CHAPTER 4

QUANTIFICATION OF CEMENTITIOUS LAYERS IN THE POSTCRACKED PHASE

CONTENTS

4.1 INTRODUCTION

A method to evaluate and quantify the effective elastic modulus of relatively weakly cemented subbase materials in the postcracked state during wet and dry periods is proposed in this chapter. The major tools in the field evaluation of the behaviour were the HVS and a theoretical procedure involving linear elastic simulations of the road structure. The different phases of cracking or states of cementitious layers during its "life" is also described.

In order to put perspective to the use of the theoretical procedure in the evaluation of road structures, Table 4.1 was extracted from work done by Otte (1978). The main elements in pavement design theory are traffic, materials, environment, analyses and modes of distress. The main advantage of the HVS system (full scale testing) is that most of the main elements of the design theory are well defined and can be accurately controlled and measured. One particular field of interest is the study of the modes of distress which can be accurately observed during an HVS test, using both permanent and resilient behavioural indicators. A few of the pertinent modes of distress observed on structures containing cementitious layers are

(a) Asphalt deformation
(b) traffic associated cracking of cementitious layers
(c) traffic associated cracking of asphalt layers
(d) surface deformation originated from the upper cementitious subbase layer owing to the lack of durability of the layer during the excess porewater pressure state,
(e) surface deformation, cracking and pumping owing to a poorly cemented upper subbase layer, and
(f) crushing of weakly cemented layers in the dry state during trafficking.

TABLE 4.1 - The development in pavement design theory (after Otte, 1978)

	California bearing ratio (CBR)	State of California	AASHTO	Theoretical procedure
Traffic:				
Number of load repetitions	Not considered	Any number; is calculated from predicted traffic volume.	Any number; is calculated from predicted traffic volume.	Any number; is calculated from predicted traffic volume but depends on materials, for example . E80 for untreated materials . Maximum wheel load for cementitious materials . Total traffic volume for bituminous surfacing.
Magnitude or unit of loading	Light : 31 kN Medium : 40 kN Heavy : 53 kN	Equivalent 22,2 kN wheel loads.	Equivalent 80 kN axles (E80)	Equivalent 80 kN axles and actual loading.
Materials				
Type considered	Untreated	Treated and untreated	Treated and untreated	All road-building materials; also those still to be developed.
Characterized by	CBR	R-value and gravel equivalency factors.	Structural coefficient	. Elastic modulus and Poisson ratio . Allowable stress and strain
Environment	-	R-value is determined from a saturated sample.	Regional Factor	. Adjustments in elastic modulus and allowable stresses and strains. . Calculates and includes effect of thermal and moisture stresses.
Analysis	Single graph	Equation	Tables and equation	Complex equations - must be solved by computer
Outcome of analysis	Total thickness; and thickness of base and subbase	. Gravel equivalent. . Thickness of various layers in structure	. Structural number . Thickness of various layers	. Number of load repetitions until distress. . Critical position in structure and likely mode of distress. . Probable maintenance requirements
Mode of distress	Deformation of subgrade	Permanent deformation in pavement materials.	Loss of riding quality.	All modes of distress

(g) horizontal interfacial layers (horizontal cracking) within a cemented layer. This leads initially to fatigue distress of the remaining "thin" layers within the main layer. These layers eventually undergo shearing and crushing or fracturing. The end result being a cemented layer with the upper part of the cemented layer crushed (granulated).

As was discussed previously in Chapter 3 each of above modes of distress occurred along a certain paths of behaviour depending on the traffic, materials and environment.

4.2 PRECRACKED AND POSTCRACKED STATES OF CEMENTITIOUS LAYERS

Most of the work done on the design and evaluation of cementitious layers in road structures, Otte (1978) and Freeme et al (1984), accept that cracking is unavoidable in cementitious layers, after construction. This cracking can be divided in non-traffic associated and traffic associated cracking. The non-traffic associated cracking involves cracking due to variations in environmental conditions such as temperature and moisture content. According to Otte (1978), little can be done to avoid these modes of cracking, except that it can be protected by an insulating structural layer on top of the cementitious layer to minimize environmental changes and to dampen reflection cracking. These types of cracking must therefore be accepted and incorporated in the design phase. It is however true that much more research in the field of the soil-cement chemistry and construction processes is needed. Factors to investigate with respect to weakly cemented materials (C3 and C4) are:

(i) Modification(i.e. reducing of the plasticity index (PI))

(ii) Cementation (i.e. hydraulic or pozzalanic action)

(iii) Delayed compaction to reduce cracking due to drying shrinkage.

(iv) Initial consumption of lime (ICL)

(v) Cementing lime content (CLC)

(vi) Modification lime content (MLC)

(vii) Both beneficial and detrimental carbonation of cementitious layers

(viii) Delayed mixing of cement after the addition of lime

(ix) Steam curing vs normal curing, etc.

All of the above factors could have a significant influence on the ultimate behaviour of these layers. It is however believed that if environmental and construction type of cracking are avoided or minimized, that the ultimate lives of these layers and hence of the total structure can be lengthened.

This type of cracking however is more important during the precracked state after which mainly traffic associated cracking occures. Work done by Otte (1978) indicated that relatively small movements (0,1 mm to 0,3 mm) occurs in a cracked treated layer owing to environmental changes in temperature. He stated further that:

> "The outcome of this study was quite unexpected in the light of previous thinking on thermal stresses in treated layers. Previously thermal stresses were considered to be very important but this study has shown it to be true only for uncracked treated layers. Once the cracks have developed the thermal stresses become virtually negligible in comparison with the traffic-associated stresses. Thermal stresses may therefore be considered as unimportant in cracked treated layers."

In this dissertion the author shares the view of Otte and for this reason no further research had been done on any of the HVS sites, with respect to the thermal conditions of cementitious layers, or the influence thereof. It is however true that the HVS tests were done on weakly cemented materials covered with a thick insulating asphalt base layer. Relatively small variations in temperature was however measured within the cementitious layers (5° to 10°C). The effect of thermal stresses in weakly cemented subbase layers is therefore not considered in the postcracked phase because it is believed that the stresses and therefore movements owing to thermal conditions will be lower than those found in strongly cemented layers.

Mechanistic analyses were used to quantify the precracked and postcracked state of cementitious layers. The postcracked state is defined as the period within which cracks develop by trafficking only i.e. traffic associated cracking. For the purpose of illustration the HVS test at Mariannhill in Natal is discussed. (See Chapter 3). As previously described, stresses and strains were calculated using linear elastic simulations of the structure at various stages of trafficking and pavement behavioural states. The input parameters to these analyses were the multi depth deflections, measured with the MDD. Using this method of evaluation the changes in effective elastic moduli with accumulated traffic of the different layers in the structure are obtained, including those of the cementitious layer. The need to obtain such a relationship was already expressed by Otte (1978). The cementitious subbase materials tested with the HVS were mainly of C3 and C4 quality and in this case appropriate relationships were found for the weakly cemented subbase layers under consideration viz : C3 and C4 materials. The definition of cementitious materials according to TRH14 (1980) are given in Table 4.2.

The relationship between effective elastic moduli of the upper subbase and accumulated trafficking is illustrated in Figure 4.1. An hyperbolic relationship was found for the postcracked phase. The correlation coefficient was approximately 75 per cent. The results from the previous Table 3.4 were used. The relationships are:

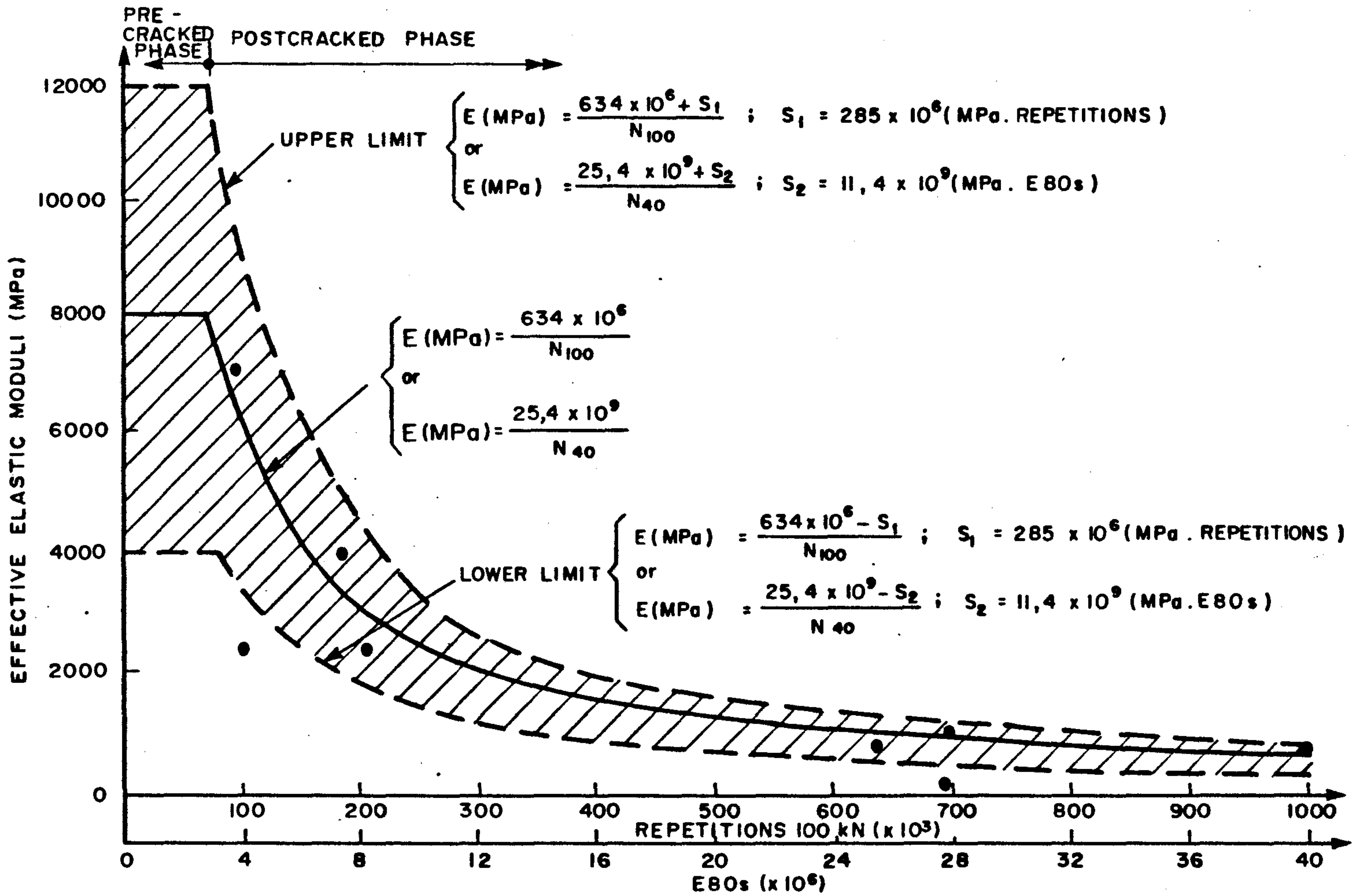

FIGURE 4.1

RELATIONSHIP BETWEEN EFFECTIVE ELASTIC MODULI OF THE CEMENTITIOUS SUBBASE LAYER AND ACCUMULATED TRAFFICKING (DRYSTATE)

TABLE 4.2 - Definition of cementitious materials (NITRR, 1980)

Code	Material	Specifications*					
		UCS at 100% Mod.AASHTO (MPa)	UCS at 95% Mod.AASHTO (MPa)	Aggregate Crushing Strength	Flakiness Index (%)	Sand Equiv. (%)	PI (%)
C1	Cemented crushed stone or gravel	6 - 12	4 - 8	10% FACT: $\geq$ 110kN ACV: $\leq$ 29%	$\leq$ 35	$\geq$ 30	-
C2	Cemented crushed stone or gravel	3 - 6	2 - 4	"	"	"	-
C3	Cemented natural gravel (Max size 63 mm)	1,5 - 3	1 - 2	-	-	-	$\leq$ 6
C4	Cemented natural gravel (Max size 63 mm)	0,75 - 1,5	0,5 - 1,0	-	-	-	$\leq$ 6
C5	Treated natural material	Modified only for Atterberg limits					

* UCS: Method A14, TMH1 (1979)

$$E = 634{,}44 \times 10^6 \pm S_1/N_{100} \quad \ldots \quad (4.1)$$

or

$$E = 25{,}4 \times 10^9 \pm S_2/N_{40} \quad \ldots \quad (4.2)$$

where E = Effective elastic modulus of upper subbase in MPa

S_1 = 285×10^6 (MPa. repetitions), standard deviation

S_2 = $11{,}4 \times 10^9$ (MPa. E80s), standard deviation

N_{100} = Repetitions of 100 kN dual wheel load

N_{40} = Repetitions of 40 kN dual wheel load, (E80s)

To obtain the upper and lower limits of the moduli, the given standard deviations (S_1 or S_2) is added or subtracted as is indicated in both equations 4.1 and 4.2. These limits are also shown in Figure 4.1. The average value of the moduli is obtained without the standard deviations, i.e.

$$E = 634{,}44 \times 10^6/N_{100} \quad \ldots\ldots (4.3)$$

or

$$E = 25{,}4 \times 10^9/N_{40} \quad \ldots\ldots (4.4)$$

The constants in the two equations differs by a multiplying factor of approximately 40, which was obtained assuming a relative damage exponent of 4 for this type of pavement structure in the relative damage expression, $(P/40)^d$. The figure indicates that a constant modulus of 8 000 MPa is proposed during the first 3 x 10^6 E80s, which is indicative of the precracked state of this layer. This modulus actually varied between 4 000 MPa and 12 000 MPa. It is difficult to measure these moduli in situ with the MDD instrument as very small relative deflections, if any, existed in cementitious layers during this phase. It is believed that some cracking, either traffic or non-traffic associated, in the cementitious layer does exist during the precracked phase, but is rather insignificant compared to that found during the postcracked phase. It is however true that anisotropy in the modulus or deflections of the cementitious layers cannot be measured or obtained by the current method using the MDD instruments. This reality was also appreciated by Otte (1978). The current method of using vertical MDDs in the pavement structure allows only the effect of oblique or horizontal cracking or granulation of the material around the MDD hole to be measured. It is however accepted that this is rather conservative, but it is the best available method of measuring in situ changes in the layer. Any major changes however within the layer can be measured, permitting the changes in effective elastic moduli to be calculated. In this case the moduli of the cementitious upper subbase changed from an average value of 8 000 MPa to approximately 500 MPa. Cracked and almost granulated material was recovered after the HVS test. Prior to testing intact material was recovered. As was discussed in Chapter 3, changes in the moduli of the other layers in the structure were also observed and calculated. It was therefore possible to define the different stages in the "life" of the road structure using linear elastic simulations. Because the main structural layers in this type of pavement structure (asphalt base, cementitious subbases), were treated materials, the effective horizontal strain was used as distress criterion to evaluate these layers. See also the theoretical procedure outlined in Table 4.1.

