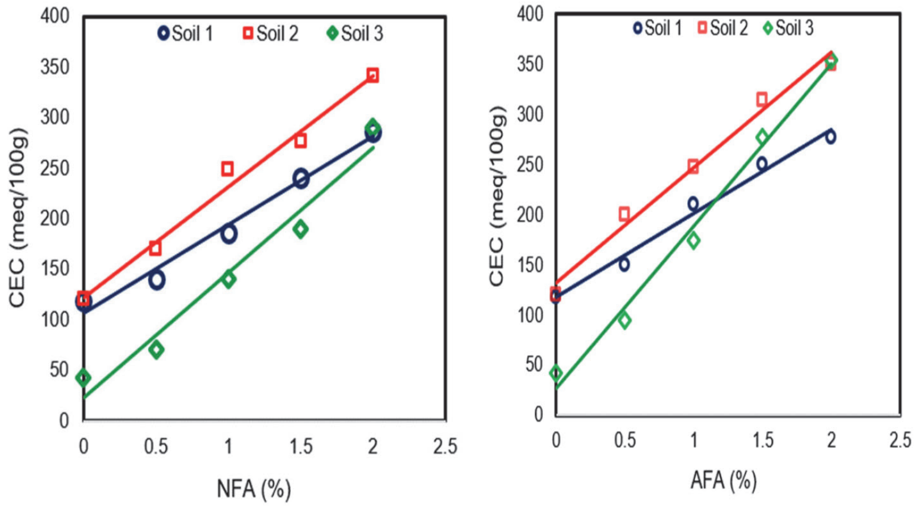

Figure 2 Variation of CEC with stabilizer contents

5.2 Effect of stabilizers inclusion on swelling stress

The variation of swelling stress with stabilizers inclusion is shown in Fig. 3. It can be interpreted from the curve that, the addition of NFA and AFA, demonstrate significant decrease in swelling stress values. It is evident that is not only relative to moisture content, thus void ratio and stiffness of the soil significantly influence the swelling behaviour of the subgrades. The initial values of Soil 1, 2 and 3 are 760 kPa, 850 kPa and 630 kPa. The result demonstrate drastic decrease in swelling stress with decrease in void ratio due to stabilizer inclusion. This imply that the dry density of the investigated soils increases, as the voids within the soil particles are filled with stabilizer.

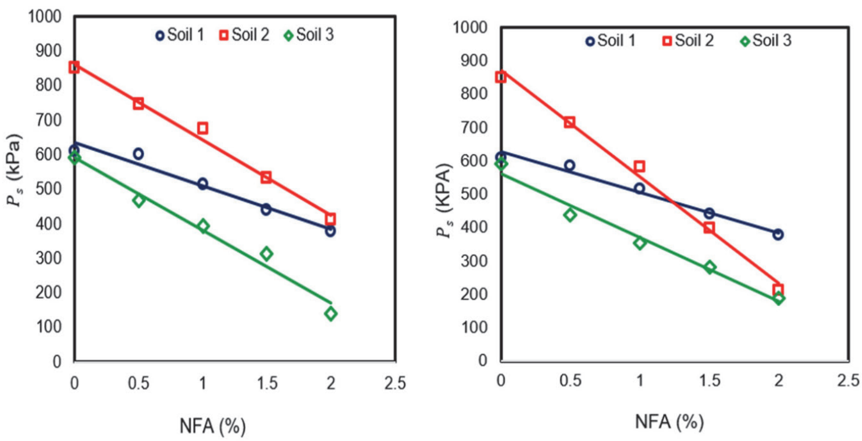

Figure 3 Variation of stabilizer contents on swelling stress

For NFA stabilized specimens, swelling stress is reduced by 63 % at stabilizer content of 1.5 % as compared to 47 % of AFA stabilized specimens with the same percentage. Generally, the NFA and AFA strikingly restrain the expansive activities of the investigated subgrades without with altering the mineralogical properties of the subgrades, as evidenced under SEM analysis (Judy et al 2016).

5.3 Effect of stabilizers inclusion on resilient modulus

The stress induced by traffic load, is represented by resilint response of subgrades under cyclic stress analysis. Figs. 4 and 5 presents the behaviour of unstabilized subgrades at OMC. It is evident that untreated subgrades exhibits considerable ductility, yielding low values with increasing deviatoric stress due to stress softening (Rahman and Tarefder 2015). It is noted that the differences in values due to change in confining pressures are small. The results revealed that average difference in values relative to deviator stress is ›4.41 % in all tested subgrade. It is observed that in at constant confining stress gradually decreased with an increase in deviator stress irrespective of subgrade type. The decreasing rate at the low deviator stress is more pronounced at high deviator stress. Soil 3 recorded the highest value, followed by soil 1 and 2. Based on the particle size distribution analysis, soil 3 is composed of 52.57 % amount of fines. Whereas, soil 1 and 2 content 75.52 % and 91.38 % amount of fine content respectively.

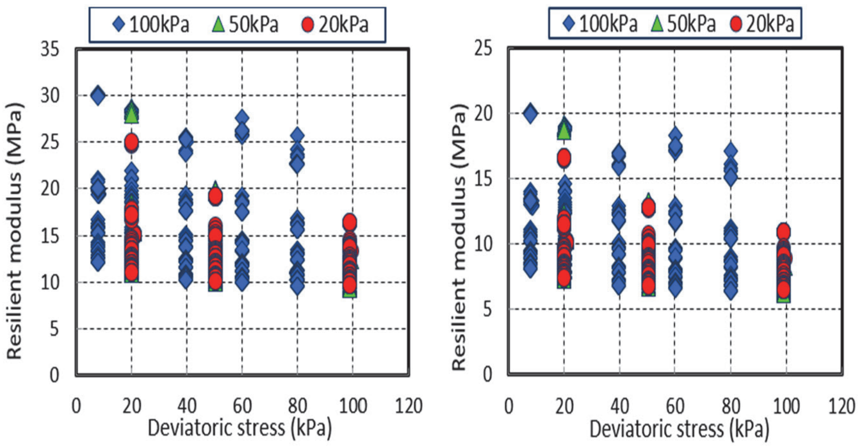

Figure 4 Soil 1 and 2 at varying deviatoric stress

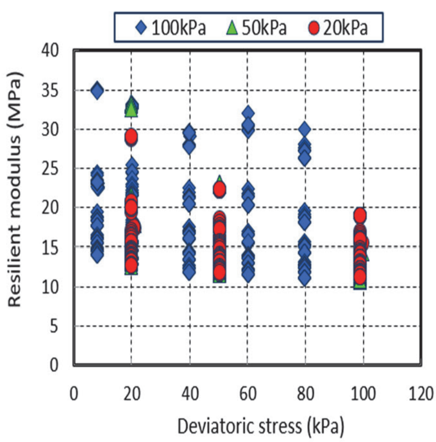

Figure 5 Soil 3 at varying deviatoric stress

Extensive tests were performed in the laboratory using the RLT machine on the stabilized soil specimens prepared at OMC with 1.5 % inclusion of stabilizer content. The variation of with stabilizer contents are shown in Figs. 6 through 9. It is indicated that increasing stabilizers content cause an increase in values. At stabilizer contents of 1.5 % for NFA improved to 46 %, 60 % and 70 % for soil 1, 2 and 3 respectively. The indicated behaviour of the nano-fly ash stabilized subgrade are in agreement with the findings reported elsewhere (Ozel 2001). The nonlinear trend of to deviator stress is similar to all treated subgrades and this result is in agreement the report published elsewhere by (Masada and Sargand 2002). based on curves slope, it can be inferred that the influence deviatoric stress is more exhibited at lower stress values irrespective of the confining stress level. The of the stabilized soils were less influenced by the deviator stress compared to unsatbilized soils. Thus, stabilized subgrades, decreases in values at low rate with deviator stress increase, which typically indicates strain hardening due to the stabilizer and denser state of grains soil particles (Maher et al. 2000).

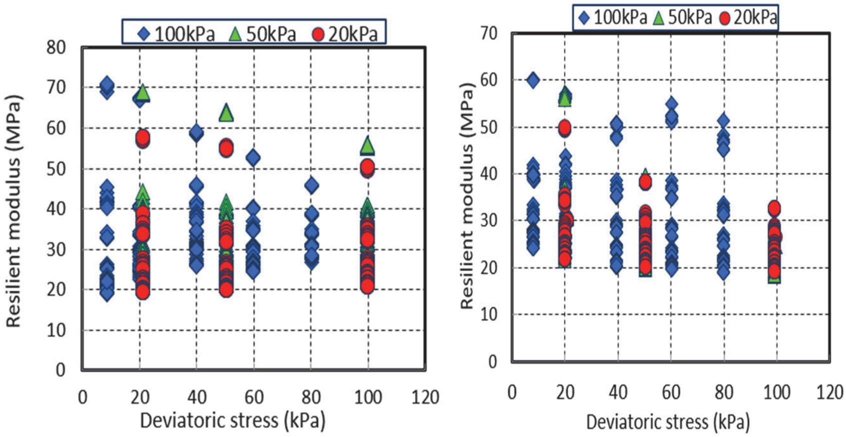

Figure 6 of 1.5 % NFA for soil 1 and 2 at varying deviatoric stress

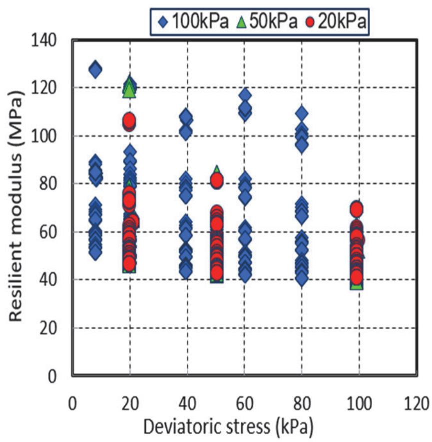

Figure 7 of 1.5 % NFA for soil 3 at varying deviatoric stress

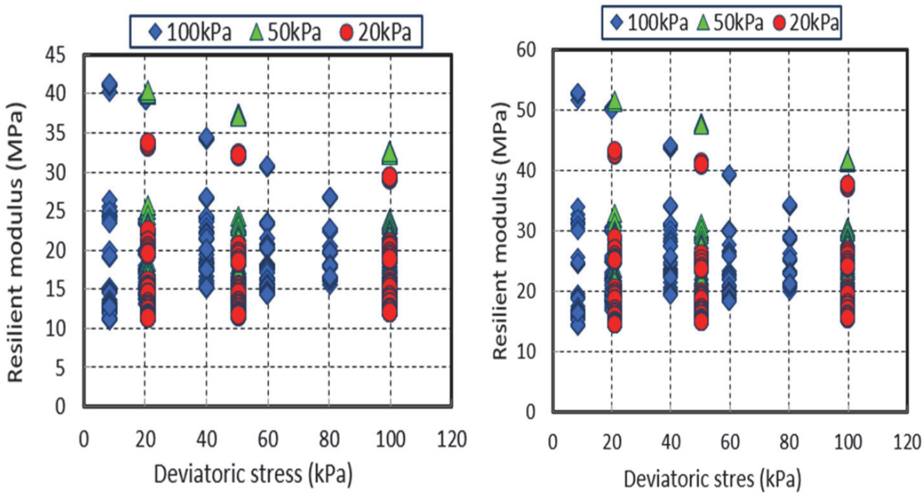

Figure 8 of 1.5 % AFA for soil 1 and 2 at varying deviatoric stress

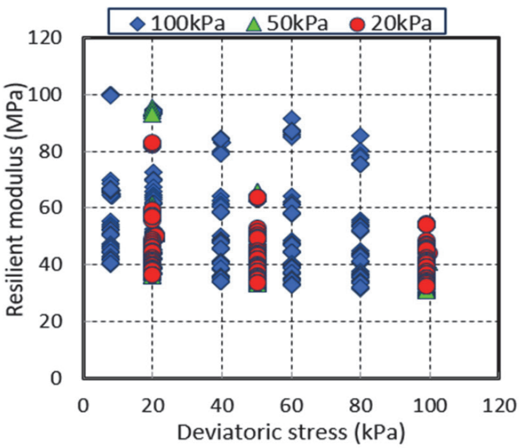

Figure 9 of 1.5 % AFA for soil 3 at varying deviatoric stress

5.4 Morphological analysis

The specimens stabilized with 1.5 % of NFA and AFA cured for 28 day period, using SEM apparatus VEGA3 TESCAN-6480 scanning electron microscope operated at 20kV are presented in Figs. 10 through 12. The specimens with 1.5 % stabilizer content were selected for the microstructure analysis, due to higher recorfed by specimens. The morphology were at magnification factor of 50µm. It is evidence that the inclusion of nano-fly ash and activated fly ash caused microstructural evolution within subgrade soils particle structures. The NFA and AFA inclusion, first produced an observed filled effect in pores between particles, which contribute to a decrease in porosity and an increase in the density of the treated subgrades. The addition of stabilizer contents propergated more clustered effect on the treated subgrades, which resulted in the formation of stronger knitted soil matrix. Whereas, the unstabilizer subgrades, appeared in a state of lose pack with larger pore.

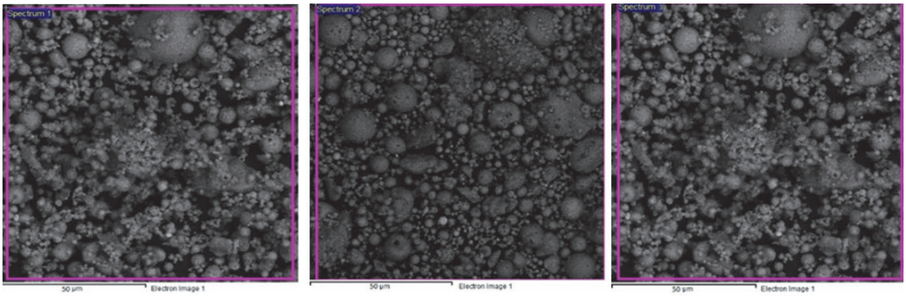

Figure 10 SEM of soil 1, 2 and 3

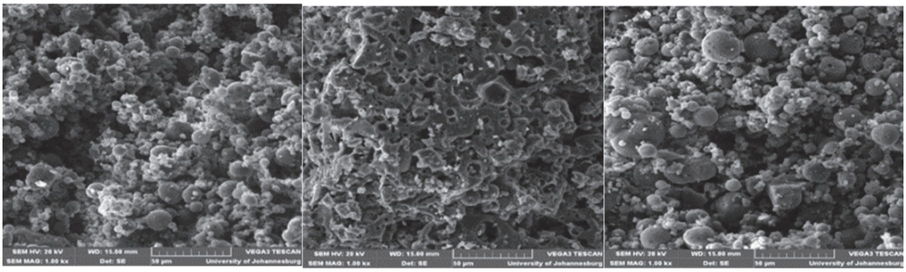

Figure 11 SEM of 1.5 % NFA stabilized specimens for soil 1, 2 and 3

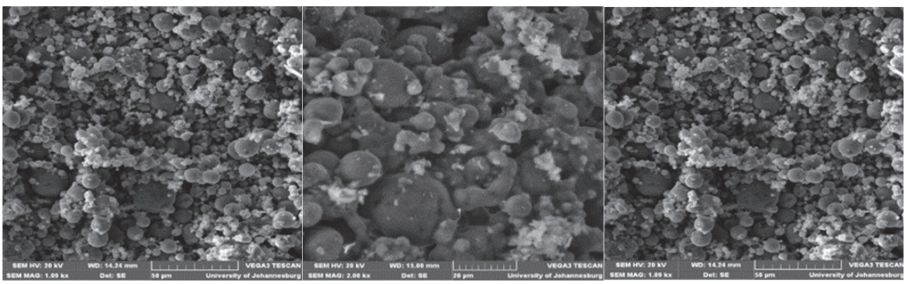

Figure 12 SEM of 1.5 % AFA stabilized specimens for soil 1, 2 and 3

Basically, micrograph of the stabilized soils depict hardened surface of pozzolanic paste. The reactive compounds of NFA and AFA compounds, such as C_3S, CASH and C_3A are more dominant on 1.5 % in NFA and AFA stabilized specimens respectively. From the above micrograph these chemicals responsible for morphological changes were traced within the capillary spaces of the stabilized soils. Based on the SEM micrograph, the pozzolanic reactions in the stabilized soil system, dissolved calcium ions from C_3S and it moves about freely in presences of moisture thus adsorbed within the surfaces of pozzolanic particles in soil. The formation of C–S–H formed by the pozzolanic effect of C_3S precipitate as the hydrates of high Ca/Si ratio on the surface of soil. The pozzolan reaction within the soil surface brings about gradual dissolution of Ca^+ causing Si and Al rich amorphous structure within soil pores. The stabilized soils with 1.5 % AFA formed predominantly amorphous phases of $3CaOSiO_2$, $CaSi_6O_{16}(OH)$, $3CaOAl_2O_3$, $3CaOAl_2O_3(OH)6$ gel within the soil pores compared with unstabilized soils.

6 Conclusions

Based on the acquired test results, the following key conclusions were drawn:

- The NFA and AFA inclusions reduced swelling stress of the stabilized soils due to an improved resistance of the specimens to swelling. An increase in stabilizer content caused increased CEC and pH properties.
- The stabilisers inclusion caused increase in for stabilizer content of 1.5 % below which reduced. The increase was attributed to the gain of stiffness of the stabilized soils due to low amount voids. The was improved by increasing level of confining pressure.
- Pozzolanic reactions were more dominant in NFA stabilized specimens, compare to AFA specimens, due to the nano size of NFA. This resulted in increasing values up to 58.67 % on the average compare to 45.32 % increase of specimens stabilized with AFA.
- Basically, this investigation revealed that NFA is more effective for stabilization compare to AFA. Though, specimens treated with both stabilizers significantly improvement the of the expansive subgrades among other geotechnical properties soil. This is expected due to the specific surface area, nano-size and higher reactivity of NFA.
- As expected, strain-softening behavior were observed as the subgrades recorded higher resilient moduli for higher confining stresses and decreased with an increase in the deviatoric stress under identical confinement.

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Reference

[1] Aneke, F.I., Mostafa, M.H., Moubarak., A.: Behaviour of Unsaturated Soils for Road Pavement Structure under Cyclic Loading, PhD Thesis, Free State, South Africa: Department Of Civil Engineering and Information Technology Central University of Technology, 2018.

[2] Aneke, F.I., Mostafa, M.H., Moubarak., A.: Resilient modulus and microstructure of unsaturated expansive subgrade stabilized with activated fly ash, International Journal of Geotechnical Engineering, 2019, doi: 10.1080/19386362.2019.1656919

[3] Azam, M.A., Cameron, A.D., Rahman, M.M.: Model for prediction of resilient modulus incorporating matric suction for recycled unbound granular materials, Canadian Geotechnical Journal, 50 (2012), pp. 1143– 1158, doi: 10.1139/cgj-2012-0406

[4] Banerjee, A.: Response of unsaturated soils under monotonic and dynamic loading over moderate suction states, Doctoral Dissertation, University of Texas at Arlington, Arlington, Texas, 2017.

[5] Jones, L.D., Jefferson, I.: Expansive Soils, ICE Publishing, New York, 2012.

[6] Judy, J., Kirby, J., McLaughlin, M., Cavagnaro, T., Bertsch, P.: Gold Nanomaterial Uptake from Soil Is Not Increased by Arbuscular Mycorrhizal Colonization of Solanum Lycopersicum (Tomato), Nanomaterials, 6 (2016) 4, pp. 1-9, 2016.

[7] Maher, A., Bennert, T., Gucunski, N.: Resilient Modulus Properties of New Jersey Subgrade Soils, Final Report, The State of New Jersey, Department of Transportation, 2000.

[8] Masada, T., Sargand, S.M.: Laboratory Characterization of Materials and Data Management for Ohio-SHRP Projects (U.S. 23), Job No. 14695(0), Final Report for Ohio Department of Transportation and Federal Highway Administration, Ohio University, Athens, Ohio 2002.

[9] Ng, C.W.W., Coo, J.L.: Hydraulic conductivity of clay mixed with nanomaterials, Canadian Geotechnical Journal, 52 (2014), pp. 808–811, doi: https://doi.org/10.1139/cgj-2014-0313

[10] Ozel, M.R., Mohajerani, A.: Resilient modulus of a stabilised fine-grained subgrade soil, Australian Geomechanics Journal, 36 (2001) 3, pp. 75-86

[11] Pei, X., Zhang, F., Wu, W., Liang, S.: Physicochemical and index properties of loess stabilized with lime and fly ash piles, Applied Clay Science, 114 (2015), pp. 77–84, doi: https://doi.org/10.1016/j.clay.2015.05.007

[12] Rahman, M.T., Tarefder, R.A.: Assessment of moulding moisture and suction on resilient modulus of lime stabilized clayey subgrade soils, Geotechnical Testing Journal, 38 (2015) 6, pp. 840-850, doi: https://doi.org/10.1520/GTJ20140237

[13] Ren, X., Hu, K.: Effect of nano silica on the physical and mechanical properties of silty clay, Nanoscience andNanotechnology Letters, 6 (2014) 11, pp. 1010–1013, doi: 10.1166/nnl.2014.1857

[14] Sun, X., Han, J., Crippen, L., Corey, R.: Back-calculation of resilient modulus and prediction of permanent deformation for fine-grained subgrade under cyclic loading, Journal of Materials in Civil Engineering, 29 (2017), 5, pp. 04016284-1- 04016284-10

[15] Titi, H.H., English, R., Faheem, A.: Resilient Modulus of Fine-Grained Soils for Mechanistic Empirical Pavement Design, Transportation Research Record, 2510 (2015), pp. 24-35, doi: 10.3141/2510-04

[16] Zhang, F., Pei, X., Yan, X.: Physicochemical and mechanical properties of lime-treated loess, Geotechnical and Geological Engineering, 36 (2018) 1, pp. 685–696, doi: 10.1007/s10706-017-0341-6

[17] Tabarsa, A., Latifi, N., Meehan, C.L., Manahiloh, K.N.: Laboratory investigation and field evaluation ofloess improvement using nano-clay - A sustainable material for construction, Construction and Building Materials, 158 (2018), pp. 454–463, doi: 10.1016/j.conbuildmat.2017.09.096

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USAGE OF WOOD ASH IN STABILIZATION OF UNBOUND PAVEMENT LAYERS

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Abstract

Unbound base layers are an important part of the pavement, which consume a considerable amount of granular stone material. Favorable grain size distribution of materials, necessary for the construction of unbound base layers, is achieved by selecting a suitable material from nature or favorable material composition is achieved through mechanical stabilization. The basic principle of mechanical material stabilization is the addition of finer or larger granular material to the material of unfavorable, uniform granulometric composition, which is inherently unstable. In designing economical pavement structures, the availability of local materials of natural or industrial origin is of great importance.
From natural local materials for the road construction in the area of Slavonia and Baranja, the Drava sand, a material of uniform granulometric composition of medium grain size D_{50} = 0.3 mm, is interesting and often in use. With the increasing number of biomass power plants in eastern Croatia, waste local material, wood ash, is also being created. Wood biomass ash, which is generated as a residue of biomass burning for electricity and heat production, is one of the newer and less explored alternative materials, which finds its application in construction as evidenced by the results of previous foreign studies. One possibility of using wood bio ashes in mixtures for unbound base layers is to modify / repair the granulometric composition of the base material. By combining the aforementioned local materials, that is, by designing a mixture of Drava sand and wood ash in appropriate proportions, it would be possible to obtain a mechanically stable mixture of increased load-bearing capacity for construction of unbound base layers.
In this paper testing of mixtures composed from different proportions of Drava sand and wood ash for unbound base layers is described with the purpose of proving the stabilizing effect of wood ash.

Keywords: unbound base layer; stabilizing effect; wood ash; Drava sand

1 Introduction

Unbound base layers are an important part of the pavement, which consume a considerable amount of granular stone material. They have a role to take static and dynamic traffic loads and transfer them to the lower structure of the road, without exceeding the bearing capacity and without causing harmful deformations of subgrade. These layers with their mechanical properties must respond to the stresses from traffic load and meet all the durability requirements during the design period of the pavement structure, while also possibly serving as a temporary pavement on the construction site before the completion of the upper layers of the structure.

In this regard, the granulometric composition of the granular material, which must meet certain requirements and its compaction, is particularly important. In conditions in which granular materials are compacted (optimal moisture content), a certain compaction is achieved, and depending on the thickness of the layer and the quality of the subgrade, a certain bearing capacity of the unbound base course. A favourable granulometric composition is achieved by choosing the right material in nature or by designing a granular material composition for optimal mechanical stability. The basic principle of mechanical stabilization of granular material is adding smaller or larger granular material to the one with an unfavourable, uniform granulometric composition.
When designing the pavement structure and selecting materials for construction, in addition to their mechanical properties, economy, sustainability and environmental impact should be taken into account. Availability of local materials of natural or industrial origin plays a big role in the sustainable and economical design of pavement constructions [1].
The area of Eastern Croatia lacks gravel and stone materials needed for road construction [2]. Because of that, and the need for more rational pavement constructions, large amounts of local materials like river and dug sand are used in road constructions. Sand from local rivers of Drava, Sava and Dunav, and sand dug from the surroundings of Valpovo, Đakovo and Slatina is used for the completion of unbound and bound base layers of pavement constructions. Encouragement of more rational road constructions in this area began in the 1980s, and many pavement constructions which contain sand proved that it is of quality and a favorable local material fit for use in their construction [3].
Of all the mentioned sands in the area of eastern Croatia, the Drava sand is especially often used in the construction of bearing layers of different traffic areas. Experiences from previous applications of Drava sand as a mechanically compacted and cement-stabilized material are good, and this is also the reason why Drava sand was chosen as the basic granular material in this research.
With the increase in the number of biomass power plants in eastern Croatia, the local waste material, wood ash, is being accumulated. Ash from wood biomass, which is formed as a residue from the combustion of biomass for the production of electricity and heat, is one of the newer and less researched alternative materials, and it can find application in construction as confirmed by previous research [4], [5]. One of the possible uses of wood bioash is in modification of granulometric composition of basic granular material, which results in a mechanically stabilized mixture used for the production of base layers [6], [7], [8]. In continuation of this paper, the examination of mixtures composed of Drava sand and wood bioash in different percentages will be described. The goal is to determine the stabilizing effect of wood ash.

2 Experimental part

2.1 Materials

2.1.1 Drava sand

Drava sand is a material of grayish-brown colour, a uniform granulometric composition with the medium grain size D_{50} = 0.3 mm. The degree of Drava sand unevenness totals is . Usually, the degree of unevenness represents a measure of good workability of a material, which is considered good for construction if U›5 [9].
The determination of Drava sand's granulometric composition was conducted on 500 g samples according to the norm HRN EN ISO 17892-4 [10], on a mechanical vibratory table with a set of sieves with openings ranging from 31,5 to 0,063 mm. The granulometric composition of Drava sand (mix 1) is visible on figure 1.

2.1.2 Wood ash

Wood ash is a complex mixture of organic and inorganic structure. It contains a large number of compounds whose composition can significantly vary. The amounts, quality and physical and chemical properties of wood ash depend on the type of burned biomass part, mineral impurity content, location of biomass growth, way of its collection and processing and the technology and temperature of burning. Wood ash in mixtures can be used in two ways: (1) as a binding component, when with its addition certain chemical reactions are initiated as a result of pozzolanic activity (indirect way of ash application) (2) as a filler, when it is necessary to improve the physical properties of mixtures by increasing the percentage of finer particles (direct way of ash application). The chemical composition of wood ash is important for initiating chemical reactions in a mixture, while its granulometric composition is important for the improvement of physical properties in mixtures. The elementary composition of wood ash depends on the type of biomass and part that is being burned, ground type, climate and anorganic composition. The mineral structure depends on the method of combustion [4].
During the combustion of wood biomass, three different fractions of ash are formed: bottom ash, cyclonic fly ash and electrostatic fly ash. For the purposes of this test, wood bottom ash was collected, which is collected under the boiler of grate furnace in the bioenergy plant "Strizivojna Hrast d.o.o." Strizivojna. „Strizivojna Hrast d.o.o." is a company located in Eastern Croatia which processes wood and produces hardwood floors and other wood products. The aforementioned company is the first one in Croatia that operated a cogeneration plant in 2011 for the production of electrical and thermal energy based on wood biomass combustion [4]. Particle size analysis of wood ash was conducted according to the norm EN 933-10 [11] used for particle size determination of filler aggregate by air jet sieving.
Chemical composition of the wood ash is shown by mass portion of individual components (mass. %): MgO=2.645, Al_2O_3=0.828, SiO_2=3.486, P_2O_5=2.346, SO_3=1.408, K_2O=7.134, CaO=43.7, Fe_2O_3=0,685. The analysis of x-ray diffraction showed that the main components of ash are calcite, quartz, CaO and, in a smaller amount, portlandite ($Ca(OH)_2$) [12].

2.2 Mixture composition and process of making

For the needs of this research, mixtures of Drava sand and wood ash of different composition were assembled. Their granulometric graphs are shown on figure 1.
- Mixture 1 (control): 100 % sand
- Mixture 2: 90 % sand and 10 % wood ash
- Mixture 3: 80 % sand and 20 % wood ash
- Mixture 4: 70 % sand and 30 % wood ash
- Mixture 5: 50 % sand and 50 % wood ash
- Mixture 6: 25 % sand and 75 % wood ash

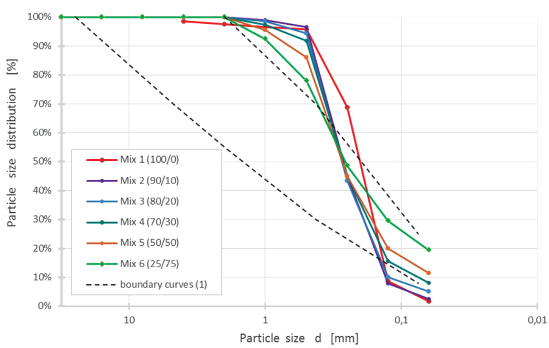

Figure 1 Particle size distribution for all mixes

From the granulometric curves it can be seen that the appearance of the curve changes significantly with the addition of a higher proportion of wood ash (especially mixture 6), the curve becomes “flatter” and approaches the limit curves [1] defined for mechanically stabilized mixtures for unbound base courses.
According to the research plan for each mixture an optimal water content and maximum dry density were determined by modified Proctor test for every mixture in order to prepare the samples for California bearing ratio (CBR) test.
The modified Proctor experiment was conducted according to the norm HRN EN 13286–2 [13]. For the purpose of the research, a Proctor’s cylindrical mold A with a 100 mm diameter and 120 mm height was used. Five layers of samples were compacted with the appropriate energy (2,7 MJ/m^3) in automatic Proctor device.
Table 1. shows the result values of Proctor’s elements (optimal water content w and maximum dry density g_d). These were used to create samples for the research of California bearing ratio CBR.

Table 1 Proctor’s elements for sample preparation

Mix No.	g_d [g/cm^3]	w [%]	Mix No.	g_d [g/cm^3]	w [%]
1	1.648	14.20	4	1.706	13.81
2	1.661	14.20	5	1.675	13.56
3	1.690	14.18	6	1.674	14.16

2.3 Determination of California bearing ratio (CBR)

California bearing ratio (CBR) is a measure of bearing capacity of a certain mixture or material. It is determined immediately after compaction or after a certain period of curing. The determination of CBR on sand - wood ash mixtures was conducted according to the norm 13286-47 [14] and three samples were used for every mixture. The samples were prepared in Proctor’s cylindrical mold B with a 150 mm diameter and 120 mm height. The samples were submerged under water with a surcharge of 4,5 kg during a period of 4 days, during which swelling was periodically recorded in order to determine increase in linear swelling. The CBR index was then determined on the samples using a Mathest CBR device (figure 2).

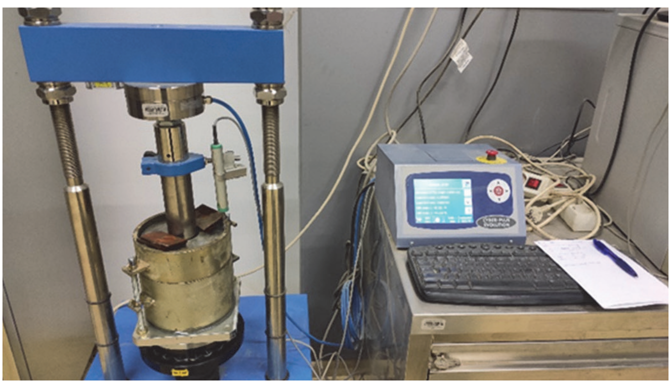

Figure 2 Examination of California bearing ratio CBR [15]

During the test, the force of piston penetration on a certain depth of the sample in the mold was measured, while it's penetration speed was 1,27 mm/min. During the test, the ratio of force and penetration was recorded, and based on the obtained diagram, the penetration force of 2.5 mm and 5 mm was read. The measured forces were then put into relation with reference forces measured at the same penetration into standardized material – crushed stone, which amount to 13,2 kN and 20 kN. The CBR index was expressed as the greater of two calculated values of the percentage of measured force in relation to the corresponding reference force. Table 2 shows the results of the bearing capacity test expressed as CBR index and linear swelling.

Table 2 Results of CBR index examination and linear swelling

Test		Mix No.				
		1	2	3	4	5
CBR 1	%	27.44	50.00	43.76	52.35	26.03
CBR 2	%	18.96	0.00	41.16	50.83	29.88
Linear swelling	%	0.02	0.53	2.14	2.33	5.11

2.4 Results commentary

Research results show that the maximum dry density increases with the increase of wood ash ratio in a mixture up to 30 %, while the optimum water content decreases. The highest value of maximum dry density of all examined mixtures (which had a value of 1,706 g/cm^3) was achieved in mixture 4, which contained 30 % wood ash. The lowest value of maximum dry density was achieved in 100 % sand (mixture 1) g_d = 1.648 g/cm^3.
The results of determining Californian bearing ratio (CBR) shown in table 2 and diagram on figure 3 show that the wood ash ratio has a direct effect on the increase of mixture bearing capacity. The control mixture of Drava sand (mixture 1) has a CBR index which is 27.44 %. Just by adding 10 % of wood ash into the sand, the bearing ration increased to 50.00 % (mixture 2). Mixture 3 (80 % sand / 20 % wood ash) had a slightly lower index of 43.76 %. The CBR was measured on mixture 4, which had 30 % wood ash and resulted in a value of 52.35 %. However, mixture 5 had a significant drop in CBR index, with a result of 29.88 %. Due to the fact that the value of CBR index in mixture 5 (50 % wood ash) was lower than those in the other mixtures and given the drop in compactness recorded by lowering of maximum dry density, the testing of mixture 6 with 75 % wood ash was not consider.

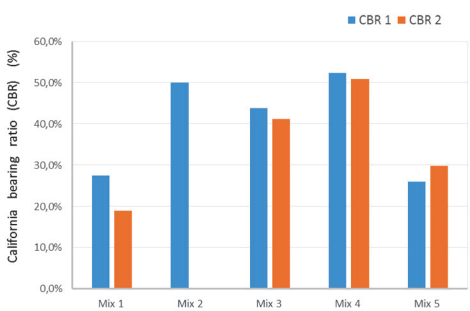

Figure 3 California bearing ratio (CBR) of all mixtures

Results of linear swelling shown in table 2 were as expected. The linear swelling increased with the increase of wood ash percentage in a mixture. For the control mixture (mixture 1), the swelling was negligible and equals 0.02 %. Swelling in mixture 2 was 0.53 %, in mixture 3 it significantly increases and was 2.14 %, while for mixture 4 it was 2.33 %. Mixture 5 with 50 % wood ash had the largest swelling which averaged at 5.11 %, making it unsuitable for base course construction.

3 Conclusion

The research on mixtures composed of Drava sand and wood ash in different ratios, shown in this paper, have proven the stabilizing properties of used wood ash. In this work, the described research of Drava sand and wood ash from biomass in different ratios has confirmed the existence of a stabilization effect in wood ash. The Californian bearing ratio (CBR) significantly increases with the increase of wood ash proportion (10 %, 20 %, 30 %) in mixtures. The highest bearing capacity was achieved in mixtures with 30 % of wood ash (Mixture 4) and amounted to CBR of 52.35 %, which is almost twice the CBR that was measured on sand (27.44 %). Also, the linear swelling of mixture 4 shows that it is suitable for base layer use in pavement constructions. These first encouraging results indicate to the possibility of applying a mixture of wood ash and sand (both local materials) in the base courses of the pavement structure of both public and commercial roads (agricultural and forest roads).

Acknowledgements

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References

[1] Braunović, P.: Kolovozne konstrukcije od stabilizovanog lokalnog materijala, Građevinska knjiga, ISBN 86-395-0284-6, p. 248, 1991.

[2] Bušić, M., Babić, B., Mintas, I., Trkmić, H.: Mogućnost primjene materijala s prostora Republike Hrvatske za izradu mehanički zbijenih slojeva kolničke konstrukcije, 1. hrvatski kongres o cestama, Hrvatsko društvo za ceste - Via Vita, pp. 544-549, 1995.

[3] Bošnjak, J.: Građenje cesta pijeskom, 1. hrvatski kongres o cestama, Hrvatsko društvo za ceste - Via Vita, pp. 536-543, 1995.

[4] Zagvozda, M., Dimter, S., Rukavina, T., Netinger Grubeša, I.: Possibilities of bioash application in road building, Građevinar, 70 (2018) 5, pp. 393–402, doi: http://doi.org/10.14256/JCE.2074.2017

[5] Zagvozda, M., Dimter, S., Rukavina, T.: The potential of bioash for utilization in road construction in eastern Croatia, Selected Scientific Papers - Journal of Civil Engineering, 12 (2017) 2, pp. 121–127, doi: http://doi.org/10.1515/sspjce-2017-0028

[6] Supancic, K., Obernberger, I.: Wood ash utilisation as a stabiliser in road construction – First results of large-scale tests, 19th European Biomass Conference & Exhibition, pp. 859–870, 2011.

[7] Šķēls, P., Bondars, K., Plonis, R., Haritonovs, V., Paeglītis, A.: Usage of Wood Fly Ash in Stabilization of Unbound Pavement Layers and Soils, 13th Baltic Sea Geotechnical Conference, pp. 122–125, Vilnius, Lithuania, 22-24.9.2016, doi: http://doi.org/10.3846/13bsgc.2016.017

[8] Šķēls, P., Bondars, K., Haritonovs, V.: Wood fly ash stabilization of unbound pavement layers, ICSMGE 2017 - 19th International Conference on Soil Mechanics and Geotechnical Engineering, pp. 2014–2017, Seoul Korea, 17-22.9.2017.

[9] HRN U.E9.022/70 - Nosivi slojevi podloga za ceste od mehanički stabiliziranog tla, Tehnički uvjeti za izradu

[10] HRN EN ISO 17892-4:2008 Geotechnical investigation and testing - Laboratory testing of soil - Part 4: Determination of particle size distribution

[11] EN 933-10:2009 Test for geometrical properties of aggregates – Part 10: Assessment of fines – Grading of filler aggregates (air jet sieving)

[12] Report on testing the mineral composition of ash samples by X-ray diffraction method (XRD) no. 65/19, Croatian Geological Survey Zagreb, 12. 2019.

[13] HRN EN 13286-2:2010 Unbound and hydraulically bound mixtures - Part 2: Test methods for laboratory reference density and water content - Proctor compaction (EN 13286-2:2010)

[14] HRN EN 13286-47:2012 Unbound and hydraulically bound mixtures -- Part 47: Test method for the determination of California bearing ratio, immediate bearing index and linear swelling (EN 13286-47:2012)

[15] Milovanović, B.: Svojstva mješavina za izradu nosivih slojeva kolničkih konstrukcija, diplomski rad, Građevinski i arhitektonski fakultet Osijek, p. 79, 2019.

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THERMO – MECHANICAL MODEL OF CONCRETE PAVEMENT IN HARDENING PHASIS

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Abstract

This paper is focused on the analysis of concrete pavements using finite element method (FEM). Specifically, it deals with the analysis of temperatures in the initial phasis of hardening and their influence on mechanical behavior of concrete pavement. High temperatures from hydration and climatic conditions in the early phase of concrete hardening co-operate and may initiate the formation of a network of micro-cracks on the surface of the concrete slab. The resulting temperatures (from hydration and climate) can theoretically be positively influenced by determining the start of concreting, so that the maximum temperatures do not meet at the same time. However, from a practical point of view the use of retarders is more realistic. Another possibility is to reduce the hydration heat by changing the composition of the concrete mixture (amount of cement, type of cement, use of alternative binders). Based on the knowledge of the material composition of the concrete and the specific temperature behavior during the concrete laying, it will be possible to predict the durability of concrete pavement in the future. Using weak formulation FEM model with quadratic base functions, the 2D heat transfer model was created. Boundary conditions were determined from experimental measurement on highway D1 in the Czech Republic. When this model was fitted to experimental data, the 3D coupled thermo - mechanical model was created. Soil and concrete elastic material characteristics had been taken over from Czech technical norms. Soil was modelled as Winkler-Pasternak 2D plate. Parameters c1 a c2 were assessed from comparison with 3D model with soil modelled as multiple layer system.

Keywords: concrete, pavement, FEM, cement hydration, heat transfer

1 Introduction

Rigid pavements are assumed to be loaded by combination of wheel load and linear temperature distribution over its thickness. The effects of thermal stress are determined for the maximum temperature differences of the upper and lower surface, usually assuming a linear temperature distribution over the thickness of the concrete slab. When determining the effects from temperature, the self-weight due to the stresses caused by the activation of the bending moment when the plate is bent (positive or negative temperature gradient) is a significant influence. The most significant fatigue stresses (tensile stresses) of the concrete pavement during its service life are from the combination of time-varying temperature and wheel loads. We can use numerical solutions [9] to analyze the phenomena, in this paper the FEM software OOFEM [1] is used.

The goal is to evaluate time dependent mechanical behavior from temperature loading. Temperature behavior is solved independently in heat transfer model, taking into account climatic boundary conditions and cement hydration. The temperature field obtained from this model is then exported to mechanical model, that evaluates stress and strain behavior of concrete pavement.
However, in order to correctly predict the future behavior of the concrete pavement, it is necessary to pay attention to the construction process of the concrete slab and take into account the conditions, under which it was made (air temperature, foundation system, sunlight, treatment after construction), in different words to monitor and influence the temperatures during the construction. The primary precondition for a more accurate determination of service life is the accuracy of the input data, such as specific concrete formula (water content, amount of cement, types and amounts of additives and admixtures, etc.) for road concrete. Changes in the approach to pavement concrete recipe [4] are currently being discussed in the Czech Republic, this should lead to more ductile concretes, rather than achieving the highest possible strength (i.e. use of less cement, use of mixed cements, higher water content). The above can affect the speed (slowing down) of hydration and thus reduce the formation of microcracks, which in the future may be a significant source of failure of the concrete pavement. Current state of work is limited to elastic material properties, the future progress should be related to usage of damage models or viscoelastic material parameters with aging, such as B3 [11] or MPS [12] model for concrete creep. Another field of interest should be moisture transfer [10] in concrete pavement and creation of complex coupled hygro-thermo-mechanical model. Heat and moisture transfer with cement hydration for concrete pavements were done before [13], but there several new viewpoints to be still observed.

2 Long-term temperature monitoring on the D1 highway

In the summer of 2018 during construction of the D1 highway (the section between Přerov and Lipník nad Bečvou) temperature and strain gauges were installed. The monitoring is still underway, and we now have data from construction and from service phasis. The monitoring is carried out in cooperation of CTU, VUT, Skanska and ŘSD (Road and Motorway Directorate of the Czech Republic) under leadership of Vít Šmilauer from CTU.
In this test section, the use of cement with slag admixture (CEM I 75 % and 25 % slag) is tested, instead of pure Portland cement (CEM I 100 % used exclusively for concrete highways in Czech Republic). The pavement consists of not reinforced concrete slabs, made using two-layer concreting. Dowel bars were inserted into the transverse joints as usual and anchors were placed in the longitudinal joints to prevent the plates from moving relative to each other. The gauges were placed at a safe distance from shrinkage joints. The lower layer of concrete was laid in the thickness of 240 mm and the upper layer in thickness of 50 mm.
As part of the monitoring a total of 18 gauges (18 temperature and 18 strain gauges) were installed in 6 places. In each place 3 gauges were installed in different height levels at (-50, -140 and -240 mm from the surface, see Fig. 1). At the same time gauges were installed to measure solar radiation and air temperature.

Figure 1 Placement of gauges (Photo: S. Šulc)

3 Use of monitoring for thermo-mechanical analysis

After performing the experimental measurement, it was necessary to process a large amount of data and create a mathematical description of the functions that would best describe the phenomena of air temperature and sun irradiation. The two functions were created to describe the weather conditions during the four days after constructing of the pavement, the functions describe representative summer days. First the function which describes the course of the air temperature (Fig.2). In the longer term this phenomenon can be described with a different function. However, for this numerical analysis it was only necessary to capture the range that had to be simulated (in this case four days). Furthermore, a more complex function was created which describes sun irradiation (based on Stefan-Boltzmann's law) again within four days after construction (Fig.2). These functions can be adjusted for specific weather conditions by simply changing the constants.

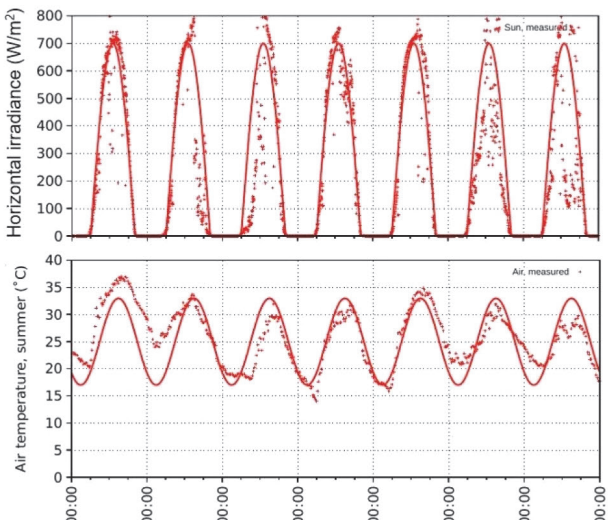

Figure 2 Comparison and validation of the two functions for sun irradiation (upper) and air temperature (lower) with experimental data

4 2D Heat transfer analysis of concrete pavement

Based on the experimental data, a 2D model was created, that contains the functions (1) and (2) as boundary conditions. Cement hydration model is implemented ("HydratingConcreteMat" [5]), it uses material parameters (potential heat of hydration, cement content, activation energy and others obtained from measurements with an isothermal calorimeter) and works with the hydration of cement over time. 2D model of heat conduction solves the balance equation

$$-\nabla^T q(x) + \bar{Q}(x,t) = \rho(x) c_v(x) \frac{\partial T(x,t)}{\partial t} \tag{1}$$

where: q is heat flux, Q is heat from hydration, T is temperature field.

The model was created as one half of the concrete slab (section in the longitudinal direction) due to the optimization of the computational task and assumption of axial symmetry. We assume that higher temperature profiles have a direct effect on reducing road durability [8], as higher temperatures accelerate cement hydration and increase shrinkage. Shrinkage cannot be completely removed but its size can be reduced significantly. There are several possible ways to achieve the reduction. It is possible to use other types of binder which would significantly slow down the so-called "kinetics" of hydration, it would reduce the development of hydration heat and thus reduce the formation of cracks and microcracks. The influence of different types of cement on the resulting temperature is evident from Fig.3. Material parameters were taken from the literature ([2] and [3]). The created model therefore allows the analysis of specific conditions for hardening phasis of road concrete in terms of temperature. Its use for various analyzes is obvious.

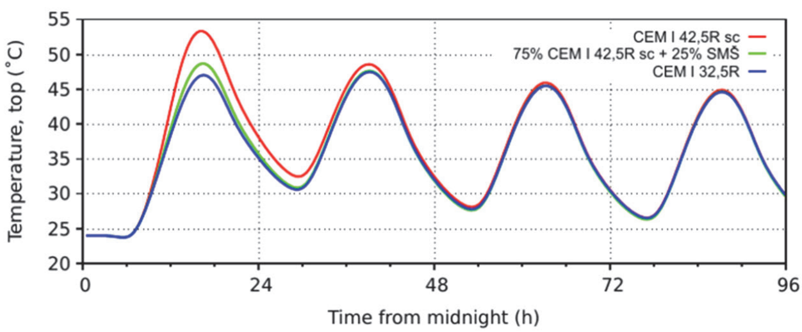

Figure 3 Influence of the type of cement binder on maximal temperature on the slab surface

5 Influence of the time, when the construction begins on the maximal temperature in the concrete slab

The created 2D model for heat conduction was also used to analyze the effect of the time, when we carry out the construction of the slab (four summer days) on the resulting temperature on the top of the slab. The results are interesting. Construction companies (for practical reasons) usually start the construction in the early morning during summer days. This setting results in the worst temperature combination where the maximal temperature from the hydration of the binder reaches a maximum value together with the highest daily air temperature. It is also favorable to delay the temperature peak because of mechanical properties of concrete. The total temperature profiles on the upper and lower surface of the slab depending on the beginning of concreting are evident from Fig. 4 and Fig. 5. Theoretically, it would be more appropriate to start the construction process in the afternoon during summer, based on this model. However, it will be necessary to validate this during real construction to approve this assumption.

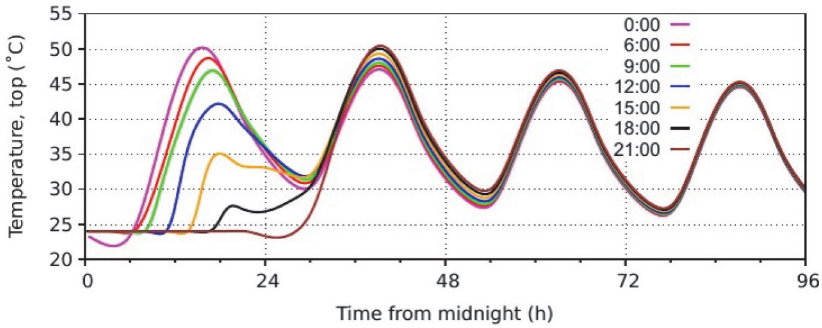

Figure 4 Influence of the time, when the construction begins on the maximal temperature on the slab surface

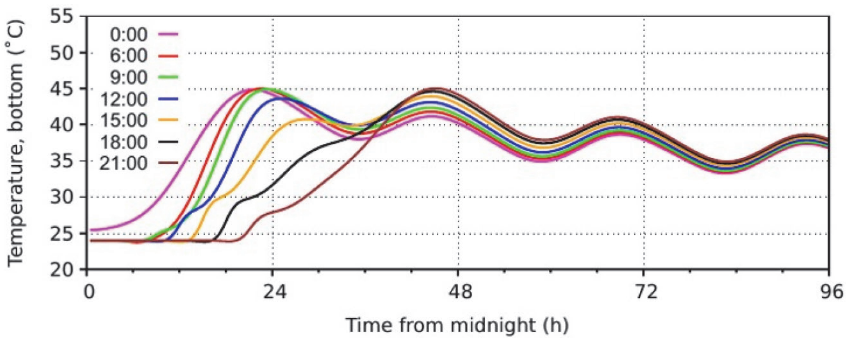

Figure 5 Influence of the time, when the construction begins on the maximal temperature on the slab bottom

6 3D Thermo - mechanical analysis of concrete pavement

The most important part for us is to obtain stress and strain fields from the time variable heat transfer task shown above. This can be achieved by weakly coupled thermo-mechanical task. The principle is that in each individual step of the calculation, the temperature field is calculated using the heat transfer model and then "exported" to the "mechanical" model as loading (MUPIF framework). This requires the creation of a "3D - heat transfer model", this can be quite easily achieved by rewriting the previous 2D model. Then we must create the 3D mechanical model for concrete pavement, that should be able to cope with the temperature field in the requested way [6]. The 3D mechanical model was considered as 3D slab with elastic material parameters on the Winkler - Pasternak (W-P) subsoil [7]. It was also necessary to assign parameters to the individual materials, relating to thermal capacity, thermal conductivity and thermal expansion. The development and more accurate modeling of this task is expected in the future, because in the hardening phase the stiffness of the cross-section changes considerably and for a more accurate description, it will be necessary to use for example viscoelastic material with aging, moisture transfer and shrinkage. The interface between W-P and concrete slab is represented by non-linear interface elements with different stiffness in tension and compression. When certain tensile deformation is exceeded, the separation of the slab is allowed. As shown in the Fig.6 and Fig.7, we obtain quite significant reduction of stress using blend of CEM I 42,5 with slag against pure CEM I 42,5, with both separated from the W-P subsoil.

