CHAPTER 3 STATE OF THE ART ON THE STRUCTURAL BEHAVIOUR AND DESIGN OF EMULSION TREATED PAVEMENT LAYERS

3.1 INTRODUCTION

Mechanistic design procedures for the structural design of pavements are well established in pavement design. The South African Mechanistic Pavement Design Procedure (SAMPDP) (Maree and Freeme: 1981, Jordaan: 1994 and Theyse et al: 1996) provides guidelines on the structural design for pavements with granular, cement treated and hot mix asphalt pavement layers. The TRH4 (COLTO: 1996) pavement design catalogue was based on the SAMPDP and provides a catalogue of pavements for new roads and rehabilitation (COLTO: 1997) for different road categories, climate types and traffic classifications. The structural design for new roads and rehabilitation of pavements containing emulsion treated layers, are not included in the TRH4 or the SAMPDP and are limited to papers published and the experience of practitioners.

3.2 THE HISTORY AND BACKGROUND TO STRUCTURAL DESIGN OF PAVEMENTS CONTAINING EMULSION TREATED MATERIALS

Emulsion was added to pavement layers initially to enhance the water resisting properties of the layer and to improve cohesion on the surface to allow the road to be opened to traffic soon after construction to prevent or limit ravelling of the base layer. In the first experiments in South Africa (Otte and Marais: 1979), no cement was added to the emulsion treated layer and it was treated as a granular layer during the structural design process.

In 1969 Kari (1969) presented a design procedure based on layer equivalency. He calculated the stresses and strains to determine the thickness of the emulsion treated layer to limit the subgrade strain.

Santucci (1977) published a design procedure for bitumen and emulsion treated materials. His research was done on high bitumen contents (11% by volume or 5 to 5.5% by mass). He developed transfer functions for emulsion treated as well as emulsion and cement treated materials. The maximum horizontal tensile strain at the bottom of the treated layer and the maximum vertical compressive strain at the top of the subgrade were used to determine the thickness of the pavement. The method assumed that the properties of the emulsion treated layer were similar to that of asphalt.

Marais and Tait (1989) made some adjustments to the method of Santucci (1977) to allow for South African conditions.

In 1993 SABITA published manual 14 (SABITA: 1993), and provided different approaches for modification and stabilisation. The structural design for the stabilisation approach was based on the work of Santucci (1977) and Marais and Tait (1989). In the modification approach, the emulsion treated material was treated similarly to a granular material.

De Beer and Grobler (1994) developed transfer functions based on research done on the Heilbron test sections. The method was regarded as too conservative because it proposed thick structures contrary to the experience of practitioners.

Theyse (1998) provided guidelines and a proposed design catalogue for pavements containing emulsion treated materials with low bitumen contents (less than 1.8% net bitumen) based on the DCP design approach.

The SABITA manual 21 (SABITA: 1999), provided guidelines on the mix design and construction of emulsion treated layers. It only proposed the use of a catalogue included in the document for structural design purposes.

3.3 FAILURE MECHANISM OF EMULSION TREATED LAYERS

The bitumen content greatly dictates the failure mechanism of emulsion treated materials. Emulsion treated materials with high bitumen contents tend to behave more like asphalt materials with the dominant failure mechanism being fatigue cracking at the bottom of the layer. Santucci (1977) and Marais and Tait (1989) used fatigue cracking at the bottom of the layer for the failure mechanism on the materials with a high bitumen content (11% by volume and 5% air voids). The SABITA manual 14 (SABITA: 1993), distinguishes between the failure mechanisms of stabilised and modified emulsion treated materials. The theory of Santucci is used for net bitumen contents exceeding 2%. For low bitumen contents, the visco-elastic behaviour of the material will be less evident and the behaviour of the material will primarily be similar to that of untreated gravels or aggregates. According to SABITA (1993) the failure mechanism for low bitumen content emulsion treated materials are similar to that of granular material, which is shear failure or gradual deformation at a steady rate under repeated loading. Both these modes of failure are believed to be related to the ratio between the applied shear stresses and the shear strength of the material under prevailing conditions of moisture and density (Theyse: 1998). The cement contents in these materials are only marginal (1 to 1.5%) to assist in the breaking of the emulsion.

De Beer and Grobler (1994) introduced the concept of fatigue cracking followed by the fracturing of the emulsion treated layer. This includes the development of fatigue-like cracking in the layer, starting mainly at the bottom of the layer, and progresses upward through the weak areas and around the larger aggregates. At an advanced stage of cracking, most of the initial fatigue cracks emerge mainly around the larger aggregates and the layer is then transformed into the fractured or advanced granular state. During this process the elastic moduli of the emulsion treated material reduces to levels known for unbound gravel materials.

Theyse (1998) stated that the general behaviour of emulsion treated materials are most likely to be equivalent to granular and lightly cemented materials.

3.4 THE BEHAVIOUR OF EMULSION TREATED PAVEMENT LAYERS

3.4.1 The Long Term pavement performance of pavements (LTPP) with emulsion treated pavement layers

A number of pavements with emulsion treated base layers in South Africa are being monitored regularly to obtain understanding and knowledge on the performance of these pavements under normal traffic and climatic conditions (Wright et al: 1991 and Steyn: 1997). Some of the sections that are being monitored are outlined in Table 3.1. The pavement structures of these sections consist of an emulsion treated base with thicknesses varying from 100 mm to 200 mm and net binder contents between 0.1 and 2.3%, with the majority below 1%. Unfortunately no reference to the presence of cement in any of these sections could be found. All the sections have cemented subbases except the P66/1 and the N7/7 (Steyn: 1997) which have granular subbases.

