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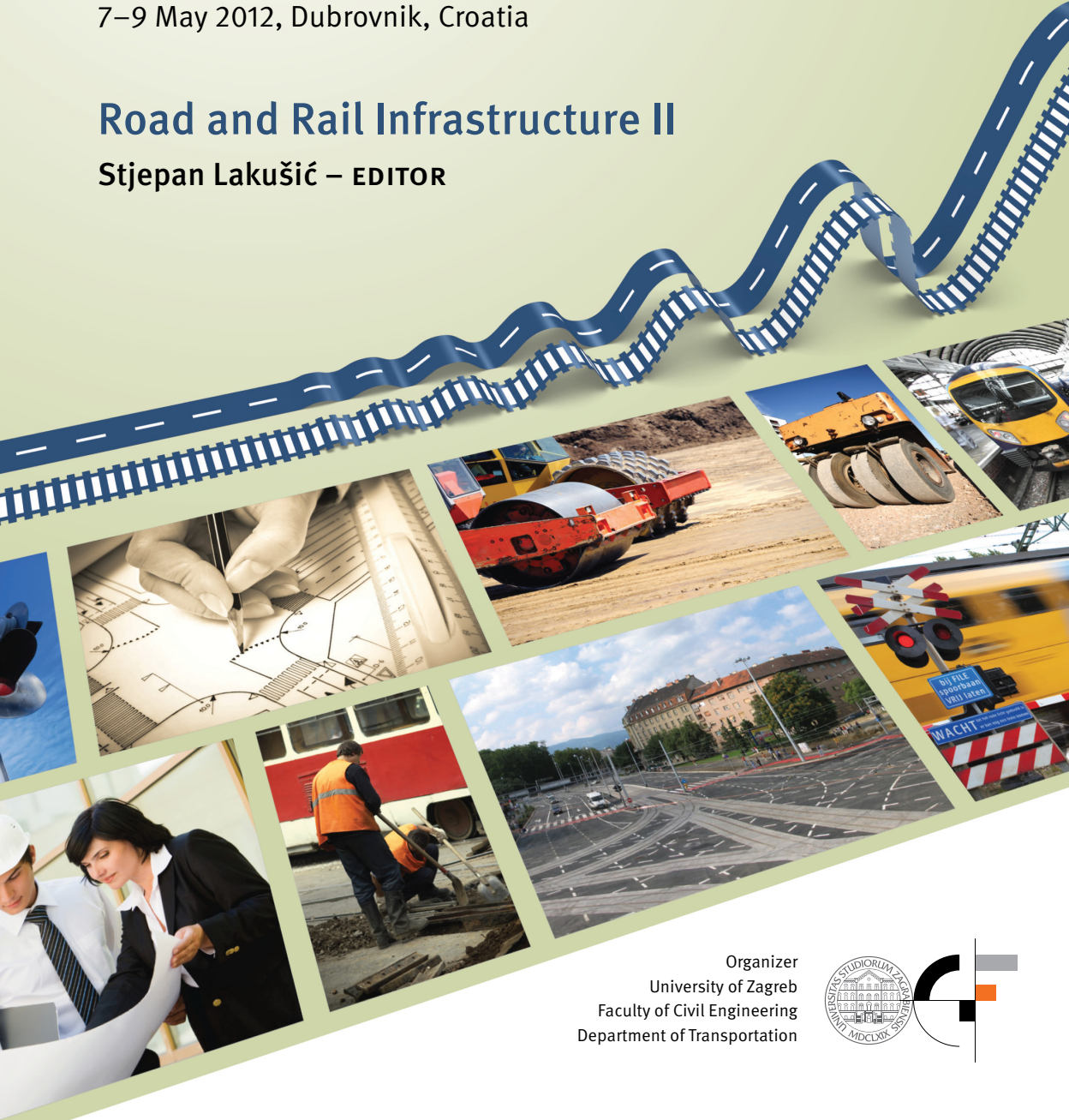**2nd International Conference on Road and Rail Infrastructure**
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Road and Rail Infrastructure II

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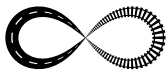
A COMPARISON OF 2D AND 3D NUMERICAL SIMULATION FOR TUNNEL EXCAVATION ACCOMPANIED BY MEASUREMENT RESULTS

