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Overview of the South African Mechanistic Pavement Design Analysis Method

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Abstract:

This paper presents a historical overview on the development of the South African Mechanistic Pavement Design Analysis Method as well as the method as it is used currently. The development of the method is briefly traced since the early 1970's. The material characterization, structural analysis and pavement life prediction are discussed for the current method. Suggested stiffness values are provided for asphalt, granular, cemented, selected and sub-grade material in the absebce of laboratory or field 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 sub-grade 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.

Keywords: South African Mechanistic Design Method, transfer functions, failure criteria, cemented materials, granular materials, subgrades

Proposals for implementation:

Formal and informal

Related documents (eg software, interim or other reports, working drawings etc):

Signatures:

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INTRODUCTION

The South African Mechanistic Design Method (SAMDM) and the development of certain components of the method have been published extensively over the last few decades. The previous publications discussed the approach related to the mechanistic design method (including material and pavement behaviour, design traffic, desired service level etc.) as well as the actual mechanistic design analysis procedure. The purpose of this document is to give an overview of the current mechanistic **design analysis** procedure and not the complete mechanistic design method. This paper discusses the historical development of the method as well as 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 off with the load and material characterization. The standard design load for South Africa is a 40 kN dual wheel load at 350 mm spacing between centres and a uniform contact pressure of 520 kPa due to the legal axle load of 80 kN allowed on public roads.

The material characterization includes layer thickness and elastic material properties for each layer in the pavement structure under consideration. The structural analysis will usually involve a linear elastic, static analysis of the multi layer system. Resulting in the pavement response to the loading condition expressed in terms of stresses ( ) and strains ( ) at critical positions in the pavement structure determined by the material type used in each layer of the pavement structure.

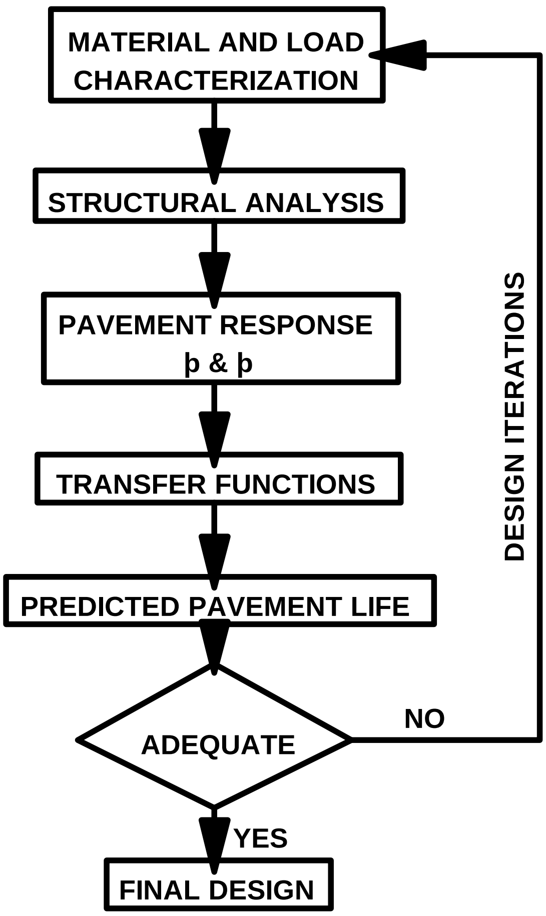


FIGURE 1: FLOW DIAGRAM FOR A MECHANISTIC DESIGN ANALYSIS PROCEDURE

The pavement response serves as input to the transfer functions for each material type. The transfer functions relate the stress/strain condition to the number of loads that can be sustained at that stress/strain level 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 THE SOUTH AFRICAN MECHANISTIC DESIGN METHOD

The first simplified mechanistic design procedure in South Africa was developed by Van Vuuren, Otte and Paterson (1) during 1974. The first comprehensive statement of the state of the art on mechanistic pavement design in South Africa is contained in a paper at the 1977 International Conference on the Structural Design of Asphalt Pavements by Walker, Paterson, Freeme and Marais (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 after Freeme and Marais (3) as a function of the maximum horizontal tensile strain at the bottom of the asphalt layer. No transfer functions were provided for thick asphalt base layers at that time. A fatigue transfer function for crack initiation of cemented material as a function of the maximum tensile strain at the bottom of the layer, developed by Otte (4,5,6) was included. The only criteria provided for granular base layers was that the working stresses should be limited to 70% of the static shear strength or that the safe working stresses should be determined from repeated loading triaxial tests. The same criteria were suggested for the selected layers and subgrade material.

In addition to providing criteria for predicting material and pavement behaviour Paterson and Maree (7) in 1978 suggested elastic properties for different road building materials in South Africa. The fatigue criteria for thin asphalt layers remained the same as those given by Walker et al (2) but Paterson and Maree (7) included transfer functions for thick asphalt base layers. The fatigue criteria for cemented material provided by Walker et al (2) were also used by Paterson and Maree (7). The concept of the safety factor for limiting the permanent deformation of granular material was introduced based on work done by Maree (8). The safety factor is calculated from the major and minor principle stresses at the midpoint of the granular layer as an indication of the ratio of the material shear strength to the working shear stress. Criteria developed by Paterson (9) for limiting the permanent deformation of the selected and subgrade material as a function of the vertical compressive strain at the top of these layers were included.

During 1981, Maree and Freeme (10) and 1983, Freeme (11) reported on the use of the South African Mechanistic Design Method for new pavement design and rehabilitation design. At that stage the method had been developed and tested extensively mainly through accelerated testing of pavements with the fleet of Heavy Vehicle Simulators (HVS's) in South Africa. The transfer functions for asphalt material were extended to include fatigue transfer functions for thick asphalt base layers as a function of the maximum horizontal tensile strain at the bottom of the layer for a range of stiffness values (1000 to 8000 MPa) after Freeme and Strauss (12). The criteria for predicting the behaviour of cemented and granular material remained the same as listed previously by Paterson and Maree (7). The criteria for limiting the permanent deformation of the selected and subgrade material were based on the criteria set by Paterson (9) and work done at the U. S. Army Engineers Waterways Experiment Station (WES) by Brabston, Barker and Harvey (13).

The South African Mechanistic Design Analysis Method was updated in 1995 by Theyse (14,15) for the purpose of revising the TRH4 1985: 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.

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

The concept of crushing in lightly cemented layers as a function of the vertical stress at the top of the cemented layers was introduced based on accelerated pavement testing (using a HVS) by De Beer (17). The original fatigue criteria for cemented layers by Otte (4,5,6) were also replaced by the effective fatigue criteria developed by De Beer (17). The design analysis method was calibrated extensively against the experience of road engineers from different road authorities in South Africa in the process of revising the TRH4 1985: Pavement Design Catalogue (16).

Although the development of the various components of the South African Mechanistic Design Analysis Procedure and more specifically the transfer functions included in the method is an ongoing process, the formal updating and comprehensive reporting of the method seems to be linked to the revision of the TRH4 Pavement Design Catalogue. The method as reported by Walker et al (2) in 1977 and Paterson and Maree (7) in 1978 was used in the development of the pavement design catalogue for TRH4 (1978). The method as reported by Maree and Freeme (10) in 1981 was used to revise the pavement design catalogue for TRH4 (1985) and the method updated in 1995 by Theyse was used for the revision of the catalogue for Draft TRH4 (1995).

MATERIAL CHARACTERIZATION FOR THE CURRENT SAMDAP.

The standard road building materials for South Africa as discussed in TRH14 (1985): Guidelines for Road Construction Materials (18) with their material codes are listed in Table 2. The suggested stiffness values contained in this section should only serve as a guideline to be used in the absence of laboratory or field measured values.

Asphalt Material

Freeme (11) suggested the elastic moduli for asphalt material listed in Table 3. Jordaan (19) suggested the values listed in Table 4 based on elastic moduli back-calculated from Multi-depth Deflectometer (MDD) deflection measurements. The values are considerably less than the values listed by Freeme due to the fact that the second set of values was obtained from back-calculation of field deflections. There is still some uncertainty over which approach to use (laboratory versus field values) and the values listed by Freeme are still preferred until the issue is resolved. The value used for the Poison's Ratio of asphalt material is assumed to be 0.44 or as measured in the laboratory.

Cemented Material

Table 5 contains the suggested elastic moduli values for cemented material in different phases of material behaviour after De Beer (20). The value used for the Poison's Ratio of cemented material is 0.35.

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	G1	Graded crushed stone	Dense - graded unweathered crushed stone ; Max size 37,5mm; 88% apparent relative density; fines PI
	G2	Graded crushed stone	Dense - graded crushed stone ; Max size 37,5 mm ; 100 - 102% mod. AASHTO or 85% bulk relative density; fines PI (min 6 tests)
	G3	Graded crushed stone	Dense - graded stone and soil binder ; max size 37,5 mm, 98 - 100% mod. AASHTO ; fines PI
	G4	Natural gravel	CBR* I Swell 0,2 @ 100% mod. AASHTO.
	G5	Natural gravel	CBR* I 2/3 per prescribed layer of usage , PI Swell 0,5 @ 100% mod. AASHTO.
	G6	Natural gravel	CBR* I 2/3 per prescribed layer of usage , PI Swell 1,0 @ 100% mod. AASHTO.
	G7	Gravel-soil	CBR* I 2/3 prescribed layer of usage , PI Swell 1,5 @ 100% mod. AASHTO.
	G8	Gravel-soil	CBR I 2/3 as per layer of usage , PI Swell 1,5 @ 100% mod. AASHTO.
	G9	Gravel-soil	CBR I 2/3 as per layer of usage , PI Swell 1,5 @ 100% mod. AASHTO.
	G10	Gravel-soil	CBR I 2/3 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.