Table 3.1 LTPP sections for pavements with emulsion treated layers (Steyn: 1997)

Test section	Description	Construction year	Traffic data AADT (year)	Traffic data % Heavy	Traffic demand class	Binder content (%)[a]	Cement content (%)[a]
N12/19	S12: Witbank	1974	12 710 (1993)	13.3	ES100	0.7[b]	0
P66/1	Wepener	1989	486 (1988)	19.0	ES1	0.9	1.5% (lime)
TR13/3	Britstown	1994	496 (1993)	21.1	ES1	2.0	N/A
N7/7	Kamieskroon	1986	715 (1993)	16.6	ES1	1.0	N/A
N1/1	Cape Town	1984	15 900 (1993)	9.9	ES100	1.0	N/A
MR27	Stellenbosch	1988	N/A	N/A	ES100	1.0	N/A
N2/16	East London	1982	3 953 (1993)	11.2	ES3	1.0	1.0
N3/4	Pietermaritz-burg bypass	1988	10 743 (1993)	17.9	ES100	1.0	N/A

[a] Wright et al (1991)

[b] From bitumen extraction test. (Steyn:1997)

Detailed tests were done on most of the sections and are presented elsewhere (Wright et al: 1991, Steyn: 1997 and Laatz: 1990). Most of the sections are still in a sound condition and behave well. During 1997 little permanent deformation was present while the DCP penetration rates compared well with that of good quality crushed stone materials. It seems therefore that after years of service, most of the pavements still have adequate shear strength to withstand the loads applied by traffic.

3.4.2 The behaviour of emulsion treated layers under repeated loading (HVS testing)

Accelerated testing with the Heavy Vehicle Simulator (HVS) system (Freeme et al, 1982) and technology have advanced the understanding of pavement behaviour and modelling in South Africa significantly (Freeme et al, 1987). The HVS is a mobile accelerated pavement testing rig that tests as-built pavements. Acceleration of testing is achieved through the backwards and forward movement of a wheel load over a selected 8m by 1m test section. The wheel load can be loaded with loads that vary from 40 kN to 200 kN. Additional to the test rig, sophisticated response measuring equipment were developed to measure deformation and deflection on the road surface and in depth, crack movement and moisture content.

Tests done on emulsion treated materials by De Beer and Grobler (1993) reported a reduction of the initial modulus after approximately 10 000 HVS repetitions (Figure 3.1) that resulted in a change in the deflection bowl (Figure 3.2). The change in the deflection bowl indicated that the load spreading characteristics of the layer had changed and that more load was being transferred to the lower layers of the pavement. This resulted in a decrease in the radius of curvature and a deeper but smaller deflection bowl. The change of load bearing characteristics was an indication that the layer had reached the end of its fatigue life and was now functioning as a cracked layer in an equivalent granular phase. This behaviour is similar to that of lightly cemented layers as described by de Beer (1989). Similar behaviour was reported by Jordaan and Horak (1991) during HVS tests on emulsion treated base layers in the Eastern Cape. Horak and Viljoen (1981) found that emulsion treated material exhibits behaviour similar to a good quality crushed stone (G1 or G2) rather than that of an asphalt layer.

The expected life in terms of E80's to 20 mm rutting for the emulsion treated layers tested by de Beer and Grobler (1994) were between 4 and 24 million. These tests were done at very high dual wheel loads of 140 kN, that may result in the end of the fatigue life of the layer after only 10 000 repetitions. The emulsion treated layer also showed a strong stress dependency, similar to granular materials, as can be seen in Figure 3.1.

The pavements showed sensitivity to overloading, but that may be as a result of the subbase support that is important to the behaviour of emulsion treated base layers, according to Horak and Viljoen (1981).

Wheel loads applied to a pavement are converted to equivalent 80 kN axle loads (E80's) by using a equivalency factor F_n, where:

$$F_n = \left(\frac{P}{80}\right)^d \quad (3.1)$$

where: F_n = equivalency factor for load P

P = axle load in kN

d = damage coefficient, which is dependent on the pavement type and material state

The damage coefficient (d) is similar to the "n" value as established during the AASHO road test. The coefficient n, determined during the AASHO road test as an average of 4.2, relates to the riding quality of a pavement. The coefficient d was later defined and relates to other forms of distress like cracking, rutting, etc. During accelerated pavement testing, the value of d may vary depending on the pavement type, material state, loading history, etc.

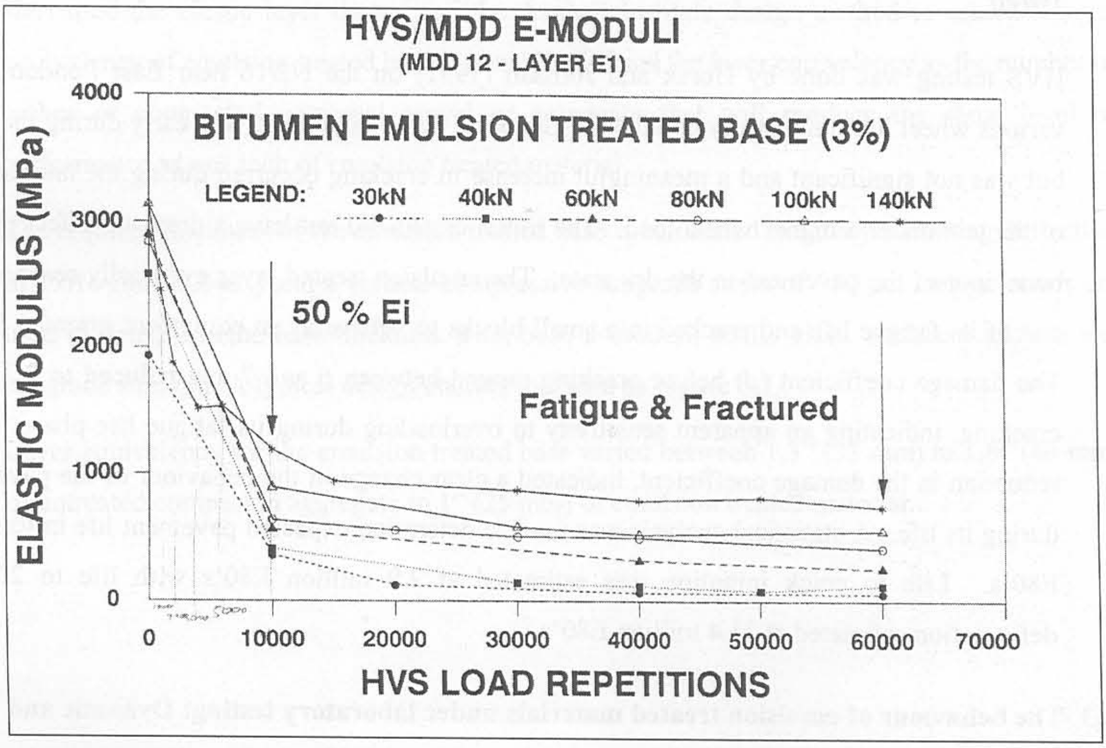

Figure 3.1 Backcalculated E-moduli at various HVS repetitions (De Beer and Grobler: 1993)