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Abstract

Tunnel excavation leads to the redistribution of stress in rock mass. The primary support is not the structure that is to assume the load of the rock mass, but instead in interaction with the rock mass it represents part of the structural system. Deformation of the mass, caused by the progress of excavation, alters its primary stress state causing stress in the primary support. This compression, amongst other factors, depends on the stiffness of the mass and support work, including the shape and size of the tunnel cross–section. The available computer methods in geotechnics, linear and non–liner constitutional equations, as well as 2D and 3D models accompanied by the use of developed computer programs, are considerably more developed and sophisticated than what is posed by utilising a familiarity and description of geotechnical properties of materials that enter such a model. The calculation results are exceptionally dependent on the chosen model and the values of chosen geotechnical parameters. Intensive measuring during tunnel works in carbonate rock mass of a Croatian karst type have shown that the measured deformations are significantly greater than that obtained in calculations which utilise stiffness parameters gained from existing relationships with rock mass classifications and that the measured forms of deformation along the depth are significantly different than those calculated or anticipated in the design. Such measurements have allowed for the development of a new approach to determining the carbonate mass deformation modulus in Croatian karst.
The paper presents a 2D and 3D numerical simulation for the Bobova Tunnel excavation on the D404 national roadway and their comparison with measurement results. In both simulations, a new approach is used to determine carbonate mass stiffness in Croatian karst which provides more reliable deformation forecasting of geotechnical structures.

Keywords: tunnel construction, numerical modelling, karst, rock mass stiffness, monitoring

1 Introduction

The design engineer of a tunnel, a typical structure in the domain of underground construction, has no choice when it comes to construction materials considering that the fundamental structural material is soil or rock in which the tunnel is constructed. Furthermore, when designing tunnel structures, loads are not important but the forces that occur due to the redistribution of the primary state of stress during excavation. The main tunnel characteristic is that it's a linear structure and, for the purpose of designing, in most cases it is irrational or mostly impossible to implement such a scope of investigation works in order to obtain reliable parameters for design purposes. This reason lies in the fact that the physical–mechanical characteristics of soil and rock may significantly vary along the tunnel route. The largest number of road tunnels in Croatia has been constructed in accordance to the recommendations

of the New Austrian Tunnel Methods (NATM) according to which the procedure for constructing a tunnel is continually adapted to the advancement of combining calculation methods, empirical manner of designing and direct interpretation of geotechnical measurements and observations. NATM, which represents a unique philosophy in tunnel construction, is based on scientifically validated and in practice verified ideas and principles, by mobilising rock mass capacities to achieve optimal safety and cost–effectiveness. It's due to the idea of mobilising the rock mass during tunnel excavation, that the primary support of the tunnel does not need to assume the rock mass load, but in interaction with it forms part of the structural system. One of the four fundamental principles of NATM is lies in in–situ measurements during works, thereby checking deformations and the process of stress redistribution, all for the purpose of fulfilling sought safety levels. Such measurements combined with numerical back–analysis contribute to the development of knowledge gained in rock mass behaviour and determination of its physical–mechanical parameters and linking them to the results of rock mass classification.

2 Numerical methods in underground construction

2.1 Numerical back–analysis

Numerical analysis, whereby material parameters change in accordance to the geotechnical measurement results, is called in professional jargon numerical back–analysis. The principle of back–analysis is that for presumed material characteristics, the stress–strain state is calculated and subsequently the state is compared to measured field results. Since in the majority of cases for presumed parameters the calculated results do not conform to measurements, it becomes necessary to alter the material characteristics until the calculated and measured values coincide with engineering precision [1]. In comparison to investigation works, numerical methods in tunnel construction are very developed and sophisticated, representing a reliable tool for determining stress–strain states which the tunnel will experience during and after its construction. However, a familiar saying is numerical modelling is 'garbage in–garbage out', thereby suggesting the fact that the utilisation of unreliable soil or rock parameters in numerical modelling results in unreliable analysis results.

2.2 The finite elements method

Considering that the problems modelled in the domain of underground construction (and in other engineering sciences) are too demanding to acquire an analytical solution, it becomes necessary to use numerical methods. Computer programs for numerical modelling of underground construction are most often based on the finite elements method or the finite differences method. According to the finite elements method, the continuum that possesses an infinite level of freedom is discretised into a particular number of mutually related (volume) elements. Each element contains a specific number of nodes, where each node has a particular, final number of levels of freedom. Such discretisation provides a particular solution for each element. Therefore, instead of solving problems for a whole volume in a single operation, we formulate a series of equations for each finite element, and mutually combine them with the aim of obtaining a solution for the whole volume. In brief, the solution for structural problems most often relates to seeking a displacement in each node and stress within each element by modelling particular geometry with set material parameters and loads [2]. A computer program, based on the finite elements model and widely used in design practice for underground engineering, is the computer package PLAXIS, with its PLAXIS 2D and PLAXIS 3D modules used in this paper in order to present complex stress–strain relations in rock mass during underground construction works.

