Overview of South African Mechanistic Pavement Design Method

H. L. THEYSE, M. DE BEER, AND F. C. RUST

A historical overview of the South African mechanistic pavement design method, from its development in the early 1970s to the present, is presented. Material characterization, structural analysis, and pavement life prediction are discussed, and stiffness values are suggested for a range of materials in the absence of measured values. The modes of failure for these material types include the fatigue of asphalt material, deformation of granular material, crushing and effective fatigue of lightly cemented material, and deformation of selected and subgrade material. The critical parameters and transfer functions for these material types and modes of failure are discussed and included in the pavement life prediction process.

The South African mechanistic design method (SAMDM) and the development of certain components of the method have been published extensively since the 1970s. These discussed the mechanistic design approach (including material and pavement behavior, design traffic, desired service level, etc.) as well as the actual mechanistic analysis procedure.

The purpose of this study is to give an overview of the current mechanistic analysis procedure and not the complete mechanistic design approach. The study discusses the historical development and the procedure as it is used currently, including components of the procedure that have been developed recently. Figure 1 illustrates the basic mechanistic design analysis procedure.

The process starts with the load and material characterization, including layer thickness and elastic material properties for each layer in the pavement structure. The structural analysis will usually involve a linear elastic, static analysis of the multilayer system, resulting in the pavement response to the loading condition expressed in terms of stresses (σ) and strains (ϵ) at critical positions in the pavement structure.

The pavement response serves as input to the transfer functions for each material type, relating the stress-strain condition to the number of loads that can be sustained before a certain terminal condition is reached. This paper focuses on the material characterization, structural analysis, and transfer function components of the procedure currently used in South Africa.

HISTORICAL DEVELOPMENT OF SOUTH AFRICAN MECHANISTIC DESIGN METHOD

Details of the first simplified mechanistic design procedure in South Africa were presented in works locally (*1*) and internationally (*2*). At that stage no values for the characterization of the pavement materials were provided and it was suggested that material characterization should be done by laboratory and field testing for each design. Transfer functions were provided for the fatigue life of thin asphalt surfacing layers (*3*) but no transfer functions were provided for thick asphalt base layers. A fatigue transfer function for crack initiation of cemented material (*4–6*) was included. The only criterion provided for granular base layers was that the working stresses should be limited to 70 percent of the static shear strength or that the safe working stresses should be determined from repeated loading triaxial tests. The same criterion was suggested for the selected layers and subgrade material.

In addition to providing criteria for predicting material and pavement behavior, elastic properties were suggested for different road building materials in South Africa in 1978 (*7*). The fatigue criteria for thin asphalt layers remained the same as those given in 1977 but transfer functions were included for thick asphalt base layers (*7*).

The same fatigue criterion given for cemented material in 1977 was used. In addition, the concept of the safety factor for limiting the permanent deformation of granular material was based on work done by Maree (*8*). Criteria developed (*9*) for limiting the permanent deformation of the selected and subgrade material were also included.

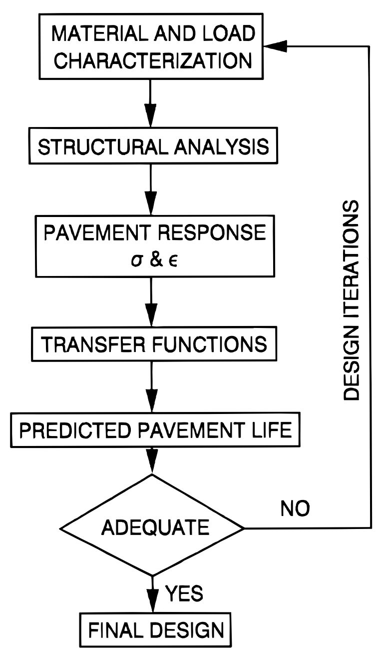


FIGURE 1 Flow diagram for mechanistic design analysis procedure.

CSIR, Division of Roads and Transport Technology, P.O. Box 395, Pretoria 0001, South Africa.

TABLE 1 Road Categories and Approximate Design Reliabilities Used in South Africa

Road Category	Description	Approximate design reliability (%)
A	Interurban freeways and major interurban roads	95
B	Interurban collectors and major rural roads	90
C	Rural roads	80
D	Lightly trafficked rural roads	50

During the early 1980s, attention was focused on the use of the SAMDM for new pavement design and rehabilitation design (*10,11*). At that stage the method had been developed and tested extensively through accelerated testing of pavements with the fleet of heavy vehicle simulators (HVSs) in South Africa. The transfer functions for asphalt material were extended to include fatigue transfer functions for thick asphalt base layers for a range of stiffness values (*12*).

The criteria for predicting the behavior of cemented and granular material remained the same as in 1978. However, the criteria set for limiting the permanent deformation of the selected and subgrade material (*9*) were modified according to work done at the U.S. Army Engineers Waterways Experiment Station (*13*). The design method was last updated in 1995 (*14,15*) to revise the South African catalogue of pavement designs (*16).*

Transfer functions were modified to include the approximate performance reliability required for the different service levels attached to the different road categories in South Africa as given in Table 1.

The concept of crushing in lightly cemented layers was introduced, based on observations under HVS accelerated pavement testing, while the original fatigue criterion for cemented layers was replaced by effective fatigue criteria (*17).*

The design method was calibrated extensively against the experience of road engineers from different road authorities in South Africa in the process of revising the catalogue of pavement designs (*16).*

MATERIAL CHARACTERIZATION FOR CURRENT SAMDM

The standard road building material classification for South Africa is summarized in Table 2 (*18*). The suggested stiffness values given in this section for these materials should only serve as a guideline to be used in the absence of laboratory or field measured values.