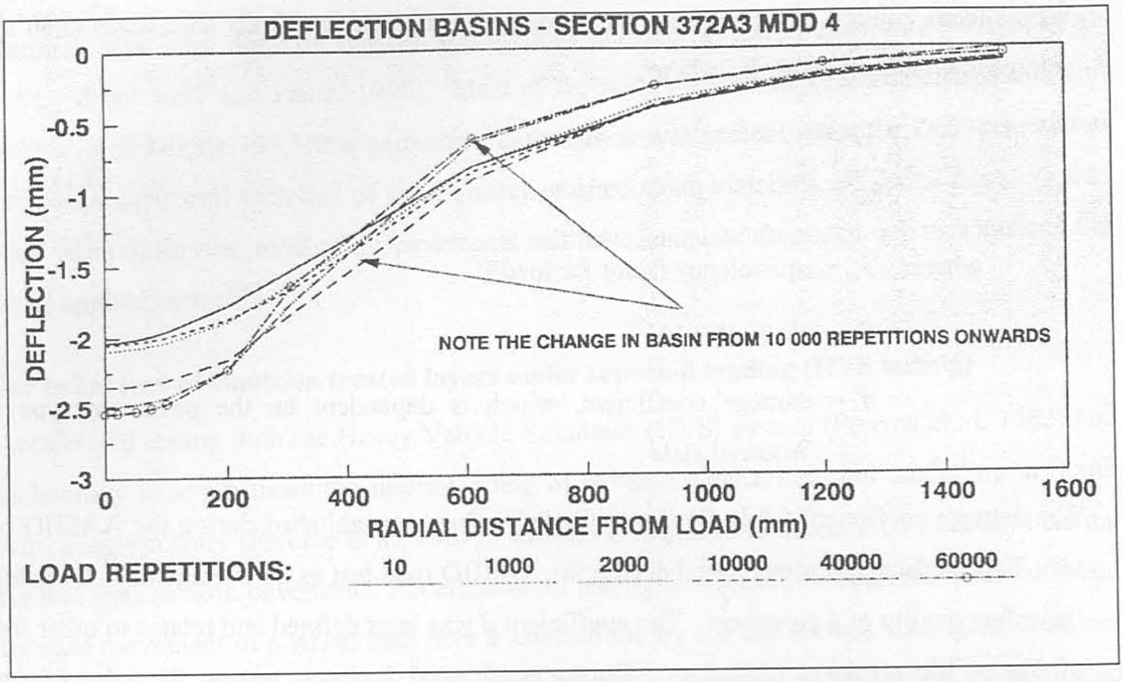

Figure 3.2 Measured deflection basins at various repetitions (De Beer and Grobler: 1993)

HVS testing was done by Horak and Jordaan (1991) on the N2/16 near East London with various wheel loads under dry conditions. Cracking was visible relatively early during the test, but was not significant and a meaningful increase in cracking occurred during the later stages of the test under a higher wheel load. The initial cracks did not have a dramatic effect on the behaviour of the pavement in the dry state. The emulsion treated layer eventually reached the end of its fatigue life and cracked into small blocks to behave as an equivalent granular layer. The damage coefficient (*d*) before cracking ranged between 6 and 7 and reduced to 2.7 after cracking, indicating an apparent sensitivity to overloading during its fatigue life phase. This reduction in the damage coefficient, indicated a clear change in the behaviour of the pavement during its life. A statistical analysis was used to determine expected pavement life in terms of E80's. Life to crack initiation was estimated at 2.9 million E80's with life to 20 mm deformation estimated at 31.4 million E80's.

3.4.3 The behaviour of emulsion treated materials under laboratory testing: Dynamic and static testing

Maree et al (1982) performed static triaxial tests on crushed stone material stabilised with small percentages of bitumen or tar. They found that the internal angle of friction reduced from values between 57° and 60° to values between 49° and 54° with an increase in emulsion or tar content. This is as a result of the lubricating effect of the bitumen emulsion or tar. The degree of saturation had little effect on the internal friction of the treated material. The treated material was less sensitive to elastic deformation, than for untreated material and failure occurred after increased axial deformation under a lower maximum deviator stress.

Maree et al (1982) found that the cohesion also reduced with an increase in emulsion content. This was against expectation. The cohesion was however less sensitive to saturation in treated samples than in untreated samples. The lubricating effect could also be responsible for this during testing. The reduction in cohesion is possibly because the curing process had not been completed and that the cohesion would increase to higher values when the material is fully cured.

Horak et al (1984) and Maree et al (1982) reported a stress sensitivity in laboratory tests for the treated materials with a similar slope to that of granular materials but with a higher initial value.

3.5 STRUCTURAL DESIGN OF EMULSION TREATED MATERIALS

3.5.1 Kari (1969)

Kari used the elastic layer theory and the Asphalt Institute design method to establish layer equivalency of emulsion treated base layers. He defined the layer equivalency as the number of inches of compacted untreated gravel or aggregate that will produce the same level of performance as one inch of emulsion treated material.

The required thickness of the emulsion treated base is calculated to satisfy a limiting deflection of 1.016 mm (0.040") and a vertical compressive subgrade strain of 980 $\mu\varepsilon$. Design charts are used to determine the base thickness from base E-moduli, traffic class, surface thickness and subgrade strength. A typical design chart is included as Figure 3.3.

Layer equivalency for the emulsion treated base varied between 1.3" (33 mm) to 1.8" (46 mm) of untreated compacted aggregate to 1" (25 mm) of emulsion treated material.

