CHAPTER 5

EFFECTIVE ELASTIC MODULI DETERMINED FROM ROAD SURFACE DEFLECTOMETER MEASUREMENTS

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1 INTRODUCTION

Deflections are measured on the surface and in depth of a pavement when tested with the accelerated testing facilities, the Heavy Vehicle Simulators (HVSs). In the past the deflections as measured in depth with the Multi-Depth Deflectometer (MDD) were used to determine the effective elastic moduli of the various layers with satisfactory results. The whole deflection basin, as measured on the surface with the Road Surface Deflectometer (RSD), was always available for determining effective elastic moduli.

The method whereby effective elastic moduli are determined with the MDD is briefly described. The author proposed a new method for determining effective elastic moduli from measured RSD deflection basins which is described in detail. The effective elastic moduli determined from RSD and MDD deflections from selected accelerated tests are compared and conclusions drawn from that. The non-linearity of the subgrade is investigated by determining the effective elastic modulus by means of the proposed method and comparing the results with the normal linear-elastic approach.

2 EFFECTIVE ELASTIC MODULI CALCULATED FROM DEPTH DEFLECTIONS

Deflections are measured at various depths within the pavement structure as a standard procedure (Freeme et al, 1982) on sections tested with the accelerated testing facility, the Heavy Vehicle Simulator (HVS). The deflections are measured with the Multi-Depth Deflectometer (MDD) (Basson et al, 1980). Measurements of resilient deflections with depth are taken throughout the test under various wheel loads and these yield a good record of the change in structural response of the pavement.

Effective elastic moduli are calculated from these MDD deflections (Maree et al, 1981) by using linear-elastic theory and a linear-elastic layered computer program such as ELSYM5 (University of California, 1972). The iteration procedure whereby moduli values of the various layers are estimated and the measured deflections in depth are matched by the calculated deflections, is illustrated in Figure 5.1 (Maree et al, 1981a). The procedure is illustrated for a 40 and 70 kN wheel load. The slope of the depth deflection curve

