INTERIM REVISION OF THE SOUTH AFRICAN MECHANISTIC-EMPIRICAL PAVEMENT DESIGN METHOD FOR FLEXIBLE PAVEMENTS

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Abstract

Pavement design methods, in combination with network level management systems must enable road authorities to develop reliable long-term financial plans based on the estimated structural capacity of the road network. Inaccurate design models at the core of such a design system could, however, result in significant design risk and inappropriate financial planning. The design model for unbound granular layers contained in the 1996 version of the South African Mechanistic-Empirical Design Method (SAMDM) for flexible pavements was shown to be overly sensitive to minor changes in certain input variables at the Conference on Asphalt Pavements for Southern Africa held in 2004 and the general accuracy of this model was challenged.

This paper presents an interim revision of the SAMDM recommending increased contact stress values at the tyre-pavement interface; updated unbound material characterisation parameters; effective stress analysis for unbound material; and revised damage models for estimating the structural capacity of unbound granular layers including the pavement subgrade. The internationally accepted subgrade vertical strain damage model is replaced with a model calibrated for local material and environmental conditions. The Factor of Safety (FoS) model unbound, granular base and subbase layers is replaced with a Stress Ratio (SR) model which is explicitly calibrated for the effect of material density and saturation levels. The revised models are shown to be less sensitive to variation in the resilient input properties of unbound pavement layers and better suited to simulating the permanent deformation of granular pavement layers under repeated traffic loading.

1. INTRODUCTION

1.1. Background

Pavement design is essentially an endurance problem and not a stability problem such as the design challenges in structural and geotechnical engineering. Pavement failures are definitely undesirable but generally the consequences of pavement failures are not as catastrophic as the collapse of a bridge or a building. Instead, deteriorating road conditions and ultimate pavement failures have a long-term negative economic impact and financial implications that are not immediately apparent. Pavement design methods, in combination with network level management systems must therefore enable road authorities to develop reliable long-term financial plans based on the estimated structural capacity of the road network. Certainly, the pavement design method should also guard against premature failure as far as possible but often these premature failures are not caused by a design error but rather attributed to:

- insufficient information collected during the design investigation;
- incorrect interpretation of the information and more specifically oversight of risk factors, and
- violation of design assumptions during construction as a result of poor construction quality.

The vision for future pavement design systems in South Africa is to incorporate the mechanistic-empirical models in a holistic design system that also addresses the interaction with network level management systems; that guides the design investigation process and ultimately base design decisions on economic considerations to alleviate some of the above aspects. However, inaccurate design models contained at the core of such a design system will result in significant, unknown design risk and it cannot be denied that the 1996 version of the South African Mechanistic Design Method (SAMDM) (Theyse *et al*., 1996) for flexible pavements contained such inaccurate design models.

1.2. Historical development

The development of mechanistic-empirical (ME) based flexible pavement design procedures have long been pursued in South Africa (Van Vuuren *et al*., 1974, Walker *et al*., 1977, Paterson and Maree, 1978, Maree and Freeme, 1981, Jordaan, 1993). The overview of the method published in 1996 (Theyse *et al*., 1996) was based largely on the content of these earlier publications. The implementation of this 1996 version of the method in software packages resulted in a much larger user group having access to the method and using the method for pavement design. The wide-scale use of the method highlighted certain of the problems associated with the method and culminated in a paper at the CAPSA 2004 conference (Jooste, 2004) questioning the value of the method, especially for the design of unbound, granular pavement layers.

Some of the problems raised at the CAPSA 2004 conference were known to the research fraternity and in certain cases research at the CSIR was already aimed at developing solutions to these problems. This research effort was significantly increased with the initiation of a project by the South African National Roads Agency Limited (SANRAL) to revise the South African pavement design method for flexible and rigid pavements (SAPDM) (www.SAPDM.co.za). A framework for the revision of the mechanistic-empirical design method for flexible pavements was presented at CAPSA 2007 and although it is not possible to present the final revised design method yet, significant improvements to the design of unbound, granular layers are already possible.

1.3. Proposed interim improvements

This paper presents an interim revision of the mechanistic-empirical design method for flexible pavements, addressing the following aspects:

1. Load characterisation
    a. Recommendations on increased tyre contact stress levels for design;
2. Material characterisation
    a. Updated resilient response characterisation of the pavement subgrade;
    b. Updated strength parameters for unbound granular layers;
3. Primary pavement response analysis
    a. The introduction of effective stress analysis incorporating suction pressure and residual compaction stress;
4. Revised damage models
    a. Interim models for the permanent deformation of unbound pavement layers including the pavement subgrade and structural layers.

The internationally accepted vertical strain damage model for the pavement subgrade is challenged by the paper based on the available experimental data. The plastic strain damage model for the upper unbound pavement layers also shows that the density and saturation of the material dominates the permanent deformation of the material in combination with the imposed stress condition.

The application of the interim solutions presented in the paper are tested against limited Accelerated Pavement Testing (APT) results and compared to results from the 1996 method, showing significant improvement in the modelling ability of the revised models. The design cases used in the critical review by Jooste (2004) are also re-analysed, showing a significant reduction in the disproportionate sensitivity of the method to variation in certain input parameters.

2. MECHANISTIC-EMPIRICAL PAVEMENT MODELLING CONCEPTS

Figure 1 illustrates the dissipation of the stress resulting from an external wheel-load through a layered pavement system. Mechanistic-Empirical pavement design methods attempt to model the resilient response of the pavement and more importantly, the damage caused by the external wheel-load throughout the pavement system. All mechanistic-empirical pavement design methods that are intended for routine pavement design separate the resilient response modelling of the pavement from the damage modelling similar to the diagram shown in Figure 2. Models such as plasticity theory that combines the elastic and plastic response of materials in a single constitutive material model are unlikely to be implemented in routine design methods in the near future as they are computationally intensive and still do not cater for all the possible responses of the materials used in pavement engineering.

