CHAPTER 4 MECHANISTIC-EMPIRICAL DESIGN MODELS IN PAVEMENT ENGINEERING

4.1 INTRODUCTION TO THE MECHANISTIC – EMPIRICAL APPROACH

The first step in the development of mechanistic design procedures took place when Boussinesq (1883) developed a mathematical solution in 1885 to determine stresses and strains in a one layer system subjected to a point load on the surface. The material was assumed to be linear elastic, homogeneous with semi-infinite dimensions. In 1926 Westergaard (1938) developed a two-layer system with the assumption that the upper layer is an elastic plate that bend through but with no vertical deflection within the layer. Timoshenko developed the general theory of elasticity where stresses and strains in a linear elastic system subject to complex loading conditions can be calculated. Burmister (1945) improved this theory for two- and three-layer systems, but it was not user friendly because of complex equations. The work of Acum and Fox (1948), Peattie (1962) and Jones (1962) provided tables for systems with a maximum of three layers. The development of computers from the 1960's provided the opportunity for analyses of systems with five and more layers. Pavement engineers were then able to analyse pavement structures with various number of layers and loading conditions. The calculated stresses and strains could be compared with values obtained from tests or experiments.

4.2 PREREQUISITES FOR INCORPORATING A NEW MATERIAL INTO THE MECHANISTIC-EMPIRICAL DESIGN PROCESS

The mechanistic-empirical design model allows the designer some latitude in the sense that designs slightly outside the domain in which the method was developed, may be investigated. The South African Mechanistic Design Procedure was developed in the 1970's from a wide range of research and was improved and refined in the 1980's and 1990's (Maree and Freeme: 1981, Otte: 1972, Otte: 1978, Maree: 1978, Maree: 1982, de Beer: 1985, de Beer: 1989, Jordaan: 1988 and Theyse et al: 1996). The method was developed for South African conditions and South African materials and verified to a large extent with the use of a number of Heavy Vehicle Simulator (HVS) tests.

The incorporation of a new or fairly unknown material into a mechanistic design process require a number prerequisites. These include:

- The basic behaviour of the particular material should be well understood. This does not imply that the full behaviour pattern nor the timescale of the total life cycle should be known, but requires at least some general description of how the material is expected to behave in terms of resilient, strength and performance characteristics. If this description agrees with the behaviour model for a well-known material already included in the design method, the process for developing a model for the new material could be simplified and accelerated.

- Once the input parameters have been identified from consideration of the expected behaviour pattern and failure mechanism for a new material, representative values of these parameters should be obtained. These input parameters will typically include the resilient properties of the material and some strength parameters related to the mode of failure of the material.
- A well established performance model (transfer function) relating the calculated stress or strain condition in the different pavement layers to the expected structural performance of the different material types used in these layers, must be available. In this case the structural performance is defined as the number of stress/strain repetitions that can be sustained by the material at that particular stress/strain level until a certain terminal condition is reached.

4.3 GRANULAR MATERIALS

Granular layers deform due to consolidation or densification and gradual shear under repeated loading. The consolidation or densification phase is usually limited to the early life of the layer and is a function of the construction quality, compaction and quality of the material used. Most of this initial deformation can be limited by proper construction and compaction. Research by Maree (1978, 1982) indicated that granular layers fail in shear when the shear strength of the layer is exceeded. He introduced the concept of a safety factor against shear failure for granular materials used in the South African Mechanistic Pavement Design Method. The safety factor against shear failure is based on the Möhr-Coulomb theory for static loading and represents the ratio of the material shear strength to the applied stresses responsible for shear. The principal stresses (σ_1 and σ_3) in the middle of the layer are regarded as the critical parameters used to calculate the safety factor. The safety factor incorporates the cohesion, internal friction, moisture regime and stress condition of the material.

The safety factor against shear is defined by:

$$F = \frac{\sigma_3 (K.\tan^2(45 + \frac{\phi}{2}) - 1) + 2K.c.\tan(45 + \frac{\phi}{2})}{\sigma_1 - \sigma_3} \qquad (4.1)$$

where F = Safety factor against shear failure

σ_3 and σ_1 = Calculated major and minor stresses acting in the middle of the layer

c = Cohesion (kPa)

ϕ = Angle of internal friction (degrees)

K = Constant (0.95 for normal conditions and 0.65 for wet conditions)

A safety factor of less than 1 implies that the shear stress exceeds the shear strength of the material and that rapid shear failure will occur under static loading. Under dynamic loading conditions, which is typically what happens on a pavement, the applied shear stress will only exceed the shear strength for a short period of time and shear failure will not occur under one or two load repetitions. The shear failure or deformation will gradually accumulate under a number of load repetitions. The rate of shear deformation of the granular layer is influenced by the magnitude of the safety factor.

Allowable safety factors proposed by Maree (1978, 1982) for different granular materials are summarised in Table 4.1.

Table 4.1 Allowable safety factors for granular materials at various traffic levels (Maree: 1978)

Road Category (TRH4)	Design traffic	Minimum safety factor
A	> 10 million E80's	1.6
	1 – 10 million E80's	1.5
B	3 – 30 million E80's	1.4
	0.1 – 3 million E80's	1.3
C	0.1 – 1 million E80's	1.2
	< 0.1 million E80's	1.0

The transfer function for granular layers to a terminal condition of 20 mm deformation at the surface is presented in Figure 4.1.

The model developed by Maree (1982), which is included in the South African Mechanistic Pavement Design Method, only consider deformation in the layer as a result of shear failure.