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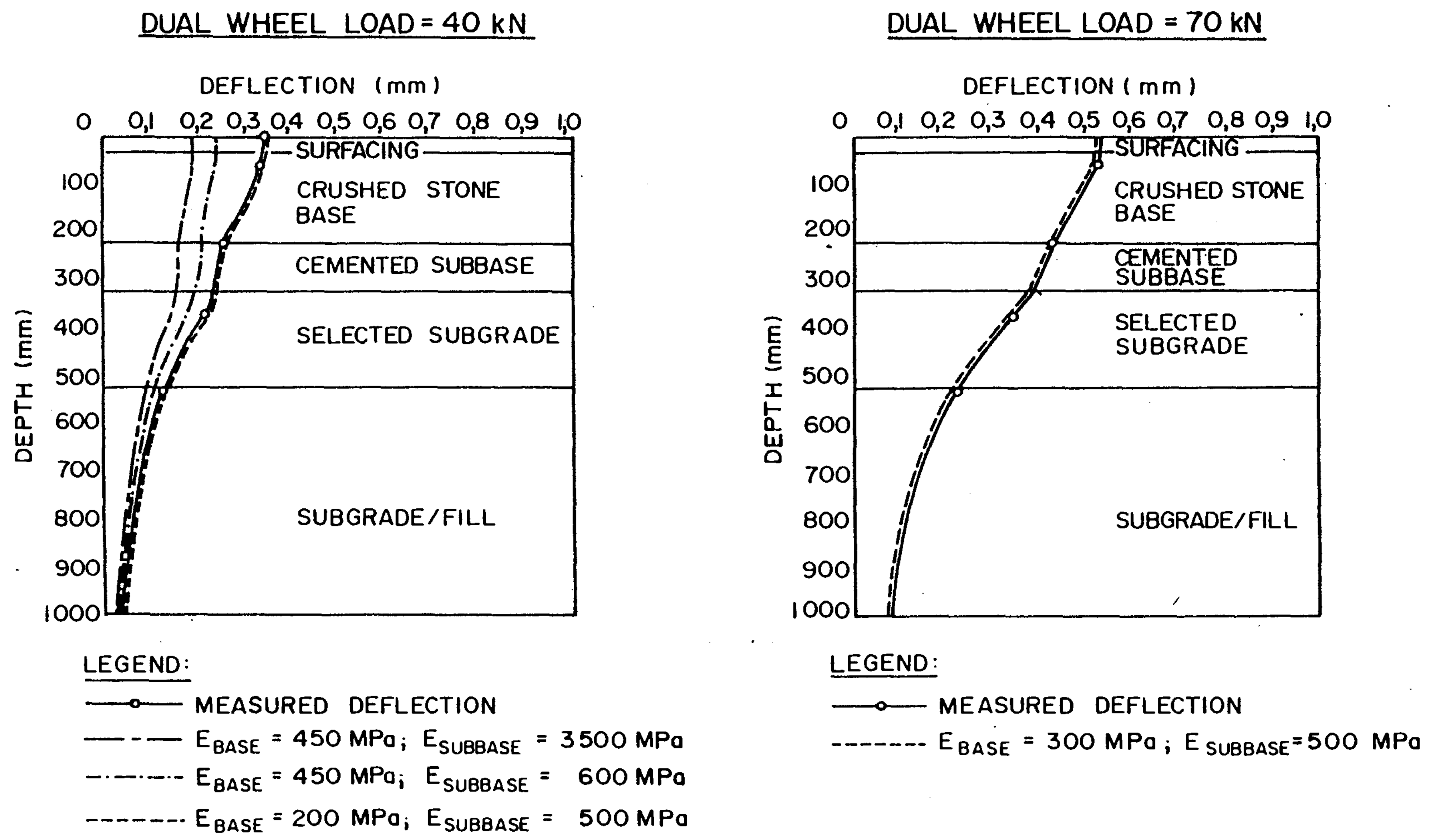

FIGURE 5.1

MEASURED AND CALCULATED DEPTH DEFLECTIONS (ROAD P157/1) at any point is an indicator of the modulus of the material at that depth.

This manual method of adjusting moduli values was automated (Coetzee and Horak, 1981) using the CHEV4 computer program (Abbot, 1977). This program was later adjusted to handle up to 15 layers (Coetzee, 1982). The moduli determined from the depth deflections are used as input values for further detailed mechanistic analyses of the pavement structures tested (Maree et al, 1981b).

3 EFFECTIVE ELASTIC MODULI CALCULATED FROM SURFACE DEFLECTIONS

3.1 General

The full deflection basin can be measured on the surface of a pavement, using the WASHO procedure (Monismith, 1979). This is measured on HVS test sections with the Road Surface Deflectometer (RSD) (Basson, 1985). In the past only the maximum deflection and radius of curvature were calculated from RSD data. Recently better use was made of the full deflection basin by calculating other deflection basin parameters too (Horak, 1985). In order to make use of the measured deflection basins more effectively, various researchers have used the full deflection basin to calculate effective elasti moduli (Horak, 1984).

In such a procedure to determine effective elastic moduli from surface deflections, at least the same number of deflections should be used as the number of layers in the pavement structure. In Figure 5.2 it is illustrated how the measured deflections, from the surface deflection basin, are used in a typical four-layered elastic system. It is assumed that the load is distributed through the pavement system by a truncated cone intersecting the pavement layers at an angle (α). Based on the concept of linear elasticity, deflection δ_4 at a distance r4 is due to the elastic compression of layer 4 while deflection δ_3 at distance r3 is due to the elastic compression of layer 3 and 4. The result is that maximum deflection δ_1 in Figure 5.2, is due to the deflection of the effective elastic moduli of all the layers (δ_1 = f(E_1, E_2, E_3, E_4).