TABLE 2 South African Road-Building Materials with Material Codes

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
▽▽▽▽ ▽▽▽	G1	Graded crushed stone	Dense - graded unweathered crushed stone ; Max size 37,5mm; 88% apparent relative density; fines PI < 4.0 (min 6 tests)
▽▽▽▽ ▽▽▽	G2	Graded crushed stone	Dense - graded crushed stone ; Max size 37,5 mm ; 100 - 102% mod. AASHTO or 85% bulk relative density; fines PI < 6 (min 6 tests)
▽▽▽▽ ▽▽▽	G3	Graded crushed stone	Dense - graded stone and soil binder ; max size 37,5 mm, 98 - 100% mod. AASHTO ; fines PI < 6
	G4	Natural gravel	CBR* ≮ 80 ; max size 53mm ; 98 - 100% mod. AASHTO ; PI < 6 Swell 0,2 @ 100% mod. AASHTO.
	G5	Natural gravel	CBR* ≮ 45 ; max size 63mm ; or $\frac{2}{3}$ layer thickness , density as per prescribed layer of usage , PI < 10 ; Swell 0,5 @ 100% mod. AASHTO.
	G6	Natural gravel	CBR* ≮ 25 ; max size 63mm ; or $\frac{2}{3}$ layer thickness , density as per prescribed layer of usage , PI < 12 ; Swell 1,0 @ 100% mod. AASHTO.
	G7	Gravel-soil	CBR* ≮ 15 ; max size $\frac{2}{3}$ layer thickness , density as per prescribed layer of usage , PI < 12 or 2 GM + 10 ; Swell 1,5 @ 100% mod. AASHTO.
	G8	Gravel-soil	CBR ≮ 10 ; at insitu density ; max size $\frac{2}{3}$ layer thickness , density as per layer of usage , PI < 12 or 2 GM + 10 ; Swell 1,5 @ 100% mod. AASHTO.
	G9	Gravel-soil	CBR ≮ 7 ; at insitu density ; max size $\frac{2}{3}$ layer thickness , density as per layer of usage , PI < 12 or 2 GM + 10 ; Swell 1,5 @ 100% mod. AASHTO.
	G10	Gravel-soil	CBR ≮ 3 ; at insitu density ; max size $\frac{2}{3}$ layer thickness , density as per layer of usage , or 90% mod. AASHTO.

* CBR at field compaction density

GM: Grading Modulus

$$GM = \frac{P_{2,00mm} + P_{0,425mm} + P_{0,075mm}}{100}$$

where $P_{2,00\ mm}$ etc., denote the percentage retained on indicated sieve size.

(continued on next page)

TABLE 2 *(continued)*

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	C1	Cemented crushed stone or gravel	UCS 6 to 12 MPa at 100 % mod AASHTO ; spec. at least G2 before treatment ; dense - graded
	C2	Cemented crushed stone or gravel	UCS 3 to 6 MPa at 100 % mod. AASHTO ; spec. generally G2 or G4 before treatment ; dense - graded
	C3	Cemented natural gravel	UCS 1,5 to 3,0 MPa and ITS ≥ 250 kPa at 100 % mod. AASHTO ; max. size 63 mm ; fines PI ≤ 6 after stabilization.
	C4	Cemented natural gravel	UCS 0,75 to 1,5 MPa and ITS ≥ 200 kPa at 100 % mod. AASHTO ; max. size 63 mm ; fines PI ≤ 6 after stabilization.
	EBM	Bitumen Emulsion Modified gravel	0,6% - 1,5% residual bitumen
	EBS	Bitumen Emulsion Stabilised gravel	1,5% - 5,0% residual bitumen
	BC1	Hot - mix asphalt	Continuously - graded ; max. size 53 mm
	BC2	Hot - mix asphalt	Continuously - graded ; max. size 37,5 mm
	BC3	Hot - mix asphalt	Continuously - graded ; max. size 26,5 mm
	BS	Hot - mix asphalt	Semi - gap - graded ; max. size 37,5 mm
	PCC	Portland cement Concrete	Modulus of rupture $\nless$ 4,5 MPa ; max size $\ngtr$ 75 mm
	AG	Asphalt surfacing	Gap graded
	AC	Asphalt surfacing	Continuously graded
	AS	Asphalt surfacing	Semi-gap graded
	AO	Asphalt surfacing	Open graded
	AP	Asphalt surfacing	Porous (Drainage) Asphalt
	S1	Surface seal	Single seal
	S2	Surface seal	Multiple seal
	S3	Surface seal	Sand seal
	S4	Surface seal	Cape seal
	S5	Slurry	Fine grading
	S6	Slurry	Medium grading
	S7	Slurry	Coarse grading
	S8	Surface renewal	Rejuvenator
	S9	Surface renewal	Diluted emulsion
	WM1	Waterbound macadam	Max. size 75 mm, PI of fines $\ngtr$ 6, 88-90% of apparent density
	WM2	Waterbound macadam	Max. size 75 mm, PI of fines $\ngtr$ 6, 86-88% of apparent density
	PM	Penetration macadam	Coarse stone + keystone + bitumen
	DR	Dumprock	Upgraded waste rock, max size $\frac{2}{3}$ layer thickness

UCS : Unconfined Compressive Strength.
ITS : Indirect Tensile Strength.

Asphalt Material

Table 3 compares the elastic moduli suggested for asphalt materials in 1983 (*11*) with the values suggested in 1993 (*19*). The latter values are effective moduli, backcalculated from multidepth deflectometer deflection measurements, and the earlier values are compression moduli based on laboratory measurements. The value used for the Poisson's ratio of asphalt material is assumed to be 0.44 or as measured in the laboratory.

Cemented Material

Table 4 contains the suggested elastic moduli values for cemented material in different phases of material behavior (*20*). The value used for the Poisson's ratio of lightly cemented material is 0.35.

Granular Material

Suggested elastic moduli for granular material, including selected and subgrade material, are listed in Table 5 (*19,20*). The value used for the Poisson's ratio is 0.35.

STRUCTURAL ANALYSIS

The structural analysis is normally done with a static, linear elastic multilayer analysis program. The standard design load for South Africa is a 40-kN dual wheel load at 350-mm spacing between centers and a uniform contact pressure of 520 kPa.