To quantify the precracked and postcracked states of the cementitious layer, traffic induced strains obtained from the linear elastic simulations at various stages of trafficking were plotted together with the strain criteria proposed by Otte (1978) and Freeme et al (1984). The comparison between the induced strain during trafficking and the fatigue criterion are illustrated in Figure 4.2. The area under the fatigue curve is defined as the precracked phase or state of cementitious layers. The remaining area (above the curve) is defined as the postcracked phase or state. In this case almost 3×10^6 E80s were applied on the upper subbase layer before major changes occurred within the layer. This effect can best be seen by comparing the slopes of the induced strain-traffic curve during the precracked and postcracked phases in Figure 4.2. On this structure almost 30×10^6 E80s were applied without serious deformation or major decrease in riding quality. The precracked phase encompasses therefore only approximately 10 per cent of the total amount of traffic applied to the structure during the dry state. From this finding the majority of the life of this pavement structure will therefore be in the postcracked phase. The concept of designing against traffic associated cracking in cementitious layers, can result in overdesigning the structure whereby uneconomical thickness of cementitious layer could be obtained. This is in agreement with finding of Otte (1978) where he stated the following:

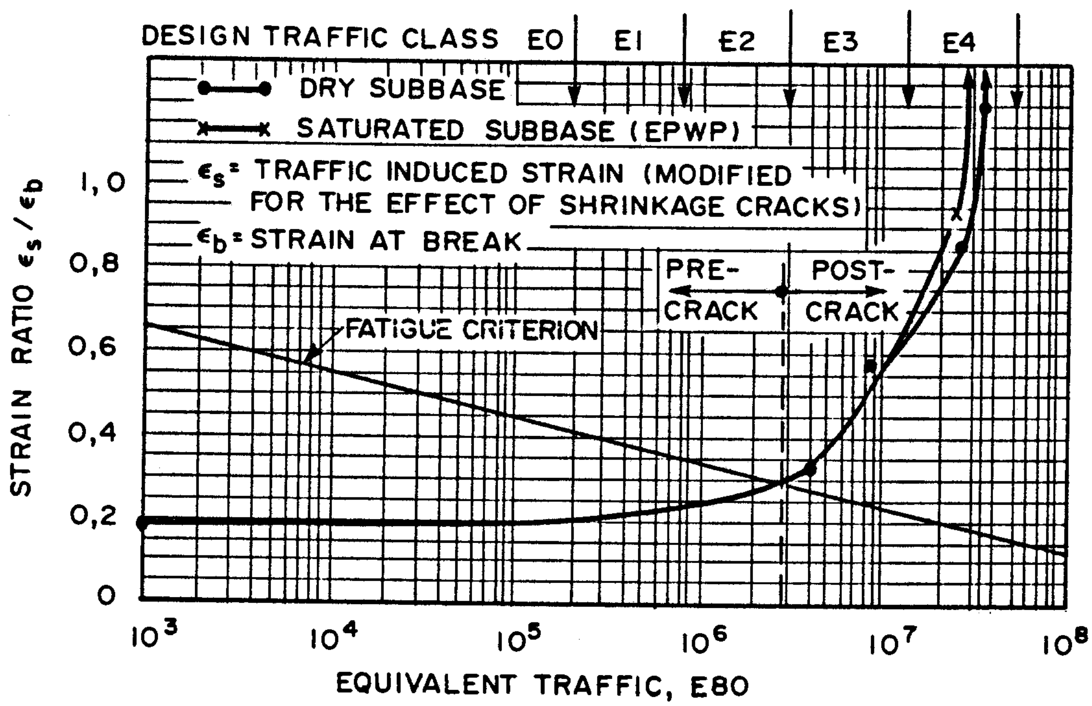

FIGURE 4.2

COMPARISON BETWEEN THE INDUCED STRAIN DURING TRAFFICKING AND THE FATIGUE CRITERION FOR CEMENTITIOUS LAYERS

"A second alternative design approach is to accept the fatigue life concept which implies that cement-treated materials can carry a limited number of load repetitions before failing due to fatigue. In this approach the treated material is considered to undergo traffic-associated cracking once the fatigue capacity of the material has been consumed by the load applications (equations 2.3 to 2.6 in section 2.2.10). During the structural design process it should therefore be endeavoured to ensure that the treated material's fatigue capacity will not be exceeded and that traffic-associated cracking will not occur before the pavement has carried the design traffic. The third alternative design approach, and probably the more practical and even less conservative one, it to accept traffic-associated cracking in the treated layer before the pavement has carried the full design traffic. The period of remaining life after traffic-associated cracking has occurred in the cement-treated layer, will be called the postcracked phase in this thesis. This is a very important phase in the overall design life of pavements with cement-treated layers and, depending on the material properties and traffic conditions, it can vary between 20 to 80 per cent of the total design life - it should therefore not be ignored! To properly analyse and include the contribution of this postcracked phase in the design life is very complex. It is currently only possible to make tentative suggestions on (i) how the postcracked phase can be included during the design, and (ii) the research work still to be done to include it with any degree of confidence."

When it is, however, considered to include the postcracked phase in the design, it is important to ensure that the pavement will not deform excessively with a consequent loss in riding quality during this phase. It is possible that deformation and a loss of riding quality may occur because of the reduction in the elastic modulus of the cementitious layer, especially during the excess porewater pressure state (EPWP), or owing to lack of durability (erosion resistance) of these layers, as was observed with the HVS test at Umgababa on N2/24, see Chapter 3. It is further possible that the vertical compressive strain within the subgrade layers could increase resulting also in deformation of the surface and subsequent loss in riding quality, especially where only one cementitious layer, 150 mm thick, exists in the structure.

It is therefore vitally important that all the layers in the pavement should be able to carry successfully the remainder of the design traffic after the onset of traffic-associated cracking in the treated layer(s).

4.3 EFFECT OF SUBBASE ON THE OTHER LAYERS IN THE STRUCTURE

In order to study the effect of the change in state of the cementitious upper subbase layer on the other layers in the structure, the induced strains on the asphalt and subgrade layers were compared with their appropriate failure criteria. The maximum horizontal strain at the bottom of the asphalt layer was compared with the fatigue criteria for thick asphalt bases (Freeme, et al, 1984) and the vertical compressive strain on top of the selected layer is compared with the limit subgrade deformation criteria. These comparisons are illustrated in Figures 4.3 and 4.4, respectively. Both figures indicate similar gradual changes to those found with the cementitious layer (Figure 4.2). The change of the slopes of these strain-traffic curves however is much smaller than those found with the cementitious layer. The effect of excess porewater pressure within the upper subbase is more accentuated with the asphalt and subgrade layers. It is thus clear that the ingress of water in the cracked cementitious subbase layer must be avoided as this will lead to excessive deformation, cracking and subsequent loss in riding quality of the pavement (failure).

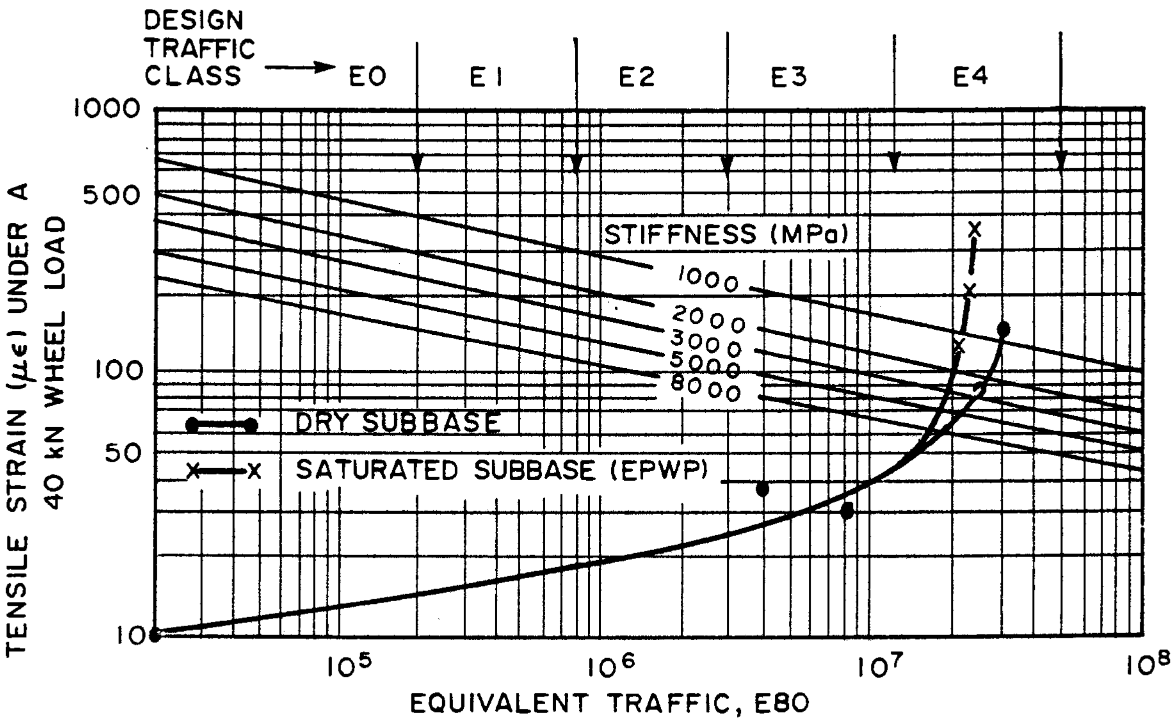

FIGURE 4.3

COMPARISON BETWEEN THE MAXIMUM INDUCED TENSILE STRAIN DURING TRAFFICKING AND THE FATIGUE CRITERIA FOR THICK ASPHALT BASES

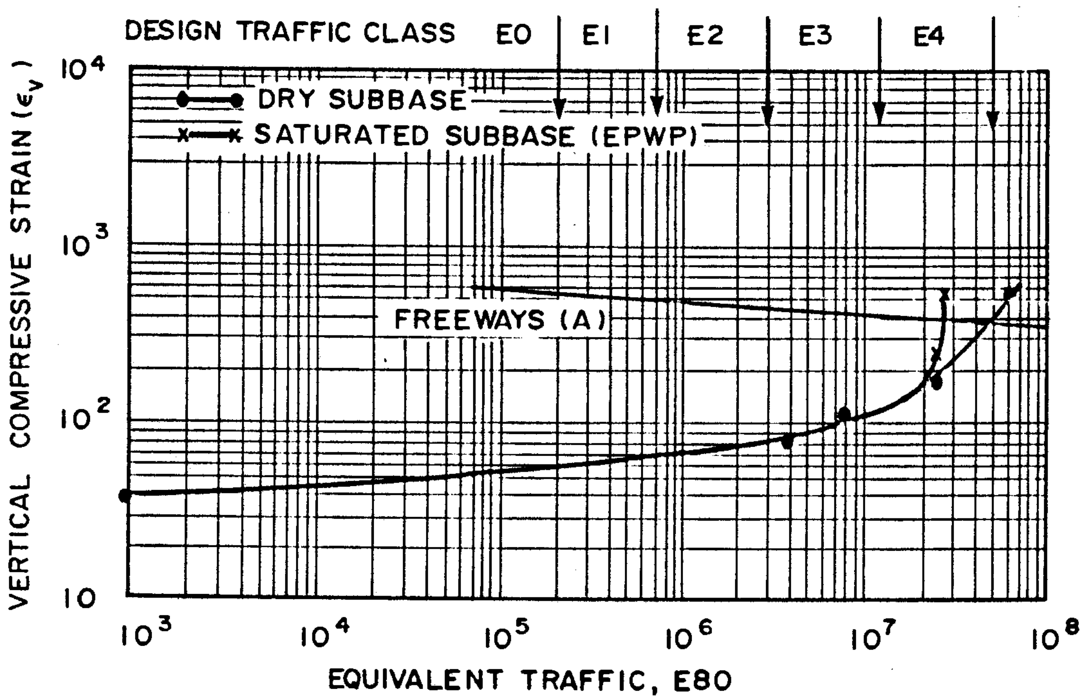

FIGURE 4.4

COMPARISON BETWEEN THE INDUCED VERTICAL COMPRESSIVE STRAIN DURING TRAFFICKING AND THE VERTICAL STRAIN CRITERION TO LIMIT SUBGRADE DEFORMATION OF FREEWAYS (A-CATEGORY)

In the dry state the figures indicate that approximately 30 x 10^6 E80s were applied to the structure before the critical strain limits were reached in the asphalt. In the subgrade layer almost 50 x 10^6 E80s could be accommodated before the critical limit is reached. This is an indication of the degree of protection which the subgrade experiences in this type of design, incorporating weakly cemented subbase layers.