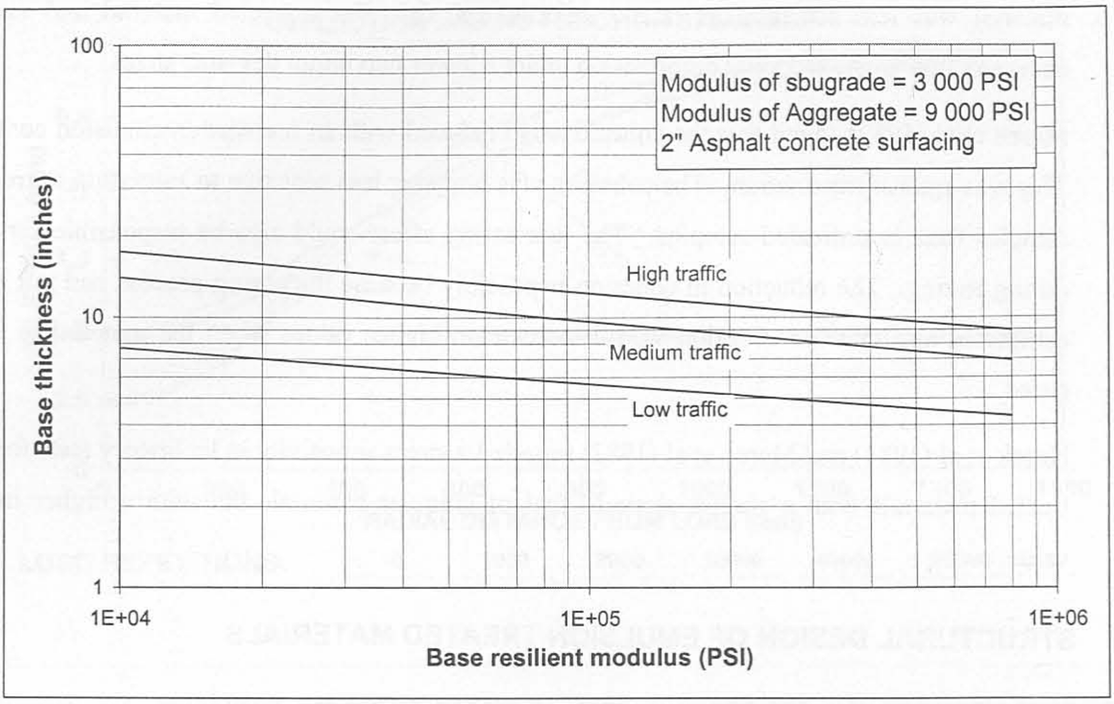

Figure 3.3 Design chart for ETB for subgrade modulus of 20 MPa (3 000 psi) (Kari: 1969)

3.5.2 Santucci (1977)

Santucci used the elastic layer principles to determine required layer thickness using the maximum horizontal tensile strain at the bottom of the emulsion treated layer and the maximum vertical compressive strain at the top of the subgrade. He presented transfer functions for materials treated with emulsion and materials treated with cement and emulsion in terms of the critical parameter (ε_t at bottom of layer) and load repetitions to failure (Figure 3.4). Failure of the emulsion treated base layer was defined when the fatigue cracks reflect through to the surface. The fatigue curves were shifted to allow for crack propagation through the layer. These fatigue lines were developed for relatively high emulsion contents (11% by volume or 5 to 6% by mass) and 5 % voids.

A correction (Equation 3.2) for different void- and bitumen contents was supplied based on research by Pell and Cooper (1975) as well as Epps (1968).