The maximum horizontal tensile strain at the bottom of asphalt and cemented layers is used as the critical parameter determining the fatigue life of these two material types. While the maxi-

TABLE 3 Elastic Moduli for Asphalt Layers

Material grading	Depth from surface (mm)	Stiffness values (MPa) based on temperature and material condition					
		Good condition or new material		Stiff, dry mixture		Very cracked condition	
		20° C	40° C	20° C	40° C	20° C	40° C
Values suggested by Freeme (11) in 1983							
Gap-graded	0 - 50	4000	1500	5000	1800	1000	500
	50 - 150	6000	3500	7000	4000	1000	500
	150 - 250	7000	5500	8000	6000	1000	500
Continuously graded	0 - 50	6000	2200	7000	4000	750	500
	50 - 150	8000	5500	9000	6000	1000	750
	150 - 250	9000	7500	10000	8000	1000	750
Values suggested by Jordaan (19) in 1993							
Gap-graded	0 - 50	1000	200	2000	300	600	200
	50 - 150	2000	300	3000	400	750	300
	150 - 250	3000	400	4000	500	800	400
Continuously graded	0 - 50	2000	300	3000	300	750	300
	50 - 150	4000	400	5000	600	800	400
	150 - 250	6000	1000	7000	1500	1000	750

TABLE 4 Suggested Elastic Moduli Values for Cemented Material

Original Code	UCS (MPa) for pre-cracked condition	Parent Material Code	Pre-cracked condition		Post-cracked condition			
			Phase 1		Phase 2	Phase 3		
			Stage 1: Intact (GPa)	Stage 2: Shrinkage cracking (MPa)	Stage 3: Traffic associated cracking, transitional phase with micro cracking (MPa)	Stage 4: Broken up in equivalent granular state (Mpa)		
						Dry condition	Wet condition	Equivalent code
C1	6 - 12	Crushed stone G1 Crushed stone G3	6 - 30	2500 - 3000	800 - 1000	400 - 600	50 - 400	EG1 EG2
C2	3 - 6	Crushed stone G2 Crushed stone G3 Gravel G4	3 - 14	2000 - 2500	500 - 800	300 - 500	50 - 300	EG2 EG3 EG4
C3	1.5 - 3	Gravel G4 Gravel G5 Gravel G6 Gravel G7 Gravel G8	2 - 10	1000 - 2000	500 - 800	200 - 400	20 - 200	EG4 EG5 EG6 EG7 EG8
C4	0.75 - 1.5	Gravel G4 Gravel G5 Gravel G6 Gravel G7 Gravel G8 Gravel G7 Gravel G8	0.5 - 7	500 - 2000	400 - 600	100 - 300	20 - 200	EG4 EG5 EG6 EG7 EG8 EG9 EG10

TABLE 5 Suggested Ranges of Elastic Moduli for Granular Materials (MPa)

Material Code	Material Description	Dry condition		Wet condition	
		Over cemented layer in slab state	Over granular layer or equivalent	Over cemented layer in slab state	Over granular layer or equivalent
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)
G7	Gravel - Soil	30 - 200		20 - 120	
G8	Gravel - Soil	30 - 180		20 - 90	
G9	Soil	30 - 140		20 - 70	
G10	Soil	20 - 90		10 - 45	

mum tensile strain in a particular layer will not necessarily occur at the bottom of the layer (*21,22*), the position of maximum tensile strain is determined by the modular ratios of the pavement layers. The transfer functions for these materials were, however, developed as a function of tensile strain at the bottom of the layer and are used as such.

Very often, the linear elastic analysis of a pavement with a granular base and subbase will predict a tensile stress in the granular subbase, resulting in almost no resistance against shear failure predicted by the safety factor approach. The occurrence of tensile stress in a granular layer is determined by the ratio of the granular layer stiffness to the subgrade stiffness (*23,24*) and is caused by the linear elastic model using the same modulus for tension and compression. The linear elastic model and the Mohr circle representation of a typical stress condition in a granular subbase are illustrated in Figure 2 (a) and (b).

A possible solution for this problem is to use a model with different tension and compression moduli as illustrated in Figure 2 (c). Direct solution, linear elastic analysis packages cannot accommodate such material models, and finite element packages will have to be considered.

Although the current SAMDM does not use the model illustrated in Figure 2 (c), an adjustment is made to the major and minor stresses calculated by linear elastic analysis to exclude tensile stress during the calculation of the safety factor against shear failure. If a tensile minor principle stress is calculated in a granular material, the value is set equal to zero and the major principle stress is adjusted under the condition that the deviator stress remain constant. This stress state is represented by the Mohr circle in Figure 2(d).

PAVEMENT LIFE PREDICTION

Two concepts are involved in pavement life prediction. The first is to predict the individual layer life for each of the pavement layers and, second, the ultimate pavement life is predicted for the layered system.

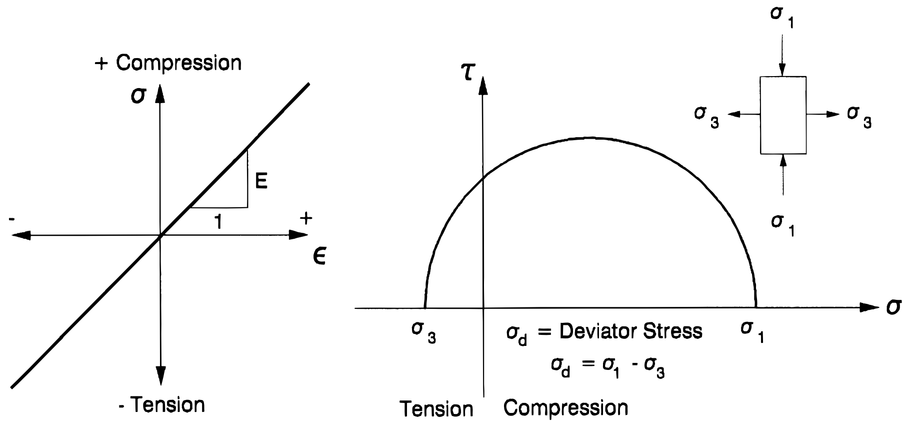
Layer Life Prediction

The basic material types considered are asphalt, cemented, granular, and subgrade materials. Each material type exhibits a unique mode of failure linked to critical parameters calculated at specific positions in the pavement structure under loading. Transfer functions provide the relationship between the value of the critical parameter and the number of load applications that can be sustained at that value of the critical parameter, before the particular material type will fail in a specific mode of failure.