3 Croatian karst rock stiffness model

Using continual and intensive measurements during numerous geotechnical operations in Croatian carbonate rock karst has shown that the measured deformations are significantly greater than deformations obtained using numerical calculations in which used stiffness parameters are obtained from existing relations with rock mass classifications. Furthermore, the measured deformation forms along the depth are significantly different from those calculated. Therefore, a new approach has been developed in determining the stiffness of carbonate rock in Croatian karst [3,4] which has shown that the parameters affecting stiffness is the geological strength index (GSI), the dispersion velocity of longitudinal wages (V_p) and the rock mass deformation index (ID_m), where the stiffness is equal to the multiple of the rock mass deformation index, the square of the geological strength index and the square of the dispersion velocity of longitudinal waves. The stated manner of determining stiffness is given using the eqn (1).

$$E_m = ID_m \cdot GSI^2 \cdot V_p^2 \tag{1}$$

where E_m is in (GPa), GSI in (%) and V_p in (km/s).

The rock mass deformation index (ID_m) for carbonate rocks in Croatian karst is equal to the rock mass quality index (IQs) determined by allocating rock mass into one of the proposed models and weathering zones, whereas the geological strength index (GSI), in completed adapted to the geological engineering properties of Croatian karst [5]. The dispersion velocity of longitudinal waves along the depth, can be successfully gained using seismic geophysical methods for seismic refraction, seismic reflection and hybrid seismic method as a combination of refraction and reflection.

4 2D and 3D numerical simulations with measurement results using the Bobova Tunnel example

4.1 Description of the Bobova Tunnel

Bobova Tunnel is located on the D404 national roadway and was penetrated in 2005. It passed under the whole Vežica–Sušak town area in Rijeka. It's 210 m long and is constructed as a three–lane tunnel along its whole length. The maximum height of the overburden above the tunnel pipes is 15 m. For the requirements of tunnel design, geological engineering research and testing was conducted previously, as well as testing samples in a laboratory, where it was shown that the tunnel route area is located mostly in Upper Cretaceous deposits, and around the entrance section in older Upper Cretaceous deposits. These deposits represent rudist limestone whose characteristic process is karstification, and therefore it becomes possible for the purpose of numerical modelling to use an approach for determining deformability modulus as described in chapter 3. When taking into account that the stated model uses longitudinal wave velocity as a parameter to calculate stiffness, figure 1 provides the longitudinal geophysical profile for the Bobova Tunnel with an illustration of the longitudinal wave velocity.
Furthermore, geomechanical classification of rock has determined that tunnel along its route is located in category 3, 4 and 5 rock mass (RMR = 19-43), and therefore constructed in two excavation phases in compliance with the guidelines from the 'New Austrian Tunnel Methods'. The selected primary support assembly for the tunnel includes two types of anchors: adhesive rock bolt 25 mm in diameter, 6 m long (besides being in places on tunnel walls where it is then 3 m in length) which is located at intervals of 2 m, and the self–penetrating injection anchor IBO 32, 6 m long at intervals of 2 m. Furthermore, for ensuring the tunnel opening, reinforcing steel

mesh type Q-257 were used, including steel trusses Pantex type 95/20/130 and 130/20/30 at intervals of 1.5 m, and shotcrete with a thickness of 0.15 to 0.25 m [6].
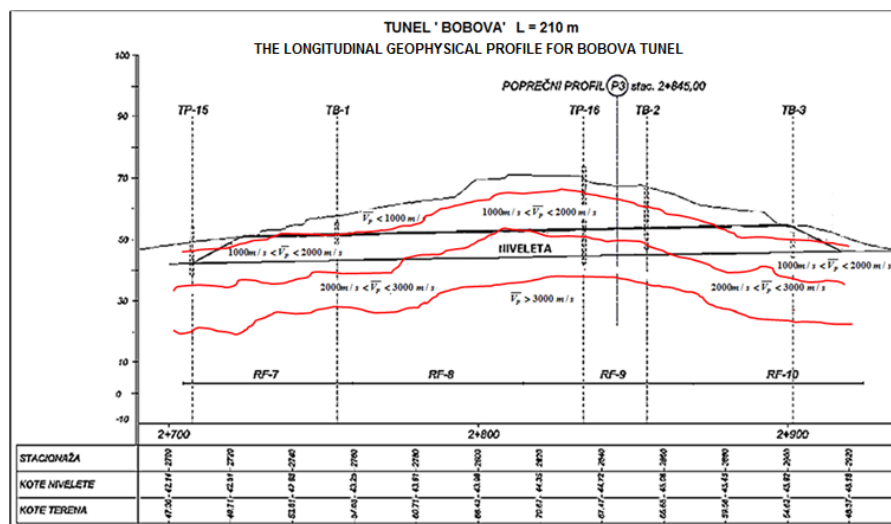