$$N_c = N_f . 10^M \tag{3.2}$$

with: N_c = Corrected number of repetitions

N_f = Number of repetitions to failure from transfer function

$$M = 4.84\left(\frac{V_B}{V_V + V_B} - 0.69\right) \tag{3.3}$$

with: V_B = Bitumen content by volume and V_V = Void content

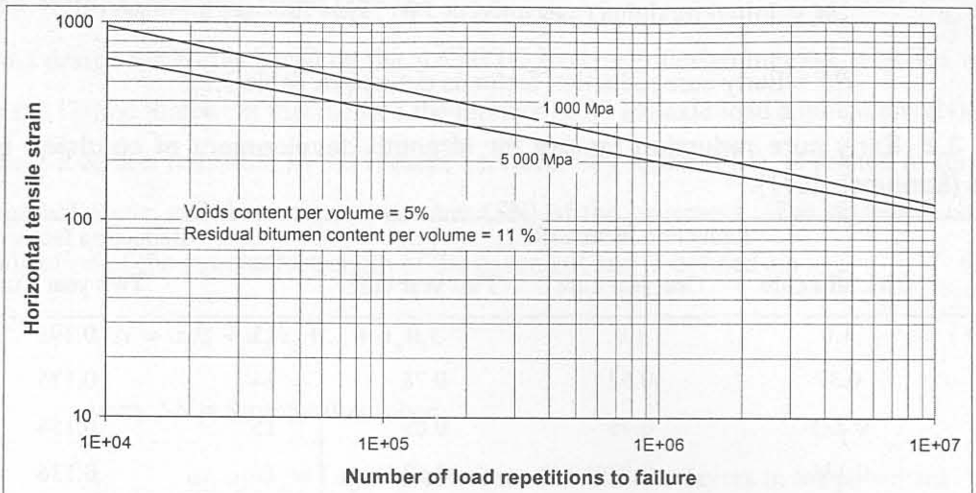

Figure 4.3a Fatigue criteria for granular emulsion mixes

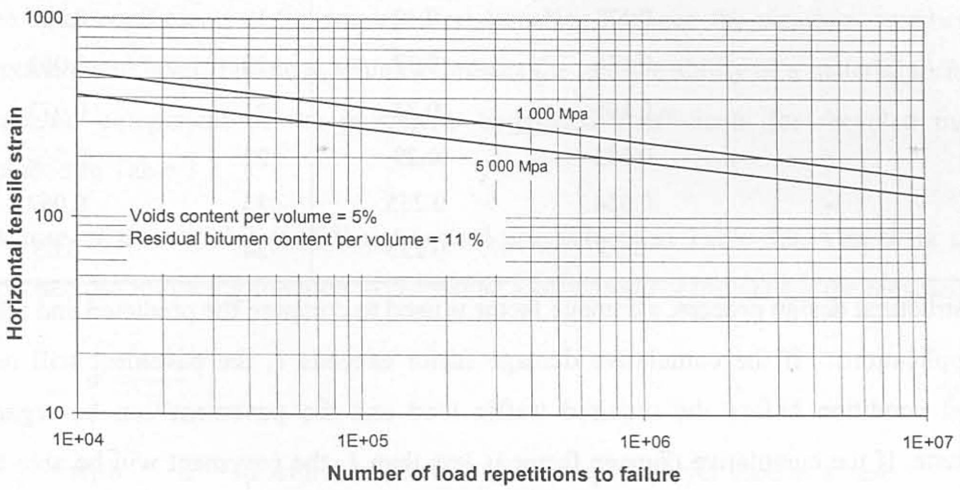

Figure 3.4b Fatigue criteria for cement modified granular emulsion mixes

Figure 3.4 Fatigue criteria for emulsion treated material for 11% bitumen and 5% air voids by volume (Santucci: 1977)

Santucci also provided a detailed procedure to allow for the influence of seasonal temperature effects and curing during the early life of the layer, on the resilient modulus of the material. The resilient modulus for design purposes at a specific time after construction can be calculated by Equation 3.4.

$$M_t = M_f - (M_f - M_i)RF \tag{3.4}$$

with M_t = Modulus at a specific time after construction at 23°C (73°F)

M_f = Final modulus (measured at 23°C after three day air cure and four day vacuum cure at room temperature)

M_i = Initial modulus (measured at 23°C after one day air cure)

RF = Early cure reduction factor as defined in Table 3.2.

Table 3.2 Early cure reduction factors for strength development of emulsion treated layers (Santucci: 1977).

Month	Reduction factors (RF)			Month	Reduction factors (RF)
	6 Months cure	One year cure	Two year cure		Two year Cure
1	1.0	1.0	1.0	13	0.198
2	0.37	0.62	0.78	14	0.175
3	0.225	0.48	0.69	15	0.154
4	0.136	0.37	0.62	16	0.136
5	0.082	0.29	0.545	17	0.120
6	0.05	0.225	0.48	18	0.105
7	-	0.175	0.42	19	0.093
8	-	0.136	0.37	20	0.082
9	-	0.105	0.33	21	0.073
10	-	0.082	0.29	22	0.064
11	-	0.064	0.255	23	0.057
12	-	0.05	0.225	24	0.05

In the structural design process, a damage factor is used to compare the predicted and allowable load applications. If the cumulative damage factor exceeds 1, the pavement will reach its terminal condition before the required traffic load and the pavement can be regarded as inadequate. If the cumulative damage factor is less than 1, the pavement will be able to carry the required load before the terminal condition is reached. The Cumulative damage factor for the ith loading is defined as the number of repetitions (n_i) of a given response parameter divided by the 'allowable' repetitions (N_t) of the response parameter that would cause failure. The Cumulative Damage Factor for the parameter is given by summing the damage factors over all the loadings in the traffic spectrum and is calculated as follows:

$$CDF = \sum_n \frac{n_i}{N_t} \tag{3.5}$$

with: CDF = Cumulative damage factor

n_i = Damage of the ith load repetition

N_t = Total number of allowable load repetitions

This is an iterative process until the required layer thickness is determined. Permanent deformation of the subgrade layer is also considered during the structural design process.

3.5.3 Van Wijk, Yoder and Wood (1984)

This design method is based on the AASHTO pavement design method, which is widely used in the United States. In this method the number of 80 kN axle load applications (N) required to cause a certain reduction in the Present Serviceability Index (PSI) is related to soil support, a regional factor and the structural number (SN) of the pavement. The structural number is an indication of the structural strength of the pavement and is defined as:

$$SN = a_1h_1 + a_2h_2 + \ldots + a_nh_n \tag{3.6}$$

with: SN = Structural number

a_1, a_2, a_n = Layer coefficients for different layers in the pavement

h_1, h_2, h_n = Respective layer thicknesses

The layer coefficients is the empirical relationships between the structural number (SN) of a pavement and layer thickness, which expresses the relative ability of a material to function as a structural component of the pavement. Suggested coefficients for recycled materials are included in Table 3.3

Because of the variability of the structural coefficients in Table 3.3, Van Wijk et al (1984) proposed the following function to determine a structural coefficient for the recycled layer.

$$a' = \frac{a.h'}{h} \tag{3.7}$$

with: a' = structural coefficient of the recycled layer used as a base

a = structural coefficient of the AASHTO bituminous layer (0.44)

h' = thickness of the recycled layer

h = thickness of the AASHTO asphalt layer to give the same strain, deformation or fatigue life

The method then proceeds to determine the required thickness of the recycled layer to satisfy the required structural number for the pavement

Table 3.3 Suggested structural coefficients for recycled layers (Van Wijk et al: 1984)

Material	Binder(s)	Coefficient Range	Coefficient Average
Cold recycled asphaltic material	Bitumen emulsion	0.30 – 0.38	-
In-situ recycled bituminous concrete base	Bitumen emulsion	0.22 – 0.39	-
Central plant recycled bituminous concrete used as base	Bitumen emulsion	0.37 – 0.59	-
In-place recycled bituminous concrete	Bitumen emulsion	0.22 – 0.49	0.39
	Lime	-	0.40
	Cement	0.23 – 0.43	0.33
AASHTO surface	-		0.44
AASHTO base		-	0.35

3.5.4 Marais and Tait (1989)

Marais and Tait published a structural design procedure for emulsion treated materials based on the work done by Santucci (1977). A number of adjustments were made to the method to allow for South African conditions. The influence of curing and seasonal temperature effects on the elastic modulus were included on a quarterly basis rather than on a monthly basis as proposed by Santucci. The value of the resilient modulus for each cured quarter is presented in Table 3.4. Resilient modulus values from laboratory tests at 23°C and 40°C at the initial and fully cured states are used as input into the structural design. The average base temperatures applicable in South Africa for each quarter are outlined in Table 3.5 and the respective moduli should be used in the structural design.