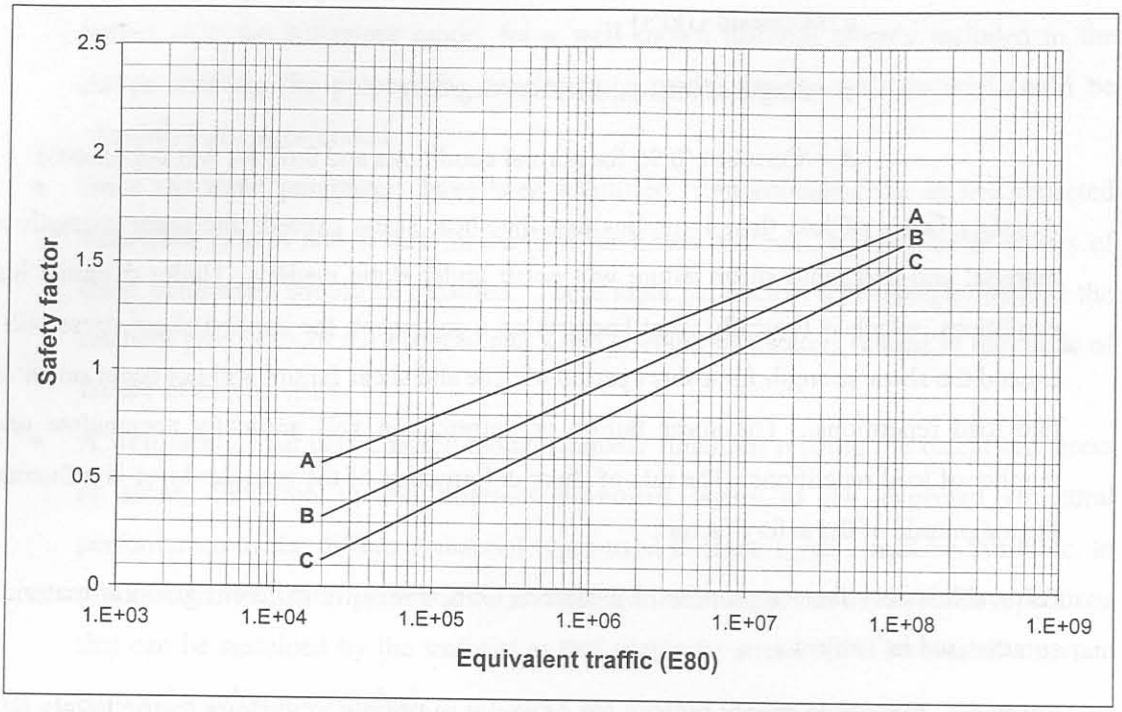

Figure 4.1 Transfer function for granular materials (Maree: 1982)

Wolff (1992) developed a design method that includes the initial rapid deformation as well as the gradual deformation at a constant rate thereafter. He developed design curves for G1 to G6 materials based on SN-curves and expressed the development of permanent deformation by the following function:

$$\varepsilon_p = (mN + a)(1 - e^{-bN}) \tag{4.2}$$

where: ε_p = cumulative permanent deformation or strain

N = number of load repetitions

m, a and b = constants

Properties of the function are described in detail by Wolff (1992) and Shackleton (1995).

The stress state, or bulk stress Θ, in the middle of the layer were used as the critical parameter in the transfer functions developed by Wolff (1992). The bulk stress is defined as the sum of the principal stresses ($\Theta = \sigma_1 + \sigma_2 + \sigma_3$). The shortcomings of the linear modelling of granular materials were addressed by introducing a non-linear elastic analysis. The K-theta model, developed by Biarez (1962), Hicks (1970), Seed et al (1967) and others were used to describe the non-linearity of the granular materials:

$$M_R = K_1 \Theta^{K_2} \quad (4.3)$$

where: M_R = resilient modulus

Θ = bulk stress which is the sum of principle stresses

K_1 and K_2 = constants

The transfer functions developed by Wolff (1992) were based on results from 22 HVS tests on base layers that varied from G1 to G6 quality. The functions were developed for various levels of permanent strains between 10 000 με and 200 000 με. The relationship between the bulk stress and the number of load repetitions was given, in a non-linear form, as:

$$\Theta = A(1 - e^{-\frac{B}{N^m}}) \quad (4.4)$$

where: Θ = bulk stress

N = number of load repetitions

m, A and B = regression constants

A typical S-N curve for G1 to G6 materials for a permanent strain of 20 000 με (equivalent to 3 mm of permanent deformation in a 150 mm layer) is presented in Figure 4.2.

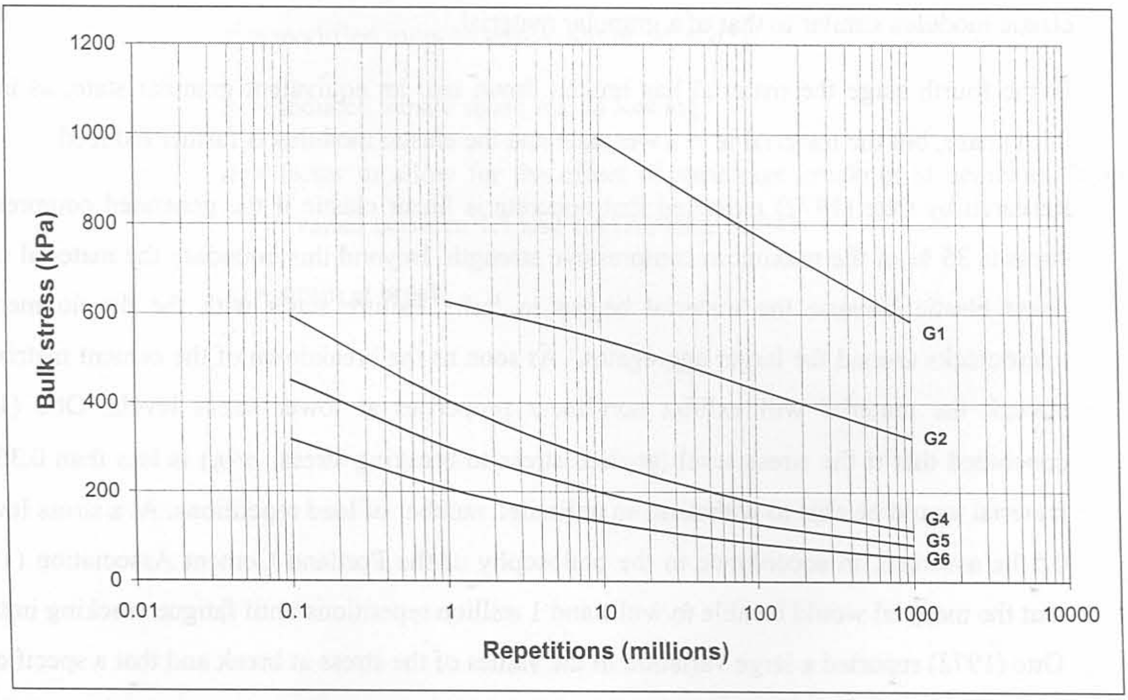

Figure 4.2 S-N curves for G1 to G6 materials at a terminal permanent strain of 20 000 με. (Wolff: 1992)

4.4 CEMENTED LAYERS

Cemented layers are analysed in terms of fatigue cracking and crushing. Fatigue cracking usually starts at the bottom of the layer and progresses upwards through the layer, while crushing occurs at the top (upper 50 to 75 mm) of a lightly cemented base layer.