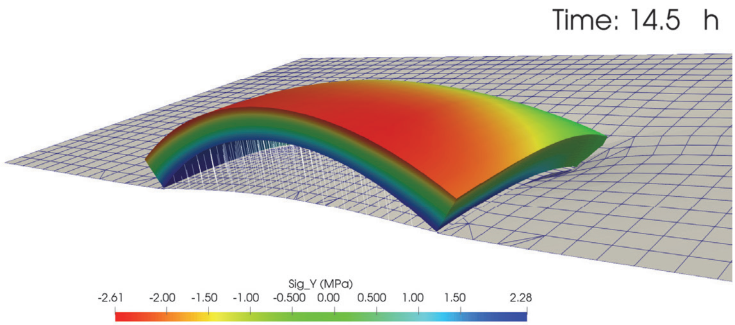

Figure 6 Stress field at 14:30, 8:30 after beginning of construction with only CEM I 42,5

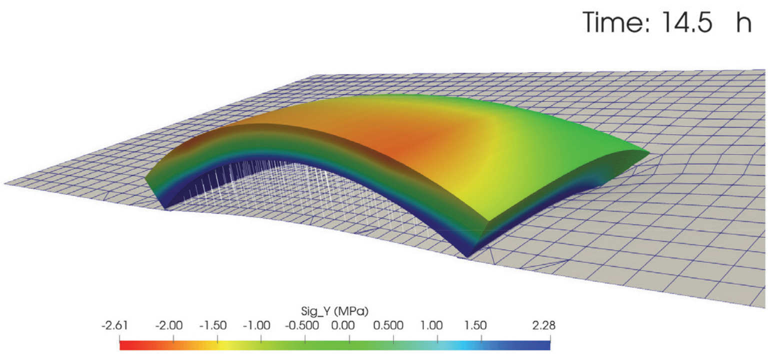

Figure 7 Stress field at 14:30, 8:30 after beginning of construction with blend of CEM I 42,5 with slag

7 Conclusion

The thermo-mechanical model of the concrete pavement helps to better understand the processes in concrete during the construction and hardening of concrete, considering the type of material and external temperature conditions. Future analysis and results of thermo-mechanical models with variable material properties and temperature conditions may lead us to determine the ideal parameters during construction. The aim should be the reduction of the formation of more microcracks in the initial phase of concrete hardening. This would positively affect (extend) the life of the concrete pavement. Further use of these models is possible in predicting the residual life of the concrete pavement and thus future applications in road management systems. We can also validate usage of less resource and environmental draining binder than pure Portland cement.

Acknowledgement

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References

[1] Patzák, B.: OOFEM - an object-oriented simulation tool for advanced modeling of materials and structures, Acta Polytechnica, 52 (2012) 6, pp. 59–66, doi: 10.14311/1678

[2] Luxemburk, F., Vébr, L., Kudrna, J., Varaus, M., Racek, I., Fiedler, J.: TP 170 Navrhování vozovek pozemních komunikací, Ministry of Transport of the Czech Republic, 2006.

[3] Šmilauer, V.: Termo – mechanické analýzy nezeslabených a zeslabených cementobetonových desek dálničních krytů, CTU, 2017.

[4] Šmilauer, V.: Betony odolné ke vzniku mikro a makrotrhlin na CB krytech, SBORNÍK Concrete roads 2018

[5] Šmilauer, V., Bittnar, Z.: Effects of Representative Cube Size on the Simulation of Portland Cement Hydration in CEMHYD3D Model, 5th International PhD Symposium in Civil Engineering, 2004. pp. 581-587

[6] Choubane, B., Tia, M.A.: Nonlinear temperature gradient effect on maximum warping stresses in rigid pavements, Transportation Research Record, 1992, p. 11

[7] Ioannides, A.M.: The problem of a slab on an elastic solid foundation in the light of the finite element method, Sixth International Conference on Numerical Methods in Geomechanics, Innsbruck, Austria, 1988, pp. 1059–1064.

[8] Förster, M.O.: Temperaturbedingte Beanspruchung von Betonfahrbahnen, Mitteilungen Aus Dem Institut Fuer Verkehrswirtschaft, Strassenwesen Und Staedtebau, Fachgebiet Konstruktiver Strassenba, 2005.

[9] Ioannides, A.M.: Concrete pavement analysis: the first eighty years, International Journal of Pavement Engineering, 7 (2006) 4, pp. 233–249

[10] Bažant, Z.P., Jirásek, M.: Creep and Hygrothermal Effects in Concrete Structures, Springer Netherlands, 2018.

[11] Bažant, Z.P., Murphy, W., P.: Creep and shrinkage prediction model for analysis and design of concrete structures-model B3, Matériaux et constructions, 28 (1995), pp. 357-65.

[12] Jirásek, M., Havlásek, P.: Microprestress–solidification theory of concrete creep: Reformulation and improvement, Cement and Concrete Research, 60 (2014) 1, pp. 51-62, doi: 10.1016/j.cemconres.2014.03.008

[13] Mukhopadhyay, A.K., Ye, D., Zollinger, D.C.: Moisture-related cracking effects on hydrating concrete pavement, Texas Transportation Institute, Texas A & M University System, 2006.

FINITE ELEMENT SIMULATION AND MULTI-FACTOR STRESS PREDICTION MODEL FOR CEMENT CONCRETE PAVEMENT CONSIDERING VOID UNDER SLAB

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Abstract

Uneven support as result of voids beneath concrete slabs can lead to high tensile stresses at the corner of the slab and eventually cause many forms of damage, such as cracking or faulting. Three-dimensional (3D) finite element models of the concrete pavement with void are presented. The accuracy of the model is verified by two methods. The analysis shows that the impact of void size and void depth at the slab corner on the slab stress are similar, which result in the change of the position of the maximum tensile stress. The maximum tensile stresses do not increase with the increase of the void size for relatively small void size. The maximum tensile stress increases rapidly with the enlargement in the void size when the size≥0.4m. The increments of maximum tensile stress can reach 183.7% when the void size are 1.0m. The increase of slab thickness can effectively reduce maximum tensile stress. A function is established to calculate the maximum tensile stress of the concrete slab. The function takes into account the void size and the slab thickness. The reliability of the function was verified by comparing the error between the calculated and simulated results.

Keywords: concrete pavement, void underneath the slab, finite element model, predictive function, maximum tensile stress

1 Introduction

Jointed plain concrete pavement (JPCP), known as its significant compressive strength and durability, is designed as one feasible ridged pavement style in those heavy traffic load areas [1]. Thin plate theory is advised as one alternative when doing analysis for cement concrete pavement. This theory is based on a previous assumption which is the foundation (base course or subgrade soil) is regarded as consistently. This opinion is also mentioned by other researchers. It can adequately simplify the mechanical analysis of pavement response. Nevertheless, One main problem is its shortage on illustration the behaviors when some undetermined but natural outcomes occur. Many studies have found that there are voids beneath the cement slab, which are an un-avoidable damage in the pavement service duration particularly near the corner or edge of the slab [2]. The occurrence of voids can result in high tensile stresses at the corner of the slab and eventually cause many forms of potential, such as cracking or faulting [3]. In recent years, most researches on concrete pavement mainly focus on the influence of other factors on the response of pavement structure, such as temperature, dowel bar [4-6]. Foundation is generally considered consistently uniform in their researches. However, the coupling action of void and traffic load remain un-clear and needs to be analyze quantitatively. Furthermore, the void depth is so large that the slab and base can never contact in the void space under traffic loads in their researches. The results of in-situ

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coring show that the slab is not completely separated from the base course in the early stage of void development. The slab and base course can still contact in the void area under the traffic load. Few studies have considered the effect of void depth on slab stress, such as the change from non-contact to contact between slab and base in void space under traffic load. The main purpose of this study is to investigate the effect of void on the maximum tensile stress. In this paper, ABAQUS is selected due to its excellent simulating ability. The reliability of the model was validated by mesh convergence analysis and comparison with the calculation results of the design standards in China. Void size and depth were analysed in the validated FEA model with a single concrete slab. Slab thickness are also considered in the validated model. A stress prediction formula is proposed based on the analysis results. Super computing resources can help to reduce the burden of large problem size.

2 Materials and methods

2.1 FEA model parameters

The analysis model of concrete pavement was worked out in three-dimensional Cartesian coordinate system and corresponded to a selected motorway pavement in China, such as Fig 1. The model consists of two structural layers, which are concrete surface course layer and cement and fly-ash stabilized macadam base course layer. The Winkler foundation is used to simulate the structural layer below the base course layer [7]. There are two methods to build Winkler foundation in ABAQUS. Several studies use spring elements of type SPRING1 to idealize the subgrade [8]. In this study, Interaction of type Elastic Foundation is used.
The surface course of the pavement consists of one concrete slab, which ignore the effect of adjacent slabs on it. Previous studies have indicated that an extended base can effectively decrease the stress of slab, which is more in line with the engineering [9]. Numerous studies have shown that linear elastic constitutive in the model can help to obtain rational results [10], which was used in this paper. The three-dimensional finite model characteristics are shown in Fig 1.

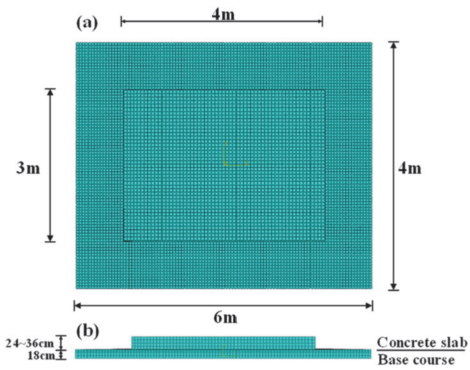

Figure 1 FEA model of concrete pavement structure: a) plane size, b) vertical size

The models were carried out by using eight-node incompatible modes linear hexahedra solid elements (C3D8I--concrete slab) [11] and eight-node reduced-integration linear hexahedra solid elements (C3D8R—base course).The relatively slide behavior between the concrete slab and the base course layer was taken into account, but not the sliding friction, i.e., the friction coefficient is 0 [12]. This can help to get the most unfavorable stress values in the slab. The wheel paths of vehicles were shown in Fig. 2. A boundary condition of the fixed placement in the horizontal direction is applied to the base course. All displacements at nodes on all side faces of the concrete slab are free [13].

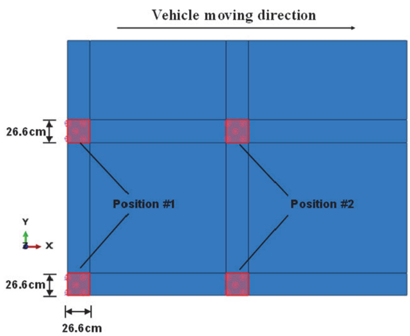

Figure 2 The shape and position of the vehicle load on the slab of FEA model

3 Results and discussion

3.1 Impact of void size at the slab corner

The void depths are provided as 5 cm to ensure that the concrete slab does not contact with the base course under vehicle load. Five sizes of void area are used: 0.02 m^2, 0.08 m^2, 0.18 m^2, 0.32 m^2 and 0.5 m^2. Four slab thicknesses (24 cm, 28 cm, 32 cm and 36 cm) were also analysed. The pressure of vehicle load is provided as 0.8 MPa. In Fig. 3, the stress of the slab remains almost constant with the increase of the void size regardless of the slab thickness for relatively small void size (0 m, 0.2 m and 0.4 m). While the stress of the panel increases rapidly with the enlargement of the void size for relatively large void size (0.6 m, 0.8 m and 1.0 m).

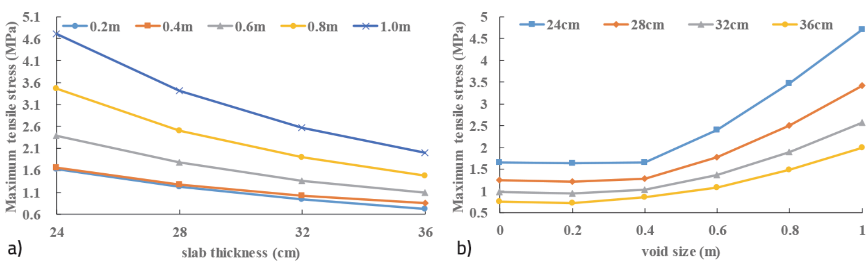

Figure 3 Variation in stress of the slab with slab thickness and void size: (a) slab thickness; and (b) void size

The maximum tensile stress can still be obtained at the bottom of the slab with 0.2 m void size. The point is that the stress at the bottom of the plate decreases slightly while the stress at the top increases rapidly compared to the stress with no void area. Further calculations show that the most unfavorable load is still at position 2. The maximum tensile stress is obtained at the top of the slab (above the edge of the void) with 0.4 m void size. It means that the stress at the slab top exceeds the stress at the slab bottom. The maximum tensile stress when the load is at the slab corner (position #1) is greater than that when the load is at the slab edge (position #2), which means that the most unfavorable load position is at position 1. Both occurred during the increase in the size of the void from 0.2 m to 0.4 m. In the following analysis, the relatively small void sizes (0.2 m and 0.4 m) will not be considered. It can also be obtained from Fig. 3 that the increase of slab thickness can effectively reduce the increase of maximum tensile stress.

3.2 Impact of void depth at the slab corner

Three void sizes (0.6 m, 0.8 m and 1.0 m) and four slab thicknesses (24 cm, 28 cm, 32 cm and 36 cm) are considered in this section. The initial height of the void area was 0.2mm and increase 0.2 mm each time until the slab stress no longer changes. The stress of different positions of concrete slab under 0.7 MPa load is recorded. The compressive stress at the bottom of the slab corner is the contact stress between concrete slab and base course. It can be seen that the stresses at the bottom and top of the slab do not change when the compressive stress at the slab corner is reduced to zero in Fig. 4. This means that the depth of voiding has been increased sufficiently so that the slab never contacts the base course and the slab stress is independent of the void depth.

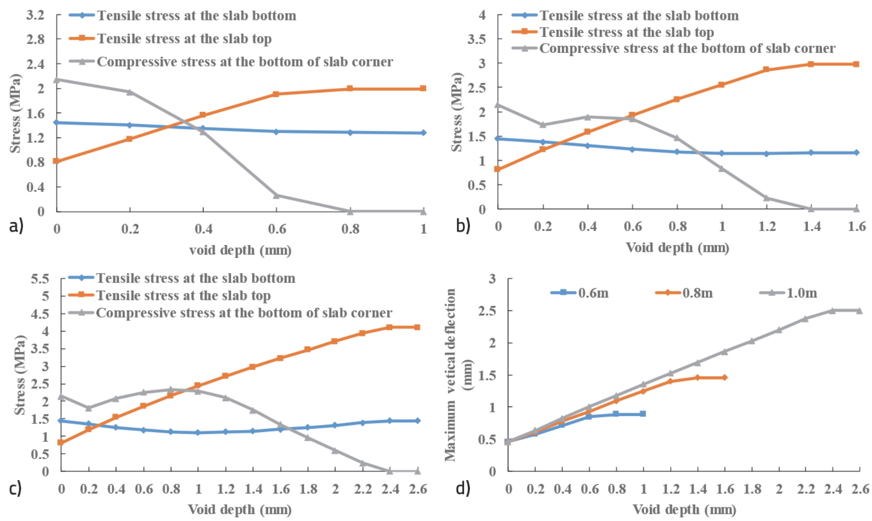

Figure 4 Stress variation of 24 cm thick slab with void depth: a) void size is 0.6m, b) void size is 0.8m, c) void size is 1.0m, d) vertical deflection

In Fig. 5, the stress at the bottom of the slab decreases first and then increases with the increase of the void depth. However, the variation is very small, which is within 20 %. The stress on the top of the slab increases with the increase of the void depth until the cement slab no longer contacts the base course. There is approximately a secondary correlation between the slab top stress and void depth. The stress of slab top gradually exceeds the stress of slab bottom when the void depth increase from 0.2 to 0.4 mm. This law is similar to that of the variation of slab stress with the void size.

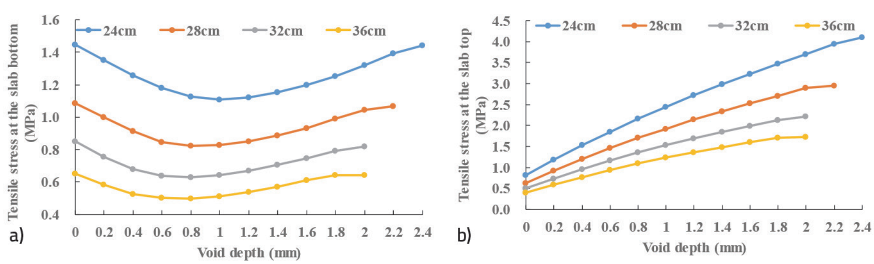

Figure 5 Tensile stress variation of 1.0m void size with void depth: (a) stress at the slab bottom and (b) stress at the slab top

3.3 Regression analysis of maximum tensile stress

In Section 3.1, the effects of slab thickness, void size and vehicle load on the maximum tensile stress of slab are analyzed. In this section, the function for obtaining the maximum tensile stress through these three factors is presented. Since the most unfavorable load position of the slab is always at the edge of the slab (position #2) when the void size is small (0.2 m), this function only considers the case when the void size is large (0.4 m, 0.6 m, 0.8 m and 1.0 m).

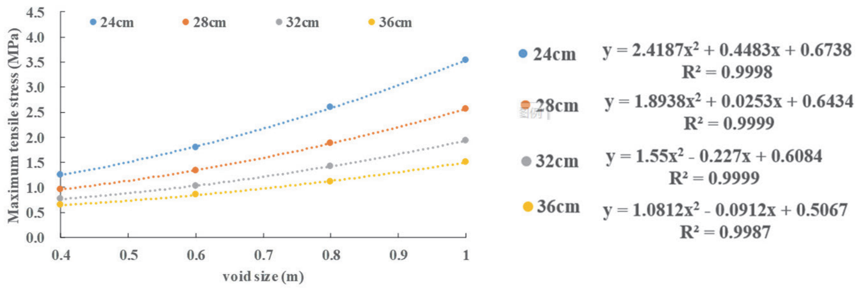

Figure 6 Regression relationship between maximum tensile stress and void size under 0.6MPa load

It can be seen from Fig. 6 that the error between the curve obtained by quadratic regression and the simulation calculation result is within 2 %. When the panel thickness is 24 cm, 28 cm, 32 cm and 36 cm, the relationship between the panel stress function and the void size as the independent variable is as follows:

$$\sigma = 2.4187X^2+0.4483X+0.6738 \tag{1}$$

$$\sigma = 1.8938X^2+0.0253X+0.6434 \tag{2}$$

$$\sigma = 1.55X^2+0.227X+0.6084 \tag{3}$$

$$\sigma = 1.0812X^2+0.0912X+0.5067 \tag{4}$$

Where σ is maximum tensile stress; X is the void size and X ≥ 0.4 m. The quadratic term, primary term and constant term of X are regressed respectively in Fig. 7 and the stress calculation function considering both void size and slab thickness is obtained:

$$\sigma =(-0.1089h+5.0032)x^2+(0.0087h^2-0.5706h+9.1255)x+(-0.0011h^2+0.0534h+0.0299) \tag{5}$$

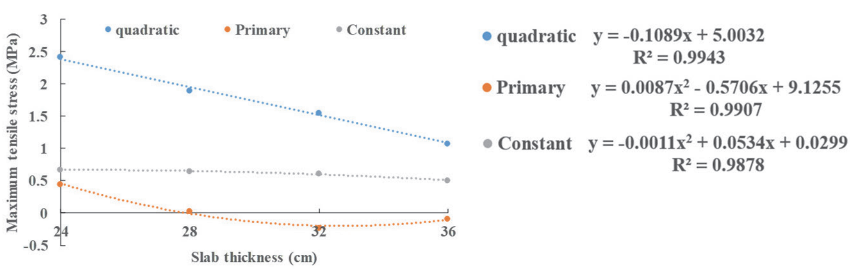

Figure 7 Regression relationship between maximum tensile stress and slab thickness under 0.6MPa load

Five void sizes (0.6 m, 0.7 m, 0.8 m, 0.9 m, 1.0 m) and three panel thicknesses (26 cm, 30 cm, 34 cm) are considered. 15 examples are additionally calculated for validation. The results are presented in Table 3. The maximum error between the results of the formula calculations and the model calculations is 2.36 %. The results show that the use of this function to predict slab stress is reliable.

4 Conclusions

In this paper, the impact of void parameters on concrete slab stress is investigated, which is supported by numerical simulation. The location, shape, and size of voids underneath slabs are indicated to have a significant effect on panel stresses. Finally, a function is established to calculate the maximum tensile stress of the concrete slab. The major findings are summarized as follows:

- Impact of void size and void depth at the slab corner on the slab stress are similar. With the increase of both, the stress at the bottom of the slab decreases slightly and the stress at the top of the slab increases rapidly. When the void size is greater than 0.4m and the void depth is greater than 0.4mm, the stress at the top of the slab exceed that at the bottom.
- A function is established to calculate the maximum tensile stress of the concrete slab. The function takes into account the void size, the slab thickness and the vehicle load. The reliability of the function was verified by comparing the error between the calculated and simulated results

References

[1] Zukang, Y.: Design theory and procedure of cement concrete pavement. China Communications Press: Beijing, China, 2003.

[2] American Association of State Highway and Transportation Officials (AASHTO), The AASHTO guide for the design of pavement structures, Washington, DC, 2011.

[3] Alland, K., Vandenbossche, J.M., Brigham, J.: Statistical Model to Detect Voids for Curled or Warped Concrete Pavements, Transp. Res. Rec., 2639 (2017), pp. 28-38

[4] Grosek, J., Zuzulova, A., Brezina, I.: Effectiveness of Dowels in Concrete Pavement, MATERIALS 12 (2019) 10, doi: 10.3390/ma12101669.

[5] Yi-Qiu, T., Yong-Kang, F., Yun-Liang, L., Chi, Z.: Responses of snow-melting airfield rigid pavement under aircraft loads and temperature loads and their coupling effects, Transportation Geotechnics, 14 (2018), pp. 107-116

[6] Wadkar, A., Mehta, Y., Cleary, D., Guo, E., Musumeci, L., Zapata, A., Kettleson, W.: Load-Transfer Efficiencies of Rigid Airfield Pavement Joints Based on Stresses and Deflections, J MATER CIVIL ENG, 23 (2011), pp. 1171-1180

[7] Bronuela, L., David Lee, H., Ryu, S., Ho Cho, Y.: Cantilever and pull-out tests and corresponding FEM models of various dowel bars in airport concrete pavement, Construction and Building Materials, 83 (2015), pp. 181-188

[8] Aure, T.W., Ioannides, A.M.: Curling effects on concrete slab-on-grade fracture, Materials and Structures, 49 (2016), pp. 2991-3004

[9] Zhou, Y.M., Tan, Z.M., Zhao, J.: Loading Stress in Cement Concrete Pavement Structure with Base Widening. Journal of Tongji University, 35 (2007) 10

[10] Mackiewicz, P.: Analysis of stresses in concrete pavement under a dowel according to its diameter and load transfer efficiency. Canadian Journal of Civil Engineering, 42 (2015) 42, pp. 845-853

[11] Huang, Y., Yuan, J., Tan, Y.: Identification of Void Beneath Airport Cement Concrete Pavement and Its Influecce. Journal of Tongji University 2012., 40(6)

[12] Jeong, J., Park, J., Lim, J., Kim, S.: Testing and modelling of friction characteristics between concrete slab and subbase layers. Road Mater Pavement, 15 (2013), pp. 114-130

[13] Nishizawa, T., Ozeki, T.: Finite Element Model Analysis of Thermal Stresses of Thick Airport Concrete Pavement Slabs, Transportation Research Record, 2095 (2009) 1, pp. 3-12, doi: https://doi.org/10.3141/2095-01

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PERFORMANCE ANALYSIS OF FLEXIBLE PAVEMENTS WITH BASE LIME

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Abstract

The performance of the pavement is influenced by several factors, such as the pavement structure, materials, traffic, and climate conditions. These factors affect the pavement response, mainly the compressive strain developed at the top of each layer and the tensile strain developed at the bottom of the asphalt concrete layer, resulting in various forms of distresses, such as fatigue cracking. The materials used in the construction of these layers are equally important for the long-term performance of the pavements as well as its structural stability. Aggregates are the most used materials in the construction of base layers in a flexible pavement structure. Moreover, the aggregate used in the base layer provides foundation for the overlying layers and needs to have enough strength, but due to the scarcity of quality materials and the rising demand, base layers are often treated with different types of stabilizing agents. In this study, various mechanistic analyses are performed using the 3-D Move Analysis software to study the effects of lime as a stabilizing agent on fatigue resistance performance. These analyses showed that the use of lime as a stabilizing agent increased the pavement performance up to 48 % for fatigue cracking resistance when compared to untreated base layers. The cost-effectiveness analysis also showed that the use of stabilizing agents would reduce the long-term cost of pavement as compared to untreated bases. The overall cost efficiency of the lime treated base is found to be 1.68 times the untreated base.

Keywords: base treatments, fatigue cracking, mechanistic analysis, finite element analysis, lime, cost efficiency

1 Introduction

Hot mixed asphalt (HMA) pavement consists typically of 3-layers: HMA surface layer, base layer and subgrade. The base layer composed of aggregates is an important layer in terms of structural performance. The load transferred from the surface of the pavement ultimately goes to the base, therefore, there is a great need for the base layer to be strong to handle the variable traffic loads and various climatic conditions (1). Various methods are being utilized for enhancing the strength of the base layer against traffic loading and climate conditions. The multiple layers in the pavement structure are required to withstand the traffic loads and various distress generated. A base layer should be strong and have rigidity to not allow distortion, lateral flow and consolidation. The base course layer is designed to have adequate thickness to reduce the traffic damage over time. (2). A base layer can be made either bounded or unbounded. A bounded layer refers to a base layer where some sort of stabilizing agent or treatments agents is utilized to make the layer more robust and stable. In contrast to bound layer, an unbounded layer does not utilize any kind of external agent and the strength of the base layer is solely depends on the strength of the aggregates. Various kinds of stabilizing agents are utilized in bounded bases such as lime, cement and asphalt.

The use of these additives is very beneficial in the construction of HMA pavement as they reduce distresses in the pavement structure. Pavement performances are also affected by the environmental conditions. Therefore, a proper study regarding the utilization of these kinds of stabilizers must be made through various mechanistic and cost-effectiveness analyses. The study presented in this paper compares various aspects of utilizing lime treated bases and untreated bases.

2 Literature review

Stabilization is the process of adding a cementing agent to the soil or crushed rock to produce materials that have greater strength than the original unstabilized ones [3,4]. There are two types of base layers generally used in the construction of flexible asphalt pavements, which includes unbound aggregate bases that consists of untreated granular materials and bound aggregate bases that consist of granular material bounded physically or chemically by a stabilizing agent (e.g., cement, asphalt emulsion or foamed asphalt [3]. The use of a stabilized base results in an increased performance of base layer with a greater stability and proper aggregate interlock. Johnson [5] studied the use of lime on bases and subgrades to increase its performance. The study found that poor subgrade and base materials can be modified to a significant level if appropriate quantities of the lime were used. The finished base was also found to be waterproof if lime was used. A study was performed by Azadegan et al. on the performance evaluation of lime and cement treated base layers in the unpaved road [6]. It was found that there bearing capacity of the base layer increased, and similarly, stiffness of subgrade and base layer should have a corresponding value. In a report submitted to the National Lime Association, the utilization of lime-stabilized layer in mechanistic-empirical pavement design is described [7]. It was basically focused on utilizing the lime stabilized layer as a structural component of the pavement system. Different issues and steps needed to be considered when incorporating lime stabilized layers in the pavement design are exclusively discussed. Mixture design and material testing were essential components. Moreover, economic benefits from the utilization of lime treated layer was an important finding from the study.
In addition to the structural benefits of the treated base layers of pavement, various economic savings are obtained [5, 7]. Koroma studied the life cycle cost analysis of pavement sections containing treated open-graded bases and compared them to traditional dense-graded untreated bases using predicted performance of the MEPDG [8]. Treated open-graded bases were found to have higher life cycle cost.
The various studies presented above showed that the addition of additive or using base treatments resulted in a great impact on the pavement structural capacity and its life. These studies have clearly provided analysis related to the strength, but the long-term impact on the cost and benefit are rarely described. This paper quantifies the recurring cost using mechanistic-empirical analysis based on bottom-up fatigue cracking for lime treated base in pavement structures.

3 3-D study objective

Base treatments are one of the most important construction practices to increase the overall pavement performance in addition to their potential long-term cost-effectiveness benefits. Various stabilizing materials are utilized for base treatments. This study focuses on the use of lime as stabilizing agent. Lime treated base was considered in determining the improved pavement performance using mechanistic analysis, which then was utilized to investigate the cost-effectiveness of such treatments using two different binder grades at four different traffic speeds.

4 3-D move mechanistic analysis

One of the One of the most powerful software packages in the design of flexible pavements is referred to as the 3-D Move Analysis. Complex surface loading, such as multiple loads and non-uniform tire pavement contact stress, are handled by the program with the continuum finite layer approach [9]. Advanced applications of the software include estimation of damage under-off-road farm vehicles and estimation of pavement performance at the intersection. This study utilized the 3-D Move Analysis software to the utmost level to find the performance of the flexible pavement base when it accounts for the bottom-up fatigue cracking for two different grade of binder and three different temperatures with two different base sections of untreated and lime treated.

This research used the HMA properties determined in the National Cooperative Highway Research Program (NCHRP) 9-44 A (10). The test results used in this study are the results presented in the project report NCHRP Report 762. The values required in the 3-D Move Analysis, such as dynamic modulus |E*|, phase angle (ø), and fatigue regression coefficient are derived from the same research project. The research effort of the NCHRP 9-44 A included the characterization of different PG asphalt binders. This study considered two PG asphalt binders which are PG 64-22 and PG 76-16. The corresponding regression coefficient k1, k2 and k3 of the generalized fatigue model of PG 64-22 are 0.000558, 3.876197 and 0.875271, respectively. Similarly, for PG 76-16 asphalt binder, the fatigue regression coefficients k1, k2 and k3 are 0.000558, 3.876197 and 0.875271, respectively [10].

5 Mechanistic Analysis of Bottom-Up Fatigue Cracking

Among the various types of the distress conditions in flexible pavements, bottom-up fatigue cracking is one of the major forms of distress. Bottom-up fatigue cracking is a series of interconnected cracks developed in the surface of the HMA surface or base under repeated traffic loading. Crack initiates at the bottom of the asphalt layer and propagates towards the surface of the pavement. The mechanistic performance of base layer under various treatments, such as lime is expected to perform better. Figure 1 shows the bottom-up fatigue cracking performance of two different types of mixtures, one with binder grade PG 64-22 and the other one with PG 76-16 under three different speeds of 40, 72, and 104 kilometre per hour.

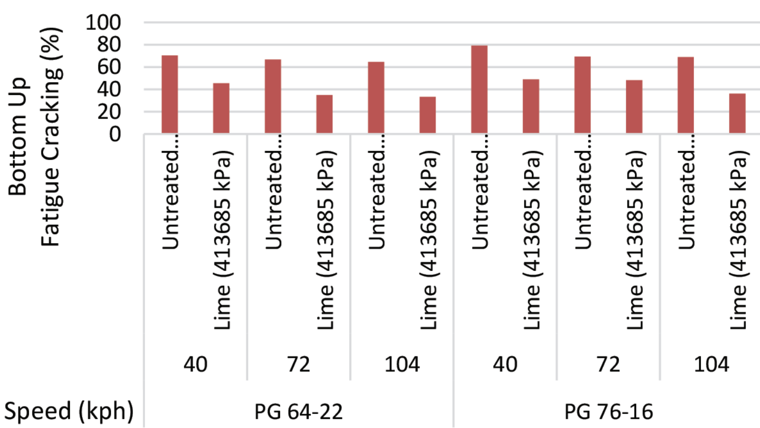

Figure 1 Bottom-up fatigue performance of pavement with lime treated base

It can be observed from Figure 1 that treated base layers had superior fatigue cracking resistance as compared to untreated sections. Lime treatment had the low predicted fatigue cracking. It can also be noticed that pavement structures with stiffer asphalt binder grade (PG 76-16) are more susceptible to fatigue cracking than softer asphalt binder grade (PG 64-22). The fatigue cracking of both PG 64-22 and PG 76-16 asphalt binders decreases as the traffic speed increases due to the viscoelastic nature of asphalt pavements where pavement structures act as a strong material under high loading frequency (high traffic speed) whereas it acts as a weak material under low loading frequency (low traffic speed).
In order to mathematically quantify the performance of base treatments with regard to their improved fatigue cracking resistance, a Fatigue Cracking Reduction Percentage (FCRP) was calculated as Eq. (1).

$$FCRP = \frac{\text{Fatigue cracking for untreated base section} - \text{fatigue cracking of treated base section}}{\text{Fatigue cracking for untreated base section}} \cdot 100\% \quad (1)$$

Table 1 shows the calculated FCRP for all structures illustrated in Figure 1. All presented lime treated bases at different traffic speeds and binder grades had a highest FCRP of 48 %. This indicates that lime base treatment has better performance than the untreated bases.

Table 1 Bottom-Up Fatigue Cracking Performance of lime treated base

Binder Grade	Speed Limit [km/h]	Base treatment	Bottom-Up Fatigue Cracking [%]	Fatigue Cracking Reduction Percentage (FCRP)
PG 64-22	40	Untreated (172369 kPa)	70.38	N/A
		Lime (413685 kPa)	45.54	35.29
	72	Untreated (172369 kPa)	66.82	N/A
		Lime (413685 kPa)	34.84	47.86
	104	Untreated (172369 kPa)	64.56	N/A
		Lime (413685 kPa)	33.32	48.39
PG 76-16	40	Untreated (172369 kPa)	79.27	N/A
		Lime (413685 kPa)	48.99	38.20
	72	Untreated (172369 kPa)	69.37	N/A
		Lime (413685 kPa)	48.29	30.39
	104	Untreated (172369 kPa)	68.88	N/A
		Lime (413685 kPa)	36.19	47.46

*N/A relates to original untreated base layer

6 Cost-effectiveness analysis of base treatments

Cost-effectiveness analysis plays an important role to determine the performance versus the cost of using different base treatment applications. In this study, cost-effectiveness analysis was conducted for lime treated bases in terms of its improved fatigue resistance as compared to untreated bases. Eq. (2) is the mathematical representation of the estimated cost-effectiveness of base treatments in terms of bottom-up fatigue cracking:

$$\text{Cost-effectiveness of base treatments in terms of fatigue} = \frac{\text{Undamaged Area of Pavement due to Bottom-up fatigue cracking}}{\text{Cost per mile of the pavement}} \tag{2}$$

Upon determining the cost-effectiveness of each base treatment, cost-effectiveness ratio can also be determined as Eq. (3):

$$\text{Cost} - \text{Effectiveness Ratio} = \frac{\textit{Treated Cost Effectiveness}}{\textit{Unteated Cost Effectiveness}} \tag{3}$$

6.1 Remaining undamaged pavement condition

By the end of the design life of 20 years, the remaining undamaged surface area of pavement due to bottom-up fatigue cracking can be estimated as the total surface area (1.600m*3.66m) minus the predicted bottom-up fatigue cracking as shown in Table 2.

6.2 Cost per mile of pavement

To estimate the cost of each base treatment and compare it to the untreated base, the cost of one ton of each of the base treated layer was calculated given the fact the unit price for aggregates, lime is $22, $220.61 per ton, respectively [11]. In this analysis, lime treatments were added to the base aggregates at a rate of 2 % by weight of the aggregates. This leads to the cost of base layer calculated as the following (assuming that the cost of plant and equipment are same for all types of bases):

- 1 Ton of Untreated Base Layer: $22/ton
- 1 Ton of Lime Treated Base: 2 % of $220.71/ton+ 98 % of $22/ton=$25.98/ton

For 8 inches of base layer thickness, the required quantity is calculated as width (3.66 m) × length (1600 m) × thickness (0.2 m) × density (2472.42 kg / m^3) = 2895.7 tons. Therefore, the cost required for paving with the given base and treatments can be calculated as:

- Cost to pave 1.6 km of untreated base case= $ 63,705
- Cost to pave 1 mile of lime treated base case= $ 75,229

6.3 Cost - effectiveness of lime base treatment in terms of bottom-up fatigue cracking

Based on the calculated remaining undamaged area of pavement due to bottom-up fatigue cracking and the cost per one mile of each base-treatment, cost-effectiveness for lime treated bases in terms of bottom-up fatigue cracking were calculated based on equation 2. Overall results are shown in Table 2.

The cost-effectiveness analysis of the base treatment in terms of bottom-up fatigue cracking shows that the use of base treatment is more economical compared to untreated bases. It can be noticed that the use of lime treatment has cost-effectiveness in comparison to untreated bases at different traffic speeds using both asphalt binder grades. The cost-effectiveness ratio of all base treatments is found to be higher using stiffer asphalt binder and for higher traffic speed cases (Table 2).

Table 2 Cost- Effectiveness of Lime Base Treatments for Bottom-Up Fatigue Cracking

Binder Grade	Speed Limit [km/h]	Base Treatment	Remaining undamaged surface area [m²]	Cost to pave 1.6 km ($)	Cost-Effectiveness (using Eq. (2))	Cost-Effectiveness Ratio (using Eq. (3))
PG 64-22	40	Untreated (172369 kPa)	1743.53	63705	0.027	N/A
		Lime (413685 kPa)	3205.70	75229	0.043	1.56
	72	Untreated (172369 kPa)	1953.09	63705	0.031	N/A
		Lime (413685 kPa)	3835.54	75229	0.051	1.66
	104	Untreated (172369 kPa)	2086.12	63705	0.033	N/A
		Lime (413685 kPa)	3925.01	75229	0.052	1.59
PG 76-16	40	Untreated (172369 kPa)	1220.24	63705	0.019	N/A
		Lime (413685 kPa)	3002.62	75229	0.040	2.08
	72	Untreated (172369 kPa)	1802.98	63705	0.028	N/A
		Lime (413685 kPa)	3043.82	75229	0.040	1.43
	104	Untreated (172369 kPa)	1831.83	63705	0.029	N/A
		Lime (413685 kPa)	3756.07	75229	0.050	1.74
Overall Cost-Effectiveness Ratio of Lime Treated Base						1.68

*N/A relates to original untreated base layer

7 Conclusions and recommendations

The purpose of this study was to conduct a mechanistic comparative analysis between treated and untreated bases in order to evaluate bottom-up fatigue cracking resistance. The base treatment considered in this study was lime treatment. In addition, cost-effective analysis was performed to investigate if such treatment was worthwhile considering their cost versus their improved field performance. Based on both mechanistic and cost-effectiveness analyses, the following conclusions are drawn:

- In terms of bottom-up fatigue cracking performance, treated base layers had superior fatigue cracking resistance as compared to untreated sections. Lime treated base had 48 % higher FCRP.
- It can also be concluded that pavement structures with stiffer asphalt binder grade (PG 76-16) were more susceptible to fatigue cracking than softer asphalt binder grade (PG 64-22). Similarly, fatigue cracking decreased as the traffic speed increased due to the viscoelastic nature of asphalt pavements.
- Cost-effectiveness analysis showed that the use of lime treated base resulted in the highest cost effectiveness considering bottom-up fatigue cracking. The overall cost-effectiveness ratio of lime was 1.68 times the untreated base for the bottom-up fatigue cracking.

Therefore, it can be concluded that the use of base treatments could potentially contribute to an overall improved fatigue cracking resistant pavement structures. In addition, such treatments present improved cost-efficiency in base construction practices. Furthermore, this research reports the preliminary mechanistic and cost-effectiveness analysis of various base treatments based on the Texas Department of Transportation (TXDOT) practices, hence, further study based on other countries practices along with other form of distresses such as rutting and reflective cracking can lead to a geographically diverse verification of the above-mentioned analysis.

References

[1] Francois, A.A.: Evaluating the impact of different types of stabilized bases on the overall performance of Flexible Pavements, Rowan University, 2016.

[2] Little, D.N., Scullion, T., Kota, P.B.V.S., Bhuiyan, J.: Identification of the Structural Benefits of Base and Subgrade Stabilization, 1995.

[3] Jaritngam, S., Somchainuek, O., Neerananon, P.T.: Feasibility of laterite-cement mixture as pavement base coarse aggregate, Iranian Journal of Science Technology. Transactions of Civil Engineering, 38 (2014)

[4] Wu, Z., Chen, X., Yang, X.: Finite element simulation of structural performance on flexible pavements with stabilized base/treated subbase materials under accelerated loading, 2011.

[5] Johnson, A.M.: Use of limes in improving bases and subgrades, Joint Highway Research Project

[6] Azadegan, O., Yaghoubi, E., Li, J.: Evaluation of the Performance of Lime and Cement Treated Base Layers in Unpaved Roads, EJCE, 2013.

[7] Mallela, J., Von Quintus, H., Kelly. L.: Consideration of Lime-Stabilized Layers in Mechanistic-Empirical Pavement Design, ERES Consultants, Virginia, 2004.

[8] Koroma, A.A.: Evaluation of the performance and cost-effectiveness of pavement sections containing open-graded base courses, 2011.

[9] Siddharthan, R.V., Yao, J., Sebaaly, P.E.: Pavement Strain from Moving Dynamic 3-D Load Distribution. Journal of Transportation Eng, 124 (1998) pp.557-566

[10] Witczak, M., Mamlouk, M., Souliman, M., Zeiada, W.: NCHRP REPORT 762: Laboratory Validation of an Endurance Limit of Asphalt Pavement, Transportation Research Board, Washington D.C, 2013.

[11] Texas Department of Transportation, Average Low Bid Unit Prices, https://www.txdot.gov/business/letting-bids/average-low-bid-unit-prices.html. Accessed 12 June 2019, May 2019.

QUALITY CHECKING OF POLYMER MODIFIED BITUMENS IN SLOVENIA

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Abstract

For many years the properties of bitumen have been determined based on mechanical tests as needle penetration, Ring&Ball and Fraass fracture temperature. For elastomer (styrene-butadiene-styrene) polymer modified bitumens these tests are not sufficient to show the important differences in bitumens. Elastic recovery and cohesion provide better insight, but rheological properties cannot be adequately described with conventional test.
The requirements of the polymer modified bitumens (PmB) in Europe were defined in EN 14023 in 2010 [1]. Since then several new tests were introduced in the research field and their procedures improved. In the European Standards Committee (CEN) TC 336, there is ongoing work to develop performance related specifications. New laboratory test methods from American standards were adapted and transformed into EN standards (bitumen laboratory aging methods and rheological tests). These test methods are not yet employed in the current PmB European standard, however, the draft prEN 14023, April 2020 [2] suggests these new tests. In the recent years at ZAG Laboratory for asphalts and bitumen-based products long-term aging of bitumen by pressure aging vessel (PAV) and rheological tests were introduced. Traditional bitumen test methods are performed together with new rheological tests e.g. Dynamic Shear Rheometer (DSR) testing, to characterize complex modulus and phase angle, and Multiple Stress Creep Recovery Test (MSCRT) in order to develop a preliminary data base on PmB's, which are frequently used in Slovenia.
The paper presents the current requirements for PmB's in Slovenia and test results on PmB 45/80-65, original, laboratory aged and extracted from produced asphalt mixtures.

Keywords: polymer modified bitumen, recovered bitumen, laboratory ageing, dynamic shear rheometer, multiple stress creep recovery test

1 Introduction

In Slovenia polymer modified bitumens (mostly type PmB 45/80-65) are regularly used on motorways for surface (SMA11) and binder (AC22bin) asphalt layers. In the past years many samples of PmB 45/80-65 were tested at ZAG Laboratory (original, laboratory aged and extracted from asphalt). ZAG laboratory for asphalt and binder products performs quality control of asphalt mixture production and asphalt layer compaction. The testing includes samples of original bitumen as well as recovered bitumen from produced asphalt mixtures. In this way, the test results can be linked to the performance of the corresponding road sections during their life span. According to CEN rules, each country may adopt suitable requirements for their climate region or other reasons in national standards. Based on EN 14023 the national delivery requirements for PmB's were set in Slovenian Institute for Standardization (SIST) document titled SIST 1035 in 2008.

Since then the requirements, as presented in Table 1, have not been changed. There are no requirements for rheological characteristics however we started to perform these tests to develop a preliminary data base.

Table 1 National requirements for modified bitumen PmB

Conventional test methods	Standard	Unit	PmB 45/80-65
Needle penetration @ 25 °C	EN 1426	mm/10	45-80
Softening point R&B	EN 1427		≥65
Fraass breaking point	EN 12593	°C	≤ -18
Cohesion, Force ductility	EN 13589	[J/cm²]	≥ 2 at 25 °C
Elastic recovery @ 25 °C	EN 13398	%	≥80
Stability against hardening (RTFOT)	EN 12607-1		
Change of mass after RTFOT	EN 12607-1	M.-%	≤ 0.50
Change of softening point after RTFOT	EN 1427		no requirement
Retained penetration after RTFOT	EN 1426	%	≥60
Elastic recovery @ 25 °C after RTFOT	EN 13398		≥70
Additional test methods			
Dynamic Shear Rheometer (DSR)			
G* (30 °C to 90 °C) at 15 kPa (25 mm plate)	EN 14770		no requirement
Multiple stress Creep and Recovery Test			
MSCRT @ 60 °C Jnr 3.2 kPa after RTFOT	EN 16659	[1/kPa]	no requirement

2 Results of tests on original and laboratory aged bitumens PmB 45/80-65

On neat bitumen the conventional tests R&B [3], Penetration [4], Fraass [5] were performed as well as new tests MSCRT [6], and DSR [7], [8]. In our study DSR test with continuous increase in testing temperature, as described in a draft EN standard [7] and guidelines [9], was performed. According to proposed [2] temperature for G* =15 kPa and corresponding δ should be evaluated. In this research all tests were performed on 25 mm plates. Original bitumens have been laboratory aged with RTFOT [10] method and all tests were repeated. Rolling Thin Film Oven Test (RTFOT) is a laboratory method that simulates or short term ageing during mixing, transport and compaction. In the last step, the five bitumens were laboratory aged with RTFOT and PAV [11] method and then re-tested. The results are presented in Tables 2 and 3 and in Fig. 1 and Fig. 2.

Table 2 Results on neat (original) bitumen PmB 45/80-65

Sample	Pen	$T_{R\&B}$	T_{Fraass}	G* at 15 kPa	δ at $T_{G^*=15kPa}$	%R at 3.2 kPa	Jnr at 3.2 kPa
No.	[0.1 mm]	[°C]	[°C]	[°C]	[°]	[%]	[kPa⁻¹]
1	53	83.4	-18	54.0	62.9	77	0.203
2	51	80.4	-18	54.4	64.2	76	0.252
3	59	81.6	-20	53.5	58.4	96	0.028
4	47	77.0	-18	54.8	62.3	60	0.350
5	59	78.4	-19	53.6	55.5	95	0.030

The tests on original bitumen are primarily intended to control the conformity of the supplied bitumen with the indications on the declarations of performance.

Table 3 Results on laboratory aged (RFTOT) bitumen PmB 45/80-65

Sample	Pen	$T_{R\&B}$	T_{Fraass}	G* at 15 kPa	δ at $T_{G^*=15kPa}$	%R at 3.2 kPa	Jnr at 3.2 kPa
No.	[0.1 mm]	[°C]	[°C]	[°C]	[°]	[%]	[kPa^{-1}]
1	37	73.2	-12	58.6	61.6	60	0.198
2	37	74.0	-10	60.3	62.1	59	0.184
3	40	79.2	-14	59.9	55.1	91	0.033
4	34	74.8	-7	61.1	59.8	63	0.138
5	41	78.4	-12	57.6	54.4	92	0.034

The results of tests after RTFOT aging indicate the characteristics of the bitumen in freshly produced and transported asphalt mixture. Typically, PmB 45/80-65 penetration is between 30 and 40 [0.1 mm]. For Slovenia climate conditions $T_{R\&B}$ should be above 70 °C, T_{Fraass} preferably as low as possible. DSR should be performed at 1.59 Hz (10 rad·s–1). The DSR test results show that the temperature at which G* = 15 kPa, is usually between 50 °C and 60 °C. Two tested samples (MSCR test) had a recovery above 90 % at 60 °C and a non-recoverable creep Jnr at 3.2 kPa of less than 0.100 kPa^{-1}, which indicates good viscoelastic response.
For PmB 45/80-45 the conventional tests do not give relevant results for quality control. The softening point $T_{R\&B}$ of original PmB is close to 80 °C, therefore the tests should be performed both in water and in glycerol. The decrease in $T_{R\&B}$ after RTFOT ageing mainly depends on the type and characteristics of polymers used, and therefore depend on the producer of polymers. The $T_{R\&B}$ of original bitumens is much higher than the minimum required value ($T_{R\&B}$ › 65 °C). From 2003 till 2006 ZAG was included in BitVal project [12]. The Bitumen Test Validation (BiTVal) project was set up by the Forum of European National Highway Research Laboratories (FEHRL) in response to a request from TC 336, Bitumen and bituminous binders, of the CEN together with other stakeholders in the industry to assess the relevance of the results of bitumen tests on the required properties of asphalt mixtures. One of the conclusions of the BitVal project was that the $T_{R\&B}$ is not appropriate test method to designate permanent deformation for asphalt layers containing PmB.

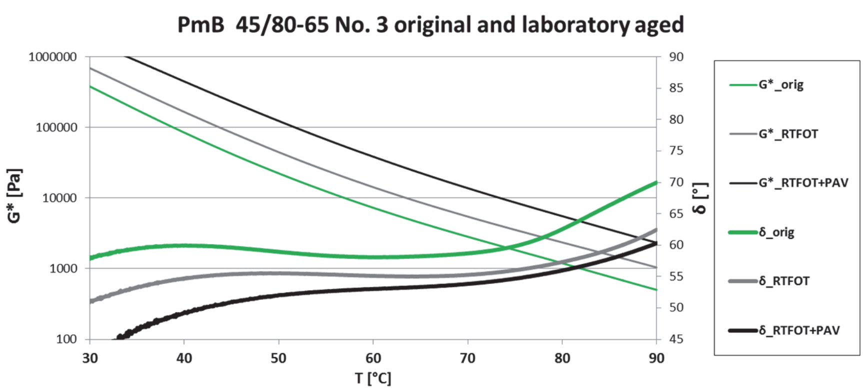

Figure 1 DSR – Temperature sweep results for a PmB 45/80-65

Fig. 1 shows results of DSR test (continuous increase of test temperature) for sample No. 3 in the range of 30°C to 90°C. The graph shows G* and δ as a function of temperature for original bitumen (_orig), after laboratory short term aging (_RTFOT) and after laboratory long term aging (_RTFOT+PAV). The bitumen gets stiffer with aging (G* increases), while δ decreases.

3 Results of rheological tests on naturally aged bitumens from SMA asphalt mixture

The same aforementioned bitumens (Samples No.1 to No. 5) were used in SMA asphalt mixtures produced in various asphalt plants. In ZAG laboratory bitumen PmB 45/80-65 extracted [13] and recovered [14] from asphalt mixtures (referred thereafter 'extracted') was investigated. For sake of clarity only results of extracted bitumens numbered No. 2, No. 3 and No. 4 (denoted by '_ex') are presented (Fig. 2). It was expected that the results would be similar to those after short-term aging (RTFOT). DSR results show that samples aged to the similar extent during mixing in the asphalt plant and during RTFOT ageing in the laboratory. However the rheological behaviour of PmB 45/80-65 of different bitumen producers can significantly differ after RTFOT ageing or mixing at asphalt plant.
We also compared the results of MSCR tests (@ 60°C) of these three bitumen samples. Comparison of non-recoverable creep compliance Jnr at 3.2 kPa and recovery R [%] at 3.2 kPa is shown in Fig. 3. The line shown in the graph is defined in AASHTO R 92-18 specifications [15] to assess elastic response for RTFOT aged samples. For heavy traffic loads MSCRT characteristics should be above this line. The results for RTFOT aged and extracted sample No. 3 are practically identical. The results for samples No. 2 and No. 4 show that the PmB's aged differently during mixing at the asphalt plant and transport than during RTFOT in laboratory. The shown differences in results of DSR and MSCRT tests may be due to the extraction and recovery process of PmB bitumen. From Fig. 3 it can be seen that all tested PmBs fulfilled the requirement in AASHTO R 92-18 specifications [15].

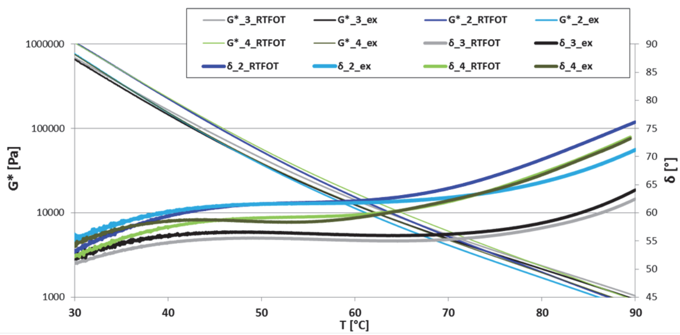

Figure 2 DSR for RTFOT aged and extracted samples No.2, No.3 and No.4

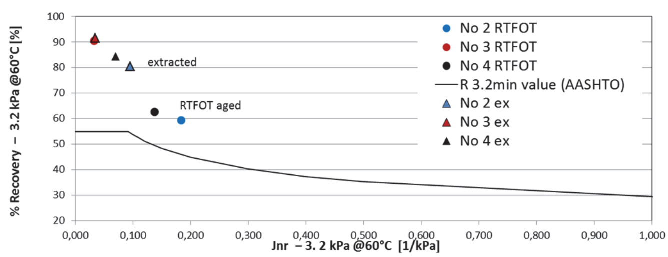

Figure 3 MSCRT results on RTFOT aged and extracted samples.