TABLE 2:SOUTH AFRICAN ROAD BUILDING MATERIALS WITH THEIR MATERIAL CODES

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 þ 250 kPa at 100 % mod. AASHTO ; max. size 63 mm ; fines PI þ 6 after stabilization.
	C4	Cemented natural gravel	UCS 0,75 to 1,5 MPa and ITS þ 200 kPa at 100 % mod. AASHTO ; max. size 63 mm ; fines PI þ 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 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 > 6, 88-90% of apparent density
	WM2	Waterbound macadam	Max. size 75 mm, PI of fines > 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.

TABLE 2:SOUTH AFRICAN ROAD BUILDING MATERIALS WITH THEIR MATERIAL CODES (CONTINUED)

TABLE 3:ELASTIC MODULI SUGGESTED BY FREEME (11) 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
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

TABLE 4:ELASTIC MODULI SUGGESTED BY JORDAAN (19) 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
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

Granular Material

The suggested elastic moduli for granular material are listed in Table 6 (Jordaan (19) and De Beer (20)). The value used for the Poison's Ratio of granular material is 0.35.

TABLE 5: 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 6: SUGGESTED RANGES OF ELASTIC MODULI FOR GRANULAR MATERIAL S (MPA) WITH EXPECTED VALUES INDICATED IN BRACKETS.

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

Selected layers and subgrade material

The suggested elastic moduli for selected layers and subgrade material are listed in Table 7 (Jordaan (19)). The value used for the Poison's Ratio of these material is 0.35.

TABLE 7: SUGGESTED ELASTIC MODULI FOR SELEC TED AND SUBGRADE MATERIAL (MPA).

Material Code	Soaked CBR	Material Description	Suggested elastic moduli	
			Dry condition	Wet condition
G7	$\geq$15	Gravel - Soil	30 - 200	20 - 120
G8	$\geq$ 10	Gravel - Soil	30 - 180	20 - 90
G9	$\geq$ 7	Soil	30 - 140	20 - 70
G10	$\geq$3	Soil	20 - 90	10 - 45

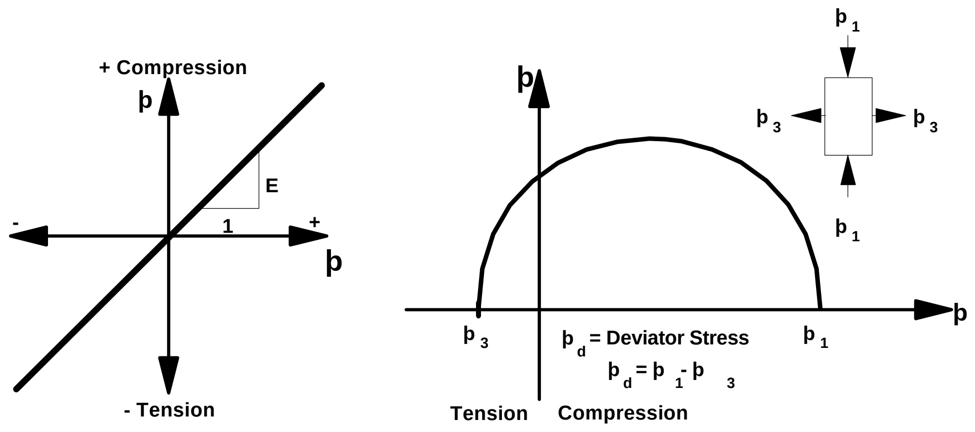
STRUCTURAL ANALYSIS

The structural analysis is normally done with a static, linear elastic multi layer analysis program. A few points related to the structural analysis that will influence the design analysis procedure should be noted.

The maximum horizontal tensile strain at the bottom of asphalt layers and the maximum tensile strain at the bottom of cemented layers are used as the critical parameters determining the fatigue life of these two material types. The position of the maximum tensile strain in a particular layer will not necessarily occur at the bottom of the layer (21,22). The position of the maximum horizontal strain will rather be determined by the modular ratios of the layers in the pavement structure. 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 structural analysis of a pavement with a granular base and sub-base will result in tensile stresses developing in the granular sub-base, resulting in almost no resistance against shear failure predicted by the mechanistic design method. This is caused by the linear elastic material models used in the static, linear elastic analysis allowing tensile stresses in the granular sub-bases resulting in very low safety factors. The occurrence of tensile stress in a granular layer is again determined by the modular ratio of the granular layer in relation to the subgrade (23,24). The linear elastic model and the resulting Mohr circle for such a case is illustrated in Figure 2.

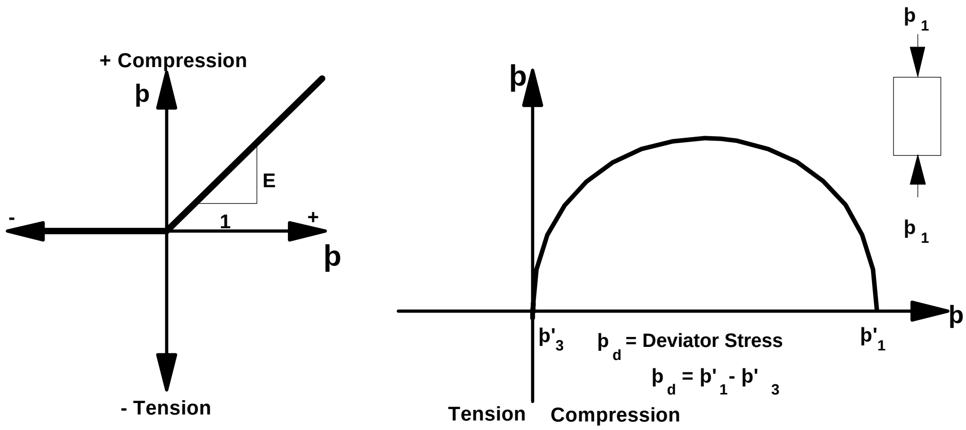
An intermediate solution is not to allow any tensile stress to develop in granular materials. If a tensile minor principle stress is calculated in a granular material, the value is set equal to zero. What this implies in practice is that the granular layer will only carry loading in compression. If the minor principle stress is set equal to zero, a re-arrangement of stresses will take place to transfer the loads by compression. The major principle stress is adjusted under the condition that the deviator stress remain constant. The Mohr circle is therefore shifted as indicated in Figure 3. Although this tentative adjustment of stresses has not been proven theoretically, it does provide more meaningful pavement designs compared to proven practice. In such cases, the correct solution is to use a material model as illustrated in Figure 3 rather than the model in Figure 2. Current linear elastic analysis packages do however, not allow for such material models and research is being conducted on the finite element analysis of pavement structures with no tension allowed in granular materials.



(a) Linear elastic material model

(b) Mohr stress circle representation of stress state

FIGURE 2: CONVENTIONAL LINEAR ELASTIC MATERIAL MODEL WITH RESULTING STRESS STATE IN GRANULAR SUB-BASES.



(a) No tension elastic material model (b) Mohr stress circle representation of stress state

FIGURE 3: SUGGESTED LINEAR ELASTIC MATERIAL MODEL WITH RESULTING STRESS STATE IN GRANULAR SUB-BASES.

PAVEMENT LIFE PREDICTION

Three concepts are involved in the pavement life prediction. The first is to predict the individual layer life for each of the layers in the pavement structure. Secondly the occurrence of crushing in cemented layers should be investigated and thirdly the ultimate pavement life should be predicted.

Failure Mode, Critical Parameters and Transfer Functions used for different Pavement Materials

The basic material types used in South Africa are asphalt, granular, cemented and subgrade materials. Each material type exhibits a unique mode of failure. The failure mode for each material type is 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, mode(s) of failure and applicable transfer functions.

Granular Material

Granular material exhibits deformation due to densification and gradual shear under repeated loading. Maree (8) developed the concept of the "safety factor" against shear failure for granular materials used in the South African Mechanistic Design Method.

The safety factor against shear failure was developed from Möhr-Coulomb theory for static loading and represents the ratio of the material shear strength divided by the applied stress causing shear.

The safety factor against shear failure for granular materials is defined by

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

or

$$F = \frac{\sigma_3 \phi_{term} + C_{term}}{(\sigma_1 - \sigma_3)} \qquad \textbf{(2)}$$

where σ_1 and σ_3 = major and minor principle stresses acting at a point in the granular layer (compressive stress positive and tensile stress negative)

C = cohesion

ϕ = angle of internal friction

K = constant = 0.65 for saturated conditions

0.8 for moderate moisture conditions and

0.95 for normal moisture conditions

Safety factors smaller than 1 imply that the shear stress exceeds the shear strength and that rapid shear failure will occur for the static load case. Under real life dynamic loading the shear stress will only exceed the shear strength for a very short time and shear failure will not occur under one load application, but shear deformation will rapidly accumulate under a number of load repetitions. If the safety factor is larger than 1, deformation will accumulate gradually with increasing load applications. In both instances the mode of failure will however, 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 safety factor or the major and minor principle stresses are referred to as the critical parameters for granular layers for the purpose of this paper. The major and minor principle stresses and hence the safety factor are usually calculated at the mid-point of granular layers. Suggested values of the C and ϕ-terms for granular materials are given in Table 8. The transfer functions, relating the safety factor to the number of load applications that can be sustained at that safety factor level, are given by Equations 3 to 6 (14) for different service level requirements and are illustrated in Figure 4.