Fatigue cracking initiated at the bottom of the layer and progressed through the layer to the surface was regarded as the mechanism of failure for the layer and the same transfer function published by Santucci (1977) was used. The subgrade strain criteria are the same as those used by Maree and Freeme (1981) for the evaluation of the pavement structures in TRH4 (CSRA, 1980).

Table 3.4 Resilient modulus values of emulsion treated base layers for each quarter for different full cure periods. (Marais and Tait: 1989)

Quarter	Full curing period		
	6 months	1 year	2 years
1	$0.46(M_f\text{-}M_i)+M_I$	$0.30(M_f\text{-}M_i)+M_i$	$0.18(M_f\text{-}M_i)+M_i$
2	$0.94(M_f\text{-}M_i)+M_I$	$0.70(M_f\text{-}M_i)+M_i$	$0.46(M_f\text{-}M_i)+M_i$
3	M_f	$0.88(M_f\text{-}M_i)+M_i$	$0.64(M_f\text{-}M_i)+M_i$
4	M_f	$0.98(M_f\text{-}M_i)+M_i$	$0.74(M_f\text{-}M_i)+M_i$
5	M_f	M_f	$0.86(M_f\text{-}M_i)+M_i$
6	M_f	M_f	$0.92(M_f\text{-}M_i)+M_i$
7	M_f	M_f	$0.96(M_f\text{-}M_i)+M_i$
8	M_f	M_f	$0.99(M_f\text{-}M_i)+M_i$

With M_f = Resilient modulus at fully cured state

M_i = Resilient modulus at initial cured state

Table 3.5 Average base temperatures in southern Africa (Marais and Tait: 1989)

Quarter	Temperature
Summer	100 % at 40°C
Autumn	50% at 40°C and 50% at 23°C
Winter	100% at 23°C
Spring	50% at 23°C and 50% at 40°C

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2	$0.94(M_f-M_i)+M_i$	$0.70(M_f-M_i)+M_i$	$0.46(M_f-M_i)+M_i$
3	M_f	$0.88(M_f-M_i)+M_i$	$0.64(M_f-M_i)+M_i$
4	M_f	$0.98(M_f-M_i)+M_i$	$0.74(M_f-M_i)+M_i$
5	M_f	M_f	$0.86(M_f-M_i)+M_i$
6	M_f	M_f	$0.92(M_f-M_i)+M_i$
7	M_f	M_f	$0.96(M_f-M_i)+M_i$
8	M_f	M_f	$0.99(M_f-M_i)+M_i$

With M_f = Resilient modulus at fully cured state

M_i = Resilient modulus at initial cured state

Table 3.5 Average base temperatures in southern Africa (Marais and Tait: 1989)

Quarter	Temperature
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Autumn	50% at 40°C and 50% at 23°C
Winter	100% at 23°C
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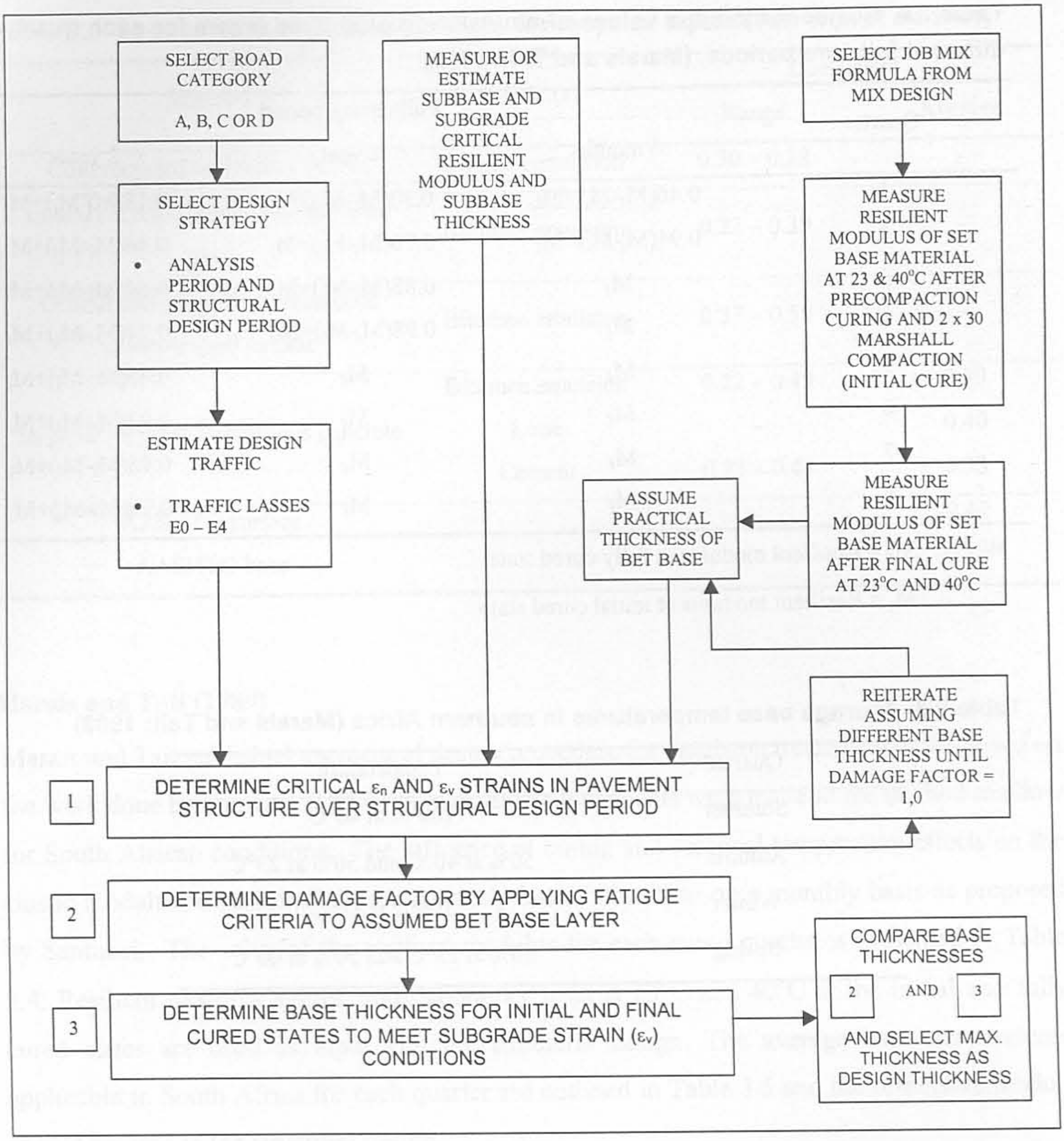

Figure 3.5 Flow diagram for structural design of emulsion treated bases (Marais and Tait: 1989)

3.5.5 SABITA manual 14: GEMS: The design and use of granular emulsion mixes (1993)

The structural design for emulsion treated layers with higher bitumen contents (greater than 2%) are regarded as stabilisation and the approach by Santucci (1977) and Marais and Tait (1985) are followed.