4.4 EFFECT OF WATER ON THE MODULUS OF THE UPPER SUBBASE LAYER

Almost every HVS test done in the wet state, i.e. when water is introduced on or within the road structure being tested, or after the ingress of rain water through surface cracks, moisture accelerated distress (MAD) occurred. The ultimate "life" of the road structure depends heavily upon the rate of progress of this distress and the definition of failure. Because it is not possible at this stage to measure the change in relevant riding quality during a the HVS test, it was decided to define failure when the average deformation measured on the surface of the test section surface equals 20 mm. The importance of decrease in riding quality is appreciated and therefore the difference in functional failure and structural distress of the road must be fully understood. Structural distress in a road leads to functional failure. Structural distress in this dissertion is defined as any major change within the structural layers of the pavement, i.e. cracking, high degree of erodibility (De Beer, 1985) excessive deformation etc. Functional failure occurs when the state of the surface of the pavement in terms of riding quality, changes to unacceptable levels, irrespective of the cause thereof. Most of the HVS tests proved that structural distress occurs rapidly when a state of excess porewater pressure develops within or between the structural layers of the pavement. When the postcracked state of the cementitious layer is considered, fracturing and even granulation of this layer occurred, which was aggravated by the presence of free water in the voids or on the layer. This degradation can quantitatively be incorporated during the design or evaluation phase of these types of pavement. As with the change in effective elastic moduli of the cementitious layer in the dry state, the change in moduli during the excess porewater pressure state was also measured during HVS testing. These tests indicated that the modulus of the cementitious layer degraded (reduced) at a rate of approximately 360 MPa per 10^6 E80s. This rate of degradation is calculated from the moduli values given in Chapter 3, Table 3.4. The average modulus in the "wet" state, i.e. at MDD8 and MDD12, of approximately 400 MPa was subtracted from the 800 MPa at the end of the "dry" state on Section 1. The rate of degradation is then calculated by dividing this change in modulus by the equivalent applied traffic of approximately 1,1 ME80s. Using this information the "shortest possible life"* of the cementitious layer is calculated. It is however necessary to obtain the earliest time when a state of excess porewater pressure will markedly effect the cementitious layers. It is believed that if non-erodible upper subbase material is used that relatively slow changes in the structural capacity or support value of the cementitious layers will occur during the precracked phase. The effect will become serious only when the rate of degradation in subbase modulus during the excess porewater pressure state exceeds the rate of degradation during the dry state.

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* The "shortest possible life" of the cementitious layer is defined as the sum of the number of repetitions during the precracked phase and the number of repetitions during the excess porewater pressure state needed in the postcracked phase to reduce the modulus of the layer to zero.

Quantitatively this canging point can be calculated as follows:

from eq 4.4: $E = 25{,}4 \times 10^9/N$ .... (4.5)

where N = E80 repetitions

$$\therefore \frac{d(E)}{d(N)} = -25{,}4 \times 10^9/(N)^2$$

But $\frac{d(E)}{d(N)} = -360$ MPa per 10^6 E80s; (measured rate of degradation in wet state).

$$\therefore -25{,}4 \times 10^9/(N)^2 = -360 \times 10^{-6}$$

$$N = \sqrt{25{,}4 \times 10^9/360 \times 10^{-6}}$$

$$= 8{,}4 \times 10^6 \text{ E80s}$$

Above calculation indicates that when non-erodible materials are used for the cementitious subbase layers, this type of structure can withstand almost E3 class of traffic (i.e. 3 to 12 x 10^6 E80s) irrespective of the presence of water within the pavement. Experience however has taught that the less free water available in the pavement the better. The author believes that although it is possible to calculate the "shortest possible life" of the cementitious layer, that this is over conservative to use in the design phase. Real wet and dry periods should be incorporated, similar to those proposed by Grant, et al (1984), during the total life of the pavement. It is further recommended that the upper limit degradation curve in Figure 4.1 should be used for relatively stronger cemented (C2) materials and the lower limit for weaker (C4) materials. The average degradation curve is recommended for C3 materials. The rate of degradation during the excess porewater pressure state within these materials can be taken as 360 MPa per 10^6 E80s for C2, C3 and C4 materials at this stage, although it is rather conservative for C2 materials.

As previously suggested wet and dry periods during the structural design period of the pavement must be incorporated in the analyses. For example, in this case, if 3 months per year were taken as the "wet" period, i.e. when a state of excess porewater pressure exists within the pavement, the rate of degradation in cementitious subbase moduli is 360 MPa per 10^6 E80s. During the "dry" periods the hyperbolic relationship (eqs. 4.1 or 4.2) must be used to calculate the effective elastic moduli of the cementitious subbase layers. The following example illustrates this principle:

Example : This example illustrates the calculation of the most probable effective elastic moduli values for C2, C3 and C4 materials, both in the "wet" and "dry" states for the cementitious upper subbase layer, incorporating a 3 monthly "wet" period per year:

The following information is given to perform the calculation:

(i) Compound growth rate in E80s, i = 10%

(ii) Cumulative growth factor, fy =

$$fy = 365 \left(\frac{(1+0,01i) \left| (1+0,01)^Y - 1 \right|}{(0,01i)} \right)$$

(iii) Analysis period, y. In this example the analysis will be done for each year from year 5 onwards including a 3 monthly wet period per year up to 20 years. Detailed calculations at years 10 and 15 in the "dry" state, and at years 14 and 15 in the "wet" state are given.

(iv) Initial traffic, Ni = 1053 E80s per day per lane

(v) Design distribution factor, Be = 0,95 (NITRR, (1985))

(vi) Non-erodible cementitious subbase material to be used in this design.

(vii) Lower subbase of C4 quality.

Solution:

Analysis for dry state only:

From eq 4.2 the following relationships holds for the moduli values of the three different materials:

(i) $EC2 = 36{,}8 \times 10^{9}/E80s$ .... (4.6)

(ii) $EC3 = 25{,}4 \times 10^{9}/E80s$ .... (4.7)

(iii) $EC4 = 14 \times 10^{9}/E80$ .... (4.8)

The cumulative equivalent traffic, Ne, can be calculated from:

$Ne = (Ni x Be) x fy$ .... (4.9)

$= (1053 x 0{,}95) fy$

$\therefore Ne = 1\ 000\ fy$

for example : y =10 years

$\therefore fy = 6399$

$\therefore Ne = 6{,}399 \times 10^{6}$ E80s

The different "dry moduli" values after 10 years can now be calculated by substituting Ne for E80s in each eq. 4.6, 4.7 and 4.8 above:

For example : $EC2 = 36{,}8 \times 10^{9}/6{,}399 \times 10^{6} = 5\ 751$ MPa

$EC3 = 25{,}4 \times 10^{9}/6{,}399 \times 10^{6} = 3\ 969$ MPa

and $EC4 = 14 \times 10^{9}/6{,}399 \times 10^{6} = 2\ 188$ MPa

Similarly:

At 15 years the three "dry moduli" values are:

EC2 = 2 885 MPa

EC3 = 1 991 MPa

EC4 = 1 097 MPa

Analyses including "dry" and "wet" alternative states:

To incorporate the 3 monthly wet periods, it is therefore necessary to analyse each year including these wet periods. It was decided to include this period at the start of each year. The cumulative equivalent traffic, associated with each analysis period, were also calculated using eq. 4.9. During the wet periods only (i.e. each 0,25 year of every year), the degradation rate of 360 MPa per 10^6 E80s was used to calculate the reduction in the moduli values.

During the alternative dry periods the previous "dry" degradation rates (hyperbolic relationships) were used. The principle of this calculation method is illustrated in Figure 4.5.

Following the figure, the "wet" moduli values can be calculated as follows:

"Dry" moduli values are : Ea, Eb, Ed and Ef. (See Figure 4.5)
"Wet" moduli values are : Ec, Ee and Eg.
$t_2 - t_1 = 0{,}25$ years $= t_4 - t_3$ etc. (yearly wet periods)
$t_3 - t_1 = 1$ year $= t_5 - t_3$ etc. (dry and wet periods)
$t_3 - t_2 = 0{,}75$ year $= t_5 - t_4$ etc. (yearly dry periods)

"Dry" rate of degradation: $E_{dry} = k_n / E80s$ with n = 1, 2 and 3

$$\left.\begin{array}{ll} \text{where } k_1 = 36{,}8 \times 10^9 & \text{(for C2 material)} \\ k_2 = 25{,}4 \times 10^9 & \text{(for C3 material)} \\ k_3 = 14 \times 10^9 & \text{(for C4 material)} \end{array}\right\} \text{.... from eq. 4.2}$$

Therefore $Ea = k_n / E80$ at t_1; $Eb = k_n / E80$ at t_2 etc. for the three types of cementitious materials in the "dry" state.

For "wet moduli" values :
$Ec = Ea - (E80 \text{ at } t_2 - E80 \text{ at } t_1)(360 \times 10^{-6})$ MPa ...(E80 in millions)

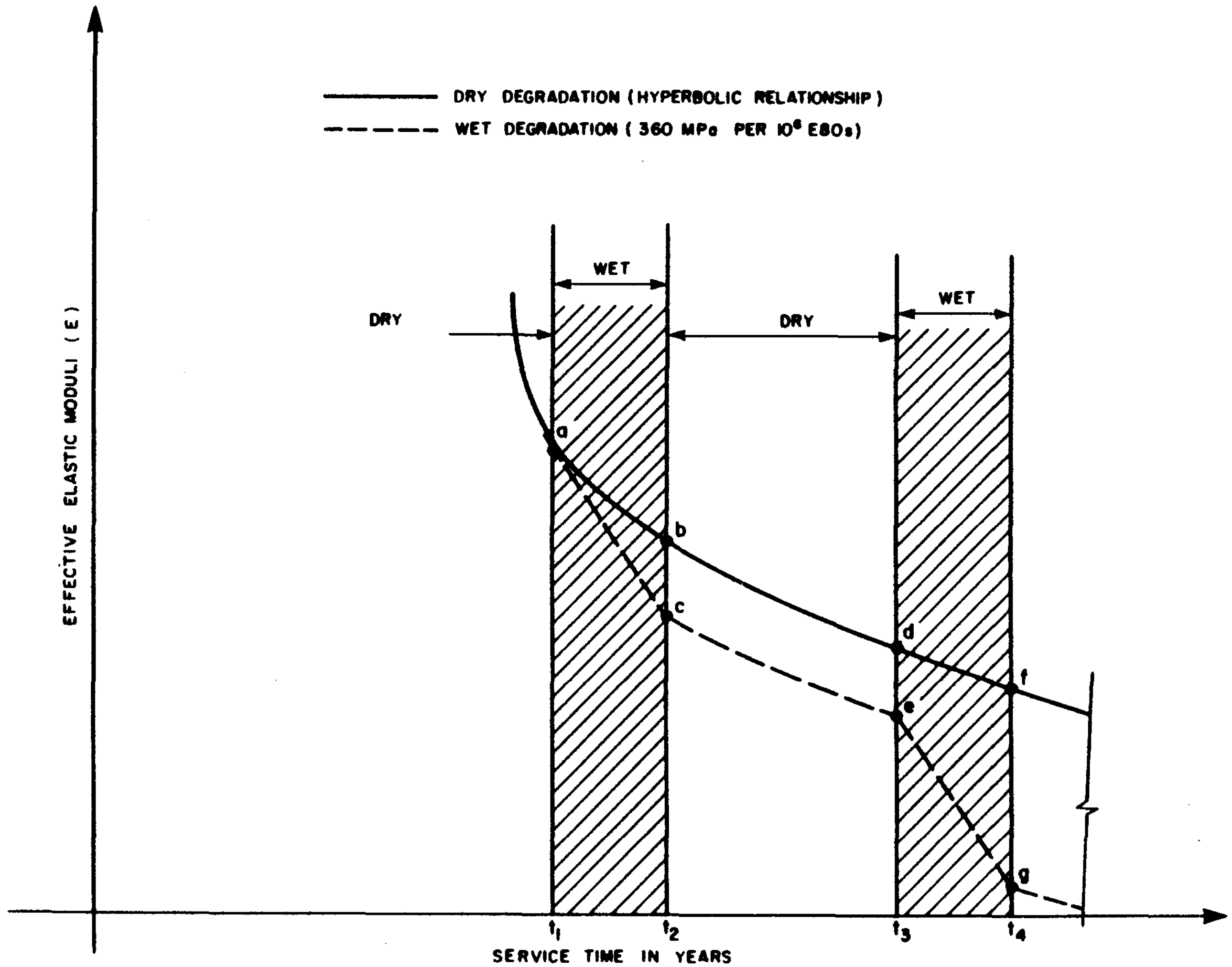

FIGURE 4.5

PRINCIPLE OF CALCULATING WET MODULI VALUES and Ee = Ed - (Eb - Ec)

= Ed - Eb + Ec

and Eg = Ee - (E80 at t_4 - E80 at t_3)(360 x 10^{-6}) MPa etc.

For example : (i) C2 - Material at year 14:

Edry = Ewet = 36,8 x 10^9/11,232 x 10^6 = 3 276 MPa (No influence of water yet)

at year 14,25: Edry = 36,8 x 10^9/11,60 x 10^6 = 3 172 MPa

and Ewet = (Edry at year 14) - (11,600 x 10^6 - 11,232 x 10^6)(360x10^{-6}) MPa

= (3 276) - (132)

= 3 144 MPa

At year 15 :

Ewet = (E dry at year 15) - (E dry - E wet) at year 14,25

= 2 885 - (3 172 - 3 144)

= 2 857 MPa etc.