4 Results of rheological tests on naturally aged bitumens from different asphalt mixtures

In Table 4 and Fig. 4 and Fig. 5 are presented results of PmB 45/80-65 extracted in 2020 from eight different asphalt mixtures (ACbase, ACbin, SMA, PA). All asphalt samples were taken at construction sites at paver and tested in ZAG's laboratory. The results of DSR test present an impression of expected rheological characteristics G* and δ for aged PmB 45/80-65 bitumens. Fig. 4 and Fig. 5 provide a good insight into the rheological behaviour of aged PmB 45/80-65 bitumens. The complex modulus of bitumen labelled as A and C are very similar through the temperature range, while there are obvious differences for the phase angles. It should be noted that original bitumen was produced in different refineries.

Table 4 Results of extracted and recovered bitumen PmB 45/80-65 from different asphalt mixtures

Sample	Pen	$T_{R\&B}$	T_{Fraass}	G* at 15 kPa	δ at $T_{G^*=15kPa}$	%R at 3.2 kPa	Jnr at 3.2 kPa
Label	[0.1 mm]	[°C]	[°C]	[°C]	[°]	[%]	[kPa-1]
A AC22base	53	79.4	-17	53.8	60.8	93	0.052
B SMA 11	40	78.8	-19	59.8	55.9	86	0.044
C AC 22bin	57	85.0	-16	53.2	60.4	96	0.032
D AC22base	40	81.0	-18	55.5	58.7	93	0.040
E SMA11	35	71.8	-15	60.7	58.0	78	0.064
F AC22base	44	79.2	-19	54.7	61.2	91	0.065
G AC22bin	42	74.8	-17	56.7	60.2	86	0.077
H PA11	38	75.8	-18	59.2	57.5	N/A	N/A
average	42	78.1	-17	57.1	58.8	89	0.054
min	35	71.8	-19	53.2	55.9	78	0.032
max	57	85.0	-15	60.7	61.2	96	0.077

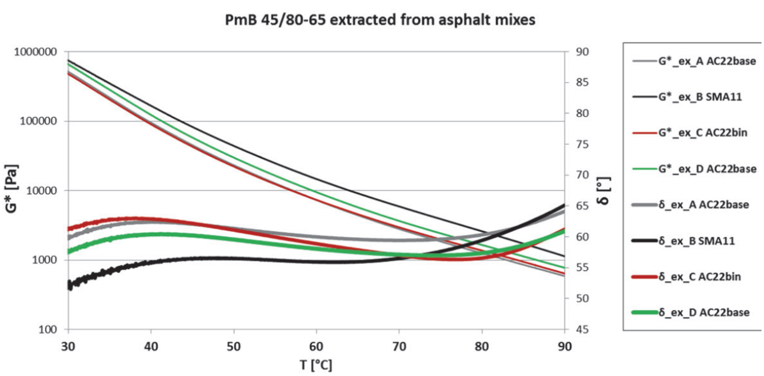

Figure 4 DSR results for PmB 45/80-65 extracted from asphalt mixes A to D.

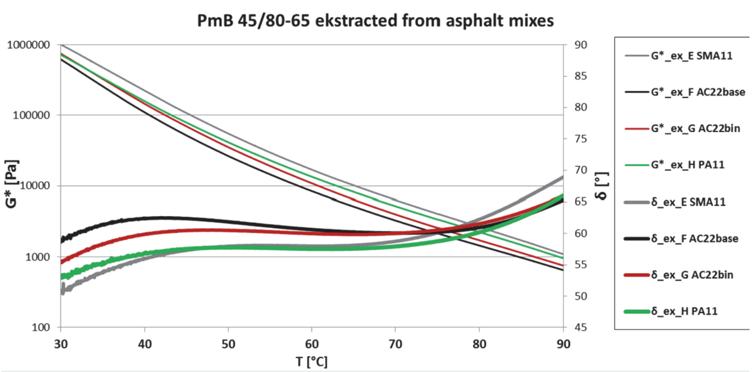

Figure 5 DSR results for PmB 45/80-65 extracted from asphalt mixes E to H.

In Fig. 6 are graphically presented MSCRT results of PmB 45/80-65 extracted from asphalt mixtures A to G in 2020. Results of MSCRT tests show that they all exhibit good elastic response, as their results lie above the AASHTO R 92-18 requirement. All PmBs included in our study fulfilled the requirement in AASHTO M 332-18 [16] specifications for extremely heavy traffic.

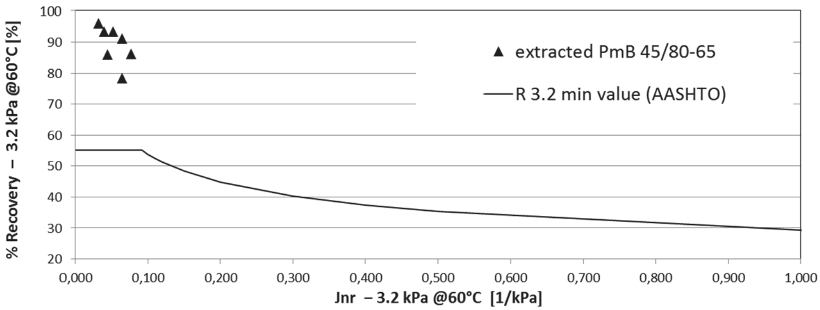

Figure 6 MSCRT results for PmB 45/80-65 extracted from asphalt mixes A to G.

5 Conclusions

From this study we concluded that test laboratories for asphalt should develop the relevant competences to perform the rheological tests, and at the same time the users (asphalt producers, road authorities, etc.) should become familiar with the results of these tests.
Goal of our study was to inform advanced and trusty paving contractors about the characteristics of the RTFOT aged bitumens, particularly PmBs, so they will be able to choose the appropriate bitumen for a specific project. Rheological DSR test, used in our study, provide good insight in bitumen behaviour through temperature range. MSCRT test provide information about elastic behaviour of bitumen, which is very important for polymer modified bitumens. The only requirement that we could rely on was AASHTO R 92-18 specification. All PmBs included in our study showed a significant elastic response for the associated value of non-recoverable creep compliance. All PmBs included in our study fulfilled the requirement in AASHTO M 332-18 [16] specifications for extremely heavy traffic. Rheological tests are thus a very good complement to conventional tests. The results show that rheological characteristics for a PmBs produced in the same refinery having an unchanging production exhibit similar rheological graphs for original bitumen and after RTFOT ageing. DSR results of extracted PmB often do not correspond to RTFOT results. We found out that results for RTFOT aged and

extracted sample No. 3 are practically identical, but the results for samples No. 2 and No. 4 show that the PmB's aged differently during mixing at the asphalt plant and transport than during RTFOT in laboratory.
For optimization of design of asphalt pavements long term monitoring of road sections should be performed. It is beneficial that a database of performance of laid asphalt and bitumen is formed. With sufficient quantitative data limit values will be set for the results of rheological tests, which will serve for quality control, for both original and (laboratory and naturally) aged bitumens.

References

[1] EN 14023: Bitumen and bituminous binders, Specification framework for polymer modified bitumens, 2010.

[2] DRAFT EN 14023: Bitumen and bituminous binders - Specification framework for polymer modified bitumens, 2020.

[3] EN 1427: Bitumen and bituminous binders-Determination of softening point - Ring and Ball method, 2015.

[4] EN 1426: Bitumen and bituminous binders - Determination of needle penetration, 2015.

[5] EN 12593: Bitumen and bituminous binders-Determination of the Fraass breaking point, 2015.

[6] EN 16659: Bitumen and bituminous binders - Multiple Stress Creep and Recovery (MSCRT), 2016.

[7] EN 17643: Bitumen and bituminous binders - Determination of equishear modulus temperature using a Dynamic ShearRheometer (DSR) - BTSV test, 2021.

[8] EN 14770: Bitumen and bituminous binders - Determination of complex shear modulus and phase angle - Dynamic Shear Rheometer (DSR)., 2012.

[9] Forschungsgesellschaft für Straßen- und Verkehrswesen, Forschungsgesellschaft für Straßen- und Verkehrswesen, 2017.

[10] EN 12607-1: Bitumen and bitumionus binders – Determination of the resistance to hardening under the influence to heat and air – Part 1: RTFOT, 2014.

[11] EN 14769: Bitumen and bituminous binders - Accelerated long-term ageing conditioning by a Pressure Ageing Vessel (PAV)., 2012.

[12] BiTVal - Analysis of Available Data for Validation of Bitumen Tests, Final report pp. 66-67, FEHRL, 2006.

[13] EN 12697-1: Bituminous mixtures - Test mehods - Part1: Soluble binder content, 2020.

[14] EN 12697-3: Bituminous mixtures - Test mehods for hot mix asphalt - Part 3: Bitumen recovery_Rotary evaporator, 2013.

[15] AASHTO R 92-18: Evaluating the Elastic Behavior of Asphalt Binders Using the Multiple Stress Creep Recovery (MSCR) Test, 2018.

[16] AASHTO M 332-18: Standard Specification for Performance-Graded Asphalt Binder Using Multiple Stress Creep Recovery (MSCR) Test, 2018.

POTENTIAL SUBSTITUTIONS OF TRADITIONAL HYDRAULIC BINDERS IN COLD RECYCLED MIXTURES USING BLAST FURNACE SLAG

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Abstract

Cold recycling techniques are known for decades in pavement engineering as a suitable rehabilitation method mainly for existing asphalt pavements. Traditionally the most common solution is to use bituminous emulsion or foamed bitumen as a binder usually in combination with small amount of cement or lime as active fillers. In some countries cement or hydraulic road binders are preferred instead of bitumen based options since it is believed that hydraulic binders can increase the bearing capacity of cold recycled layer especially for pavements with underestimated structures which were designed ›40 years ago. Based on that the Faculty of Civil Engineering, CTU Prague is for more than 10 years evaluating and developing further alternatives for the cement-based approach of cold recycled mixtures. In the past experience with fly-ashes or activated fly-ash based alternative binders were presented. Presently the focus is concentrating on the potentials of using blast furnace slags which are not generally usable for the cement industry (e.g. because of limited content of glassy compounds which are very typical mainly for granulated blast furnace slags). Air-cooled blast furnace slags were selected and activated by high-speed milling to get a material with latent hydraulic properties. This modified slag was applied in several options to cold recycled mixtures and standard strength and deformation tests were performed, including the determination of resistance to water immersion. Separately pastes based on used treated fine-grained slags were tested and evaluated. Data of the pastes are discussed jointly with the results for experimentally tested cold recycled mixtures.

Keywords: cold recycled mixtures, blast furnace slag, fly-ash, cement substitution, strength properties, reclaimed asphalt

1 Introduction

In the last five years there were several experimental tasks which focused on the potentials of using alternative hydraulic binders which would effectively use existing by-products or wastes. The CTU Prague started firstly with fly-ash binders like DASTIT or SORFIX, which were invented in the Czech Republic and are applicable to hydraulic bon mixtures or concretes. Later it was decided to seek for suitable combinations or effective use of some types of slags including eventually necessary activating agents. The aim was to replace traditional cement where it would be possible and suitable. In case of utilizing alternative hydraulic binders traditional characteristics for the particular mixtures are analyzed – in case of cold recycled mixtures it is indirect tensile strength and resistance to water immersion. In most cases these results are accompanied by testing of cement pastes, which is important to precisely

understand the potentials of cement substitution, especially with respect to the suitable level of substitution.
The cold recycling technology is a well-established technique and there have been presented in the last 20 years many scientific papers, conference contributions and research reports (e.g. [1, 2, 3, 4, 7, 9, 13]) focusing on different aspects of characterizing this technology, manners of carrying out cold recycled pavement layers by using either bituminous binders with cement as well as studying performance related characteristics (e.g. [5, 6, 11, 12]) and advanced test methods. There are also diverse papers where several cement substituting binders are used in these composites (e.g. [8, 10, 14–17]). The reference list would be in reality very spacious and only a review paper cover many pages). At the same time it shall not be forgotten, that in the last 10-15 years some European research projects were realized as well, e.g. SAMARIS, Re-ROAD, CoRePaSol, SCORE etc. They helped to extend the knowledge about cold recycling.
Cold recycling is widely used in the practice for repair works and rehabilitations of pavements. In case of using the in-situ option the construction process can be accelerated ad be made more efficient. The true advantage is the fact that the existing pavement is 100 % reused by this kind of recycling often without the necessity of hauling huge amounts of building materials. It helps also to reduce waste creation and in case there are in the pavement some environmentally problematic materials used in the past, like e.g. tar, they can be safely treated and embedded in the pavement structure again. This helps to avoid not only the creation of dangerous waste but provides also effective protection to air and water pollution.
Since the early use of cold recycled mixtures in the Czech Republic the technological option based on combination of hydraulic binder – usually portland cement – and bituminous binder gained the highest popularity. There were even many road projects where the contractor gave precedence to hydraulic road binder, which must fulfill the criteria of EN 13282-1 or 2. The reason for the mentioned combination of two binders and even preference for solutions with only hydraulic binders is advocated by securing higher strength properties, which helped to increase the bearing capacity of the recycled pavement layer. Of course such approach needs some caution since a stiffer pavement layer can in these cases result in higher susceptibility to cracking. Nevertheless the regular use of cement in cold recycled mixtures has provided interesting potential for substitutes which would be suitable to be used instead of cement.

2 Used materials and cold recycled mix designs

The objective of the more sample experimental study, partly covered by theses [18, 19] was to evaluate suitable substitutions of regularly used cement in cold recycled mixtures by variants of blast furnace slags (BFS) eventually combined or compared with fly-ash based binders. For the preparation of experimental mixtures granular material was used which was formed by combination of granular base material 0/32 mm and reclaimed asphalt RA 0/11 mm with 1:1 ratio or entirely reclaimed asphalt 0/22 mm or site-won asphalt of particle size 0/45 mm and 0/32 mm. Designed cold recycled mixtures contained always slow-setting bituminous emulsion C60B10 and some of the traditional or alternative hydraulic binders as described further. The content of these binders varied in the proposed mix designs. For the tested experimental series the intension was to keep the bituminous emulsion content always the same to avoid too many factors which could influence the resulting mix characteristics.
All compared experimental mixtures (see Tables 2, 3, 4) and gained results for test specimens were compared with corresponding reference mixtures (REF) by laboratory tests as provided by TP 208. Commonly required test were for some mixtures supplemented by determination of stiffness using the repeated indirect tensile stress test (ČSN EN 12697-26, method C) and by determination of resistance to crack propagation according to ČSN EN 12697-44 (not pre-

sented in this paper). These characteristics were usually tested on specimens cured for either 28 days or 56 days. The already mentioned ratio of granular base material and reclaimed asphalt used in one of the test series was selected to provide conditions and especially cold recycled mix composition which will be in terms of mix properties and material composition as close as possible to the conditions of a pavement where it was assumed to be used. In general it is normally not common in the Czech Republic that the cold recycled layer is formed only by a single-kind material and ordinarily there is some ratio of granular base material and site-won asphalt. Of course there are specific exceptions, like the modernization of the key Czech motorway D1, where original cement bond base course was cold recycled. On the other hand the above mentioned rule is well represented by mixed reclaimed pavement material where usually asphalt layers and part of the base layer are treated together. Such material used in this paper originates from a 3rd class road and introduces a good example of common practice.
The composition of reference cold recycled mixtures is presented in Table 1. The alternative cold recycled mixtures with cement substitutions are presented in Tables 2, 3, 4. Grading curves and modified Proctor test result are in more detail presented in [23].

Table 1 Composition of the reference mixtures

Mix constituents [%]	Mix design		
	REF 1	REF 2 (R1)	REF 3 (G1)
Granular base material 0/32 mm	45,05	–	–
Mixed reclaimed pavement material 0/32 mm	–	–	63,1
Reclaimed asphalt 0/11 mm	45,05	–	27,1
Reclaimed asphalt 0/22 mm	–	90,0	–
Water	3,9	3,5	3,8
Bituminous emulsion	3,0	3,5	2,0
Cement CEM II 32,5R	3,0	3,0	4,0

Table 2 Composition of the cold recycled mixtures, series I [18]

Mix constituents [%]	Mix design					
	A	B	C	D	E	F
Granular base material 0/32 mm	44,0	44,1	43,85	43,45	43,85	43,4
Reclaimed asphalt 0/11 mm	44,0	44,1	43,85	43,45	43,85	43,4
Water	4,0	3,8	4,3	5,1	4,3	5,2
Bituminous emulsion	3,0	3,0	3,0	3,0	3,0	3,0
Cement CEM II/ 32,5R	1,0	1,0	1,0	1,0	1,0	–
MS-PT (ladle slag)	4,0	–	–	–	–	–
MS-KVP (air cooled BFS)	–	4,0	–	–	–	–
MS-TG (granulated BFS)	–	–	4,0	–	–	–
MS-TG + DASTIT fly-ash binder (ratio 1:1)	–	–	–	4,0	3,0	4,0

Table 3 Composition of the cold recycled mixtures, series II [19]

Mix constituents [%]	Mix design							
	R2	R3	R4	R5	R6	R7	R8	R9
Reclaimed asphalt 0/22 mm	90	90	90	91	91,5	90	90	90
Water	3,5	3,5	3,5	3,5	3,0	3,5	3,5	3,5
Bituminous emulsion	3,5	3,5	3,5	1,5	3,5	3,5	3,5	3,5
Cement CEM II 32,5R	–	–	–	4,0	2,0	–	–	–
Commercial road binder TB	–	–	–	–	–	–	3,0	–
MS-TG (granulated BFS)	3,0	–	–	–	–	–	–	–
MS-KVP (air cooled BFS)	–	3,0	–	–	–	–	–	–
Milled recycled concrete (D2)	–	–	3,0	–	–	–	–	–
Ternary fly-ash based binder SORFIX	–	–	–	–	–	–	–	3,0
DASTIT fly ash binder with CEM I (ratio 4:1)	–	–	–	–	–	3,0	–	–

Table 4 Composition of the cold recycled mixtures, series III

Mix constituents [%]	Mix design								
	G2	G3	G4	G5	G6	G7	G8	G9	G10
Mixed reclaimed pavement material 0/32 mm	63,1	63,1	63,1	63,7	63,1	63,1	63,1	62,5	63,1
Reclaimed asphalt 0/11 mm	27,1	27,1	27,1	27,5	27,1	27,1	–	26,3	27,1
BFS 0/4 mm	–	–	–	–	–	–	27,1	–	–
Water	3,8	3,8	3,8	3,8	3,8	3,8	3,8	4,2	3,8
Bituminous emulsion	2,0	2,0	2,0	2,0	2,0	2,0	2,0	2,0	2,0
Cement CEM II 32,5R	–	–	–	3,0	–	–	–	–	–
Cement CEM I 42,5R	–	–	–	–	0,5	0,5	0,5	–	0,5
MP-TTA with CEM I	–	4,0	–	–	–	–	–	5,0	–
MP-TTA with $Ca(OH)_2$	–	–	4,0	–	–	–	–	–	–
MS-KVP : fly-ash from fluidized combustion (1:1)	–	–	–	–	–	–	–	–	3,5
Ternary fly-ash based binder SORFIX	4,0	–	–	–	3,5	–	–	–	–
Fly-ash based binder DASTIT	–	–	–	–	–	3,5	3,5	–	–

The assessed mix variants differ in use of either slag or fly-ash of manifold origin. Combinations were used as well. All these byproducts were firstly treated by high-speed grinding (mechanical chemical activation). In the Czech Republic the BFS sources are limited to Kladno, Třinec and Ostrava. Further option to these locations can be found e.g. in Linz (Austria). Fly-ash based binder DASTIT® is actually produced only in Pilsen as a fully certified product which is mostly used for solidifications of clays, argillaceous rocks and sludge. Utilization in cold recycled mixtures was firstly verified by a trial section on a rural road II/118 near Prague [17]. Another example of fly-ash utilization is ternary binder SORFIX, which is a product of two fly-ash types, which are combined with lime hydrated and chemical additives. Last but not least a commercially available hydraulic road binder TP was used for sake of comparison within the second test series. Similarly high-speed ground recycled concrete of original particle size 0/4 mm was used as well. In this case the material originated from the reconstruction of D2 motorway Brno – Bratislava.

In the case of used BFS the marking MS-PT stays for ladle slag from Třinec, where CaO is found in trace amount. Chemically this slag is dominated by SiO_2 and Fe_2O_3. MS-KVP sample represents stabilized air-cooled blast furnace slag which is stored for decades in the city of Kladno. The material was landfilled on the existing stockpile between the beginning and the 80´ of the last century. The BFS was high-speed milled by the partnering company Lavaris, whereas the determined surface area according to Blaine was 370 m^2/kg. Higher content of free CaO (around 38 %) is a good prerequisite for latent hydraulic activity of this material. It can be expected, that such BFS can be used as partial substitute for traditional cement. This blast furnace slag has again also higher contents of SiO_2 and Al_2O_3. The particle size distribution was within the interval of 0,1–50 μm, whereas the highest count was found for particles between 10 μm and 15 μm. The activity of this slag and other slag-based binder or active fillers is expressed by the efficiency index which was tested according to EN 15167-1 (par. 5.3.2.3). These results, however, are not part of this article, some of the findings can be found e.g. in [22]. The MS-GT sample represents granulated (water cooled) BFS which was ground by high-speed disintegrator and originates in Třinec. The surface area of this sample was only 84 m^2/kg. During further steps of milling (results for these options are not presented in this paper) it was possible to reach a better surface area of 250-300 m^2/kg. Originally very low surface area might be interpretable also by the particle size distribution, as was determined by laser granulometry. Interval of 0,1–250 μm was reached, whereas the highest count was found for particles between 40 μm and 100 μm. This BFS is dominated by SiO_2, CaO (42 %) and MgO. Spectroscopic XRF analyses of the described slags can be found e.g. in [18].

3 Experimental results

Findings obtained from three selected series of cold recycled mixtures, whose design is defined above, are presented in this section. Reclaimed material used in these series was obtained from different locations, and therefore potentials or benefits of it can be observed for some variants, where the same type of alternative hydraulic binder is used. In case of other tested alternative binders, the selection is based on the gradual assessment of inputs like fly ashes from fluidized combustion and blast furnace slags (BFS). In general, the intention is to focus on wastes which are not commonly used in civil engineering (commonly used are e.g. granulated slags used in cement industry, or silica fly-ashes, which traditionally are very well used in concrete production). In many cases, tested fly-ashes and slags have not been used regularly due to concerns of their uncontrolled activity or considerable heterogeneity of such materials. This is, after all, one of the reasons why CTU in Prague has been over a long period supporting the development of high-speed milling technology, which helps to minimize many of these weaknesses in presented wastes or by-products.

In case of series “I” cold recycled mixtures, in the first step variants REF, A, B, C and D were evaluated by indirect tensile strength. The minimum values of indirect tensile strength R_{it} as defined by Technical Specifications TP 208 valid in the Czech Republic were not fulfilled by variant A (0,27 MPa) and variant B (0,30 MPa). The other two variants (C, D) met this minimum requirement without any problems. It should be point out that ladle furnace slag used in variant A has generally lower activity, but on the contrary, as it has been shown in other applications it can improve workability of the mixture. Its composition and especially the relatively large stockpiles, that do not find any reasonable application, could perhaps be used as alternative filler in asphalt mixtures. Variant B is based on blast furnace air-cooled BFS, which works better with an increased amount of a suitable activator or in combination with a suitable type of CaO rich fly-ash, as indicate results of series “III” as well.

Table 5 Volumetric and strength properties of cold recycled mixtures – series I

Variant		**REF1**	**CR-A**	**CR-B**	**CR-C**	**CR-D**	**CR-E**	**CR-F**
Bulk density (before test)	[g.cm^{-3}]	2,287	2,297	2,299	2,298	2,291	2,288	2,292
Air voids content	[%-vol.]	13,71	13,31	13,28	13,33	13,58	13,71	13,55
Moisture content of fresh mixture	[%-mass]	3,89	3,99	3,79	4,26	5,07	4,33	5,21
Indirect tensile strength, 7 days ($R_{it,7}$)	[MPa]	0,48	0,33	0,30	0,35	0,43	0,35	0,37
Indirect tensile strength, 7+7 days ($R_{it},_{7+7}$)	[MPa]	0,42	0,27	0,30	0,32	0,43	0,38	0,35
Water susceptibility	[%]	87,3	81,9	101,2	91,1	98,9	106	104,5
Indirect tensile strength, 7 days ($R_{it,28}$)	[MPa]	1,10	0,71	0,62	0,80	0,91	0,83	0,77
Indirect tensile strength, 7 days ($R_{it,56}$)	[MPa]	1,15	0,86	0,81	0,81	1,15	-	-

Furthermore, comparing the results of variants REF and C, the variant C did not reach higher strength. Likewise the variant D reached slightly lower ITS in comparison to reference variant (D = 0,43 MPa vs. REF = 0,48 MPa), but the water susceptibility exceeded in this case the reference mixture. Even the individual indirect tensile strength $R_{it,7+7}$ was higher for variant D (D = 0,43 MPa vs. REF = 0,42 MPa).
In the first phase of testing, the variant D, which contained 1.0 % cement and 4.0 % blended binder of granulated and milled BFS with DASTIT fly-ash based binder, performed best compared to the reference mixture. For this reason, two additional variants of cold recycled mixtures (E, F) were subsequently designed on the basis of variant D. These variants contained the same binder (blend of BFS from the same source and DASTIT), only the amount of blended binder varied. Although these last two mixtures met the requirements of TP 208 for minimum values of indirect tensile strength ($R_{it,\ 7\ min}$ = 0.30 MPa) and water susceptibility (min. 75 %), the results were below the expected values compared to the reference mixture.
If we focus only on water susceptibility (ITSR), the limit was fulfilled for all recycled mixtures with a considerable reserve. The best result was achieved by the variant E (1.0 % cement + 3.0 % MS-GT slag + DASTIT), for which the value of ITSR exceeded 107 %, i.e. the mixture gained additional strength due to the effect of water conditioning, which can be for mixtures with increased amount of hydraulic binder explained by additional hydration and hardening of composite matrix.
Slower, but over time continuing strength increase if BFS-based binders are used corresponds well with the strength development of variant D (see $R_{it,7}$, $R_{it,28}$ and $R_{it,56}$). While in the case of ITS after 7 and 28 days this variant showed still lower strength than reference mixture containing standard portland cement, after 56 days the strength characteristics were equal. It can be assumed that after this time the proposed substitution is fully comparable with the reference mix, while this solution will be more cost effective and at the same time it can be declared as low-emission, since for the production of alternative hydraulic binders a low-energy disintegrator and existing by-products, which are anyway produced by the human activity, are used. On the contrary, the cement cannot be manufactured without emitting higher amounts of CO_2.
The second series of cold recycled mixtures differed in several aspects. Firstly, only reclaimed asphalt with maximum particle size of 22 mm (without use of additional aggregates) was used. Furthermore, the amount of water was same for all mix variants, so the fact that for example fly-ash normally requires a slightly increased content of water to reach optimal

humidity, was not reflected. The amount of water was only reduced in case of the variant R6, where the proportion of cement in the mixture was reduced. The effect of increased water consumption is evident from the results of fresh mix moisture – the reference mixture showed the highest value.
In terms of physical properties, the slightly different bulk density is obvious, the variants R1 to R3 represent either a reference mixture with cement or variants with milled BFS as a substitute for the cement. On the contrary, the variants R7 to R9 contained either fly-ash based binders or commercial hydraulic road binder TB according to ČSN EN 13282-1. The density of a fly-ash is generally lower than that of cement or BFS, and this is reflected in the bulk density of cold recycled mixture. However, this aspect does not have any influence on air voids content, whose comparison is problematic for heterogeneous materials like reclaimed asphalt.

Table 6 Volumetric and strength properties of cold recycled mixtures – series II

Variant		R1	R2	R3	R4	R5	R6	R7	R8	R9
Bulk density	[g.cm^{-3}]	2,249	2,239	2,220	2,175	2,164	2,179	2,151	2,166	2,092
Air voids content	[%-vol.]	10,04	10,46	12,83	13,73	15,00	11,51	13,20	9,97	13,01
Moisture content of fresh mixture	[%-mass]	6,1	5,6	6,0	5,6	5,6	5,9	5,2	4,9	5,7
Water suscept. (ITSR)	[%]	101,6	86,5	95,0	118,0	96,3	117,2	107,2	91,2	125,6

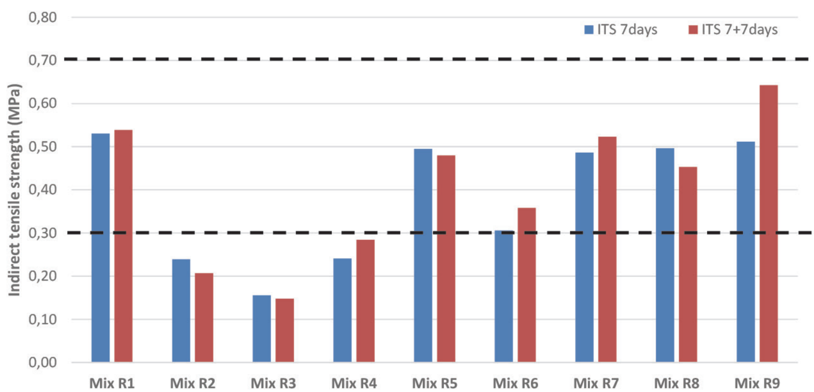

Figure 1 Indirect tensile strength of cold recycled mixtures – series II

The results mentioned above probably caused the differences in strength properties, which are shown in Figure 1. In this series, both variants with milled BFS completely failed. Similarly, the variant R4, where micro-milled recycled concrete was applied, did not reach the minimum required value of $R_{it,7}$. However, it should be emphasized that, on the contrary to series "I", the mixtures with alternative binders did not contain any cement as an activating additive. This obviously plays an important role in the case of latent hydraulic binders, which blast furnace slags undoubtedly are.
To certain extent, the comparison of R1 and R5 is surprising since the latter mixture contained increased amount of cement and reduced amount of bituminous emulsion. This combination does not always necessarily need to lead to better strength results. The variant with a ternary fly-ash based binder SORFIX (R9) clearly achieved the best results. In this case, it is necessary to pay attention to the increase in strength, which occurred after 7 days in water

bath conditioning. The water immersion significantly increased the activation of the fly-ash, which, unlike from slags, had sufficient amount of $Ca(OH)_2$ as an activating additive.
What is necessary to point out, both tested fly-ash binders can work very well as an alternative to hydraulic binders and can effectively substitute cement. The economic impact of price reduction for alternative binders would be at least 20-25 %. A certain disadvantage of these binders is lower bulk density, which partially complicates their usage – it is in these cases necessary to spear such binders in front of the recycler in a thicker layer. This can cause problems with even small wind gusts, when the binder can be blow away in front of the recycler. The solution of this problem could be for example use of binder suspension instead of dry binder spreading.
The third test series was based on the results of previous two. For its performance another type of mixed site-won asphalt was obtained (material resulted from milling 20 cm of an existing asphalt pavement). This material was combined with RA 0/11 mm to adjust particle size distribution of the final mixture. The ratio of the site-won asphalt and RA 0/11 mm was 70:30. It is not possible to make any conclusions from volumetric characteristics. The variants G1 and G5 contained only cement. The variant G10 combined fly-ash from fluidized combustion with blast furnace slag. The remaining mixtures were variations of different types of fly-ash binders. The lightest mixture seems to be the one, which combines chemically very active high speed milled fly-ash from fluidized combustion with lime hydrate (mix G4). Lime hydrate is present in the ternary binder SORFIX (mix G2) as well.

Table 7 Volumetric and strength properties of cold recycled mixtures- series III

Variant		**G1**	**G2**	**G3**	**G4**	**G5**
Bulk density	[g.cm^{-3}]	2,307	2,273	2,254	2,206	2,425
Air voids content	[%-obj.]	12,4	12,4	13,1	14,4	9,8
Moisture content of fresh mixture	[%-hm.]	3,9	4,3	4,5	4,5	3,6
Water susceptibility (ITSR)	[%]	101,5	62,3	103,4	68,0	106,2
Variant		G6	G7	G8	G9	G10
Bulk density	[g.cm^{-3}]	2,330	2,320	2,314	2,247	2,239
Air voids content	[%-obj.]	13,8	14,2	14,4	16,7	14,6
Moisture of fresh mixture	[%-hm.]	3,8	3,6	4,3	3,6	3,6
Water susceptibility (ITSR)	[%]	100,4	98,1	94,9	79,5	102,2

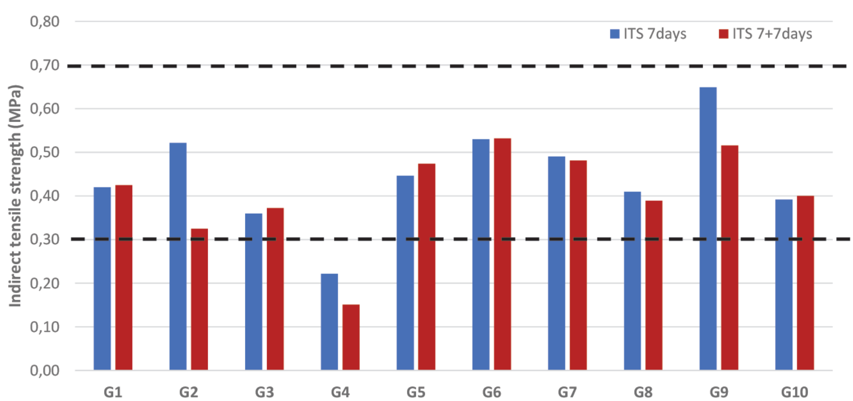

Figure 2 Indirect tensile strength of cold recycled mixtures – series III

The results may thus indicate the effect of slightly different densities of different fly-ash based binders. In terms of air voids content, only mix G5 (reduced cement content if compared to the reference mix) and mix G9 (increased amount of fly-ash based binder "MP-TTA+CEM I") deviate from quite narrow interval of gained results.
From the results of indirect tensile strength (Figure 2), it is obvious that only variant G4, containing fly-ash binder MP-TTA+$Ca(OH)_2$, does not meet the required limits given in TP 208. In all other cases, the limits were exceeded with good comparability of the mixtures. The highest $ITS_{7\ days}$ were achieved by mixtures G2, G6 and G9, i.e. variants where SORFIX binder was applied or 5 % of fly-ash binder MP-TTA+CEM I was used. Interesting is also the comparison of the reference mix and G5. The reference mix design was based on requirements for the rehabilitation of 2nd class road. In this particular case, the comparison shows that higher cement content does necessarily not provide better results. After all, in this comparison, the resistance to water susceptibility (ITSR) is also better for the variant G5.
In terms of water susceptibility, the behaviour of cold recycled mix variants can be assumed as relatively stable, and the water doesn´t have any considerable effects on mix properties. This is a relatively common phenomenon for cold recycled mixtures with hydraulic binders. Only the variant G2 did not meet the requirements of TP 208 and variant G9 showed a significant decrease in strength characteristics. For G2 it can be stated that in comparison with mix G6 it is obvious that in case of Sorfix fly-ash based binder SORFIX it is probably effective to combine it with at least small amount of cement, which subsequently helps to keep water susceptibility of the mixture within required limits.

Table 8 Stiffness modules of cold recycled mixtures

Mix variant	Stiffness modulus @ T(°C) [MPa]			Thermal susceptibility
	0	15	27	S_0/S_{27}
REF 1	12 258	8 056	6 706	1,83
CR – A	7 764	4 449	3 331	2,33
CR – B	6 794	4 874	3 441	1,97
CR – C	9 196	6 834	4 663	1,97
CR – D	11 658	8 447	6 269	1,86
CR – E	12 239	7 849	7 363	1,66
CR – F	10 500	6 326	4 997	2,10
REF – R1	8 684	4 982	3 172	2,74
R2	6 051	2 339	841	7,19
R3	6 445	2 438	985	6,54
R4	4 834	1 989	999	4,84
R5	9 974	8 147	6 193	1,61
R6	5 938	3 886	2 557	2,32
R7	7 424	4 725	3 207	2,31
R8	6 762	4 292	3 204	2,11
R9	6 897	4 708	3 219	2,14

The results of stiffness modulus correlates well with strength properties of mixtures of series "I" and "II". The stiffness modulus was determined at 3 test temperatures (0°C, 15°C and 27°C). Thermal susceptibility was calculated as a ratio of stiffness for the lowest and highest

determined temperature. This characteristic is always significantly lower for cold recycled mixtures than for hot asphalt mixtures.
In case of series “I” it should be emphasized that stiffness of mix D was after 28 days already fully comparable to the reference mixture. It can be even stated that mix E, with a slightly smaller amount of alternative binder, is also well comparable.
The results of mixtures from series “II” are totally opposite and none of the conclusions apply in this case. The highest stiffness values (Table 8) were achieved by the mix R5 designed with higher cement content. This mix variant, due to the reduced content of bituminous emulsion, resulted also in a very low thermal susceptibility. It is apparent that substitution of cement by existing commercial hydraulic road binder (TB) or fly-ash based binder (DASTIT, SORFIX) lead to similar stiffness values. This supports the legitimacy of such kind of substitution in production of cold recycled mixture on site.

4 Conclusions

Following the European-wide effort to reduce energy demand, carbon footprint as well as polluting emissions it would be pity if prospective alternative binders were not be considered as an option. Several secondary resources as by-products of the industrial activities will always exist. These by-products already are represented by a certain amount of embodied energy and there is a carbon-footprint related to their production. There conversion to suitable products normally does not involve in no manner significant increment of energy demand and they can be therefore considered as a “green” solution. Moreover, if we prioritize reasonable recycling of these materials instead of their landfilling, we might reach additional economic added values.
From the realized experimental tests and comparisons of various alternative binders it is possible to conclude that the substitution of traditional cement by fly-ash or BFS based binders is possible and technically even provides justification. It is usually important that the alternative latent hydraulic material is complemented by a suitable activating agent, which can be small portion of cement, hydrated lime etc. Another important aspect without doubts is the influence of particle size the by-products can have. Results presented in this paper have again confirmed the previous findings of the research done at CTU in Prague using high-speed disintegrators for milling, homogenisation and mechanical activation of fly-ash or slag materials. Production costs and energy demand of such process are disproportionally lower that for the traditional cement production. At the same time investment demanding ball mills are not required. In applications, where we are not expecting compression strength for the resulting pavement structures higher than 20-25 MPa, alternative binders or cement substitutes based on fly-ash or slags are a reasonable solution. Nevertheless, the potential will be most probably reachable also for higher strength classes (so far done tests for shotcrete, self compacting concrete or concrete pavement mixtures have indicated such possibility). It is therefore important to stress, that attention is paid to fly-ashes from fluidized combustion and to those BFS, which are not traditionally used for blended cements. If the mentioned potentials would be combined with construction volumes in infrastructural projects the economic leverage of these alternatives could be significant and the contractors could gain inconsiderable cost savings – price for alternative fly-ash or BFS based binders usually do not exceed 55-60 EUR/t (cement price is 75-80 EUR/t), in contrary some solutions can even reach level of max. 45 EUR/t. Carrying out a very simplified comparison for mix variants presented in this paper one can quickly discover the economic benefit of proposed cement substitutions.
The persisting concern in many regions is already the term “slag” or “fly-ash” itself. Many sub-types of these industrial by-products are stigmatized by earlier problematic applications in some construction project or by general distrust just because these materials can contain

higher portions of free CaO and there can be a higher risk of uncontrolled reactions and volume changes. It is nevertheless important to keep in mind that the amount of such fly-ashes or slags in hydraulic bond mixtures or similar composites is quite low which keeps the potential risk in controllable range. Additionally processes like the used high-speed disintegration helps to further avoid some of the problems related to unfavourable reactions.

Remark

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References

[1] AUSTROADS: Technical Report – Review of Foamed Bitumen Stabilisation Mix Design Methods, Austroads Publication No. AP–T178/11, 2011.

[2] Asphalt Academy: Technical Guideline: Bitumen Stabilised Materials: A Guideline for the Design and Construction of Bitumen Emulsion and Foamed Bitumen Stabilised Materials. Second edition. South Africa, Pretoria, 2009.

[3] Valentin, J., Mondschein, P.: Foam asphalts in cold recycling - current foreign experience. Reuse of materials for construction purposes, Štrbské Pleso, Košice: House of technics ZSVTS, pp. 117–122, 2006.

[4] Valentin, J., Mondschein, P.: Comparison of selected properties of cold recycling mixtures with asphalt emulsion and asphalt foam, VIII. scientific conference of the Faculty of Civil Engineering TU in Košice, Košice, 2007, pp. 53-58

[5] Valentin, J., Mondschein, P.: Monitoring of selected rheological properties (stiffness and fatigue modulus) of different cold recycling mixtures, Realization and economics of constructions, Štrbské Pleso, Košice: House of technics ZSVTS, 2007, pp. 123-131

[6] Valentin, J., Mondschein, P.: Analysis of stiffness modulus characteristics in selected cold recycling mixtures, Asphalt roads 2009, České Budějovice, Praha, Pragoprojekt, pp. 1–11, 2009.

[7] Batista F., Valentin J., Čižkova Z.: Report on available test and mix design procedures for cold-recycled bitumen stabilised materials, CEDR Transnational Road Research Programme Call 2012: Recycling: Road construction in a post-fossil fuel society, 2014.

[8] Faltus, M.: New types of hydraulic binders based on waste materials, Ecology and new building materials and products, 2009.

[9] Suda, J., Čížková, Z., Valentin, J., Šedina, J., Valentová, T.: Influence of test specimen compaction and curing condition for pavement cold recycled mixtures, 12th International Symposium on Concrete Roads, Prague, 2014, pp. 148–149

[10] Souček, V., Suda, J., Matoušek, D., Kubinčanová, L.: Mixtures bonded with alternative binders with the use of by-products in road construction, Juniorstav 2011, Technical University in Brno, 2011.

[11] Krpálek, O.: Deformation behavior of cold recycling mixtures with different types of binders, Bachelor thesis, Faculty of Civil Engineering, CTU in Prague, 2013.

[12] Valentin, J.: Problems of cold recycling mixtures in terms of selected functional characteristics, dissertation, Faculty of Civil Engineering, Czech Technical University in Prague, 2009.

[13] Čížková, Z.: Problems of Laboratory Testing of Cold Recycled Asphalt Mixtures, dissertation, Faculty of Civil Engineering, Czech Technical University in Prague, 2017.

[14] Suda, J.: Design and evaluation of recycled and emulsion asphalt mixtures using energy by-products, dissertation, Faculty of Civil Engineering, Czech Technical University in Prague, 2018.

[15] Šedina, J.: Alternative Types of Binders in Bonded Mixtures, dissertation, Faculty of Civil Engineering, Czech Technical University in Prague, 2019.

[16] Suda, J., Valentin, J.: Use of mechanically activated fluid fly ash in cemented cold recycling mixtures, Recycling 2012 - Possibilities and perspectives of construction waste recycling as a source of full-fledged raw materials, Technical University, Faculty of Mechanical Engineering, Brno, 2012, pp. 141–147

[17] Valentin, J.: Findings from the implementation of a test section with the application of cold recycling technology and an alternative binder based on activated fluid ash (DASTIT®), RECYCLING 2018 - Possibilities and perspectives of construction waste recycling as a source of full-fledged raw materials, Brno, 2018.

[18] Živný, M.: Composite mixtures for cold recycling of structural layers of the road using modified blast furnace slag, diploma thesis, University of Pardubice, 2019.

[19] Tarifa, N.: Alternative hydraulic binders for cold recycling techniques used in pavement rehabilitation, Diploma Thesis, Faculty of Civil Engineering, Czech Technical University in Prague and École Nationale d'Ingénieurs de Tunis, 2019.

[20] Srb, J., Růžičková, Z.: New methods of using secondary raw materials, Praha: SNTL – Technical Literature Publishing House, 1980.

[21] Vlček, J., Fiedor, J.: Industrial waste: slags from iron and steel production, ECOFER, 2015.

[22] Valentin, J., Faltus, M.: Innovative Technologies for the Utilization of Inorganic Industrial Wastes or By-Products - Interim Report 2019, Project TH04020043, Praha, 2020.

[23] Valentin, J., Živný, M., Tarifa, N.: Knowledge with the application of some slags in cold recycling technology, Roads Bridges 03/2019, Praha, 2019.

COMPARATIVE STUDY ON THE DESIGN METHODS FOR FLY ASH-FLEXIBLE PAVEMENT

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Abstract

Advancement in the design of pavement structures in the recent decade has brought about the use of finite element modelling (FEM) tools. Numerical simulation of flexible pavement through these models are yielding positive results and enhancing pavement design year after year. Various factors contribute to this success; yet, material characterization model in FEM is a major/critical factor. However, in using FEM, there are various material characterization input methods which are; input through laboratory testing; secondly, through correlation and lastly a backward calculation from deflection measurements. Overall, input methods are more realistic and give a better understanding of the mechanical behaviour of the material, nevertheless quite difficult to obtain. Although, the use of fly-ash stabilizer in pavement structure is not new yet its use has not been fully implemented in FEM design. As a result, a comparative study is considered based on input and correlation parameters on fly ash stabilized flexible pavement using Abaqus. Furthermore, the results show that the material input method provides better results and gives some amount of certainty on the design life of the pavement.

Keywords: flexible pavement; finite element modelling; empirical design methods; material characterization; Non-linear model; fly ash

1 Introduction

Flexible pavement design is based on load distributing characteristic of the component layers. The asphalt surface depending on time and temperature behaves as a viscous material and the pavement foundation matrix (coarse-grained unbound granular materials in base / sub-base course and fine-grained soils in the sub-grade) exhibit stress-dependent non-linear behaviour [1]. Furthermore, with the introduction of soil stabilization resulting in alternative materials, the design of flexible pavement has become more complex. Thou the analysis of pavement via empirical methods sometimes result in errors [2]. Therefore, this study presents a comparative analysis of material characterization input methods for fly ash stabilized flexible pavement using FEM. FEM has been applied extensively in road engineering over the years [3]. So far, it is the most versatile of all analysis techniques, with capabilities for 2D and 3D geometric modelling, able to analyze stable (static), time-dependent problems, non-linear material characterization, large strains/deformations, dynamics analysis and oth-

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er sophisticated features [4]. However, the application of FEM to solve any problem consists of three separate stages; pre-processing (Modelling), processing (Evaluation) and post-processing (Simulation). Yet, the use of 3D appears to be the best approach [5]. FEM has been successfully used in the analysis of the major forms of failure in pavement structure such as rutting and fatigue cracking at different layers [6] and also used to determine the accurate positioning of geogrid materials [6], the thickness of each layer [7] and the interaction between pavement and its instrumentation.

2 Critical factors in finite element analysis of flexible pavement

In general, creating an FE model for flexible pavement analysis involves the consideration of all the steps in the pre-processing (Modelling), which are; the geometry of pavement (dimensions), material characterization, the relationship between parts (assembling and interactions), loading and boundary conditions (constraints), and analysis type. Although critical considerations need to be given to the aforementioned, any FEM simulation's success depends greatly on them. If these factors are not properly considered, it can result in errors in the design. However, based on the scope of this study, material characterization would be discussed.

2.1 Material characterization

Proper material characterization is a critical/major aspect of FEM based design of pavement, as it determines the reliability of response prediction in pavement design. However, accurate material characterization, selection and formulation of proper constitutive equations to represent the behaviour of the materials under loading is considered [1]. Qualitative choice is needed in material characterization and the model must capture the major features of material behaviour while minor features may be ignored in the model [8]. Furthermore, resilient modulus (MR) alongside Poisson's Ratio as input material property for characterizing all unbounded layers and soils in any FEM model for flexible pavement design is considered [9]. MR values although estimated directly from laboratory testing such as; Triaxial, Oedometer and Shear test; indirectly through correlation with other laboratories/field tests which are CBR, Isotropic compression test, Uniaxial strain test, Indirect tensile strength and unconfined compression strength (UCS) or back-calculated from deflection measurements [9], [10]. On this note, correlation is selected due to the difficulty in laboratory testing of input parameters.

2.2 Material characterization via correlation equations

AASHTO recommends MR from repeated Triaxial test. Due to the complexity of the test and time required, results are not readily available. In view of this, Sas et al. [11] and Rao et al. in the Technical Report [12] improvised the use of correlation equations for readily available test results. Thus, it is necessary to evaluate design MR of a stabilized base layer available data of UCS. Table 1, amongst others, suggests equations to estimate MR considering UCS test.

Table 1 Summary of correlations between the unconfined compressive strength and resilient modulus of underlying pavement layers [13]

Correlation	Source of the correlation	Application area
MR (ksi) = 500 + UCS (psi)	American Coal Ash Pavement Manual (1990)	Lime-cement-fly ash stabilized soils
MR (psi) = 1200 UCS (psi)	Barenberg (1977)	Cement stabilized coarse-grained sandy soils
MR (psi) = 440 UCS (psi) + 0.28 UCS2 (psi)	Barenberg (1977)	Cement stabilized fine-grained soils
MR (ksi) = 0.124 UCS (psi) + 9.98	Thompson (1966)	Lime stabilized soils
MR (psi) = 0.25 UCS2 (psi)	McClelland Engineers (unpublished)	Lime-cement-fly ash mixtures
MR (MPa) = 2240 UCS0.88 (MPa) + 110	Australian Road Research Laboratory (1998)	Cemented natural gravel

2.3 Material characterization using resilient modulus model

The use of Triaxial, Oedometer and Shear test results as material characterization is considered more accurate. Using any of the aforementioned test results requires at least one to two laboratory tests for calibration in the FE model. Over the years, various models have been developed for obtaining MR through Triaxial laboratory results. In NCHRP [14], few of the several models available were suggested. Overall, amongst the models in that study, the LTTP model (equation 1), – a modification of the Universal model – is adopted in the Design Guide (United States Department of Transportation – Federal Highway Administration [15], this will be considered in this study based on its general acceptance. The result in terms of MR obtained is inputted in constitutive material models in the FE Model.

$$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left[\left(\frac{\tau_{oct}}{P_a}\right)+1\right]^{k_3} \tag{1}$$

Where

M_R - Resilient modulus,
θ - Bulk stress (σ1 +σ2 +σ3),
P_a - Atmospheric pressure,
σd - Deviator stress,
σ3 - Confining stress,
k_i - Regression coefficient,
τ_{oct} - Octahedral stress ($\sqrt{\frac{2}{3}}\sigma_d$).

2.4 Failure criteria in numerical simulation

This is the empirical portions of the M-E design, known as the damage models, and it is developed to provide the resistance of pavement to failure [16]. These models require results from FEM such as stress, strain or deflection to give the behaviour of pavement in terms of performance, cracking, rutting, roughness and life span with equations derived from observation and performance of pavement to observed failure and initial strain under various loads. Various types of failure criteria exist depending on the type of pavement layer in question; Asphalt surface – (Fatigue cracking); Unbound granular base and sub-base layer – (Permanent deformation); Cemented base and sub-base layers – (Crushing failure, Effective fatigue and Permanent deformation); Subgrade – (permanent deformation or rutting), nonetheless two are widely recognized; fatigue cracking in asphalt and deformation in the subgrade [17]. Permanent deformation is induced in any layer of the structure, making it more difficult to predict than fatigue cracking. Yet, critical rutting can be attributed mostly to a weak pavement layer (subgrade). This is typically expressed in terms of the vertical compressive strain at the top of the subgrade layer and is given by Asphalt Institute by equation (3).

$$N_f = 0.0796\left(\varepsilon_t\right)^{-3.291}\left(E\right)^{-0.854} \tag{2}$$

Where
N_f - Number of repetitions for fatigue cracking;
ε_t - Tensile strain at the bottom of the asphalt surface in microstrain;
E - resilient modulus of asphalt in psi.

$$N_r = 1.365 \cdot 10^{-9}\left(E_c\right)^{-4.477} \tag{3}$$

Where
Nr - Number of repetitions for subgrade rutting failure;
Ec - Compressive strain on top of the subgrade.

Overall, the failure analysis models define the point at which failure occurs in pavement by determining the incremental damage.

3 Methodology

The research design adopted for this study incorporates Finite Element analysis of pavement layers subjected to traffic loading conditions. The design principle also allows for the response analysis to be analysed based on the pavement's characteristic input values; the analysis result is developed in form of failure deformation patterns considering resilient modulus of the underlying asphalt base layers down to the subgrade layer. A scenario of a paved flexible pavement is developed for a three-layered system of the pavement structure which is; asphalt surface, 18 % fly ash with 1 % cement stabilized base and subgrade layer. 3D FEM was used in the development of these models. The thicknesses of the asphalt layer ranges (25mm – 100mm) while that of base and subgrade layer were kept constant at a specific depth (300 mm and 2000 mm respectively).
Additionally, the 3D model is 3000 mm in length by 3000 mm breadth and the total depth of 2350 mm. This geometry is also similar to that used by Tiliouine and Sandjak [18], intending to avoid edge error when loaded. Furthermore, 8-node solid continuum elements (C3D8R) with reduction integration were used. The asphalt, stabilized base and subgrade layer was seed at 0.025 m at the loading area, while other areas were seed at 0.1 m; as a result, meshes are fine in/near loading area and coarse at distances away from the applied load for an efficient model as suggested by Peng and He [3].