4.4.1 Fatigue life

De Beer (1985) describes the behaviour of cemented layers as a four staged behaviour that influences the effective elastic moduli of the material.

The first stage, also known as the pre-cracked phase, involves the possible presence of shrinkage cracks but at large block sizes (greater than 5 times layer thickness) and the material acts as a slab similar to a concrete slab.

In the second phase loading and/or the environment have reduced the block size, but the behaviour is still predominantly controlled by the large blocks of material relative to the layer thickness (block size between 1 and 5 times layer thickness). The material still acts as a slab but with a reduced elastic moduli.

In the third phase the material has broken down in small blocks (block size smaller than layer thickness) and the material is equivalent to that of a granular material. The material also has an elastic modulus similar to that of a granular material.

In the fourth stage the material has broken down into an equivalent granular state, as in the third phase, but the material is in a wet state and the elastic modulus is further reduced.

Research by Otte (1972) indicated that concrete is linear elastic if the generated compressive stress is 35 % of the maximum compressive strength. Beyond this boundary the material is not linear elastic because the material begins to fail. Failure starts with the development of microcracks around the larger aggregates. As soon as the breakdown of the cement matrix had started, the material will exhibit non-linear properties at lower stress levels. Otte (1978) concluded that if the stress level (applied stress to breaking stress, σ/σ_b) is less than 0.35, the material would be able to withstand an unlimited number of load repetitions. At a stress level of 0.5 he assumed, in accordance to the philosophy of the Portland Cement Association (1959), that the material would be able to withstand 1 million repetitions until fatigue cracking initiates. Otte (1972) reported a large variation in the values of the stress at break and that a specification in terms of strain at break resulted in less variation. The corresponding strain ratios ($\varepsilon/\varepsilon_b$) for unlimited and 1 million repetitions are 0.25 and 0.33 respectively. The fatigue life function developed by Otte was mainly for strongly (4 to 6 % cement content or C1 and C2) (CSRA: 1986) stabilised materials and is defined in Equation 4.5.

$$N_f = 10^{9.1\left(1-\frac{\varepsilon}{\varepsilon_b}\right)} \tag{4.5}$$

where: N_f = Number of load repetitions to failure

ε = Horizontal tensile strain at bottom of the layer

ε_b = Strain at break

Otte (1972) determined average values for the strain at break on various projects, with cement stabilised base layers with 4% cement, at 145 µε. He recommended that a maximum allowable strain of 25 % of the strain at break, should never be exceeded.

Research by de Beer (1989) and Jordaan (1988) point out that the transfer function developed by Otte underestimates the effective fatigue life of lightly cemented layers. An effective fatigue life function for lightly cemented (C3 and C4) materials was developed by de Beer from various HVS tests and is defined in equation 4.6. A comparison between the fatigue life functions developed by Otte and de Beer is presented in Figure 4.3.

$$N_f = 10^{7.19\left(1-\frac{\varepsilon_s}{8\varepsilon_b}\right)} \tag{4.6}$$

where: N_f = Effective fatigue life

ε_s = modified induced tensile strain: $\varepsilon_s = d^*\varepsilon_i$

ε_i = induced tensile strain due to loading

d = factor to allow for the effect of shrinkage cracking in cemented layers varies between 1.1 and 1.4 (Jordaan: 1994)

ε_b = strain at break

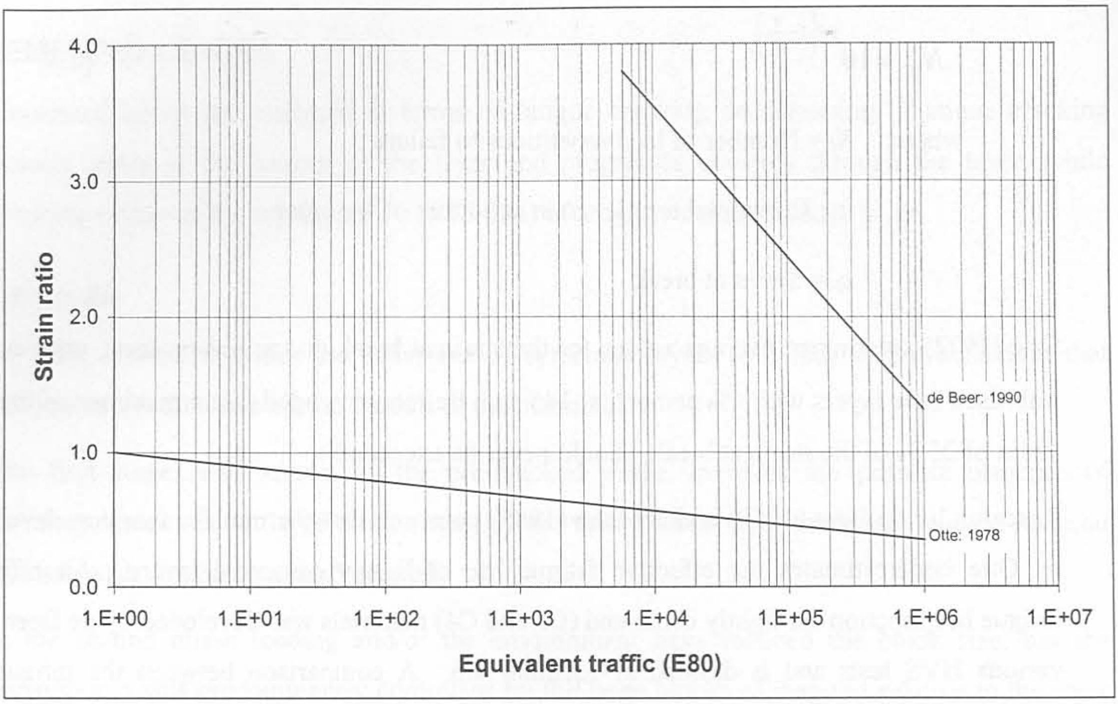

Figure 4.3 Comparison of the relationship between maximum tensile strain ratio ($\varepsilon_s/\varepsilon_b$) and number of strain repetitions to initiate effective fatigue cracking in cemented layers (de Beer: 1989).

