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COMPARATIVE DESIGN AND PERFORMANCE EVALUATION OF PAVEMENT DESIGN METHODS FOR LOW VOLUME ROADS USING EMPIRICAL MECHANISTIC AND FEM APPROACHES

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Abstract

Low volume roads (LVRs) are an integral part of rural and peri-urban infrastructure, which require low-cost and reliable techniques for pavement design. The study provides a comprehensive comparison of empirical, mechanistic empirical and finite element modelling (FEM) methods for pavement design with LVRs sealed and unsealed. The six methods tested: California Bearing Ratio (CBR), AASHTO 1993, TRH 20, Odemark stress-based approach, MePADS elastic modelling, and 2D/3D FEM simulation with Abaqus. For two separate study areas, Northern Cape soil (NCS) and KwaZulu-Natal soil (KZNS) (which represent dry and wet climates), we standardized the design input parameters: resilient modulus (M_R), serviceability indices, and traffic loading (ESALs). All three empirical approaches (CBR, AASHTO and TRH 20) yielded different values for pavement thickness; only TRH 20 included a gravel wearing layer. One of the mechanistic methods Odemark used, did not depend on changes in the environment for estimates of performance to intermediate measures. Advanced FEM calculations incorporating 2D and 3D models involved material characterization established from DCP and triaxial resilient modulus data. So, while 2D models can serve as good design guidelines for load limits in pavement applications, this research demonstrated that 3D material properties are required to replicate the stress distribution on pavements more accurately. The pavement life (years) predicted for the DCP elastic models, despite being shorter than those projected for other LVR elastic and visitation models, represented better material behavior under the field circumstances that unfitted-pavements experience; hence only DCP elastic models are mandatory in unsurfaced LVR design. Comparison with MePADS findings showed a good correlation for 2D FEM results and minor deviations (12%) in the predicted subgrade strains through use of 3D FEM. This points towards the need for calibrated context sensitive models. Fine-tuning material characterization and utilizing advanced approaches to the FEM for assessing pavement life and guiding eco-sustainable LVR design, particularly in resource-constrained environments, are thereby critical based on the findings from this study.

Keywords: low volume roads, pavement design, resilient modulus, FEM analysis

1 Introduction

This study presents the performance and suitability of a range of pavement design methods for low-volume roads (LVRs). The methods, California bearing ratio (CBR) method, AASHTO method 1993 [1], TRH 20 method [2], Odemark's method [3], mePADS software [4], and finite element method (FEM) approach [5] are compared against each other.

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All of these unique principles and computational techniques address numerous factors, such as soil conditions, material behavior, and load-carrying capacity. This study identifies the most reliable and cost-effective approach for ensuring long-term pavement durability under low-traffic volume scenarios.

1.1 CBR design, AASHTO 1993, TRH 20 and Odemark's input parameters

The design input parameters and criteria necessary for flexible pavement design were defined in accordance with the guidelines provided in the CBR, AASHTO, and TRH 20 design guides. For the CBR, TRH 20, and Odemark methods, the pavement layer properties are considered on the basis of the CBR value, whereas in the AASHTO design, they are considered on the basis of the M_R value. Table 1 presents a summary of the design input parameters and the various methods applicable to the input parameters. The expected cumulative traffic over the design life of 10-15 years is also presented. Input parameters such as the initial and terminal serviceability, layer coefficients, and design reliability are among those listed in table 1. The initial serviceability of a newly built pavement was set at 4.2, and at the end of the design life, the terminal serviceability was specified as 2.0. These parameters were established via recommendations from AASHTO. The coefficients for the layers constitute another key group of input variables, which are determined on the basis of the available materials. AASHTO recommends a typical value of 0.07 for sandy gravel (the in-situ material). Finally, the design reliability was set at 80% for low-volume roads, as recommended by AASHTO.