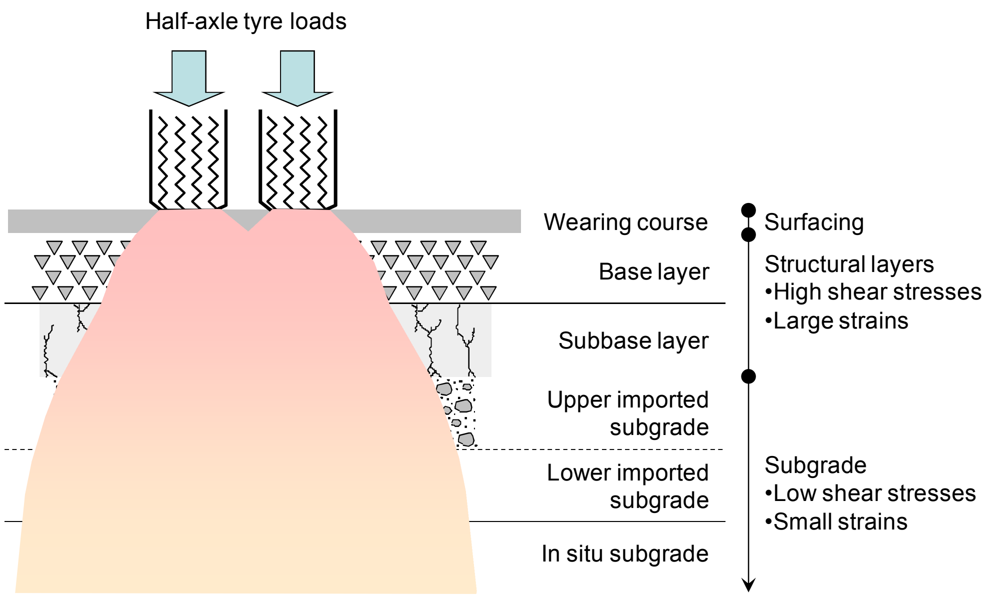


Figure 1. Dissipation of externally applied stress in a layered pavement system (*from* Theyse, 2009)

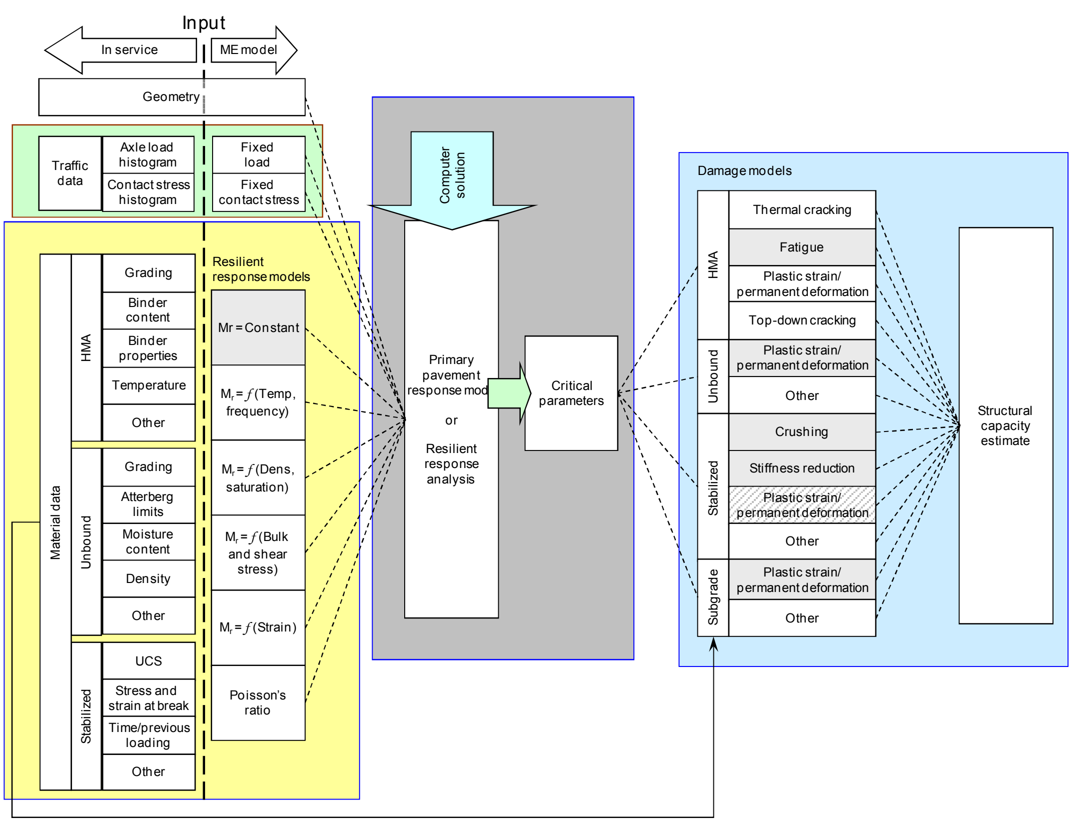


Figure 2. Schematic representation of the main components of a mechanistic-empirical pavement model (*from* Theyse, 2008)

Mechanistic-empirical design methods may be classified according to the manner in which the damage modelling is done. Typically, when new pavements are opened to traffic there is an initial rapid accumulation of distress or damage (bedding-in), followed by a period of linear accumulation of damage and finally an accelerated increase in the damage as illustrated by paths ① and ② in Figure 3. The damage models in classical mechanistic-empirical methods are calibrated to estimate the number of load repetitions required to progress from a condition of zero initial distress or damage to the terminal condition which is merely a predefined, unacceptable condition and does not imply complete failure of the facility. Given that the damage models of classical ME-design methods do not retain any information regarding the progression of damage, a linear progression following path ③ in Figure 3 is implied. Damage models that relate a critical stress or strain parameter (S) to the number of repetitions (N) that can be sustained before the terminal condition is reached are referred to as S-N damage models in this paper.

Another characteristic of classical ME-design methods is the use of a design standard load. The structural capacity of the design pavement is expressed in terms of the number of design standard loads required to reach the terminal structural condition.

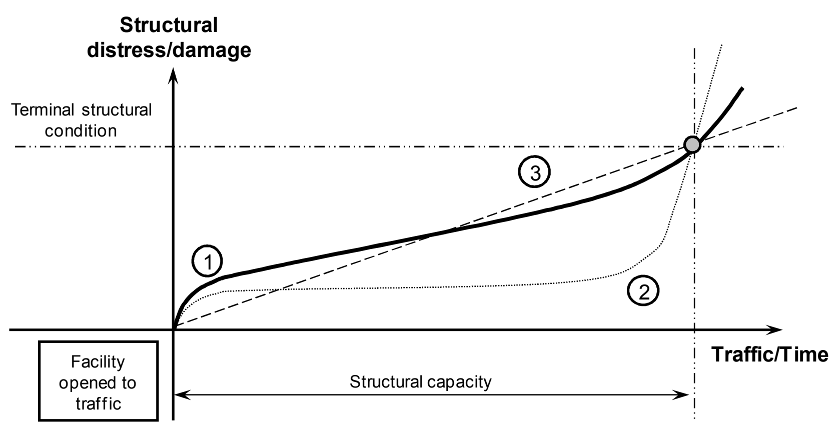


Figure 3. Illustration of various hypothetical damage progression paths for pavements (*from* Theyse, 2009)

More recently, ME-design methods have evolved to accommodate mixed traffic and time-dependent input variables by utilising recursive modelling. These recursive methods may in turn be classified as linear recursive methods, which utilise the damage models from classical ME design methods in combination with Miner's law; or non-linear recursive methods that are based on incremental damage models and the strain-hardening approach.