3.1 Material input classification for analysis

Material properties of the stabilized base layers were obtained from laboratory test (UCS) and by correlation formula [13]. Although, other material properties are selected from research reported in [17] represented by the linear elastic model. Parameters such as (bulk stress = 1854kPa) are obtained from Heyns and Mostafa Hassan [19] and regression coefficients (k1 = 3000psi and k2 = 0.5) suggested by AASHTO [13]. A Drucker-Prager, (D-P) elasto-plastic model and plasticity model in Abaqus was used for the material characterization

to be non-linear. The D-P shear criterion is assumed 'exponent form' to allow for the use of sub-option (Triaxial test data), and the dilation angle is assumed to be 15^{o}.
Furthermore, the non-linear material characterization for the stabilized base layer is analyzed in a static-general analysis procedure, to consider the non-linear effect. All laboratory test result were conducted by [19]. Tables 2-3 presents the material properties used in this study.
3.2 Boundary Conditions and load parameters
The pavement layers are assumed to be perfectly bond together, and the model is fixed at the bottom of the subgrade and roller constraints on the vertical boundaries. A static standard equivalent single axle load with dual tires is used. TRH4 [20] specified that the maximum stress at a specific point in the pavement occurs when the wheel load is directly above it, while the stress can be assumed zero when the load is quite far from that point. The contact area of 72557 mm^2 with a rectangular area of contact was placed above the asphalt layer [2]. These loads were standard equivalent single axle load (80 kN) with dual tires and applied uniformly with a pressure of 0.65 MPa following South African standard.

Table 2 Material properties of conventional pavement interlayers profile

Layer	Material code (Colto 2008)	Modulus of Elasticity [MPa]	Poisson's Ratio
Surface	AG	3000	0.44
Granular Base	G5	200	0.35
Subgrade	G10	45	0.35

Table 3 Material properties of the stabilized base layer

Stabilized Base (%18Flyash+1%Cement)	Material code (Colto 2008)	USC [kPa]	Modulus of Elasticity [MPa] (Level 1)	Modulus of Elasticity [MPa] (Level 2)	Poisson's Ratio
18	C3	2133	1301	2560	0.35

4 Result and discussion

4.1 Finite Element Deformation Models

The non-linear material characterization over linear gives a close field measurement; thus, a comparative analysis of non-linear and linear material characterization was undertaken in this study. Figures 1 – 3 show the contour plots for displacements, strains and stresses in the 25 mm asphalt thickness layer. From Figure 1, it was observed that the maximum magnitude of deflection (rutting - 4.544 x 10^{-4} m) was higher in Figure 1B, which is for the non-linear model, implying the material acts like an elasto-plastic membrane; thus did not return to the original state. Similarly, from Figure 2, the maximum strain (1.838 x 10^{-4} m) was higher in the non-linear model but also worth to noting that the minimum strain (-5.076 x 10^{-6} m) was higher in the linear model, thus implying that strain in the linear model extended to the lower part of the sub-grade which will overall fail.
In Figure 3, the maximum stress transfer (tyre load) through the linear model was high, implying that more stress is transferred to the rest of the layers. Overall, there are not many differences in the results obtained, despite the MR (1301 MPa) used in the non-linear model is smaller when compared with that of the linear model (2560 MPa).

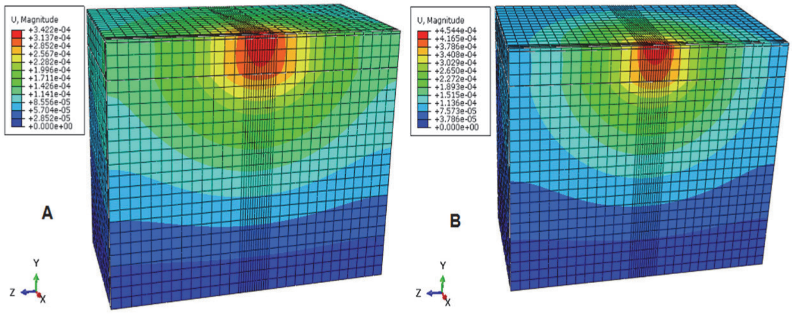

Figure 1 Displacement Failure Mode 25mm Asphalt Layer Thickness (A – Linear Model; B – Non-Linear Model)

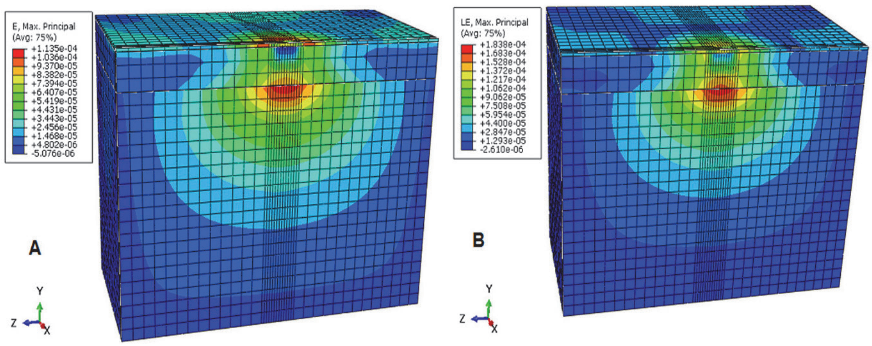

Figure 2 Strain failure Mode 25mm Asphalt Layer Thickness (A – Linear Model; B – Non-Linear Model)

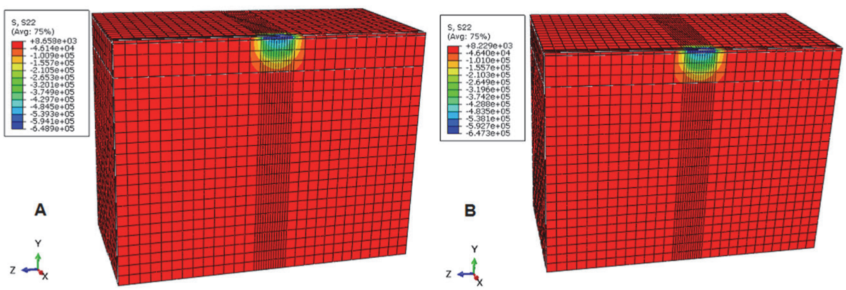

Figure 3 Stress Failure Mode 25mm Asphalt Layer Thickness (A – Linear Model; B – Non-Linear Model)

The non-linear model [Table 4] experienced an increase in the compressive strain for the stabilized base in 50 mm thickness asphalt layer and after that a decrease. Conversely, the horizontal strain in asphalt layer decreases in the 50 mm thickness and thereafter increases for subsequent thickness, thus, implying that the thickness of asphalt layer beyond 50 mm may result in bottom-up fatigue cracking. Overall, it is worth noting that the use of 50 mm thickness of asphalt layer over the stabilized base layer by developing countries, is not only justifiable by economic reasons, but also on its effectiveness to prevent failure such as bottom-up fatigue cracking which can be experienced in thicknesses beyond 50 mm.

Table 4 Asphalt Layer Response (Linear and Non-Linear Model)

Asphalt Layer Thickness (mm)	Linear Model		Non-linear Model	
	Vertical Strain εc (10^{-6}) in Stabilized Base Layer	Tensile Strain εt (10^{-6}) bottom of Asphalt Layer	Vertical Strain εc (10^{-6}) in Stabilized Base Layer	Tensile Strain εt (10^{-6}) bottom of Asphalt Layer
25	120.0	31.94	259.1	38.57
50	131.9	27.53	285.7	30.92
75	135.2	23.08	273.9	41.46
100	129.8	35.60	247.5	61.55

4.2 Structural Capacity Comparative Analysis

Table 5 presents the pavement structural capacity results obtained from the use of mechanistic-empirical structural capacity estimation (mePADS), 3D FEM Linear and Non-Linear Material with the Asphalt Institute model. The mePADS; which serves as a check for 3D FEM models' performance, although within a close range yet, tend to be higher than those of 3D FEM models. This is so because the South African Pavement Design Method (SAPDM) damage model used in software in question is outdated and currently under review [13]. Furthermore, results from linear models are higher than those of non-linear, which show that the linear model is over-designed as a result of the MR of the stabilized base layer considered. Thus it can be concluded that MR has a significant effect on the design of pavement through FEM.

Table 5 Effect of Asphalt layer thickness (linear and non-linear material model)

Asphalt Layer Thickness (mm)	Sub-grade Bearing Capacity (mePADS Results)	No. of Load Repetitions to Failure Nr (Linear Model)	No. of Load Repetitions to Failure Nr (Non-Linear Model)
25	30.70×10^{12}	2.92×10^{6}	5.41×10^{5}
50	12.70×10^{14}	5.60×10^{6}	1.13×10^{6}
75	43.11×10^{14}	9.89×10^{6}	2.13×10^{6}
100	10.00×10^{15}	17.04×10^{6}	3.91×10^{6}

5 Conclusion

In this study, a comparative analysis of material characterization inputs methods was undertaken. According to literature reviews, material characterization is one of the major factors contributing to the success in pavement design through FEM. Firstly, this study's results showed that an increase in the asphalt layer's thickness increases pavement resistivity to failure; yet, an increase in thickness beyond minimum allowance based on the design requirement may result in bottom-up fatigue cracking. Secondly, results showed that the use of non-linear (level 1) material characterization model is more efficient than linear material characterization. However, as a result of the Triaxial test results' unavailability, the linear material characterization model can be used as a preliminary study. Overall, the non-linear material characterization model stands a chance to provide better results and gives some certainty on the pavement's design life.

References

[1] Kim, M.: Three-dimensional finite element analysis of flexible pavements considering nonlinear pavement foundation behavior, University of Illinois at Urbana-Champaign, 2007.

[2] Huang, Y.H.: Pavement Analysis and Design, 2nd Edition, Pearson Education Inc, NJ., 2004.

[3] Peng, Y., He, Y.: Structural characteristic of cement-stabilized soil bases with 3D finite element method, Frontiers of Architecture and Civil Engineering in China, 3 (2009) 4, pp. 428-434

[4] NCHRP: Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures: Appendix RR-Finite Element Procedures for Flexible Pavement Analysis, Transportation Research Board, National Research Council, Washington D.C., 2004.

[5] Shafabakhsh, G.A., Motamedi, M., Family, A.: Influence of Asphalt Concrete Thickness on Settlement of Flexible Pavements, Electronic Journal of Geotechnical Engineering, 18 (2013), pp. 473-483

[6] Abed, A.H., Al-Azzawi, A.A.: Evaluation of Rutting Depth in Flexible Pavements by Using Finite Element Analysis and Local Empirical Model, American Journal of Engineering and Applied Sciences, 5 (2012) 2, pp. 163-169

[7] Sinha, A.K., Chandra, S. Kumar, P.: Finite Element Analysis of Flexible Pavement with Different Subbase Materials, Technical Paper, Indian Highway. 42 (2014) 2, pp. 53-63

[8] Abaqus, Inc.: Analysis of geotechnical problems with ABAQUS. 2003. ABAQUS, Inc.

[9] Ji, R., Siddiki, N., Nantung, T., Kim, D.: Evaluation of Resilient Modulus of Subgrade and Base Materials in Indiana and Its Implementation in MEPDG, The Scientific World Journal, 2014., pp.1-14, doi: https://doi.org/10.1155/2014/372838

[10] Al-Jhayyish, A.K.: Incorporating Chemical Stabilization of the Subgrade in Pavement Design and Construction Practices, PhD Thesis, Ohio University, 2014.

[11] Sas, W., Głuchowski, A., Szymański, A.: Determination of the resilient modulus MR for the lime stabilized clay obtained from the repeated loading CBR tests, Annals of Warsaw University of Life Sciences-SGGW, Land Reclamation, 44 (2012) 2, pp. 143-153

[12] Rao, C., Titus-Glover, L., Bhattacharya, B., Darter, M.I.: User's Guide: Estimation of Key PCC, Base, Subbase, and Pavement Engineering Properties from Routine Tests and Physical Characteristics, technical Report, FHWA-HRT-12-03, Federal Highway Administration, Washington, DC, 2012.

[13] Adedeji, J.A.: Simulation of flexible pavement utilizing fly ash as alternative stabilizer. Master's thesis, Department of Civil Engineering, Central University of Technology, Free State, South Africa, 2015.

[14] NCHRP, National Cooperative Highway Research Program. Mechanistic-Empirical Pavement Design Guide, Transportation Research Board, Washington, DC, http://onlinepubs. trb.org/onlinepubs/archive/mepdg/home.htm, 14.04.2010.

[15] United States Department of Transportation - Federal Highway Administration (USDT-FHA), Geotechnical Aspects of Pavements Reference Manual, http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/05b.cfm, 3.1.2020.

[16] South African National Road Agency, Pavement Design: South African Pavement Engineering Manual, An initiative of the South African national roads agency ltd, South Africa, 2013.

[17] Ekwulo, E.O., Eme, D.B.: Fatigue and rutting strain analysis of flexible pavements designed using CBR methods, African Journal of Environmental Science and Technology, 3 (2009) 12, pp. 412-421

[18] Tiliouine, B., Sandjak, K.: Numerical simulation of granular materials behaviour for unbound base layers used in Algerian pavement structures, International Journal of Civil and Structural Engineering, 4 (2014) 3, pp. 419-429

[19] Heyns, M.W., Mostafa Hassan, M.: South Africa Class F Fly Ash for roads: physical and chemical analysis, Interim: Interdisciplinary Journal, 12 (2013) 3, pp. 28-41

[20] TRH 4: Structural Design of flexible pavements for interurban and rural roads. Pretoria: Committee of Land Transport Officials (COLTO), Department of Transport (DoT). (DoT Technical Recommendations for Highways; Draft TRH 4, 1996.

COMPARISON OF STANDARDS AND REQUIREMENTS FOR POROUS ASPHALT MIXTURES

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Abstract

Porous asphalt is a bituminous material prepared in such a manner that it has a very high content of interconnected voids that allow passage of water and air in order to provide the compacted mixture with better drainage and noise reducing characteristic. The analysis of available regulations and scientific literature shows that the national standards of porous asphalt mixtures differ in type of granulometric envelopes, the type of bitumen and its content in the mixture and the limits of the air voids content in the mixture. In this paper, standards used for porous asphalt in Croatia are compared with the US, Australian and Dutch standards for this type of asphalt. In addition, samples prepared based on HRN EN 13108-7 and HRN EN 12697-17 requirements were tested and results were compared using Croatian and other available standards. This paper also investigates the effects of ambient temperature on abrasion loss of porous asphalt. The abrasion loss results are compared to the other standards.

Keywords: porous asphalt, Cantabro test, abrasion loss

1 Introduction

Porous asphalt pavement is a type of porous pavement structure designed to allow rainfall and runoff to flow into and through the pavement structure. The main difference compared to the standard asphalt mixture is a higher proportion of coarse aggregate and a higher content of air voids in the mixture. The total amount of bitumen in the porous asphalt mixture is equal or slightly higher than the amount of bitumen in dense asphalt mixtures with the same maximum size of aggregate. The main characteristics of porous asphalt pavement are high permeability and reduced traffic noise [1]. Common applications of porous asphalt pavements are parking lots, sidewalks and pathways [2]. In addition, in the Netherlands porous asphalt is also used for pavements for heavy wheel loads. In 2009 about 80 % of total surface of the Dutch motorways was covered with porous asphalt [3]. Porous asphalt pavement is not recommended at intersections due to the possibility of oil and fuel leaking from the vehicles and infiltrating into porous asphalt air voids [4]. Some advantages of porous asphalt pavements are: maintaining a high skid resistance, noise reduction, providing a storm-water management system that promote infiltration, reducing the dangers of aquaplaning and reducing a glare at night during wet weather [5, 6, 7]. The main disadvantages are: the need for early preventive maintenance, clogging of air voids, shorter expected service life compared to dense asphalt pavements and higher total project cost [2, 6, 8].

This study was prepared with the aim to compare standards used for porous asphalt mixture in Croatia with other countries (the USA, the Netherlands and Australia). Test results for porous asphalt mixtures are presented in the paper, with emphasis on Cantabro test - abrasion loss of porous asphalt specimen. In addition, the aim was to investigate the effect of ambient temperature on abrasion loss of porous asphalt specimens according to national standard HRN EN 12697-17 and to compare obtained results with limit values of foreign standards.

2 Standards and guidelines

The porous asphalt mixture in Croatia is defined in standard HRN EN 13108-7 [9]. The limit values for porous asphalt mixture are defined in Croatian technical requirements for asphalt pavements [10]. The guidelines applied in different countries and the review of past research related to Cantabro test are presented below.

2.1 Porous asphalt requirements in Croatia, USA and Australia

Croatian technical requirements for asphalt pavements [10] defines properties and requirements for construction products and usability of asphalt layers in road construction, reconstruction and maintenance. Depending on the aggregate gradations, three types of porous asphalt mixtures are used in Croatia (PA 8, PA 11 and PA 16). In Croatia, depending on the traffic load, conventional bitumen 50/70 or polymer-modified bitumen 40/100-65, 45/85-65 and 45/80-55 is used. The porous asphalt mixture can be classified in two categories M1 and M2, and must satisfy the required technical properties (air voids content, indirect tensile strength ratio ITSR, and abrasion resistance). These parameters are listed in Table 1.
In the USA, properties and requirements for porous asphalt mixtures are defined in the Standard Practice for Open-Graded friction Course (OGFC) Mix Design [11]. Limit values and characteristics of mixtures in the USA are presented in table 1. The draindown potential can be decreased by adding fiber stabilizers (cellulose fiber or mineral fiber) in porous asphalt mixture [10]. For some mixtures that use polymer-modified bitumen or asphalt rubber, fiber additives may not be required to control draindown or to obtain good performance. The Cantabro test has been used in Europe for many years, but it is optional in the USA and it has seen very little use in the USA [11]. The operating temperature should be 25 ± 5 °C and the machine operates at 300 rotations. Abrasion loss values are shown in Table 1. In the Netherlands [3,6], the standard wearing course is performed as single layer PA 16 with air voids content of 20 %. It can also be performed as two layered, combining PA 8 and PA 16. The porous asphalt mixture PA+ is also used. PA+ contains 5.2 % bitumen 70/100 and drainage inhibitors that increase the pavement lifespan. In Australia [4], a porous asphalt mixture is grouped as Type I and Type II depending on indicative traffic volume (commercial vehicles per lane per day). The requirements for the mixture are presented in Table 1.
The porous asphalt mixture in the USA and Australia has a higher bitumen content compared to Croatia. The use of additives is recommended in cases where no polymer-modified bitumen is used. The maximum air voids content in the USA and Australia is not defined. In Croatia the maximum air voids content amounts to 26 % for M2 or 28 % for M1. The minimum air voids content in Croatia and in the USA is lower compared to Australia, where the minimum air voids content in the mixture amounts to 20-25 %.

Table 1 Comparison of requirements for a porous asphalt mixture by country

	Croatia		The USA	Australia	
Standard	**HRN EN 13108-7: Bituminous mixtures – Material specifications – Part 7: Porous Asphalt [8], Hrvatske ceste: Croatian technical requirements for asphalt pavements [10]**		**D7064/ D7064M-08: Standard Practice for Open-Graded friction Course (OGFC) Mix Design [11]**	**STANDARDS AUSTRALIA (1995). Hot Mix Asphalt. AS 2150 (Standards Association of Australia) [4]**	
Types of Porous asphalt mixture	M1	M2		Type II	Type I
Minimum bitumen content (%)	3% polymer modified bitumen	3% conventional bitumen	6-6.5% conventional bitumen, 5.5.-10% modified bitumen	4.5-6.5% conventional/ modified bitumen	3.5-5.5% conventional/ modified bitumen
Minimum air voids content (%(V/V))	18%	16%	18%	20-25%	20%
Maximum air voids content (%(V/V))	28%	26%	not specified	not specified	not specified
ITSR (%)	80%		80%	80%	80%
Maximum value of abrasion loss, PL, %(m/m)	30% 30% (aged specimens)		20% (unaged specimens)	20% (unaged specimens)	25% (unaged specimens)
			30% (aged specimens)	35% (aged specimens)	
Fillers and additives	The use of own filler or Portland cement as an added filler is not allowed.		Cellulose or mineral fiber 0.2-0.5% by mixture mass	Cellulose or mineral fiber 0.2-0.5% by mixture mass. Fillers: Portland cement, hydrated lime, fly-ash, ect.	

2.2 Cantabro test

Abrasion loss is tested in laboratory and the test is called Cantabro test. The test evaluates the resistance of porous asphalt specimens to abrasion loss due to abrasion and impact forces. Cantabro test is a quick and simple test so it is used in many countries. For instance, it is used in Japan [12] to determine the resistance of abrasion loss of porous asphalt in winter conditions. Test method for determining the abrasion loss of porous asphalt specimens in Croatia is specified in HRN EN 12697-17 [13].

This standard [13] specifies a test method for porous asphalt specimens in the Los Angeles machine (without steel balls). Cantabro test is applied to laboratory compacted cylindrical specimens where the upper sieve size does not exceed 22.4 mm. During the test, the ambient temperature should be measured near the Los Angeles machine. Ambient temperature usually used for this test is between 15°C and 25°C and the temperatures above 35°C are not suitable for this test [13]. At least five specimens are required and they should have a mass of (1.0±0.2) kg. Bulk density and air voids content are determined by the standard HRN EN 12679-6 and HRN EN 12697-8. Specimens should be kept at the ambient temperature for at least 4 hours before testing. Then the specimen is placed into the drum of the Los Angeles machine which operates at the speed of 3,1 rad/s up to 3,5 rad/s (30 to 33 rpm) for a total of 300 rotations. The mass of specimen should be determined before and after placing it inside of the Los Angeles machine. The mass difference gives the value of abrasion loss. Specimens before and after the Cantabro test are shown in Figure 1.

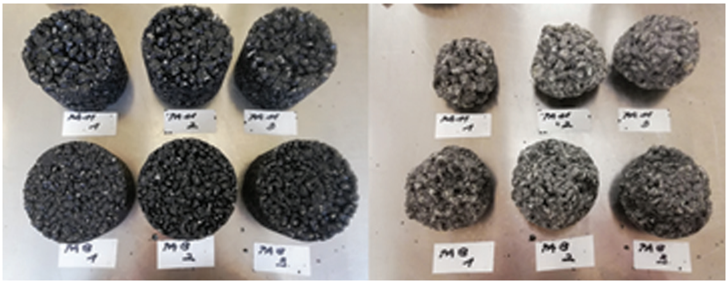

Figure 1 Specimens before and after Cantabro test

Recent studies related to Cantabro test show the influence of ambient temperature and bitumen content in the porous asphalt mixture. The authors [14] considered the effects of ambient temperatures on abrasion loss and temperature change of porous asphalt according to the standard EN 12697-17 and Standard Specification for Road Works in Malaysia. The test was performed in the Netherlands (the average laboratory temperature was 24 °C) and Malaysia (the average laboratory temperature was 30 °C). It was concluded that the abrasion loss in the mixture decreases with an increase in the ICT (initial conditioning temperature) and with an increase of the bitumen content. In this study [14], specimens are prepared with granite aggregate (two different aggregate gradations) and two different type of bitumen (conventional bitumen 60/70 and modified bitumen PG76) with 4.0 to 5.5 % bitumen content. Type of bitumen that is used in mixture are marked according to AASHTO standard. Also, modified bitumen PG76 is referred to SMA that has softening point min. 60 and penetration at 25°C is specified in ASTM D5. According to the authors, [14] the type of bitumen, bitumen content, ICT and surface temperature of specimens have a significant impact on the Cantabro test results. Porous asphalt specimens, which contain modified bitumen, showed better results than those specimens that contain conventional bitumen. It was also concluded that an increase in ambient temperature leads to an increase in the surface temperature of the specimen and to an increase of the temperature inside the drum of the Los Angeles machine. In the paper, the authors [2] studied a porous asphalt mixture with different bitumen contents (5-7.5 %) on the unaged and aged specimens. It was concluded that samples containing 6 % of bitumen obtained the best results, taking into account air voids content, abrasion loss, aging characteristics and drain-down potential.

3 Laboratory testing

Cylindrical specimens were prepared in a standard Marshall mold. Two tests were conducted and six specimens were made for each. The ambient temperature for the first test was 23.2°C-25.4°C (on average 24.3°C) and for the second test temperature was not precisely determined at the time, but it was continuously above 30°C and below 35°C. In this study, the temperature of 30°C will be assumed. Figure 2 (left) shows the aggregate gradations PA8 for three specimens of porous asphalt mixture type M1 with aggregate designation AG1. The other three specimens were prepared as porous asphalt mixture type M1 with aggregate designation AG1 and aggregate gradations PA11 shown in Figure 2 (right). Porous asphalt specimens contain 3.0-4.0 % of polymer modified bitumen 45/80-65. The specimens were kept at the ambient temperature for at least 4 h before testing.

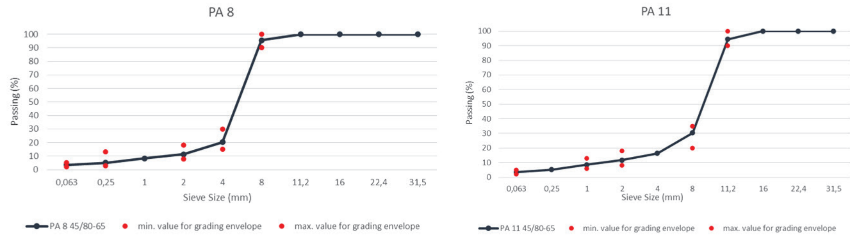

Figure 2 Aggregate gradations PA8 (left) and PA11 (right) adopted in this study

The diameter and height of the specimens were measured and these data were used to obtain density and air voids content of porous asphalt. The specimens were weighed before and after placing inside the drum of the Los Angeles machine.

4 Results and discussion

The specimens presented in Table 2 and Table 3 are marked as PA8 1-3 and PA11 1-3, and for every specimen the dimensions, density and air voids content are shown. These parameters are determined according to previously mentioned standards for ambient temperature of 30°C and for temperature between 23.2- 25.4°C. The tables show the mean height, mean diameter and volume of specimens. The bitumen content ranges from 3 to 4 %. The ρ_{MV} value represents the asphalt mixture bulk density obtained by the volumetric method. Bulk density ρ_{bdry} is obtained from measured dimensions of dry specimen. VMA stands for the calculated air voids content in aggregate, V_m for the calculated air voids content in the specimen, and VFB for the air voids content filled with bitumen. The air voids content in the specimens ranges from 25.0 % to 28.9 %. These values meet the minimum air voids content, which is 18 % according to the Croatian technical requirements for asphalt pavements [10]. However, six out of nine specimens do not meet the maximum air voids content of 28 %. The percentage of air voids filled with bitumen is lower when PA8 aggregates are used in the porous asphalt mixture.

Table 2 PA mixture characteristics (ambient temperature of 30 °C)

SPECIMEN	PA8-1	PA8-2	PA8-3	PA11-1	PA11-2	PA11-3
h (mm)	69,8	70,0	69,8	68,9	68,4	68,2
d (mm)	101,5	101,6	101,7	101,6	101,6	101,6
V (mm^3)	565347,57	567253,86	566488,61	558423,58	554257,44	553037,51
bitumen content (%)	3,0	3,5	4,0	3,0	3,5	4,0
ρ_{MV} (g/mm^3)	2,600	2,580	2,560	2,600	2,580	2,560
ρ_{bdry} (g/mm^3)	1,848	1,856	1,869	1,865	1,897	1,914
V_M (%)	28,9	28,1	27,0	28,3	26,5	25,2
VMA (%)	34,3	34,4	34,3	33,7	32,9	32,7
VFB (%)	15,7	18,4	21,3	16,2	19,7	22,8

Table 3 PA mixture characteristics (ambient temperature of 23.2 to 25.4°C)

SPECIMEN	PA8-1	PA8-2	PA8-3	PA11-1	PA11-2	PA11-3
h (mm)	69,5	69,8	69,7	68,9	68,0	68,0
d (mm)	101,6	101,6	101,6	101,6	101,5	101,5
V (mm^3)	563256,03	566093,60	565282,86	557895,98	549743,34	549873,18
bitumen content (%)	3,0	3,5	4,0	3,0	3,5	4,0
ρ_{MV} (g/mm^3)	2,600	2,580	2,560	2,600	2,580	2,560
ρ_{bdry} (g/mm^3)	1,852	1,856	1,868	1,863	1,908	1,919
V_M (%)	28,8	28,1	27,0	28,4	26,1	25,0
VMA (%)	34,2	34,4	34,3	33,8	32,6	32,5
VFB (%)	15,8	18,4	21,3	16,1	20,0	23,0

Table 4. shows the results obtained according to standard HRN EN 12697-17 at the ambient temperatures of 30°C and 23.2°C-25.4°C.

Table 4 Test results according to standard HRN EN 12697-17

			Temp. 30 °C			
SPECIMEN	**PA8-1**	**PA8-2**	**PA8-3**	**PA11-1**	**PA11-2**	**PA11-3**
mass before (g)	1044,7	1052,8	1058,6	1041,5	1051,5	1058,3
mass after (g)	485,5	653,3	796,4	485,4	662	818,9
abrasion loss (%)	53,5	37,9	24,8	53,4	37	22,6
			Temp. 23,2°C - 25,4°C			
SPECIMEN	PA8-1	PA8-2	PA8-3	PA11-1	PA11-2	PA11-3
mass before (g)	1043,1	1050,8	1056,2	1039,1	1048,8	1055,3
mass after (g)	498,7	667,7	651,9	328,2	565,4	680,9
abrasion loss (%)	52,2	36,5	38,3	68,4	46,1	35,5

Data from Table 4 are shown in Figure 3. In addition, abrasion loss values and abrasion loss limits from different countries are presented in Figure 3. Specimens PA8-3 and PA11-3 meet the limit value for abrasion loss and the lowest value is shown for ambient temperature of 30°C (24.8 % for PA8-3 and 22.6 % for PA11-3). In addition, these specimens have the highest bitumen content (4 %) and lower air voids content compared to the other specimens. According to the Croatian technical requirements for asphalt pavements [10] abrasion loss should not exceed 30 %. Specimens tested in the ambient temperature of 23.2°C-25.4°C do not meet the limit value for abrasion loss. Also, presented results do not meet the other guidelines shown in Table 1.

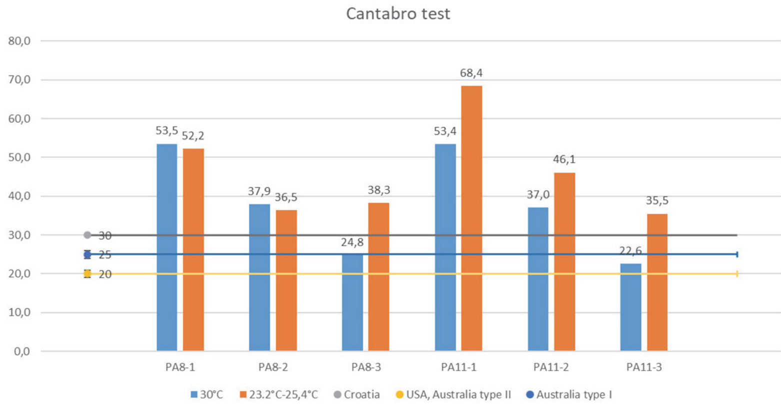

Figure 3 Abrasion loss – results

According to Colonna [15], at temperature of 30 °C, interpolation yields a maximum abrasion loss of 16 %. Abrasion loss in this test amounts between 22.6 % to 53.5 % and does not meet the results recommended by Colonna [15]. Also, abrasion loss values for ambient temperature 25°C do not meet the value of 20 % recommended by Colonna [15]. For ambient temperature 23.2°C-25.4°C abrasion loss amounts between 35.5 % and 68.4 %. For the specimen PA11-1, the abrasion loss at lower temperature is 15 % higher than the abrasion loss at higher temperature. The smallest difference in the result is observed in the specimen PA8-1 and the difference is 1.3 %. For PA11-1 and PA11-3 at ambient temperature of 23.2°C- 25.4°C, the difference in results is 32.9 %. This is also the greatest difference in the results at the given temperature. At ambient temperature of 30°C, for the specimen PA11-1 abrasion loss is 30.8 % higher than the abrasion loss for the specimen PA11-3. Generally, specimens PA8-1 and PA11-1 have the highest abrasion loss values. These specimens have a lower bitumen content (3 %) and a higher air voids content compared to other specimens. Air voids content for specimen PA8-1 for different ambient temperature amounts to 28.9 % and 28.8 %. For PA11-1 air voids content amounts to 28.3 % and 28.4 %. Four specimens (PA8-3, PA11-1, PA11-2 and PA11-3) show lower abrasion loss at higher ambient temperature and two specimens (PA8-1 and PA8-2) show higher abrasion loss.

5 Conclusions

According to the standard HRN EN 12697-17, Cantabro test is used to evaluate bonding properties between aggregate and bitumen. During the test, the specimens are exposed to abrasive force between the surface of the specimen and the walls of the Los Angeles machine. Two tests were conducted at ambient temperatures of 23.2°C-25.4°C and 30°C on cylindrical porous asphalt specimens. It can be concluded from the results that the ambient temperature affected the abrasion loss value. In addition, abrasion loss value is affected by bitumen content and air voids content. Specimens with higher bitumen content have lower abrasion loss value. Comparison of results with other analyzed guidelines shows the need to design a porous asphalt mixture with a higher bitumen content and lower air voids content. This indicates the need to repeat the testing with new parameters in order to determine whether the abrasion loss value shall be within the permited value.

References

[1] Roberts, F.L., Kandhal, P.S., Brown, E.R., Lee, D., Kennedy, T.W.: Hot mix asphalt materials, mixture design and construction, Second edition, Hrvatski savez građevinskih inženjera, 1996.

[2] Kiran Kumar, V., Manjunatha, S., Shiva Prasad, N.: Porous asphalt pavement a tentative mix design guidelines – by new generation open graded friction course approach, International Journal od Engineering Science Invention Research & Development, 1, issue III, pp. 105-112, 2014.

[3] Voskuilen J., Van de Ven, M.: Winter problems with Porous Asphalt in the Netherlands, 11th International Conference on Asphalt Pavements, Nagoya, Japan, 2010.

[4] Australian Asphalt Pavement Association: Open Graded Asphalt – Design Guide, Implementation guide No. 1, 1997.

[5] Halstead, W.J.: Open-Graded Friction Courses for Highways, Transportation Research Board, NCHRP Syncthesis 49, 1978.

[6] Van der Zwan, J.T., Goeman, T., Gruis, H.J.A.J., Swart, J.H., Oldenburger, R.H.: Porous Asphalt Wearing Courses in the Netherlands: State of the Art Review, Porous asphalt pavements: an international perspective 1990., pp. 95.-110., 1990.

[7] Vaitkus, A., Andriejauskas, T., Gražulytė, J., Šernas, J., Vorobjovas, V., Kleizienė, R.: Qualitative criteria and thresholds for low noise asphalt mixture design, IOP Conference Series: Materials Science and Engineering, 356 (2018), doi: 10.1088/1757-899X/356/1/012027

[8] Arrieta, V.S., Maquilon, J.E.C.: Resistance to degradation or cohesion loss in Cantabro test on specimens of porous asphalt friction courses, Procedia – Social and Behavioral Sciences, 162 (2014), pp. 290-299, doi: 10.1016/j.sbspro.2014.12.210

[9] HRN EN 13108-7:2016 - Bituminous mixtures – Material specifications – Part 7: Porous Asphalt (EN 13108-7:2016)

[10] Hrvatske ceste: Croatian technical requirements for asphalt pavements, Zagreb, june 2015.

[11] D7064/D7064M-08: Standard Practice for Open-Graded friction Course (OGFC) Mix Design, 2013.

[12] Takahashi, S., Poulikakos, L.D., Partl, M.N.: Evaluation of improved porous asphalt by various test methods, 6th Int. RILEM symposium on performance testing and evaluation of bituminous materials, pp. 230-236, Zurich, Switzerland, 2003.

[13] HRN EN 12697-17:2017 - Bituminous mixtures - Part 17: Particle loss of porous asphalt specimen (EN 12697-17:2017)

[14] Hamzah, M.O., Hasan, M.R.M., van de Ven, M., Yahaya, A.S.: The effects of initial conditioning and ambient temperatures on abrasion loss and temperature change of porous asphalt, Construction and Building Materials, 29 (2012), pp. 108-113, doi: 10.1016/j.conbuildmat.2011.08.050

[15] Colonna, P.: Design and performance of porous asphalt in the Puglia region of southern Italy, First Eurasphalt & Eurobitume Congress, Strasbourg, France; 1996.

COMPARATIVE STUDY ON USING THERMOPLASTIC POLYMERS TO IMPROVE ASPHALT MIXTURES CHARACTERISTICS

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Abstract

Asphalt mixtures are composite building materials consisting of a mineral skeleton mixed with a bituminous binder, following a recipe which may also include fibres and/or polymers. The natural aggregates sustain the mixture structure, but adequate bitumen behaviour under various temperature and mechanically-induced stresses is also essential for the structural durability. Much research effort was directed towards improving the asphalt mixtures' resistance to permanent deformation, implying an increase in mixture stiffness. At the same time, the mixture must exhibit enough low temperature cracking resistance. Six reference asphalt mixture samples were prepared and tested: mixtures M1 and M2 for base and binder courses respectively, as well as four mixtures for wearing courses (two asphalt concrete - AC1 and AC2, a stabilised mixture SMA containing fibres, and a porous mixture - PM). A 50/70 penetration grade bitumen was used to prepare all mixtures. In some cases, the obtained results did not meet the standard requirements. Bitumen or mixture modification is commonly performed by adding thermoplastic or elastomeric polymers, to improve the asphalt mixture behaviour. In this study, the effects of four thermoplastic polymers on the stiffness modulus, dynamic creep and fatigue resistance were studied. All tested polymers were introduced as grains during mixture preparation. Polymer addition led to a 31 % to 104 % increase in mixture stiffness modulus. A 220 % average increase in fatigue resistance was observed for mixtures M1 and M2. For the wearing course mixtures, creep resistance is expressed through a 99 % reduction in deformation speed and a 50 % to 80 % reduction in rut depth. The obtained results met the standard requirements. Using grain polymers is currently an effective alternative to polymer-modified bitumen, because of several technological and economical advantages. Polymer quality is essential to obtain adequate mixture characteristics.

Keywords: asphalt, thermoplastic polymer, stiffness, deformation, fatigue

1 Introduction

Asphalt mixtures are composite building materials consisting of a mineral skeleton mixed with a bituminous binder, following a recipe which may also include fibres and/or polymers, in order to improve the mixtures' behaviour in operation [1, 2]. The natural aggregates sustain the mixture structure, but adequate bitumen behaviour under various temperature and mechanically-induced stresses is also essential for the structural durability. Much research effort was directed towards improving the asphalt mixtures' resistance to permanent deformation, implying an increase in mixture stiffness at high temperatures. At the same time, the mixture must exhibit workability and enough low temperature cracking resistance. Modified bitumens are binders whose rheological properties have been modified by adding one or

several modifying chemical agents, in certain dosages, to improve the bitumen's viscoelastic properties and behaviour [3-6].
Polymer-modified bitumen (PMB) is produced in specially designed refineries or units consisting of quality and performant equipment. Its production and distribution costs are rather high. Furthermore, technological difficulties (e.g. segregation) when applying or using PMB can be encountered, especially in developing countries. Therefore, a technological alternative was developed, addressing the improvement of asphalt mixtures' properties instead of bitumen modification, by adding grain polymers directly during asphalt production [3-4, 7]. This material is also known as polymer-modified asphalt (PMA).
The polymers currently employed in the business can be divided in two main categories: elastomers (with the ability to regain their initial shape after an action) and plastomers (forming a rigid tridimensional network in the asphalt mixture, capable to reduce/limit deformations under increased actions).
Thermoplastic polymers (plastomers) become soft and ductile when heated at certain temperatures. Afterwards, when temperature decreases, they become rigid and maintain their shape. These polymers do not exhibit elastic properties in ordinary thermal conditions, as their molecular chains' ability to move and recover their original position is limited. They increase the mixture's stiffness, however they do not significantly influence the bitumen elasticity or Fraass breaking point.
The main plastomers used for bitumen modification are: ethyl-vinyl-acetate (EVA), polyvinyl chloride (PVC), polyethylene (PE), polypropylene (PP), polystyrene (PS), ethylene-methyl-acrylate (EMA) and ethylene-butyl-acrylate (EBA) [3].

2 Materials

The polymers' efficiency and their influence on asphalt properties were the primary purpose of this study. Therefore, to perform a relevant comparison between different polymers, the analyses started by preparing and testing six reference asphalt mixture samples. Two of them, M1 and M2, were designed for bituminous base and binder courses, respectively. The rest of the mixtures were designed for wearing courses. Tests were performed on two asphalt concrete mixtures (AC1, AC2), as well as on a stabilised mixture (SMA) and a porous one (PM). All materials used to prepare the asphalt mixtures were tested according to the European regulations.
A 50/70 penetration grade bitumen was used to prepare all mixtures. Its properties follow the standard requirements. The natural aggregates used to prepare all the tested mixtures were extracted from a medium-density, slightly acid and porous dacite. The performed tests for aggregates' mechanical and physical properties indicated the following results, according to EN 13043 [8]: LA_{15} resistance to fragmentation, MDE_{15} resistance to wear, SI_{20} shape index.
The reference asphalt mixtures consisted of: natural aggregates (fine and coarse), bitumen, mineral filler, as well as cellulose fibres in the case of SMA. Mixtures M2, SMA and especially PM had high contents of 8/16 coarse aggregates fraction. 90 % of aggregates used to prepare the porous mixture (PM) were coarse. This mixture had the lowest content of fine aggregates of the six tested mixtures. SMA had a 22 % content of fine aggregates, while the rest of them contained 30-35 % fine aggregates.
The main physico-mechanical properties of the reference mixtures are indicated in Table 1 and Table 2. Shaded table cells indicate values outside the recommended standard intervals. Allowable values are shown where results were unsatisfactory.

Table 1 Reference mixtures' properties

Ref. mix.	Characteristics							
	Bitumen content [%]	Stability (S)[kN], 60 °C	Flow (F) [mm]		S/F [kN/mm]		Water abs. [%]	
			tests	limits	tests	limits	tests	limits
M1	4.7	9.8	3.2	≤ 3.0	3.1	≥ 6.0	5.8	≤ 5.0
M2	4.8	11.6	3.3		3.5		1.8	
AC1	5.7	11.4	3.3	≤ 3.0	3.5	≥ 4.0	2.5	
AC2	5.7	10.4	3.6	≤ 3.0	2.9	≥ 4.5	2.1	
PM	4.3	10.5	2.8	≤ 2.5	3.8	≥ 5.0		

Table 2 SMA reference mixture properties

Ref. mix.	Characteristics					
	Bitumen content	Binder drainage [%]		Void content [%]	Voids filled with bitumen [%]	Water sensitivity [%]
		test	limit			
SMA	5.9	1.1	≤ 0.2	3.6	79.3	89

Mixture recipes design have shown that the best laboratory results are obtained using a low bitumen content, compared to the standard recommended values: 4.5 % for M1/M2, 5.7 % for AC, 4.0 % for PM and 5.9 % for SMA. PM reference void content was 18.1 %. All mixtures passed the stability requirements, with all reference values extracted from the Romanian standard applicable at the time of study. This latter aspect shows that the obtained flow values exceed the maximum allowable ones with up to 10-20 %, except mixture M2 for the binder course.

Thermoplastic grain polymers consist of small and flexible grains which are added during the preparation of hot bituminous mixtures in order to improve their physico-mechanical properties, mainly their stiffness and their resistance to permanent deformations. They can be added directly in the mixer during production, after the aggregates and before the bitumen. In this study, the effects of four thermoplastic polymers (P1, P2, P3, P4) on the stiffness modulus, dynamic creep and fatigue resistance of bituminous mixtures were analysed. All tested polymers were introduced as grains during mixture preparation. Product P4 was added only to mixture AC1. The main properties of the thermoplastic polymers used in this study are presented in Table 3.

Table 3 Thermoplastic polymers properties

Properties	Polymers			
	P1	P2	P3	P4
Composition	low-density PE, EVA	low-density PP, PE	low-density polymers	low-density polymers
Aspect / colour	gray, 1.5-4.0 mm soft grains	black powder / grains	brown, soft grains	transparent, soft grains
Soft. point [°C]	150-160	150	160	100
Melt. point [°C]	180	155-170	180	120
Density at 20 °C [kg/m^3]	900-980	900-980	550-650	550-650
Dosage [% of bitumen]	4-6	4-6	4-8	4-8

3 Methodology and results

3.1 Stiffness modulus

Under normal loading conditions, asphalt mixtures show a viscoelastic behaviour, therefore the fundamental elastic modulus used for linear elastic materials does not apply. Asphalts are characterised by the stiffness modulus, where the stress-strain relation depends on the temperature and loading rate, unlike the fundamental Young's modulus. The commonly used stiffness modulus is the dynamic one, where the applied load is dynamic (short loading rate) [3]. The stiffness modulus is determined according to EN 12697-26 [9]. Indirect tensile tests, bending tests or direct uniaxial tests can be performed. In this study, indirect tensile tests on cylindrical specimens were performed at 20 °C, under controlled strain conditions.

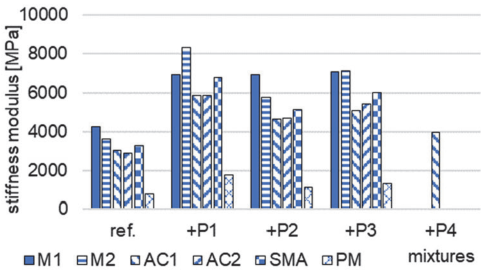

Figure 1 Stiffness modulus

Adding thermoplastic grain polymers clearly improves the mixtures' stiffness moduli (Figure 1). On average, polymer addition led to a 31 % (P4) to 104 % (P1) stiffness improvement. Since P4 was only used for mixture AC1, its relevance is limited. The largest increase was obtained with polymer P1, which doubled every mixtures' stiffness, except M1. Adding any of the tested polymers to mixture M1 improved its stiffness modulus with 65 %. Mixture M2 has shown the highest average stiffness increase (+95 %). The lowest improvement was obtained using polymer P2, which increases the mixture stiffness with 55-60 % on average.

3.2 Permanent deformation

The permanent deformation of an asphalt mixture is closely related to its low-stiffness response (i.e. response at high temperature and long loading time). It depends on the aggregate and bitumen volume, grading, shape, texture and hardness, as well as on the degree of interlocking and compaction. The mixture deformation and strain quickly increase during the beginning of the loading time, and become quasi-constant afterwards. The total deformation is partially recovered instantaneously (elastic deformation) when removing the applied load. Another fraction of the total deformation is recovered gradually (viscoelastic deformation), whereas the rest cannot be recovered (permanent deformation). The latter represent the mixture rutting [3].

3.2.1 Cyclic compression tests

Testing a bituminous mixture's resistance to permanent deformation by cyclic compression with confinement is described in EN 12697-25 [9]. The creep test is conducted under dynamic loading and allows ranking various mixes or checking the acceptabiity of a given material. They do not provide the possibility to predict field rutting from a quantitative point of view. In this study, method B was used, testing the mixtures by means of the triaxial cyclic compression test. This method is usually employed to develop and evaluate new mixture types. The confining stress is induced by vacuum as the mixture specimen is subjected to a cyclic axial stress. Mixtures M1 and M2 were tested at 40 °C, with a 200 kPa axial stress. The other mixes were tested at 50 °C and 300 kPa, being designed for the wearing course. The deformations recorded after 10,000 loading/unloading cycles are synthesised in Figure 2.

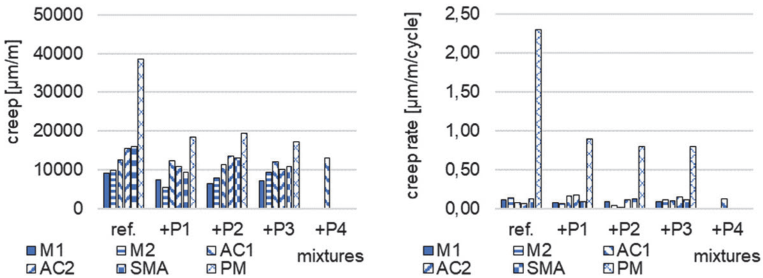

Figure 2 yclic compression tests: creep (left) and creep rate (right)

Tested polymers reduce creep for all mixture specimens on average with 23 % (P2) to 31 % (P1). A significant effect is observed on the PM porous mixture. Its reference creep of almost 40,000 µm/m, as well as its creep rate of 2.30 µm/m/cycle, exceed the standard allowable values. However, adding thermoplastic grain polymers reduces creep by half and creep rate by two thirds. Although creep is reduced by adding polymers, in some cases (i.e. asphalt concrete mixes AC1 and AC2) creep rate increases at the same time.

3.2.2 Wheel tracking tests

The purpose of the wheel-tracking test is to determine the asphalt susceptibility to permanent deformation under a moving load. The test procedure is described in EN 12697-22 [9]. It is performed in a controlled environment, simulating real conditions. In this study, rectangular specimens were prepared in the laboratory and tested using a small-size device, procedure B, testing in air. The applied load is fixed, whereas the table with the specimen's mould is repeatedly moving forwards and backwards. Tests were performed at 60 °C, on mixtures designed for wearing courses.

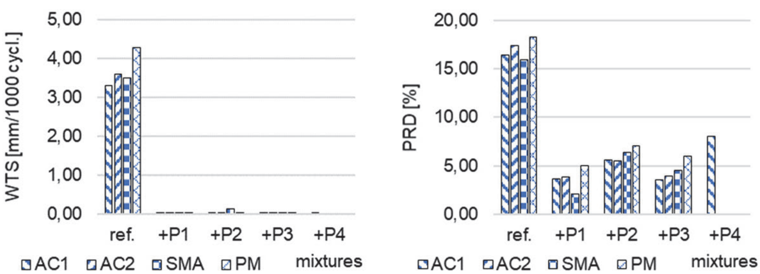

Figure 3 Wheel tracking tests: WTS (left) and PRD (right)

The wheel-tracking slope (WTS) is the average value of the tested specimens (two in each case). While the values obtained for the reference mixtures exceed the allowable limits, adding grain polymers substantially improves WTS (Figure 3). The proportional rut depth (PRD [%]) is determined by dividing the vertical specimen displacement after 10,000 cycles to the initial specimen height. In this study, the reference mixtures' PRD values were three times the recommended limits. Asphalt mixtures including P1 or P3 polymers exhibit improved PRD with 75 % to 80 % on average. The results (PRD ≤ 5,00 %) indicate these mixtures could be used on almost any type of road. Polymers P2 and P4 improve the PRD with 50 % to 65 % on average. However, the obtained values (5.5 % to 8.0 %) recommend those asphalt mixtures for secondary roads only.

3.2.3 Resistance to fatigue

In this study, the resistance to fatigue tests were performed on cylindrical specimens of M1 and M2 hot mix asphalts, according to EN 12697-24 [9]. This standard indicates five alternative test methods: an indirect tensile test on cylindrical specimens and four bending direct tensile tests on prismatic and trapezoidal specimens. In this case, the indirect tensile test on cylindrical specimens was used.
The resistance to fatigue may be used to rank asphalt mixtures or as a pavement performance indicator to estimate/evaluate its structural behaviour. The conventional failure criterion is the number of load application when the mixture's complex stiffness modulus decreases to half its initial value.
The laboratory-prepared specimens were 40 mm thick and 100 mm in diameter. After ageing, they were conditioned and tested at 15 °C. Three specimens were tested at each of the three constant stress levels of pulse.
The reference mixtures performed poorly, with values as low as 30 % from the minimum allowable number of loads. As Figure 4 shows, adding thermoplastic grain polymers significantly improved the tested mixtures' resistance to fatigue: 140 % average increase for mixture M1 and 300 % for mixture M2. The obtained results exceed the minimum allowable values with 5 to 20 %.

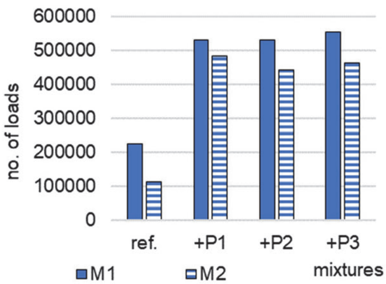

Figure 4 Resistance to fatigue

4 Discussion and conclusions

This study, as well as others [7-12], have confirmed that thermoplastic polymers, added as PMB components or as grains during PMA production, increase the asphalt stiffness and its resistance to deformations. Therefore, these agents are recommended to limit or eliminate common pavement distresses such as rutting. However, a careful and rational approach must be ensured in order to obtain the required results without negatively affecting other characteristics.
PMAs provide equilibrium between asphalt properties, ensuring both stiffness under high temperatures as well as flexibility at low temperatures. This is a useful method to extend the pavement lifespan and reduce maintenance costs [5]. Using grain polymers is currently an effective alternative to PMB, because of several technological and economical advantages. Polymer quality is essential to obtain adequate mixture characteristics.