The following sections will describe each basic material type with its accompanying critical parameters, modes of failure, and applicable transfer functions.

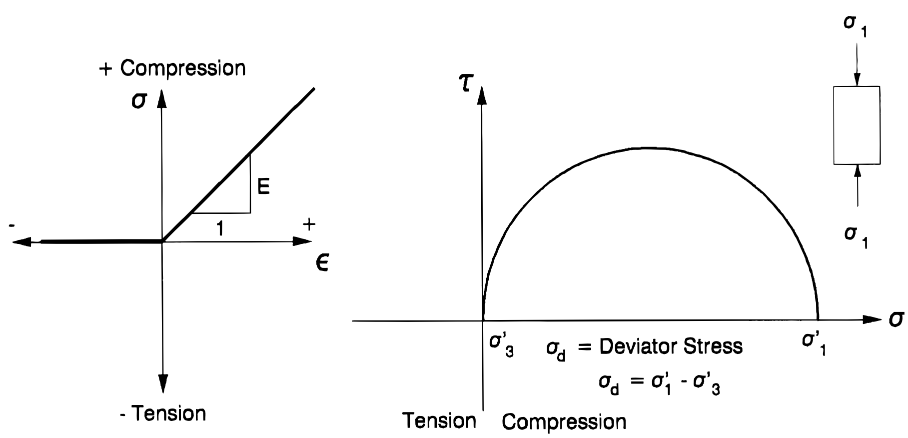
Asphalt Material

Asphalt material fails because of fatigue cracking under repeated loading as a result of tensile strain (ϵ_t) at the bottom or in the layer. A distinction is made between thin asphalt surfacing layers (<50 mm) and thick asphalt bases (>75 mm). Transfer functions are provided for continuously and gap graded surfacing layers and asphalt base layers with stiffnesses from 1000 to 8000 MPa. The fatigue crack initiation transfer functions for asphalt surfacing layers are illustrated in Figure 3 (*14*).



(a) Linear elastic material model

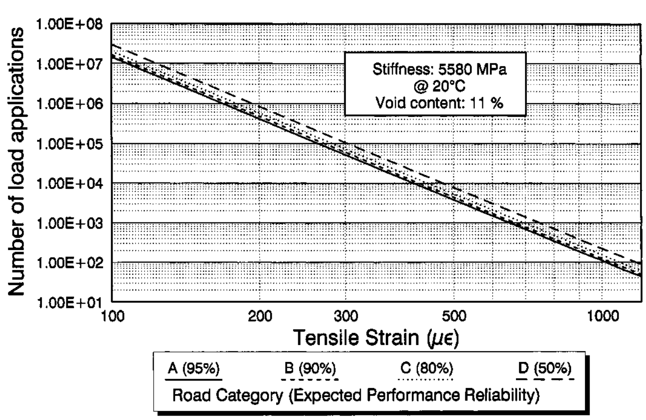
(b) Mohr stress circle representation of calculated stresses in a granular sub-base



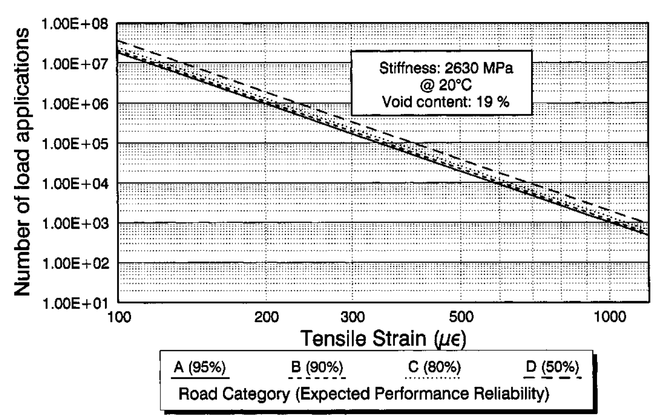
(c) No tension elastic material model

(d) Mohr stress circle representation of adjusted stresses in a granular sub-base

FIGURE 2 Elastic material models and stress conditions in granular subbases.



(a) Continuously Graded



(b) Gap Graded

FIGURE 3 Fatigue crack initiation transfer functions for thin asphalt surfacing layers.

Figure 4 illustrates the fatigue crack initiation transfer functions for thick asphalt bases. Figure 5 shows the shift factor to convert the crack initiation life to the total fatigue life after surface cracks appear on the road surface. The total asphalt depth should be considered to determine the shift factor.

Cemented Material

Cemented material may exhibit two failure modes, namely effective fatigue and crushing (*17*). The critical parameters for cemented material are (*a*) maximum tensile strain (ϵ) at the bottom of the layer controlling the effective fatigue life and (*b*) vertical compressive stress (σ_v) on top of the cemented layer controlling crushing life.

The effective fatigue transfer functions for cemented materials are illustrated in Figure 6 (*14*). The default values suggested for the strain at break ϵ_b (με) and the unconfined compressive strength (UCS) (kPa), of cemented material are given in Table 6.

These transfer functions (Figure 4) do not allow different layer thicknesses. A shift factor for cemented material was therefore introduced, allowing thicker layers to have an extended effective fatigue life compared with thinner layers subjected to the same strain. This shift factor is illustrated in Figure 7.