The interim solutions proposed in this paper are formulated in the context of a classical ME design method which may also be applied in a linear recursive modelling process. Ultimately the revised South African Pavement Design Method will be based on non-linear recursive modelling. However, for the interim models to be immediately useful, they have to be implemented within the current classical ME-design framework used in South Africa. This interim version of the classical South African Mechanistic-Empirical Design Method for flexible pavements is, therefore, referred to as SAMDM2011 to distinguish from the 1996 version.

3. REVISED ME-DESIGN MODELS FOR SAPDM2011

3.1. Load characterisation

The fact that the SAPDM2011 remains a classical ME-design method has the implication that the method relies on the concept of a standard design load to quantify the structural capacity of the pavement. The 80 kN single axle standard design load with a dual-wheel configuration is therefore retained in SAMDM2011 similar to the standard design load in the 1996 version. However, the revised method deviates from the 1996 version regarding the tyre-pavement contact stress that is used for design analysis. Stress-In-Motion (SIM) measurements results have shown the value of 520 kPa recommended by the 1996 version to be too low (De Beer *et al.,* 1999) and SAMDM2011, therefore, makes provision for using increased tyre-pavement contact stress values.

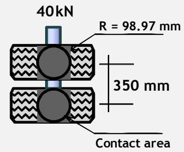
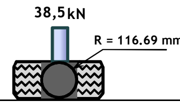
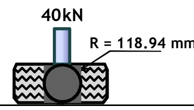
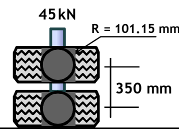
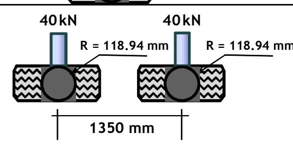
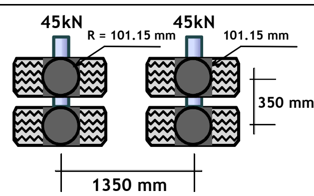
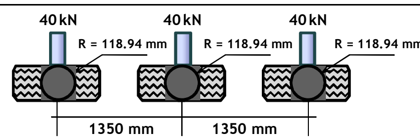
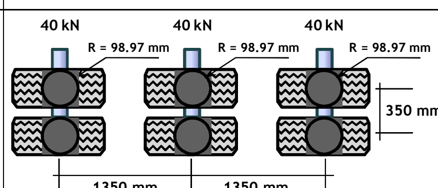
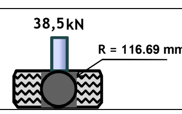
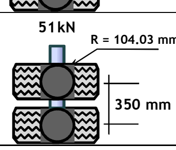
Although the concept of a standard design load is retained in SAPDM2011, recommendations are provided for the axle mass and contact stress on other axle groups and wheel configurations to allow assessment of the effect that these load configurations have on the structural capacity of a pavement. Table 1 provides a summary of typical axle and wheel-load configurations with recommended contact stress values for design analysis. The static axle loads listed in Table 1 are based on the current legal axle loads for these axle groups and wheel configurations.

3.2. Material characterisation

As indicated previously, the mechanistic-empirical method consists of two main modelling components, the primary pavement response model that calculates the elastic response of the pavement to loading and the damage models that quantify the damage in all the pavement layers given certain elastic response parameters. The primary pavement response model used in most mechanistic-empirical design methods is a multi-layer, linear-elastic continuum mechanics model that requires Young's modulus and Poisson's ratio to characterise the resilient response of the materials found in each of the pavement layers. Young's modulus and Poisson's ratio are, however, theoretical concepts that apply to perfectly elastic materials. The Young's modulus that represents the "stiffness" of materials can only be approximated from experimental results and is most often approximated by the resilient modulus for unbound granular material. The resilient modulus is a measure of the elastic recovery of a specimen of material given the repeated application and removal of an axial load under compressive stress conditions. Figure 4 shows a simplified case representing a single cycle of load application and removal. The resilient modulus therefore represents the secant slope through the two extremes of a load-unload, stress-strain hysteresis loop in repeated load tri-axial testing.

Most road-building materials also exhibit stress-dependent and apparent anisotropic behaviour. The magnitude of the resilient modulus therefore depends on the level of confinement of the material and differs under tensile and compressive stress conditions, not to mention the effect of density and saturation. Strictly speaking, there is no single resilient modulus value for a given unbound, granular material but rather an infinite range of possible values. The resilient modulus values recommended for design in this paper, therefore, represents typical stiffness values in a compressive stress region underneath the external wheel-load.

Table 1 Typical axle and wheel-load configurations for pavement design

Vehicle type	Axle group configuration	Wheel configuration	Typical half-axle configuration for analysis *	Static axle group load (kN)	Recommended tyre contact stress (kPa) *
All	Standard design load	Dual		80	650
Truck	Steering axle	Single		77	900
	Single axle	Single		80	900
		Dual		90	700
	Tandem axle	Single		160	900
		Dual		180	700
	Tridem axle	Single		240	900
		Dual		240	650
Bus	Steering axle	Single		77	900
	Single axle	Dual		102	750

* Load radii (R) and tyre contact stresses estimated from SIM measurements on 12R22.5 and 315/80 R22.5 tyres.