Emulsion treated materials with low bitumen contents (lower than 1.5%) are treated the same as granular materials. For mechanistic analysis the cohesion (c) normally used for the untreated material can be doubled for a modification design of emulsion treated materials. The proposed values for cohesion and internal friction are given in Table 3.6.

Table 3.6 Material properties for emulsion treated materials using the modification approach (SABITA: 1993)

Material code	Moisture state	Apparent Cohesion, c (kPa)	Internal friction, ϕ (°)
G1	Wet	130	55
	Dry	90	55
G2	Wet	110	52
	Dry	80	52
G3	Wet	100	50
	Dry	70	50
G4	Wet	90	48
	Dry	70	48
G5	Wet	80	43
	Dry	60	43

A safety factor against shear failure is calculated as follows:

$$FOS = \frac{\sigma_3 (K.\tan^2(45+\frac{\phi}{2})-1)+2K.c.\tan(45+\frac{\phi}{2})}{\sigma_1-\sigma_3} \qquad (3.8)$$

where FOS = Safety factor against shear failure

σ_3 and σ_1 = Calculated major and minor principal stresses acting in the middle of the layer (kPa)

c = Cohesion (kPa)

ϕ = Angle of internal friction (degrees)

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

The allowable minimum safety factor for emulsion treated layers at various traffic levels are given in Table 3.7.

The transfer function (Figure 3.6) used to determine the number of load repetitions to shear failure is similar to the one used by Maree and Freeme (1981) for the evaluation of the pavement structures in TRH4 (CSRA: 1980).

Table 3.7 Allowable minimum safety factors at various traffic levels (SABITA: 1993)

Road Category	Design traffic class	Minimum safety factor
A	E4 (12 – 50 million E80's)	1.60
	E3 (3 – 12 million E80's)	1.40
B	E3 (3 – 12 million E80's)	1.30
	E2 (0.8 – 3 million E80's)	1.05
	E1 (0.2 – 0.8 million E80's)	0.85
C	E2 (0.8 – 3 million E80's)	0.95
	E1 (0.2 – 0.8 million E80's)	0.75
	E0 (<0.2 million E80's)	0.50

Figure 3.6 Transfer function for shear failure for granular materials (Maree and Freeme: 1981)

3.5.6 De Beer and Grobler (1994)

Structural design criteria for emulsion treated materials in terms of fatigue cracking and fracturing was developed by de Beer and Grobler (1994) on sections with 3% emulsion (1.8 % net bitumen) and pre-treated with lime. The research was done on weathered dolerites classified as a G4 according to the TRH14 (CSRA, 1987) classification system.

The initial failure point was defined where the initial elastic modulus reduced by 50%. A failure criteria for the fatigue and total fractured (granular) state was also developed. This curve represents a condition where the maintenance is good and the pavement is in a relatively dry state with approximately 10 mm rutting.

Initial fatigue: $$N_{if} = 10^{8.6[2.756-\log(\varepsilon_t)]} \quad (3.9)$$

Fatigue and total fracture: $$N_{ff} = 10^{4.6[3.435-\log(\varepsilon_t)]} \quad (3.10)$$

With: N_{if} = Initial fatigue life (Approximately 50% of E_i)

N_{ff} = Fatigue and fracture life (approximately 10 mm rutting)

ε_t = Calculated initial horizontal tensile strain at bottom of emulsion treated layer

They recommended that for design purposes of category B and C pavements, the fatigue and total fracture curve (Equation 3.10) be used. A design catalogue was developed from the above transfer functions and is included as Figure 3.7. No provision was made for curing and the early reduced strength of emulsion treated layers.

3.5.7 Theyse (1998)

Theyse provides interim guidelines on the structural design of pavements with emulsion treated base layers based on the Dynamic Cone Penetrometer (DCP). The DCP design method is an empirical design method, similar to the CBR cover design approach, mainly developed for the evaluation and design of granular pavements. The DCP is primarily used for the evaluation of existing pavements and the processing and interpretation of DCP data is well advanced in South Africa (Kleyn: 1984 and de Beer: 1991).

The principles of the DCP design method, combined with DCP data measured from various long term pavement performance (LTPP) sections, were used to do some quantitative design analysis (Theyse, 1998). A design catalogue was developed and distributed to practitioners for review. The proposed design catalogue is included as Figure 3.8.

ROAD CATEGORY	EMULSION TREATED BASE LAYERS PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE) E0-1 $< 0.5 \times 10^3$	E0-2 $0,5 - 3,0 \times 10^4$	E0-3 $0,3 - 1,0 \times 10^5$	E0-4 $1,0 - 2,0 \times 10^5$	E1-1 $2,0 - 4,0 \times 10^5$	E1-2 $4,0 - 8,0 \times 10^5$	E2 $0,8 - 3,0 \times 10^6$	E3 $3,0 - 10 \times 10^6$
A: Major interurban freeways and roads. (95% approximate design reliability)								
B: Interurban collectors and major rural roads. (90% approximate design reliability)								
C: Lightly trafficked rural roads and strategic roads. (80% approximate design reliability)	S 150 GEMS 150 G4 150 G8 G10	25A 100 GEMS 100 C3 G10	25A 100 GEMS 120 C3 G10	25A 150 GEMS 140 C3 G10	25A 150 GEMS 160 C3 G10	25A 150 GEMS 200 C3 G10	35A 200 GEMS 300 C3 G10	35A 300 GEMS 320 C3 G10
D: Lightly pavement structures, rural access roads. (80% approximate design reliability)								