Transfer functions are provided for two crushing conditions, namely crush initiation with roughly 2-mm deformation on top of

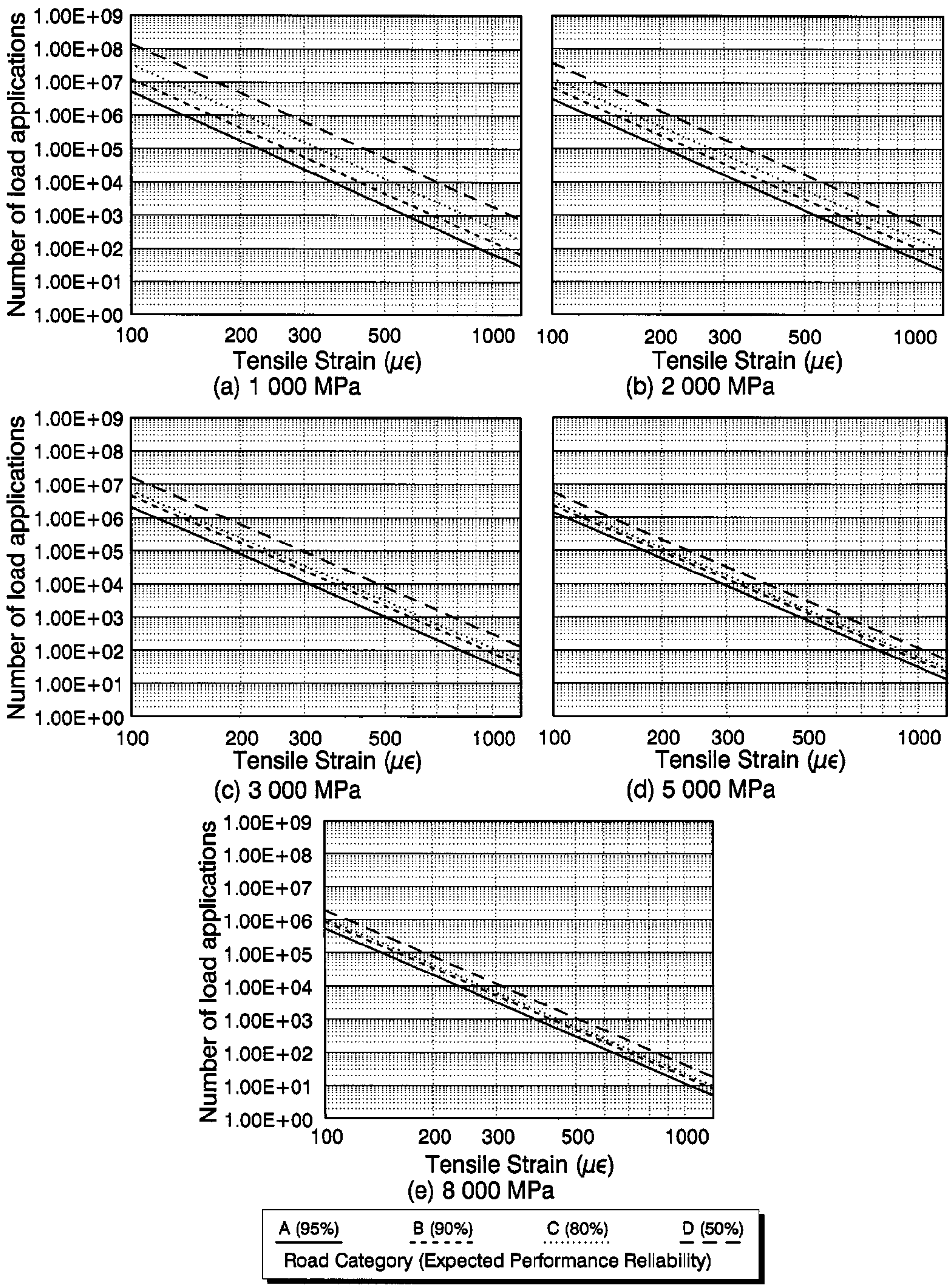


FIGURE 4 Fatigue crack initiation transfer functions for thick apshalt base layers.

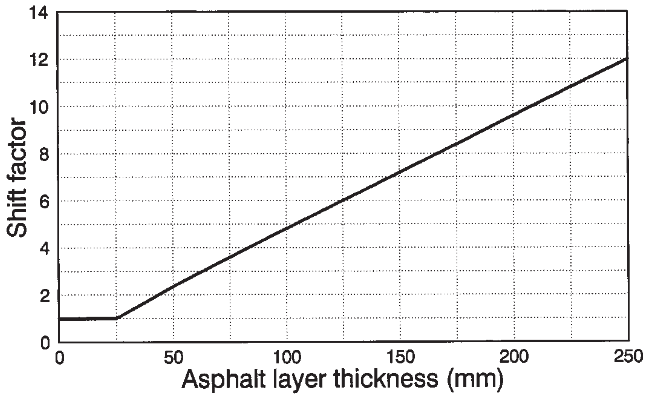


FIGURE 5 Fatigue crack propagation shift factor for asphalt layers.

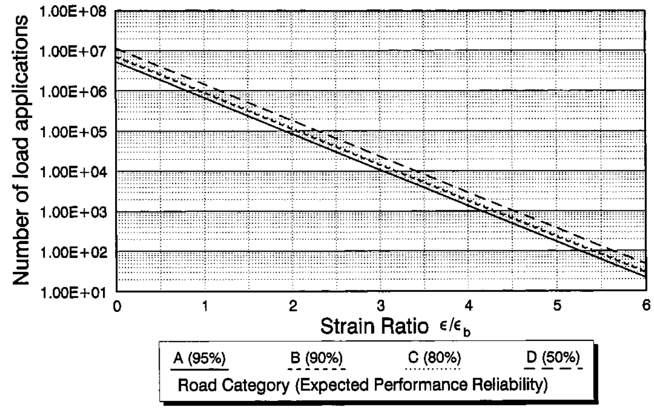


FIGURE 6 Effective fatigue life transfer functions for cemented material.

the layer and advanced crushing with 10-mm deformation and extensive breakdown of the cemented material. Figure 8 (*14*) illustrates the crush initiation (N_{Ci}) and advanced crushing (N_{Ca}) transfer functions for cemented material.