Calculation of the accumulative traffic (E80s) at the first stage when "dry" moduli will be affected by water for the different types of cementitious layers, (C2, C3 and C4):

The rate of degradation in moduli during the wet periods is 360 MPa per 10^6E80s. Because the "dry" relationships is hyperbolic, the time when water will first effect the moduli of the cementitious layer can be determined by first order differentiation of the moduli relationships with respect to the number of repetitions, N (E80s):

Thus for C2 : $EC2 = 36{,}8 \times 10^9/N$

$$\therefore \quad \frac{d(EC2)}{d(N)} = -36{,}8 \times 10^9/(N)^2$$

but $$\frac{d(EC2)}{d(N)} = -360 \times 10^6$$ (Rate of degradation in wet state)

$$N = \sqrt{\frac{36{,}8 \times 10^9}{360 \times 10^{-6}}}$$

i.e. for C2: $N|_{C2} = 10{,}11 \times 10^6$ E80s

For C3 : $N|_{C3} = 8{,}40 \times 10^6$ E80s
and C4 : $N|_{C4} = 6{,}24 \times 10^6$ E80s

From above calculation it is clear that the "weaker" the subbase material, the earlier it will be effected during the excess porewater pressure state. In other words the moduli of the three materials, C2, C3 and C4 will be reduced because of water only after $10{,}11 \times 10^6$, $8{,}4 \times 10^6$ and $6{,}2 \times 10^6$ E80s, respectively. These moduli values will be lower than those calculated for the equivalent dry state. The calculated moduli values incorporating the 3 monthly wet period per year for 20 years are given in Table 4.3.

The values from the table are graphically illustrated in Figure 4.6. If a "failure condition" is defined using a "minimum allowable effective elastic modulus" of the cementitious layer during the design life of the structure, the ultimate lives of the three types of subbases, can be obtained from Table 4.3 or Figure 4.6. These values can also be calculated. A minimum effective elastic modulus of 350 MPa is recommended for all three types cementitious subbase layers. According to this failure criterion, the ultimate lives of the three types of cementitious materials are approximately 20 to 25 years for the C2 material; 20 years for the C3 material and 17 years for the C4 material. See Table 4.3 also. It is however important to note that these lives depends heavily on the initial traffic, the real compound growth rate and the actual wet periods etc.

TABLE 4.3 Effective elastic moduli values for the three types of cementitious materials, incorporating in the dry and wet periods.*

Prediction Period ,y (yrs)	fy	Accum. Traffic, (E80s) ($x10^6$)	Effective elastic moduli (MPa) EC2 Dry	Wet	EC3 Dry	Wet	EC4 Dry	Wet
0	-	-	12 000	12 000	8 000	8 000	4 000	4 000
5	2 451	2,451	12 000	12 000	8 000	8 000	4 000	4 000
6	3 098	3,098	11 879	11 879	8 000	8 000	4 000	4 000
6,25	3 269	3,269	11 257	11 257	7 770	7 770	4 000	4 000
7	3 809	3,809	9 661	9 661	6 668	6 668	3 676	3 676
7,25	3 998	3,998	9 205	9 205	6 353	6 353	3 502	3 502
8	4 592	4,592	8 014	8 014	5 531	5 531	3 049	3 049
8,25	4 799	4,799	7 668	7 668	5 293	5 293	2 917	2 917
9	5 452	5,452	6 750	6 750	4 659	4 659	2 568	2 568
9,25	5 680	5,680	6 479	6 479	4 472	4 472	2 465	2 465
10	6 399	6,399	5 751	5 751	3 969	3 969	2 188	2 188
10,25	6 650	6,650	5 534	5 534	3 820	3 820	2 105	2 098
11	7 440	7,440	4 946	4 946	3 414	3 414	1 882	1 875
11,25	7 716	7,716	4 769	4 769	3 292	3 292	1 814	1 776
12	8 586	8,586	4 286	4 286	2 958	2 958	1 631	1 593
12,25	8 890	8,890	4 139	4 139	2 857	2 849	1 575	1 484
13	9 846	9,846	3 738	3 738	2 580	2 572	1 422	1 331
13,25	10 180	10,180	3 615	3 615	2 495	2 452	1 375	1 211
14	11 232	11,232	3 276	3 276	2 261	2 218	1 246	1 082
14,25	11 600	11,600	3 172	3 144	2 190	2 086	1 207	950
15	12 757	12,757	2 885	2 857	1 991	1 887	1 097	840
15,25	13 161	13,161	2 796	2 712	1 930	1 742	1 064	695
16	14 434	14,434	2 550	2 466	1 760	1 572	970	601
16,25	14 879	14,879	2 473	2 306	1 707	1 412	941	441
17	16 279	16,279	2 261	2 094	1 560	1 265	860	360
17,25	16 768	16,768	2 195	1 918	1 515	1 089	835	184
18	18 308	18,308	1 733	2 010	1 387	961	765	114
18,25	18 846	18,846	1 953	1 539	1 348	767	743	0
19	20 540	20,540	1 792	1 378	1 237	656	682	0
19,25	21 132	21,132	1 741	1 105	1 202	443	663	0
20	22 996	22,996	1 600	1 024	1 105	346	609	0

* Dry Period : 0,75 year, each year
Wet period : 0,25 year, beginning of each year.

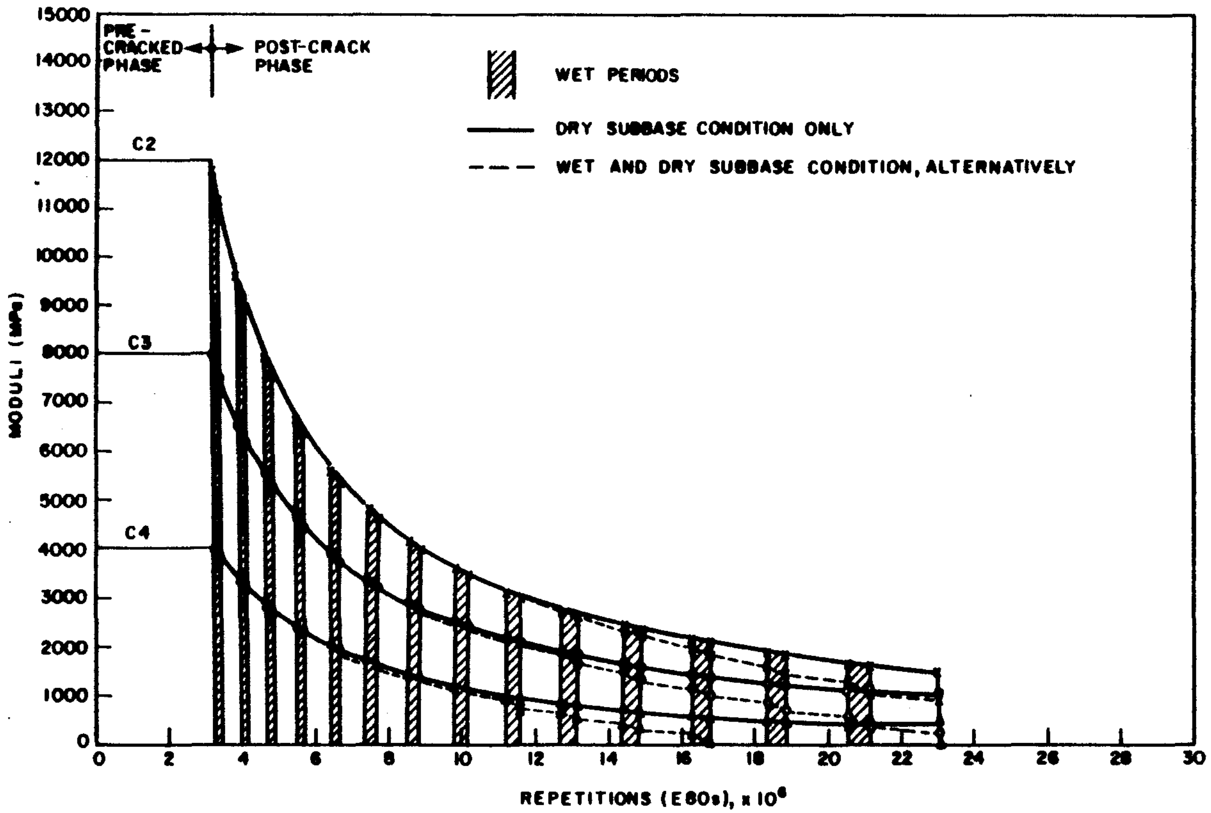

FIGURE 4.6

CHANGE IN EFFECTIVE ELASTIC MODULI OF CEMENTITIOUS SUBBASE LAYERS DURING TRAFFICKING INCLUDING WET AND DRY PERIODS

4.5 CONCLUSIONS AND RECOMMENDATIONS

In this chapter some of the results obtained from HVS testing were evaluated using theoretical procedures, i.e.: linear elastic simulation, in this case. It was shown that the different behavioural states of cementitious layers (pre-cracked and postcracked) in the road structure can be distinguished and quantified using the induced tensile strain at the bottom of these layers in comparison with the fatigue crack criterion proposed by Otte (1978). Comparisons were also made between the failure criteria and induced strain values of the asphaltic and subgrade layers, respectively.

It is concluded that the degradation of the effective elastic modulus during the postcracked phase of the cementitious layer is hyperbolic during the dry state, and linear during the wet (excess porewater pressure) state. A detailed example illustrates the principle of calculating the effective elastic modulus values, incorporating specified wet periods over the structural design life of the structure.

The following relationships are recommended in evaluating or designing pavement structures consisting of a 100 mm thick bitumen base and two 150 mm thick cementitious subbase layers:

Degradation in dry state:

(i) For C2 materials : $EC2 = 36{,}8 \times 10^9/E80s$ .... (4.10)

(ii) For C3 materials : $EC3 = 25{,}4 \times 10^9/E80s$ .... (4.11)

(iii) For C4 materials : $EC4 = 14 \times 10^9/E80s$ .... (4.12)

Rate of degradation during wet state:

$$\frac{d(E)}{d(E80s)} = -360 \text{ MPa per } 10^6 \text{ E80s} \quad \text{.....................} \quad (4.13)$$

A lower limit of 350 MPa is recommended as "failure" of a cementitious layer during the postcracked phase. It can further be concluded that:

(i) The "life" of a cementitious subbase layer is strongly dependent on

- (a) Real accumulated traffic
- (b) Material cementitious quality (C2, C3 or C4)
- (c) Environmental conditions - mainly excessive moisture on or within the cementitious layer during the postcracked phase, (wet and dry periods).
- (d) Erodibility of the cementitious material.

Last but not least, the traffic associated degradation (i.e. gradual break down of the cementitious layer) must be accepted and incorporated in the design phase. This includes possible rehabilitation design. It was shown that less than 10 per cent of the "life" of these layers are in the precracked phase (i.e. before any traffic induced fatigue distress). In this chapter a quantitative method to describe and define effective modulus elastic values for cementitious subbase layers are proposed.

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C H A P T E R 5

THE EFFECT OF INTERLAYERS WITHIN BITUMEN BASE STRUCTURES

CONTENTS

5.1. INTRODUCTION

It is often found with the in situ road structure that it differs from the initial design. One of the most important differences is that relatively soft interlayers (lenses) exists between the structural layers of the pavement. These interlayers often causes failures owing to the lack of supporting value especially in the wet state when excess porewater pressure (EPWP) develops. (De Beer, 1984(a); De Beer, 1984(b); Opperman, 1984; De Beer, 1984(c))

In this case the effects of weak interlayers on the asphalt base pavement design (Freeme, et al, (1984 (a)) are studied. The critical strain parameters of the structural layers of the pavement will be investigated with the aid of linear elastic simulation, using the ELSYM computer program.

Failure potential for fatigue distress and permanent deformation are introduced which indicates which structures are most sensitive to distress, originating mainly from these interlayers. Moduli values for the interlayers are assumed to represent dry and wet conditions. These moduli values are 300 MPa and 50 MPa for the dry and wet conditions, respectively. The moduli for the base, subbases and subgrade layers being kept constant at 3 000, 8 000, 4 000 and 200 MPa, respectively. These moduli values were deliberately chosen to represent the initial state of the pavement structure after construction. A few identified reasons for the existence of these interlayers (which could easily be up to 50 mm or more in thickness) are :

(a) Poor construction;
(b) inadequate layer thickness of stabilised layers, whereby correction layers are introduced;
(c) over or faulty compaction (crushing of top 10-20 mm);
(c) Inadequate mixing depth;
(d) possible carbonation owing to wetting and drying of unprotected stabilised layers (some materials weakens because of carbonation), (Netterberg, 1984).

5.2. ANALYSES

In this analyses four different structures are evaluated. The structures are numbered from 1 to 4. Structure 1 consists of no interlayers representing the designed structure. The rest of the structures (2, 3 and 4) each contain one interlayer, but at a different position. Firstly the effect of the position of a 5 mm interlayer will be studied both in the wet and dry states. Secondly, the effect of the thickness of the interlayer at the bottom of the asphalt (similar to structure 2) will be studied also in the dry and wet states. In this report "dry" and "wet" will be differentiated by the magnitude of the modulus of the interlayer. A modulus of 300 MPa will reflect the "dry" condition and a modulus of 50 MPa will reflect a "wet" condition. The main structural layers however, retain their initial modulus values throughout.

5.2.1 Effect of position of the interlayer

For the purpose of this section of the thesis, the position of the interlayer will be either on top of the upper subbase, between the two subbases or at the bottom of the lower subbase. Because of a limitation in the computer program (ELSYM), a maximum of five layers only can be analysed, therefore only one interlayer per structure could be incorporated.

The effect of these interlayers is described in terms of the fatigue distress parameter, viz. maximum horizontal tensile strain (ε_t) at the bottom of the asphalt or cementitious layers, and in terms of the permanent deformation parameter, viz. vertical compressive strain (ε_v) at the top of the interlayers and the subgrade layer, including all layer interfaces. A dualwheel load of 40 kN at 690 kPa tyre pressure was used in this study. The position midway between the dualwheels was used as the point of analysis.