Table 1 Pavement design input values used in CBR, AASHTO, TRH20 and Odemark's methods

Parameter	Design values		Comments
	Northern Cape soil (NCS)	KwaZulu-Natal soil (KZNS)	
Expected ESAL, SCw18	0.892/1.488 x 10^6	0.892/1.488 x 106	CBR, AASHTO, TRH20, DCP, Odemark
Structural design period	10/15 years	10/15 years	CBR, AASHTO, TRH20
Reliability, R	80%	80%	AASHTO, TRH20
Zr	- 0.841	- 0.841	AASHTO
So	0.45	0.45	AASHTO
M_R base (CBR)	142 MPa (35)	362 MPa (106)	CBR, AASHTO, TRH20, DCP, Odemark
M_R subbase (CBR)	155 MPa (39)	185 MPa (48)	
M_R subgrade (CBR)	201 MPa (52)	154 MPa (13)	
Layer coefficients	a2: 0.07 (Sand gravel)		AASHTO
Pi	4.2	4.2	AASHTO
Pt	2.0	2.0	AASHTO
PSI = (Pi – Pt)	2.2	2.2	AASHTO
Weinert N-values	2	10	AASHTO, TRH20,
Tyre pressure and radius of contact	80 kN/195 mm		Odemark

1.2 CBR design, AASHTO and TRH 20 pavement thickness

Based on table 1, two pavement sections were designed using the CBR approach through AASHTO 1993 and TRH 20. AASHTO produced 150 mm granular base and subbase layers for both NCS and KZNS, while the CBR method gave NCS 0 mm base and 120 mm subbase due to its strong, dry subgrade. TRH 20 resulted in 125 to 150 mm for both layers with a double surface seal, specifically addressing LVR design. Although most empirical methods excluded a seal, designers often add a 30 mm non-structural layer. Odemark's stress based method yielded 100 mm base and 150 mm subbase for NCS, and 80 mm and 120 mm for KZNS, aligning with Paul et al. [3].

Table 2 Pavement design thickness established via the CBR, AASHTO 1993, TRH 20 and Odemark methods for different cases

Design method	Case description	Surface layer [mm]	Granular base thickness [mm])	Granular subbase thickness [mm]
CBR	NCS	-	-	120
	KZNS	-	-	280
AASHTO 1993	NCS	-	150	150
	KZNS	-	150	150
TRH 20	NCS	WC (80)	100 G5	-
	KZNS	WC (90)	100 G5	100 G7
Odemark's stress-based method	NCS	-	100	150
	KZNS	-	80	120

2 LVR pavement design using 2D and 3D FEM modelling analysis for unseal LVRs

In South Africa, LVRs typically consist of a surface seal, a 150 mm granular base, a 150 mm stabilized subbase, and a subgrade including selected and in situ layers [4, 5]. Due to cost constraints, these roads are often left unpaved, so only three layers excluding the surfacing were analyzed. Two material characterizations were applied using both 2D and 3D Abaqus models for two study areas. The 2D model represented a three layer system with a compacted subgrade, comprising a 150 mm granular base, a 150 mm subbase, and a 2000 mm natural subgrade foundation designed according to AASHTO 1993. The 3D model used the same layer configuration to assess the effects of unsealed pavements. All models were also analyzed in MePADS for validation and comparison. Key FEM inputs such as geometry, material properties, element type and mesh size, boundary conditions, and loading were carefully considered [6].

2.1 Model geometry, element types and mesh size

The models were simulated in Abaqus using an axisymmetric approach with a 3000 mm horizontal dimension and 2300 mm depth. The base and subbase were each 150 mm thick, with a 2000 mm subgrade. Four node axisymmetric elements CAX4 and CAX4R were used for 2D models, and C3D8R elements for 3D models. A refined 5 mm mesh was applied near the load, gradually increasing to 500 mm away from it [5, 7], with element distribution shown in table 3. MePADS modelled the pavement as an axisymmetric linear elastic system and was used to validate mesh configuration. Results were consistent with mesh sensitivity analysis and comparable to other multilayer elastic software.

Table 3 Abaqus model mesh configuration analysis

Pavement structures	2D model	3D model	2D model	3D model
	No. of elements along the layer thickness	No. of elements along the layer thickness	Total number of elements CAX4R	Total number of elements C3D8R
Gravel base granular subbase subgrade	41	12	4,346	271,272
	8	7	848	158,242
	8	6	848	135,636

2.2 Material characterization

All the layers of the pavement are assumed to be linearly elastic in behavior for simplicity, and the effects of material characterization on the FEM models are determined. Additionally, two different material characterizations were used in the two-study area model: material characterization based on the obtained DCP test results and resilient modulus test results. The material characteristics of the gravel base, subgrade, and subgrade are closely related to those presented in [8]; these material characteristics are presented in table 4.