Granular Material

Granular material exhibits deformation caused by densification and gradual shear under repeated loading. The safety factor against shear failure for granular materials used in the SAMDM (Equation 1) was developed from Mohr-Coulomb theory for static loading and represents the ratio of the material shear strength divided by the applied stress causing shear (*8*).

$$F = \frac{\sigma_3\left[K\left(\tan^2\left(45+\frac{\phi}{2}\right)-1\right)\right]+2KC\tan\left(45+\frac{\phi}{2}\right)}{(\sigma_1-\sigma_3)} \tag{1}$$

or

$$F = \frac{\sigma_3\phi_{\text{term}} + c_{\text{term}}}{(\sigma_1 - \sigma_3)} \tag{2}$$

where

- σ_1 and σ_3 = major and minor principal stresses acting at point in granular layer (compressive stress positive and tensile stress negative),
- C = cohesion,
- ϕ = angle of internal friction, and
- K = constant = 0.65 for saturated conditions, 0.8 for moderate moisture conditions, and 0.95 for normal moisture conditions.

Maree found that at values of the safety factor below a certain critical value the permanent deformation of granular material will increase rapidly under a few load applications because of shear failure; and at values above the critical value the permanent deformation increases gradually with increasing load applications. In both instances, however, the mode of failure will be the deformation of the granular layer, and the rate of deformation is controlled by the magnitude of the safety factor against shear failure.

The major and minor principal stresses, and hence the safety factor, are usually calculated at the midpoint of granular layers. Suggested values of the C and ϕ terms for granular materials are given in Table 7.

The transfer functions, relating the safety factor to the number of load applications that can be sustained at that safety factor level, are illustrated in Figure 9.

Subgrade Material

The mode of failure for the selected and subgrade material is the permanent deformation of these layers, resulting in the deformation of the road surface. The critical parameter for these materials is the vertical strain (ϵ_v) on top of the layer. Transfer functions are provided for two terminal conditions, a 10- or 20-mm surface rut caused by the deformation of the selected or subgrade material (Figure 10) (*14*).

TABLE 6 Suggested Values of ϵ_b and UCS for Cemented Material

Material code	ϵ_b ($\mu\epsilon$)	UCS (kPa)
C1	145	7500
C2	120	7500
C3	125	2250
C4	145	1125

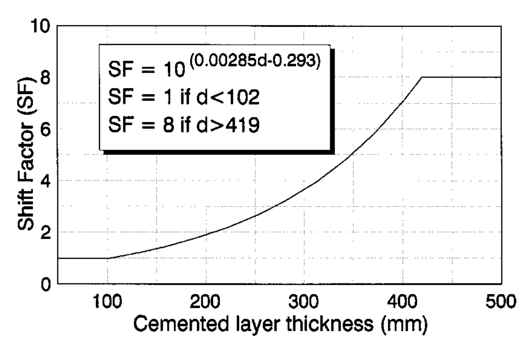


FIGURE 7 Shift factor for effective fatigue life of cemented material.

Pavement Life Prediction

After the layer life for each individual pavement layer is predicted from the transfer functions in the previous section, they should be combined to predict the ultimate life for the layered system. If there are no cemented layers in the pavement structure, the ultimate pavement life is determined by the shortest individual layer life predicted. If cemented layers are incorporated in the pavement, distinct phases in the pavement life cycle may be identified and should be included in the ultimate pavement life prediction.

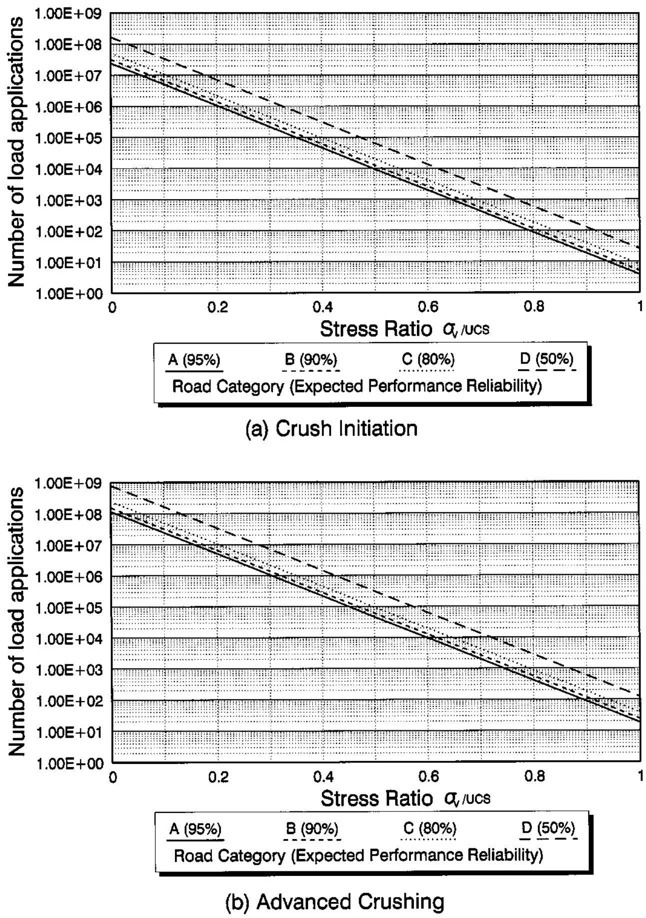


FIGURE 8 Crushing life transfer functions for lightly cemented material.

Modeling Long-Term Behavior of Cemented Layers

Figure 11 illustrates the long-term behavior of a lightly cemented layer in a pavement structure. During the precracked phase, the elastic modulus of the layer will be in the order of 3000 to 4000 MPa, and the layer will act as a slab with the slab dimensions a few times larger than the layer thickness. This *E*-value reduces rapidly to values in the order of 1500 to 2000 MPa at the onset of the effective fatigue life phase during which the layer is broken down from large blocks, with dimensions of approximately one to five times the layer thickness, to particles smaller than the thickness of the layer. During the equivalent granular phase the elastic modulus is in the order of 200 to 300 MPa, and the cemented material acts typically like a granular layer. The effective fatigue life phase and equivalent granular phase of cemented material are used to calculate the layer life for the cemented layer. The precracked phase is considered very short (*17*) in relation to the other phases and is therefore not included in predicting the layer life for the cemented layer.