5.2.1.1 Tensile strain (ε_t) at the bottom of the treated layers

In Figure 5.1 the effect of the 5 mm dry and wet interlayer on the maximum horizontal tensile strain at the bottom of the asphalt base layer is illustrated. The figure indicates maximum strain levels with the interlayer between the asphalt and the upper stabilized subbase, both in the dry and wet conditions. The other structures indicate virtually no significant tensile strains. Above finding is important regarding the fatigue distress potential of the asphalt layer.

The effect of the interlayers on the maximum horizontal tensile strain at the bottom of the upper stabilized subbase layer is illustrated in Figure 5.2. In this case the maximum strains occurred when the interlayer is positioned between the two stabilized subbase layers. Similar strains resulted within the other structures.

The effect of the interlayers on the maximum horizontal tensile strain at the bottom of the lower stabilized subbase layer is illustrated in Figure 5.3. In this case the maximum strains occurred when the interlayer is positioned between the asphalt and the upper subbase. Minimum strains even in the wet state occurred at this position in Structure 3, where the interlayer are positioned between the two subbase layers.

5.2.1.2 Fatigue distress potential, (Fdp)

In order to differentiate between the total sensitivity of the different structures with regard to fatigue distress (for the treated layers only), it is necessary to define a new parameter, called the "Fatigue distress potential". The fatigue distress potential (Fdp) is defined as the sum of the maximum horizontal tensile strains at the bottom of all the treated layers in the structure, viz :

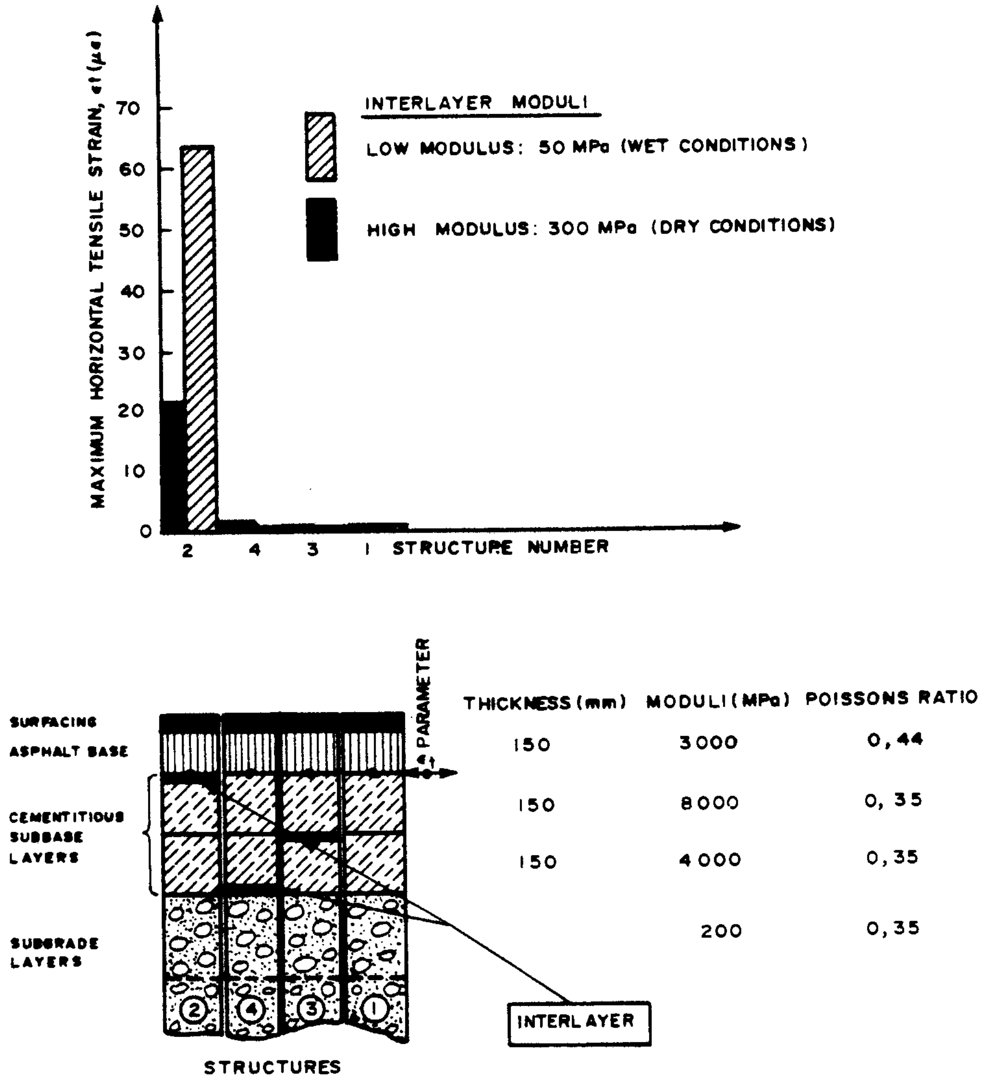


FIGURE 5.1

EFFECT OF 5 mm WET AND DRY INTERLAYER ON THE MAXIMUM HORIZONTAL TENSILE STRAIN AT THE BOTTOM OF THE ASPHALT BASE LAYER

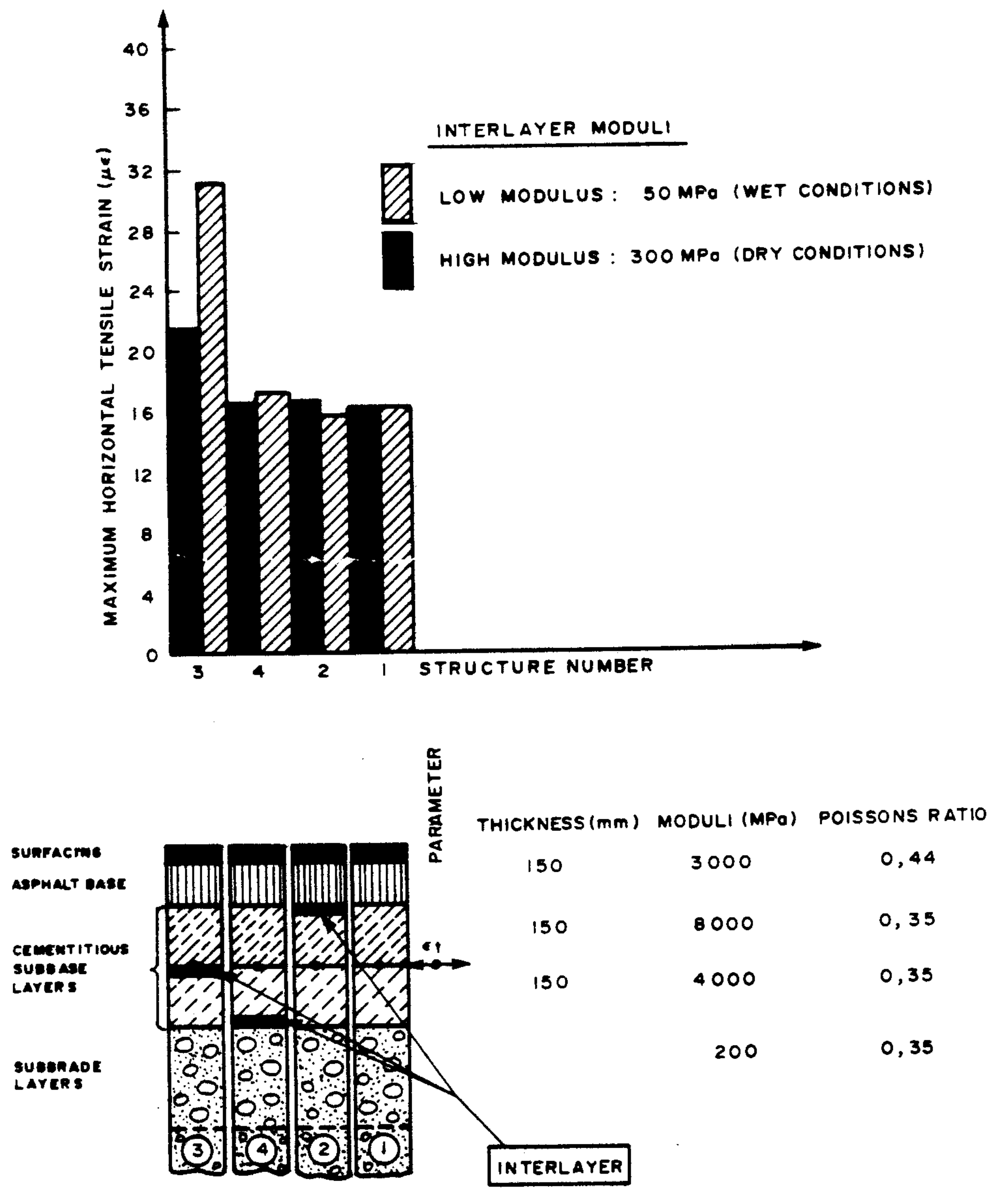
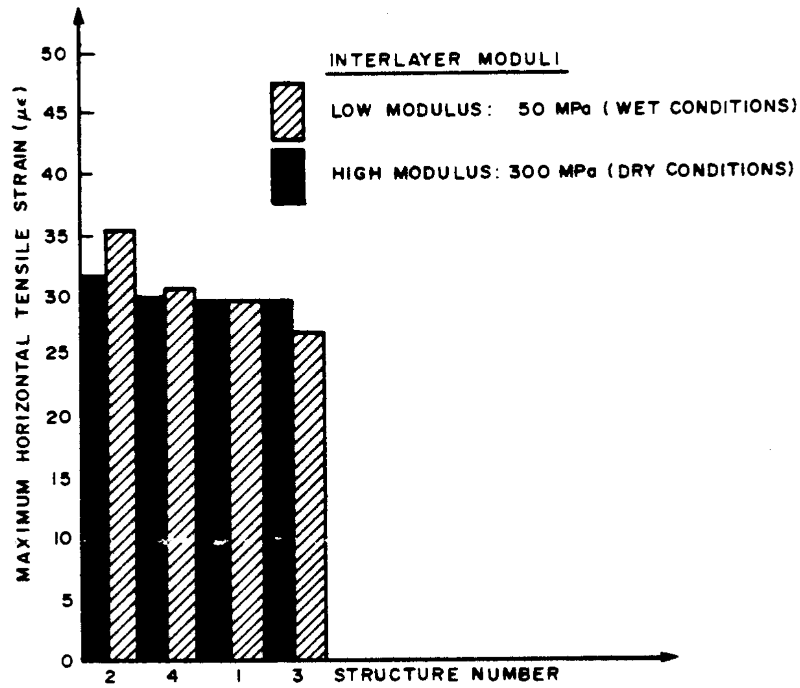


FIGURE 5.2

EFFECT OF 5 mm WET AND DRY INTERLAYER ON THE MAXIMUM HORIZONTAL TENSILE STRAIN AT THE BOTTOM OF THE UPPER CEMENTITIOUS SUBBASE LAYER



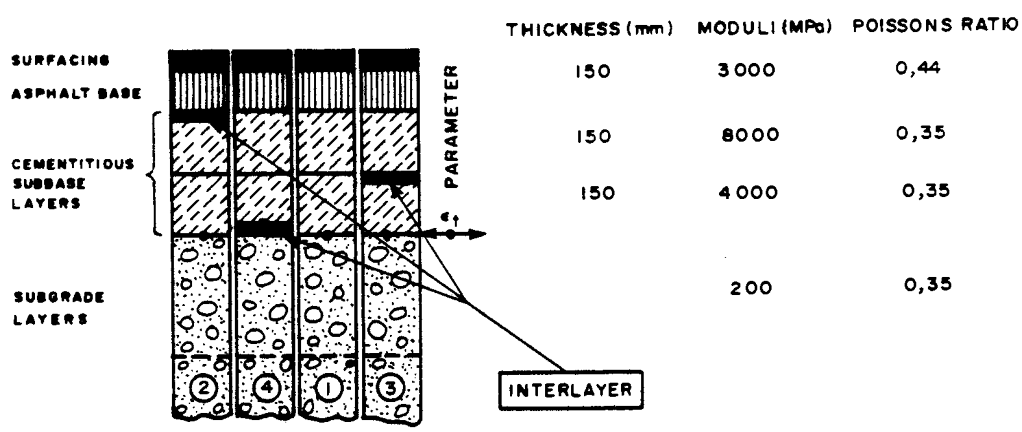


FIGURE 5.3

MAXIMUM HORIZONTAL TENSILE STRAIN AT THE BOTTOM OF THE LOWER CEMENTITIOUS SUBBASE LAYER

$$\text{Fatigue distress potential} = \text{FdP} = \sum_{i=o}^{n} \varepsilon_{t_i},$$

n = number of treated layers

The Fdp is measured in microstrain ($\mu\varepsilon$).

The different Fdp's for the four structures analysed are illustrated in Figure 5.4. The figure indicates that Structure 2 is most sensitive to fatigue distress both in the dry and wet conditions. Similar Fdp's were obtained for the three remaining structures. It can be concluded at this stage that an interlayer between the asphalt and upper subbase is relatively more detrimental than interlayers deeper in the structure.

5.2.1.3 Vertical compressive strain, (ε_v)

In order to study the potential for permanent deformation in these structures the vertical compressive strain parameter, ε_v, was used. The vertical compressive strains at the bottom of the asphalt layer (i.e. top of upper subbase - interlayers included) are illustrated in Figure 5.5. The figure indicates a remarkable increase in strain level of Structure 2 when the interlayer is wet. Insignificant strain levels were obtained for the other structures and even for the dry condition in Structure 2.