Figure 1 The longitudinal geophysical profile for the Bobova Tunnel

4.2 Two–dimensional numerical model of the Bobova Tunnel

The two–dimensional stress–strain analysis of the Bobova Tunnel was conducted using the PLAXIS 2D computer program. Considering that it is not possible to show tunnel progression using this model, excavation was carried out in two steps. In the first phase, part of the rock was excavated in calottes up to half–way on the tunnel walls, and subsequently the support system elements were assembled. In the next step, the remaining section of the tunnel was excavated, and other elements of the support system assembly were installed. The analysed model with the visible network of finite triangular elements, following the second step, is shown on figure 2. The model comprises of 11 174 elements and 90 149 nodes.
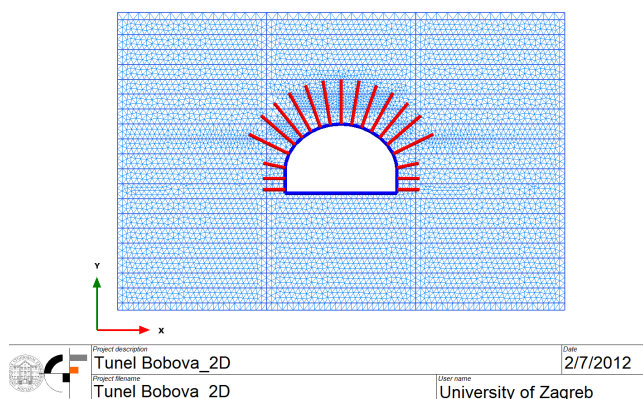

Figure 2 Numerical model of the Bobova Tunnel used in 2D analysis

4.3 Three–dimensional numerical model of the Bobova Tunnel

The three–dimensional stress–strain analysis of the Bobova Tunnel was carried out using the PLAXIS 3D computer program. In comparison to the two–dimensional model, the three–dimensional stress–strain analysis can be taken into consideration for incremental excavation progress. Consequently, the number of calculation phases increases, but at the same time the actual state of tunnel construction is simulated, whereby it is then possible to observe displacements in rock mass during tunnel construction. Incremental progress represents the length of tunnel excavation whereby the rock mass remains unsupported. Therefore, following excavation of rock mass in a particular length (length of incremental progress), the support work for the rock mass is carried out, and only then is it possible to continue with the following excavation phase. Incremental progress for Bobova Tunnel three–dimensional model is equal to two metres and conforms to the interval of anchors in the direction of tunnel progression. When taking into consideration what has been said, tunnel excavation was simulated in 38 increments or steps, whereas the model comprised of a total of 140 369 15–node wedge elements, and 202 985 nodes. In Figure 3, the 18th step in tunnel excavation is shown with the visible mesh of finite elements. Since the tunnel is axial symmetric, only one–half of the tunnel is modelled.
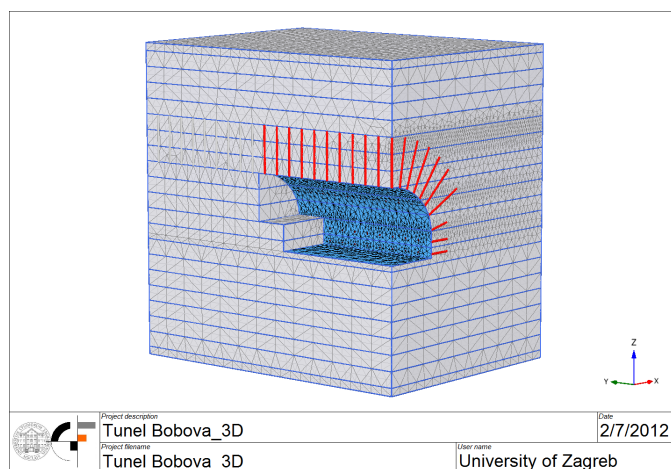