TABLE 8: SUGGESTED C_{TERM} AND $_{TERM}$ VALUES (MODIFIED FROM REF 11)

	Moisture Condition					
	Dry		Moderate		Wet	
Material Code	-term	C-term	-term	C-term	-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

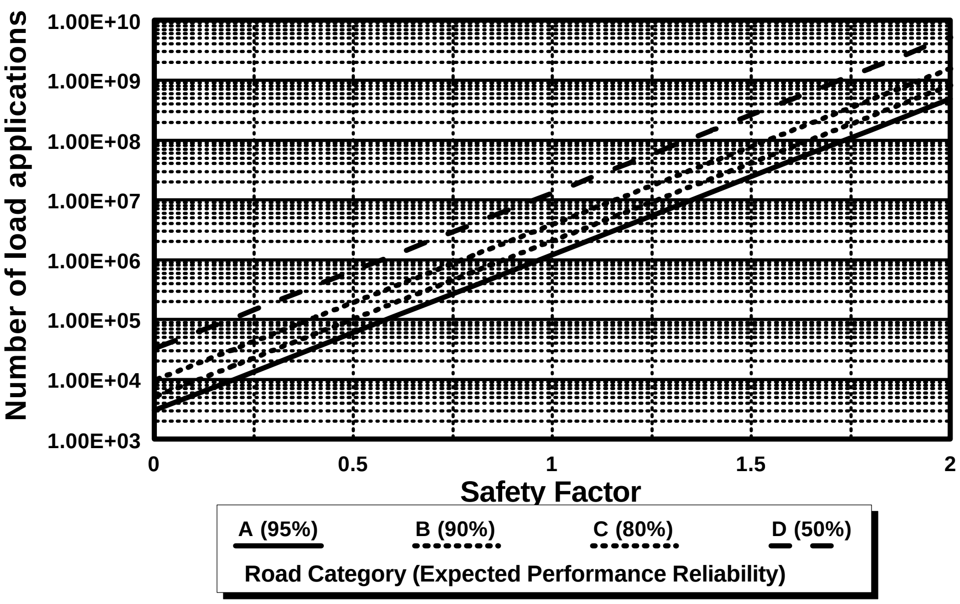


FIGURE 4: TRANSFER FUNCTIONS FOR GRANULAR MATERIALS

$$N_A \quad 10^{(2.605122F \quad 4.480098)} \quad \textit{for category A roads} \qquad \textbf{(3)}$$

$$N_B \quad 10^{(2.605122F \quad 3.707667)} \quad \textit{for category B roads} \qquad \textbf{(4)}$$

$$N_C \quad 10^{(2.605122F \quad 3.983324)} \quad \textit{for category C roads} \qquad \textbf{(5)}$$

$$N_D \quad 10^{(2.605122F \quad 4.510819)} \quad \textit{for category D roads} \qquad \textbf{(6)}$$

Cemented Material

Cemented material exhibits two failure modes, namely the effective fatigue (17) and the crushing (17). The critical parameters for cemented material is the maximum tensile strain (µ), at the bottom or in the layer for controlling the effective fatigue life and the vertical compressive stress $_v$ (kPa) , on top of the cemented layer controlling crushing life. Transfer functions are provided for two crushing conditions, namely crush initiation with roughly 2 mm deformation on top of the layer and advanced crushing with 10 mm deformation and extensive breakdown of the cemented material.

Equations 7 to 10 (14) contain the effective fatigue transfer functions for cemented materials as a function of the tensile strain (µ) at different service levels, illustrated in Figure 5. The default values suggested for the strain at break $_b$ (µ) and the Unconfined Compressive Strength UCS (kPa), for cemented materials are given in Table 9.

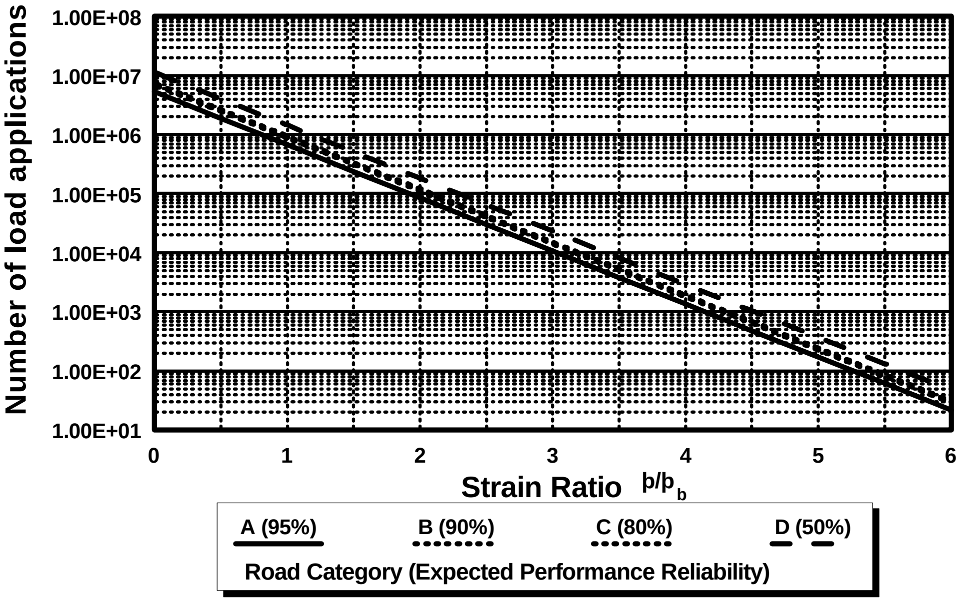


FIGURE 5: EFFECTIVE FATIGUE LIFE TRANSFER FUNCTIONS FOR CEMENTED MATERIAL

$$N_{eff} \quad 10^{6.72\left(1 \quad \frac{}{7.49 \;{}_{b}}\right)} \qquad \textit{for category A roads} \tag{7}$$

$$N_{eff} \quad 10^{6.84\left(1 \quad \frac{}{7.63 \;{}_{b}}\right)} \qquad \textit{for category B roads} \tag{8}$$

$$N_{eff} \quad 10^{6.87\left(1 \quad \frac{}{7.66 \;{}_{b}}\right)} \qquad \textit{for category C roads} \tag{9}$$

$$N_{eff} \quad 10^{7.06\left(1 \quad \frac{}{7.86 \;{}_{b}}\right)} \qquad \textit{for category D roads} \tag{10}$$

TABLE 9: DEFAULT VALUES USED FOR $_b$ AND UCS OF CEMENTED MATERIAL.

Material code	$_b$ (μ)	UCS (kPa)
C1	145	7500
C2	120	7500
C3	125	2250
C4	145	1125

The transfer functions in Equations 7 to 10 represent the effective fatigue life of the cemented material. At the end of the effective fatigue life for a cemented material, the material is assumed to behave similarly to granular material. These transfer functions do not allow for different layer thicknesses. A shift factor for cemented material was therefore introduced allow thicker layers to have an extended effective fatigue life compared to thinner layers subjected to the same strain. The shift factor based on the cemented layer thickness, for the effective fatigue life of cemented material is illustrated in Figure 6.

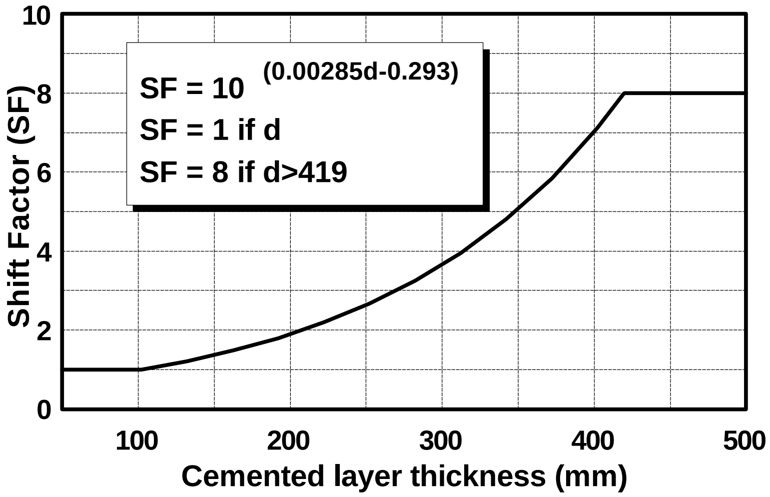


FIGURE 6: SHIFT FACTOR FOR THE EFFECTIVE FATIGUE LIFE OF CEMENTED MATERIAL (19)

Equations 11 to 14 (14) contain the transfer functions for crush initiation (N_{Ci}) and Equations 15 to 18 (14) the transfer functions for advanced crushing (N_{Ca}) of cemented material, illustrated in Figures 7 and 8 respectively.