Table 4 Material characterizations used in the models

Study Area	Layer	Code	t [mm]	Density [kg/m³]	M_R [MPa]	E_{DCP} [MPa]	μ
NCS	Gravel	G4	150	1908	362	142	0.02
	Gravel	G5	150			141	
	Subgrade	G7	2000			155	
KZNS	Gravel	G4	150	1870	185	362	0.12
	Gravel	G5	150			185	
	Subgrade	G7	2000			154	

Code = Material code; t = Thickness of layer; E_{DCP} = DCP Elastic Modulus; μ = possion ratio

2.3 Boundary condition, loading and contact modelling

The models were analyzed under static loading using linear perturbation. Horizontal restraints were applied, while the subgrade was fixed in all directions. In Abaqus, a circular load radius of 195 mm with 0.67 MPa pressure was applied [5, 9], assuming fully bonded layer interfaces. In MePADS, the load area was 119, 355 mm² (285 × 419 mm) with an 80, 000 N load.

3 Results and discussion

In this analysis, the structural importance of the granular base was evaluated by considering material characterization on the basis of the results of the DCP test and resilient modulus test. The decision to leave these pavements unsurfaced is justified by the need to minimize the cost of low-volume roads while maximizing the strength and performance of the granular base in the design of LVRs. To assess the impact of the granular base, critical areas such as the compressive vertical strain and stress at the top of the subgrade were analyzed as part of the pavement verification process.

3.1 Effect of material characterization on an unsealed surface Utilizing DCP results and triaxial m_r results

Technically, the use of DCP test results should provide a better option for pavement design, as it shows a balanced structure of the pavement layers, especially for the KZNS result (figure 1) in terms of the stress, strain, and deflection developed in the pavement layers. However, this was not the case, as the triaxial elastic material performed better. It could be argued that the elastic modulus for the triaxial M_R model is greater than that of the DCP model. Nevertheless, the redistribution of stress and strain resulting from the input of the DCP elastic model needs to be better understood (figure 1). The analysis in this study revealed that the compressive vertical strain/stress at the top of the subgrade was drastically lower (from 1395×10^{-6} to 432.0×10^{-6}) in the elastic model (triaxial test results) than in the DCP elastic model (figure 2). Although the DCP elastic model showed a larger area affected by strain/stress, this reflects the effect of changes in layer properties, which more accurately depicts the pavement layer structure in real-life scenarios. This subsequently affects the structural capacity of the subgrade layer (DCP elastic model: 8.29×10^3, and elastic model: 1.57×10^6 ESALs). Overall, when designing low-volume roads, the DCP elastic model needs to be considered, as it provides a closer depiction of real-life pavement structures. However, a proper correlation between the DCP elastic model and the elastic (triaxial resilient modulus) model needs to be established. According to Ikechukwu et al. [10], there is a close correlation between the in situ DCP and triaxial test results in terms of material characterization. Therefore, the use of a single elastic modulus for pavement layer design tends to lead to overdesign of the pavement structure.

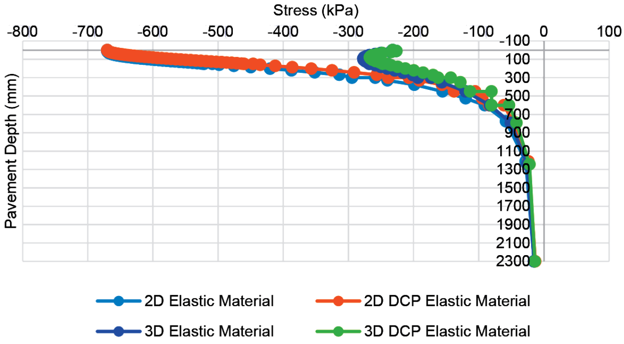

Figure 1 Pavement layer meshing configuration and bias factor meshing for 2D and 3D models

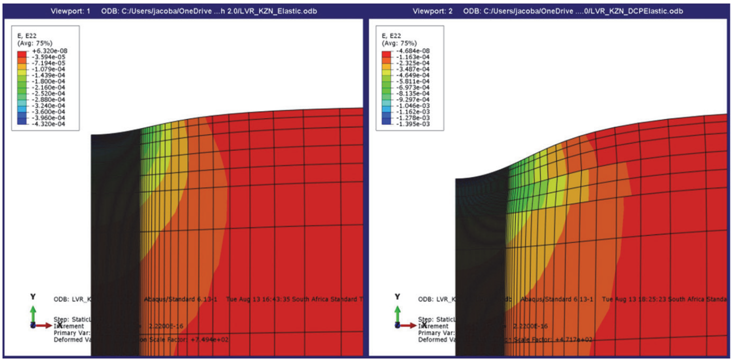

Figure 2 Vertical strains on top of the subgrade for the 2D model using the elastic and DCP elastic models (KZNS)