Remark

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References

[1] Ciont, N., Har, N., Iliescu, M.: On the importance of aggregate mineralogy in asphalt pavement distress, 5th International Conference on Road and Rail Infrastructure - CETRA 2018, pp. 395-401, Zadar, Croatia, 17-19 May 2018, doi: 10.5592/CO/CETRA.2018.761

[2] Har, N., Lăzărean, A., Iliescu, M., Ciont, N., Abrudan, I.F.: Degradation processes of iron-sulfides and calcite containing aggregates from asphaltic mixtures, Construction and Building Materials 212 (2019), pp. 745-754, doi: 10.1016/j.conbuildmat.2019.04.018

[3] Nikolaides, A.: Highway Engineering, ISBN 978-1-138-89376-4, CRC Press, 2017.

[4] Toraldo, E., Mariani, E.: Effects of polymer additives on bituminous mixtures, Construction and Building Materials 65 (2014), pp. 38-42, doi: 10.1016/j.conbuildmat.2014.04.108

[5] Hunter, R.N., Self, A., Read, J.: Shell Bitumen Handbook, 6th edition, Shell International Petroleum Company, ISBN 978-0-7277-5837-8, doi: 10.1680/tsbh.58378.149, 2015.

[6] Cardone, F., Ferrotti, G., Frigio, F., Canestrari, F.: Influence of polymer modification on asphalt binder dynamic and steady flow viscosities, Construction and Building Materials 71 (2014), pp. 435-443, doi: 10.1016/j.conbuildmat.2014.08.043

[7] Brovelli, C., Crispino, M., Pais, J., Pereira, P.: Using polymers to improve the rutting resistance of asphalt concrete, Construction and Building Materials, 77 (2015), pp. 117-123, doi: 10.1016/j.conbuildmat.2014.12.060

[8] EN 13043: Aggregates for bituminous mixtures and surface treatments for roads, airfields and other trafficked areas.

[9] EN 12697: Bituminous mixtures. Test methods.

[10] Munera, J.C., Ossa, E.A.: Polymer modified bitumen: Optimization and selection, Materials and Design, 62 (2014), pp. 91-97, doi: 10.1016/j.matdes.2014.05.009

[11] Meena, S., Biligiri, K.P.: Binder rheological predictive models to evaluate rutting performance of asphalt mixtures, Construction and Building Materials, 111 (2016), pp. 556-564, doi: 10.1016/j.conbuildmat.2016.02.151

[12] Domingos, M.D.I., Faxina, A.L., Bernucci, L.L.B.: Characterization of the rutting potential of modified asphalt binders and its correlation with the mixture's rut resistance, Construction and Building Materials, 144 (2017), pp. 207-213, doi: 10.1016/j.conbuildmat.2017.03.171

THE EFFECT OF BITUMEN AGEING TO FRACTIONAL COMPOSITION

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Abstract

Bitumen is manufactured from different crude oil sources also by various refining technologies, therefore, the impact of ageing is different. Bitumen is a complex chemical mixture consisting of a large number and diversity of organic compounds, mostly hydrocarbons, and varying in molecular mass, polarity and aromaticity. Various polar and non-polar fragments in bitumen interacting in-between form the certain structures which changes bitumen behaviour. Since bitumen is assigned as a colloidal system, consisting of high molecular weight asphaltene micelles dispersed in a lower molecular weight maltenes (saturates, aromatics, resins), the bitumen structure changes over time. Since ageing is one of the main factors effecting bitumen properties and asphalt pavement performance, it is essential to understand how the fractional composition of bitumen is affected by the long-term ageing simulation in the laboratory. The main purpose of this article is to analyse bitumen ageing process and influence to bitumen fractional composition (saturates, aromatics, resins, and asphaltenes), i.e. to show what happens to the bitumen fractional composition and colloidal stability when bitumen reaches a critical ageing point. The saturates, aromatics, resins, and asphaltenes were determined with the Thin Layer Chromatography with flame-ionisation detector (TLC/FID), the IATROSCAN MK-6s.

Keywords: bitumen, fraction composition, SARA, ageing, oxidation, colloidal stability

1 Introduction

Determination of detailed bitumen chemical composition has been a major challenge for many years since it varies every time depending on the crude oil source, the batch of crude oil as well as the refining technology [1, 2]. Crude oil is the base for lots of products which variety depends on the market demand. Since crude oil products include transportation fuels such as diesel, gasoline, fuel oils used for heating and electricity generation, synthetic fibres as well as many other products, bitumen, as the end distillation product, differs with each production process [3]. The increase in demand for petroleum products encourages scientists to take a deeper interest in the properties of bitumen. The knowledge of crude oil source and processing is essential because it helps to understand bitumen behaviour under different conditions [4]. In general, crude oil mainly contains hydrocarbons and small quantities of nitrogen, oxygen, and sulphur compounds as well as some metal components such as iron, nickel, copper and palladium [5, 6]. The chemical structure of different crude oil directly effects the refining technology and the bitumen quality. Naphthenic type of crude oil is one of the best option for bitumen production since it contains a large amount of naphthene, whereas paraffinic and aromatic crude oils contain large amounts of saturated and aromatic hydrocarbons accordingly.

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Another relevant aspect effecting bitumen chemical structure is non-reversible ageing process [7, 6, 8]. Bitumen ageing is caused by the processes of oxidation, volatilization and steric hardening which are directly related to oxygen, ultraviolet light and heating. Volatilization and oxidation of bitumen are effected by chemical reactions or the changing of the bitumen structure, whereas steric hardening of bitumen occurs by intermolecular reconstruction [9, 7, 10]. Bitumen starts quickly ageing during the storage, asphalt mixing, transportation, laying process (short-term ageing) as well as under the influence of oxidation in the asphalt pavement structure during the service life (long term ageing) [11]. During these two ageing periods, two phases of bitumen ageing where determined: in the first phase (short term ageing) - mainly sulphur oxidation, in the second phase (long term ageing) – benzylic position oxidation. Petersen, C. and Glaser, R. (2011) and other authors analysed bitumen oxidation mechanisms, especially the formation of ketones, identified as a major factor leading to asphaltenes formation, which leads to viscosity increase on ageing [7, 12-14].
Bitumen consists of an infinite variety of organic compounds that vary widely in molecular structure, molecular size and polarity [15, 16]. Analysing bitumen chemical composition, especially fractional composition (SARA), the attention falls into the components (saturates, aromatics, resins and asphaltenes) interaction which influence bitumen behaviour and changes the sensitivity to ageing. SARA fractions have been commonly used to study the chemical changes in bitumen due to oxidative ageing [17]. Bitumen itself is non-polar comparing with the environment however it contains slightly more polar molecules (such as asphaltenes) and non-polar molecules (saturates). The polarity is very important aspect since it effects the oxidation process in bitumen [18]. Changes in chemical fractions (SARA) due to oxidation process have been interpreted as a movement of components from non-polar fractions up to the more polar fractions (i.e. from saturates to asphaltenes) [9]. The increase of polarity creates a way to oxygen-containing functional groups in bitumen's molecules which have different reactivity to oxidation. Finally, we get a loss of aromatics and resins, with a consequent increase in the number of asphaltenes. Saturate fraction is highly resistant to oxidation since they are non-polar and, as a consequence, has low reactivity [9, 12].
Lots of investigations have been done on bitumen ageing process and influence to bitumen fractional composition [19, 20]. However, there is limited research done on how the fractional composition of bitumen changes after extremely long term ageing and how it effects asphalt pavement performance under the real conditions. Chemical changes of bitumen before and after ageing include the formation of functional groups, transformation of fractions, changes in microstructure and in molecular weight. Seki, H. and Kumata, F. investigated structural changes of asphaltenes and resins by hydrodemetallization (HDM) reaction. From the Laser Desorption Mass Spectrometry (LD-MS), the molecular weight of both asphaltenes and resins decreased with HDM temperature, showing an opposite change in molecular weight distribution [21]. Cuciniello, G. et all investigated the changes in the microstructure of SBS modified bitumen. The results showed that neat bitumen binders after one cycle of long term ageing (PAV), bitumen components, such as aromatics, resins and asphaltenes, shifted towards heavier molecules [22]. Aguiar-Moya, J. P. et all (2017) concluded that colloidal stability of bitumen fraction is affected by a loss of low molecular weight components as well as air oxidising the bitumen. It was determined that increase in stiffness depends on polar components, while adhesiveness is associated mainly with nonpolar components. Other scientists Redelius and Soenen (2015) determined that increased stiffness of bitumen is due to increased interactions between the molecules. During the chemical reaction with oxygen, increases the polar interaction in bitumen, the size of molecules as well as increases the condensed aromatics, as a consequence, increasing other molecular interactions such as and dispersive interactions [23]. Filippelli, L. et al. 2012 determined that aged bitumen binders have higher reprocessing temperatures because some of their aromatic components and resins, which are responsible for a certain grade of mobility, are oxidized to asphaltenes.

Hence, asphaltene micelles became larger so that the fluidity of the system is reduced [24]. The main purpose of this article is to analyse ageing process at molecular level and to investigate how bitumen fractional composition can be effected by extended long-term ageing. In order to structurally characterize bitumen, the SARA (saturates, aromatics, resins, asphaltenes) method was used [17], which relies on Chromatography separation, based on polarity and molecular mass.

2 Experimental research

2.1 Materials

The original (unaged) neat 100/150 bitumen was chosen from two kind of crude oils (paraffinic and naphthenic) for the experimental research (Table 1). This type of bitumen was selected due to the research orientation to one-layer asphalt pavements, which usually are used for lower volume roads. Moreover, previous research showed that there is no big difference in bitumen structural composition depending on bitumen penetration grade.

Table 1 Summary of tested materials

Bitumen Penetration Grade	Ageing	Code (Paraffinic Crude Oil)	Code (Naphthenic Crude Oil)
100/150	Unaged	P-NS	N-NS
100/150	RTFOT	P-RT	N-RT
100/150	PAV I (20 h)	P-PV20	N-PV20
100/150	PAV II (40 h)	P-PV40	N-PV40
100/150	PAV III (60 h)	P-PV60	N-PV60
100/150	PAV VI (80 h)	P-PV80	N-PV80
100/150	PAV V (100 h)	P-PV100	N-PV100
100/150	PAV VI (120 h)	P-PV120	N-PV120
100/150	PAV VII (140 h)	P-PV140	N-PV140

2.2 Methods

2.2.1 Laboratory ageing procedure

The neat bitumen 100/150 from naphthenic and paraffinic crude oils were aged in eight steps: (1) short-term ageing to simulate the ageing effect of asphalt mixture production and layer compaction; (2) long-term ageing (20 h) to simulate 5-15 years of pavement in-service; (3) extended (40 h) long-term ageing; (4) extended (60 h) long-term ageing; (5) extended (80 h) long-term ageing; (6) extended (100 h) long-term ageing; (7) extended (120 h) long-term ageing; (8) extended (140 h) long-term ageing. The Rolling Thin Film Oven Test (RTFOT) was used for short-term ageing of bitumen samples according to EN 12607-1 [25]. The bottles with 35±0.5 g of bitumen were placed in the oven with carousel at 163ºC, where hot air was periodically injected inside at a rate of 4000 ± 200 ml/min for 75 min. The Pressure Ageing Vessel (PAV) test was used for long-term and extended long-term ageing of bitumen after RTFOT according to EN 14769 [26]. The pans with 50±0.5 g of bitumen were placed in the pressure camber. The long-term ageing for 20 h and 40 h were performed at 100°C and 2.1±0.1 MPa air. The extended long-term ageing for 60 h, 80 h, 100 h, 120 h, 140 h were performed at 85°C and 2.1±0.1 MPa air pressure following the standard EN 14769 requirements.

2.2.2 Bitumen fractional composition (SARA)

The bitumen fractional composition (saturates, aromatics, resins, and asphaltenes - SARA) was determined using the IATROSCAN MK6s Thin-Layer Chromatograph with Flame-Ionization Detection (TLC-FID) with referring to IP 469/01 [27]. The samples were prepared dissolving 1 % ±0.1 % (m/V) bitumen in toluene. After cleaning the activated quartz rods (Chromarods), the 1 μl of sample solution was spotted on each rod with semi-automatic spotter. The frame with ten Chromarods was placed in the drying chamber at 80°C for around 2 min to evaporate the residual toluene. The bitumen fractional composition was separated placing rod frame into three tanks with different solvents: Tank A - n-heptane (100 %) to extract saturates; Tank B - toluene (80 %) and n-heptane (20 %) to extract aromatics; and Tank C - dichloromethane (95 %) and methanol (5 %) to extract resins. Finally, the quartz rods were scanned in the TLC-FID analyser. For each quartz rod the four peak areas were integrated and calculated the percent concentrations for saturates, aromatics, resins and asphaltenes.

3 Results and discussion

3.1 Ageing processes at molecular level

During the short-term ageing, efforts are made to reproduce as closely as possible the conditions for storing, mixing, transporting and laying asphalt mixture, i.e. constant air supply and slow mixing of bitumen at high temperature. On the chemical side, during short-term ageing, the oxidation of a fragment of asphaltene or resin molecule occurs by reacting heteroatoms with atmospheric oxygen to form oxidised functional groups. [O] is a general representation of an oxidizer - it can be oxygen (O_2) or an oxygen atom from other functional oxidative properties, for example, from sulfoxide.

During the short term ageing of bitumen most often occurs oxidation of sulphur. Oxidation of thioether to the sulfoxide during a short-term ageing is shown in Figure 1.

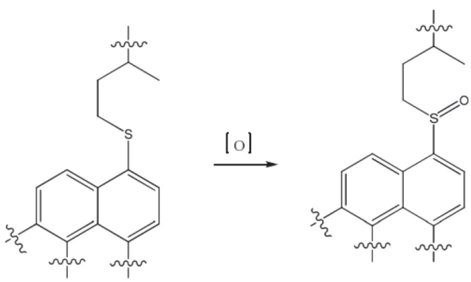

Figure 1 Oxidation of thioether to the sulfoxide during a short-term ageing

Long-term ageing seeks to influence the rheological properties of bitumen therefore the temperature is reduced up to 85-100 °C in order to reduce chemical reactions, to create atmospheric pressure, and maintain such conditions for longer. On the chemical side, during the long-term ageing, the oxidation of the asphaltene or resin fragment of molecule with atmospheric oxygen occurs when the oxygen molecule tears the hydrogen atom from the benzylic position or near the double bond to form carboxyl groups, ketones and etc. (Figure 2).

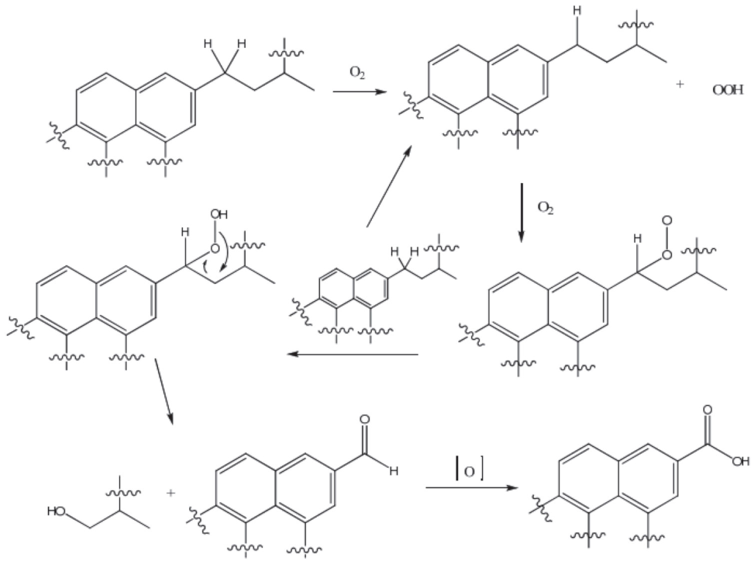

Figure 2 Oxidation of benzylic-type situation during a long-term ageing

3.2 SARA results

Four fractions, namely saturates, aromatics, resins and asphaltenes (SARA), were obtained from the same type of asphalt binder (100/150) but different crude oil sources (naphthenic and paraffinic). The results of SARA fractions were analysed by a Thin Layer Chromatography to evaluate their sensitivity to ageing (Table 1). The main idea was to check how bitumen fractions (saturates, aromatics, resins, asphaltenes) are changing depending on the ageing time as well crude oil type. The average values of five rod readings of bitumen fractional composition was used for analysis.

Investigation of the SARA fractions of bitumen after extended long-term ageing up to 140 h proved that ageing has a direct effect on transformation of fractions interpreted as a movement of components from non-polar fractions up to the more polar fractions (i.e. from saturates to aromatics – resins - asphaltenes). The results showed that unaged bitumen had the highest amount of aromatics and its amount decreased over ageing time. The dramatic decrease of aromatic fractions occurred in the period of short term ageing and 20 h of long term ageing and its content constantly decreased over ageing time. In terms of resins, the unaged bitumen had the lowest amount of this fraction which increased with the ageing time. The significant increase of resins occurred in the period of short term ageing and 20 h of long term ageing. It was expected that amount of asphaltenes will increase significantly over ageing time however asphaltene fraction increased slowly.

Table 2 Results of SARA fractions

Bitumen	Asphaltenes	Resins	Aromatics	Saturates
P-Neat	14,67	25,42	54,34	5,57
N-Neat	19,99	23,27	50,01	6,73
P-RT	16,94	25,13	52,33	5,60
N-RT	21,96	21,34	49,54	7,15
P-PV20	16,50	38,32	39,81	5,63
N-PV20	23,29	35,00	34,97	6,75
P-PV40	18,86	42,70	32,81	5,63
N-PV40	24,61	37,33	31,43	6,96
P-PV60	14,74	42,89	36,27	5,70
N-PV60	19,43	38,13	35,37	7,06
P-PV80	15,37	42,99	36,38	5,54
N-PV80	20,33	40,13	32,43	6,94
P-PV100	13,37	51,27	29,42	5,78
N-PV100	19,28	46,58	26,59	7,02
P-PV120	16,17	52,10	26,21	5,52
N-PV120	22,90	46,44	23,49	7,04
P-PV140	16,99	53,38	24,28	5,36
N-PV140	22,82	46,95	24,25	6,52

Since saturate fraction is known as stable paraffin's fraction, a mixture of pure aliphatics, aliphatics with side chains, cycloaliphatics, and cycloaliphatics with side chains, the content of saturates remains almost constant during the overall ageing process. To conclude, the ageing limit of bitumen was not reached. Longer ageing or different ageing procedure is needed in order better represent long term ageing and also so that the chromatography could show resins transformation into asphaltenes (Figure 3).

Analysis of bitumen SARA fractions from different crude oil showed that bitumen from naphthenic crude oil has obviously larger amount of asphaltenes comparing with the paraffinic crude oil, less resin and aromatic fractions. Since naphthenic crude oil contains a higher amount of sulphur, the short term ageing is faster in bitumen from naphthenic crude oil. However, in the long term perspective, the naphthenic crude oil is more resistance to ageing than paraffinic crude oil.

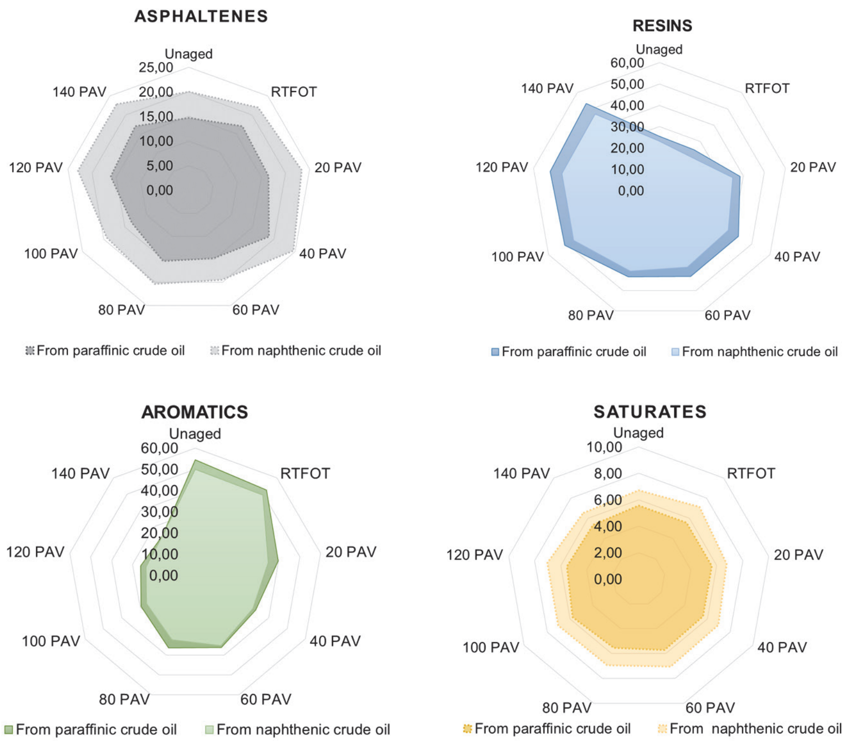

Figure 3 Comparison of SARA fractional composition depending on the ageing time and crude oil type

4 Conclusions

The study presented in this article analysed the ageing effect of neat (100/150) bitumen from naphthenic and paraffinic crude oils. The bitumen was aged up to 140 hours and after every 20 hours of ageing tested with a Thin-Layer Chromatograph. The following conclusions are summarised below:

- Short and long term ageing processes of bitumen were presented at molecular level which showed the main chemical processes and final chemical products obtained. During these two ageing processes, two phases of bitumen ageing where determined: in the first phase (short term ageing) - mainly oxidation of sulphur occurred, in the second phase (long term ageing) – oxidation of benzylic position.
- Investigation of SARA fractions of bitumen after extended long-term ageing (up to 140 h) proved that ageing has a direct effect on transformation of fractions interpreted as a movement of components from non-polar fractions up to the more polar fractions. Increasing the ageing time, the evident transformation was noticed on decrease of aromatics and increase of resins. Asphaltene fraction increased slightly, not as it was expected, since the size of asphaltene-type molecules has not increased in molecular mass yet enough so that Chromatography would attribute it as asphaltene fraction. The saturates fraction remained stable because of their low reactivity and non-polarity.
- SARA fractions showed their sensitivity to ageing especially aromatics and resins. The significant increase of resins and decrease of aromatics occurred during the period of short term ageing and 20 h of long term ageing as well as during the period of 80 h and 100 h of extended long term ageing.

- After maximum 140 h of ageing, the ageing limit or the expiry date of bitumen was not reached. That means that standard long-term ageing procedure do not represent natural long term ageing. Longer ageing or different ageing procedure is needed so that Chromatograph could show the transformation of resins into asphaltenes.

References

[1] Rossini, F.D.: Hydrocarbons in Petroleum, 1906; 554–61

[2] Sakib, N., Bhasin, A.: Measuring polarity-based distributions (SARA) of bitumen using simplified chromatographic techniques, International Journal of Pavement Engineering, 20 (2019) 12, pp. 1371-1384, DOI: 10.1080/10298436.2018.1428972

[3] Speight, J.: The Chemistry of Technology of Petroleum, Taylor Francis Group, 2006.

[4] Paliukaitė, M., Vaitkus, A., Zofka, A.: Influence of bitumen chemical composition and ageing on pavement performance, Baltic Journal of Road and Bridge Engineering, 10 (2015), pp. 97–104

[5] Nagy, B.U.C.: Fundamental Aspects of Petroleum Geochemistry, 1967.

[6] Al-Dahhan, W.H., Mahmood, S.M.A.: Classification of Crude Oils and its Fractions on the Basis of Paraffinic, Naphthenic and Aromatics, 2019.

[7] Petersen, J.C.: TRB Transportation Research Circular E-C140: A Review of the Fundamentals of Asphalt Oxidation: Chemical, Physicochemical, Physical Property, and Durability Relationships, 2009, doi: DOI: 10.17226/23002

[8] Taylor, P., Airey, G.D.: State of the Art Report on Ageing Test Methods for Bituminous Pavement Materials, International Journal of Pavement Engineering, 4 (2003) 3, pp. 165–176, doi: 10.10850/1029843042000198568

[9] Petersen, J.C., Harnsberger P.M. R.: Factors affecting the kinetics and mechanisms of asphalt oxidation and the relative effects of oxidation products on age hardening, Chemical Society, Division of Fuel Chemistry, 41 (1996) 4

[10] Yao, H., Dai, Q., You, Z.: Fourier Transform Infrared Spectroscopy characterization of aging-related properties of original and nano-modified asphalt binders, Construction and Building Materials, 101 (2015), pp. 1078–87, doi: org/10.1016/j.conbuildmat.2015.10.085

[11] Jing, R., Varveri, A., Liu X., Scarpas, A.: Ageing effect on chemo-mechanics of bitumen, Road Materials and Pavement Design, 22 (2019) 5, pp. 1044–1059, doi: org/14680629.2019.1661275

[12] Petersen, J.C.: The Chemistry of Technology of Petroleum J.C.: Chemical Composition of Asphalt As Related To Asphalt Durability: State of the Art, Transportation Research Record, pp. 13–30, 1984.

[13] Petersen, J.C., Glaser, R.: Asphalt Oxidation Mechanisms and the Role of Oxidation Products on Age Hardening Revisited, Journal Road Materials and Pavement Design, 12 (2011) 4, pp. 795-819, 2011.

[14] Petersen, J C.: Quantitative functional group analysis of asphalts using differential infrared spectroscopy and selective chemical reactions – application and theory, Transportation Research Record, pp.106–119, 1986.

[15] Wang, T., Wang, J., Hou, X., Xiao, F.: Effects of SARA fractions on low temperature properties of asphalt binders, Road Materials and Pavement Design, 2019, pp. 1–18, doi.org/14680629.2019.1628803.

[16] Lesueur, D.: The colloidal structure of bitumen: consequences on the rheology and on the mechanisms of bitumen modification. Advances in Colloid and Interface Science, 145 (2009), pp. 42–82

[17] Corbett, L.: Composition of asphalt based on genericfractionation, using solvent deasphaltening, elution-adsorption chromatography, and densimetric characterization. Analytical Chemistry, 41 (1969) 4, pp. 576–579

[18] Eberhardsteiner, L., Füssl, J., Hofko, B., Handle, F., Hospodka, M., Blab, R.: Towards a microstructural model of bitumen ageing behaviour, International Journal of Pavement Engineering, 16 (2015), pp. 939–49

[19] Stangl, K., Jäger, A., Lackner, R.: Microstructure-based identification of bitumen performance, Road Materials and Pavement Design, 7 (2006), pp. 111–42, doi: abs/10.1080/14680629.2006.9690061.

[20] Qin, Q., Schabron, J.F., Boysen, R.B., Farrar, M.J.: Field aging effect on chemistry and rheology of asphalt binders and rheological predictions for field aging, Energy Fuels, 121 (2014), pp. 86–94, doi: org/10.1016/j.fuel.2013.12.040.

[21] Seki, H., Kumata, F.: Structural Change of Petroleum Asphaltenes and Resins by Hydrodemetallization, Energy Fuels, 14 (2000) 5, pp. 980–985, doi: https://doi.org/10.1021/ef000009m

[22] Cuciniello, G., Leandri, P., Filippi, S., Presti, D., Polacco, G., Losa, M., Airey, G.: Microstructure and rheological response of laboratory-aged SBS-modified bitumens, Road Materials and Pavement Design, 22 (2019) 2, pp. 372-396, doi: org/14680629.2019.1621771.

[23] Redelius, P., Soenen, H.: Relation between bitumen chemistry and performance, Energy Fuels, 140 (2015), pp. 34–43, doi: 10.1016/j.fuel.2014.09.044

[24] Filippelli, L., Gentile, L., Rossi, C.O., Ranieri, G.A., Antunes, F.E.: Structural Change of Bitumen in the Recycling Process by Using Rheology and NMR, Industrial & Engineering Chemistry Research, 51 (2012) 50, doi: 10.1021/ie301899v.

[25] LST EN 12607-1: Bitumen and bituminous binders - Determination of the resistance to hardening under influence of heat and air - Part 1: RTFOT method, Lithuanian standards board, Vilnius, 2015.

[26] EN 14769: Bitumen and bituminous binders - Accelerated long-term ageing conditioning by a Pressure Ageing Vessel (PAV), 2012.

[27] IP 469/01: Determination of saturated, aromatic and polar compounds in petroleum products by thin layer chromatography and flame ionization detection, 2006.

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A REVIEW OF THE BEST EXPERIENCE ON CRUMB RUBBER – DRY PROCESS MODIFIED ASPHALT MIXTURE PERFORMANCE

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Abstract

The use of crumb rubber made from end of life tyres for asphalt mixtures modification in order to improve their properties or just utilize waste products may be considered as potential solution. Crumb rubber can be used as a bitumen modifier (wet process) or supplementary component of the asphalt mixture (dry process). Dry modification process has more potential due relatively unsophisticated technology and higher possible to use amount of crumb rubber comparing to the wet process. The performance of asphalt mixtures modified by dry process mainly depends on several factors as crumb rubber type, content and size. However, limited number of publications reported the results of dry method crumb rubber modified asphalt mixtures performance. This paper summarizes the latest findings from literature review on the modification technologies and specifications related to dry modification process, the effect of crumb rubber type and amount on modified asphalt mixture performance in terms of stiffness, rutting resistance, water sensitivity, resistance to fatigue and low temperature cracking. The algorithm of crumb rubber modified asphalt mix design was introduced.

Keywords: tyre recycled rubber, crumb rubber, dry process, rubber modified asphalt mix

1 Introduction

Each year increasing amount of vehicles is the reason of endless number of used tyre, which now is a worldwide problem. Nevertheless, according to European Tyre and Rubber Manufacturers' Association (ETRMA) 3.5 million tonnes of used tyres were collected and treated for material recycling or energy recovery [1]. One of possible solution may be tyre recycling [2, 3]. Recycled tyres rubber can be reused in other products. One popular way is to obtain crumb rubber from tyres and to use it in road section [3]. In order to effectively apply crumb rubber into pavement the most rational process must be chosen. There are two traditional ways to do it: wet process and dry process. Many researches prefer wet process [4–7] because it has better performance than dry process. Usually better performance is caused by adhesion between binder and crumb rubber, because in wet process firstly are mixed binder and crumb rubber, while in dry process binder and crumb rubber are added directly into mixer with the others mixture components. Due to, uncertain bond is form between crumb rubber and binder in dry process. Nevertheless, from 2 to 4 times higher amount of crumb rubber can be used during dry process [4, 7, 8]. Moreover, dry process does not require any special equipment and it is more easier to apply crumb rubber into mixture because crumb rubber is added to mixture into common mixer just before binder [4, 9, 10].

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Moreover, there is lack information of amount of required crumb rubber, how to correctly apply crumb rubber to asphalt mixture, if additives are needed or not. This article/paper represents literature review of general crumb rubber applying into asphalt mixture in dry process.

2 Existing crumb rubber modifying technologies

There are two main technologies of applying crumb rubber into asphalt mixtures. First one is wet process, the second one is dry process. Essential difference between these methods is that crumb rubber is added to into bitumen in wet process and like other component of mixture in dry process (Fig. 1). Dry process is described more in details (mixing queue, temperature, time) in 2.2 paragraph.

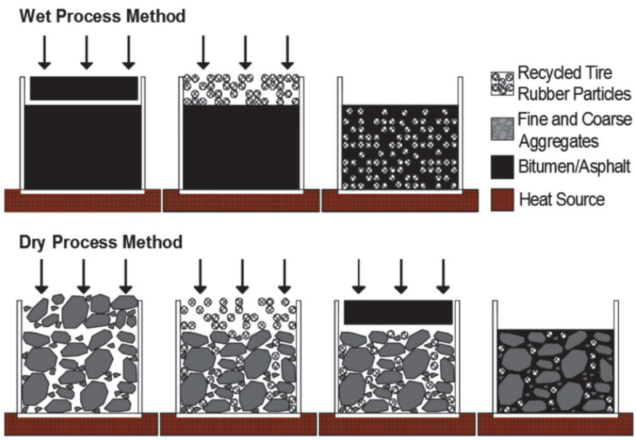

Figure 1 Wet and dry processes principles [11]

2.1 Crumb rubber size and amount in dry process

In the past, there were popular to use larger size crumb rubber, which could be up to 10 mm [8, 12, 13], but due to poor performance of asphalt mixture, smaller particles were started to use. It is important to note that smaller crumb rubber particles react faster and better with bitumen. Moreover, swell up to five times while contacting with the binder, because crumb rubber absorbs binder molecules. Due that fact, some researches advice to increase the amount of binder [14, 15].

Nowadays most commonly used amount of crumb rubber in dry method is 1 – 3 % by total weight of mixture [8, 9, 14, 16–19]. However, Moreno et al. [5, 20] used 0.5-2.0 % crumb rubber, which size ≤ 0.6 mm.

2.2 Mixing procedure in dry process

In many cases this will not be necessary as this template is programmed In order to correctly apply crumb rubber into mixture authors [9, 18, 21, 22] suggest following steps. Firstly, aggregates are preheated at 175 - 190 ºC and mixed. Secondly, crumb rubber is added to mixture and again mixed and mixed for 1 - 1.5 min, while Arabani et al. [9] and Moreno et al. [5] suggest mixing 20 seconds and according to technical regulations by FGSV [22] it is possible mix only 10 seconds. Thirdly, binder, which was preheated at 160 ºC, are added to the mixture and once again mixed for 1 - 2 min at 175 - 190 ºC. Finally, mixture is conditioned in the oven at 160 ºC for 60, 90 or 120 min [5, 6].

2.3 Crumb rubber additives

Using unmodified crumb rubber usually cause poor performance asphalt mixture's properties. Poor performance mostly comes from insufficient adhesion between crumb rubber and bitumen.
One of possible additives are filler and cross-linking agent modified crumb rubber. Usually this additive consists of 62 – 65 % crumb rubber, 20 – 25 % soft bitumen, 15 – 20 % filler and small amount of cross-linking agent. Soft bitumen improves viscosity and workability even if high amount of crumb rubber is used while filler is used to improve interaction between crumb rubber and bitumen. Better adhesion leads to reduced water sensitivity. Elastomeric crumb rubber particles evenly mixes with bitumen, then filler molecules creates an interconnected network with the rubber particles, thereby, forming a cohesive blend of asphalt, rubber, and the stabilizer [23]. This type modified crumb rubber additive is suitable for use with any type of Hit Mix Asphalt: Dense, Open Graded, Gap-graded, SMA to improve resistance to permanent deformation, noise reduction and increase fatigue strength [23, 24].
Another possible crumb rubber additive is a semicrystalline polyoctenamer, which chemically reacts with both crumb rubber and bitumen, due that fact rubber-like, homogenous composite is formed, while alone crumb rubber is recognized as non-reactive additive to bitumen. It can be used with all kind of bitumen and in every sort of asphalt mixtures. This additive is applied to the aggregate mixture before bitumen in dry process to improve workability, properties of mixture such as rutting, cracking, traffic noise. This additive allows to work in lower temperatures as a result decrease emissions, also it helps to avoid sticking between rubber particles. Moreover, rubber less stick to machines of compacting or transporting [25].
To sum up, all additives are smaller than 1 mm, and can be added during both wet and dry process, dosage varies on the type of additive. All crumb rubber additives improve reaction between crumb rubber particles and bitumen. Effect of crumb rubber and additives on asphalt mixtures performance

2.4 The effect of crumb rubber on modified asphalt mixture properties

Many researchers agree that the use of crumb rubber as component in asphalt mixture can not only reduce environmental problems, but also increase asphalt mixture properties: stiffness, rutting resistance, water sensitivity, fatigue, low temperature cracking [9, 20, 26–28].
Arabani et al. [9] found that stiffness modulus using 1 %, 3 % and 5 % of crumb rubber has lower values than conventional mixtures. It is because of low adhesion between crumb rubber and binder during dry process. This theory was supported by other researches Navarro et al. and Rahman et al. [26, 29]. Hassan et al. [14] states that finer crumb rubber particles reacts faster and better in which case values of stiffness is higher. Navarro et al. [30] found that after stiffness test in different temperatures crumb rubber improves stiffness, yet SBS has greater impact on mixture stiffness.
Gradually using crumb rubber from 1 % to 3 % improve rutting resistance of asphalt concrete [13, 31]. Comparing wet process and dry process it was found that increasing amount of crumb rubber increase resistance to rutting of asphalt mixtures in both process. Moreover, using dry process helped to achieve even better result than during wet process [20] or even there is no important difference between values during both process [32]. Rahman et al. [33] adds that using 3 – 5 % and ≤ 1.0 mm crumb rubber fatigue resistance and stiffness improves. Besides, higher amount of crumb rubber helps to achieve better resistance to rutting.
Because of poor adhesion between crumb rubber and binder moisture sensitivity is higher in dry process compared to conventional mixtures [34, 35]. Rahman et al. [33] established that using 3 – 5 % crumb rubber by amount of aggregate, asphalt mixtures are more sensitive to water comparing to conventional asphalt mixture. Navarro et al. [30] found that indirect ten-

sile strength slightly decreases with usage of crumb rubber, nevertheless, water sensitivity remains great even after freeze-thaw cycles.

Crumb rubber applied to asphalt mixture improves fatigue resistance comparing to traditional mixtures [18, 29]. Higher resistance to fatigue is due to higher amount of bitumen and crumb rubber [14, 18]. Tahami et al. [4] found that increasing amount of crumb rubber cause contrary reaction on fatigue.

The addition of 3 % crumb rubber into asphalt mixture cause the best results of resistance of deformation at high (-60 ºC) and low (-10 ºC) temperatures [19]. Cao [19] performed an experiment where crumb rubber was used in amounts 1 %, 2 % and 3 % (crumb rubber size was 1 – 3 mm). It was observed that increasing amount of crumb rubber improved asphalt resistance to cracking at low temperatures (-10 ºC) and the most rational amount of crumb rubber is 3 %.

Although there are many possible crumb rubber additives in crumb rubber market, and all manufactures ensure that their product improves performance, there is lack of published test, which would confirm advantages of different additives – the most of above analysed researches involves not raw crumb rubber modified asphalt mixtures results. According to this, it is necessary to perform experimental research and evaluate the effect of modified crumb rubber on asphalt mixture properties.

3 The algorithm of crumb rubber modified asphalt mixture design

After analysis of literature following chart is drawn, which represents the usage of crumb rubber in asphalt mixture algorithm (Fig. 2). Research should start with proper selection of components, which includes selection of binder, crumb rubber and aggregates. If all components meet requirements, then laboratory test starts. In first testing stage, these properties are determined: air voids, water sensitivity and stiffness. If acceptable values are obtained, then test continue to second testing stage, if not – test returns to selections of proper components. In second testing stage, values of rutting resistance, resistance to fatigue and low temperature cracking are determined. If all properties show good performance, then laboratory test succeed, if not – all test are repeated from proper selection of components.

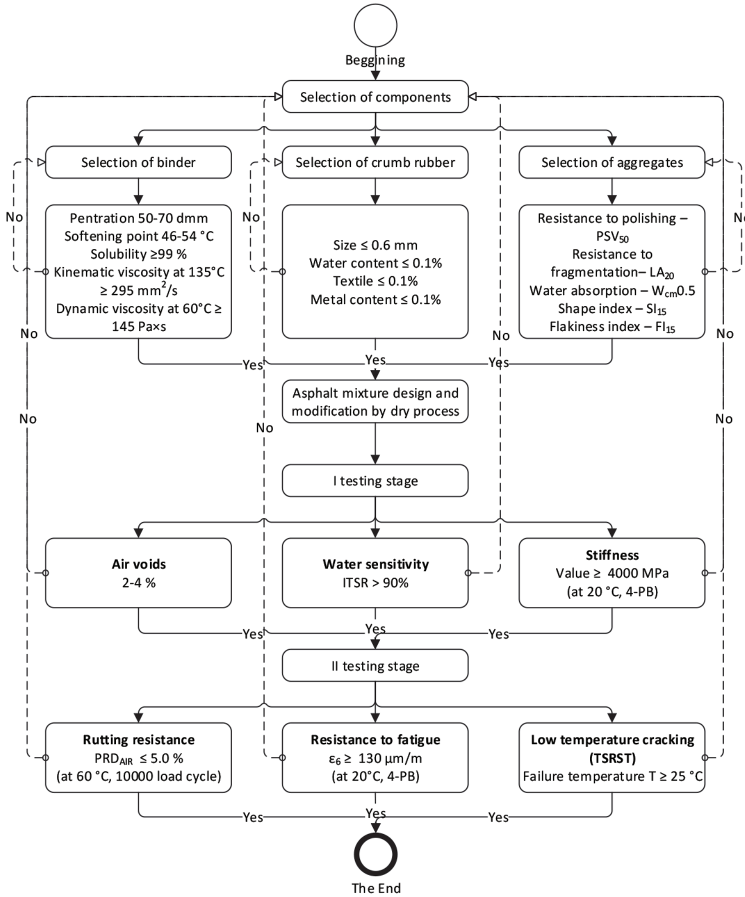

Figure 2 The algorithm of crumb rubber modified asphalt mix design

4 Conclusion

This paper has intended to collect the newest information about the experience of crumb rubber usage for asphalt mixture modification by dry process. Following conclusions can be drawn:

- Literature analysis has shoved the nominal size of crumb rubber – dry process is ≤1 mm (mostly ≤0.8 mm). Smaller particles react better and faster with bitumen, better adhesion between these two components is achieved. Moreover, larger particles in asphalt mixture distributes unevenly and can segregate, while fine particles distribute homogeneously. The most commonly used amount of crumb rubber is 1 – 3 % by mixture weight.
- Most successful dry asphalt modification process starts with aggregate preheating, addition of crumb rubber and intermixing at least 20 seconds. Finally mixing of asphalt mixture for 1-2 minutes after bitumen insertion.
- In general crumb rubber insufficiently decrease asphalt stiffness, but sufficiently increase rutting resistance. Moreover, raw crumb rubber decreases water sensitivity values because of poor adhesion between crumb rubber and bitumen. There is still lack of relative results regarding resistance to low temperature cracking.

- There are several modified crumb rubber additives in the market. Some additives are based on semicrystalline polyoctenamer, which chemically reacts with both crumb rubber and bitumen, while other additives are based on filler and cross-linking agent. The purpose of these additives is to improve interaction between crumb rubber and bitumen. The addition of modified crumb rubber to asphalt mixture should improve properties as rutting resistance and stiffness, but there is still lack of comprehensive research that would prove it.

It can be stated that there is no unified opinion about the usage of crumb rubber for asphalt mixtures modification by dry process and how it impacts the performance of asphalt mixture. Because of many contradictory opinions from researches and the fact that the effect of modified crumb rubber on asphalt mixture performance still is not completely analysed, following experimental research of the modified crumb rubber effect on asphalt mixture properties should be done. Moreover, the difference between raw crumb rubber and modified crumb rubber should be determined.

Remark

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References

[1] European Tyre & Rubber Manufacturers' Association (ETRMA), https://www.etrma.org/wp-content/uploads/2020/09/Copy-of-ELT-Data-2018-002.pdf, 08.03.2021.

[2] Zanetti, M.C., Fiore, S., Ruffino, B., Santagata, E., Dalmazzo, D., Lanotte, M.: Characterization of Crumb Rubber from End-Of-Life Tyres for Paving Applications, Waste Management, 45 (2015), pp. 161-170, doi:10.1016/j.wasman.2015.05.003

[3] Lo Presti, D.: Recycled Tyre Rubber Modified Bitumens for Road Asphalt Mixtures: A Literature Review, Construction and Building Materials, 49 (2013), pp. 863-881, doi:10.1016/j.conbuildmat.2013.09.007

[4] Tahami, S.A., Mirhosseini, A.F., Dessouky, S., Mork, H., Kavussi, A.: The Use of High Content of Fine Crumb Rubber in Asphalt Mixes Using Dry Process, Construction and Building Materials, 222 (2019), pp. 643-653. doi:10.1016/j.conbuildmat.2019.06.180

[5] Moreno, F., Rubio, M.C., Martinez-Echevarria, M.J.: The Mechanical Performance of Dry-Process Crumb Rubber Modified Hot Bituminous Mixes: The Influence of Digestion Time And Crumb Rubber Percentage, Construction and Building Materials, 26 (2012) 1, pp. 466-474, doi:10.1016/j.conbuildmat.2011.06.046

[6] Shen, J., Li, B., Xie, Z.: Interaction Between Crumb Rubber Modifier (CRM) and Asphalt Binder in Dry Process, Construction and Building Materials, 149 (2017), pp. 202-206, doi:10.1016/j.conbuildmat.2017.04.191

[7] Farouk, A.I.B., Hassan, N.A., Mahmud, M.Z.H., Mirza, J., Jaya, R.P., Hainin, M.R., Yaacob, H., Yusoff, N.I.Md.: Effects of Mixture Design Variables on Rubber–Bitumen Interaction: Properties of Dry Mixed Rubberized Asphalt Mixture, Materials and Structures, 50 (2017) 1, pp. 1-10, doi:10.1617/s11527-016-0932-3

[8] Bressi, S., Fiorentini, N., Huang, J., Losa, M.: Crumb Rubber Modifier in Road Asphalt Pavements: State of The Art And Statistics, Coatings, 9 (2019) 6, doi:10.3390/COATINGS9060384

[9] Arabani, M., Tahami, S.A., Hamedi, G.H.: Performance Evaluation of Dry Process Crumb Rubber-Modified Asphalt Mixtures With Nanomaterial, Road Materials and Pavement Design, 19 (2018) 5, pp. 1241-1258, doi:10.1080/14680629.2017.1302356

[10] Bakheit, I., Xiaoming, H.: Modification of The Dry Method For Mixing Crumb Rubber Modifier With Aggregate And Asphalt Based on The Binder Mix Design, Construction and Building Materials, 220 (2019), pp. 278-284, doi:10.1016/j.conbuildmat.2019.06.050

[11] Alfayez, S.A., Suleiman, A.R, Nehdi, M.L.: Recycling Tire Rubber in Asphalt Pavements: State of The Art, Sustainability, 12 (2020) 21, pp. 1-15, doi:10.3390/su12219076

[12] Hernandez-Olivares, F., Witoszek-Schultz, B., Alonso-Fernandez, M., Benito-Moro, C.: Rubber-Modified Hot-Mix Asphalt Pavement by Dry Process, International Journal of Pavement Engineering, 10 (2009) 4, pp. 277-288, doi:10.1080/10298430802169416

[13] Tai Nguyen, H.T., Nhan Tran, T.: Effects of Crumb Rubber Content and Curing Time on The Properties of Asphalt Concrete and Stone Mastic Asphalt Using Dry Process, International Journal of Pavement Research and Technology, 11 (2018) 3, pp. 236-244, doi:10.1016/j.ijprt.2017.09.014

[14] Hassan, N.A., Airey, G.D., Jaya, R.P., Mashros, N., Aziz M.A.: A Review of Crumb Rubber Modification in Dry Mixed Rubberised Asphalt Mixtures, Jurnal Teknologi, 70 (2014;) 4, pp. 127-134, doi:10.11113/jt.v70.3501

[15] Moreno, F., Rubio, M.C., Martinez-Echevarria, M.J.: Analysis of Digestion Time and The Crumb Rubber Percentage in Dry-Process Crumb Rubber Modified Hot Bituminous Mixes, Construction and Building Materials, 25 (2011) 5, pp. 2323-2334, doi:10.1016/j.conbuildmat.2010.11.029

[16] Bingham, N., Saboundjian, S., Brunette, B.: Use of Rubber-Modified Hot-Mix Asphalt to Reduce Studded Tire Wear And Plastic Deformation, Alaska Departament of Transportation, 2010.

[17] Ng Puga, K.L.N.: Rheology And Performance Evaluation of Polyoctenamer as Asphalt Rubber Modifier in Hot Mix Asphalt, (2013), doi:10.31274/etd-180810-3582

[18] Feiteira Dias, J.L., Picado-Santos, L.G., Capitão, S.D.: Mechanical Performance of Dry Process Fine Crumb Rubber Asphalt Mixtures Placed on The Portuguese Road Network, Construction and Building Materials, 73 (2014), pp. 247-254, doi:10.1016/j.conbuildmat.2014.09.110

[19] Cao, W.: Study on Properties of Recycled Tire Rubber Modified Asphalt Mixtures Using Dry Process, Construction and Building Materials, 21 (2007) 5, pp. 1011-1015, doi:10.1016/j.conbuildmat.2006.02.004

[20] Moreno, F., Sol, M., Martín, J., Pérez, M., Rubio, M.C.: The Effect of Crumb Rubber Modifier on The Resistance of Asphalt Mixes to Plastic Deformation, Materials and Design, 47 (2013), 274-280, doi:10.1016/j.matdes.2012.12.022

[21] Rodríguez-Fernández, I., Cavalli, M.C., Poulikakos, L., Bueno, M.: Recyclability of Asphalt Mixtures with Crumb Rubber Incorporated by Dry Process: A Laboratory Investigation, Materials, 13 (2020) 12, doi:10.3390/ma13122870

[22] FGSV: Empfehlungen Zu Gummimodifizierten Bitumen Und Asphalten E GmBA, 2012, https://www.fgsv-verlag.de/e-gmba, 08.03.2021.

[23] Sousa, J.B., Vorobiev, A., Ishai, I., Svechinsky, G.: Elastomeric Asphalt Extender – A New Frontier on Asphalt Rubber Mixes, 2012, http://www.consulpav.com/shrp/?publications=elastomeric-asphalt-extender-a-new-frontier-on-asphalt-rubber-mixes, 08.03.2021.

[24] Kedarisetty, S., Biligiri, K.P., Sousa, J.B.: Advanced Rheological Characterization of Reacted and Activated Rubber (RAR) Modified Asphalt Binders, Construction and Building Materials, 122 (2016), pp. 12-22, doi:10.1016/j.conbuildmat.2016.06.043

[25] Yadollahi, G., Mollahosseini, H.S.: Improving the Performance of Crumb Rubber Bitumen by Means of Poly Phosphoric Acid (PPA) and Vestenamer Additives, Construction and Building Materials, 25 (2011) 7, pp. 3108-3116, doi:10.1016/j.conbuildmat.2010.12.038

[26] Navarro, F.M., Gámez, M.C.R.: Influence of Crumb Rubber on The Indirect Tensile Strength And Stiffness Modulus of Hot Bituminous Mixes, Journal of Materials in Civil Engineering, 24. 6, pp. 715-724, 2012.

[27] Subhy, A., Lo Presti, D., Airey, G.: An Investigation on Using Pre-Treated Tyre Rubber as a Replacement of Synthetic Polymers for Bitumen Modification, Road Materials and Pavement Design, 16 (2015), pp. 245-264, doi:10.1080/14680629.2015.1030826

[28] Kebria, D.Y., Moafimadani, S.R., Goli, Y.: Laboratory Investigation of The Effect of Crumb Rubber on The Characteristics and Rheological Behaviour of Asphalt Binder, Road Materials Pavement Design, 16 (2015) 5, pp. 946-956, doi:10.1080/14680629.2015.1042015

[29] Rahman, M.M., Airey, G.D., Collop, A.C.: Moisture Susceptibility of High and Low Compaction Dry Process Crumb Rubber-Modified Asphalt Mixtures, Transportation Research Record, (2010) 2180, pp. 121-129, doi:10.3141/2180-14

[30] Moreno-Navarro, F., Sol-Sánchez, M., Rubio-Gámez, M.C., Segarra-Martínez, M.: The Use of Additives for The Improvement of The Mechanical Behavior of High Modulus Asphalt Mixes, Construction and Building Materials, 70 (2014), pp. 65-70, doi:10.1016/j.conbuildmat.2014.07.115

[31] da Silva, L., Benta, A., Picado-Santos, L.: Asphalt Rubber Concrete Fabricated by the Dry Process: Laboratory Assessment of Resistance Against Reflection Cracking, Construction and Building Materials, 160 (2018), pp. 539-550, doi:10.1016/j.conbuildmat.2017.11.081

[32] Moreno-Navarro, F., Rubio-Gámez, M.C., Jiménez Del Barco-Carrión, A.: Tire Crumb Rubber Effect on Hot Bituminous Mixtures Fatigue-Cracking Behaviour, Journal of Civil Engineering and Management, 22 (2016) 1, pp. 65-72, doi:10.3846/13923730.2014.897982

[33] Rahman, M.: Characterisation of Dry Process Crumb Rubber Modified Asphalt Mixtures, 2004, https://www.nottingham.ac.uk/research/groups/ntec/documents/theses/rahman,mphdthesis.pdf, 08.03.2021.