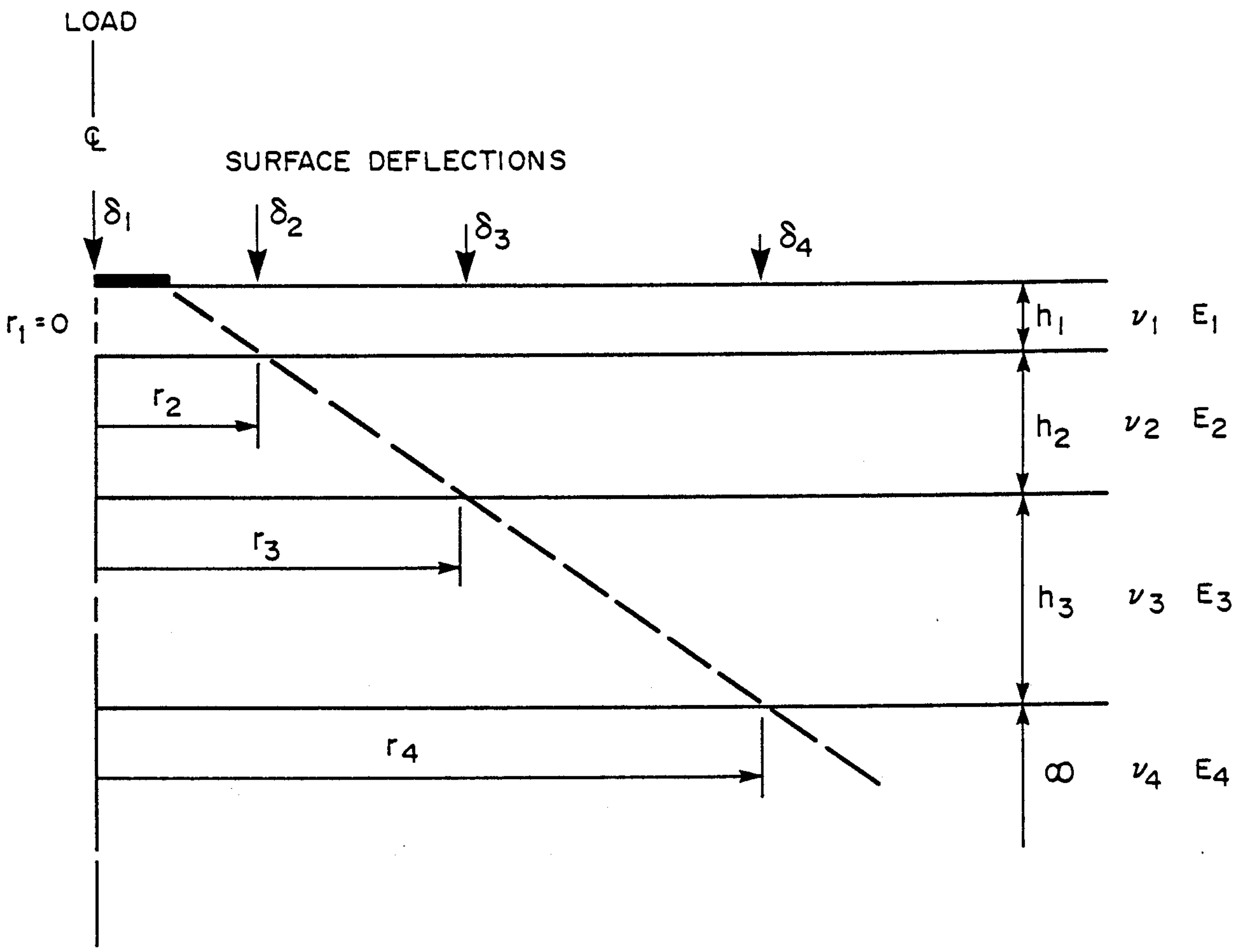

FIGURE 5.2

FOUR LAYER ELASTIC REPRESENTATION OF A PAVEMENT SYSTEM (FHWA, 1984)

3.2 The back-calculation procedure

In the back-calculation procedure the effective elastic modulus of the subgrade (E_4) is calculated first by fitting the deflection at distance r4 to the measured deflection δ_4. Then follows the calculation of E_3 by calculating and fitting the deflection at distance r3 to the measured deflection δ_3. Effective elastic moduli are therefore calculated from the lower layers upwards by fitting the deflections radially inwards towards maximum deflection.