$$N_{C_i} \quad 10^{7.386(1 \quad \frac{v}{1.09\ UCS})} \qquad for\ \ category\ \ A\ \ roads \qquad \textbf{(11)}$$

$$N_{C_i} \quad 10^{7.506(1 \quad \frac{v}{1.10\ UCS})} \quad \textit{for category B roads} \qquad \textbf{(12)}$$

$$N_{C_i} \quad 10^{7.706(1 \quad \frac{v}{1.13\ UCS})} \quad \textit{for category C roads} \qquad \textbf{(13)}$$

$$N_{C_i} \quad 10^{8.516(1 \quad \frac{v}{1.21\ UCS})} \quad \textit{for category D roads} \qquad \textbf{(14)}$$

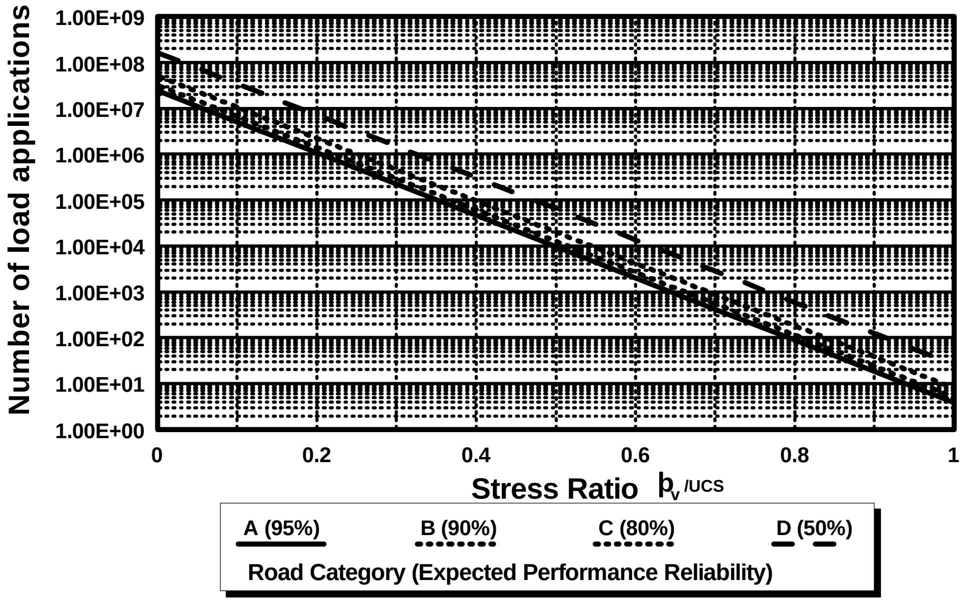


FIGURE 7: CRUSH INITIATION TRANSFER FUNCTIONS FOR LIGHTLY CEMENTED MATERIAL

$$N_{C_a} \quad 10^{8.064(1 \quad \frac{v}{1.19\ UCS})} \quad \textit{for category A roads} \qquad \textbf{(15)}$$

$$N_{C_a} \quad 10^{8.184(1 \quad \frac{v}{1.2\ UCS})} \quad \textit{for category B roads} \qquad \textbf{(16)}$$

$$N_{C_a} \quad 10^{8.384(1 \quad \frac{v}{1.23\ UCS})} \quad \textit{for category C roads} \qquad \textbf{(17)}$$

$$N_{C_a} = 10^{8.894\left(1 - \frac{v}{1.31\ UCS}\right)} \quad \textit{for category D roads} \qquad \textbf{(18)}$$

FIGURE 8: ADVANCED CRUSHING TRANSFER FUNCTIONS FOR LIGHTLY CEMENTED MATERIAL

Asphalt material

Asphalt material fails due to fatigue cracking under repeated loading as a result of tensile strain ε_t (µε) 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 a continuously graded or gap graded surfacing layer and asphalt base layers with stiffnesses from 1000 MPa to 8000 MPa.

Continuously Graded Asphalt Surfacing Layers

The fatigue crack initiation transfer functions for continuously graded material at different service levels are listed in Equations 19 to 22 (14) and illustrated in Figure 9.

$$N_f = 10^{17.40\left(1 - \frac{Log\ \varepsilon_t}{3.40}\right)} \quad \textit{for category A roads} \qquad \textbf{(19)}$$

$$N_f = 10^{17.46\left(1 - \frac{Log\ \varepsilon_t}{3.41}\right)} \quad \textit{for category B roads} \qquad \textbf{(20)}$$

$$N_f \quad 10^{17.54\,(1 \quad \frac{Log\ \ _t}{3.42})} \qquad \textit{for category C roads} \tag{21}$$

$$N_f \quad 10^{17.71\,(1 \quad \frac{Log\ \ _t}{3.46})} \qquad \textit{for category D roads} \tag{22}$$

Gap Graded Asphalt Surfacing Layers

The fatigue crack initiation transfer functions for gap graded material at different service levels are listed in Equations 23 to 26 (14) and illustrated in Figure 10.

$$N_f \quad 10^{15.79\,(1 \quad \frac{Log\ \ _t}{3.71})} \qquad \textit{for category A roads} \tag{23}$$

$$N_f \quad 10^{15.85\,(1 \quad \frac{Log\ \ _t}{3.72})} \qquad \textit{for category B roads} \tag{24}$$

$$N_f \quad 10^{15.93\,(1 \quad \frac{Log\ \ _t}{3.74})} \qquad \textit{for category C roads} \tag{25}$$

$$N_f \quad 10^{16.09\,(1 \quad \frac{Log\ \ _t}{3.77})} \qquad \textit{for category D roads} \tag{26}$$

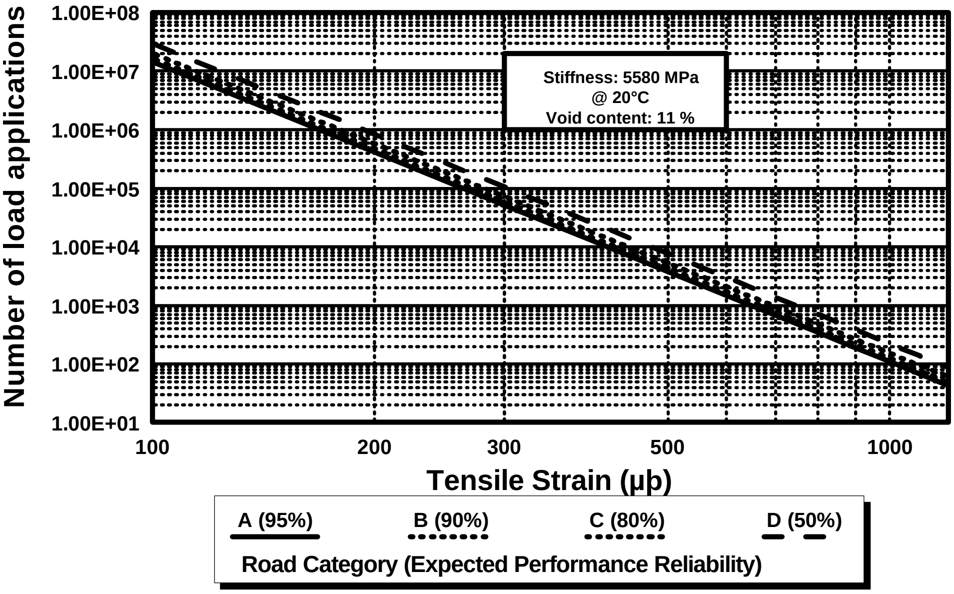


FIGURE 9: FATIGUE CRACK INITIATION TRANSFER FUNCTIONS
FOR CONTINUOUSLY GRADED THIN ASPHALT SURFACING LAYERS

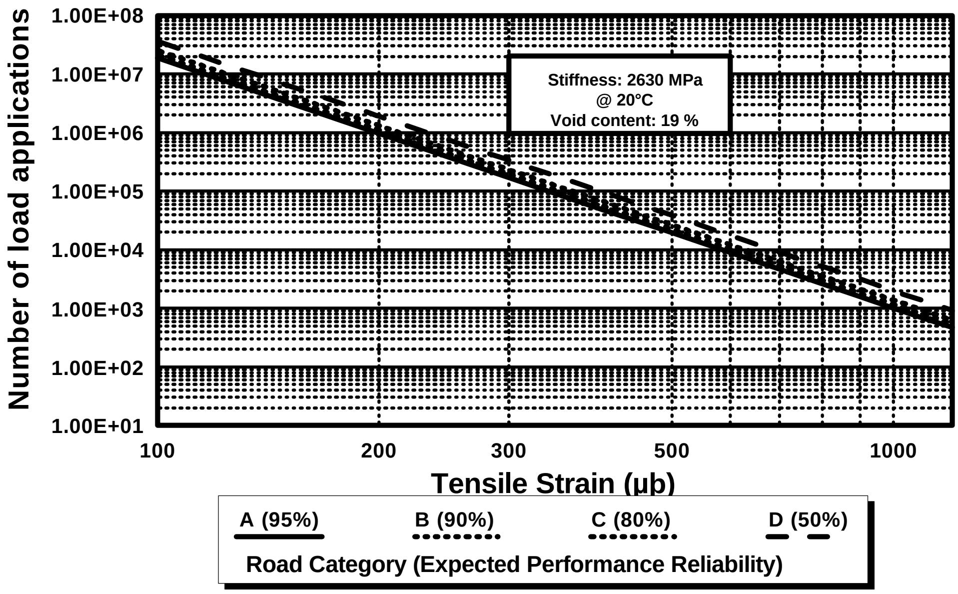


FIGURE 10: FATIGUE CRACK INITIATION TRANSFER FUNCTIONS
FOR GAP GRADED THIN ASPHALT SURFACING LAYERS

Thick Asphalt Bases

The general form of the fatigue crack initiation transfer functions for thick asphalt bases, is given in Equation 27 (14). Table 10 (14) contains the regression coefficients for Equation 27, for combinations of road category and approximate asphalt mix stiffness. Figure 11 to 15 illustrates the fatigue crack initiation transfer functions for different approximate asphalt mix stiffnesses.

$$N_f = 10^{A\left(1 - \frac{Log\,\varepsilon_t}{B}\right)} \quad \textit{for all road categories} \qquad \textbf{(27)}$$

TABLE 10: REGRESSION COEFFICIENTS FOR THE GENERAL FATIGUE CRAC K INITIATION TRANSFER FUNCTION FOR THICK ASPHALT BASES.