[34] Mohammad, L.N., Cooper, S.B., Elseifi, M.A.: Characterization of HMA Mixtures Containing High Reclaimed Asphalt Pavement Content with Crumb Rubber Additives, Journal of Materials in Civil Engineering, 23. 11, pp. 1560-1568, 2011.

[35] Jones, D., Liang, Y., Harvey, J.: Performance Based Specifications: Literature Review on Increasing Crumb Rubber Usage by Adding Small Amounts of Crumb Rubber Modifier in Hot Mix Asphalt, 2017, https://escholarship.org/uc/item/9cw18024, 08.03.2021.

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ASPHALT CONCRETE MIXTURES WITH ADDITION OF RECLAIMED ASPHALT PAVEMENTS

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Abstract

The effect of using reclaimed asphalt pavements (RAP) to asphalt concrete mixtures besides their utilization is to reduce the amount of the new bituminous binder and aggregate added to hot mix asphalt. This publication presents studies on asphalt mixtures with an increased up to 40 % amount of RAP additive with the simultaneous use of 2 types of added bitumen, i.e. 35/50 and PMB 25/55-60. The aim of the paper is the evaluation of the basic mixture properties in a wide range of operating temperatures, as a part of the AC testing at high temperatures, the resistance to rutting at 60 °C and indirect tensile strength at 40 °C. The assessment of properties at intermediate operating temperatures is based on indirect tensile tests, including: elastic stiffness modulus at 5 °C, 15 °C and 30 °C and static strength at 25 °C. The low temperature properties have been tested in water and frost resistance tests by indirect tensile strength ratio. The results of the study were subjected to the analysis of the statistical significance of differences, which showed an improvement in the resistance of AC with the addition of RAP to the formation of permanent deformations and an increase in the stiffness modulus as well as indirect tensile strength. There was no adverse effect of the RAP additive on asphalt mixtures resistance to water and frost action.

Keywords: reclaimed asphalt pavements (RAP), asphalt concrete (AC), indirect tensile strength (ITS), stiffness modulus, wheel tracking

1 Introduction

RAP is the most popular recycled material used in the production of asphalt mixtures around the world, and at the same time, the most obvious. In the American report prepared by Copeland [1] it was stated that the most economical use of RAP is in asphalt mixtures. The technical regulations of specific countries in the field of using this material differ quite significantly and depend primarily on the experience with this technology. As reported Swamy et al. [2], properties of the asphalt mixtures with RAP percentage up to 15 % differ negligibly. In the work of Noferini et al. [3] RAP can be incorporated into the investigated mixture at percentages up to 10 % with no significant effects on properties of bitumen. As a practical method for testing of bitumen binder with RAP content 20 % or higher Noferini et al. [3] proposed application of the DSR test with complex modulus, phase angle isochrones, the black space and the Cole-Cole diagram. According to work by Sontag et al. [4], addition of RAP to a mixture increased the resilient modulus and it is also affected by the source of RAP. At the same time in the report prepared by Lee et al. [5] it was stated that RAP addition generally increases the stiffness, reduces the rut depth and wheel tracking rate and reduces the fatigue life.

There is an upward trend in the frequency of using RAP in HMA as well as the proportion of this material in the composition of the mixture. Al-Qadi et al. [6] concluded that it is possible to design high-quality asphalt mixtures with up to 50 % RAP. Proper processing and fractionation of the RAP material at asphalt plant as well as binder-grade bumping (using more softer bitumen binder grade) is also recommended [6]. As Sorociak's research has shown [7], it is possible to produce a mixture containing almost 100 % RAP (with the addition of bitumen rejuvenating agent) that meets all functional requirements for new mixtures. To assess the effectiveness of the rejuvenating agents Sorociak recommends analysing the relationship of AC stiffness as a function of phase angle. Hagos et al. [8] also confirmed that 100 % RAP mixtures with an addition of an innovative rejuvenator can be applied as a base and binder layer in pavements of all traffic classes including heavy duty.
The main problems of mixtures with RAP addition are: material quality and heterogeneity, bitumen ageing causing a drop in the mixture resistance to cracking, lack of additional installation for RAP dosing and lack of experience [1].
The purpose of this publication is the evaluation of the basic properties of the asphalt concrete mixture with an increased up to 40 % amount of RAP additive (with the simultaneous use of 2 types of added bitumen, i.e. 35/50 and PMB 25/55-60) in a wide range of operating temperatures.

2 Materials and methods

A mixture of asphalt concrete AC 16 for the bonding course with paving bitumen 35/50 was used for the test as reference mix. The results obtained on this mix were compared with the results of mixtures with the addition of 40 % RAP, differing in the type of bitumen used, i.e. 35/50 and PMB 25/55-60.
Testing mixtures were designed in such a way that the total content of bitumen binders and the grading curves of all mixes were constant. Details concerning the test mixtures composition and gradation is given in Table 1 and in Table 2, respectively.

Table 1 AC composition

O.N.	Components	Participation in AC [%] for the HMA			
		(35/50)	PMB	RAP (35/50)	RAP (PMB)
1	Limestone filler	4.8	4.8	1.9	1.9
2	Dolomite 0/4	28.7	28.7	19.4	19.4
3	Dolomite 2/8	33.4	33.4	21.3	21.3
4	Dolomite 8/11	14.3	14.3	9.7	9.7
5	Dolomite 8/16	14.3	14.3	5.8	5.8
6	RAP	-	-	38.8	38.8
7	Fresh bitumen	4.5	4.5	3.1	3.1

Table 2 AC gradation

Sieve size [mm]	Grading curve [%]		
	RAP	Reference mixtures	Mixtures with RAP addition
22.4	100	100	100
16.0	94.9	98.6	96.8
11.2	85.5	84.6	85.6
8.0	68.2	70.8	68.9
5.6	55.8	57.7	51.4
2.0	35.6	28.0	27.6
0.5	21.1	13.2	14.3
0.125	11.8	8.4	8.3
0.063	10.3	6.9	6.9
Bitumen amount	3.5	4.5	4.5

For each of the above 4 mixtures, the following samples were prepared:

- Loose mixture for maximum density test in pycnometer according to EN 12697-5;
- Cylindrical with a diameter of 101.6 mm and a height of approx. 63.5 mm, compacted with a Marshall hammer at 2 x 75 blows and 2 x 35 blows acc. to EN 12697-30;
- Cylindrical with a diameter and a height of 100 mm, compacted with gyratory press up to 200 rotation acc. to EN 12697-31;
- Plates with dimensions of 305x305x60 mm, compacted in a roller compactor according to EN 12697-33.

The program of the AC study is given in Table 3.

Table 3 Program of AC tests

O.N.	Tested property	Testing standard	Number of samples for the AC type			
			REF (35/50)	(PMB 25/55-60)	RAP (35/50)	RAP (PMB 25/55-60)
1	Maximum density	EN-12697-5	3	3	3	3
2	Bulk density	EN-12697-6	8	8	5	5
3	Air voids	EN-12697-8	8	8	5	5
4	Wheel tracking at +60°C	EN-12697-22	1	2	1	2
5	Stiffness modulus at 3 temp.: +5°C, +15°C and +30°C	EN-12697-26	5	5	5	5
6	ITS, samples 2 x 75 blows (and 200 gyration), tested at +40°C	EN-12697-23	4 (4)	4 (4)	5 (4)	5 (4)
7	ITS, samples 2 x 35 blows on wet and (dry) condition at +25°C	EN-12697-23	5 (5)	5 (5)	5 (5)	5 (5)

3 Results

The results of the individual tests are summarized in tables and figures, for tests where the number of samples in the series was at least 4, statistical analyses were performed to verify the hypothesis that the results for RAP mixtures differ significantly from the results of the reference mix. Statistical analyses were conducted with the use of computer program Statgraphics Plus v. 5.1. [9] according to the procedure given in [10]. Tests were performed to find out if there was any statistically significant difference between the averages of the variable at a given confidence level equal 0.95. ANOVA Table was used for this purpose. To determine which interlayer systems differ significantly from one another, the analysis of multiply range tests with application of LSD (least square differences) option was used.

3.1 Physical parameters

Before executing performance tests, the maximum density and bulk density of HMA samples were tested, then content of the air voids in compacted samples were calculated, examples of obtained results are included in Table 4.

Table 4 Results of air voids content in Marshall samples [%]

Mixture type	count	average	std. dev.	coef of var.	minimum	maximum	range
Samples compacted 2 x 35 blows							
REF (35/50)	10	7.4	0.26	3.5	7.0	7.8	0.8
RAP (35/50)	10	8.7	0.27	3.1	8.3	9.1	0.8
(PMB 25/55-60)	10	7.7	0.30	3.9	7.3	8.2	0.9
RAP (PMB 25/55-60)	9	7.8	0.40	5.2	7.1	8.5	1.4
Samples compacted 2 x 75 blows							
REF (35/50)	8	6.4	0.76	11.8	5.2	7.2	2.0
RAP (35/50)	8	6.4	0.40	6.2	5.9	6.8	0.9
(PMB 25/55-60)	5	6.2	0.38	6.1	5.8	6.8	1.0
RAP (PMB 25/55-60)	5	5.6	0.30	5.4	5.1	5.8	0.7

3.2 Wheel tracking

For wheel tracking tests each mixture was tested at 60°C using the method of small apparatus acc. to PN-EN 12697-22. The test results covering proportional rut depth (PRD) and wheel tracking speed (WTS) are summarized in Table 5.

Table 5 Wheel tracking results for AC

Mixture type	Rut depth		WTS_{AIR}
	FRD [mm]	PRD_{AIR} [%]	[mm/1000 cycles]
REF (35/50)	3.50	5.8	0.078
RAP (35/50)	2.10	3.5	0.052
(PMB 25/55-60)	1.95	3.3	0.040
RAP (PMB 25/55-60)	1.80	3.0	0.038

3.3 Stiffness modulus

The tests were performed on cylindrical Marshall samples, compacted 2 × 75 blows with indirect tensile method, at 3 temperatures: + 5°C, + 15°C and + 30°C. On each sample, tests were performed on 2 mutually perpendicular diameters using 5 pulses of load, results are given in Table 6, while their statistical tests in Table 7.

Table 6 Results of AC stiffness modulus (E) [MPa]

Mixture type	**count**	**average**	**std. dev.**	**coef of var.**	**minimum**	**maximum**	**range**
Temperature of test +5°C – E(5)							
REF (35/50)	10	11959	1107	9.3	10502	13400	2898
RAP (35/50)	10	13660	793	5.8	12453	14985	2532
(PMB 25/55-60)	6	13422	569	4.2	12735	14012	1277
RAP (PMB 25/55-60)	6	16355	906	5.5	15246	19966	2720
Temperature of test +15°C – E(15)							
REF (35/50)	10	7065	427	6.0	6229	7568	1339
RAP (35/50)	10	9093	741	8.1	7816	10101	2285
(PMB 25/55-60)	10	7944	316	4.0	7499	8382	883
RAP (PMB 25/55-60)	8	10348	589	5.7	9522	11131	1609
Temperature of test +30°C – E(30)							
REF (35/50)	10	2415	163	6.8	2167	2688	521
RAP (35/50)	10	3523	166	4.7	3192	3830	638
(PMB 25/55-60)	10	2542	97	3.8	2432	2711	279
RAP (PMB 25/55-60)	10	3955	295	7.5	3607	4558	951

Table 7 Results of significance tests for AC stiffness modulus

Contrast	**+5°C**		**+15°C**		**+30°C**	
	difference	**+/- limits**	**difference**	**+/- limits**	**difference**	**+/- limits**
REF (35/50) - RAP (35/50)	1701*	820	2029*	491	1108*	176
REF (35/50) - (PMB 25/55-60)	1463*	946	879*	491	126	176
REF (35/50) - RAP (PMB 25/55-60)	4396*	946	3284*	520	1539*	176
RAP (35/50) - (PMB 25/55-60)	238	946	1149*	491	982*	176
RAP (35/50) - RAP (PMB 25/55-60)	2694*	946	1254*	520	431*	176
(PMB 25/55-60) - RAP (PMB 25/55-60)	2933*	1058	2404*	520	1413*	176
* denotes a statistically significant difference						

3.4 Indirect tensile strength

The tests were performed on cylindrical Marshall samples, compacted 2×35 blows and 2×75 blows respectively. The basic temperature of the study was 25 °C, whereas samples compacted 2×75 blows were also tested at the temperature of 40 °C. In the case of 2×35 blow samples, 2 types of conditioning were used, i.e. dry and wet conditions with one freezing cycle, according to Polish Technical Requirement WT-2 [11] and next indirect tensile strength ratio (ITSR) was calculated according to equation (1).

$$ITSR = \frac{ITS_{wet}}{ITS_{dry}} \cdot 100\% \quad (1)$$

where: $ITS_{(wet/dry)}$ – indirect tensile strength for (wet/dry) series of samples

Results of ITS tests at the temperature 25 °C as well as ITSR results are given in Table 8, while results of ITS tests at the temperature 40 °C are presented in Table 9. Statistical tests in for ITS results are given in Table 10.

Table 8 Results of indirect tensile strength (ITS) [kPa] at the temperature of 25 °C

Samples compacted 2 x 35 blows, wet conditions								**ITSR [%]**
Mixture type	**count**	**average**	**std. dev.**	**coef of var.**	**minimum**	**maximum**	**range**	
REF (35/50)	5	960	55	5.7	864	996	132	90.0
RAP (35/50)	5	985	54	5.4	907	1054	147	92.2
(PMB 25/55-60)	5	1046	84	8.1	961	1171	210	91.0
RAP (PMB 25/55-60)	4	1041	58	5.6	961	1089	128	89.1
Samples compacted 2 x 35 blows, dry conditions								
REF (35/50)	5	1067	108	10.1	980	1247	267	
RAP (35/50)	5	1068	92	8.6	962	1168	206	
(PMB 25/55-60)	5	1149	70	6.1	1074	1239	165	
RAP (PMB 25/55-60)	4	1168	60	5.2	1081	1217	136	

Table 9 Results of indirect tensile strength (ITS) [kPa]

Samples compacted 2 x 75 blows, dry conditions, +40 °C							
Mixture type	**count**	**average**	**std. dev.**	**coef of var.**	**minimum**	**maximum**	**range**
REF (35/50)	4	481	37	7.6	443	523	80
RAP (35/50)	4	618	49	8.0	564	670	106
(PMB 25/55-60)	4	583	29	5.0	544	614	70
RAP (PMB 25/55-60)	4	747	65	8.8	690	827	137
Samples compacted up to 200 gyrations, dry conditions, +40 °C							
REF (35/50)	4	564	25	4.4	541	594	53
RAP (35/50)	4	761	30	3.9	717	779	62
(PMB 25/55-60)	4	820	38	4.6	785	873	88
RAP (PMB 25/55-60)	4	850	43	5.0	805	907	102

Table 10 Results of significance tests for ITS

Contrast	2 x 35, wet, +25 °C		2 x 35, dry, +25 °C		2 x 75, dry, +40 °C		200 gyr., dry, +40 °C	
	difference	+/- limits	difference	+/- limits	difference	+/- limits	difference	+/- limits
REF (35/50) – RAP (35/50)	26	87	1	116	137*	60	197*	60
REF (35/50) – (PMB 25/55-60)	86	87	82	116	101*	60	255*	60
REF (35/50) – RAP (PMB 25/55-60)	81	92	101	123	265*	60	285*	60
RAP (35/50) – (PMB 25/55-60)	60	87	81	116	35	60	58	60
RAP (35/50) – RAP (PMB 25/55-60)	56	92	100	123	129*	60	88*	60
(PMB 25/55-60) – RAP (PMB 25/55-60)	5	92	19	123	164*	60	30	60

* denotes a statistically significant difference

4 Discussion

Summarizing the results of the physical characteristics tests, it was found that the designed mixtures, for each series of samples compacted with the same energy, had similar air voids. The above condition allows to compare the strength properties of mixtures with the negligible influence of its physical characteristics.
In the case of the elastic stiffness modulus, the statistical tests showed that the mixtures with RAP addition were significantly stiffer than the reference mixture, regardless of the type of the added binder (35/50 or PMB 25/55-60) and the temperature of the test. Moreover, the RAP mixtures have proven to be less thermal sensitive than the reference mixtures, which is a beneficial phenomenon. The stiffness modulus ratio, calculated as a stiffness modulus at the temperature of 5°C divided by the stiffness modulus at the temperature of 30°C, for samples with the addition of RAP is 3.9 and 4.1, respectively (for bitumen 35/50 and PMB), while for the reference mixture it is 5.0 and 5.3, respectively. The above observation can be explained by the lower thermal sensitivity of the binder contained in the used RAP.
Wheel tracking test shows better performance for RAP mixtures, especially in the case of paving bitumen application, where rutting parameters were reduced by about 33 % (PRD) to 40 % (WTS), while in the case of PMB mixtures effect of RAP addition caused the reduction of rutting parameters by less than 10 %. Studies have also confirmed the beneficial effect of the use of polymer modified bitumen, the rutting parameters have been reduced by almost half, which is in line with most studies.
All of tested mixtures obtained a similar assessment in terms of resistance to water and frost action, calculated ITSR values are within the limits of 89.1 - 92.2 %, i.e. significantly above the value required by WT-2 [11] (minimum 80 %).
ITS results at the temperature of 25°C for RAP mixtures are not significantly different from reference mixtures, regardless of sample seasoning conditions (wet or dry).
In accordance with works [12, 13] the ITS test at elevated temperature can be used to assess the resistance of asphalt mixtures to permanent deformations. This was confirmed by the results of ITS tests at a temperature of 40°C, both on samples compacted in the gyratory press and in the Marshall hammer. PMB mixtures give significantly higher tensile strength than paving bitumen mixtures and at the same time, RAP mixtures obtained higher ITS results

than reference ones. In addition, the strength of gyratory samples was found to be higher than Marshall samples, which can be explained by the lower air voids content in gyratory samples (higher compaction energy).

5 Conclusions

The presented test results indicate the potential possibility of increased up to 40 % addition reclaimed asphalt pavement to asphalt concrete mixture. The asphalt concretes for binding course (AC 16 W 35/50 and AC 16 W PMB 25/55-60) with 40 % of RAP addition meet all the requirements given in Polish Technical Requirements WT-2 [11], e.g.: content of the air voids, the proportional rut depth, the wheel tracking speed, and the resistance to water and frost action as the indirect tensile strength ratio. The RAP mixtures, despite the same content of the air voids in samples, obtained much higher stiffness modules (regardless of the test temperature) and, moreover, they are characterized by lower thermal sensitivity than reference mixtures. In terms of the resistance of the mixture to the formation of permanent deformation, the addition of RAP proved to be an effective solution, especially in the case of the paving bitumen mixtures. The above conclusion from the rut research was also confirmed by the results of indirect tensile strength (ITS) tests at a temperature of 40°C in contrast to the results of the ITS study at 25°C (no differences between the tested mixtures).
Finally, the results show, that mixture with an increased up to 40 % amount of RAP improves the AC properties in the elevated and the intermediate temperatures, not affecting the water and frost action. Fatigue tests and low-temperature fractures are necessary for a full assessment of these mixtures.

References

[1] Copeland, A.: Reclaimed Asphalt Pavement in Asphalt Mixtures: State of the Practise, Report No. FHWA-HRT-11-021, Federal highway Administration, USA, 2011.

[2] Swamy, A.K., Mitchell, L.F., Hall, S.J., Daniel, J.S.: Impact of RAP on volumetric, stiffness, strength, and low-temperature properties of HMA, Journal of Materials in Civil Engineering, 23 (2011) 11, pp. 1490 – 1497, doi: https://doi.org/10.1061/(ASCE)MT.1943-5533.0000245

[3] Noferini, L., Simone, A., Sangiori, C., Mazzotta, F.: Investigation on performance of asphalt mixtures made with Reclaimed Asphalt Pavement: Effect of interaction between virgin and RAP bitumen, International Journal of Pavement Research and Technology, 10 (2017) 4, pp. 322-332, doi: https://doi.org/10.1016/j.ijprt.2017.03.011

[4] Sontag, M.S., Chadbourn, B.A., Drescher, A.: Investigation of recycled asphalt pavement (RAP) mixtures, Report No. MN/RC – 2002-15, University of Minnesota, USA, 2002.

[5] Lee, J., Dennerman, E., Choi, Y.: Maximising the re-use of reclaimed asphalt pavement – Outcomes of year two: RAP mix design, Austroads Publication No. AP-T286-15, Austroads, 2015.

[6] Al-Qadi, I.L., Aurangzeb, Q., Carpenter, S.H., Pine, W.J., Trepanier, J.: Impact of high RAP contents on structural and performance properties of asphalt mixtures, Report No. FHWA-ICT-12-002, University of Illinois at Urbana-Champaign, USA, 2012.

[7] Sorociak, W.: Badanie wpływu dodatków regenerujących na właściwości mieszanek mineralno-asfaltowych podlegających procesom starzenia, rozprawa doktorska, Politechnika Śląska, Gliwice, 2018.

[8] Hagos, E., Shirazi, M., van de Wall, A.: The development of 100 % RAP mixtures with the use of innovative rejuvenator, 6th Eurasphalt & Euobitume Congress, Prague, Czech Republic, 1-3.6.2016, doi: dx.doi.org/10.14311/EE.2016.093

[9] Statgraphics Plus v. 5.1, A Manugistics Inc. Product, Rockville, MD USA, 2000, No. 537672031051732

[10] Dobosz, M.: Computer-aided statistical analysis of research results, Problems of contemporary science, Theory and applications, STATISTICA, AOW EXIT, Warsaw, Poland, 2001

[11] WT-2, Hot Mix Asphalt, Technical Requirements for National Roads, General Directorate For National Roads and Motorways, Warsaw, Poland, 2014.

[12] Zaniewski, J.P, Srinivasan, G.: Evaluation of indirect tensile strength to identity asphalt concrete rutting potential, Department of Civil and Environmental Engineering, West Virginia University, Morgantown, USA, 2003.

[13] Christensen, D.W., Bonaquist, R.: Using the Indirect Tension Test to Evaluate Rut Resistance in Developing Hot-Mix Asphalt Designs, Transportation Research Circular, Washington, 2007, pp. 62–77

BEHAVIOUR OF HIGH-MODULUS ASPHALT MIXTURES FROM THE PERSPECTIVE OF STRAIN CHARACTERISTICS (STIFFNESS)

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Abstract

High modulus asphalt concrete (HMAC) presents a concept of an asphalt mixture with advanced performance which is suitable mainly for heavy loaded pavement structures. The mix concept was developed more than 25 years ago in France and became a standard in many countries. In the Czech Republic this type of asphalt mixtures is used since the early years of this millennium, when original technical requirements have been set. After almost 20 years a volunteer technical assessment started to validate whether the technical requirement set mainly for stiffness values and partly also for flexural strength or resistance to crack propagation are still up-to-date or if some reasonable modification is needed like was done several years ago in France when high modulus asphalt concrete of so called EME II or GP5 generation were brought to the practice. Based on this a study with focus on stiffness determination for more than 40 different HMACs was started. The stiffness was tested at different temperatures. At the same time virgin and aged asphalt mixtures were compared. Results from this study are presented by the paper.

Keywords: asphalt mixtures, high modulus asphalt concrete, HMAC, EME, stiffness, SCB test

1 Introduction

High modulus asphalt concretes (HMAC) were firstly design and used in France nearly 40 years ago [1]. HMAC or interchangeable term of EME (Enrobé a Module Élevé) and are a special type of asphalt concrete with strong aggregate structure, slightly higher amount of binder and elevated stiffness usually balanced with good fatigue life. This type of mixtures is used in both heavy duty and structural rehabilitation projects where it is desirable to minimize the impact of grade change yet still ensure pavement longevity. Use of HMAC in base or potentially binder pavement layer can potentially lead to a reduction of thickness of asphalt layers in pavement structure in comparison to pavement structure with conventional asphalt concretes while the service life of such a construction remains unchanged. Effort to reduce the thickness of asphalt layer is related to reduction of construction costs and also later life cycle costs related to maintenance LCC optimization [2, 4]. In addition, the material resources can be saved. Some of the published papers or research outputs e.g. [5, 6] present a reduction in thickness between 25 and 30 percent in the pavement structure. In the case of long-life pavements, the overall costs have to be assessed not only from the perspective of construction costs but mainly from the view point of life cycle costs. Primary cost can be higher but the pavement shows less demand for repairs and rehabilitation actions and therefore the life cycle costs are significantly lower than for other types of asphalt mixtures. For these reasons it is necessary to focus on the life cycle cost assessment during selection of the right pavement design and not only on the lowest construction price as currently often

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happens. Espersoon [1] showed the results of the experimental research that has been done to calculate the reduction in thickness of the base layer with HMAC compare to a base layer with conventional paving grade bitumen for runway pavements at the different temperatures. Rys et al. [3] presented analysis of 80 selected road sections in Poland of total length of about 1300 km and compared low temperature cracking properties of pavements with HMAC mix type and conventional asphalt concrete base. It was revealed that pavements with high modulus asphalt bases have several times higher odds of cracked than pavements with conventional asphalt concrete base.

2 Assessed variants of asphalt mixture

For the assessment of further specified characteristics of high modulus asphalt concretes (denoted VMT 22 or HMAC) in total 47 variants were included. These mixtures were produced and tested in 2019-2020. The HMACs were divided in two groups depending on the used bituminous binder – either paving grade/hard paving grade or polymer modified bitumen (PMB). For the PMB set 5 variants containing PMB 10/40-65, 24 variants containing PMB 25/55-60 or -65 and 4 variants with commercial Polybitume EP were included. In total this group involved 33 variants of HMAC mixture. The “non-modified” set of asphalt mixtures contained 11 variants with 20/30 bitumen, on variant with hard binder 15/25 and two variants where 30/45 bitumen was used. In total there were for this set 14 variants.
Asphalt mixtures of HMAC type have generally similar grading (representation of particle size distribution) like asphalt concrete used for binder courses (AC_{bin}). The difference is in a closer grading envelope. In comparison with AC_{bin} 22 the HMAC 22 mixture do have a requirement for higher bituminous binder content. The national standard CSN 73 6121 (for the Czech Republic) specifies for AC_{bin} 22 of superior class a minimum bitumen content 4,0 % by mass. For HMAC 22 the required binder content interval is 4,1 to 5,4 % by mass depending on the coefficient of richness and the volumetric content of bitumen in the mixture (min. 10,5 % by vol.). These requirements have so far been defined by the technical specifications TP 151. According to the new standard CSN 73 6120 (in final review process) the bitumen content is even set between 4,4 and 5,6 % by mass. The requirement for coefficient of richness which is typical for French asphalt mix design as well as the minimum volumetric binder content will not be requested in the future.

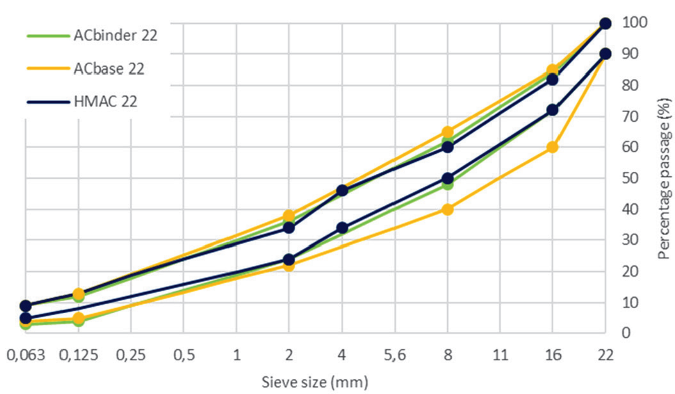

Figure 1 Comparison of grading envelopes for HMAC and asphalt concrete according to Czech specifications

For asphalt mixtures VMT 22 (HMAC 22) the interval of required voids content ins closer as well. The TP 151 specifications prescribe a voids content for mix designing of 3,0 to 5,0 % and for control testing 2,5 to 6,0 %. For common asphalt concrete AC_{bin} 22 for superior applica-

tions the voids content requirements are 4,0 to 6,0 % for type testing and 3,0 to 8,0 % for control testing.
One of the most fundamental characteristics for HMAC mix type is without any doubts its stiffness. In the case of the Czech Republic the stiffness modulus is determined at the test temperature of 15 °C (S15). This temperature is the most decisive according to the pavement design manual which is defined by specifications TP 170. In the so far still valid technical specifications TP 151 for HMAC mixtures the minimum required value of stiffness is $S_{15,min}$=9000 MPa. There is only on stiffness category. This limit is valid for determination of stiffness modulus either on trapezoidal test specimens according to EN 12697-26, annex A, or on cylindrical (Marshall) test specimens according to EN 12697-26, annex C. In the new standard CSN 73 6120 the minimum requirement has been already set depending on the used test method for its determination. For 2PB test using trapezoidal test specimens the minimum required stiffness value stays 9000 MPa, for IT-CY test method (repeated indirect tensile stress on cylindrical specimens) the minimum requirement has been increased to 9500 MPa. The reason for such differentiation is based on experience gained during the last 20 years. It has been repeatedly identified that if the identical HMAC mixture is tested using trapezoidal test specimens and in parallel cylindrical test specimens, in most case the results form 2PB test are lower. The common practice was then, that many asphalt mix producers to fulfil the criterion of stiffness after failing with 2PB test, ordered the IT-CY test to meet the minimum requirement. Such approach in general is technically not correct and therefore for both test methods different minimum required value has been proposed. The difference of 500 MPa between both test methods is still rather affable and less conservative. In general, this follows the ongoing issue existing in Europe where EN 12697-26 defines several test methods for stiffness but there isn´t any relevant functionality between the determined values of the particular methods.
The asphalt mixtures presented and compared in this paper were assessed from the viewpoint of their bulk densities, voids contents, stiffness values determined at temperatures of 0, 15, 27 and 40 °C on Marshall test specimens (IT-CY test method according to EN 12697-26, annex C) and the resistance to thermal induced cracking determined by modified test method based on CSN EN 12697-44 (semicircular bending test). The temperatures for stiffness testing are based on the established practice in the Czech Republic which is used for more than 30 years. The selected temperatures represent typical average conditions on a pavement during a year.

3 Results for tested HMAC mixtures

3.1 Voids content

Results shown in this paper were grouped based on larger number of various commercial construction projects (sites). Not for each of the 47 available variants all characteristics were tested or determined.
The bulk density of test specimens has been determined for all available HMAC variants, however, the maximum density used for calculation of voids content was tested only for approx. 70 % of all mixtures. From the voids contents which were determined result that roughly in half of the cases does not fulfil the limits set for mix design (type testing). If more benevolent limit for control testing is used, that still about 20 % of tested variants shows voids content beyond this limit.
The problem with voids content has been detected mainly in the set of HMAC mixtures with polymer modified binders. This finding is crucial with respect to the workability of the asphalt mixtures, but might have influence on other properties as well. Voids content and in case of paved asphalt layer its compaction rate does significantly influence behavior and

performance of the asphalt layer as such. It is possible, that higher content of usually harder bituminous binder type and increased content of fines in the mixture can lead to partially worsened workability. On the other hand, voids content is the fundamental characteristic which can be influenced during the mix design. If an asphalt mix with inconvenient voids content is identified, it is necessary either modify the grading of the mixture or increase content of the used bituminous binder. In the case of utilizing hard bituminous binders it is necessary to mix and compact the mixture at sufficiently high temperatures. The specifications TP 151 define working (processing) temperatures in the interval between 170 °C and 195 °C depending on the type of used binder. Newly drafted national standard ČSN 73 6120 adjusts this interval for a range between 160 °C to 190 °C for paving grades and hard paving grades and between 155 °C to 180 °C for polymer modified binders. Reduction of the processing temperature is without any doubt beneficial with regard to environmental protection or cost efficiency of asphalt mix production. All the more it is important to care about an accurate asphalt mix design including requirements for voids content and further corresponding properties of HMAC. Last but not least the workability can be improved by using the warm mix asphalt concept as well.

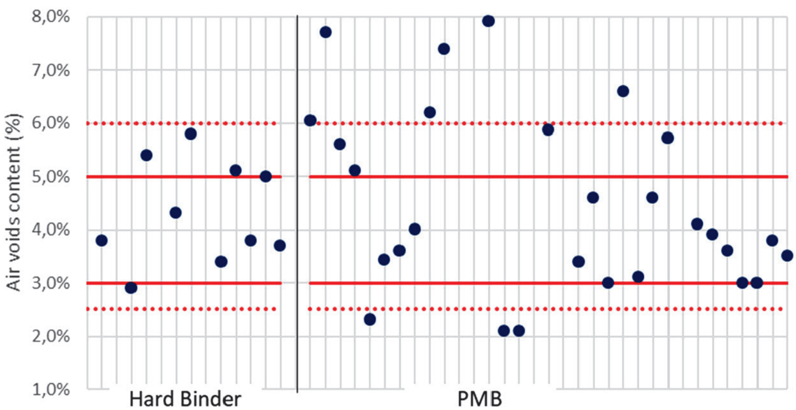

Figure 2 Voids content for assessed HMAC mixtures

Stiffness was determined by IT-CY test method at 4 temperatures: 0 °C, 15 °C, 27 °C and 40 °C. As has been mentioned earlier, the decisive temperature is 15 °C, for which the minimum required value of 9000 MPa is set (according to TP 151) and will be in the future 9500 MPa (according to the draft CSN 73 6120). The minimum value was in case on non-modified mixtures not fulfilled by one mixture containing 30/45 paving grade bitumen. For the set of modified asphalt mixtures the overall results are worse. From 32 tested variants 14 mixtures did not comply with was the existing minimum value, which is nearly half of all assessed variants. If the stricter limit would be considered, additional 2 variants would not fulfill the requirement since they showed a stiffness between 9000 MPa and 9500 MPa. This finding means that from all received and commercially used or designed HMAC mixtures the half in fact are not high modulus asphalt concretes but regular asphalt concretes for base or binder course containing just elevated content of bitumen.

Figure 3 is split in two parts. First part shows all stiffness results for non-modified HMAC variants whereas the second part contains only modified variants. The asphalt mixtures are in each group ordered according to the reached stiffness S_{15} and this order is kept for all presented graphs. First five "PMB variants contain harder PMB 10/45-65 binder. All remaining modified variants are just ranked according to S_{15} values without any further division according to the used PMB. In the case of HMAC mixtures with paving grades or hard paving grade binders the correlation for 15 °C and for other temperatures works quite well (R^2 = 0,82 to 0,94). For these mixtures it is therefore with some accordant caution be stated and forecast what will be the stiffness for other temperatures as well. Such statement is nevertheless not valid unconditionally and it is necessary to accentuate that for the evaluation only 14 variants have been included. This is rather a smaller number of determinations and the correlation results need to be considered with providence.

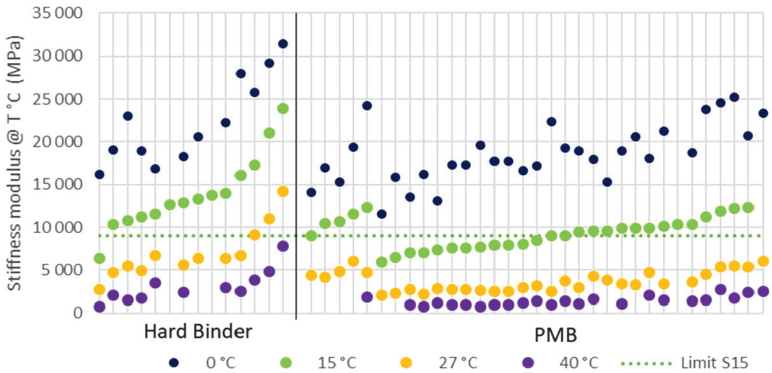

Figure 3 Stiffness of assessed HMAC mixtures

For the HMAC mix variants with polymer modified binders the variance of analyzed characteristics is higher. At elevated test temperatures there is obvious similar trend like in the case of paving grades and hard binders – the tilt of the tangent for the regression curve is very similar, but the coefficient of determination is lower (R^2 0,79, resp. 0,66).

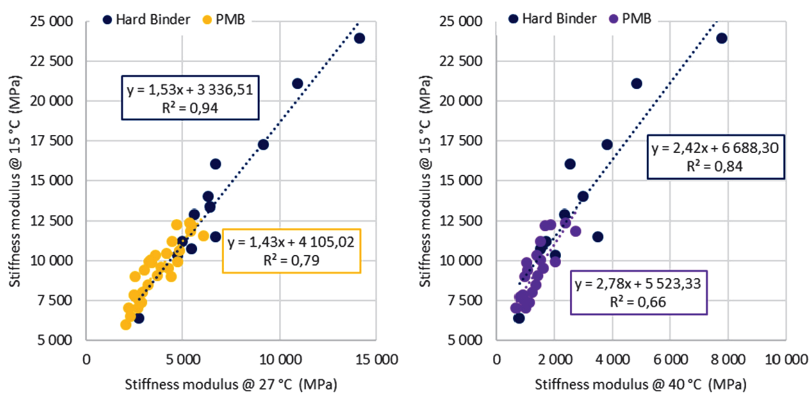

Figure 4 Comparison of stiffness S_{15} and S_{27}, as well as S15 and S_{40} for assessed HMAC mixtures

At the test temperature of 0 °C there is fully apparent a different impact of particular modifications. The coefficient of determination in case of the PMB mixture group reaches only a value of 0,54, which indicates some dependence between the parameter, but it is only medium strong. The tilt of the regression curve tangent is completely different from the tilt for non-modified mixtures showing a gentler progress. This means that there is a slower accrual of stiffness with decreasing test temperature. If the set of HMAC mixtures containing PMBs is further divided in sub-groups following the particular binder types (PMB 10/40-65 and PMB 25/55-60 or -65 including Polybitume EP) a higher variability for “softer” PMBs can be induced from the results. This can be entrained by the fact that binders commercially offered by different producers have the same PMB category, but the source including production processing, original bitumen master-batch and used type of modifier is different which results in dissimilar properties even if fulfilling the standard requirements for the bitumen as prescribed by EN 14023 and national requirements. That might be also one of many explanations why it is not as simple to interchange same type of a PMB coming from different producers. This works quite well for a paving grade but shall be followed with caution in case of modified bituminous binders.

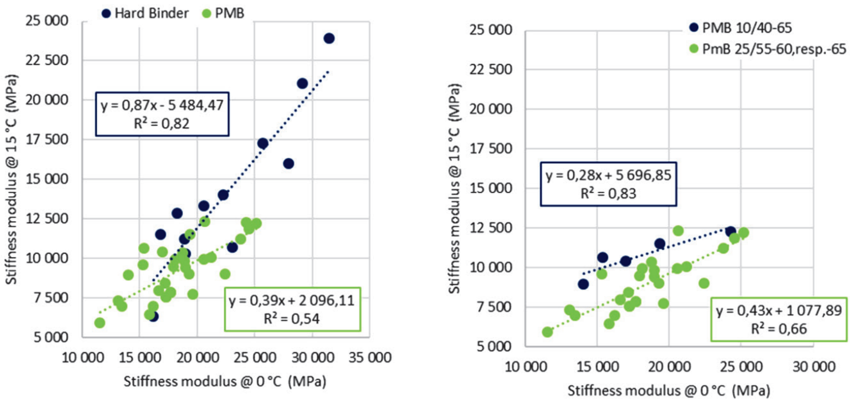

Figure 5 Comparison of stiffness modules S_{15} and S_0 of HMAC mixtures

Variability of the results for HMAC set of mixtures containing PMB is apparent also for the comparison of stiffness S15 and bulk density. In this respect we are aware of the fact that the comparison to bulk density and not voids content which was not determined for all mix variants presented by this study might be partially misleading and not fully predicative.

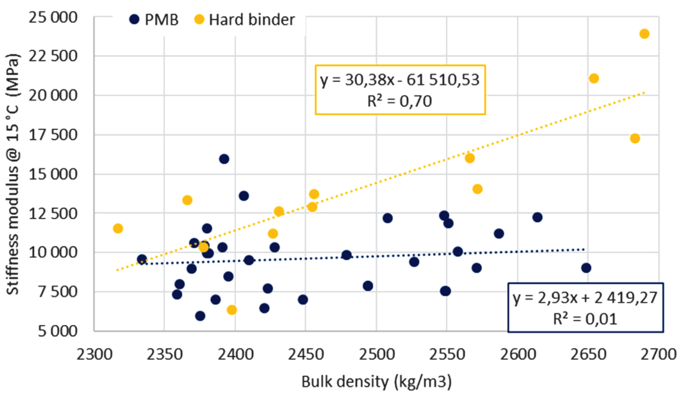

Figure 6 Comparison of stiffness S_{15} and bulky density of HMAC mixtures

Table 1 Statistical quantities for stiffness modules of assessed HMAC mixtures

Statistical quantity	0 °C			15 °C			27 °C			40 °C	
	HB	PMB 10	PMB 25	HB	PMB 10	PMB 25	HB	PMB 10	PMB 25	HB	PMB 25
Mean	22445	17994	18607	13932	10770	9410	6992	4841	3456	3076	1377
Stand. error	1465	1798	655	1203	557	420	888	332	215	584	117
St. deviation	5074	4020	3403	4502	1245	2259	3077	743	1119	1938	562
Minimum	16213	14036	11533	6345	8976	5951	2739	4165	2030	778	693
Maximum	31426	24254	25164	23930	12271	15932	14159	6085	5981	7790	2732
Range	15213	10218	13630	17585	3296	9981	11420	1920	3952	7012	2039
Count	12	5	27	14	5	29	12	5	27	11	23

HB = Hard bituminous binders; PMB 10 = PMB 10/40-65; PMB 25=PMB 25/55-60 or PMB 25/55-65 or Polybitume EP

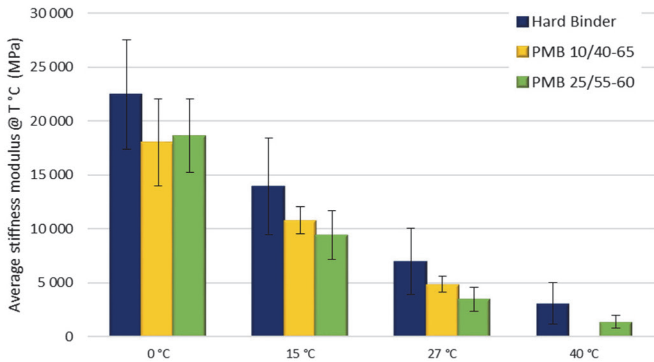

Figure 7 Mean value of stiffness modulus of HMAC mixtures

The highest mean stiffness moduli were determined for asphalt mixtures with hard binders. This is expected phenomena due to lowered penetration and increased stiffness of the binder. For modified mixtures, the stiffness modulus determined at temperate of 15 °C and 27 °C is higher for mix variants with PMB 10/40-65, which again confirms the influence of penetration of binder to mixture‘s stiffness. At the temperature of 0 °C the asphalt mixtures with PMB 25/55-60, resp. 65 + Polybitume EP reached higher stiffness.

Behaviour in the range of low temperature is crucial for HMAC mixture. The mixtures are very stiff and usually more susceptible to low temperature cracking. Technical conditions of Czech ministry of transportation TP 151 defines the minimum flexural strength determined by three-point bending test 6 MPa. This requirement was left out from the new standard CSN 73 6120 which is actually in final approval process. For this research study the SCB test according to modified method was used, instead of three-point bending test. The modified method is based on standard EN 12697-44. The modified method is elaborately described e.g. in [7]. The important modifications in the methodology are e.g. smaller diameter of test specimens (ø100 mm), different compaction of test specimens (according to EN 12697-30), lower loading rate (2.5 mm/min), new test parameters (e.g. fracture energy) etc.

The SCB test performed at temperature of 0 °C, 15 °C and 25 °C. The higher test temperatures relate to ‘fatigue’ cracking.

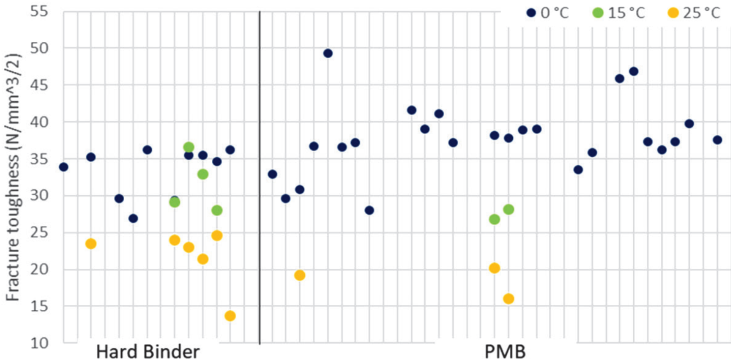

Figure 8 Fracture toughness of HMAC mixtures

In figure 8, there are results for both groups of HMAC mixtures. The individual variants are still in the same sequence according to the stiffness modulus determined at 15 °C. From the results, it might be deducable, that there is a certain trend between stiffness modulus at 15 °C and fracture toughness, but it is actually very low. From the coeffients of determination (Figure 9) it can be seen, that these two parameters do not relate to each other. For fracture parameters the opposite trend is apparent in comparison to stiffness modulus – with higher penetration and modification of binder, the fracture parameters increase, Table 2.

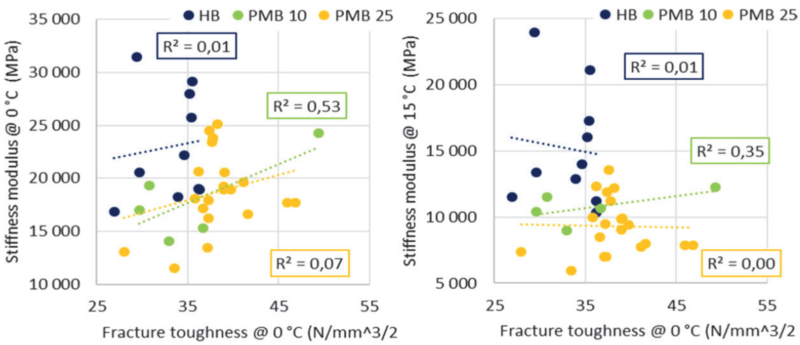

Figure 9 Comparison of fracture toughness determined at 0 °C and stiffness modulus determined at 0 °C a 15 °C

Table 2 Statistical quantities for SCB test parameters of assessed HMAC mixtures determined at 0 °C

Statistical quantity	Fracture toughness ($N/mm^{3/2}$)			Fracture energy till Fmax (J/m^2)			Total fracture energy (J/m^2)		
	HB	PMB 10	PMB 25	HB	PMB 10	PMB 25	HB	PMB 10	PMB 25
Mean	33	36	38	835	1052	1111	1193	1295	1416
Stand. error	1	4	1	87	176	72	148	209	82
St. deviation	3	8	4	276	392	313	467	467	358
Minimum	27	30	28	492	704	733	583	1004	998
Maximum	36	49	47	1257	1668	2169	2171	2114	2389
Range	9	20	19	765	964	1436	1588	1110	1392
Count	10	5	20	10	5	19	10	5	19

HB = Hard bituminous binders; PMB 10 = PMB 10/40-65; PMB 25=PMB 25/55-60 or PMB 25/55-65 or Polybitume EP

4 Conclusions

The presented study offered a comprehensive preview of the commonly gainable characteristics of HMAC mixtures as used in the Czech Republic. It was possible to collect more than 40 variants representing different types of aggregates and different binders as they are regularly used for pavement structures. It provided also a better understanding about the stiffness values ant potential weaknesses. It is true – and was supported by the results – that harder binder, especially if applied as paving grades provide usually high values of stiffness. This must necessarily not correspond with appropriate resistance to cracking and fracture behavior, which is critical especially in the case that HMAC is used in a binder course.
It has been shown that usually PMBs do result in slightly lower stiffness and in general there are more variants which do not meet the minimum required stiffness, mainly if PMB 25/55-60 or -65 is used. It might be well explainable by a higher elasticity of such bitumen. On the other hand it was demonstrated that HMAC mixtures with modified binders result in better fracture characteristics and higher resistance to cracking. If such finding would be compared with the fact that binders do continuously age, then the option with slightly softer PMB might result in an overall better long-term performance.
It has been also analyzed if there is some stronger relation between stiffness and either bulk density or the characteristics used for assessment of behavior in low-temperature range and resistance to cracking. In case of bulk density there was a moderate dependency between this characteristic and stiffness for hard paving grades. For fracture characteristics or flexural strength there is more or less no clear relation, even if the characteristics have been tested at same test temperature.

Remark

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References

[1] Espersson, M.: Effect in the high modulus asphalt concrete with the temperature, Construction and Building Materials, 71 (2014), pp. 638-643

[2] Miró, R., Valdés, G., Martinez A., Segura P., Rodriguez, C.: Evaluation of high modulus mixture behaviour with high reclaimed asphalt pavement (RAP) percentages for sustainable road construction, Construction and Building Materials, 25 (2011) 10, pp. 3854-3862, doi: 10.1016/j.conbuildmat.2011.04.006

[3] Rys, D., Judycki, J., Pszcola, M., Jaczewski, M., Mejlun, L.: Comparison of low-temperature cracks intensity on pavements with high modulus asphalt concrete and conventional asphalt concrete bases, Construction and Building Materials, 147 (2017), pp. 478-487

[4] Corté, J.-F.: Development and uses of hard-grade asphalt and of high-modulus asphalt mixes in France, Perpetual Bituminous Pavements, Transportation Research Circular, 2001, pp. 12-31

[5] Newcomb, D.: Perpetual Pavements - A Synthesis. Asphalt Pavement Alliance, 2002.

[6] Geng, H., Clopotel, C.S., Bahia, H.U.: Effects of high modulus asphalt binders on performance of typical asphalt pavement structures, Construction and Building Materials 44 (2013), pp. 207-213

[7] Vacková, P., Valentin, J.: What information can be provided by the asphalt crack propagation test done on semicylindric specimens, World Multidisciplinary Civil Engineering - Architecture - Urban Planning Symposium 2020.

10

TRAFFIC: SIMULATIONS, COMPUTER TECHNIQUES, TRAVEL TIME AND SERVICE QUALITY

SELECTION AND ANALYSIS OF INPUT PARAMETERS INFLUENCING PEDESTRIAN MICRO-SIMULATED CROSSING TIME

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Abstract

Pedestrian unrestrained behaviour, sudden movements and vulnerability are elements, which can highly affect road safety, especially when interacting with motorized vehicles. Therefore, it is important to have a deep insight in pedestrian behaviour. A way to tackle this issue is micro-simulation. Modern micro-simulation tools, indeed, allow, thanks to the implemented mathematical formulation of the problem, to model and repeat a real situation in a virtual environment. Nevertheless, they need to well-fit the real observed behaviour: the calibration step allows to make the model reliable, by adapting selected, influential model input parameters. By dealing with pedestrian issues, software Vissim/Viswalk has been selected for micro-simulation, which implements Helbing's Social Force model. This model is based on several parameters, like relaxation time, side preference, strength and range of pedestrian interactions, amount of anisotropy, parameters governing the forces among pedestrians, noise, number of reacting pedestrians, queue order and straightness, which need to be set by the user when creating the model, but they can be hardly measured. This paper presents a selection of the recalled input parameters, on which statistical tests are carried out to understand their influence on the behavioural output – crossing time - that is supposed to describe pedestrian crossing behaviour. This is the first step towards the development of a new calibration methodology, which will keep advantage of artificial intelligence tools to fine-tune micro-simulation input parameters.

Keywords: pedestrian, micro-simulation, input parameters, statistics

1 Introduction and related works

Moving through urban area by foot, i.e. been a pedestrian, is one of the most wide spreading transportation ways. On the one hand it is an eco-friendly way of moving, it is healthy and it permits to easily and fast reach close facilities. On the other hand, the increase of walkers produces a greater interaction with motorized vehicles, letting the problem of pedestrian safety arise. As a matter of fact, pedestrians are the most free-to-move, but also the most vulnerable road users, and their behaviour is often not totally predictable for the other traffic participants. All these statements found the need of a deeper knowledge about walking behaviour, specifically in areas where interaction with motorized users exists. A way to tackle this problem, without affecting reality, is micro-simulation. This powerful and very promising tool allows, indeed, to reproduce and study a selected location on a virtual environment (computer), and to repeat many times the same external conditions – which in reality would

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always change. One of the most relevant issues highly affecting the results of a micro-simulation model is the choice of the parameters to fine-tune. [1] carried out an overview of all parameters used in previous research studies about traffic simulation model calibration, noticing that the most redundant ones are mean headway, mean reaction time, speed and, more generally, driver behavioural parameters. As regarding pedestrian simulation model calibration, the efforts spent in that way are much lower than for motorized traffic. In Table 1, the main works about calibration found in literature are reported and the utilized parameters are listed. 8 of the listed papers deal with social force model –may it be implemented in Vissim, or in a modified version of the recalled approach.

Table 1 Literature review about calibrated micro-simulation model parameters.

Authors	Calibrated parameters
[2]	free speed, anisotropy of social force; interaction strength and its range
[3]	not specifically told
[4]	parameters related to attractive and repulsive forces
[5]	interaction strength and range, obstruction effects of physical interactions, 4 social force parameters + 8 scenario specific parameters
[6]	pedestrian count, flow, passage time, no of overlaps
[7]	radius and comfort speed; comfort speed, neighbor distance, radius, time horizon; comfort speed, radius, 2 error-quantifying parameters
[8]	radius of pedestrians, A social, B social, B physical, border, A social Isotropie, B social Isotropie, τ, friction force, side preference right, velocity dependence, λ, longitudinal scale consider at maximum n pedestrians
[9]	pedestrian size, desired speed, time pressure
[10]	interaction strength and range, anysotropy
[11]	Interaction strengths and ranges for repulsive and attractive forces, relative distance, relative conflicting time, "footprint" effect.