According to Jordaan (1988) the layer will be in its transitional (second) phase, still intact but with microcracks present, and with a reduced elastic moduli, if the tensile strain exceeds 25 % of the strain at break. The values for the elastic modulus for cemented material are included in Table 4.2.

The method by Otte was further improved by Jordaan (1988) to allow for the in-situ pavement conditions by adjusting the strain at break (ε_b) as determined in the laboratory for strongly cemented layers. This is done by increasing the strain at break by a factor of m which has a value of 4.7. Figure 4.4 presents a comparison between the failure criteria of Otte and the improved one by Jordaan.

It might be possible that the maximum horizontal strain does not occur at the bottom of the cemented layer. Jordaan (1988) derived a method to test if the maximum strain does occur at the bottom of the layer that is similar to the formula recommended by SHELL (1978) for the use on asphalt layers to determine when the maximum horizontal strain shifts from a position at the bottom of the asphalt layer to a position within the asphalt layer.

Table 4.2 Typical effective range of elastic moduli for cement treated materials in various stages of behaviour. (Jordaan: 1994)

Original code	UCS (MPa) Pre-cracked material	Parent material	Phase 1: Pre-cracked phase with large shrinkage cracks (MPa)	Phase 2: Transitional phase: Layer intact, but microcracks present (MPa)	Phase 3: Post-cracked phase: layer broken up in equivalent granular state — Dry state (MPa)	Phase 3: Post-cracked phase: layer broken up in equivalent granular state — Wet State (MPa)
C1	5 – 12	Crushed stone G2 – G3	2 500 – 3 000	800 – 1 000	400 - 500	50 – 400
C2	3 – 6	Crushed stone G2 – G3, and Natural Gravel G4	2 000 – 2 500	500 – 800	300 - 800	50 - 300
C3	1.5 – 3	Natural Gravel G4 – G10	1 000 – 2 000	500 - 800	200 - 400	20 - 250
C4	0.75 – 1.5	Natural Gravel G4 G10	500 – 2000	400 - 600	100 - 350	20 - 200

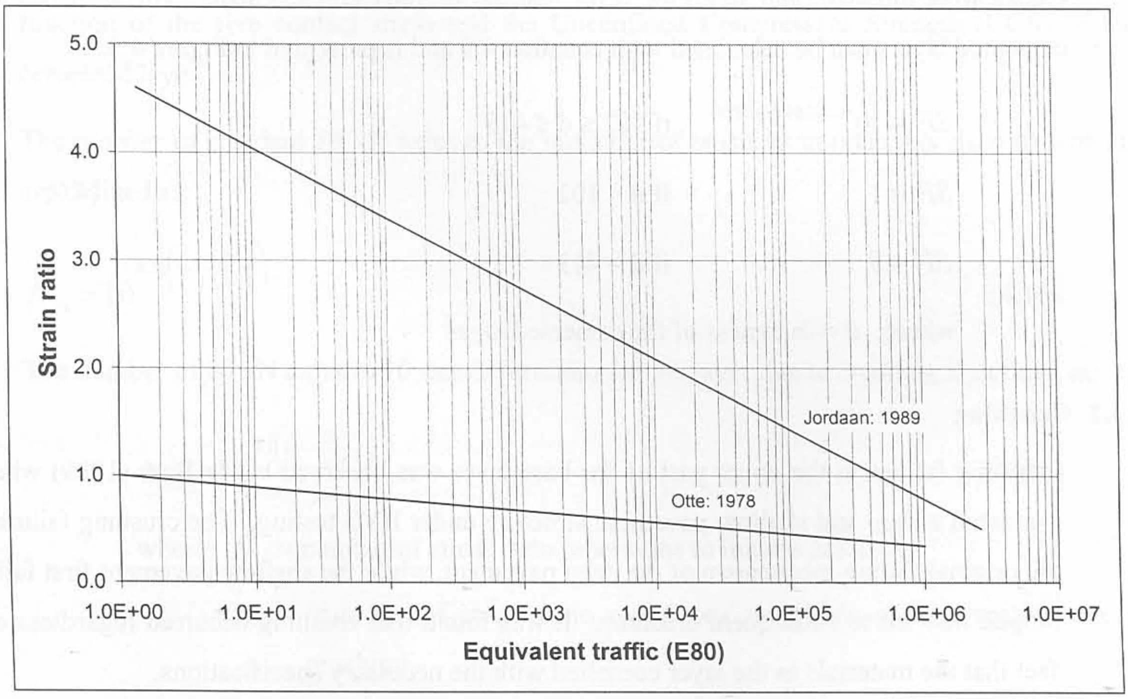

Figure 4.4 Comparison of the fatigue criteria applicable to strongly cemented layers (Jordaan: 1988)

The position of the maximum horizontal strain is at the bottom of the cemented layer when:

$$\left(\frac{E_3}{E_2}\right)^2 h_c < K \tag{4.7}$$

where: $$h_c = h_1\left(\frac{E_1}{E_3}\right)^{\frac{1}{3}} + h_2\left(\frac{E_2}{E_3}\right)^{\frac{1}{3}} \tag{4.8}$$

E_1 = elastic modulus of the asphalt layer (MPa)

E_2 = elastic modulus of the cement treated base layer (MPa)

E_3 = elastic modulus of the subbase layer (MPa)

h_1 = thickness of the asphalt layer (mm)

h_2 = thickness of the cement treated layer (mm)

K = constant = 128

After the initiation of cracks at the bottom of a cemented layer, the crack progresses through the layer with time as the layer is subjected to traffic loading. The progression of cracks through the layer is addressed by means of a shift factor to allow for the time between when the cracks were initiated, and when the layer reached its fully cracked state. The shift factor for cemented layers can be calculated with equation 4.9 and is presented in Figure 4.5.

$$\begin{aligned} SF &= 10^{(0.00285d-0.293)} && \text{if } 102 \le d \le 419 \\ SF &= 1 && \text{if } d < 102 \\ SF &= 8 && \text{if } d > 419 \end{aligned} \tag{4.9}$$

where: d = thickness of the cemented layer

4.4.2 Crushing

Crushing failure in the upper part of the base layer was observed by de Beer (1989) when he evaluated a deep and shallow pavement structure under HVS testing. The crushing failure was the original failure mechanism of the deep pavement, while the shallow pavement first failed in fatigue that led to subsequent crushing. It was found that crushing occurred regardless of the fact that the materials in the layer complied with the necessary specifications.