The BISAR computer program (Horak, 1985) is used in this back-calculation procedure because of its superior accuracy in the vicinity of the loaded wheels (Tam, 1985). Estimate or seed values of elastic moduli are initially assumed for all the layers based on material information, as-built information and material type. These seed values are then adjusted by comparing the calculated deflection with the respective measured deflection at that point considered. This adjustment procedure is done in two phases. The first phase makes use of an interpolation procedure to save computer time. The second phase uses these calculated effective elastic moduli as input or as seed values and calculates the effective elastic moduli more accurately.

In Figure 5.3 the flow diagram of the first phase of adjustment is illustrated. The deflection (δ_i^{seed1}), as calculated from seed values (E_i^{seed1}), is calculated and compared with the measured deflection ($\delta_i^{meas.}$). A tolerance of 5 μ-meter (0,005 mm) is used to decide whether an adjustment is necessary. If there is an adjustment necessary, $E_i^{Calc.1}$ is calculated as shown. The factor 2 in the calculation is used to deliberately overcompensate. The BISAR program is run again and now a linear interpolation is used in a log-log space (deflection versus modulus) as indicated in Figure 5.3. The value $E_i^{interp.}$ is now used as the new seed value in the calculation and adjustment of the other layers in phase 1 until all the layers are adjusted in such a way.

In phase 2 the seed values of the elastic moduli are those calculated as $E_i^{interp.}$ in phase 1. This is illustrated in Figure 5.4.