3.2 Comparative analysis of 2D and 3D models in the FEM

Pavement life based on rutting (table 5) was estimated as load repetitions to failure (Nr) for unsealed LVRs in NCS and KZNS using elastic and DCP elastic models in 2D and 3D FEM. In both regions, 3D models predicted higher Nr than 2D, indicating improved stress representation. For KZNS, elastic results were 2.13×10^6 in 3D versus 1.58×10^6 in 2D, while DCP elastic predicted 11.46×10^3 and 8.29×10^3, respectively. The large variation between elastic and DCP elastic outcomes emphasizes the importance of material characterization. NCS failed to meet the 1.488×10^6 design life (table 1), supporting the need for surface sealing and careful model selection.

Table 5 Rutting failure analysis for unsealing LVRs via elastic and DCP elastic models

Study area	MePADS	2D Abaqus model	3D Abaqus model	Percentage difference (%) btw MePADS &2D/3DAbaqus
	Vertical strain εc (10^{-6}) in subgrade	Vertical strain εc (10^{-6}) in subgrade	Vertical strain εc (10^{-6}) in subgrade	
		NCS		
Elastic	699	676.5	631.6	3.27/10.13
DCP Elastic	1793	1748	1637	2.54/9.1
		KZNS		
Elastic	453	432	403.7	4.75/11.5
DCP Elastic	1426	1395	1298	2.2/9.4

3.3 Validation of the FEM

Validation of the FEM was conducted to compare its results (in terms of stress and strain) with those obtained from linear elastic theory [11]. The validation analysis was performed for Study Area 1 (NCS) and Study Area 2 (KZNS) using a granular base and subbase layer of 150 mm each and a subgrade of 2,000 mm. Linear elastic properties were applied to all the layers. MePADS, a linear elastic layered analysis program, was employed to validate the FEM results. Additionally, an axisymmetric model in Abaqus was developed, and the results closely matched those obtained from the layered elastic software. The design model details used in MePADS are highlighted above. The 2D Abaqus model displayed great similarity to those of MePADS (table 6). However, the results of the 3D FEM model revealed a 12% difference in vertical strain on the subgrade compared with those of the other methods, with fewer discrepancies in the other layers, despite the use of a similar mesh configuration. This suggests that while the FEM provides reliable results, some variations can occur depending on the modelling approach and complexity, emphasizing the need for careful calibration and validation in different contexts.

Table 6 Rutting failure analysis for unsealing LVRs via elastic and DCP elastic models

Model	Fatigue criterion No. of load repetitions to failure N_r			
	NCS-2D	**NCS-3D**	**KZNS-2D**	**KZNS-3D**
Elastic	211.80×10^3	288.10×10^3	1.58×10^6	2.13×10^6
DCP Elastic	3.02×10^3	4.05×10^3	8.29×10^3	$\times 10^3$

Model	Fatigue criterion No. of load repetitions to failure N_r			
	NCS-2D	**NCS-3D**	**KZNS-2D**	**KZNS-3D**
Elastic	211.80×10^3	288.10×10^3	1.58×10^6	2.13×10^6
DCP Elastic	3.02×10^3	4.05×10^3	8.29×10^3	$\times 10^3$

4 Conclusion

This study presents a comparative analysis of various pavement design methods for LVRs, focusing on empirical and mechanistic-empirical methods. It evaluates the performance of design methods such as the CBR, AASHTO 1993, TRH 20, and Odemark methods, alongside 2D and 3D FEM models, which were validated via MePADS layer elastic software. Overall, this study highlights the consideration of a gravel-wearing surface in the TRH 20 empirical method of pavement design, whereas other methods do not include the use of such. However, at the engineers' discretion and on road user demands, a thin layer, which is also considered a non-structural layer, such as a surface seal, could be applied. The chapter also notes that while 3D FEM models generally offer more accurate predictions of pavement performance by better-capturing stress distributions, DCP elastic models provide a closer depiction of real-life conditions, especially for unsealed LVRs. However, the DCP elastic model did not produce satisfactory results, which may explain the continuous maintenance requirements for unsealed roads. Additionally, the validation of the FEM results against linear elastic theory and MePADS software further supports the reliability of these models, although some variations were noted, particularly in 3D modelling. Overall, this chapter underscores the need to carefully select and calibrate pavement design models to ensure accurate, cost-effective, and sustainable designs for low-volume roads.

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