Maximum traffic: 10 x 10^6 E80's
Symbol A denotes AG, AC or AS
S denotes Double Surface Treatment (seal or combinations of seal and slurry)

Catalogue of GEMS Designs on a Poor Subgrade (E = 15 Mpa)

ROAD CATEGORY	EMULSION TREATED BASE LAYERS PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE) E0-1 $< 0.5 \times 10^3$	E0-2 $0,5 - 3,0 \times 10^4$	E0-3 $0,3 - 1,0 \times 10^5$	E0-4 $1,0 - 2,0 \times 10^5$	E1-1 $2,0 - 4,0 \times 10^5$	E1-2 $4,0 - 8,0 \times 10^5$	E2 $0,8 - 3,0 \times 10^6$	E3 $3,0 - 10 \times 10^6$
A: Major interurban freeways and roads. (95% approximate design reliability)								
B: Interurban collectors and major rural roads. (90% approximate design reliability)								
C: Lightly trafficked rural roads and strategic roads. (80% approximate design reliability)	S 100 GEMS 100 C3 G7	25A 100 GEMS 100 C3 G7	25A 100 GEMS 100 C3 G7	25A 125 GEMS 125 C3 G7	25A 125 GEMS 125 C3 G7	25A 150 GEMS 160 C3 G7	35A 200 GEMS 250 C3 G7	35A 300 GEMS 230 C3 G7
D: Lightly pavement structures, rural access roads. (80% approximate design reliability)								

Maximum traffic: 10 x 10^6 E80's
Symbol A denotes AG, AC or AS
S denotes Double Surface Treatment (seal or combinations of seal and slurry)

Catalogue of GEMS Designs on a Strong Subgrade (E = 90 Mpa)

Figure 3.7. Proposed design catalogue for emulsion treated materials by De Beer and Grobler (1994)

EMULSION TREATED BASE LAYERS

ROAD CATEGORY	PAVEMENT CLASS AND DESIGN BEARING CAPACITY (80 kN AXLES/LANE)										Foundation
	ES0.003 0,1 - 0,3 x 10^4	ES0.01 0,3 -1,0 x 10^4	ES0.03 1,0 - 3,0 x 10^4	ES0.1 3,0 - 10 x 10^4	ES0.3 0,1 - 0,3 x 10^6	ES1 0,3 - 1,0 x 10^6	ES3 1,0 - 3,0 x 10^6	ES10 3,0 - 10 x 10^6	ES30 10 - 30 x 10^6	ES100 30 - 100 x 10^6	
A: Major interurban freeways and roads. (95% approximate design reliability)								40A 125 E1 250 C4	40A 150 E1 150 C3 150 C4		
B: Interurban collectors and major rural roads. (90% approximate design reliability)						S 100 E1 150 C4 S 125 E1 150 G5	S*/30A 125 E1 150 C4 S*/30A 150 E1 150 G5	40A 125 E1 150 C4 40A 150 E1 150 C4			150 G7 150 G9 G10
C: Lightly trafficked rural roads and strategic roads. (80% approximate design reliability)				S 100 E2 150 G5	S 100 E2 150 G5	S 100 E2 125 C4					
D: Lightly pavement structures, rural access roads. (80% approximate design reliability)				S 100 E2 150 G5	S 100 E2 150 G5						150 G9 G10

Symbol A denotes AG, AC or AS
AO, AP may be recommended as a surfacing measure for improved skid resistance when wet to reduce water spray
S denotes Double Surface Treatment (seal or combinations of seal and slurry)
S1 denotes Single Surface Treatment
* is Seal is used, increase C4 and G5 subbase thickness to 200 mm

Most likely combinations of road category and design bearing capacity

Figure 3.8. Proposed design catalogue for emulsion treated materials by Theyse (1998)

3.5.8 SABITA manual 21: ETB: The design and use of emulsion treated bases (1999)

No formal structural design procedure is given in the document and it recommended that the catalogue in the manual be used. The catalogue is the same catalogue proposed by Theyse (1994), which is included above as Figure 3.8.

3.6 Conclusions

This chapter addressed the current state of the art on the structural behaviour and design procedures for pavements with emulsion treated layers. Most of the emulsion treated materials have net bitumen contents between 0.6 and 3%, with cement content generally below 2%. The following conclusions that is of importance to this study may be drawn from this literature study:

- There is general consensus that the behaviour of emulsion treated layers is a two staged behaviour. One stage being the pre-cracked phase where the layer acts as a slab will be governed by fatigue and the other being the equivalent granular phase where the layer will exhibit properties similar to that of granular materials.
- The end of the fatigue life of the treated layer is not yet clearly defined and need to be addressed in this study. The end of the fatigue life may be a function of the emulsion and cement contents and should be done in terms of elastic and plastic parameters, e.g. deflection and horizontal strain. The contribution of the fatigue layer to the total life of the layer is also not clear.
- Emulsion treated materials fail at lower stresses, but at increased deformation than cement treated and crushed stone materials. It is able to withstand more deformation before failure than cemented or granular materials. Higher deflections in emulsion treated layers might therefore not necessarily be an indication of a weak layer. This property needs to be considered when a failure criteria for the end of the fatigue phase is defined.
- The influence of the addition of cement on the structural properties of an emulsion treated material is unknown. The literature suggests that the properties of an emulsion treated layer without cement are similar to that of high quality crushed stone.
- The curing of emulsion treated layers without cement or lime may take several months to complete. This process can be speeded up with the addition of cement or lime. A design modulus for input into the mechanistic design procedure need to be defined.
- Transfer functions that are available are for high bitumen contents in terms of horizontal tensile strain at the bottom of the layer. For low bitumen contents the same transfer function for granular materials in terms of shear strength are used.

- Design catalogues based on DCP tests and experience are available. Their development were limited to some of the long term pavement performance (LTPP) sections in South Africa.
- The influence of cement on the fatigue properties of the emulsion treated layer are not known. In the first phase is it a cement treated layer with emulsion or an emulsion treated layer with cement?

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