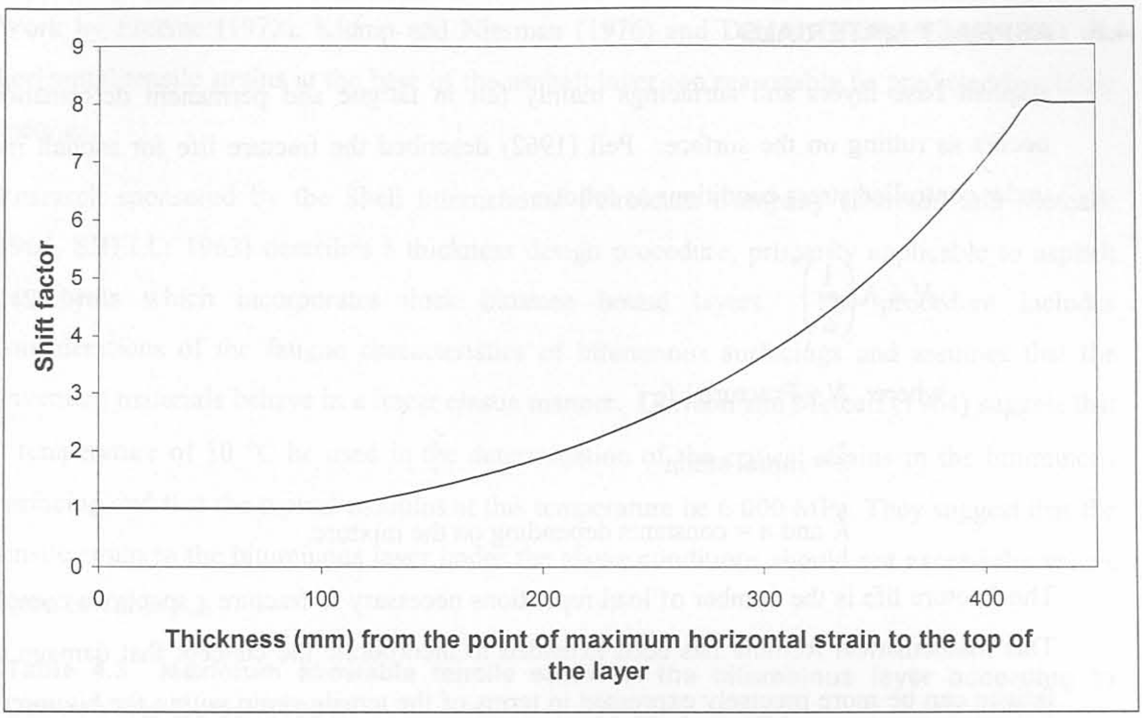

Figure 4.5 Shift factor for cemented layers (Jordaan: 1994)

This failure mechanism differs from the fatigue failure normally associated with cemented materials as described by Otte (1972) and de Beer (1989). This lead to the development of the "crushing life" concept for lightly cemented gravel pavement materials. The crushing is a function of the tyre contact stress and the Unconfined Compressive Strength (UCS) of the cemented layer.

The number of standard 80 kN axles to the initiation of crushing in relatively deep pavements are defined as:

$$N_{c1} = 10^{8.21\left(1-\frac{\sigma_t}{1.2UCS}\right)} \tag{4.10}$$

The number of 80 kN axles to 10 mm deformation in the layer due to crushing is defined as:

$$N_{c2} = 10^{9.1\left(1-\frac{\sigma_t}{1.2UCS}\right)} \tag{4.11}$$

where: N_{c1} = number of stress ratio repetitions to initiate crushing

N_{c2} = number of stress ratio repetitions to 10mm of deformation due to crushing

σ_t = vertical stress at the top of the layer (kPa)

UCS = Unconfined Compressive Strength of the in-situ cementitious material

4.5 ASPHALT MATERIALS

Asphalt base layers and surfacings mainly fail in fatigue and permanent deformation that occurs as rutting on the surface. Pell (1962) described the fracture life for asphalt material under controlled stress conditions as follows:

$$N = K\left(\frac{1}{\varepsilon}\right)^{n} \tag{4.12}$$

where: N = Fracture life

ε = initial strain

K and n = constants depending on the mixture.

The fracture life is the number of load repetitions necessary to fracture a specimen completely. This mathematical formula has been extended to incorporate the concept that damage due to fatigue can be more precisely expressed in terms of the tensile strain within the bitumen (Pell: 1976), and resulted in the following equation:

$$N = K\left(\frac{\alpha B_v}{\varepsilon_M}\right)^{n} \tag{4.13}$$

where: N = number of load repetitions to cause crack initiation

α = a factor depending on the amount of filler voids and/or voids present in the mix

B_v = proportion of bitumen present in the total volume of mix

ε_M = tensile strain in the mixture

K and n = constants depending on the mixture.

Epps and Monismith (1969) presented fatigue regression lines from controlled stress laboratory testing for granite mixes graded to meet the extremes, as well as the middle of the State of California 13.2 mm maximum sieve-size grading specifications which contains 6 % bitumen (by mass of aggregate). These regression lines can be presented by the following equation:

$$N_f = 1.15 * 10^{-6}\left(\frac{1}{\varepsilon_{mix}}\right)^{2.92} \tag{4.14}$$

where: N_f = number of load applications to fracture

ε_{mix} = bending strain repeatedly applied

Work by Freeme (1972), Klomp and Niesman (1976) and Dehlen (1969) indicated that the horizontal tensile strains at the base of the asphalt layer can reasonably be predicted by elastic theories.

Research sponsored by the Shell International Petroleum Company (Dormon and Metcalf: 1964, SHELL: 1963) describes a thickness design procedure, primarily applicable to asphalt pavements which incorporates thick bitumen bound layers. The procedure includes considerations of the fatigue characteristics of bituminous surfacings and assumes that the pavement materials behave in a linear elastic manner. Dormon and Metcalf (1964) suggest that a temperature of 10 °C be used in the determination of the critical strains in the bituminous surfacing and that the typical modulus at this temperature be 6 000 MPa. They suggest that the tensile strain in the bituminous layer under the above conditions should not exceed the values given in Table 4.3.