What can be noticed is that the main parameters, which are modified by the authors to better fit real data, are the ones linked to the interaction strength and range of repulsive and attractive forces: these are also the parameters that majorly influence the dynamics in Vissim/Viswalk social force model. As stated in [12], the most important difficulty when thinking about parameter selection, fine tuning and calibration, is the interpretation of these magnitudes, which greatly affect various and different behavioural aspects. Focusing on Vissim/Viswalk social force model, the parameters which can be set up in the model are: reaction time, anisotropy, strength and range governing the interaction forces among pedestrians, time VD connected to pedestrian relative speeds, noise, number of pedestrians influencing the considered agent, queue order and straightness and side preference [13]. In the calibration attempt developed in [14], the authors use as initial parameter set a group of 13 magnitudes (Table 2, second column), which have been reduced to 4 in the other sets (Table 2, third column). As can be inferred from Table 2, not only the number of parameters changed, but also their values.

Table 2 Literature review about calibrated micro-simulation model parameters.

Parameters	Initial parameter values	2nd parameter set values	3rd parameter set values	Unit of measure
Radius of pedestrians	0.15			m
A social	0.5	0.1	2.5	m/s²
B social	2.8 m			1/s²
B physical, border	100			1/s²
A social, isotropic	25	10	100	m/s²
B social, isotropic	0.2	0.05	0.3	m
T	0.4			s
Friction force	0			
Side preference	Right			
Velocity dependence VD	2 s			s
λ	0.1			
Longitudinale scale	0.25			
maximum n pedestrians	5	15	15	

The present research focuses on pedestrian behaviour at roundabout crossings and aims at modelling pedestrian crossing time. Specifically, it will be developed in further steps a prediction model for the selected output (i.e. pedestrian crossing time) thanks to neural networks, which will be the tool to calibrate the developed micro-simulation model. The first step towards the development of such a calibration, is the selection and statistical analysis of the input parameters chosen as starting point for both models, as well as the understanding of their influence on pedestrian crossing time. In the following structure, the first chapter will recall the studies about pedestrian parameter selection and the choice made for this research. The second paragraph will describe the normality test held on the parameter datasets, while the third part will focus on the two statistical tests: non-parametric Kruskal-Wallis and parametric one-way ANOVA tests. Finally, the discussion of the results is provided and the first important conclusions of this statistical analysis are drafted.

2 The selection of input parameters

This study focuses on the problem of pedestrian crossing action, described by pedestrian crossing time. An existing roundabout located in an urban area and specifically the unsignalized crossing set on the main entry leg of the same have been chosen as study area. The crossing is 10.25 m long and 4 m wide and passes through two traffic lanes. After the definition of the geometrical features of this zone and their setting up in Vissim/Viswalk micro-simulation software, the most important step is the identification of which model parameters are influencing the expected output, i.e. crossing time. As summarized in the introduction, many authors already dealt with the right selection of input parameters to achieve an accurate model of pedestrian behaviour. Following the cited examples about parameter fine tuning [1] [2] [3] [4], it has been decided to select for this study 5 pedestrian parameters - tau, lambda, Asoc_iso, Bsoc_iso, side_pref - and their ranges (Table 3). Since in the situation analysed in this study there is a strict interaction with vehicular flow, also vehicular parameters have to be considered: the selected model to rule vehicular behaviour is Wiedemann 74, and the parameters chosen to be used are the three most affecting the car-following model, i.e. average standstill distance, additive part of safety distance and multiplicative part of safety distance (Table 3).

Table 3 Selected micro-simulation model input parameters.

Input	Name	Description	Min.	Max.
I1	Tau	Relaxation time	0.05	2
I2	Lambda	Amount of anisotropy	0	0.4
I3	Asoc_iso	Parameter governing pedestrian forces	3	7
I4	Bsoc_iso	Parameter governing pedestrian forces	0.1	10
I5	Side_pref	Side preference	-1	1
I6	Avg standstill distance [m]	Average standstill distance	1	3
I7	Additive part of safety distance [m]	Additive part of safety distance	1	5
I8	Multiplicative part of safety distance [m]	Multiplicative part of safety distance	1	6

Starting from this selection of input parameters and their ranges, a database of 100 random combinations of the same has been produced by applying a changing step of 0.1. Each one of these combinations has been then simulated via Vissim/Viswalk, and the simulated pedestrian crossing time has been added to the database. The following analyses have been based on the complete dataset, containing both the input combinations and their relative simulated output.

3 Results

3.1 Normality Test for crossing time simulation results

The first features that are essential to be discovered are the main statistical characteristics of the data and the type of distribution followed by the same. Descriptive statistics (Table 4) run on Vissim/Viswalk simulation results, shows that on a set of 100 data, the mean value is 6.407 s, while the median is 4.575 s. This let us infer that the distribution of the data could be non- normal.

Table 4 Descriptive statistics about pedestrian crossing time.

variable	N	N*	Mean [s]	StDev	Min [s]	Q1 [s]	Median [s]	Q3 [s]	Max [s]
VISSIM	100	0	6.407	3.502	2.440	3.673	4.575	7.773	14.5

To confirm this result, Anderson-Darling test has been set up. This test is based on the hypothesis that if the calculated p-value is lower than the set significance level, the distribution is not normal (null hypothesis for this test). Otherwise, it cannot be stated that the distribution is not normal – the null hypothesis is rejected, but no additional conclusions about the normality of the distribution can be made. In this study a significance level of 0.05 has been set, and the results reported in Table 5 have been obtained.

Table 5 Numerical results of Anderson-Darling test

Mean	StDev	N	AD	P-Value
6.407	3.502	100	6.355	<0.005

Also, the data plotting shows the non-normality of the selected dataset (Figure 1).

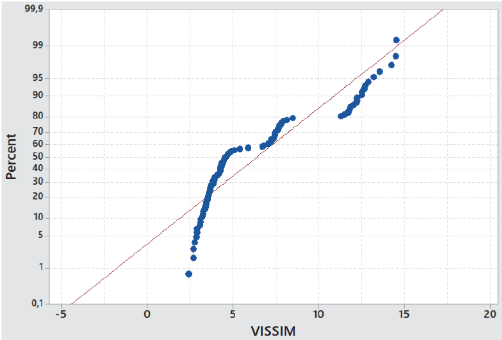

Figure 1 Probability plot of pedestrian crossing time: normal vs. data distribution.

3.2 Comparison of non-parametric and parametric test results

The next outcome, which will provide very useful information, is about the influence of the parameters on the selected output. Since the previous analysis confirmed the non-normal distribution of the data, firstly the non-parametric Kruskal-Wallis test has been used to examine this factor. Considering the large amount of available data and the strength of ANOVA, and following the suggestions of various authors [15]–[17], who recommend the use of this method also on data partially deviated from normality, when the recalled preconditions are available, the parametric ANOVA test has also been applied.

3.3 Kruskall-Wallis test

Kruskal-Wallis test is based on the comparison of the differences between medians. Specifically, it compares this magnitude to the set significance level – in this case 0.05 – and states if the null hypothesis „the population medians are all equal“ is valid or must be rejected. If the p-value is less than the chosen significance level, the null hypothesis must be rejected, otherwise it can be confirmed. In the considered study, Kruskall-Wallis has been applied to all the selected input parameters to understand their influence of the output „crossing time“. It turned out that all P-values are lower than the significance level (Table 6), and therefore all parameters are important in the calculation of the chosen outcome. For seek of completeness it has to be clarified that P-values reported in Table 6 (as regarding Kruskal-Wallis test) and Table 7 (referring to ANOVA analysis) are all null: actually, they differentiate one from the other for such low values, that do not influence the results, when compared to the significance level and thus they have been omitted. To understand which parameter influences the most the crossing time, the same test provides H-values. The greater this value is, the most influential the relative parameter. Table 6 summarizes also these results.
From Table 6 it can be inferred that the 3 most influential parameters are I1, I3 and I8, i.e. τ, Asoc_iso and multiplicative part of safety distance.

Table 6 Results of Kruskal-Wallis test on the selected parameters.

Input parameters	Description	H-values	P-values	Significance level
I1	Tau	80.63	0	0.05
I2	Lambda	47.67	0	0.05
I3	Asoc_iso	80.65	0	0.05
I4	Bsoc_iso	69.69	0	0.05
I5	Side_pref	27.15	0	0.05
I6	Avg standstill distance	48.04	0	0.05
I7	Additive part of safety distance	53.55	0	0.05
I8	Multiplicative part of safety distance	80.68	0	0.05

3.4 ANOVA

One-way ANOVA is a parametric test, which is considered by many authors [15]–[17]strong also for large dataset, which do not follow a normal distribution. In this case, ANOVA has been applied to compare the results achieved by Kruskal-Wallis test and establish if they agree or not. Being a parametric test, the precondition of ANOVA should be that means and medians equal each others, and its null hypothesis states that „the population means are equal “. When p-value is lower than the significance level this hypothesis must be rejected, otherwise it cannot. Analogously to Kruskal-Wallis test, there is a value, F-value, which highlight the degree of influence which each parameter has on the output. In Table 7 the results of ANOVA test, p- and F-values, are reported.

Table 7 Results of one-way ANOVA test on the selected parameters.

Input parameters	Description	F-values	P-values	Significance level
I1	Tau	229.52	0	0.05
I2	Lambda	27.85	0	0.05
I3	Asoc_iso	144.562	0	0.05
I4	Bsoc_iso	108.67	0	0.05
I5	Side_pref	10.25	0	0.05
I6	Avg standstill distance	19.76	0	0.05
I7	Additive part of safety distance	21.70	0	0.05
I8	Multiplicative part of safety distance	202.02	0	0.05

4 Discussion

The parametric and non-parametric statistical analysis of the same large database has allowed to make some considerations about the parameters chosen as influential on pedestrian behaviour, specifically pedestrian crossing time. Indeed, the first selection of those parameters has been made based on literature considerations and observations, but the application of these tests provides an insight in the importance and influence of the preliminary chosen parameters from a mathematical point of view. It is very promising that both the parametric one-way ANOVA test and the non-parametric Kruskal-Wallis test turn the same results. As can be seen from previous paragraphs, both tests underline the importance of all parameters, which – consequently – have to be considered in the further model. They also allow to score the parameters, from the most to the last influential, underlining that multiplicative part of safety distance, τ and Asoc_iso are the most powerful ones, followed respectively by Bsoc_iso, additive part of safety distance, average standstill distance, lambda and side preference. The only difference between the results of the two methods is that ANOVA recognizes a more powerful parameter in τ, followed by the multiplicative part of safety distance, Asoc_iso and Bsoc_iso. Also, the comparison between the two methods confirms the suggestions that ANOVA can be a strong statistical tool also for non-normal distribution datasets, if they are large enough.

5 Conclusions

Micro-simulation is a powerful tool to study traffic dynamics. This is valid also when dealing with pedestrians. In this paper the selection of input parameters for a micro-simulation model and its further calibration is presented. The first set of parameters has been chosen on the basis of literature observations, and a database with 100 random combinations of the selected parameters has been produced. Each one of this combination have been simulated via Vissim/Viswalk in order to obtain the chosen output, pedestrian crossing time. A first statistical analysis of the output results showed that it does not have a normal distribution. This statement led to the decision of applying a non-parametric test, Kruskal-Wallis one, to the database in order to analyse parameter influence on pedestrian crossing time.
Since the data base was large enough, it has been established the precondition set by [15] – [17], who states that also one-way ANOVA can be strong enough to statistically analyse which parameters are influential for the considered problem. The achieved results confirmed that the two methods were equivalent for the studied issue. Both Kruskal-Wallis and ANOVA tests turned out that all selected parameters are influential in the calculation of crossing time and must be considered in further elaborations. Also, thanks to H- and F-values respectively, they allowed to score the parameters from the most to the last influential. The results of this study are essential for next steps: indeed, they will be used as input parameters not only for the micro-simulation model, but they will also be implemented in the creation of a prediction model of crossing time. This independent model, developed thanks to neural networks, will be the tool used to develop a methodology to calibrate pedestrian micro-simulation models.

References

[1] Lee, J. B., Ozbay, K.: Calibration of a Macroscopic Traffic Simulation Model Using Enhanced Simultaneous Perturbation Stochastic Approximation Methodology, 2008.

[2] Liao, W., Chraibi, M., Seyfried, A., Zhang, J., Zheng, X., Zhao, Y.: Validation of FDS+Evac for pedestrian simulations in wide bottlenecks, 17th IEE International Conference on Intelligent Transportation Systems, ITSC 2014, 2014, pp. 554-559.

[3] Berrou, J.L., Beecham, J., Quaglia, P., Kagarlis, M.A., Gerodimos, A.: Calibration and validation of the Legion simulation model using empirical data, Pedestrian and Evacuation Dynamics 2005, Springer Berlin Heidelberg, 2007, pp. 167-181

[4] Rudloff, C., Matyus, T., Seer, S., Bauer, D.: Can Walking Behavior be Predicted? Analysis of Calibration and Fit of Pedestrian Models, Transportation Research Record: Journal of the Transportation Research Board, 2264 (2011) 1, pp. 101-109, doi: https://doi.org/10.3141/2264-12

[5] Zhong, J., Hu, N., Cai, W., Lees, M., Luo, L.: Density-based evolutionary framework for crowd model calibration, Journal of Computer Science, 6 (2015), pp.11-22

[6] Voloshin, D., Rybokonenko, D., Karbovskii, V.: Optimization-based Calibration for Micro-level Agent based Simulation of Pedestrian Behavior in Public Spaces, Procedia Computer Science, 2015, pp. 372-381

[7] Wolinski, D., Guy, S.J., Olivier, A.-H., Lin, M., Manocha, D., Pettrè, J.: Parameter estimation and comparative evaluation of crowd simulations, Comput. Graph. Forum, 33 (2014) 2, pp. 303-312

[8] Kretz, T., Hengst, S., Vortisch, P.: Pedestrian Flow at Bottlenecks: Validation and Calibration of Vissim Social Force Model of Pedestrian Traffic and its empirical Foundations

[9] Hongfei, J., Lili, Y., Tang, M.: Pedestrian Flow Characteristics Analysis and Model Parameter Calibration in Comprehensive Transport Terminal, Journal of Transportation Systems Engineering and Information Technology, 9 (2009) 5, pp. 117-123

[10] Johansson, A., Helbing, D., Shukla, P.K.: Specification of a Microscopic Pedestrian Model by Evolutionary Adjustment to Video Tracking Data

[11] Zeng, W., Chen, P., Yu, G., Wang, Y.: Specification and calibration of a microscopic model for pedestrian dynamic simulation at signalized intersections: A hybrid approach, Transportation Research Part C: Emerging Technologies, 80 (2017), pp. 37-70

[12] Kretz, T., Lohmiller, J., Sukennik, P.: Some indications on How to Calibrate the Social Force Model of Pedestrian Dynamics, Transp. Res. Rec. J. Transp. Res. Board, 2672 (2018) 20, pp. 228-238

[13] VISSIM 5.30-05 User Manual, 2011.

[14] Kretz, T., Hengst, S., Vortisch, P.: Pedestrian Flow at Bottlenecks – Validation and Calibration of Vissim's Social Force Model of Pedestrian Traffic and its Empirical Foundations, May 2008.

[15] McDonald, J.H.: Handbook of Biological Statistics, Second Edition, 2014.

[16] Hoskin, T.: Parametric and Nonparametric: Demystifying the Terms

[17] Kitchen, C.M.R.: Nonparametric vs Parametric Tests of Location in Biomedical Research, American Journal of Ophthalmology, 147 (2009) 4, pp. 571-572

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THE ARCHITECTURE OF DECISION-MAKING SUPPORT SYSTEMS USED FOR THE RATIONAL RAILWAY CAPACITY MANAGEMENT

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Abstract

Decision-making support systems in railway transport are systems that make it easier for traffic controllers and dispatchers involved in the regulation of train traffic to make individual decisions more easily and accurately. Without such systems, dispatchers usually make decisions based on previous experiences and feelings they have developed working in train traffic control. However, quality decision-making support systems are based on large amounts of data processed by one or several different artificial intelligence techniques. This paper will examine the architecture of such a system in railway transport, which helps the dispatcher to make decisions based on different criteria and values of individual criteria. The architecture of this decision-making support system has been developed to equal or, if necessary, use the maximum available double-track railway line capacity to resolve delays caused by lack of capacity for any given route. This system has been developed for the specific configuration of a double track, whereby each track is intended for one direction of train traffic. This paper will lay the foundation for understanding decision-making support systems and for the development of a specific model of decision-making support system in practice.

Keywords: railway capacity optimization, decision-making support system, dynamic traffic control

1 Introduction

The railway is a specific transport system because it comprises vehicles that can operate only on specific infrastructure. Furthermore, vehicles and tracks require particular signalling regulation and are limited to a specific direction of travel. Despite certain flaws (inability to change the direction of travel) ensures numerous benefits whilst ensuring a high level of safety. The railway can transport a great number of passengers or a large amount of cargo on a relatively small surface. For that reason, the railway has become the transport method of choice for mid-haul distances (150–600 km). From the expenses standpoint, the railway provides an additional advantage because it remains competitive for mid-haul journeys and is being used more and more. Many tracks that were built a long time ago require investments and modernization. Because of the excessive use due to numerous economic and ecological advantages and due to lack of investments, tracks have reached the limits of their transport capacities. This lack of available capacity on some tracks has led to operational traffic management whose aim is to minimize delays and maximize infrastructure utilization. This paper analyses the assistance systems used in making decisions in railway transport management. Also, the paper sets out to provide a comparative overview of systems to determine their benefits and drawbacks. Furthermore, the paper puts forward a new proposal for system ar-

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chitecture which would help make decisions in specific conditions that may arise on double tracks. Such systems are devised, planned, and implemented to help dispatchers to make decisions on train movement without or with minimum delays. At certain intervals of the day, month and year, there can appear disbalance in transport demand for some directions of transport. These situations lead to large capacity utilization and can generate substantial delays and traffic issues. However, since double tracks are constructed so that each track is intended for one direction of train travel, each track is limited in capacity, for both directions of travel.

2 Systems of decision-making in track capacity management

This part of the paper examines the systems that were developed for managing railway capacity. They can be used in managing various situations and assist in making decisions based on real-time processed and analysed data. These decision-making support systems follow the same principles based on which a dispatcher makes decisions. Dispatchers are in charge of operational railway transport management. They supervise the overall situation on a certain section of the track and based on knowledge, real-time data, and experience, they make professional decisions to normalize delays and reducing them to an absolute minimum.

According to [1] delays and disorders in transport impact the reliability and stability of the railway. They examine the simultaneous availability of larger disorders (temporary complete or partial blockage of tracks) as stochastic variation as the source of these disorders. The paper defines the time of the blockage and the time required to resolve the blocked track. A model was devised based on simulation and it includes dynamic priority rules for dispatching that aims to reduce the overall delay time. Also, a meta-heuristic scheme was devised that would search for available tracks within the limited capacity near the blockage. The model also calculates new arrival, departure, and journey times. It was tested on the Iranian railway network and, according to the authors, it yields very good and quick solutions equal to ones generated by commercial simulation and optimization software. The proposed solutions also include a lesser deviation compared to the solutions widely accepted today both by dispatches and standard software packages.

Author in [2] describes realistic solutions to transport coordination and coordination of transportation signalization in urban transport networks. The authors present a clock-face timetable that expands the network using a static and linear model. Such a transport flow model is based on the most modern statistic and dynamic models enabling at the same time the optimization of the timetable and coordination of the transport light signalling using precise techniques of mathematical programming. This paper also examines the inherent properties of journey times, demonstrating their capabilities in simulations made in high-quality simulation solutions.

In [3] authors write about railway signals, which is the most significant component of the railway system as it is the only aspect that ensures safety during the realization of railway services. Therefore, when there is an interruption in railway transport, dispatchers aim to adjust the affected timetable and minimize the negative effects during and after the interruption. Dispatchers manage train traffic using railway signals to communicate with train operators at fixed blocks. In such systems, signals depend on the movement of previous trains on that particular section or in the entire network. Previous works written on this have not concerned the impact of a signal on train movement and regulation. This paper proposes a new set of signal restrictions to describe the impact of signalling terms. Signal restrictions are based on the if-then rule. The authors assert the policy of “green lights”, that is, trains operate whenever possible without limitation.

Paper [4] describe the model of managing urban-suburban transport while adopting dynamic timetable. This includes a support system used when adjusting the timetable to the real-time situation in railway transport by means of a genetic algorithm defined by rules of regulating railway transport and train movement. Such a model provides a response to increased demands for railway capacity, that is, better utilization of available track capacity. At the same time, the paper describes the use of an expert system to manage capacity utilization – to save electricity. Case studies in the paper prove the advancement of railway urban and suburban transport by saving power energy.
Based on an optimization method for solving issues in train timetables, in the [5] authors presented their own optimization method. The aim is to reduce the overall train journey time in the railway network. SIMARAIL is the software that is used to plan timetables, which takes into account capacity limitation and infrastructure features. The simulation model uses a detailed micro-model of tracks and stations and infrastructural information.
A deeper look into the papers and what they deal with reveals that all decision-making support systems work based on already existing criteria, values, and rules, based on which these decisions are made. An adequate definition of the criteria is, therefore, an important factor in making optimal decisions. Subsequent research should pay particular attention to defining the criteria so as to develop a quality decision-making support system. The substance and approach will be of great assistance in that process.
The criteria include entry parameters and optimal time and duration of opening a second track for the same direction which would quickly and efficiently resolve delays.

3 Decision-making support systems in operational transport management

This part of the paper examines the support systems used in decision making which can be of use in managing various situations or when making decisions based on processed data.
There are support systems for making decisions on train movement in real-time and real-world transport conditions. They aim to reduce pollution and energy use. Such systems help make decisions based on the same principles that dispatches follow. In a way, the systems enable the making valid business decisions or selecting the optimal variants. However, the most important in decision-making support systems are systems that help make an optimal decision on regulating train traffic, i.e. decisions that help dispatchers.
Decisions made by dispatchers mainly concern train transport on tracks with a large amount of traffic or the regulation of trains at peak times when the maximum track capacity utilization is likely. Dispatcher decisions are founded on experience, intelligence, and intuition, taking into account the situation in transport and the traffic structure itself. Commonly, this includes decisions on the number of passenger and cargo trains currently operating, and their type (speed). For the decisions to be more rational, dispatchers are advised to make decisions based on indicators and the change in indicators over time. Since dispatchers cannot monitor and process large amounts of data in real-time, specialized tools are used to make their jobs easier. This is the reason why decision-making support systems need to be developed, as they can handle huge amounts of data quickly. Numerous authors have developed similar systems [6], [7]. Therefore, as can be seen from previous efforts, decision making is an important issue, which many authors have struggled to resolve.
In [8], the authors dealt with the criteria that may affect decisions when track capacity must be increased in a single direction. It is vital to define the normative values of the criteria that are used in the decision-making process. To do so, decision-makers must be familiar with the criteria, how they impact the potential increase/decrease in value, as well as the train operation process. In addition, the entire process must include an automatic value monitoring so that the parameters could be used as entry data for the decision-making system.

In fact, the most important aspect of the criteria is their ranking and the cost of waiting for the capacity to increase. The importance of every criterion needs to be defined so that they can later be compared and used in the decision-making process.
The paper [8] also includes a solution simulation for additional capacity on the case study of the M104 line. Additional capacity would include another track to be used for train traffic for the direction opposite of the one planned. The simulation-obtained result which claims that the second track would use as much as half of the capacity for the direction that had been planned. The next chapter provides the architecture of such a system which would help dispatchers to make decisions based on monitoring certain parameters.

4 The architecture of decision-making support systems in railway track capacity management

The previous chapter described the support system used in decision making. However, for the system to function properly and assist dispatchers to make decisions, it needs to be adequately described and devised. To do so, we first need to have a good understanding of how dispatchers make decisions. Since the system cannot think on its own, it needs to be made "smart" by providing it with quality input parameters. These parameters must be properly defined. The logic of the system has to rely on it providing a satisfactory solution.
For the system to be operational, it needs to be simulated using quality tools because this system is designed to in some way optimize the adequate use of the railway traffic network, or part of the double-track railway in this case. As this system has been conceived to optimize the adequate utilization of transport networks, the simulation should adopt a Petri net, given that it is a mathematical tool that can represent discrete systems such as train traffic on Fig. 1. [9]

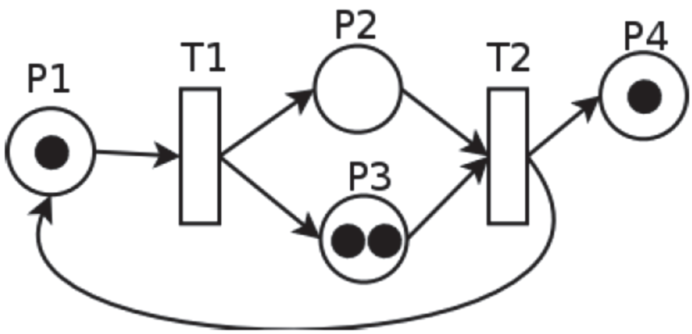

Figure 1 A Petri net model chart

Fig. 1 illustrates the Petri net places (P), transitions (T), markings, and input and output places. Places are marked with circles and transitions with rectangles or short straight lines. Markings represent tokens at places. The number of tokens equals the value of the marking. Input and output places are marked with one-way arcs.
Apart from the discrete events of time flow, the model needs to define the requirements and restrictions imposed by the system. Petri nets are first and foremost focused on conditioning the transition from one state to another after a condition or an event has been met, whereby the event can be the moment a certain time has passed in a state.
As Petri nets do not have features required to simulate such complex dynamic processes, they have to be upgraded to:

- High-level Petri nets: New generation of Petri nets with the same features
- Hierarchical Petri nets: have the option of creating hierarchical nets where the movement of tokens is monitored at several levels. Furthermore, Petri nets here can become a token of the Petri net
- Timed Petri nets: an important feature because it allows tracking of trains moving through railway junctions
- Stochastic Petri nets: train delays can be added to the model. Delays are modeled based on some of the established distributions or laws
- Colored Petri nets: have a practical way of using different colors to mark every train or type of train. This ensures train movement management in the model as well as a detailed tracking and analysis of train movement.

Therefore, the combination of all these upgrades together with external tools used in decision-making makes it possible to develop a system which can help dispatchers to make decisions on utilizing additional track capacity, based on tracking the criteria and parameters that affect those decisions.

In that conceptualized overview of the railway system modeled using Petri nets, places (p) are tracks or blocks of tracks. Transitions (t) indicate block signals and other elements that impact or can impact the normal train operation. Tokens indicate trains that operate on the network or parts thereof. Using colored Petri nets, additional train parameters can be marked. Such concepts of train simulation and mathematical modeling the Petri net can be used to devise and simulate a specific system that can be used in the decision-making process that regulates capacity management. This is particularly useful for when transport demand exceeds capacity supply. [9]

5 Conclusion

The railway is a complex dynamic system which regularly faces new demands and increase in traffic, that is, more and more transport services combined with fresh organizational and technological challenges. The challenges are mainly the inability to respond to ever greater demands for unutilized capacity put forward by railway undertakings. For this reason, experts are continuously trying to answer the challenges and utilize the capacity to the fullest by optimizing train operation. Dispatchers are in charge of operational train transport. With their skill and experience, they make decisions on how trains should operate. However, sometimes these decisions are not optimal, which is why some infrastructure managers are looking into specialized decision-making support systems. The proposals for potential solutions are based more amounts of data which are often more precise than the ones at dispatcher's disposal at any given moment. It is essential that these systems can in a very short time process large amounts of data with great precision, based on which they then can make a decision.

For exactly that reason, this paper has outlined the architecture of the decision-making support system for operational train transport management which would help improve dispatcher decisions concerning maximum capacity utilization. The system is largely based on input criteria which are constantly tracked. Based on these values and the impact these criteria have on each other, optimal solutions are provided.

The concept of such a system has already been tested in various case studies and by using simulation tools for model railway systems. This paper has put forth a proposal for upgrading the system and simulate it by using Petri nets as foundations for further system development.

Acknowledgements

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References

[1] Shakibayifar, M., Sheikholeslami, A., Corman, F.A.: Simulation-based optimization approach to rescheduling train traffic in uncertain conditions during disruptions, SCI Iran, 25 (2018) 2A, pp. 646–662

[2] Köhler, E., Strehler, M.: Combining Static and Dynamic Models for Traffic Signal Optimization Inherent Load-dependent Travel Times in a Cyclically Time-expanded Network Model, Procedia - Social Behavior Science, 54 (2010), pp. 1125–1134, http://dx.doi.org/10.1016/j.sbspro.2012.09.827

[3] Luan, X., De Schutter, B., Corman, F., Lodewijks, G.: Integrating Dynamic Signaling Commands Under Fixed-Block Signaling Systems into Train Dispatching Optimization Problems, Transportation Research Record, 2018, https://doi.org/10.1177/0361198118791628

[4] Toš, Z., Mlinarić, T.J., Haramina, H.: Managing Rail Traffic on Commuter Lines Based on Dynamic Timetable Application, PROMET - Traffic&Transportation, 23 (2012) 6, pp. 413–420

[5] Sajedinejad, A., Mardani, S., Hasannayebi, E.S., Ahmad Reza Mir Mohammadi, K., Kabirian, A.: Simrail: Simulation based optimization software for scheduling railway network, pp. 703–715, 2011.

[6] Lindfeldt, O.: An analysis of double-track railway line capacity, Transport Planning Technology, 34 (2011) 4, pp. 301–22

[7] Abril, M., Barber, F., Ingolotti, L., Salido, M.A., Tormos, P., Lova, A.: An assessment of railway capacity, Transportation Research Part E: Logistics Transportation Review, 44 (2008) 5, pp. 774–806

[8] Ljubaj, I., Mlinarić, T.J.: The Possibility of Utilising Maximum Capacity of the Double-Track Railway By Using Innovative Traffic Organisation, Transportation Research, Procedia, 40 (2019), pp. 346–53

[9] Milinković, S.: Modeliranje saobradajnih procesa u složenim sistemima sa rasputnicom, Univerzitet u Beogradu, Saobraćajni fakultet, 2007.

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ENHANCING CAPACITY ON ETCS LEVEL 2 LINES IN AUSTRIA

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Abstract

The European Train Control System (ETCS) is also applied in the network of Austrian Railways (ÖBB). In ETCS Level 2 it is possible to skip conventional trackside signals. This opportunity is very useful on recently built new high speed lines. Level 2 allows to increase capacity by adding additional blocks even on existing lines. This paper presents the potential increase of capacity on the high speed line between Vienna and St. Pölten which has been opened in the year 2012 and is operated by ETCS Level 2. Trackside signals are only used as a fall back option. Simulation software OPENTRACK has been applied to calculate headways for different block layouts. Results are presented in this paper and look promising for an increase of capacity on railway lines.

Keywords: ETCS, headway calculator, capacity

1 Introduction

This paper presents the application of software OpenTrack for the calculation of headways for different scenarios of ETCS levels on the Austrian railway line from Vienna to St. Pölten. While chapter 2 describes the functionality of software OpenTrack, chapter 3 presents the specific features of ETCS Level 1, 2 and 3. Finally, Chapter 4 shows the application for the railway line from Vienna to St. Pölten. Chapter 5 concludes the findings from the showcase.

2 Functionality of Software OpenTrack

OpenTrack was developed at the Swiss Federal Institute of Technology's Institute for Transportation Planning and Systems (ETH IVT). The project's goal was development of a user-friendly railroad simulation program that can run on different computer platforms and can answer many different questions about railway operations. [1] Figure 1 illustrates the three main elements of OpenTrack: data input, simulation, and output.

OpenTrack is a microscopic synchronous railroad simulation model. As such it simulates the behaviour of all railway elements (infrastructure network, rolling stock, and timetable) as well as all the processes between them. It can be easily used for many different types of projects including testing the stability of a new timetable, evaluating the benefits of different long-term infrastructure improvement programs, and analyzing the impacts of different rolling stock.

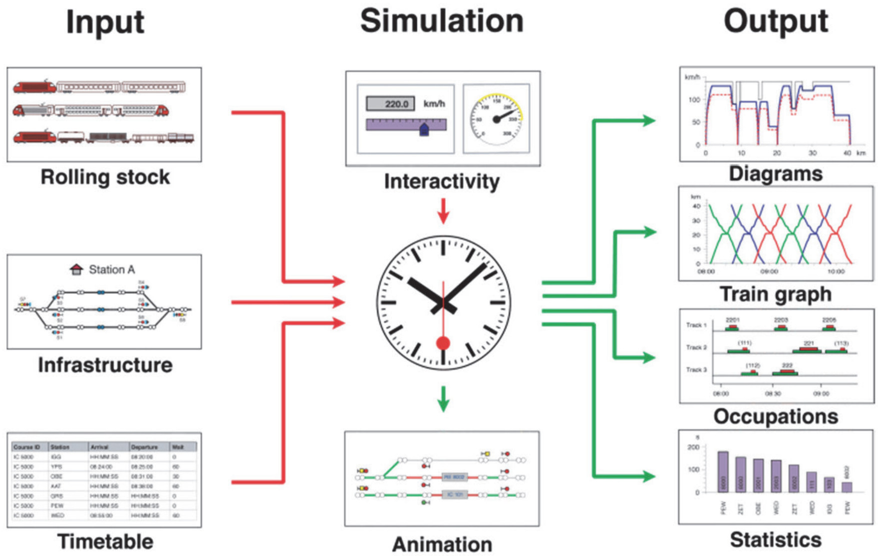

Figure 1 Data flow in a simulation project

2.1 Input data

OpenTrack administers input data in three modules: rolling stock (trains), infrastructure, and timetable. Users enter input information into these modules and OpenTrack stores it in a database structure. Once data has been entered into the program, it can be used in many different simulation projects. For example, once a certain locomotive type has been entered into the database, that locomotive can be used in any simulation performed with OpenTrack. Similarly, different segments of the infrastructure network can be entered separately into the database and then used individually to model operations on the particular segment or together to model larger networks.
Train data (locomotive and wagons) is entered into the OpenTrack database with easy to use forms displayed using pull down menus. Infrastructure data (e.g. track layout, signal type/location) is entered with a user-friendly graphical interface; quantitative infrastructure data (e.g. elevation) is added using input forms linked to the graphical elements. Following completion of the RailML data structure for rolling stock and infrastructure, OpenTrack will be modified to enable train and infrastructure data to be directly imported from RailML data files [2].
Timetable data is entered into the OpenTrack database using forms. These forms include shortcuts that enable data input to be completed efficiently. For example, users can designate hourly trains that follow the same station stopping pattern an hour later. Since OpenTrack uses the RailML structure for timetable data, timetable data can also be entered directly from various different program output files as well as database files.

2.2 Simulation

In order to run a simulation using OpenTrack the user specifies the trains, infrastructure and timetable to be modeled along with a series of simulation parameters (e.g. animation formats) on a preferences window. During the simulation, OpenTrack attempts to meet the user-defined timetable on the specified infrastructure network based on the train character-

istics. OpenTrack uses a mixed continuous/discrete simulation process that allows a time driven running of all the continuous and discrete processes (of both the vehicles and the safety systems) under the conditions of the integrated dispatching rules.
The continuous simulation is dynamic calculation of train movements based on Newton's motion formulas. For each time step, the maximum force between the locomotive's wheels and the tracks is calculated and then used to calculate acceleration. Next, the acceleration function is integrated to provide the train's speed function and is integrated a second time to provide the train's position function.
The discrete simulation process models operation of the safety systems; in other words, train movements are governed by the track network's signals. Therefore, parameters including occupied track sections, signal switching times, and restrictive signal states all influence the train performance. OpenTrack supports traditional multi-aspect signalling systems as well as new moving block train control systems (e.g. European Train Control System – ETCS signalling).
Finally, dynamic simulation enables users to run OpenTrack in a step-by-step process and monitor results at each step. Users can also specify exactly what results are displayed on the screen. Running OpenTrack in a step-by-step mode with real time data presented on screen helps users to identify problems and develop alternative solutions.

2.3 Output

One of the major benefits of using an object oriented language is the great variety of data types, presentation formats, and specifications that are available to the user. During the OpenTrack simulation each train feeds a virtual tachograph (output database), which stores data such as acceleration, speed, and distance covered. Storing the data in this way allows users to perform various different evaluations after the simulation has been completed.
OpenTrack allows users to present output data in many different formats including various forms of graphs (e.g. time-space diagrams), tables, and images. Similarly, users can choose to model the entire network or selected parts, depending on their needs. Output can be used either to document a particular simulation scenario or as an interim product designed to help users identify input modifications for another model run.

3 Simulation of the European Train Control System (ETCS)

Regarding the equipment of the lines, ETCS specifications distinguish five application levels: the levels 0, 1, 2, 3 and STM [3]. Level 0 just describes the situation where a vehicle which is equipped with ETCS moves in an unequipped area. Level STM (Specific Transmission Module) is designed for situations where a train which is equipped with ETCS moves on a line without ETCS, but with a national train protection system. This level has been developed for the migration period.
In Level 1 the main transmission medium are so called Balises which transmit movement authorities and profile data to the train. Balises can be fix data or switchable balises. The former store the data content in the balise itself (only static data), whereas the latter, a Lineside Electronic Unit (LEU) selects the data according to input information (e.g. signal aspects). Besides the balises, linear infill devices can be used locally to transmit changes of signal aspects beyond. These are Euroloops (cable loops in the rail) or radio infill units. Therefore Level 1 provides continuous guidance functions by movement authority.

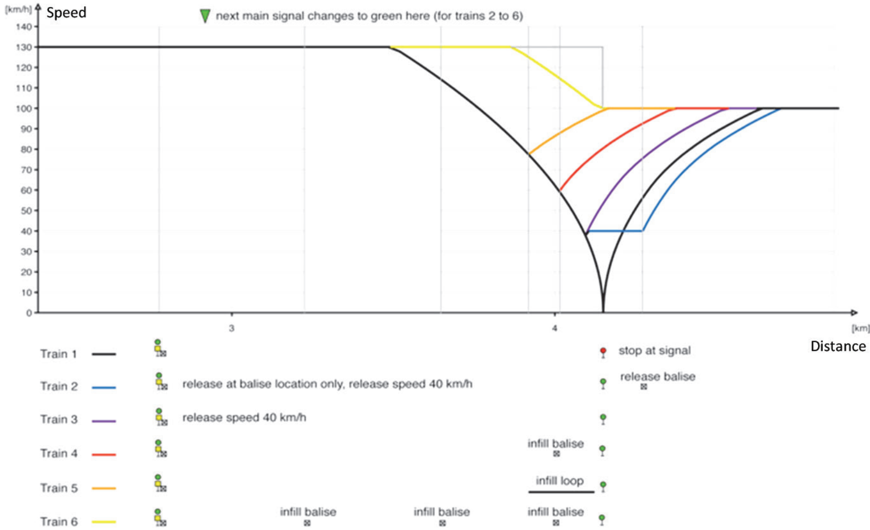

Figure 2 Operational impact of balises and infill loop in ETCS Level 1

The main operational challenge of ETCS Level 1 can be explained by figure 2. Train 1 shows the safety case of the ETCS application. If the next main signal shows some stop aspect, the braking curve of the approaching train is supervised. Trains 2 to 6 passed the closed distant but then at the green triangle point the next main signal is upgraded because the block is free again. Most of traditional signalling systems have therefore designed some liberating function, e.g. the driver has to confirm the upgraded signal aspect in sight distance and is thereby allowed to accelerate again. In ETCS Level 1 the infrastructure has to be equipped with balises or loop to transmit the upgraded signal aspect to the engine. Train 2 to 6 show the different scenarios for infrastructure equipment. Train 2 has to pass a release balise behind the main signal to be allowed to accelerate again. Of course this is the most restrictive approach. Train 3 has only to brake until the release speed of 40 km/h is reached. This scenario is not possible in ETCS but in many traditional signalling systems and is therefore used as a benchmark. Train 4 is able to receive the upgraded signal aspect by an infill balise. Train 5 is using the opportunity of receiving the upgraded signal aspect by a loop. As the installation costs for a loop are typically high the solution with several infill balises like for train 6 is more convenient.
Figure 3 shows the infrastructure layout for a railway line where only the section in the middle is equipped with ETCS Level 2.

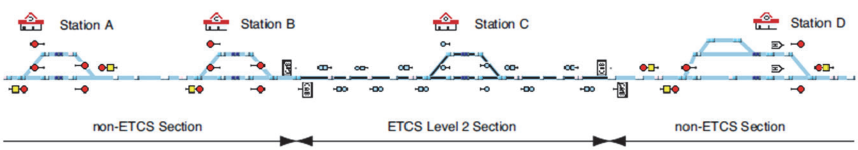

Figure 3 Infrastructure layout with non-ETCS and ETCS Level 2 sections

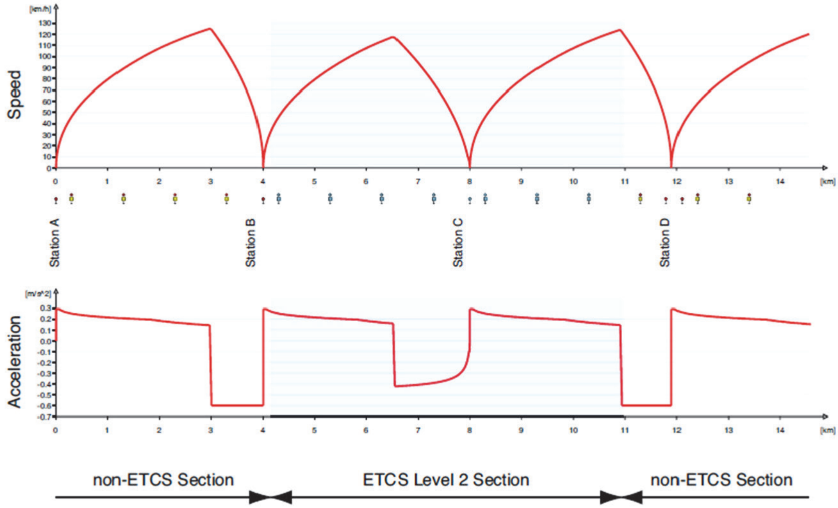

Figure 4 Speed-distance and acceleration-distance diagrams for non-ETCS and ETCS Level 2 sections

In Level 2 and 3 information can be continuously and bidirectionally transmitted by Euroradio, a radio standard based on GSM-R. The central trackside unit is the Radio Block Centre (RBC). In contrast to Level 1 the trains are individually known in the RBC. The train requests new movement authorities in regular time intervals or at particular events. A difference between Level 2 and 3 is that in Level 2 ETCS only takes the responsibility for signal and train protection functions, whereas Level 3 also replaces the interlocking-based track clear detection by continuously checking train completeness on the train and transmitting this information to the RBC.
The interesting investigation of non ETCS and ETCS Level 2 sections is the behaving of trains in terms of speed and acceleration. Finally the running time is influenced and thereby the timetable has to be modified. Figure 4 shows the impact of more restrictive braking curves in Level 2. The maximum speed achieved by the train is lower and the braking procedure has to start earlier due to reduced deceleration.
As ETCS Level 3 allows the application of moving block the investigation of switch operations becomes relevant. Figure 5 shows an example of a junction where three trains follow each other. In the left part they are all using the same track. In the right part they are coming from different tracks which requires a switch operation. Thereby the minimum headway is increased by 14 seconds. Moving block is today typically applied for new metro systems to achieve short headways.

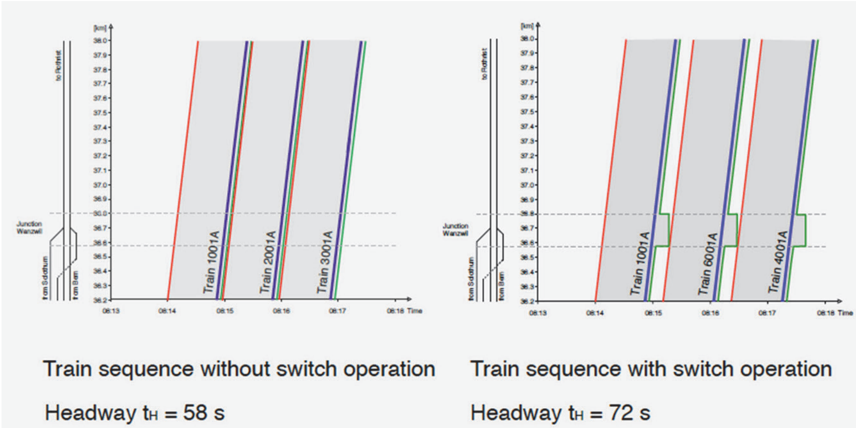

Figure 5 Impact of switch operations on headway in ETCS Level 3

4 Show case in Austria: Vienna – St. Pölten High Speed Line

Since 2012 the new high speed line between Vienna and St. Pölten is in operation. The line is designed for speeds up to 250 km/h. Unfortunately, there is no train in Austria available to reach this speed limit. Nevertheless, Austrian Railways use their RailJet services with a speed limit of 230 km/h. RailJet consists out of a Taurus engine and seven trailers. Taurus engines are also used for freight trains with a maximum speed of 100 km/h. For both headway scenarios the headway calculator of OpenTrack has been used to compare headways with ETCS Level 2 vs. ETCS Level 3.

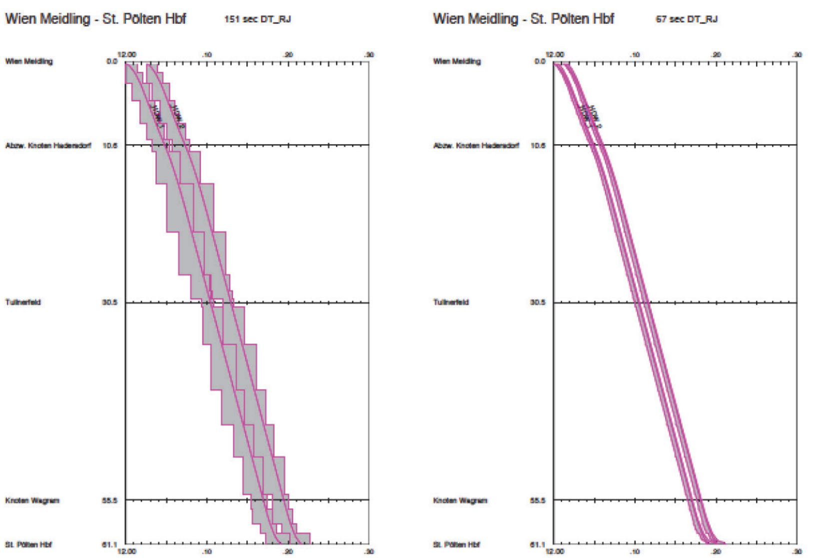

Figure 6 Headway for two RailJets with ETCS Level 2 (left) and 3 (right) in Time-Distance-Diagram

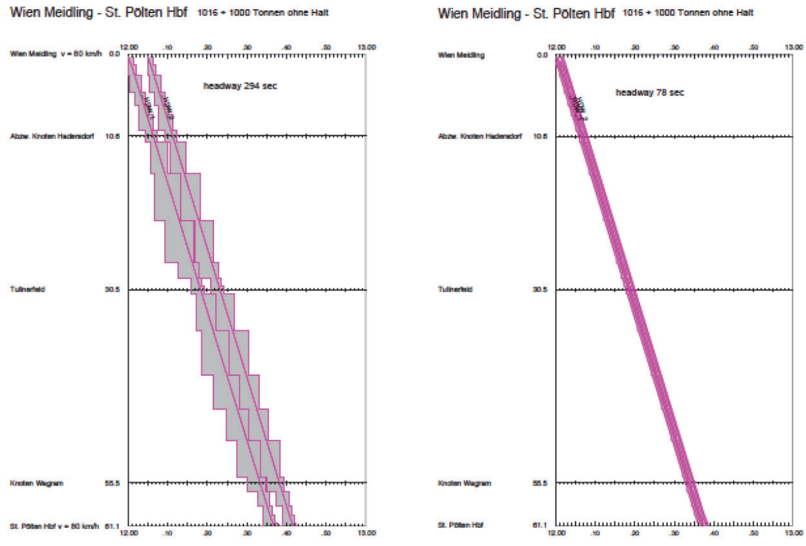

Figure 7 Headway for two Cargo trains with ETCS Level 2 (left) and 3 (right) in Time-Distance-Diagram

Results are promising since ETCS Level 3 would allow to shorten the headway from 151 to 67 seconds for two RailJet services and from 294 to 78 seconds for two cargo trains. During day time this would allow to double the frequency of passenger services. Even more interesting is the possible increase of capacity during night time for cargo trains. ETCS Level 3 with moving block would allow to have almost four times more cargo trains. Of course, ETCS Level 3 is currently not available as a solution on the market but with short blocks in ETCS Level 2 headways can be pretty close to the ones possible in ETCS Level 3. Signals can be only virtual in ETCS Level 2 while axle counters have to be physically placed wayside wherever a short block ends.

5 Conclusions

OpenTrack is an efficient and effective railway simulation program. It has been used successfully in many different railway planning projects throughout the world. The program's use of object oriented programming and the RailML data structure makes it particularly effective since the program can be modified relatively easily to address specific applications and since data can be transferred easily to and from other programs based on RailML. Therefore it seems to be highly recommended to apply it also for the evaluation of the operation performance for the planned introduction of ETCS in the network of Croatian Railways. As the Austrian show case has clearly shown an increase of capacity even on high speed lines is possible. Another field of application is the implementation of ETCS Level 2 on existing lines with shorter blocks than in the existing layout.

References

[1] Huerlimann, D.: Object oriented modelling in railways, ETH Dissertation Nr. 14281, 2001 (in German)

[2] railML www.railml.org

[3] Theeg, G., Vlasenko, S.: Railway Signalling & Interlocking. PMC Media GmbH, 2019, ISBN 978-3-96245-169-1

MOTORWAY WORK ZONES CAPACITY ESTIMATION USING FIELD DATA FROM SLOVENIA

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Abstract

Work zones represent typical bottlenecks on motorway sections. Due to aging of Slovenian motorway network, roadworks and the corresponding delays are becoming a major issue. While the majority of roadworks is planned, it is not straightforward to predict the delays they will cause even with a good prediction of the traffic volumes. The problem is to determine the work zone capacity, which is affected by several work zone characteristics, road elements and traffic structure. Besides, the stochastic nature of capacity cannot be ignored, since the same traffic values can result in different traffic conditions. This paper presents work zone capacity estimation based on the data of the recent work zones on Slovenian motorways. We collected data of various short- and long-term work zones that were recently present on Slovenian motorway network. We have studied the effect of heavy vehicles, longitudinal grade, crossovers and lane narrowing on the work zone capacity. Two statistical methods were applied to study those factors, namely regression analysis and survival analysis combined with maximum likelihood method. For each work zone type we have estimated the Weibull cumulative distribution function of capacity and queue discharge rate. The estimated capacity can provide an estimate of breakdown probabilities, delays and levels of service of motorway sections in work zones and as such it provides a useful tool in planning maintenance of the motorway network.

Keywords: capacity, cumulative distribution function, maximum likelihood method, regression analysis, work zone

1 Introduction

Safe and reliable motorway network requires regular maintenance resulting in common work zones, i.e. part of the motorway sections under roadworks. While different parts of a work zone can have different capacity, the capacity of a road zone is considered the part of the work zone with the lowest capacity. Several factors appear to affect the capacity; however, determination of their effect can be a difficult task. Researchers also note that no appropriate model has been developed to accurately determine the impact of various factors on work zone capacity, as different studies take into account different factors ([1], [2]).

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The potential work zone capacity affecting factors can be divided into five categories:

- Work zone configuration: number of opened/closed lanes, work zone length, grade, speed limit, distance to side barriers, type of side barriers, lane narrowing, traffic signalization and presence of construction site sight blocking equipment.
- Road conditions: area urbanization, presence of on- and off-ramps in work zone area, lane width.
- Road works conditions: intensity of roadworks, working hours (day/night) and work duration (short-term/long-term work zones).
- Weather conditions: rain, snow, visibility etc.
- Traffic conditions: traffic structure and driver population (heavy vehicle percentage, percentage of drivers unfamiliar with the network), presence of ITS for traffic management upstream of the work zone and in the work zone.

Length of the work zone is shown in some studies not to affect the work zone capacity [3]. Some studies also show that the capacity of long-term work zones does not increase with time, eliminating the effect of work zone duration ([4], [5], [6]). In [6], it turns out that the most influencing factors are: heavy vehicle percentage, intensity and type of roadworks, traffic lanes width and presence of a crossover. A crossover can represent a bottleneck, especially under inappropriate traffic management and signalization. The negative impact of work zone on section capacity can be reduced by appropriate signalization and construction site sight blocking equipment. According to [6], those measures can increase work zone capacity by 10 %.