Asphalt mix stiffness (MPa)	Road Category/ Service level	A	B
1000	A	16.44	3.378
	B	16.81	3.453
	C	17.25	3.543

	D	17.87	3.671
2000	A	16.09	3.357
	B	16.43	3.428
	C	16.71	3.487
	D	17.17	3.583
3000	A	15.78	3.334
	B	16.11	3.403
	C	16.26	3.435
	D	16.68	3.524
5000	A	15.52	3.317
	B	15.73	3.362
	C	15.83	3.383
	D	16.10	3.441
8000	A	15.09	3.227
	B	15.30	3.272
	C	15.39	3.291
	D	15.65	3.346

Figure 16 illustrates 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.

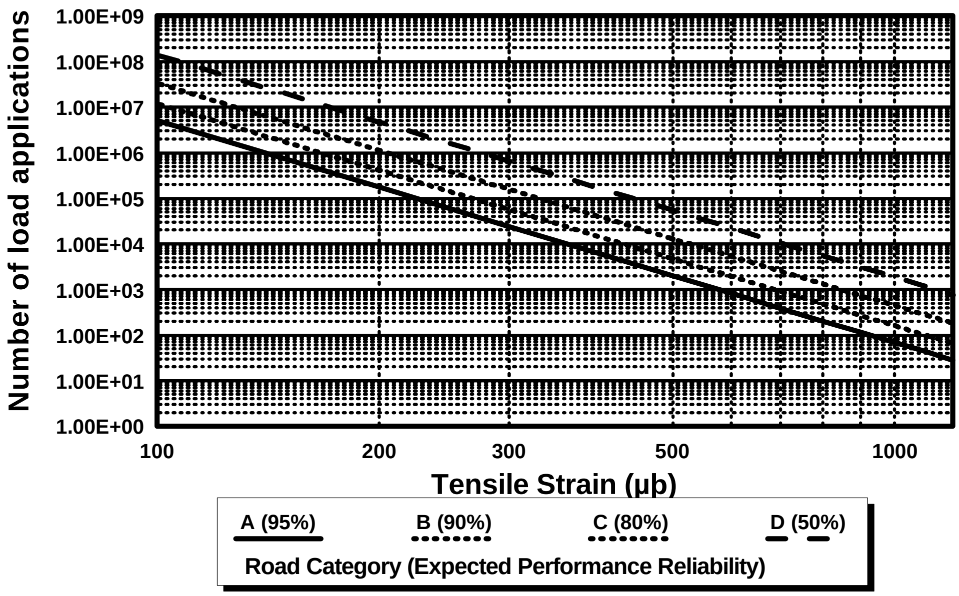


FIGURE 11: FATIGUE CRACK INITIATION TRANSFER FUNCTIONS FOR THICK ASPHALT BASE LAYERS AT 1000 MPa STIFFNESS

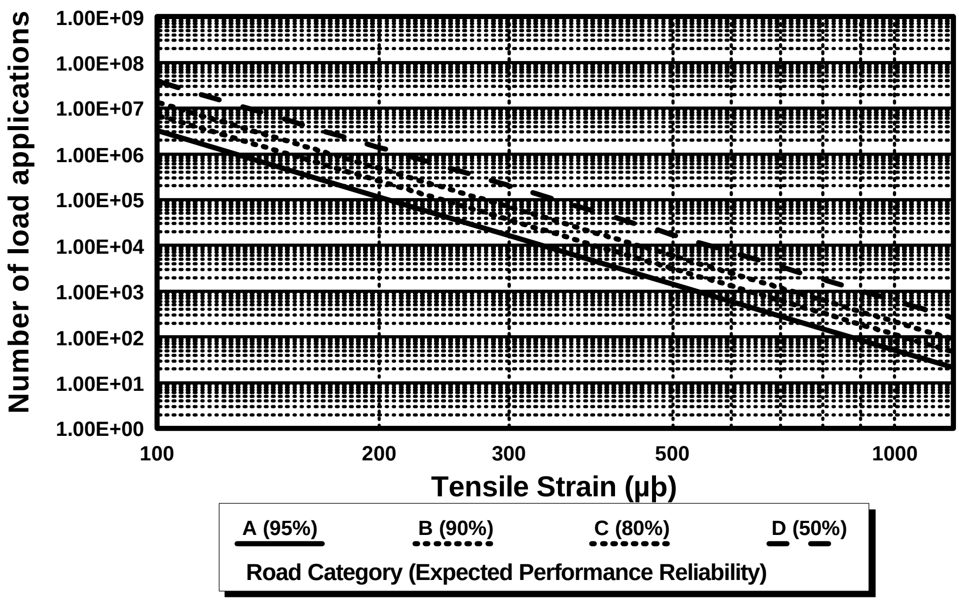


FIGURE 12: FATIGUE CRACK INITIATION TRANSFER FUNCTIONS FOR THICK ASPHALT BASE LAYERS AT 2000 MPa STIFFNESS

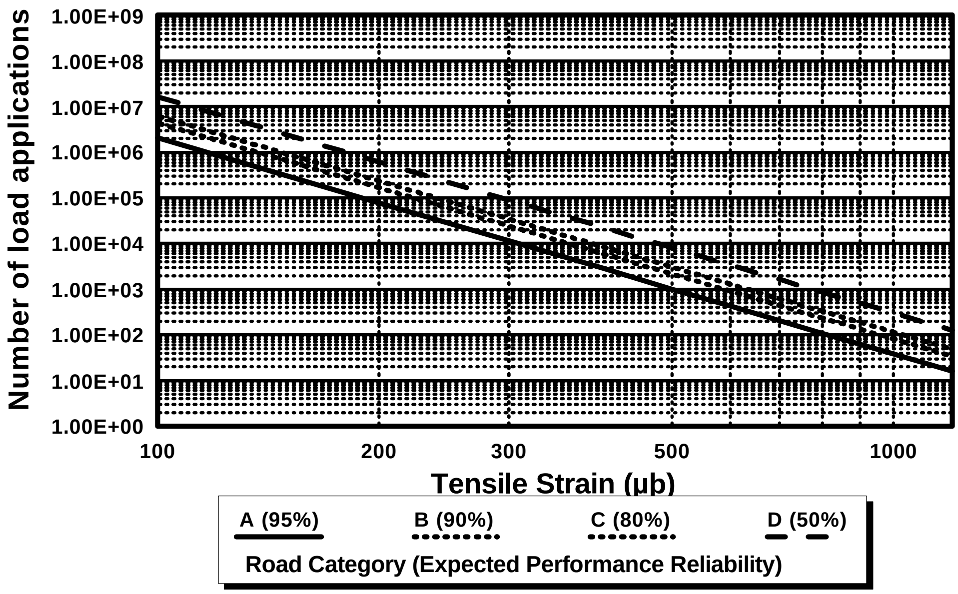


FIGURE 13: FATIGUE CRACK INITIATION TRANSFER FUNCTIONS FOR THICK ASPHALT BASE LAYERS AT 3000 MPa STIFFNESS

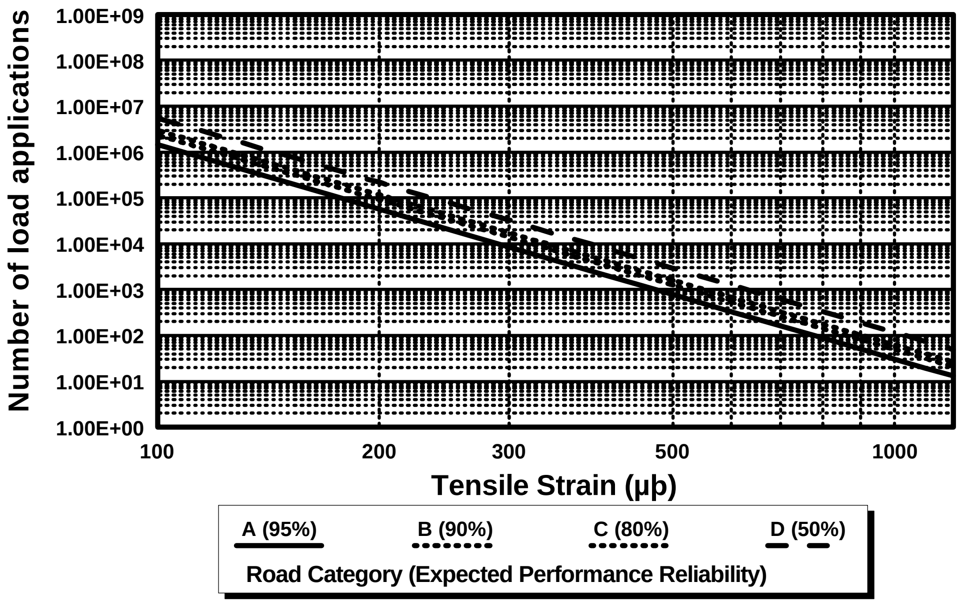


FIGURE 14: FATIGUE CRACK INITIATION TRANSFER FUNCTIONS FOR THICK ASPHALT BASE LAYERS AT 5000 MPa STIFFNESS

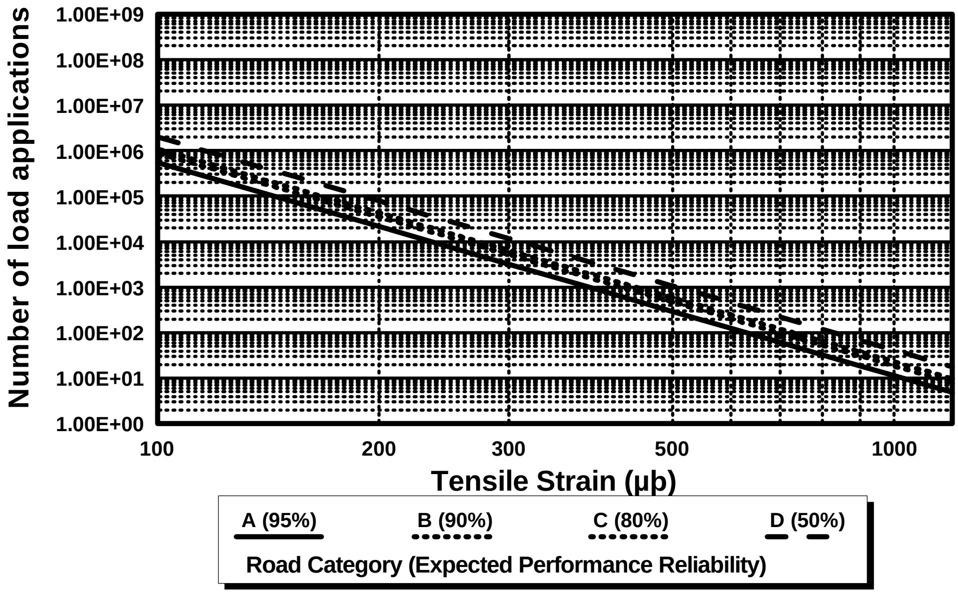


FIGURE 15: FATIGUE CRACK INITIATION TRANSFER FUNCTIONS FOR THICK ASPHALT BASE LAYERS AT 8000 MPa STIFFNESS

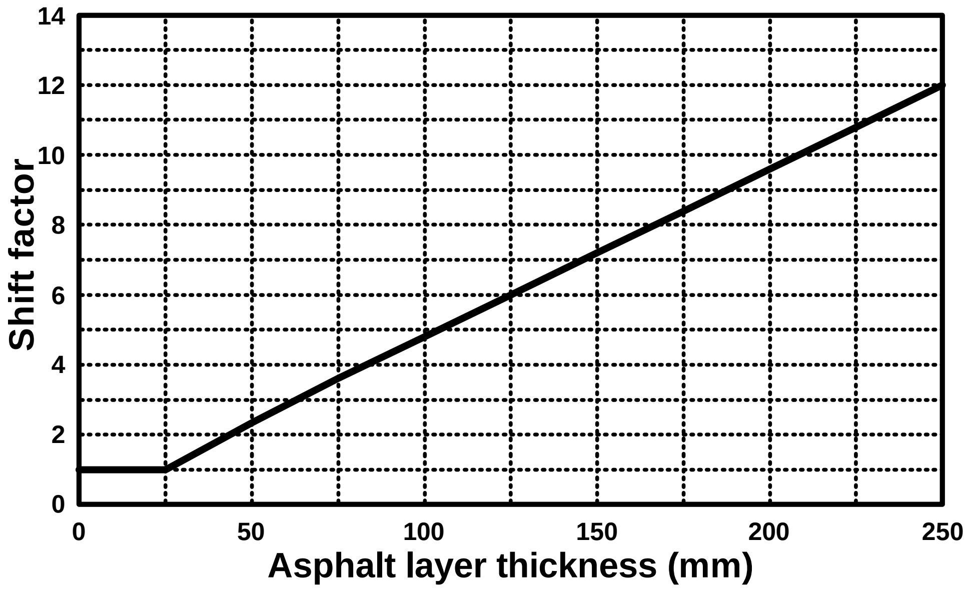