Figure 3   A numerical model of the Bobova Tunnel used in 3D analysis, 18th step

4.4 A comparison of measured displacements in the Bobova Tunnel accompanied with numerical simulations

During construction of the Bobova Tunnel, continual measurements were taken of the surface terrain using vertical inclinometers with the aim of acquiring horizontal displacements and measurements using sliding deformeters with the aim of acquiring vertical deformations. The measuring profile with positions for the installed inclinometers and deformeters is shown in Figure 4.
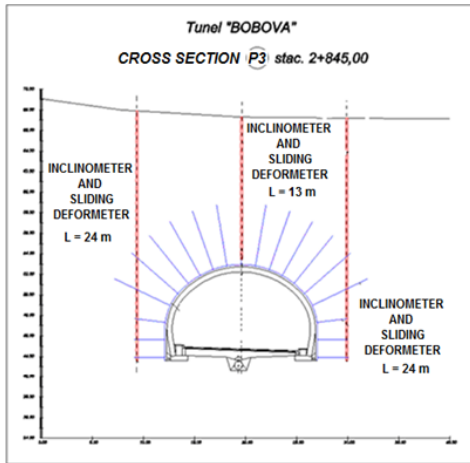
Figure 4 Measuring profile of the Bobova Tunnel with designated positions of the installed inclinometers and deformeters

Figure 5a shows the comparison of horizontal displacements (in millimetres) gained using numerical simulations (two–dimensional and three–dimensional) with inclinometer measurement results, while comparison of the vertical deformations (permille) acquired using numerical simulations (two–dimensional and three–dimensional) with sliding deformeter results is shown in Figure 5b.

–▲–2D model horizontal displacements perpendicular to tunnel axis

–●–3D model horizontal displacements perpendicular to tunnel axis

–■–MEASUREMENTS horizontal displacements perpendicular to tunnel axis

a

–●–3D model vertical deformations

–▲–2D model vertical deformation
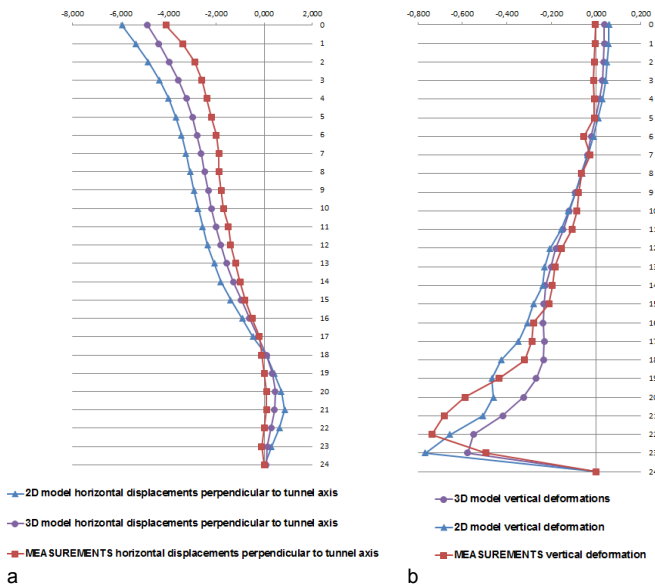
–■–MEASUREMENTS vertical deformation

b

Figure 5 Comparison of horizontal displacements perpendicular to tunnel axis (a) and vertical deformations (b) using numerical simulations and in–situ measurements

Using non–linear ground stiffness model, described in chapter 2, the numerical stress–strain analysis of the Bobova Tunnel showed that the acquired horizontal and vertical deformations in rock mass, occurring due to excavation, have an approximately identical trend along the depth such as has been obtained from measurement results. In regards to the size of the

deformations themselves, it is evident that the three–dimensional simulations acquire deformations which are closer according to the size of deformations obtained from measurements, than those deformations obtained using two–dimensional analysis. The stated is valid for horizontal and vertical deformations.

5   Conclusion

Implementation of numerical back–analysis for tunnel excavation in rock, described in this paper, presents the conclusion that numerical simulations, two–dimensional and three–dimensional, along with the use of the non–linear models for rock stiffness can obtain deformations that based on trends conform to measured deformations. This especially relates to three–dimensional simulations which, besides deformation trends, provide approximate equal values also for the deformation size in comparison to the measured values. The determined differences exist primarily for the reason that numerical simulations take rock mass as the continuum, while in reality it is represented as a discontinuum containing fissure systems that are intersecting. Furthermore, an essential advantage of the three–dimensional model, besides the fact that it provides us with insight into the spatial state of stress, it also provides the ability to analyse deformation in each tunnel construction phase. This analysis allows forecasting rock mass deformation before a particular section has been excavated, and accordingly the necessary measures may be implemented. On the basis of what has been said, the use of three–dimensional numerical simulation is proposed when analysing the state of rock mass deformation during tunnel excavation.

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