The capacity can be considered as an exact value (deterministic approach) or as a random variable (stochastic approach). Stochastic methods for capacity estimation are more accurate and therefore suitable for assessing traffic flow stability and probability of traffic breakdown under certain conditions. However, less accurate deterministic methods enable some insight in capacity-affecting factors due to their simplicity and input data requirements.

Deterministic models for capacity estimation are usually based on regression analysis of queue discharge rates (QDR) as the capacity estimate (more accurate capacity estimate is the pre-breakdown capacity PBC). However, short-term work zones often cause a breakdown immediately after the work zone setting up, making determining PBC impossible.

Stochastic approach considers capacity as the maximum flow while traffic is still stable, i.e. does not cause a traffic breakdown. If the demand exceeds the capacity, traffic breakdown occurs. However, the measured pre-breakdown flow rates usually differ even in case of several measurements on the same section. This confirms that the breakdown is a random phenomenon, which depends on driver behaviour and specific traffic conditions [7]. The survival analysis is able to take into account these maximum pre-breakdown flows.

The objective of this paper is estimating work zone capacity and capacity-affecting factors based on the data of the recent work zones on Slovenian motorway network. We aim to determine Weibull distribution of capacity, as several researchers [7], [8] as well as HCM [9] claimed it to be a good estimate of the actual capacity distribution.

2 Methodology

This chapter consists of describing the data collection and methodology, used for analysis of short-term and long-term work zones.

2.1 Data collection

We collected data about work zones and motorway sections in the zones (road geometry and grade, presence of tunnels), precipitation and traffic data (traffic volumes, speeds, structure). 151 short-term work zones (less than 24 hours) and 5 long-term work zones (more than 3 weeks) with congestions were analysed. They were classified by types, described in Table 1. All the analysed long-term work zones were on the same motorway section.

Table 1 Types of work zones in analysis

Type	Description	Right lane width	Left lane width	No. of different WZ, duration	No. of congestions
0	No workzone	3,75 m	3,75 m	1, long-term	10
A	Right lane closed	0	3,75 m	89, short-term	103
B	Left lane closed	3,75 m	0	62, short-term	70
C	Narrowed lanes 1	3,25 m	2,75 m	1, long-term	32
D	Narrowed, deviated lanes	3,25 m	2,75 m	1, long-term	54
E	Narrowed lanes 2	3,00 m	2,30 m	1, long-term	33
F	Narrowed lanes, crossover	3,05 m	2,50 m	1, long-term	40
G	Shoulder lane closed	3,75 m	3,75 m	1, long-term	12

2.2 Analysing traffic flow

Traffic flow data from automatic traffic counters were provided by traffic manager DARS, d.d. We analysed the data of traffic counters that were located upstream of the work zone. All the 15-minute intervals were qualified by traffic stability type (stable traffic, period with capacity reached – C and period with traffic jam – QDR) An example is shown in Table 2.

Table 2 An example of traffic counter data and determination of interval types.

Time interval	Avg speed [km/h]	Rel speed change	Flow [PC/h/ln]	Interval type
08.06 13:15	85	5%	1440	stable
08.06 13:30	96	-3%	1456	stable
08.06 13:45	93	-32%	1486	C
08.06 14:00	63	-17%	1298	QDR
08.06 14:15	52	15%	1430	QDR
08.06 14:30	60	33%	1544	QDR
08.06 16:30	80	10%	1510	stable

QDR data were used in both short-term and long-term work zones. For short-term work zones, QDR was used as a capacity measure for a regression model. Least-squares multiple regression analysis was performed for each work zone type. QDR data for long-term work zones was used in a different manner as a better capacity estimate could be obtained (PBC). Therefore, we calculated the average QDR values for long-term work zones to compare QDR and PBC. To estimate PBC, we used survival analysis [7] to obtain capacity cumulative distribution function:

$$F_c(q) = P(c \leq q) \tag{1}$$

where:
$F_c(q)$ - capacity cumulative distribution function,
$P(c \leq q)$ - probability, that traffic demand exceeds capacity.

All the periods with stable traffic and periods with traffic jam were furtherly analysed. The transition between stable and congested traffic was determined based on occurrence of a breakdown regarding multiple criteria; breakdown is followed by a sudden drop in speed (at least 25 %), speed drops below a certain threshold (50 – 75 km/h, depending on the work zone type). The transition between stable traffic and traffic breakdown occurs when speed reaches 70 to 80 km/h on motorway sections without work zones [8]. However, this speed is usually lower in work zones, therefore flow-speed diagrams were used to determine the thresholds (see diagram on Figure 1 for comparison).

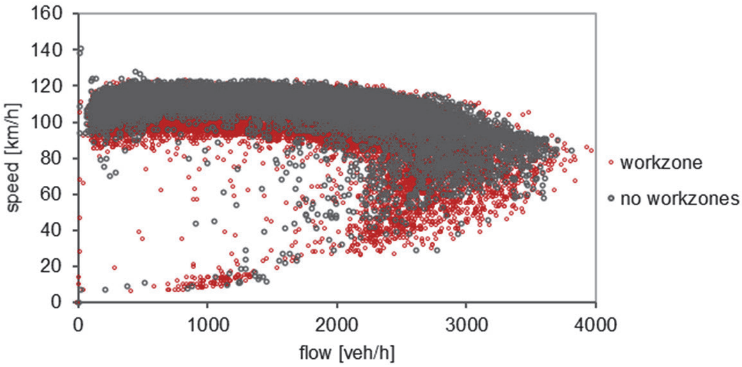

Figure 1 low rates and average speed data in roadwork period (red) and comparable period without work zones (gray)

We will determine Weibull cumulative distribution function of capacity represent the relationship between flow and probability of traffic breakdown:

$$F_C(q) = 1 - e^{-\left(\frac{q}{\beta}\right)^{\alpha}} \tag{2}$$

where:

$F_c(q)$ - cumulative distribution function of capacity,
q - flow (veh/h),
a - Weibull shape parameter,
β - Weibull scale parameter.

Parameters a and β were estimated using maximum likelihood method. The resulting capacity cumulative distribution function estimates the probability of a traffic breakdown for a given flow (demand) value. Therefore, it can be used as an estimate of the effect of a work zone on traffic flow under certain traffic conditions. The capacity is estimated as the flow, under which the traffic remains stable with a certain (high) probability, i.e. the probability of the breakdown for such flow is low (for example 5 % or 15 % according to HCM).
The maximum likelihood method analysis for determining Weibull capacity distribution functions can be used in work zones, where sufficient pre-breakdown data can be collected, i.e., we are not able to determine PBC values for short-term work zones that caused a breakdown immediately after the work zone setting up.

3 Results and discussion

Results of the analyses are hereby summarized. The first part consists of results of the regression analysis of QDR for short-term work zones, whereas Weibull cumulative capacity distribution functions for each work zone type and average QDR values for each long-term work zone are shown in the second part.

3.1 QDR in short-term work zones and factors affecting capacity

Several possible capacity affective factors were studied in the regression analysis. Besides the factors that were identified as statistically significant (light goods vehicle percentage, heavy goods vehicle percentage, grade, high share of drivers unfamiliar with the network – weekend/seasonal traffic), we studied several factors that turned out to be insignificant and were excluded from the model (presence of a shoulder lane, precipitation, time of day (day/night) and tunnel area). The results of the regression analysis are shown in equations (3) and (4) for work zones of type A A (QDR_A in equation (3)) and B (QDR_B in equation (4)).

$$QDR_A = 1596 - 1345 \cdot GV - 17800 \cdot I \cdot HGV - 230 \cdot WE \tag{3}$$

$$QDR_B = 1591 - 1419 \cdot GV - 28000 \cdot I \cdot HGV - 180 \cdot WE \tag{4}$$

where:
GV - percentage of commercial goods vehicles (light goods vehicles and heavy goods vehicles),
I - longitudinal grade,
HGV - percentage of heavy goods vehicles,
WE - weekend or seasonal traffic (drivers unfamiliar with the network).

Longitudinal grade only reduces capacity when heavy goods vehicles are present. Grade shows a bigger effect for work zones of type A than B (17800 compared to 28000), however, both equations don't differ significantly. QDR for work zones type A and B can therefore be determined as an average of both equations (also HCM [9] doesn't differ between both types of work zones). The effect of negative grades was further analysed; the effect is the best described if we set negative grades to 0, unless they are long and steep (≥3 % in ≥ 700 m), where they act similar as a positive grade (represent a capacity reduction in presence of heavy vehicles).
Factors, that turned out to be statistically insignificant may still have some effect on the capacity, however, their effect can be difficult to measure due to insufficient data. For example, daily precipitation data may not be sufficient and may lead to a false result, since most of the short-term work zones were probably implemented during dry hours, even though it may have rained earlier/later in the day.

3.2 Cumulative distribution functions

We performed stochastic analysis for both short- and long-term work zones. Short-term work zones on steep (grade › 2 %, length › 800 m) and non-steep (grade ‹ 2 %) motorway sections were analysed separately, with the same heavy vehicle equivalent factor (E_T=2). This clearly shows the effect of a steep grade (Figure 2), with up to ten times higher probabilities for a breakdown with the same flow. Curve "one ln closed (HCM 2016)" on Figure 2 shows cumulative distribution function of capacity, where HCM [9] heavy vehicle equivalent factors were used (E_T=2) for non-steep and E_T=3 for steep roads). Clearly, the HCM methodology with the homogenous traffic flow concept captures the grade effect well.

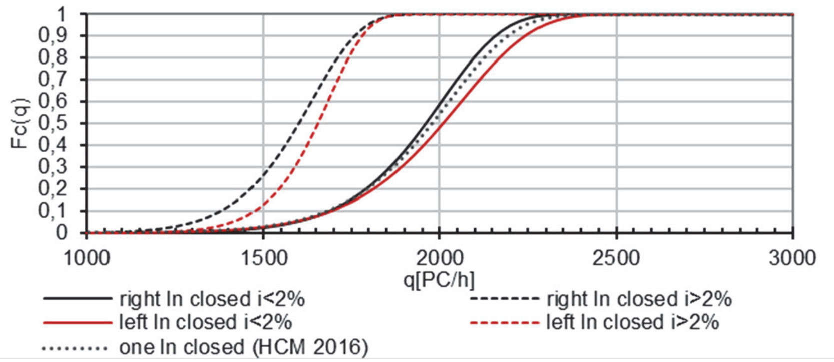

Figure 2 Weibull distribution functions for work zones of types A and B for steep/nonsteep grades (‹2 % and ›2 %).

The results of stochastic analysis of long-term work zones (Figure 3) show, as expected, that capacity is the highest for type G work zone (closed shoulder lane, no lane narrowing). The capacity for type G work zone is approx. 100 100PC/h/lane lower than in the case with no work zone (type 0). Work zones with crossovers (type D and F) and work zone with the narrowest lanes (type E) achieve the lowest capacity.

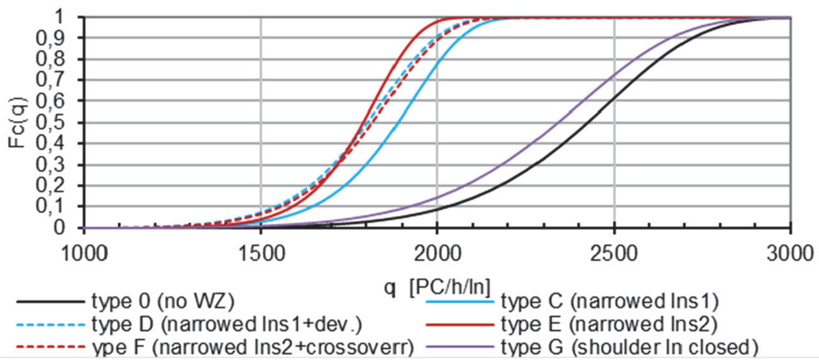

Figure 3 Weibull capacity distribution functions for various long-term work zone types

Table 3 shows summary of results with estimated capacity PBC as 15th percentile of capacity distribution function (C_{15}), whereas QDR is the average value of QDR data. Hereby, values for work zones with similar results are averaged. It should be noted that QDR values in work zones of type G can be underestimated, since breakdowns were rarely reported in this type of work zone and additionally, when they were reported, they commonly lasted less than 15 minutes (time period of data).

Table 3 Estimated work zone capacity C_{15} [PC/h/lane] and capacity drop α_{wz}

Work zone type	A or B One ln closed	G Shoulder ln closed	C Narrowed lns (3,25; 2,75)	E Narrowed lns (3,00; 2,30)	D or F Narrowed lns + crossover
C_{15} [PC/h/lane]	1754	2004	1694	1630	1600
QDR [PC/h/lane]	1594	1572	1600	1474	1445

Results of the analysis are comparable of those proposed in HCM only in case of work zones with no lane narrowing and no crossovers. In case of lane narrowing and crossover, HCM methodology does not provide good fit, as already noted in HCM [9].

4 Conclusions

The presented research shows analysis of 151 short-term work zones of two types (left or right lane closed) and long-term work zones of five types (various lane narrowing and crossovers). QDR values and 15th percentile of capacity distribution function (C_{15}) were calculated for the work zones. The presented analyses show that the work zone capacity is strongly affected by heavy vehicle percentage, grade, high share of drivers unfamiliar with the network – weekend/seasonal traffic, lane narrowing and crossovers. While some other factors might also affect capacity (daylight, precipitation etc.), their effect cannot be measured due to work zone planning/insufficient data (e.g. short-term work zones only in sunny days).
The results are useful as an orientational estimates for capacity values of different types of work zones and helpful for planning work zone configurations in the future. Furthermore, the estimated capacity values can provide an estimate of breakdown probabilities, delays and levels of service of motorway sections under work zones and as such they enable the motorway managers to be better prepared for possible scenarios during roadworks.

References

[1] Strömgren, P., Olstam, J.: A Model for Capacity Reduction at Roadwork Zone, Transport Research Procedia, 15 (2016), pp. 245–256

[2] Weng, J., Meng, Q.: Estimating capacity and traffic delay in work zones: An overview, Transport Research Part C, Emerging Technology, 35 (2013), pp. 34–45

[3] Heaslip, K., Kondyli, A., Arguea, D., Elefteriadou, L., Sullivan, F.: Estimation of Freeway Work Zone Capacity Through Simulation and Field Data, Transportation Research Record, Journal of the Transportation Research Board, 2130 (2009), pp. 16–24

[4] Yeom, C., Hajbabaie, A., Schroeder, B.J., Vaughan, C., Xuan, X., Rouphail, N.M.: Innovative Work Zone Capacity Models from Nationwide Field and Archival Sources, Transportation Research Record: Journal of the Transportation Research Board, 2485 (2015), pp. 51–60

[5] Hajbabaie, A., Yeom, C., Rouphail, N.M., Rasdorf, W.: Freeway Work Zone Free-Flow Speed Prediction from Multi-State Sensor Data, Transportation Research Board 94th Annual Meeting - 2015, Washington, D.C., USA, 11-15 January 2015.

[6] Al-Kaisy, A., Hall, F.: Guidelines for Estimating Capacity at Freeway Reconstruction Zones, Journal of Transportation Engineering, 129 (2003), pp. 572–577

[7] Brilon, W., Geistefeldt, J., Regler, M.: Reliability of Freeway Traffic Flow, Transp. Traffic Theory: A stochastic Concept of Capacity, 16th International Symposium on Transportation and Traffic Theory 2005, College Park, Maryland, July 2005, pp. 125 – 144

[8] Brilon, W.: Implementing the Concept of Reliability for Highway Capacity Aanalysis, Transportation Research Board of the National Academies 2027, pp. 1–8, 2007, doi: 10.3141/2027-01

[9] Transportation Research Board 2016.: Chapter 10: Freeway Facilities Core Methodology (Chapter), Highway capacity Manual 6th ed. Washington D.C., National Academies of Sciences, Engineering, and Medicine, pp. 1-64

UNCERTAINTY ESTIMATION ON ROAD SAFETY ANALYSIS USING BAYESIAN DEEP NEURAL NETWORKS

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Abstract

Deep neural networks have been successfully used in many different areas of traffic engineering, such as crash prediction, intelligent signal optimization and real-time road surface condition monitoring. The benefits of deep neural networks are often uniquely suited to solve certain problems and can offer improvements in performance when compared to traditional methods. In collision prediction, uncertainty estimation is a critical area that can benefit from their application, and accurate information on the reliability of a model's predictions can increase public confidence in those models. Applications of deep neural networks to this problem that consider these effects have not been studied previously. This paper develops a Bayesian deep neural network for crash prediction and examines the reliability of the model based on three key methods: layer-wise greedy unsupervised learning, Bayesian regularization and adapted marginalization. An uncertainty equation for the model is also proposed for this domain for the first time. To test the performance, eight years of car collision data collected from Highway 401, Canada, is used, and three experiments are designed.

Keywords: road safety, road surface condition, crash prediction, neural networks

1 Introduction

The technology of deep neural networks (DNNs) is becoming ubiquitous in both research and industry fields. By stacking layers of linear convolutions with appropriate non-linearities, abstract concepts can be learnt from high-dimensional input alleviating the challenging and time-consuming task of hand-crafting algorithms [1-2]. With this advance, almost in each passing month, new applications on a variety of problems like unsupervised-learning based classification, object detection, forecasting, and reinforcement learning based game playing, continue to be found by using DNNs [3-6]. Moreover, in recent years, a number of deep learning-based methods have also been applied in traffic engineering, such as road surface condition monitoring, traffic sign recognition, autopilot, intelligent traffic optimization, etc [7-10]. In the domain of road safety analysis, deep learning-based models have been shown as effective when compared to the traditional negative binomial and to other techniques such as kernel regression [12]. In this paper, we focus on solving the problem of predicting collisions for highways and achieving high accuracy and fast training speed. We have also explored the relation between the model size, data size and trained model accuracy. While previous researches have done similar works using traditional artificial neural networks

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(ANNs), surprisingly, DNN-based solutions for road safety applications have so far been suggested without any uncertainty management [10-12]. This study aims to fill this gap in the literature by quantifying and estimating the uncertainty of DNN predictions.
Information about the reliability of automated predictions is a key requirement for them to be integrated into alarm systems for the public. No matter whether data is short or abundant, difficult and unseen scenarios are unavoidable. Therefore, DNNs should report, in addition to the decision, an associated estimate of uncertainty. Estimating the uncertainty from a DNN-based prediction on a single testing sample requires a distribution over possible outcomes, for which a Bayesian perspective is best suited. Indeed, Bayesian approaches to uncertainty estimation have been proposed to assess the reliability of model predictions in many domains [13-15]. The integration of the Bayesian methods and DNNs is an active topic of research, but the practical value of the developed methods has yet to be demonstrated in traffic engineering. There are two major types of uncertainty, aleatoric uncertainty and epistemic uncertainty. The first one captures noise inherent in the observations, while the later one accounts for uncertainty in the model which can be explained away given enough data. Aleatoric uncertainty is unavoidable, while epistemic uncertainty is more practical and has been mostly focused on [16-18]. Our previous attempts have focussed on deep learning models and their applications. Specifically, in the domain of road safety analysis, a global DNN model was introduced as an alternative to the traditional regression models for crash modelling [12]. An extensive empirical study was conducted using three real world crash data sets covering six classes of highways as defined by location (urban vs. rural), number of lanes, access control, and region. As an improvement in previous work in this area, this paper proposes a Bayesian deep neural network (BDNN) version that analyses the stability and epistemic uncertainty automatically and gives more reasonable suggestions.
Our focus in this paper is to propose a Bayesian regularization based deep neural network for road safety analysis. The core techniques of this model are, 1) layer-wise greedy unsupervised learning used for data feature learning; 2) Bayesian regularization for knowledge distribution management and uncertainty estimation; and 3) the use of a specifically designed output layer called marginalization which uses the Softmax function predict the probabilities and uncertainties for different scenarios. This paper's structure is organized as follows: Section 2 introduces our methodology, including the improvements on the model and the equation to quantify model's uncertainty; Section 3 demonstrates a case study based on our previous research, more details on implementing the model will be explained; Section 4 summarises the study as well as directions in the future.

2 Methodology

2.1 The training of Bayesian Deep Neural Networks

The structure of proposed model is shown in Figure 1. In the model, three parts are included. The first layer is input layer, it receives sampling values from the original road observed features. Two hidden layers are placed after the input layer and are used for feature extraction. Finally, the output from the hidden layers is marginalised and fed to the output layer. The marginalization layer helps to resort the prediction from exact collision amounts to risk-level based distributions, and uses Softmax to output the result as a percentage for each level.

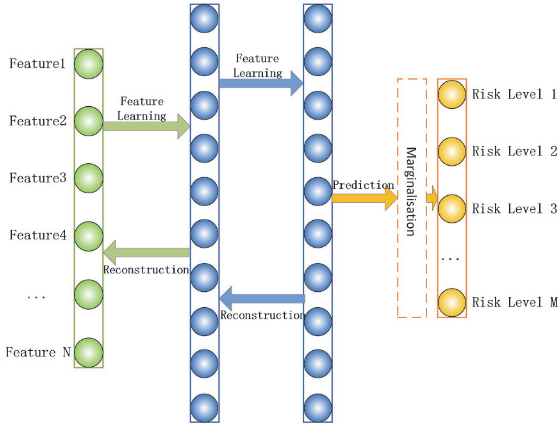

Figure 1 The structure of Bayesian Deep Neural Networks

The training of a BDNN has two steps, the first step is a layer by layer greedy unsupervised learning in which each two neighbouring layers form a restricted Boltzmann machine (RBM). Each RBM has a two-way structure with full connectivity and no specific connections between each same layer. When training each RBM, the hidden layer (the later layer of an RBM) extracts features and information from input to form an order and more abstract representation. Features are transferred in this way until they pass through the last layer and reach the output. The following equations are form the basis of how input is transformed in RBMs and how training takes place:

$$y_j = \varphi_j\left(\sum_j w_{ij}x_i + \sigma \cdot N_j(0,1)\right) \tag{1}$$

$$x_i = \varphi_i\left(\sum_j w_{ij}y_j + \sigma \cdot N_i(0,1)\right) \tag{2}$$

$$\varphi(X) = \theta_L + (\theta_H - \theta_L) \cdot \frac{1}{1+e^{-a_i X}} \tag{3}$$

x_i and y_i is the continuous value of unit i in input layer and j in output layer; w_{ij} is the weight between them, N(0,1) is a Gaussian random variable with mean 0 and variance 1; σ is a constant; ϕ(X) denotes a sigmoid-like function with asymptote of θ_H and θ_L; and a is a variable that controls noise, which means it controls the gradient of the transfer function. When training an RBM, the states are recorded as X^0 (input values, the vector of x_i), Y^0 (hidden layer state using equation 1, the vector of y_j), X^1 (input layer state using equation 2), and Y^1(hidden layer state using equation 1 again). The weights' updating function is

$$W^{(t+1)} = W^{(t)} - \left(X^0 \cdot Y^0 - X^1 \cdot Y^1\right) \tag{4}$$

In the second step, the BDNN is fine-tuned in terms of model structure and learning rate. In doing this, the network will be unfolded, while keeping the trained weights unchanged, to a multilayer back propagation network that uses a gradient descent algorithm to fine-tune the weights. Bayesian regularization is implemented using the following equations:

$$F_W = \alpha E_W + \beta R_W \tag{5}$$

$$E_W = \frac{1}{T}\sum_{t=1}^{T}\left(Y_{target} - Y_{observed}\right)^2 \tag{6}$$

$$R_W = \frac{1}{I \cdot J}\sum_{j=1}^{J}\sum_{i=1}^{I} w_{ij}^2 \tag{7}$$

E_W is the traditional objective function used in a back-propagation network, which calculates the error between target output Y_{target} and observed output $Y_{observed}$ T the testing set; R_W is the Bayesian regularization item that controls the weight between two layers to prevent it from becoming too big in iterations; and α and β are called performance parameters that can be calculated during the iteration. If α›β , then the first part of F_W dominates, which means that the objective of the training is to decrease the training error, if α‹β , the training will focus on decreasing the weights. Finally, I and J are the amount of units in each layer. The weights updating algorithm is,

$$W_k^{t+1} = W_k^t - \Delta W \tag{8}$$

$$\Delta W = \frac{\partial F_W}{\partial W_k} = \alpha \frac{\partial E_W}{\partial W_k} + \beta \frac{\partial R_W}{\partial W_k} \tag{9}$$

$$\frac{\partial E_W}{\partial W_k} = \frac{\partial F_W}{\partial Y_n} \cdot \frac{\partial Y_n}{\partial X_{n-1}} \cdot \frac{\partial X_{n-1}}{\partial Y_{n-2}} \cdots \frac{\partial X_k}{\partial W_k} \tag{10}$$

$$\frac{\partial R_W}{\partial W_k} = \frac{2}{I \cdot J}\sum_{j=1}^{J}\sum_{i=1}^{I} w_{ij} \tag{11}$$

n is the number of layers, so Y_n is the output of the whole network, $Y_n = Y_{observed}$; X_k is the vector of layer k. Therefore, by introducing this regularization term, we can expect that in an iteration, if one weight is affected by its neighbour, which should not be happening, the change of amount of the affection will be reduced using equations (8-11). Furthermore, using this method will ensure weights that do not contribute to the response will be minimized, keeping the network in a sparse connection state.

2.2 Marginalization and Uncertainty Estimation Equations

Marginalization layer is another technique adopted in BDNN to ensure realistic output. To perform this, in training and testing, if the prediction in a category is 0 % while the neighbouring ones are not, the predictions on each category will be re-distributed using equation (12). After training, the marginalization index in equation (13) is applied to estimate how well the distributions are. Finally, when testing the model, testing accuracy can be estimated using equation (14), and the uncertainty with equation (15).

$$P\left(y_i\right) = \partial_1 \cdot P\left(y_{i-1}\right) + \partial_2 \cdot PP\left(y_{i+1}\right) \tag{12}$$

$$MI = f(P\left(y_i\right) > \delta) \tag{13}$$

$$Acc = \frac{1}{M}\sum\left(y_i - \widehat{y}_i\right)^2 \tag{14}$$

$$Mu = \frac{1}{M} \cdot \frac{MI - T \cdot Acc}{R \cdot T} \tag{15}$$

$P(y_i)$ is the probability of car collision on risk level i; ∂_1 and ∂_2 are the percentage probabilities taken from the specific categories; $f(P(y_i) > \delta)$ means the total case amount that $P(y_i)$ is over δ; T is the total number of observations and R is the total output neurons; MI is the marginalization index; M is the running times of the model; y_i the ground truth; $\widehat{y}_i$ is the integrated confidence; Mu is the marginalized model uncertainty and Acc is the marginalized model accuracy.

Traditionally, Acc (in some papers MAE or RMSE are used) is the only standard to evaluate a model and the higher Acc the better. However, this can obscure the true accuracy of a model in some circumstances. For example, in BDNN, the output is the probabilities for each risk level (explained in next section) and thus if Acc is the only evaluation used then the model will try to even the prediction on each category, which means that during the training the model will try to make a tie between all the categories, if so the truth ground will definitely fall into a category and it will be 100 % correct. This prediction, however, is totally meaningless because it cannot provide any alarms or suggestions to the public and department for management. Mu is a compliment to Acc because it shows how much confidence the model has over a certain observation and how stable the model's prediction is. The lower Mu the better because a lower Mu means the distribution on the prediction is more confident. Thus, in this research, we propose to utilise both Acc and Mu for model evaluation.

3 Experiment

3.1 Experimental Design

This case study is based on historical collisions and related data from Highway 401 in Ontario, Canada. This highway, which plays a crucial role for the region's socio-economic development, is one of the busiest highways in North America. The total length of the highway is 817.9 km of which approximately 800 km is used in this study. According to 2008's traffic volume data, the annual average daily traffic (AADT) ranges from 14,500 to 442,900 indicating comparatively a very busy road corridor. Traffic count data consists of AADT and average annual commercial vehicle counts for the period 2000 - 2008. As each observation records the LHRS and offset information, traffic counts can be spatially located using a linear referencing GIS tool. The whole highway is divided into 418 homogeneous sections (HS) with length ranging from 0.2 km to 12.7 km. Each HS is then assigned the nearest traffic observation. Finally, the processed crash and traffic data are integrated into a single dataset with HS and year as the mapping fields, resulting in a total of 3762 records. The selected input features used in the study are exposure, AADT, left of shield, median width, right of shield, and curve deflection.

In this case study, the model's performance is estimated based on accuracy (Acc), model uncertainty (Mu) and marginalization index (MI), as defined in previous equations. Three experiments are designed below, to testify the performance of the model. We use a network structure of 6-10-10-4 (6 units for the input layer - 10 units for the hidden layer 1 - 10 units for the hidden layer and 2 - 4 units for the output layer) for the model. This structure is selected on the basis of our previous work, which finds that when the training dataset is around 3,000 two hidden layers with no more than 10 are the most effective. Also, different learning rates varying from 0.5 to 3 are tried, with the learning rate of 1.0 being the most effective. In terms of training, we use a total of 50 unsupervised iterations and 1000 fine-tuning iterations to ensure model convergence. Six input neurons are used based on six key features: exposure, AADT, left of shield, median width, right of shield, and curve deflection. Four output neurons are provided corresponding to four risk levels designed based on the annual collision rate. Risk level 1 covers sections with less than 3 collisions per year, risk level 2 is covers sections with 4 to 10 collisions per year, risk level 3 covers sections with 11 to 20 collisions per year and risk level 4 covers sections with over 21 collisions each year. The data distribution is shown in figure 2. In calculating MI, we set $\partial_1 = \partial_2 = 2$.

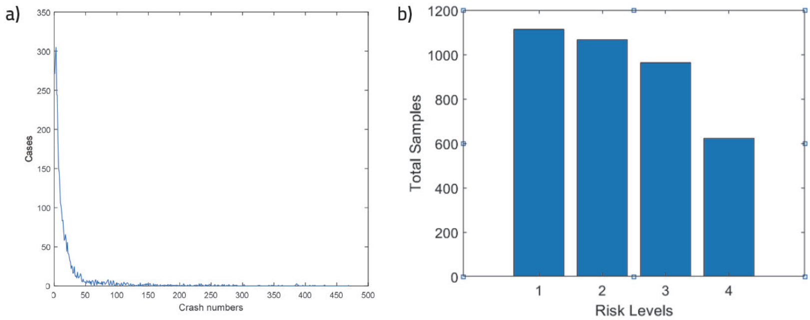

Figure 2 a) crash amount on each segmentation observed each year, b) the re-sorted risk levels based on (a)

3.2 Uncertainty Test with Pre-set Marginalization

A total eight years of collision data with 3762 sets are collected. The first 6 years with 2926 sets are used for model training and the rest 836 sets are for model testing. The marginalization parameters (∂_1, ∂_2) are set to be 2. Other model parameters have been given in last section. The results are shown in Table 1, Table 2 and Figure 4.

Table 1 shows the predictions on ten selected samples. When comparing the predicting risk with the exact risk level (or collision number) using traditional models, BDNN outputs the probabilities on each level and evens the distributions more realistically, thanks to marginalization and Softmax. For some cases (2 and 3), BDNN outputs are given with a very high confidence level, while for most other samples, it is more conservative and provides more possibilities for the risk level. This characteristic allows the model to provide a complete picture that is clearer, such as the situation on cases 5 and 6, providing better results than traditional models that often fail to capture these situations. In table 2, the advances of using BDNN is clearer. In our previous work, the R-DBN-based proposed deep learning model had the best performance when compared to other popular methods. However, when predicting risk levels, the model only achieves an accuracy level of 61 % because the uncertainty is too high. It is outperformed significantly by BDNN, which has a higher accuracy of 83.35 % and an uncertainty value (Mu) only 0.2344, which is a clear sign of better stability. Full comparison results are shown in Table 2, along with results for BDNN models with different confidence ranges, and without marginalisation. The finding proves that the proposed model receives lower prediction accuracy, but higher stability if it is restricted, which is quite reasonable. Figure 3 shows the results of table 1 and 2. The blue lines in both sub-figures are the ground truth. Red line in (a) represents the exact predictions using R-DBN, and the green band in (b) shows the confidence area.

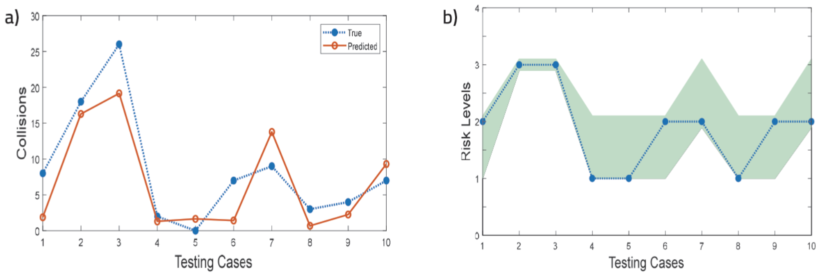

Figure 3 Comparisons on randomly selected testing samples.

Table 1 Predictions on Randomly Selected Testing Samples

Testing Cases		**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
Ground truth on risk levels		**2**	**3**	**3**	**1**	**1**	**2**	**2**	**1**	**2**	**2**
Predicted risk levels (%)	1	13.5	0.0	0.0	99.6	9.4	90.6	0.0	67.4	45.0	0.0
	2	86.5	0.0	0.0	0.4	90.6	9.4	99.8	32.6	55.0	99.3
	3	0.0	100.0	100.0	0.0	0.0	0.0	0.2	0.0	0.0	0.7
	4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Table 2 Results comparison using different models

Methods	**R-DBN**	**BDNN without marginalization**	**BDNN (99% confidence)**	**BDNN (95% confidence)**
Testing Acc	61.00%	79.07%	83.35%	73.74%
Model Uncertainty	0.6746	/	0.2344	0.1619
MI	2255	/	1459	1158

3.3 Uncertainty Test with Growing Training Data

Further experiments were also conducted to test the uncertainty of the model as the training data size changes. The following bootstrapping process is followed: 1) Split the given dataset into two subsets: a training set and a testing set. The training set includes seven of data (2000 - 2006) while the testing set includes the remaining two years of data (2007- 2008). 2) A subset of data at a specific amount of years is drawn from the training data set. 3) The subset of data of a specific size is then used to calibrate or train the candidate models, which is subsequently used to predict the collisions at the testing data set. The Acc and Mu are then calculated. 4) Repeat Step 2) to 3) for 10 times. After done, calculate the average, minimum and maximum. Results are shown in Table 3 and Figure 4.

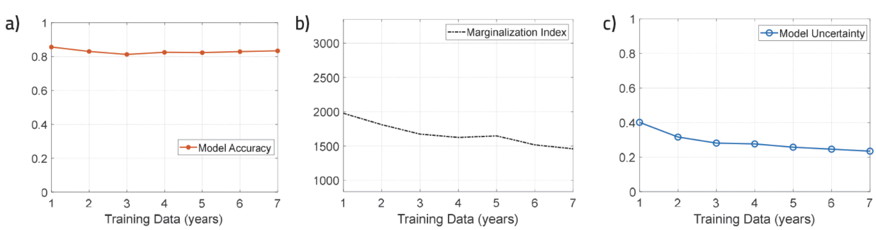

Figure 4 Effect of data size on the model

The training and testing results for the BDNN model are shown in Figure 5. When the size of the training data increases, the testing MI and Mu of the BDNN model both decrease, which is a sign of model improvement. It should be noted that the testing MI and Mu both showed an initial quick drop but then begin to level off, suggesting that the model is reaching its limit. This also suggests that BDNN learns more information using fewer data as with our previous model, the testing MAE and RMSE kept dropping dramatically as more data was added. Furthermore, figure 5(a) shows that the testing accuracy remains between 80-90 % all the time, which further suggests that the model has achieved a high level of stability.

Table 3 Testing results with increased data size

Training data (years)	1	2	3	4	5	6	7
Testing Accuracy	0.8557	0.83	0.8125	0.8249	0.8227	0.8285	0.8335
Model uncertainty	0.4012	0.3157	0.2811	0.2762	0.2569	0.2453	0.2344
Marginalization Index	1977	1811	1674	1625	1648	1517	1459

3.4 Uncertainty Test with Unknown Observation

This experiment is designed to testify the generalization and uncertainty of the BDNN. We hope once the model is well trained, it can not only perform well on the existing dataset, but also learn and act well on new or limited databases with unpredictable scenarios, and that it make continued predictions as the new information is updated, even if it has not been observed previously in its past training data sets. The following experimental process is followed. 1) Some specific datasets are carefully picked out for model training. All the datasets are in the range of specific risk levels, yet the exact collision amounts are not observed before. 2) Train a BDNN using the new designed database as in experiment 1, then test the trained model on unseen observations. 3) Repeat the process for 10 times.
In the database designed, all 305 cases that have 2 collisions (risk level 1), 102 cases with 10 collisions (risk level 2) and 27 observations that are more than 300 collisions are removed and reserved for model testing (risk level 4). Thus, during the training process, only samples with less than 300 collisions will be learned by the model. The result is shown in table 4 and 5.

Table 4 Model comparison with unseen observations

Methods	Acc	MI	Mu
DBN	14.29%	1645	0.6976
BDNN	70.83%	591	0.1632

Table 5 Model predictions

Ground truth		1	1	2	2	4	4	4	4	4
DBN predictions		2	2	3	4	3	2	3	2	2
BDNN Predictions (%)	1	13.52	99.36	0.00	0.00	0.17	0.00	0.01	0.00	0.00
	2	86.48	0.64	0.00	0.00	50.00	1.00	50.00	1.00	25.00
	3	0.00	0.00	1.00	1.00	49.83	0.00	49.99	0.00	50.00
	4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	25.00

Table 4 compares the results of different models and training methods. When compared to our previous work, substantial improvements in the accuracy, marginalization index and model uncertainty are achieved in for unseen scenarios. When using BDNN, the testing accuracy stays as high as 70.83 % in the total new dataset. Model uncertainty is only 0.1632 for the BDNN model compared to 0.6976 using DBN and 0.2344 in experiment 1. In table 5, 9 randomly selected samples are shown, with 2 samples chosen from risk level 1, 2 from risk level 2 and 5 from level 4. All the predictions by DBN are incorrect, while the predictions by BDNN are much more reasonable, especially on the five examples with risk level 4 where all the samples have over 300 collisions and not in the range of training data. Despite this, the BDNN model shows strong performance and adaptability and can still make reasonable predictions.

4 Conclusion

In this paper, we proposed an improved deep neural network approach for modelling road collisions with low uncertainty and high stability. The research has made two main contributions. First, a BDNN model is introduced as an alternative to traditional machine learning for predicting expected collision risk levels of highways. This model is able to receive and process continuous real-world data and output probabilities on each level using the techniques of Bayesian regularization, marginalization and Softmax. Second, an uncertainty analysis method is proposed, and three experiments are designed to test the model's accuracy, marginalization index and model uncertainty. Finally, results showed that the proposed method is much more stable than our previous model and gives more reasonable suggestions to the public in real-world applications.

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References

[1] Voosen, P.: How AI detectives are cracking open the black box of deep learning,Science, 357 (2017) 346, pp. 22-28

[2] LeCun, Y., Bengio,Y., Hinton, G.: Deep learning, Nature, 521 (2015) 7553, pp. 436-444

[3] Szegedy, C., Vanhoucke, V., Ioffe, S., Shlens, J., Wojna, Z.: Rethinking the inception architecture for computer vision, Proceedings of the IEEE Conference on Computer Vision and Pattern, pp. 2818-2826, 2016.

[4] Chollet, F: Xception: Deep Learning with Depthwise Separable Convolutions, Proceedings of the IEEE conference on computer vision and pattern recognition, pp. 1800-1807, 2016.

[5] He, K., Zhang, X., Ren, S.: Identity Mappings in Deep Residual Networks, European Conference on Computer Vision, pp. 630-645, 2016.

[6] Chen-Mccaig, Z., Hoseinnezhad, R., Bab-Hadiashar, A.: Convolutional neural networks for texture recognition using transfer learning, International Conference on Control, Automation and Infor Recognition mation Sciences, pp. 187-192, 2017.

[7] Zhang, Z., He, Q., Gao, J.: A deep learning approach for detecting traffic accidents from social media data, Transportation Research Part C: Emerging Technologies, 86 (2018), pp. 580-596

[8] Lv, Y., Duan, Y., Kang, W.: Traffic flow prediction with big data: A deep learning approach, IEEE Transportation, Intelligent Transportation Systems, 16 (2015) 2, pp. 865-873

[9] Pan, G., Fu, L., Yu, R. Muresan, M.: Winter Road Surface Condition Recognition Using a Pre-trained Deep Convolutional Neural Network, Transportation Research Board 97th Annual Meeting, Washington, D.C, 2018.

[10] Pan, G., Fu, L., Thakali, L., Muresan, M., Yu, M.: An Improved Deep Belief Network Model for Road Safety Analyses, The Transportation Research Board Annual Meeting, Washington, D.C, 2018.

[11] Muresan, M., Fu, L., Pan, G.: Adaptive Traffic Signal Control with Deep Reinforcement Learning - An Exploratory Investigation, The Transportation Research Board Annual Meeting, Washington, D.C, 2018.

[12] Pan, G., Fu, L., Thakali, L.: Development of a Global Road Safety Performance Function Using Deep Neural Networks, International Journal of Transportation Science & Technology, 6 (2017) 3, pp. 159-173

[13] Yarin, G.: Uncertainty in deep learning, Discussion, PhD thesis, University of Cambridge, 2016.

[14] Kendall, A., Roberto C.: Modelling uncertainty in deep learning for camera relocalization, IEEE international conference on Robotics and Automation (ICRA), pp. 4762-4769, 2016.

[15] Kendall, A., Yarin G.: What uncertainties do we need in Bayesian deep learning for computer vision?, Neural information processing systems, pp. 5574-5584, 2017.

[16] Lakshminarayanan, B., Alexander, P., Charles, B.: Simple and scalable predictive uncertainty estimation using deep ensembles, Neural Information Processing Systems, pp. 6402-6413, 2017.

[17] Gurevich, P., Hannes, S.: Pairing an arbitrary regressor with an artificial neural network estimating aleatoric uncertainty, Neurocomputing, 350 (2019), pp. 291-306

[18] Zhu, L., Nikolay, L.: Deep and confident prediction for time series at uber, ,2017 IEEE International Conference on Data Mining Workshops (ICDMW), pp. 103-110, 2017.

[19] Leibig, C., Vaneeda, A., Murat, S.A., Philipp, B., Siegfried, W.: Leveraging uncertainty information from deep neural networks for disease detection, Scientific reports, 1 (2017), pp. 8-16

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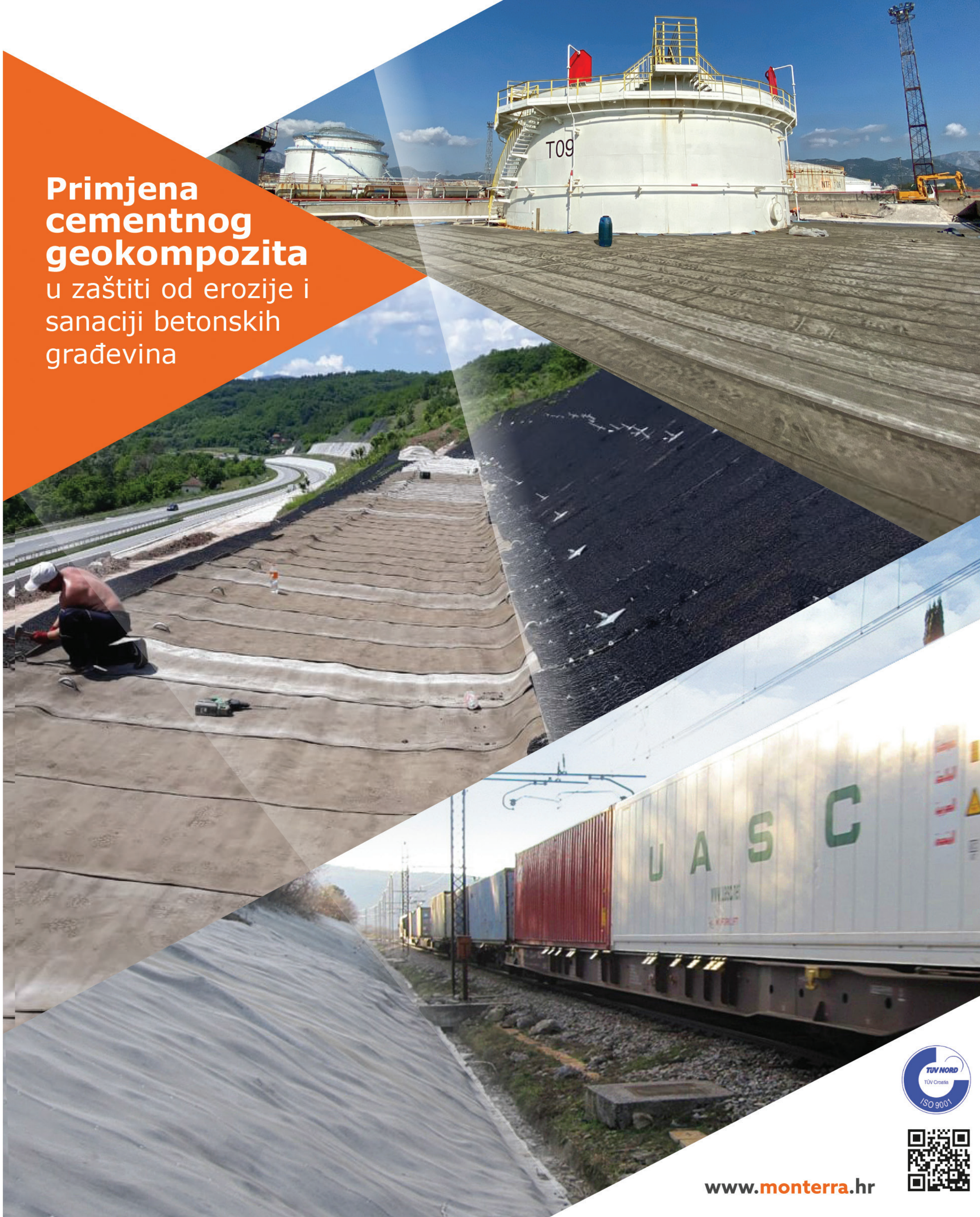

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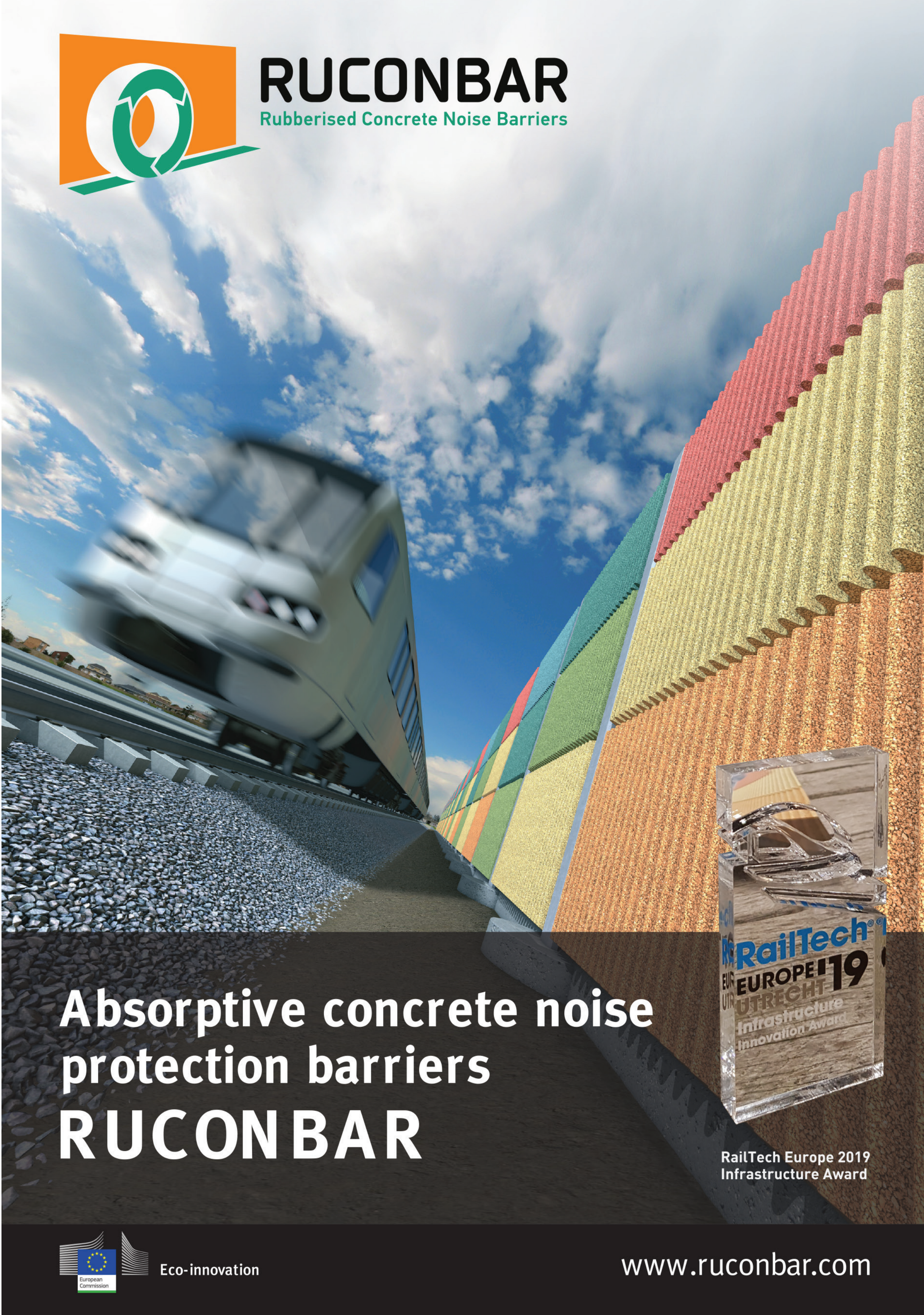
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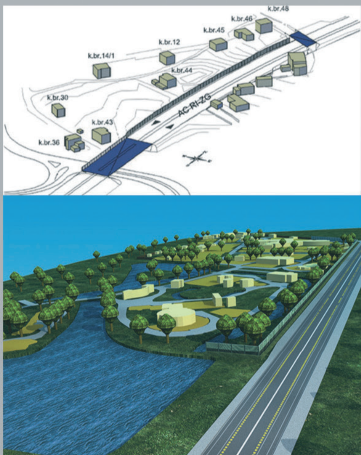

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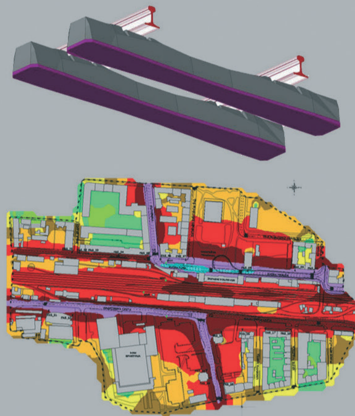

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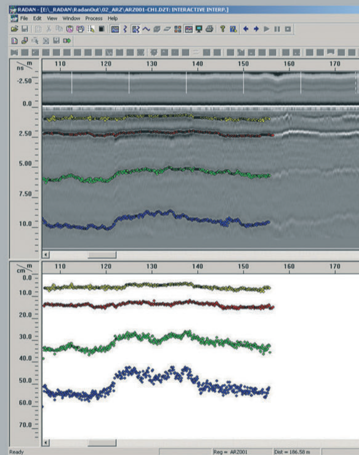

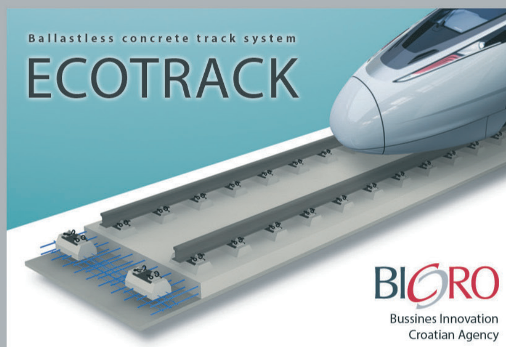

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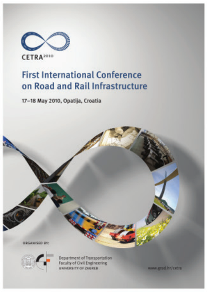

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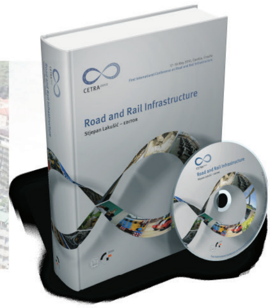

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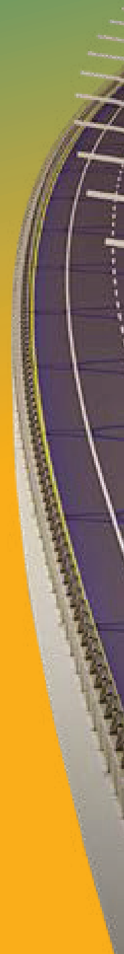

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