FIGURE 16: FATIGUE CRACK PROPAGATION SHIFT FACTOR FOR ASPHALT LAYERS (19)

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 ($_v$) on top of the layer. Transfer functions are provided for two terminal conditions, a 10 mm or a 20 mm surface rut due to the deformation of the selected or subgrade material.

Equation 28 (14) gives the general form of the transfer function for the selected and subgrade material with the regression coefficients for the 10 and 20 mm terminal rut condition listed in Table 11 (14) for different service levels / road categories. The transfer functions for the 10 mm terminal rut condition are illustrated in Figure 17 and those for the 20 mm terminal rut condition in Figure 18.

$$N \quad 10^{(A \quad 10\ Log \quad _v)} \qquad \textit{for all road categories} \qquad \textbf{(28)}$$

TABLE 11: REGRESSION COEFFICIENTS FOR THE GENERAL SUBGRADE DEFORMATIO N TRANSFER FUNCTION.

Terminal rut condition (mm)	Road Category / Service Level	A
10	A	33.30
	B	33.38
	C	33.47
	D	33.70
20	A	36.30
	B	36.38
	C	36.47
	D	36.70

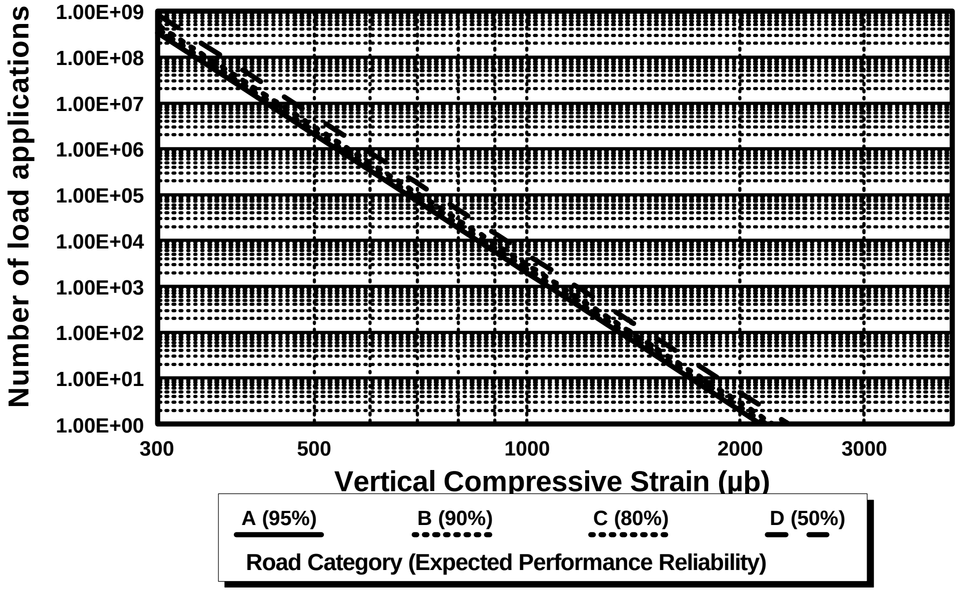


FIGURE 17: 10 mm SUBGRADE DEFORMATION TRANSFER FUNCTIONS

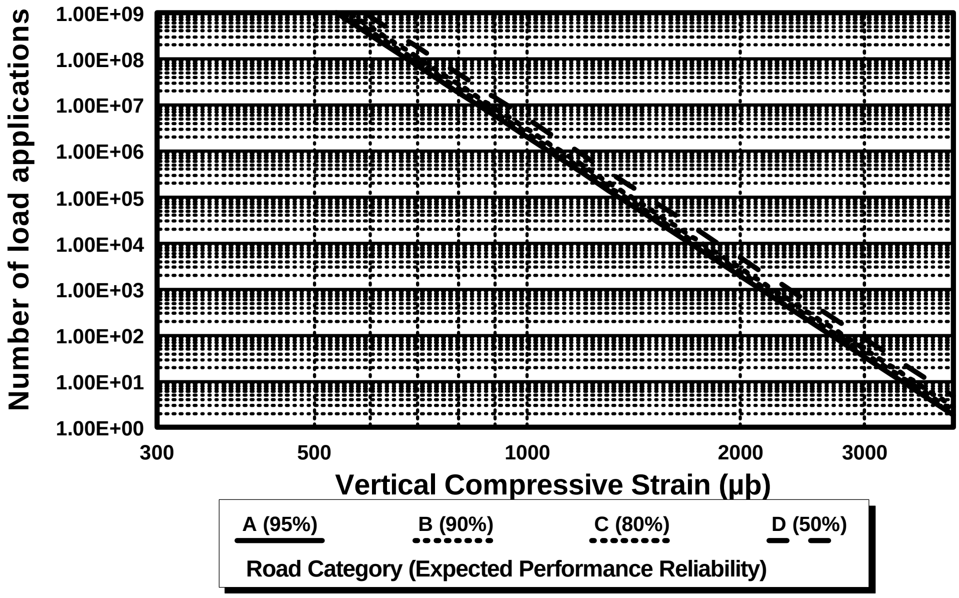


FIGURE 18: 20 mm SUBGRADE DEFORMATION TRANSFER FUNCTIONS

Incorporation of Crushing Failure for Cemented Material.