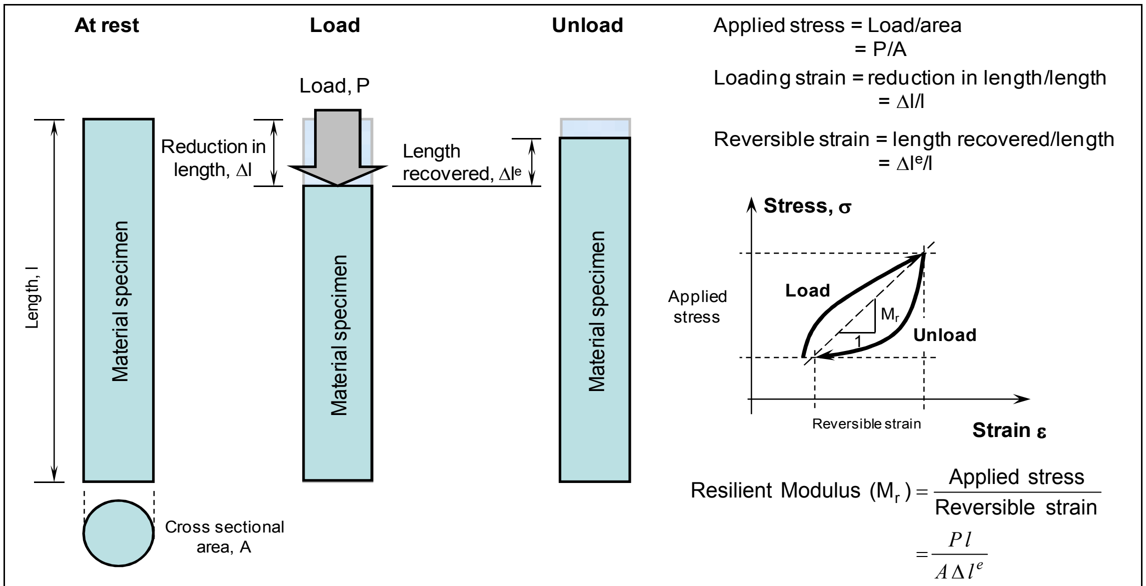


Figure 4. Approximation of Young's modulus by the resilient modulus (*from* Theyse, 2009)

3.2.1 Resilient response characterisation of the pavement subgrade

The resilient modulus values recommended for the pavement subgrade are based on back-calculation from multi-depth deflectometer (MDD) deflection profiles recorded during Heavy Vehicle Simulator (HVS) testing in South Africa. The subgrade was modelled as a semi-infinite half-space in the back-calculation process although the MDD system is normally anchored at a depth of 2,5 to 3 metres. Modelling the subgrade as a semi-infinite half-space results in an over-estimation of the resilient modulus of the subgrade by roughly 20 to 25 %. However, when this "incorrect" modulus is used in a forward design calculation in combination with a semi-infinite half-space, the subgrade deflection which is the proposed new critical parameter for subgrade design is calculated correctly. Using the over-estimated subgrade modulus values with a vertical subgrade strain damage model is not recommended as the structural capacity of the subgrade will be over-estimated.

Table 2 provides a summary of average, long-term resilient modulus values for different subgrade types and moisture conditions for a 40 kN dual wheel-load. A Poisson's ratio of 0,35 is recommended for subgrade materials.

Table 2 Typical resilient modulus values for subgrade material

Subgrade type	Typical subgrade material	Grading Modulus (GM)	Plasticity Index (PI)	Moisture condition (MC)	Resilient modulus (MPa)
Gravel	Sandstone conglomerate (Gauteng)	2,2	Non-plastic	Dry	200 – 300
				Wet	100 – 150
	Ferricrete (Gauteng)	1,2	10	Dry	100 – 200
				Wet	60 – 100
	Decomposed granite (Limpopo)	2,1	14	Dry	300 – 500
				Wet	100 – 150
	Calcrete (Karoo)	1,4 – 1,6	Non-plastic	Dry	200 – 300
				Wet	100 – 150
Sand	Deep sand (Western Cape and KwaZulu-Natal	1,0 – 1,2	Non-plastic	Dry	100 – 150
				Wet	75 – 125

In general, course material tends to have a higher modulus and the higher the plasticity of the material the bigger the difference between the dry and wet modulus of the material.

3.2.2 Resilient response characterisation of unbound granular material

The resilient modulus values recommended for unbound granular base and subbase layers remain the same as for SAMDM1996 and are summarised in Table 3. The recommended Poisson's ratio for these materials is 0,35.

Table 3 Recommended resilient modulus values for unbound granular base and subbase layers

Moisture condition		Dry		Wet	
Support condition		**Cemented layer in slab state**	**Over granular or equivalent granular layer**	**Cemented layer in slab state**	**Over granular or equivalent granular layer**
Material Code (CSRA, 1985)	**Material Description**				
G1	High quality crushed stone	250 - 1000 (450)	150 - 600 (300)	50 - 250 (250)	40 - 200 (200)
G2	Crushed stone	200 - 800 (400)	100 - 400 (250)	50 - 250 (250)	40 - 200 (200)
G3	Crushed stone	200 - 800 (350)	100 - 350 (230)	50 - 200 (200)	40 - 150 (150)
G4	Natural gravel (base quality)	100 - 600 (300)	75 - 350 (225)	50 - 200 (200)	30 - 150 (150)
G5	Natural gravel	50 - 400 (250)	40 - 300 (200)	30 - 150 (150)	20 - 120 (120)
G6	Natural gravel (sub-base quality)	50 - 200 (200)	30 - 200 (150)	20 - 150 (150)	20 - 120 (120)

Although the resilient modulus ranges in Table 3 are wide, the sensitivity of the design model for unbound material to stiffness variation is countered by the introduction of suction pressure and residual compaction stress that act as beneficial pre-stressing of the unbound layers.

3.3. Primary pavement response modelling

The relationship between the ratio of applied stress to the shear strength (Stress Ratio or Factor of Safety) and the plastic strain of unbound material has been confirmed by local (Maree, 1978 and Theyse, 2008) as well as international research (Huurman, 1997 and van Niekerk *et al*, 1998). One of the major problems associated with the SAPDM1996 is the calculation of inadmissible stress conditions (FoS < 1, SR > 1) for unbound layers. These inadmissible stresses are partly the result of not considering the effective stress condition and the behaviour of unbound granular material depends largely on the effective stress condition to which the material is subjected.

SAMDM1996 only considered the stress resulting from the application of the external wheel-load in the primary response analysis of unbound granular layers. While the damage (primarily permanent deformation) of unbound granular layers is the direct result of the stress caused by the external wheel-load, the amount of damage is determined by the effective stress regime in the unbound granular layers. The introduction of effective stress, especially the residual compaction stress component, causes the unbound pavement layers to be constrained similar to pre-stressed concrete thereby preventing the development of tensile stress in the unbound material.

SAMDM2011 deviates from the past approach by introducing effective stress analysis in the primary pavement response model for unbound granular base and subbase layers by considering the following stress components:

- The vertical overburden stress in combination with residual compaction stress;
- Internal suction pressure resulting from the partial saturation of the material; and
- The stress caused by the external wheel-load.

The development of residual compaction stress behind retaining walls have long been studied and documented in geotechnical engineering. Similarly, residual compaction stress occurs in pavement layers and Uzan (1985) formulated theoretical models for the residual compaction stress in pavement layers based on static equilibrium conditions. Dehlen (1958) provides experimental proof of residual compaction stress. A simplified residual compaction stress model was developed for SAPDM2011 using the theory formulated by Uzan, the shear strength properties of a selection of unbound granular road-building materials and the experimental results from Dehlen.