Table 4.3 Maximum allowable tensile strain in the bituminous layer according to Dormon and Metcalf (1964)

Number of equivalent 80 kN axle load applications	Tensile strain in bituminous layer (με)
10^5	230
10^6	145
10^7	92
10^8	58

The work of Monismith et al (1971), Heukelom and Klomp (1964) and Kingham et al (1972) were used by Freeme and Strauss (1979) to develop the transfer functions included in the South African Mechanistic Pavement Design Method. These researchers all related the crack initiation fatigue life to the tensile strain in the sample.

The transfer function currently included in the South African Mechanistic Design Method for thick asphalt bases is as follows:

$$N_f = 10^{\alpha\left(1 - \frac{\log \varepsilon_t}{\beta}\right)} \tag{4.15}$$

where: N_f = Number of 80 kN equivalent axle repetitions to crack initiation

ε_t = tensile strain at the bottom of the asphalt layer

α and β = regression constants depending on mix stiffness and percentile level

The transfer function for thin asphalt surfacings is similar to the one for thick asphalt bases and is as follows:

$$N_f = 10^{A\left(1-\frac{\log \varepsilon_t}{B}\right)} \qquad (4.16)$$

where: N_f = Number of 80 kN equivalent axle repetitions to crack initiation

ε_t = tensile strain at the bottom of the asphalt surfacing layer

A and B = regression constants depending on mix type (gap graded, continuous graded etc.) and percentile level

The propagation of cracks through the layer will occur as a result of repeated loading after cracks have been initiated at the bottom of the layer. Factors that accelerate or provide the necessary conditions for cracks to propagate through the layer, are those which induce tensile stress in the material. A shift factor to allow for this crack propagation is proposed by Jordaan (1994) and is as follows:

$$SF = \frac{d}{20} - 0.25 \qquad (4.17)$$

where: SF = Shift factor to allow for crack propagation

d = thickness of asphalt layer in mm

Work done by Harvey et al (1995) include the mix properties such as bitumen content, air void content as well as voids filled with bitumen into fatigue life predictions from controlled strain laboratory testing. They indicated that an increase in bitumen content and a decrease in air voids enhance the fatigue life properties of asphalt mixes. An increase in the voids filled with bitumen also benefits the laboratory fatigue life. Maximum bitumen content and minimum void content are for given conditions not only limited by economics, but also by other distress mechanisms such as rutting, instability and bleeding.

The permanent deformation of asphalt materials is not included in the South African Mechanistic Pavement Design Method. It is assumed that proper mix design will prevent excessive permanent deformation before the end of the fatigue life. Asphalt is visco-elastic and permanent deformation is dependent on a number of factors that include time of loading, temperature, magnitude of load and the various mix properties. To accurately model the permanent deformation of asphalt would require complex analysis procedures that are beyond the scope for routine structural design of pavements. These procedures are more useful in the mix design of asphalt materials.

4.6 SUBGRADE MATERIALS

Subgrade soils are usually found at a depth of 450 mm and more in the pavement structure. The dominant failure mechanism assumed is permanent deformation as a result of the combined action of consolidation and shear. The critical parameter for evaluating this failure mechanism is the vertical compressive strain at the top of the subgrade layer. The first criteria for subgrade performance for South African subgrades, was published by Paterson (1978) and is presented in Figure 4.5. The background for these criteria is the work done by Paterson (Paterson and Maree: 1978) and by the US Army Engineer Waterways Experiment Sation (WES) (Brabston et al: 1975). Paterson's criteria was based on the analysis of data from the AASHO road test done by the personnel from SHELL (Dormon: 1962, Dormon and Metcalf: 1965). The AASHO data are the only source of original data for the prediction of performance of subgrade material used as background for the South African Mechanistic Design Method. Criteria were published for three road categories that were applicable for any natural gravel or gravel-soils layers underneath the subbase.

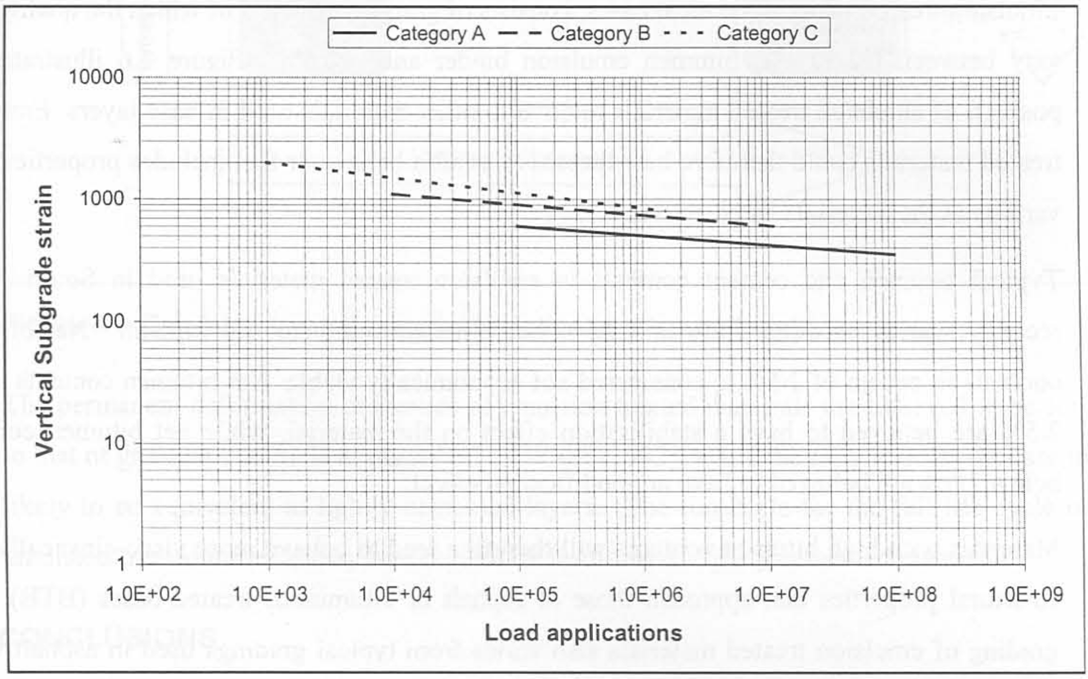

Figure 4.5 Criteria for prediction of subgrade performance in Southern Africa Paterson (1978)