Although these changes in the behavior of the cemented material will gradually occur with time, they are modeled as stepwise phases in the life of a cemented layer. The modulus of a cemented layer is modeled as a constant value for the duration of a particular phase with a sudden change at the end of each phase. Such a reduction in stiffness of a cemented layer will result in a redistribution of the initial calculated stresses and strains in the layered system with a reduction in the layer life predicted initially for the other pavement layers.

The first cemented layer introduces two phases in the pavement life prediction process and the rest of the cemented layers introduce one phase each, as illustrated in Figure 12 (a) and (b).

Calculating Ultimate Pavement Life

Consider the situation in Figure 13 in which the stresses and strains calculated for each layer during each phase will yield a predicted

TABLE 7 Suggested C_{term} and ϕ_{term} Values for Granular Material

Material Code	Moisture Condition					
	Dry		Moderate		Wet	
	φ-term	C-term	φ-term	C-term	φ-term	C-term
G1	8.61	392	7.03	282	5.44	171
G2	7.06	303	5.76	221	4.46	139
G3	6.22	261	5.08	188	3.93	115
G4	5.50	223	4.40	160	3.47	109
G5	3.60	143	3.30	115	3.17	83
G6	2.88	103	2.32	84	1.76	64
EG4	4.02	140	3.50	120	3.12	100
EG5	3.37	120	2.80	100	2.06	80
EG6	1.63	100	1.50	80	1.40	60

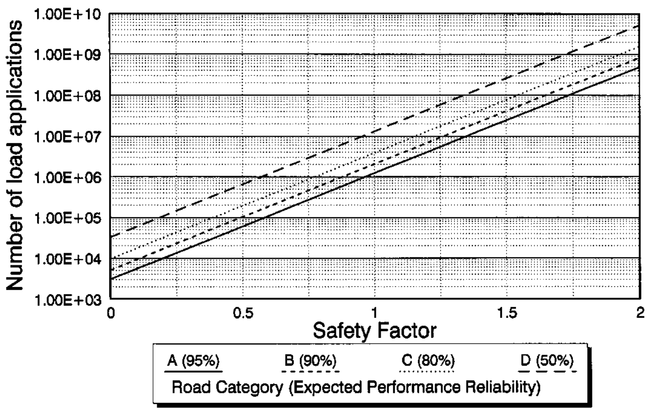


FIGURE 9 Transfer functions for granular material.

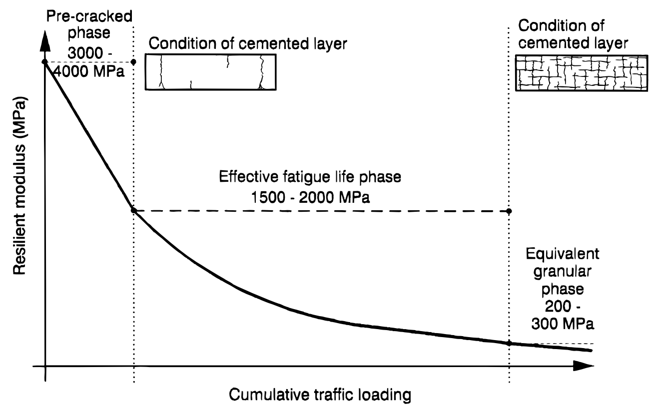


FIGURE 11 Long-term behavior of lightly cemented material.

layer life for each layer during each phase. At the end of Phase 1, the modulus of the cemented layer is suddenly reduced, resulting in higher stress-strain conditions in the other layers, similar to an increase in the load on the pavement. The remaining part of the Phase 1 predicted layer life for the other layers, or the residual life of the other layers, is then reduced because of the increased stress conditions. The method assumes that the rate of decrease in the residual life of the other layers during the second phase is equal to the ratio of the Phase 1 predicted layer life to the Phase 2 predicted layer life for a particular layer, similar to a load equivalency factor. The only exception is the cemented layer that will start with a clean sheet for the second phase because of a change in material state and therefore terminal condition. The predicted equivalent granular layer life for the original cemented layer will therefore be allocated to the cemented layer in total for the second phase. Also note that if the top layer is a surfacing layer such as a surface seal or thin asphalt layer, the predicted layer life for the top layer will not affect the ultimate pavement life. The reason is that surface maintenance should be done at regular intervals, and it is not possible to design the thin asphalt surfacing layers for the total structural design life of the pavement structures, especially for high design traffic classes.

The ultimate pavement life is calculated as the sum of the duration of Phase 1 and the minimum adjusted residual life for Phase 2

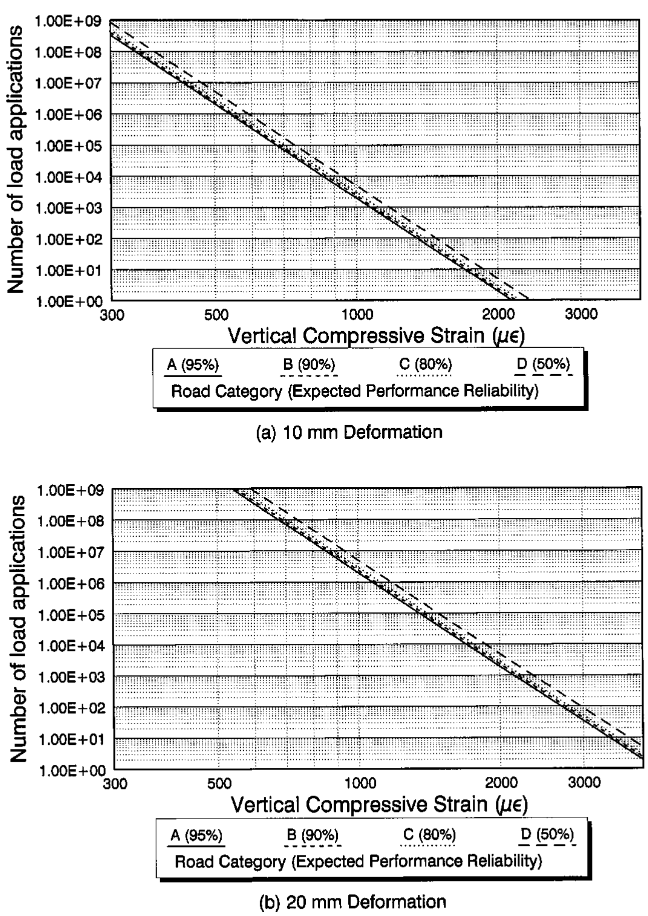


FIGURE 10 Subgrade deformation transfer functions.