The concepts of matric suction and suction pressure are also well accepted in geotechnical engineering (Fredlund, 1985: 465 - 472, Vanapalli *et al*, 1996a: 259 – 268, Vanapalli *et al*, 1996b: 379 – 392, Vanapalli and Fredlund, 1999: 93 – 96 and Vanapalli and Fredlund, 2000: 195 - 209) and Heath (Heath *et al*, 2002 and Heath, 2002) introduced suction pressure in the analysis of granular pavement materials in California. Locally, a suction pressure model has been developed based on a linear approximation of the soil-water characteristic curve for a range of unbound granular material (Theyse, 2008) and the suction pressure model parameters were found to be largely determined by the grading of the material. This model is incorporated in SAPDM2011. Research is continuing to develop a general suction pressure model for design application as part of the ongoing SANRAL research project.

The stress caused by the external wheel-load is calculated using a multi-layer, linear elastic software programme such as GAMES (Maina and Matsui, 2004). The combination of the residual compaction stress, suction pressure and external load stress in an effective stress analysis has been coded into a software package for mechanistic-empirical pavement design that will be released during 2012.

3.4. Damage models

SAMDM2011 provides revised damage models for the pavement subgrade and unbound granular base and subbase layers.

3.4.1 Subgrade permanent deformation damage models

Similar to the 1996 version, SAMDM2011 makes provision for two levels of subgrade permanent deformation, namely 10 and 20 mm. Using multi-depth deflection data from HVS tests, Theyse (2001) showed that the vertical subgrade strain at the top of the subgrade is not a good predictor of subgrade permanent deformation. The subgrade deflection was found to have a better correlation with the subgrade permanent deformation. This does not imply that the subgrade plastic strain is not correlated to the subgrade vertical elastic strain. In fact, the integration of the subgrade vertical elastic and plastic strain over the full depth of the subgrade result in the subgrade deflection and subgrade permanent deformation respectively. Such an approach, however, requires that the exact subgrade stiffness (and hence vertical strain) profile is known. This detailed information is rarely available for calibration data sets generated from controlled testing, let alone pavement design cases.

Subgrade deflection is therefore used as the critical parameter for subgrade permanent deformation. The resilient modulus values listed in Table 2 should be used in combination with a semi-infinite subgrade to calculate the subgrade deflection between wheel-loads for design purposes.

A range of S-N type subgrade permanent deformation models were calibrated as part of the SANRAL SAPDM project, providing for subgrade deformation levels from 1 to 20 mm. It was also found that a separation had to be made between the models for deep sand subgrades typical of KwaZulu-Natal and the Western Cape and gravel subgrades. The S-N models for 10 and 20 mm subgrade permanent deformation included in SAMDM2011 are illustrated in Figure 5.

The scatter in the data plotted in Figure 5 is wide but such variation is an inherent characteristic of pavement performance. The distinction between sandy and gravel subgrades as well as subgrade capacities of a particular subgrade type at different levels of subgrade deflection are, however, clear for a given level of subgrade permanent deformation. Although there is significant overlap in the data ranges for 10 and 20 mm permanent deformation, the risk profiles are different for the two levels of permanent deformation for a given traffic demand.

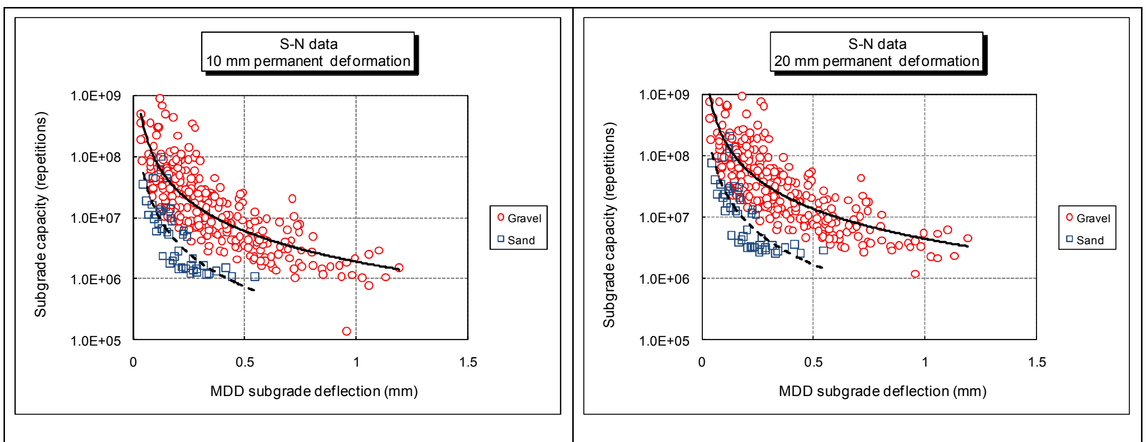


Figure 5. Subgrade permanent deformation damage models

3.4.2 Plastic strain damage models for unbound granular layers

Plastic strain data from repeated load tri-axial tests by Theyse (2008) were used for the development of a series of S-N type plastic strain damage models under the SANRAL SAPDM project. It was found the volumetric density and degree of saturation of the material had to be incorporated in the formulation of the critical parameter in addition to the stress ratio. The stress ratio is based on effective stress according to Equation (1). A better correlation was found between the plastic strain laboratory results and the stress ratio formulated in terms of major principal stress than the stress ratio formulated in terms of deviator stress.

Implementation of the stress ratio in a pavement system requires that the stress ratio be calculated from the effective vertical and horizontal stress under the wheel instead of the major and minor principal stress. The reason for this being that the overburden pressure and residual compaction stress are uniform stress fields that affect the major and minor principal stress but do not contribute to the vertical permanent deformation of the pavement layer, only the vertical stress component of the external wheel-load does.

$$SR = \frac{\sigma'_v}{\sigma'_h \tan^2\left[45° + \frac{\phi}{2}\right] + 2C\tan\left[45° + \frac{\phi}{2}\right]} \qquad \text{Eq. (1)}$$

where SR = stress ratio
σ'_v = effective vertical stress
σ'_h = effective horizontal stress
ϕ = friction angle of the material
C = cohesion of the material (kPa)

Recommended Mohr-Coulomb shear strength parameters for unbound granular base and subbase materials are summarised in Table 4 for preliminary design. It is strongly recommended that shear strength tests be performed on the materials selected for the construction of a road and that the design should be finalised using project specific material properties.