Jordaan (1994) developed a regression equation from the available international literature in which the failure criteria is a function of the adopted terminal rut depth.

$$N = \left(\frac{\varepsilon_v}{20^{2.37} Rut} \right)^{-10} \quad \text{(after Jordaan: 1994)} \tag{4.18}$$

where: ε_v = Maximum vertical compressive strain at the top of the subgrade

Rut = Allowable rutting (mm)

In the revision of the TRH4 Flexible Pavement Design document, (Theyse et al: 1995 and Theyse: 1996) the following failure criteria is included into the SAMPDM that incorporate the terminal rut condition as well as the percentile level required:

$$N = 10^{A - 10\log(\varepsilon_v)} \qquad \text{(after Theyse et al: 1996)} \qquad (4.19)$$

where: ε_v = Maximum vertical compressive strain at the top of the subgrade

A = Regression coefficient that includes the percentile level and the terminal rut condition

4.7 MECHANISTIC-EMPIRICAL MODELS APPLICABLE TO EMULSION TREATED MATERIALS.

Emulsion treated material, in most cases, consists of granular material of which the quality may vary between G2 to G7, bitumen emulsion binder and cement. Figure 4.6 illustrates the position of emulsion treated materials relative to other materials used in base layers. Emulsion treated materials could therefore be expected to exhibit behaviour that includes properties from various of the materials listed above.

Typical bitumen and cement contents in emulsion treated materials used in South Africa recently, varies between 1.0% and 2.5% net bitumen and 1 to 2% cement. Net bitumen contents in excess of 2.5% is considered not economically viable. Net bitumen contents above 2.5% are believed to have a stabilisation effect on the material, while net bitumen contents below 2.5% are believed to have an modification effect.

Materials with high bitumen contents will therefore tend to behave more visco-elastically with structural properties that approach those of asphalt or bituminous treated bases (BTB). The grading of emulsion treated materials also varies from typical gradings used in asphalt where the grading requirements of asphalt material are more closely controlled. Materials with lower (less than 2.5%) contents of net bitumen would therefore not behave similar so asphaltic materials.

Materials with low bitumen- and cement contents (typically less than 1%) would tend to behave more according to the Möhr-Coulomb theory with structural properties like unbound granular material. The applicability of the Möhr-Coulomb theory to materials with cement- and net bitumen contents typically expected in emulsion treated materials, is not researched, but is expected to be applicable.

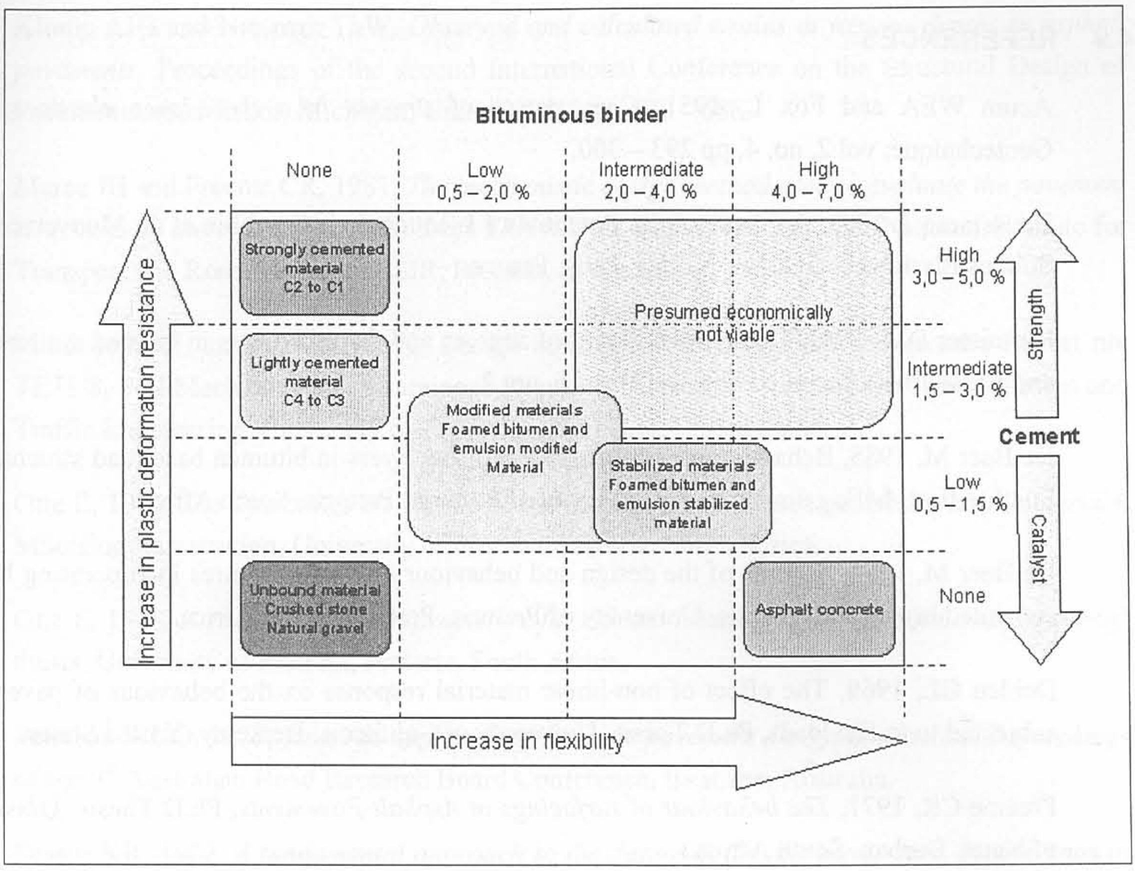

Figure 4.6 Emulsion treated materials relative to other materials

The permanent deformation properties of emulsion treated materials are believed to be similar to that of granular materials as described by Wolff (1992), while the fatigue properties are more likely to be equivalent to lightly cemented layers. The timescale for the full life cycle of an emulsion treated material may however differ from that of lightly cemented materials.

4.8 CONCLUSIONS

The South African Mechanistic Pavement Design Method includes design models for four generic material types commonly used for pavement structural layers in South Africa. These include granular materials, strongly and lightly cemented layers, bituminous surfacings and bases as well as subgrade materials. Of these materials, the lightly cemented material and the granular material, seem to also represent an emulsion treated material, in terms of general behaviour. No transfer function for emulsion treated material is included in the current South African Mechanistic Pavement Design method.