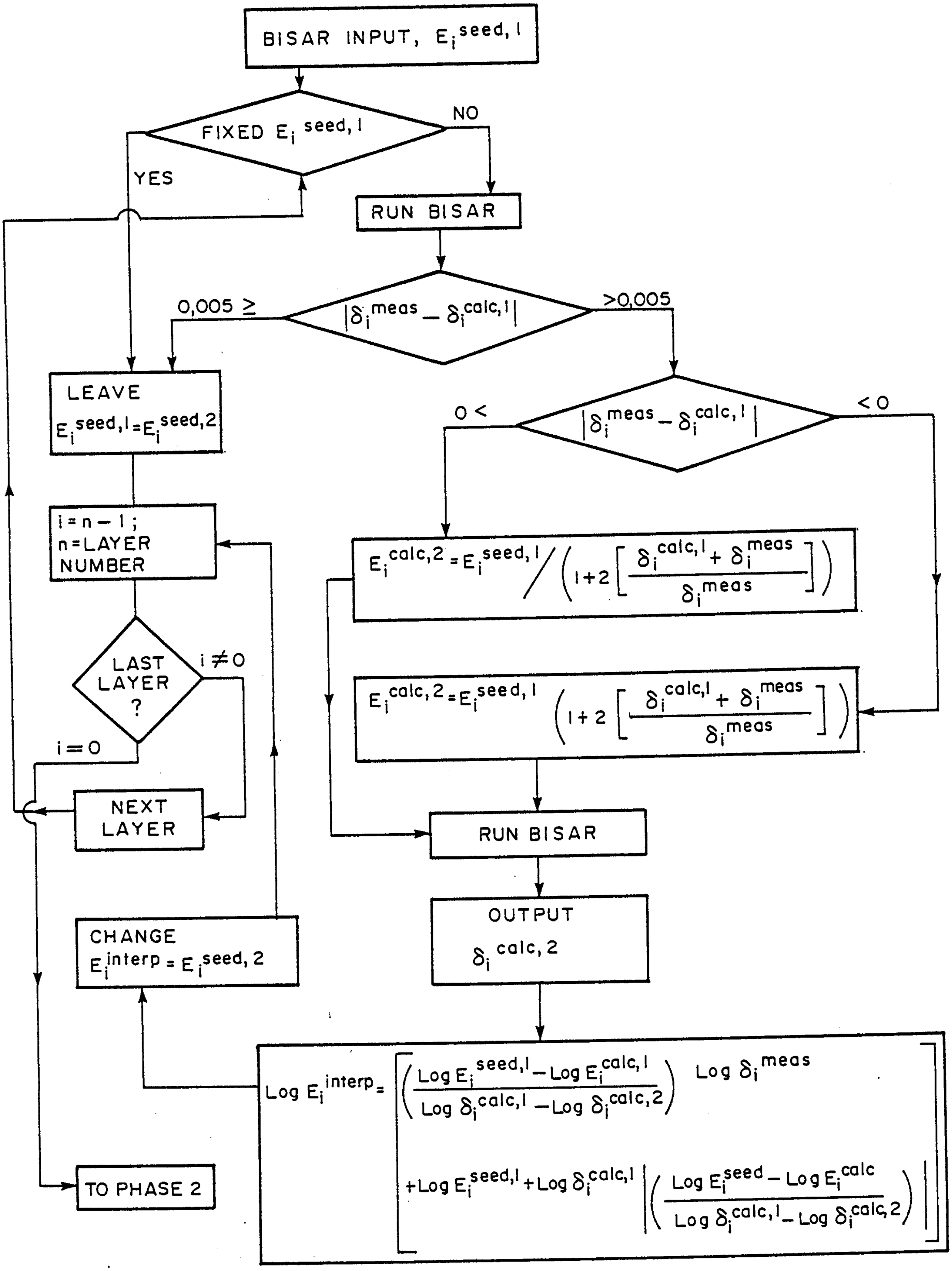

FIGURE 5.3

FLOW DIAGRAM OF PHASE I OF PROGRAM BISFT

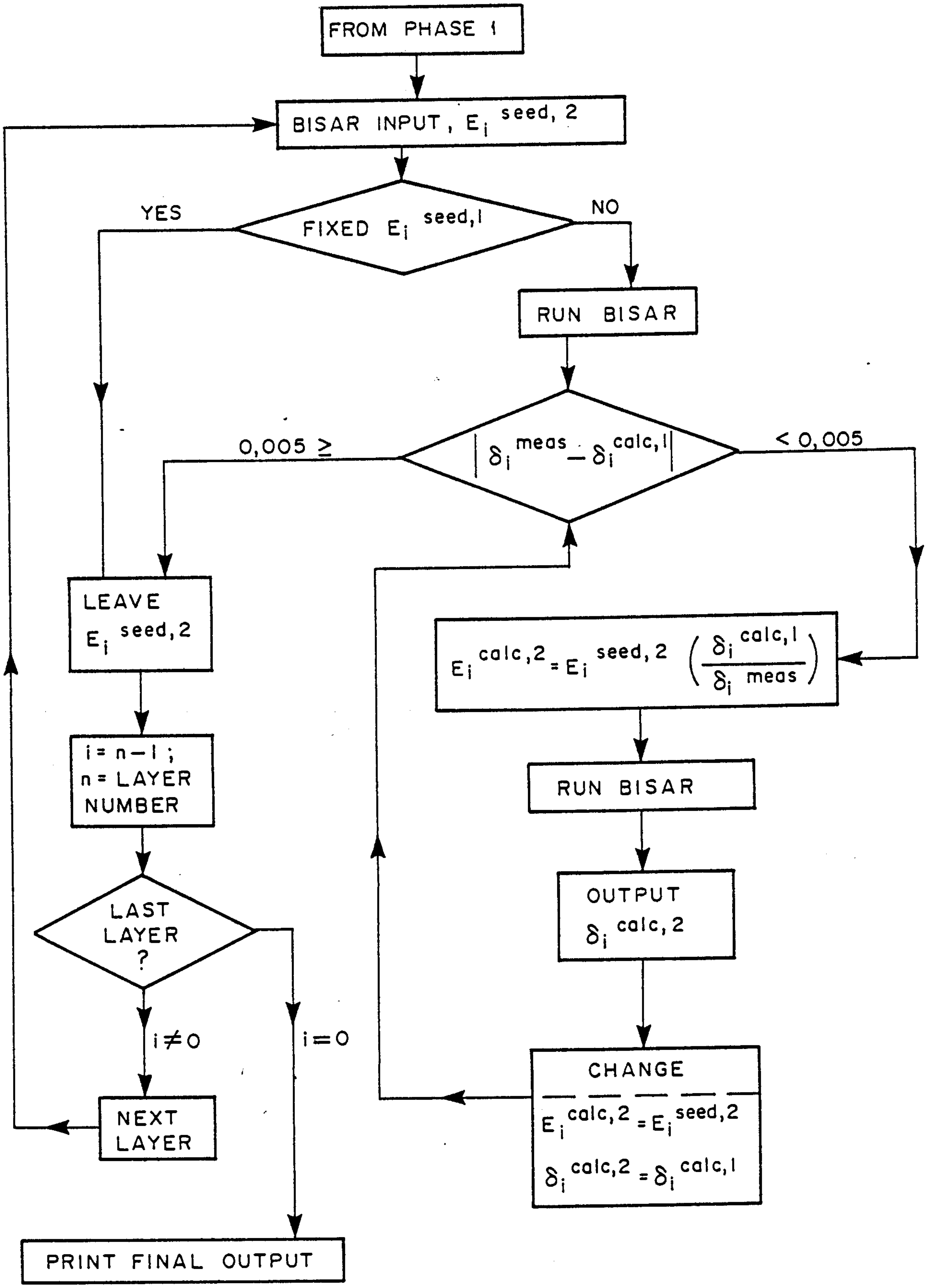

FIGURE 5.4

FLOW DIAGRAM OF PHASE 2 OF PROGRAM BISFT

The deflections calculated ($\delta_i^{calc,1}$) and measured ($\delta_i^{meas.}$) are checked against the specified tolerance of 5 μ-meter in order to adjust as indicated. Each layer is adjusted in the sequence as discussed earlier. A maximum of 5 total iteration loops are allowed at this phase to limit computer time. An iteration loop is seen as the adjustment of all the layers.

As the adjustment of effective elastic moduli of the layers near the surface are more difficult to determine and more costly in terms of computer time (Tam, 1985) an option to fix the effective elastic modulus (E_i^{Seed},fixed) was built into the program. In general the material information, as-built information and knowledge of these layers are such that the material can be classified accurately (Freeme, 1983). The non-linearity of the subgrade is not considered yet in the BSFT back-calculation program. This can however be done manually beforehand.

3.3 Non-linearity of the subgrade

The mechanistic design and analysis procedure in South Africa does not yet accommodate the non-linearity of the subgrade. The back-calculation procedure of the MDD inherently makes use of a non-linear approach though. With the assumption that deflection at a depth of 2 meter (where the MDD is anchored) is zero, it is implied in the back-calculation procedure that below 2 meter there exists a semi-infinite rigid layer or a layer with a rather high effective elastic modulus.