The crushing failure of cemented material was described by De Beer (17) but has not been included in the South African Mechanistic Design Method up to date. The transfer functions for crush initiation and advanced crushing were listed in a previous section.

Figure 19 illustrates the long-term behaviour of a lightly cemented layer in a pavement structure. During the pre-cracked 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 1 to 5 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.

Although these changes in the behaviour of the cemented material will gradually occur with time, they are modelled as stepwise phases in the life of the cemented material. The effective fatigue life phase and the equivalent granular phase of cemented material is used to calculate the layer life for the cemented layer. The pre-cracked phase is considered to be very short (17) in relation to the other phases and is therefore not included in predicting the layer life for the cemented material.

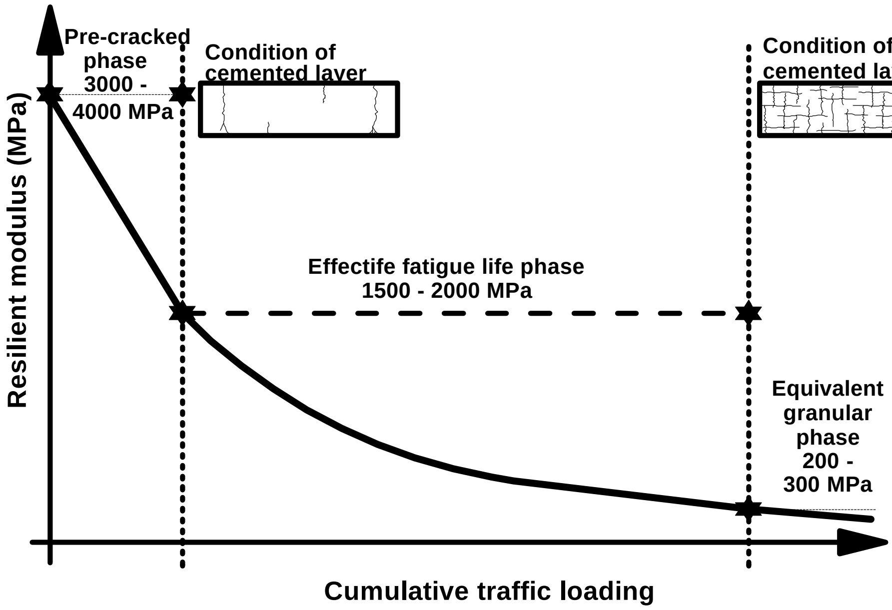


FIGURE 19: LONG-TERM BEHAVIOUR OF LIGHTLY CEMENTED MATERIAL

Consider a pavement structure with a cemented base and sub-base with a stepwise model for the cemented material behaviour. It is clear that at the end of the effective fatigue life phase for the sub-base, there will be a sudden change in the elastic modulus of the sub-base, resulting in a re-arrangement of stresses and strains in the pavement structure. The stresses and strains calculated during the effective fatigue life phase will therefore not be valid any more, except for the vertical stress on top of the cemented base which will remain almost the same as this parameter is influenced more by the applied contact stress than the structural conditions below it. This argument allows the concept of crushing failure to be introduced in the South African Mechanistic Design Method. The procedure for determining if crushing failure will occur is best illustrated by Figure 20.

In Figure 20(b) the granular state is reached in the base before crush initiation or advanced crushing take place. During this equivalent granular stage the modulus of the base is too low to allow crushing to continue to the same extent as a cemented base. In Figure 20(c) the predicted crush initiation life is shorter than the predicted effective fatigue life and crush initiation will occur with approximately 2 mm deformation on top of the cemented material. In Figure 20(d) the predicted effective fatigue life exceeds both the crush initiation and advanced crushing life and advanced crushing will occur with approximately 10 mm deformation on top of the cemented material as a result.

Pavement Life Phases and the Residual Life Concept

The concept of pavement life phases has already been introduced in the previous section. The phases are caused by changes taking place in pre-dominantly the cemented layers in a pavement structure. The modulus of a cemented layer is modelled as a constant value for the duration of a particular phase with a sudden change at the end of each phase.

The first cemented layer therefore introduces two phases to the pavement design analysis and the rest of the cemented layers one each. For example, a pavement structure with a cemented base and sub-base will have the first phase up to the point where the sub-base reaches the end of its predicted effective fatigue life. The modulus of the sub-base will suddenly reduce and the base will still be in it's effective fatigue life phase for the second phase. At the end of the predicted effective fatigue life phase for the base, the modulus of the base reduces and both the base and the sub-base are in an equivalent granular phase for the third and last phase. This process is illustrated in Figure 21.

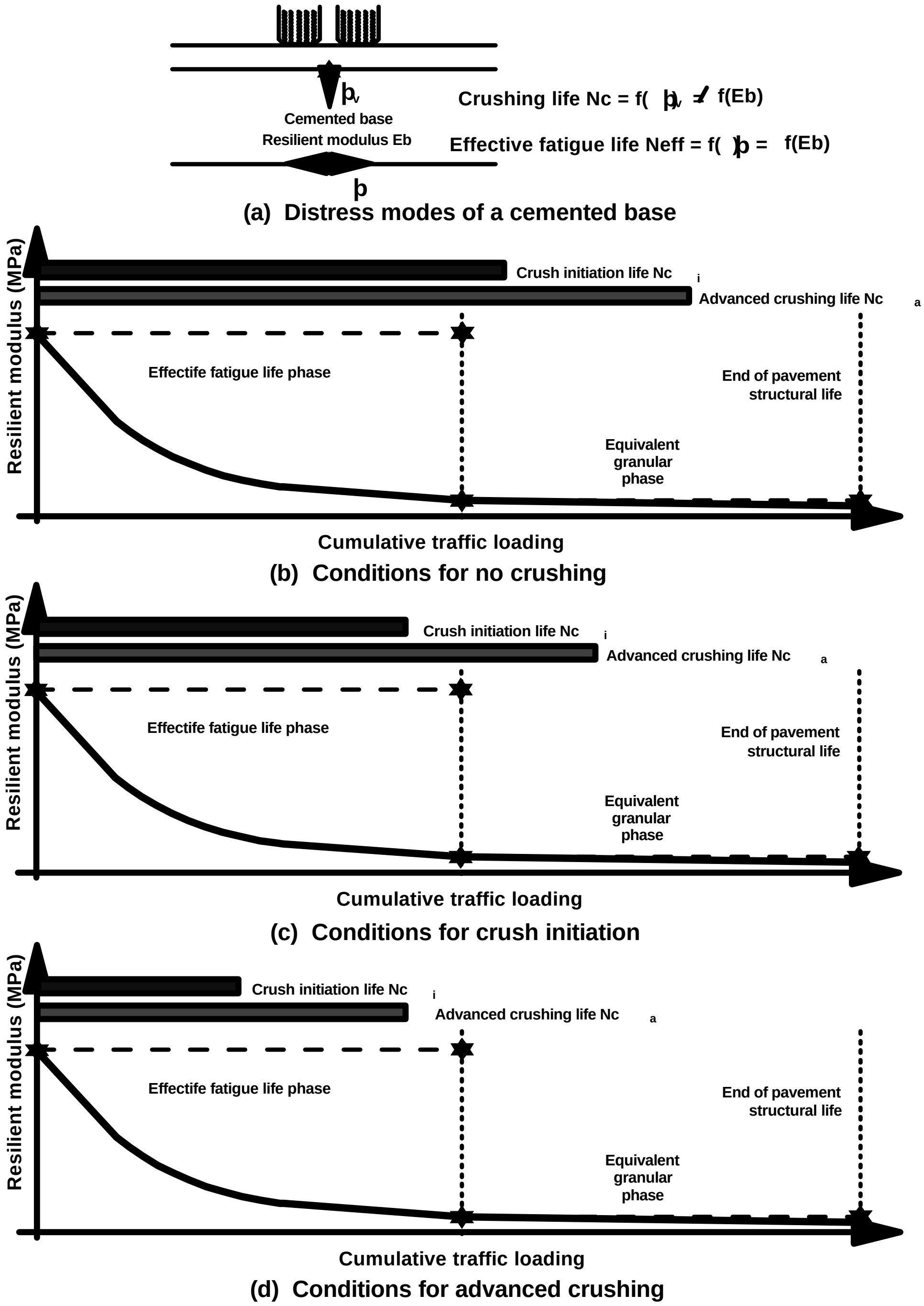
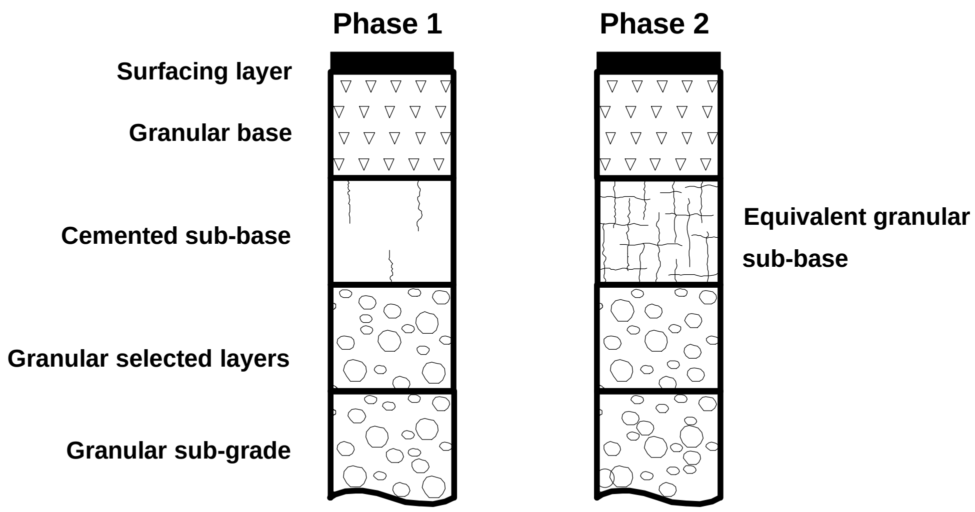
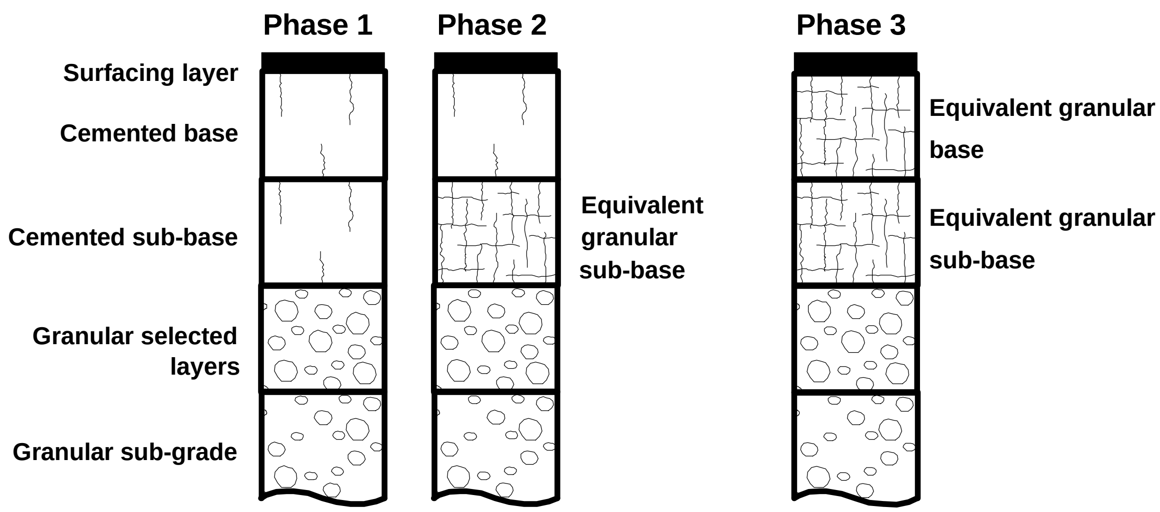