The fatigue life transfer function for lightly cemented material should be evaluated to predict the fatigue life for emulsion treated materials. The transfer functions for permanent deformation developed by Maree (1982) as well as Wolff (1992) should be evaluated for applicability to predict the permanent deformation of emulsion treated layers.

4.9 REFERENCES

Acum WEA and Fox L, 1951, *Computation of stresses in a three-layer elastic system*, Geotechnique, vol.2, no. 4, pp 293 – 300.

Boussinesq J, 1883, Application des Potentials à L'Etude de L'Equilibre et du Mouvement des Solides Elastiques, Gauthier-Villars, Paris, France.

Burmister DM, 1945, The general theory of stresses and displacements in layered systems I, II and III, Journal of applied Physics, Vol. 16, no. 2.

De Beer M, 1985, Behaviour of cementitious subbase layers in bitumen base road structures in South Africa, MEng dissertation, University of Pretoria, Pretoria, South Africa.

De Beer M, 1989, Aspects of the design and behaviour of road structures incorporating lightly cemented layers, PhD Thesis, University of Pretoria, Pretoria, South Africa.

Dehlen GL, 1969, The effect of non-linear material response on the behaviour of pavements subjected to traffic loads, Ph.D Thesis, University of California, Berkeley, United States.

Freeme CR, 1971, *The behaviour of surfacings in Asphalt Pavements*, Ph.D Thesis, University of Natal, Durban, South Africa.

Freeme CR and Strauss JA, 1979, *Towards the Structural Design of More Economical Pavements in South Africa*, Proceedings of the Third Conference on Asphalt Pavements for Southern Africa, Durban, South Africa.

Harvey JT, Deacon JA, Tsai B and Monismith CL, 1995, *Fatigue performance of asphalt and its relationship to asphalt concrete pavement performance in California*, Asphalt Research Program, CAL/APT Program, Institute of Transportation Studies, Iniversity of California, Berkeley, United States.

Heukelom W and Klomp AJG, 1964, *Road design and dynamic loading*, Proceedings of the Association of Asphalt Paving Technologists, Vol I pp 92 –123.

Jones A, 1962, *Tables of stresses in three-layer elastic systems*, Highway Research Board Bulletin 342.

Jordaan GJ, 1988, *Analysis and developments of some pavement rehabilitation design methods*, PhD thesis, University of Pretoria, Pretoria, South Africa.

Jordaan GJ, 1994, *The South African Mechanistic Pavement Rehabilitation Design Method*, Department of Transport, Research Report RR91/242, Pretoria, South Africa.

Kingham IR and Kallas BF, 1972, *Laboratory fatigue and its relationship to Pavement Performance*, Proceedings of the 3rd International Conference on the Structural Design of Asphalt Pavements, Vol I pp 849 – 865, London, United Kingdom.

Klomp AJG and Niesman ThW, *Observed and calculated strains at various depths in asphalt pavements*, Proceedings of the second International Conference on the Structural Design of Pavements, Ann Arbor, Michigan, United States, pp 671 – 688.

Maree JH and Freeme CR, 1981, *The mechanistic design method used to evaluate the pavement structures in the catalogue of the draft TRH4:1980*, Report no. RP/2/81, National Institute for Transport and Road Research, CSIR, Pretoria, South Africa.

Monismith CL and Mclean, 1972, *Design considerations for asphalt pavements*, Report no. TE71-8, Soil Mechanics and Bituminous Research Laboratory, Institute for Transportation and Traffic Engineering, University of California, Berkeley, United States.

Otte E, 1972, *Die Spannings en Vervormingseienskappe van Sementgestabiliseerde Materiale*, MSc(Eng) dissertation, University of Pretoria, Pretoria, South Africa.

Otte E, 1978, *A structural design procedure for cement-treated layers in pavements*, DSc(Eng) thesis, University of Pretoria, Pretoria, South Africa.

Paterson WDO, 1978, *Towards applying mechanistic pavement design in practice*, Proceedings of the 9th Australian Road Research Board Conference, Brisbane, Australia.

Peattie KR, 1962, *A fundamental approach to the design of flexible pavements*, Proceedings of the First International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, United States, p 403.

Pell PS, 1962, *Fatigue characteristics of bitumen and bituminous mixes*, Proceedings of the First International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, United States, pp 310 – 323.

Pell PS, 1967, F*atigue of Asphalt Pavement Mixes*, Proceedings of the Second International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, United States, pp 577 – 593.

Portland Cement Association, 1959, *Soil-cement laboratory handbook*, Portland Cement Association, Chicago, United States.

Shackleton MC, 1995, *Modelling of the permanent deformation of untreated pavement layers*, MEng dissertation, University of Pretoria, Pretoria, South Africa.

SHELL, 1963, *Charts for flexible pavements*, Shell International Petroleum Company Limited, London, United Kingdom.

SHELL, 1978, *Shell pavement design manual – asphalt pavements and overlays for road traffic*, Shell International Petroleum Company Limited, London, United Kingdom.

Theyse HL, de Beer M, Prozzi J and Semmelink CJ, 1995, *TRH4 Revision (1995): Phase I: Updating the South African Mechanistic Design Method*, Report no. I/PA/13/95, National Service Contract NSC-24/1, Transportek, CSIR, Pretoria, South Africa.

Theyse HL, de Beer M and Rust FC, 1996, *Overview of the South African Mechanistic Pavement Design Analysis Method*, Divisional Publication DP-96/005, Transportek, CSIR, Pretoria, South Africa.

Theyse HL, 1998, *Towards Guidelines on the Structural Design of Pavements with Emulsion-treated Layers*, Contract Report CR-97/045, Transportek, CSIR, Pretoria, South Africa.

Theyse, HL, 2001, Personal correspondence and communication.

Westergaard HM, 1938, *A problem of elasticity suggested by a problem in Soil Mechanics: Soft material reinforced by numerous strong horizontal sheets*, in Contribution to the Mechanics of Solids, Stephan Timoshenko 60th Anniversary Vol., Macmillan, New York, Unites States.

Wolff H, 1992, *The Elasto-plastic behaviour of granular pavement layers in South Africa*, PhD thesis, University of Pretoria, Pretoria, South Africa.