The approach that is suggested, is based on the procedure described by Tam (1985). The subgrade is subdivided into four layers with thicknesses of 0,6 m, 1,0 m, 1,0 m and 2,0 m respectively, overlaying a semi-infinite layer. The BISAR computer program is then used to calculate effective elastic moduli at mid-depth of each layer and 2,0 m into the semi-infinite layer. The seed values for the effective elastic moduli as derived by means of BSFT are used as input for the other structural layers. The deflection at distance (n-1)305 mm (where n is the number of structural layers) from maximum deflection (δ_o) is first checked for non-linearity. This will be 1220 mm for a a five layered system. If the measured deflection (δ^{m}_{1220}) differs more than 0,005 mm from the calculated deflection (δ^{c}_{1220}), non-linearity is assumed in the subgrade. The seed effective elastic moduli of the subgrade layers are then altered by using the equation below as derived by Brown (1979);

$$E_{subg.} = A \frac{P_o}{q}^{1-B}$$

where E is the sugrade layer effective elastic modulus (MPa)
P^1_o is the initial effective overburden stress (MPa),
($P_o = {}^1/_3\ (\sigma^1_1 + 2(\sigma^1_3))$)
q is the deviatoric stress due to applied load (MPa),
($q = \sigma^1_1 - \sigma^1_3$)

A is the amplification factor (MPa)
B is the nonlinear power coefficient

σ^1_1 and σ^1_3 are effective major and minor principle stresses (MPa)

In the iteration procedure, the value of A is taken as the subgrade effective modulus as determined in the linear BSFT program. The value of B is taken as 0,2 as suggested by Tam (1985). In a typical five-layered pavement structure, the deflection at 1220 mm (δ_{1220}) is used to adjust the value of A first, using:

$$A_{i+1} = A_i\left(\frac{\delta_c}{\delta_m}\right)$$

where A_i and A_{i+1} are the i-th and (1+i)th iterative value of A
δ_c and δ_m are the calculated and measured deflections respectively.

With the new value of A and the same value of B, a new set of subgrade effective elastic moduli are computed. The effective elastic moduli of the other structural layers are kept the same.

This process is repeated until the difference between the measured and calculated deflections at 1220 mm is less than 0,005 mm. The deflection at n*305 mm, which is 1525 mm for a 5 layered system, is then used for comparison purposes. The non-linear power coefficient, B, is then adjusted as follows:

$$B_{i+1} = B_i \frac{(\delta_c + \delta_m)}{2\delta_c}$$

where B_{i+1} and B_i are the (i+1)th and i-th iterative values of B and δ_c and δ_m are the calculated and measured deflections at 1525 mm from load centre. This process of adjusting B and th effective elastic moduli of the subgrade layers is repeated until the criterion of 0,005 mm deflection difference is satisfied.

3.4 Determining effective elastic moduli

3.4.1 Assuming linearity of the subgrade

Effective elastic moduli were determined from selected pavement structures as tested with the accelerated testing facility, the HVS. The layer depths are known and the effective elastic moduli had already been determined in some cases using the depth deflections from the MDD. The effective elastic moduli were calculated using the measured surface deflection basin data as described in Section 3.2. The angle α, shown in Figure 5.2 was taken as 38° as standard after α was varied between 30°, 45° and 60°. The value of 30° was found to give effective elastic moduli that were better related to those determined with the MDD measurements. This is however also a function of pavement type and material type.

The wheel load used in the calculations was 40 kN with a 520 kPa tyre pressure. In Table 5.1 the calculated effective elastic moduli are shown for a typical granular base pavement tested at Erasmia (Horak, 1986a). The effective elastic moduli as determined from MDD and RSD measurements are shown for each structural layer at various repetitions. In order to compare the effective elastic moduli determined with the two measurements, the modular ratios (RSD method/MDD method) are also shown. A modular ratio of about 1 would indicate nearly the same effective elastic moduli as determined by both measuring techniques.

The BSFT back-calculation program uses increasingly more time to calculate effective elastic moduli nearer to the surface. This is related to the calculation procedure as outlined in Section 3.2 and the difficulty of achieving the specified accuracy. In order to save computer time, the effective elastic moduli of the surfacing and the granular base layer were fixed. The fixing was based on other material information and prior determined effective elastic moduli with MDD deflections.