The vertical compressive strains at the bottom of the upper subbase are illustrated in Figure 5.6. Maximum strains were obtained on the interlayer of Structure 3. Similar strains were obtained for the remaining structures. But here again it is virtually only the wet condition which is significantly higher than the rest.

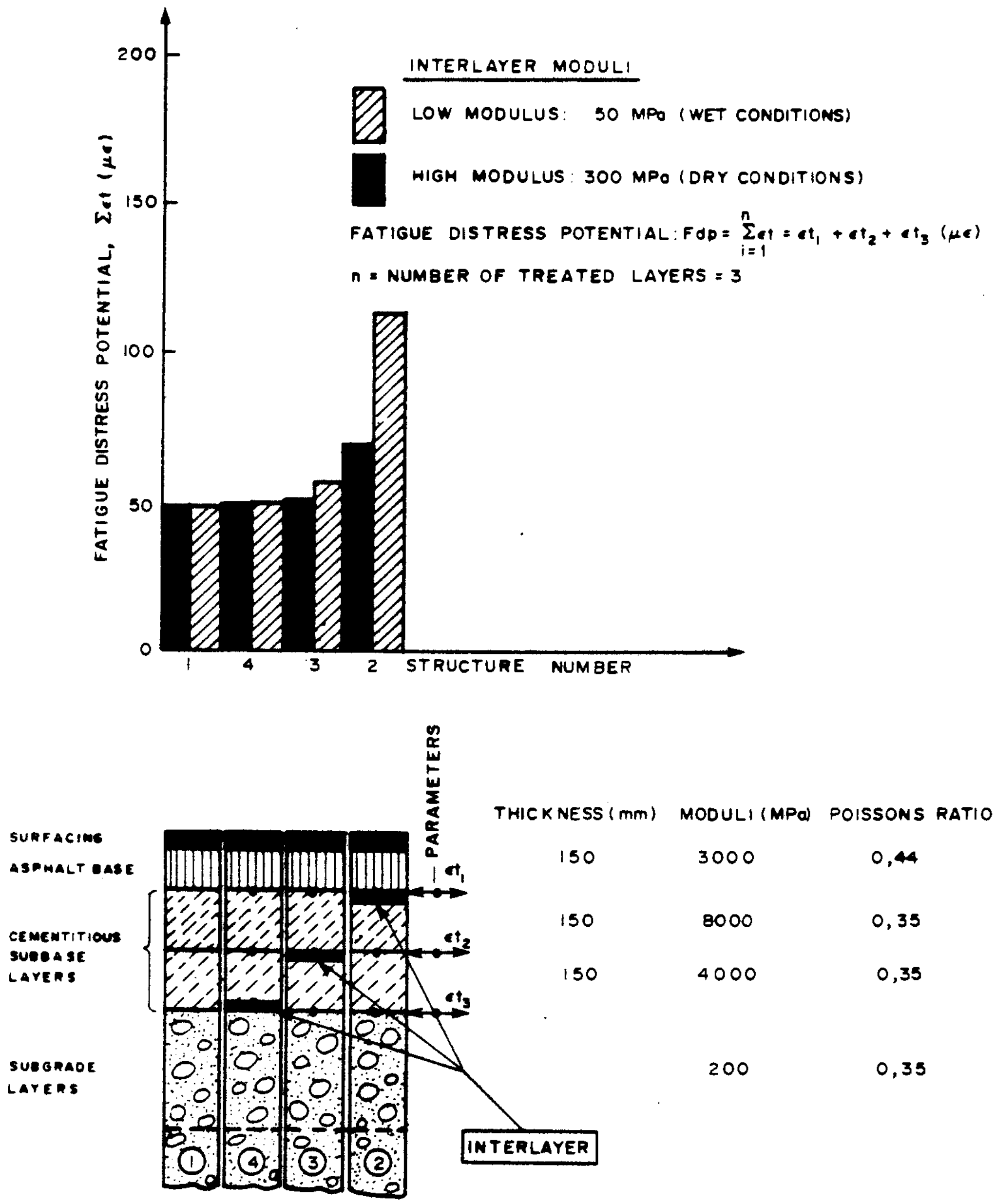
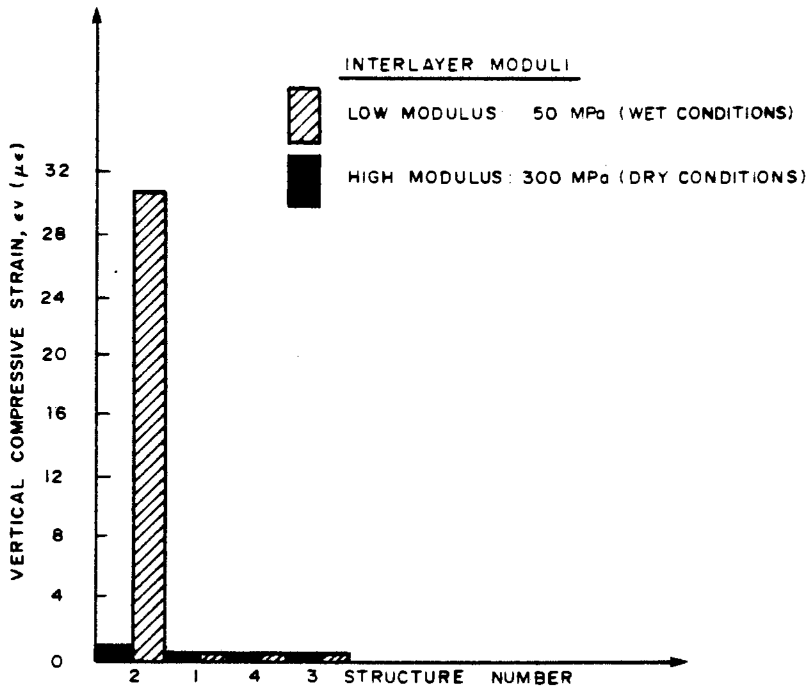


FIGURE 5.4

FAILURE DISTRESS POTENTIALS OF THE TREATED LAYERS IN THE 4 DIFFERENT STRUCTURES



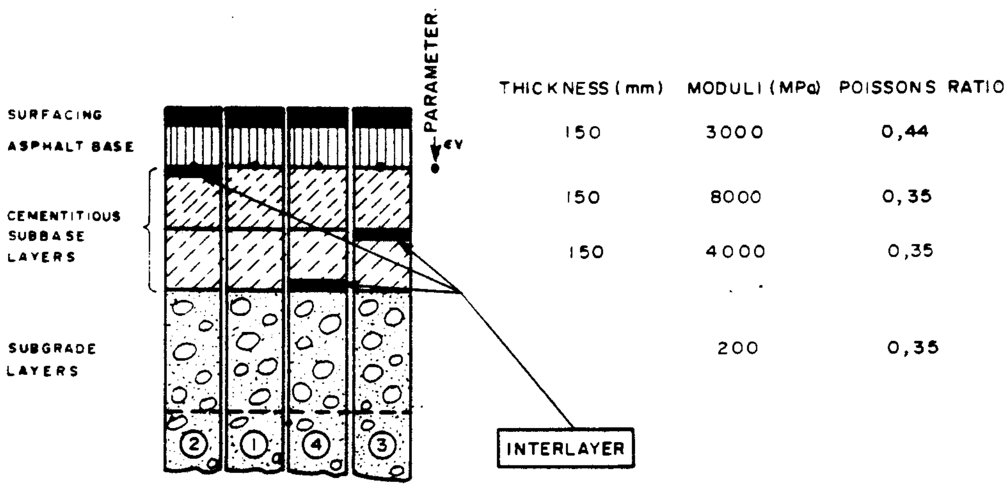


FIGURE 5.5

VERTICAL COMPRESSIVE STRAIN AT THE BOTTOM OF THE ASPHALT LAYER (TOP OF UPPER SUBBASE - INTERLAYER INCLUDED)

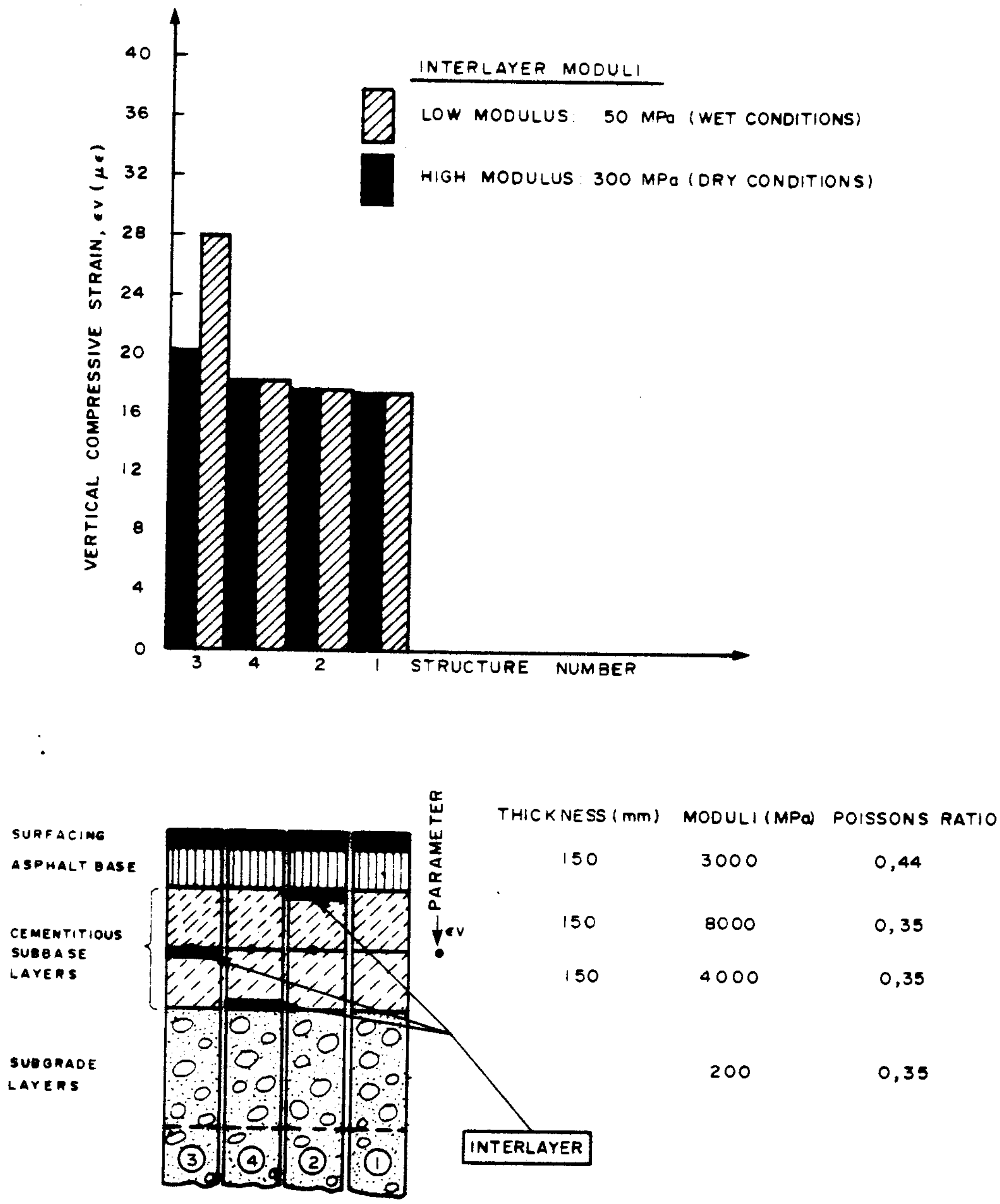


FIGURE 5.6

VERTICAL COMPRESSIVE STRAIN AT THE BOTTOM OF THE UPPER SUBBASE LAYER

In Figure 5.7, the vertical compressive strains at the bottom of the lower subbase (top of interlayers and subgrade included) are illustrated. In this case relatively high strain values were obtained with Structure 2, again resulting in the highest strain values both in the dry and especially the wet condition, but very little different from the no interlayer case (Structure 1).

5.2.1.4 Permanent deformation potential, (Pdp)

Similar to the Fdp, it was necessary to define the permanent deformation potential (Pdp) for these structures. The permanent deformation potential is defined as the sum of the vertical compressive strain values at the interfaces of the different structural layers. The vertical compressive strains at the bottom of the interlayers are ignored. Therefore :

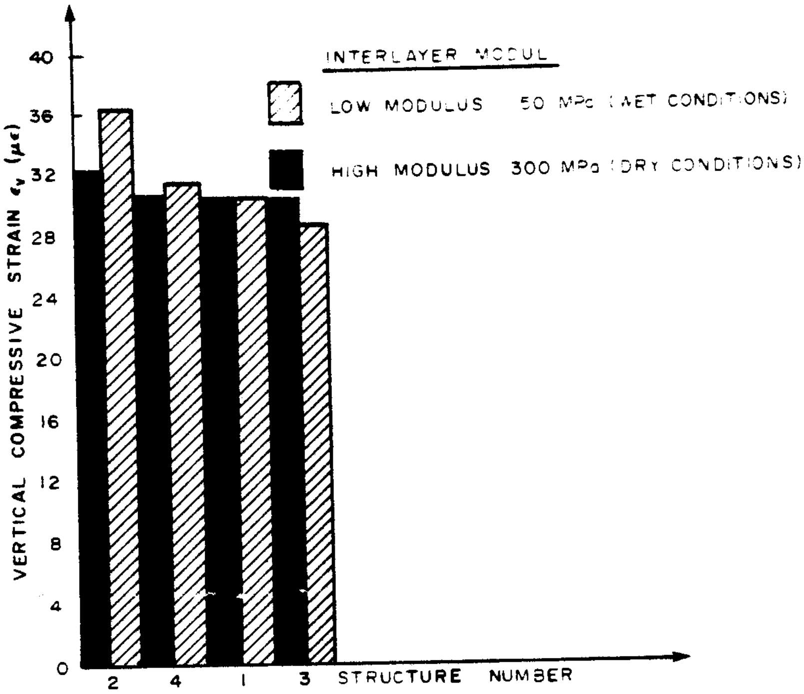
$$\text{Permanent deformation potential (Pdp)} = \sum_{i=o}^{n} \varepsilon_{vi},$$

n = number of treated layers.