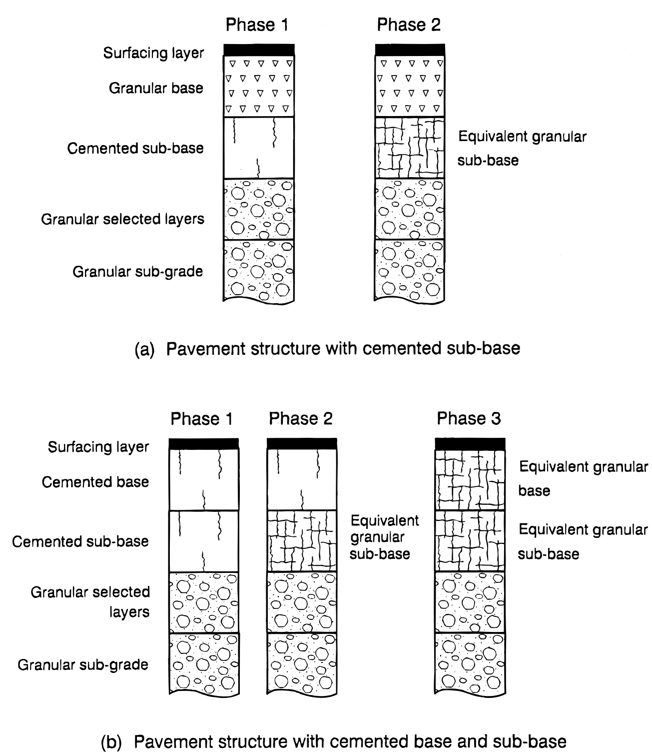


FIGURE 12 Pavement life phases.

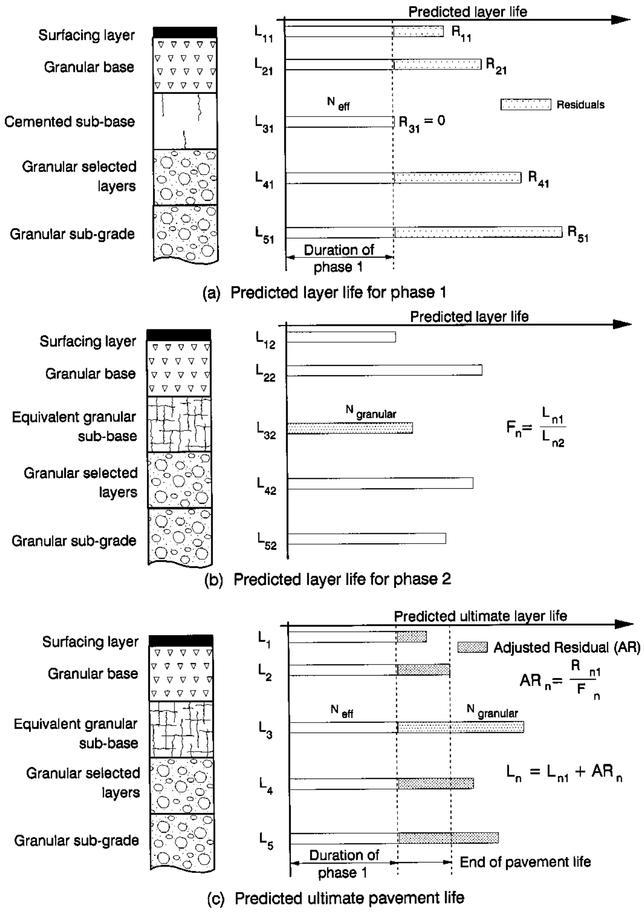


FIGURE 13 Calculation of ultimate pavement life for pavement structure with cemented layers.

or the Phase 2 predicted equivalent granular layer life for the original cemented layer, whichever is the smaller. The process is extended along similar principles for a three-phase analysis of a pavement structure incorporating two cemented layers. In addition to calculating the ultimate pavement life, pavement structures incorporating a cemented base must also be checked for possible crushing of the cemented material under the surfacing layer.

If only a cemented base layer is used in the pavement structure, crush initiation and advanced crushing will occur only if the predicted crush initiation or advanced crushing life exceeds the predicted effective fatigue layer life. On the other hand, if a cemented base and subbase are used, the effective fatigue life predicted initially for the cemented base will reduce at the end of the effective fatigue life of the subbase as a result of an increase in the tensile strain at the bottom of the cemented base. The crushing failure life for the base, however, will remain more or less the same as the vertical stress at the top of the base depends largely on the applied vertical stress and not so much on the support conditions below. After reducing the effective fatigue life of the cemented base according to the calculation illustrated in Figure 13, the same test for crushing failure as described is applied.

CONCLUSION

The SAMDM has been used for new and rehabilitation pavement design since the 1970s. The development and verification of the method was assisted by accelerated pavement testing done with heavy vehicle simulators. The most recent development in the SAMDM has been the introduction of some measure of design reliability in the transfer functions contained in the method. This latest version of the SAMDM has been used to develop standard pavement designs for different road categories contained in a catalogue for the design of interurban and rural roads on a national level and has been calibrated against the experience of road engineers from various road authorities in South Africa during this process.

Current research is aimed at converting SAMDM from a critical layer approach to a system approach, in which each pavement layer will contribute to the total permanent deformation of the pavement structure. The use of nonlinear analysis models will also be investigated with the emphasis on obtaining methods more suited to calculating the stress and strain condition in granular layers.

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Publication of this paper sponsored by Committee on Flexible Pavement Design.