FIGURE 20: CRUSH INITIATION AND ADVANCED CRUSHING IN CEMENTED BASES



(a) Pavement structure with cemented sub-base



(b) Pavement structure with cemented base and sub-base

FIGURE 21: PAVEMENT LIFE PHASES

The stresses and strains calculated during one phase, are not valid during the following phase. A structural analysis is therefore done for each phase with the applicable reduced moduli for the cemented layers. The stresses and strains calculated for each phase will yield a predicted layer life for each layer during each phase. The transfer functions used for pavement design were however, developed from initial conditions of no distress. After Phase 1, the predicted layer life for both Phase 1 and 2 therefore becomes invalid but by combining the two values, an ultimate layer life may be calculated.

Consider the situation in Figure 22 where the layer life for each layer has been predicted for Phase 1. 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 loading 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 due to 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 which will start with a clean sheet for the second phase because there is 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 the if the top layer is a surfacing layer such as a surface seal or thin asphalt layer, then the predicted layer life for the top layer will not affect the ultimate pavement life. The reason for this 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

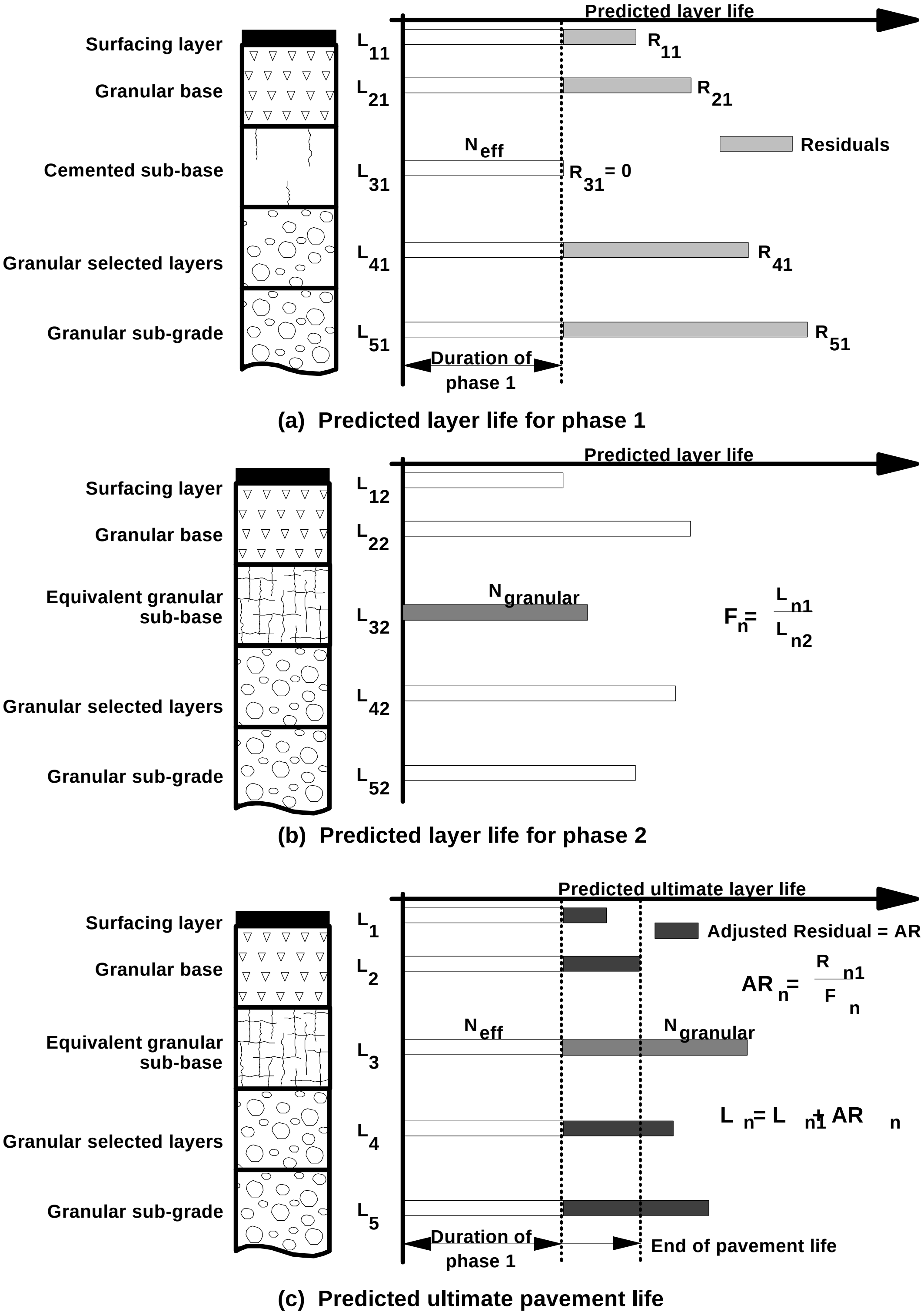


FIGURE 22: CALCULATING THE ULTIMATE PAVEMENT LIFE FOR A PAVEMENT STRUCTURE WITH CEMENTED LAYERS

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 or the Phase 2 predicted equivalent granular layer life for the original cemented layer whichever is the smallest.

The process is extended along similar principles for a three phase analysis of a pavement structure incorporating two cemented layers.

CONCLUSION

The South African Mechanistic Design Method has been used for new and rehabilitation pavement design since the late 1970's. The development of the method and components of the method was done over the last two decades and is still continued today.

The development and verification of the method was assisted by accelerated testing done with Heavy Vehicle Simulators over the last twenty years.

The South African Mechanistic Design Method has also been used to develop standard pavement designs contained in a pavement design catalogue used for the design of interurban and rural roads on a national level. The latest version of the mechanistic design method has been calibrated against the experience of road engineers from various road authorities in South Africa.

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