The Pdp's of the different structures are illustrated in Figure 5.8. The figure indicates that Structure 2 is also most sensitive to permanent deformation with virtually only the wet case being of real significance. Relatively insignificant differences were obtained in the Pdp's of the remaining three structures.

5.2.2 Effect of the thickness of the interlayer

The previous strain analysis indicated that a 5 mm interlayer positioned between the asphalt layer and the upper subbase layer is responsible for resulting in a structure with the highest relative fatigue distress and permanent deformation potentials. It was decided to evaluate this structure further include to the effect of the thickness of the interlayer, both in the dry (300 MPa) and wet (50 MPa) conditions.



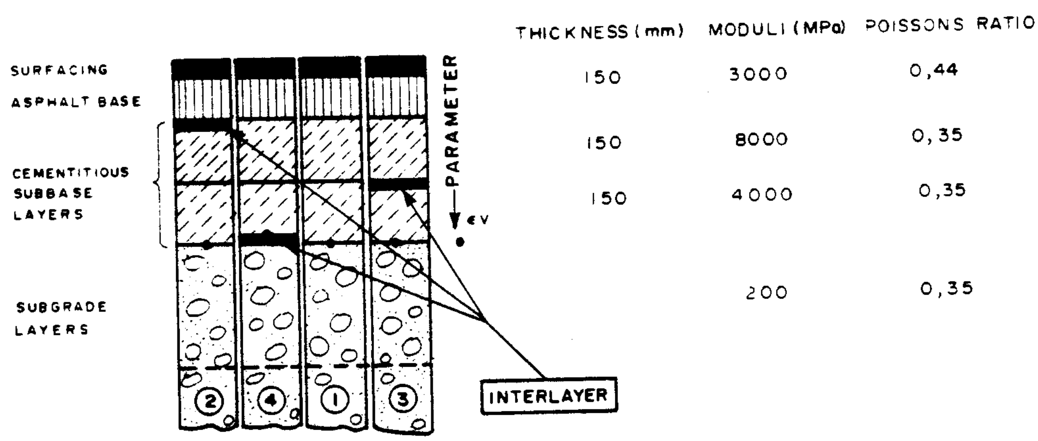


FIGURE 5.7

VERTICAL COMPRESSIVE STRAIN AT THE BOTTOM OF THE LOWER SUBBASE (TOP OF INTERLAYER INCLUDED)

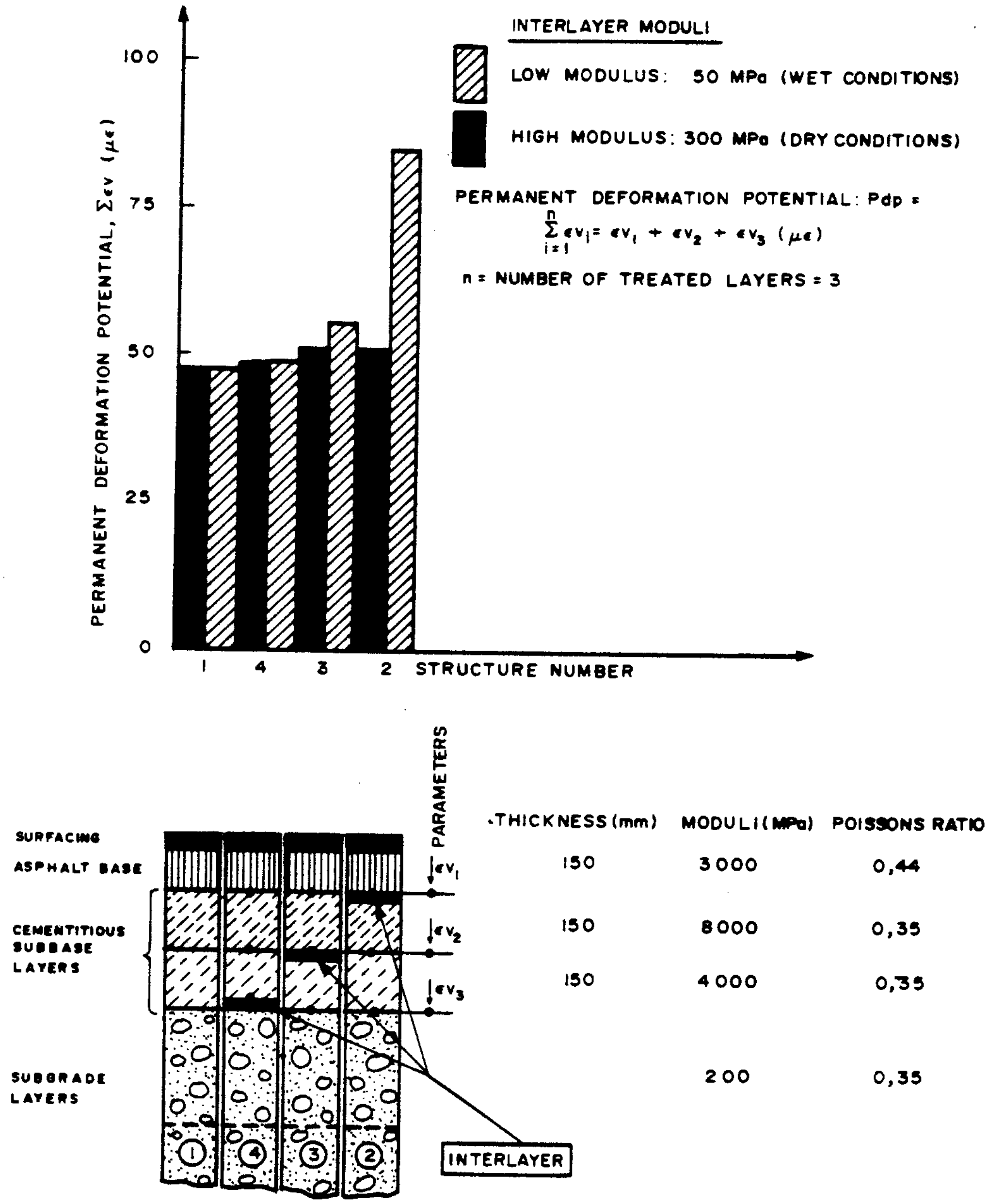


FIGURE 5.8

PERMANENT DEFORMATION POTENTIALS OF THE SOIL LAYERS IN THE 4 DIFFERENT STRUCTURES

5.2.2.1 Tensile strain, (ε_t)

The relationships between the maximum horizontal tensile strain at the bottom of the asphalt layer and the thickness of the interlayer are illustrated in Figure 5.9. The figure indicates relatively rapid increases in strain with layer thickness. The strains resulted from a wet interlayer are more than than those found in the dry state. In the wet state the structural capacity of the asphalt (fatigue life) is reduced by almost 100 per cent with an interlayer 50 mm thick and almost 50 per cent reduction with an interlayer of 20 mm thickness. The horizontal tensile strains at the bottom of the asphalt in the wet state, with interlayer thickness of 20 mm and 50 mm are approximately 115 $\mu\varepsilon$ and 155 $\mu\varepsilon$, respectively. Using these strain values in comparison with the fatigue criteria for asphalt bases (Freeme, et al, 1984 (b)), the relatively short fatigue life in comparison to those in the dry state can be calculated.

In Figure 5.10 the relationships between the maximum horizontal tensile strain at the bottom of both cementitious subbase layers and the thickness of the interlayer are illustrated. In general, the induced strains at the bottom of the lower subbase layer are almost double that induced at the bottom of the upper subbase layer. The upper subbase layer experienced however reduced strains with increase in thickness of the interlayer in both the dry and wet conditions. It is believed that the reason for this reduction in strain level is accomplished by the full friction interface assumed between the two cementitious subbase layers.

The lowest strain values were obtained at the bottom of the upper subbase during the wet conditions (interlayer modulus : 50 MPa). The reverse was true in the lower subbase for interlayer thicknesses less than approximately 30 mm. This indicates longer fatigue lives for both

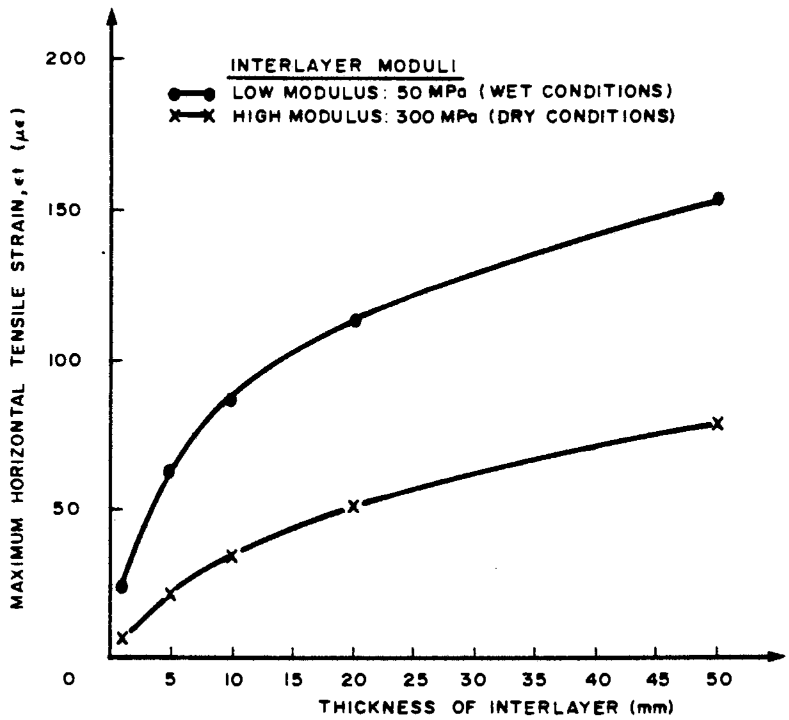


FIGURE 5.9

RELATIONSHIPS BETWEEN THE MAXIMUM HORIZONTAL TENSILE STRAIN AT THE BOTTOM OF THE ASPHALT LAYER AND INTERLAYER THICKNESS

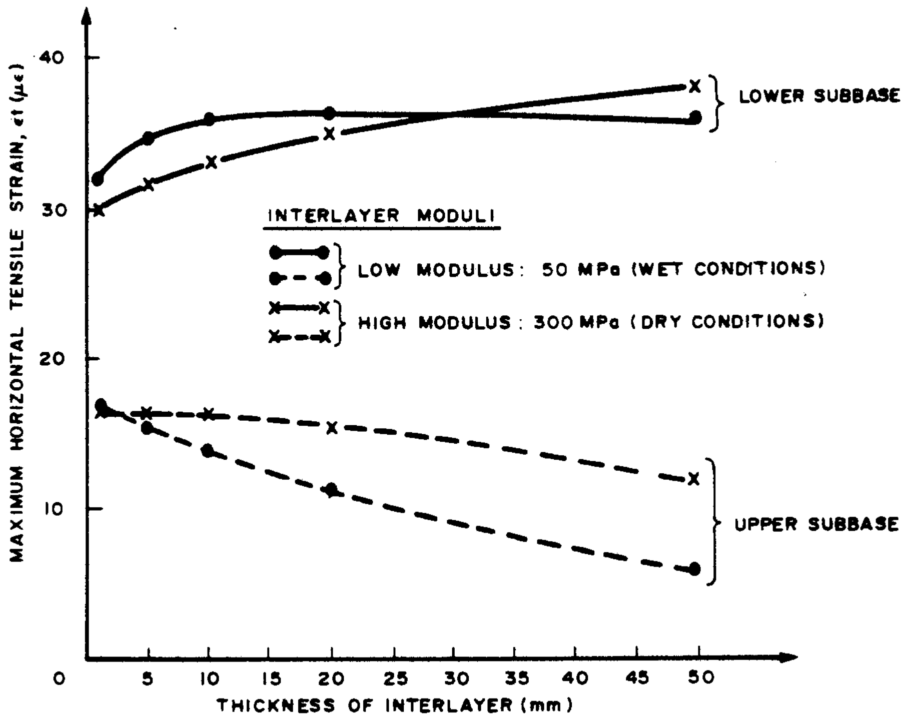


FIGURE 5.10

RELATIONSHIPS BETWEEN THE MAXIMUM HORIZONTAL TENSILE STRAINS AT THE BOTTOM OF THE TWO CEMENTITIOUS LAYERS AND THE THICKNESS OF THE INTERLAYER cementitious subbase layers when the thickness of the interlayer more than 30 mm thick and relatively soft. It is believed that the soft interlayer acts as a stress absorbing layer thus reducing the deflections (strains) deeper in the structure. The asphalt layer in consequence becomes overstrained.

5.2.2.2 Vertical compressive strain, (ε_v)

The relationships between the vertical compressive strains at the top of both interlayer and the subgrade layer against the thickness of the interlayer are illustrated in Figure 5.11. Very low strain values are experienced by the subgrade layers, both in the wet and dry conditions of the interlayer. On top of the interlayer remarkable strain levels were reached during the wet (50 MPa) state. This becomes serious with interlayer thickness of more than approximately 10 mm. This is an indication of permanent deformation potential of this interlayer. The thicker the interlayer the worse, if wet. This will leads to serious fatigue distress of the asphalt layer, resulting in cracks. Rain water could now enter and will result in consequent further weakening (softening - reducing the modulus) of the interlayer and hence again the asphalt layer.