Table 4 Recommended shear strength properties for unbound granular material to be used during preliminary design

Application	Material	Saturation level	Recommended shear strength parameters	
			Cohesion (kPa)	Friction angle (°)
Base	G1	20 % (dry)	90 - 130	53 - 57
		50 % (moderate)	75 - 100	51 - 55
		80 % (wet)	50 - 75	50 - 53
	G2	20 % (dry)	100 - 125	54
		50 % (moderate)	50	52
		80 % (wet)	45	50
	G3	20 % (dry)	75	51
		50 % (moderate)	40	51
		80 % (wet)	20	50
	G4	20 % (dry)	75	51
		50 % (moderate)	40	47
		80 % (wet)	20	45
Subbase - coarse material	G5/6 (BLS x P425 < 170)	20 % (dry)	100 - 125	45 - 49
		50 % (moderate)	50 - 100	41 - 45
		80 % (wet)	10 - 50	39 - 42
	G5/6 (BLS x P425 > 170)	20 % (dry)	225 - 275	45 - 49
		50 % (moderate)	50 - 100	41 - 44
		80 % (wet)	25 - 35	31 - 33
Subbase - fine material	G5/6 (BLS x P425 < 100)	20 % (dry)	125 - 250	43 - 45
		50 % (moderate)	40 - 50	43 - 45
		80 % (wet)	10 - 25	40 - 43

Notes: Course material - grading modulus (GM) 1,7 to 2,3; maximum particle size 26,5 to 37,5 mm;
Fine material - grading modulus (GM) 1,5 to 1,6; maximum particle size < 13,2 mm;
BLS - Bar Linear Shrinkage;
P425 - Percentage passing 0,425 mm sieve

The damage models for unbound base and subbase layers makes provision for different levels of plastic strain ranging from 1 to 19 % plastic strain. The appropriate level of plastic strain depends on the terminal deformation level selected by the designer i.e. a terminal condition of 20 mm permanent deformation equates to 13 % plastic strain for a 150 mm thick layer and 20 % plastic strain for a 100 mm thick layer. Figure 6 illustrates the S-N damage model for 19 % plastic strain as a function of the Stress Ratio (SR), saturation level (S) and volumetric density (VD) of the material.

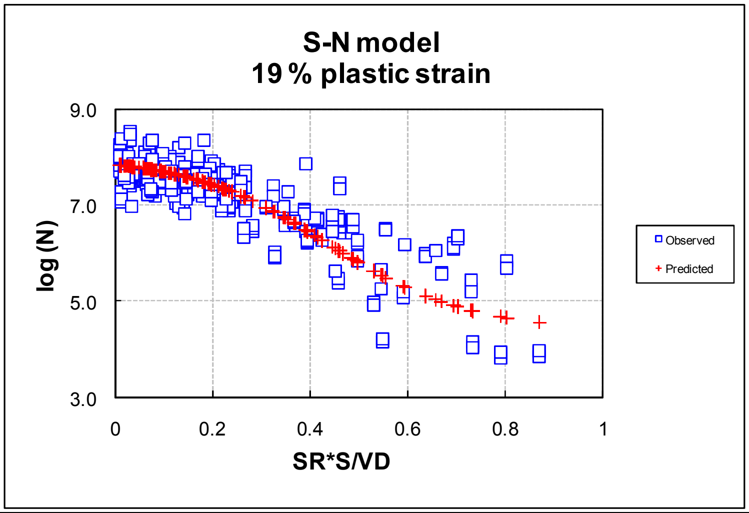


Figure 6. Plastic strain damage model for 19 % plastic strain of unbound base and subbase layers

4. VALIDATION OF SAMDM2011

The effective stress analysis and damage models presented in the preceding sections were coded into a software package for mechanistic-empirical pavement design (*me*-PADS®) developed at the CSIR. As indicated earlier, the revised models are implemented in a classical ME-design approach meaning that an estimate of the structural capacity of each pavement layer is made independently but with due consideration of the interaction between layers in the pavement system. The revised models were evaluated at the hand of the sensitivity of the models to changes in the resilient response characteristics of the pavement materials and the ability of the models to simulate permanent deformation damage recorded during HVS testing.

4.1. Assessment of the sensitivity of the method to input variation

The critical review of SAMDM1996 by Jooste (2004) focused on the sensitivity of the method, specifically the models for unbound materials, to changes in the resilient response parameters of the unbound layers and their supporting layers. Jooste established a base case for analysis consisting of the pavement structure summarised in Table 5. Jooste (2004) analysed four variations of the base case of which two related to variation in the resilient properties of the pavement layers and two related to changes in the shear strength properties of the G1 base layer. The modified resilient response parameters are highlighted in Table 5 for the first two variations analysed by Jooste.

The base case and first two variations were recalculated using the proposed new design models and the same resilient and shear strength properties used by Jooste (2004). The pavement was analysed using a 20 kN dual-wheel load at 750 kPa contact pressure similar to Jooste. Table 6 summarises the base layer capacity reported by Jooste and the results from the revised models. The revised method is far less sensitive to variation in the resilient input parameters than the 1996 method for the particular case investigated.

Table 5 Pavement structure used by Jooste (2004) to perform a sensitivity analysis of SAMDM1996

Layer thickness	Material Description	Base case		Variation 1		Variation 2	
		Resilient modulus (MPa)	Poisson's ratio	Resilient modulus (MPa)	Poisson's ratio	Resilient modulus (MPa)	Poisson's ratio
40 mm	Asphalt overlay	2500	0,40	2500	0,40	2500	0,40
150 mm	G1 overlay	650	0,35	650	0,35	650	0,38
250 mm	Old cemented layer	450	0,35	400	0,35	450	0,35
150 mm	Selected subgrade	140	0,35	140	0,35	140	0,35
Semi-infinite	Sandy subgrade	90	0,35	90	0,35	90	0,35

Table 6 Comparison of the G1 base layer capacity according to SAMDM1996 and SAMDM2011

Method	Parameter	Base case	Variation 1	Variation 2
SAMDM1996 (from Jooste, 2004)	Base capacity	$10{,}7 \times 10^6$	$4{,}2 \times 10^6$	$54{,}5 \times 10^6$
	Deviation from base	0 %	-25 %	+ 168 %
SAMDM2011 (*me*-PADS®)	Base capacity	26×10^6	25×10^6	27×10^6
	Deviation from base	0 %	-3 %	+5 %

4.2. Simulation of limited Accelerated Pavement Testing (APT) results

A full depth granular pavement consisting of a crushed stone base on top of a crushed stone subbase with a deep sandy subgrade was tested on the National Road 7 (N7) near Cape Town during 2001-2002 with the HVS. Figure 7 shows the pavement structure on the fast lane of the southbound carriageway that was tested while Table 7 summarises the test programmes for two HVS tests done on this pavement.