5.2.2.3 Strain potential

The fatigue distress potentials (Fdp) of the structure with different interlayer thicknesses are illustrated in Figure 5.12. In this case the potential increased rapidly in both wet and dry conditions. The wet condition resulting in strain potentials of 50 to 60 per cent higher than those in the dry state. The figure indicates in general that the thicker the interlayer the higher the fatigue distress potential, both for dry and wet conditions.

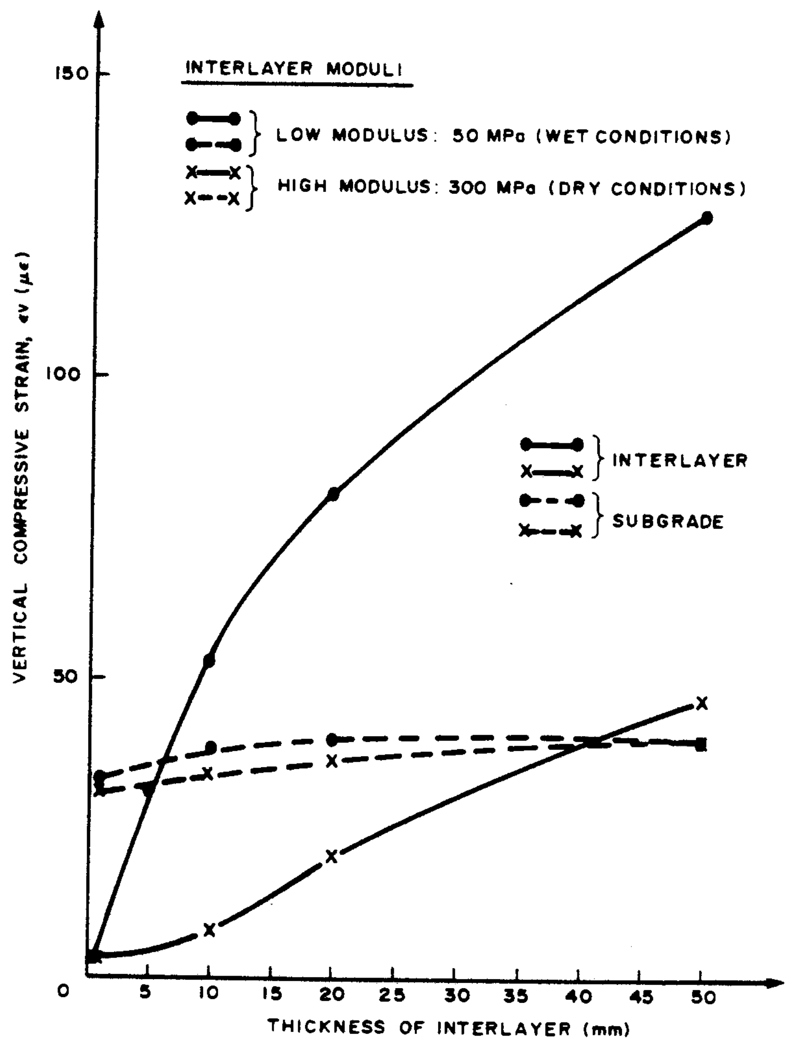


FIGURE 5.11

RELATIONSHIPS BETWEEN THE VERTICAL COMPRESSIVE STRAINS AT TOP OF THE INTER-AND SUBGRADE LAYER AND THE THICKNESS OF THE INTERLAYER

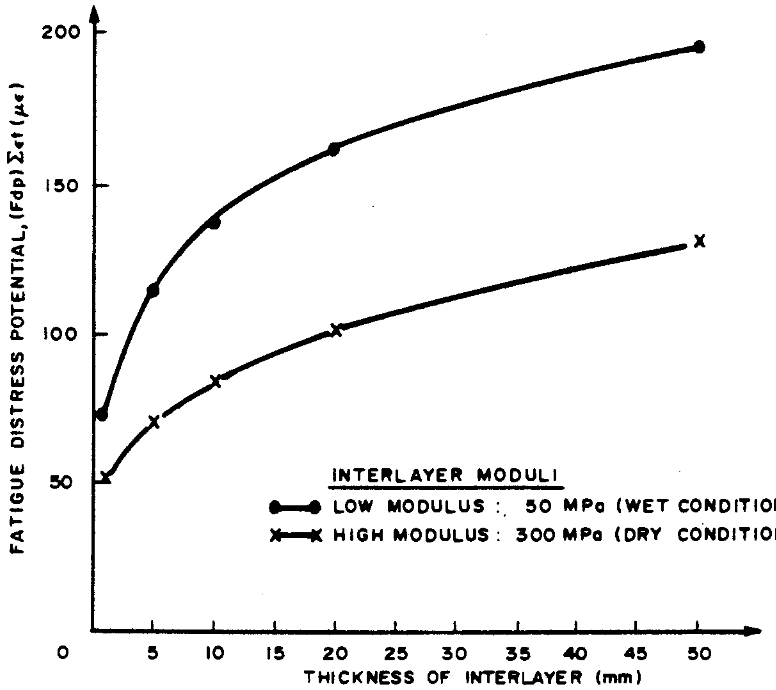


FIGURE 5.12

RELATIONSHIPS BETWEEN FATIGUE DISTRESS POTENTIAL AND THE THICKNESS OF THE INTERLAYER

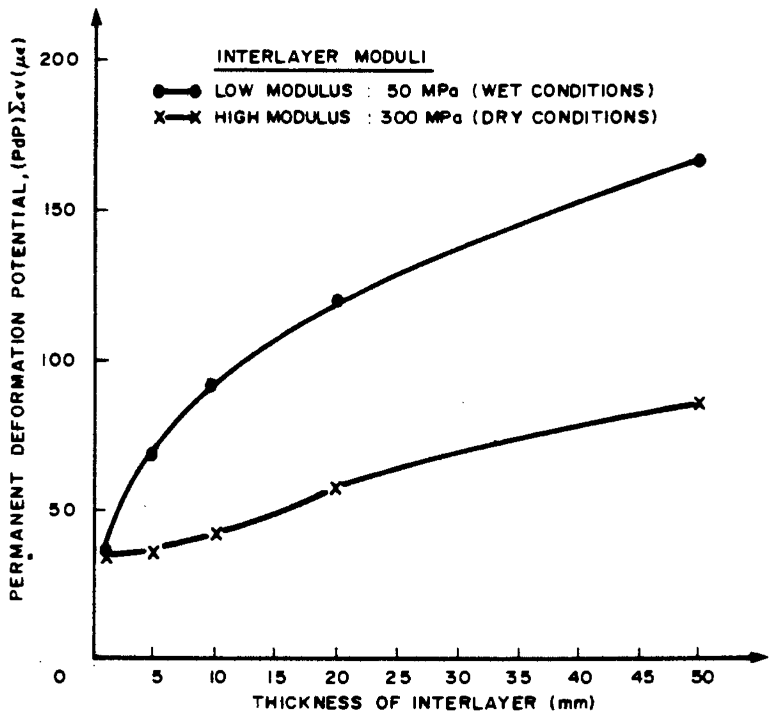


FIGURE 5.13

RELATIONSHIPS BETWEEN THE PERMANENT DEFORMATION STRAIN POTENTIAL AND THE THICKNESS OF THE INTERLAYER

In Figure 5.13, the relationships between permanent deformation potential (Pdp) and thickness of the interlayer are illustrated. Similar results as with the Fdp were found.

5.3. CONCLUSIONS

From this analitical study it can be concluded that the existence of any interlayer between the upper structural layers of this type of pavement design is detrimental and must be avoided. The thickness of the interlayer and especially its state (dry or wet) is very important. The Fdp and Pdp analyses indicated that the thicker and weaker the interlayer the highest potential for fatigue distress and permanent deformation exists.

It is thus important and the author's belief that interlayers must be avoided especially during the construction process. Layer thickness quality control should be done more carefully. Where cementitious subbase layers are used it is strongly recommended that control of the layer thickness must include the rapid field test for carbonation proposed by Netterberg, 1984. It must be fully understood that this test can identify unstabilized interlayers or carbonated interlayers, including poor mixing (spreading of stabiliser through the layer). It is further important to note that only some materials, not all, weaken as a result of carbonation. This fact is not fully quantified as yet, but is currently being studied in more detail at the NITRR.

This study further emphasizes that one must not only take cognisance of the weakness resulting from interlayers, but also ways and means must be found to optimise the construction processes to narrow the gap between the designed structure and the actual as- built structure. This will lead to even more economical pavement behaviour in future.

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CHAPTER 6

SUMMARY, DISCUSSION AND RECOMMENDATIONS FOR FURTHER RESEARCH

CONTENTS

6.1 SUMMARY AND DISCUSSION

In this thesis the behaviour of mainly weakly cemented subbase layers in asphalt base pavements is discussed. The behaviour of these layers was studied with the aid of the Heavy Vehicle Simulator (HVS).

It was shown that the maximum horizontal tensile strain (ε_t) criterion is only applicable during the precracked (uncracked) phase of the weakly cemented subbase layers. This phase, however, constitutes less than 10 per cent of the total life of these layers. From this information the importance to quantify the behaviour of these layers in the postcracked (after traffic associated cracking) is realized. Extensive HVS testing on ten different HVS test sections showed that these weakly cemented subbase layers give adequate support to the asphalt base layer provided that;

(i) the subbase moisture condition is relatively dry,

(ii) no soft and wet interlayers exist between the upper layers in the structure,

(iii) initially the cemented layers not in a granulated (poor) state,

(iv) the cemented layers non-erodible (De Beer, 1985),

(v) the slope instabilities are minimized before the construction of the relatively rigid cemented layers,

(vi) adequate and more precise quality control during construction is done to avoid inadequate mixing depth or uncemented interlayers.

If above criteria are met the protection to the subgrade layers is also be adequate for this type of design (bitumen base structures).

Although the concept of equivalent granular state was proposed during the postcracked phase of these layers, the different assumed C and ø values (in the granular state) in Chapter 2 were not evaluated for applicability. This is an area for further research.

It was further shown that the relationship between effective elastic modulus of the weakly cemented upper subbase and the traffic loading is approximately hyperbolicly related during the postcracked phase in the dry state. Rapid detoriation (break down or degradation) of the modulus occurred during the excess porewater pressure (EPWP) state. This degradation was found to be linear at approximately 360 MPa per million E80s. It is however over conservative (and incorrect) to assume this rate of degradation for the "total life" of the weakly cemented layers. It is therefore proposed to incorporate alternative wet and dry periods during the "design life" of these layers in Chapter 4. The linear (wet) degradation is used during the wet periods and the hyperbolic (dry) degradation is used during the dry periods. It is however important to note that these calculations are dependant on the expected growth rate, i.e. in E80s, duration of wet periods and dry periods, initial traffic in E80s and the structural design period. The above factors must be assessed by the design engineer. After the calculation of the moduli values, linear elastic simulations can be done in the postcracked phase. The criteria during this state are the current maximum horizontal tensile strain (ε_t), criterion at the bottom of the asphalt base layer and the vertical compressive strain (ε_v) on top of the selected subgrade and subgrade layers. No other criterion is proposed for the weakly cemented layers in the postcracked phase at this stage. It is however believed that durability (erodibility) of the weakly cemented materials is one of the most important parameters to investigate in the future.

A very important aspect resulting from this study is the effect of relatively weak interlayers in this type of pavement design. The highest fatigue distress and permanent distress potentials occurs when the interlayer is positioned between the asphalt base layer and the weakly cemented upper subbase layer. The worst situation occurs when this interlayer is thick and wet for pavements in this thesis. It is thus of utmost importance to avoid any interlayers in this type of pavement structure. This should be accomplished by adequate quality control and construction methods on site together with chemically stable cementing agents to be used in the weakly cemented materials.

6.2 RECOMMENDATIONS FOR FURTHER RESEARCH

As a result of discussions with Clauss (1982) and Savage (1985) the author belief that the following aspects of weakly cemented (and possibly strongly cemented) materials should be investigated further:

(a) The cementing agent chemistry and basic reactions on different types of weathered materials should be better understood.

(b) The critical factors to ensure stable cement reaction products should be established (organic impurities etc).

(c) The chemical and structural implications of both beneficial and detrimental carbonation.

(d) The rate of increase in the pH of the soil with increase in percentage stabilizer including determination of the initial consumption of lime (ICL) should be measured and analysed for the different weathered materials.

(e) The lime content when cementing reaction products start pozzalanic action to develop should also be established for these materials (cementing lime content, CLC).

(f) The lime content which completes modification, as defined by Clauss (1982) should also be established for these materials. (modification lime content, MLC).

(g) The effect of two phase reactions where lime is added initially to reduce the PI and then later the addition of cement to produce strength. This should be done over a range of delays between the adding of lime and cement; viz 12 hours, 24 hours, etc.

(h) The effects of delayed mixing and compaction on strength and the development non-traffic associated cracking.

(i) The effects of mixing and compaction moisture content which is relatively dry or relatively wet of optimum moisture content.

(j) The effects of particle size and mixing time of weathered soils.

(k) The swell and shrinkage characteristics of weakly cemented materials.

(l) Changes in effective elastic modulus of the weakly cemented materials, especially during the precracked phase.

(m) Erodibility of weakly cemented materials during the excess porewater pressure state (EPWP), both in the precracked and postcracked phases should be quantified. It is the author's belief that erodibility requirements must take precendence over strength requirements in weakly cemented materials in asphalt pavements. It is therefore necessary also to re-evaluate the applicability of existing durability test methods for cementitious materials such as the wet/dry durability test (NITRR, 1979). The possibility of additional new test methods such as the erodibility test (De Beer, 1985). A critical evaluation should be done of the current specifications for cementitious materials in South Africa.

(n) Different mixing and compaction methods to reduce the formation of weak interlayers, including the avoidance of smooth interfaces between the different structural layers in the pavement. The more friction between the layers the better.

6.3 REFERENCES

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