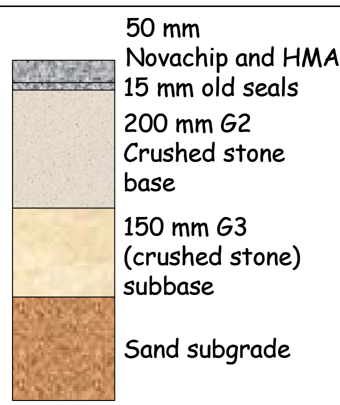


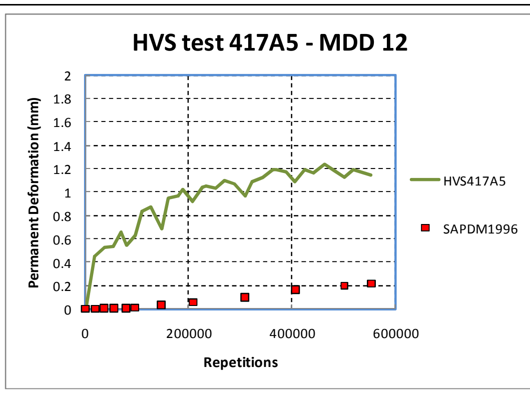
Figure 7. Pavement structure tested by the HVS on the N7 near Cape Town

Table 7 HVS test programme for the crushed stone pavement on the N7

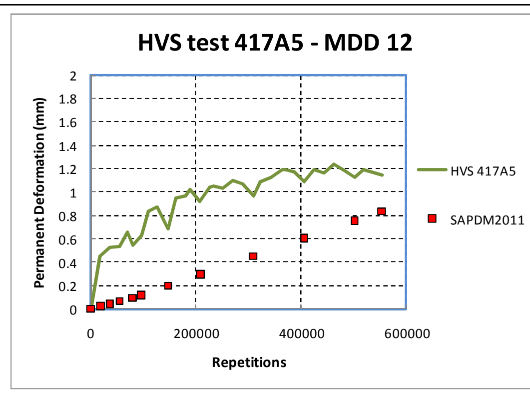
		Trafficking wheel-load and tyre pressure			
		Dry Conditions			Wet conditions
		40 kN, 620 kPa	60 kN, 620 kPa	80 kN, 850 kPa	40 kN, 620 kPa
Test Section	417A5	0 – 55 000	55 000 – 147 444	147 444 – 553 644	-
	418A5	0 – 616 806	-	-	616 806 – 1 143 423

Deflections and permanent deformation data were recorded at three locations (MDDs 4, 8 and 12) on the HVS tests. The resilient moduli back-calculated for each of the pavement layers using the depth-deflections recorded at regular intervals during the HVS tests were used as common input to SAMDM1996 and SAMDM2011 to estimate the structural capacity of each pavement layer to a 20 mm terminal rut condition. The evolution of permanent deformation during the two HVS tests was subsequently modelled using a linear recursive method based on Miner's law.

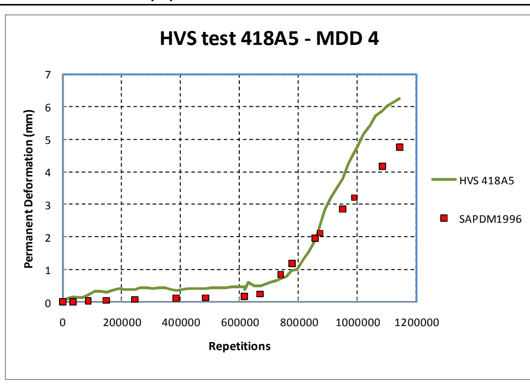
Figure 8 shows examples of the observed and modelled base layer permanent deformation for HVS tests 417A5 and 418A5 using SAMDM1996 and SAMDM2011. In the case of test 417A5 during which the wheel-load was increased from 40 to 60 and then 80 kN the SAMDM1996 models did not respond well and the base layer permanent deformation only started to increase slightly under the 80 kN load. Although the SAMDM2011 models do not simulate the initial bedding-in that occurred, the rate of permanent deformation increased for both the 60 and 80 kN load portions of test 417A5. In the case of test 418A5 during which the section was soaked with water from about 600 000 repetitions onwards, both models responded equally and realistically to the increase in saturation levels of the base layer.



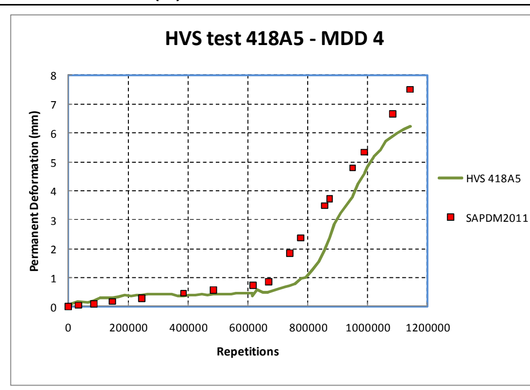
(a) SAMDM1996 – 417A5



(b) SAMDM2011 – 417A5



(c) SAMDM1996 – 418A5



(d) SAMDM2011 – 418A5

Figure 8. Base layer permanent deformation for HVS tests 417A5 and 418A5

The observed and modelled permanent deformation at the top of the base (20 mm), subbase (255 mm) and subgrade (405 mm) are shown in Figure 9 for the two HVS tests. In addition to the poor modelling accuracy of SAMDM1996 for the base layer of test 417A5, the subgrade permanent deformation is also under-estimated significantly resulting in almost no permanent deformation being modelled for the duration of the test. Although the pavement structure permanent deformation is over-estimated by SAMDM2011 for test 417A5, the model response to increasing wheel-load is far more realistic than that of SAMDM1996. In the case of test 418A5 no permanent deformation other than that originating from the base layer during the wet portion of the test is predicted by SAMDM1996. SAMDM2011 shows an accumulation of permanent deformation at all levels in the pavement structure during the dry and wet portions of the tests that agrees with the observed permanent deformation behaviour.

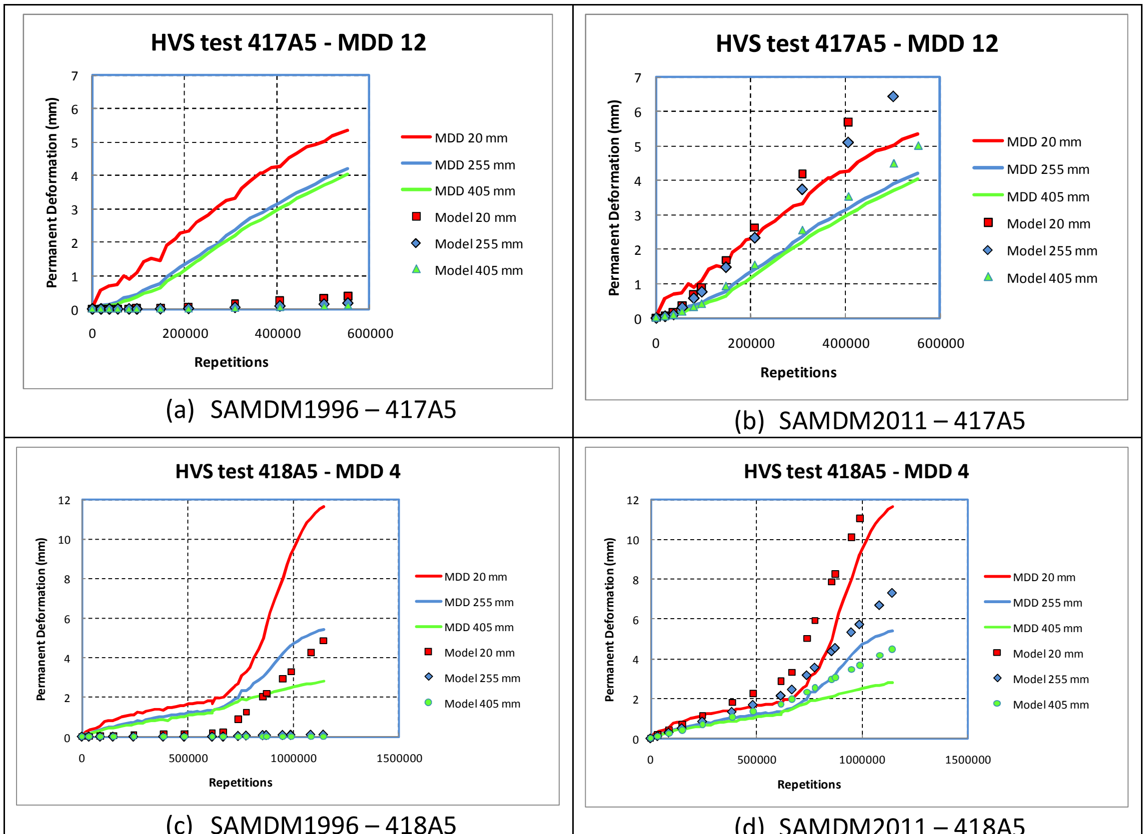


Figure 9. Pavement structure permanent deformation evolution for HVS tests 417A5 and 418A5

Table 8 provides a summary of the final deformation results recorded at three locations (MDD4, MDD8 and MDD12) on HVS test sections 417A5 and 418A5. In general, SAMDM1996 underestimates the base and total pavement deformation with the error in some cases exceeding 80 % and approaching 100 %. In all cases, the SAMDM1996 predicts almost no deformation from the subgrade which does not correspond with the observed data. Given this underestimation of permanent deformation, the structural capacity of the pavement will be overestimated significantly resulting in very high design risk. The sensitivity of SAMDM1996 to variations in the input variables is highlighted by the results for HVS test 417A5 with the base deformation being overestimated at MDD4 and underestimated at the other locations. Although the base and pavement deformation estimated by SAMDM2011 is not very accurate, the results from SAMDM2011 are consistent and more accurate than the SAMDM1996 results.

Table 8 Summary of results for the crushed stone pavement on the N7

			Deformation at the end of the each test		
			HVS	**SAMDM 1996**	**SAMDM 2011**
Test 417A5	**Base deformation**	**MDD4**	1.6	2.2 (+37.5%)	0.9 (-43.8%)
		MDD8	0.9	0.1 (-88.9%)	0.8 (-11.1 %)
		MDD12	1.1	0.2 (-81.8%)	0.8 (-27.3%)
	Pavement deformation	**MDD4**	5.4	2.2 (-59.3%)	4.8 (-11.1%)
		MDD8	5.9	0.2 (-96.6%)	9.2 (+55.9%)
		MDD12	5.4	0.4 (-92.6%)	8.0 (+48.1%)
Test 418A5	**Base deformation**	**MDD4**	6.2	4.8 (-22.6%)	7.5 (+21.0%)
		MDD8	3.9	3.2 (-17.9%)	6.6 (+69.2%)
		MDD12	5.2	4.4 (-15.4%)	7.5 (+44.2%)
	Pavement deformation	**MDD4**	11.7	4.8 (-58.9%)	14.8 (+26.5%)
		MDD8	8.0	3.3 (-58.8%)	13.8 (+72.5%)
		MDD12	9.6	4.5 (-53.1%)	14.0 (+45.8%)

5. CONCLUSIONS AND RECOMMENDATIONS

In addition to safeguarding against inappropriate (over- and under-designed) pavement designs, pavement design methods and specifically mechanistic-empirical design methods must provide realistic pavement performance information for long-term financial planning. In the process of modelling the pavement performance, the design method should not be overly sensitive to variation in secondary design inputs but should respond realistically to changes in the primary inputs that are known to govern the performance of pavements.

Jooste (2004) illustrated that the 1996 version of the South African Mechanistic-Empirical Design Method for flexible pavements (SAMDM1996) was overly sensitive to small changes in the resilient response parameters of pavement layers. As shown in this paper, the simulation of Heavy Vehicle Simulator (HVS) permanent deformation data for full-depth granular pavements also revealed that the models for unbound granular layers were not sufficiently sensitive to changes in loading conditions.

The paper presents revised permanent deformation damage models for unbound granular pavement layers. The internationally accepted vertical subgrade strain design model was found to be inadequate and replaced with a model based on the total subgrade deflection. The factor of safety model for unbound granular base and subbase layers was replaced with a stress ratio model with the distinction that the stress ratio is calculated from effective stress conditions, not only the stress associated with the external wheel-load. The effective stress includes the vertical overburden pressure, residual compaction stress, suction pressure and the stress resulting from the external wheel-load.

The revised method shows considerably less sensitivity to changes in the resilient response parameters of pavement layers compared to the SAMDM1996 models with appropriate responsiveness to changes in input variables such as wheel-load, density and saturation levels as well as material shear strength. Using the revised models, it was possible to simulate the development of the permanent deformation of a full-depth granular pavement with reasonable accuracy.

Although some of the known problems associated with the design models for unbound granular layers have been alleviated by the proposed revised models, the models for other material types are still under revision as part of the SANRAL SAPDM project to ensure that all materials are assessed according to true performance potential in the South African Mechanistic-Empirical Pavement Design Method.

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KEY WORDS

Mechanistic-Empirical design, unbound granular layers, effective stress analysis, damage models, linear recursive simulation.