TABLE 5.1 - Calculated effective elastic moduli (Erasmia)

Actual repetitions	Layer description	Effective elastic moduli (MPa) determined from		Modular ratio**
		MDD*	RSD**	
10	Base	390	400	1,03
	Subbase	1 180	499	0,42
	Selected	210	210	1,00
	Subgrade	200	230	1,15
196 000	Base	150	150	1,00
	Subbase	230	163	0,71
	Selected	125	43	0,34
	Subgrade	85	194	2,28
564 000	Base	235	240	1,02
	Subbase	220	145	0,66
	Selected	130	58	0,45
	Subgrade	85	220	2,59
700 000	Base	205	210	1,02
	Subbase	230	151	0,66
	Selected	100	23	0,23
	Subgrade	55	210	3,82
1 009 000	Base	195	200	1,03
	Subbase	220	155	0,71
	Selected	120	37	0,31
	Subgrade	55	270	4,91

* MDD measured depth deflections and RSD surface deflections
** Ratio of RSD/MDD effective elastic moduli

For that reason the modular ratio of the base layer is constantly in the region of 1 in Table 5.1.

In the pre-cracked phase of the cemented subbase, the effective elastic moduli determined from RSD measurements indicate a lower value than the one determined from MDD measurements. In the cracked phase of the subbase, the modular ratio increases from the initial low 0,42 to a rather constant 0,6 to 0,7. This still indicates an undercalculation with the RSD measurements.

The same tendency of undercalculation by using RSD measurements is also true when the effective elastic moduli of the selected layer are inspected. In this case though, the undercalculation becomes worse with the increase in actual repetitions. The subgrade, in contrast, is however constantly overcalculated when the effective elastic moduli determined with RSD measurements are compared with those determined with MDD measurements. This overcalculation increases with the increase in actual repetitions.

It is therefore obvious from the calculated subgrade effective elastic moduli that the non-linearity of the subgrade is not initially influencing the effective elastic moduli of the subgrade, but at a later stage of trafficking it seems to become more influential. The shift in balance, when the subbase cracks, seems to have an effect on both the subbase and the selected layer underneath. This shift in balance can also be reflected by the effective elastic moduli of the subgrade.

In Table 5.2 the effective elastic moduli as calculated from RSD deflection measurements are shown for a typical light-structured granular pavement at Malmesbury (Horak, 1986a).

In this case no effective elastic moduli were determined with MDD deflections. It is clear though that the pavement balance did change to a much deeper structure as the actual repetitions increased. The base and selected layers showed a steady decrease in effective elastic moduli as actual repetitions increased. The structurally stronger subbase and the subgrade showed initial decreases in effective elastic moduli, but increased again towards the end. The effective elastic moduli of the base and surfacing were again fixed at values as determined with the aid of other material information.

TABLE 5.2 - Calculated effective elastic moduli (Malmesbury)

Actual repetitions	Layer description	Effective elastic moduli from RSD deflections (MPa)
10	Base	80
	Subbase	200
	Selected	60
	Subgrade	230
50 000	Base	60
	Subbase	138
	Selected	35
	Subgrade	161
200 000	Base	50
	Subbase	100
	Selected	28
	Subgrade	157
350 000	Base	45
	Subbase	123
	Selected	30
	Subgrade	230
496 000	Base	45
	Subbase	107
	Selected	28
	Subgrade	285
591 000	Base	40
	Subbase	143
	Selected	30
	Subgrade	521

3.4.2 Assuming non-linearity of the subgrade

The procedure as set out in Section 3.3 was followed in a few selected cases. This was done to determine whether the subgrades do exhibit non-linear behaviour and whether it affects the calculated effective elastic moduli of the other structural layers. The results are shown in Table 5.3.

In the case of the Erasmia test it is clear at the start of the test that there is no strong evidence of non-linearity of the subgrade in depth of the pavement. The average subgrade effective elastic modulus of 350 MPa is higher than the 230 MPa of the linear approach. That difference is due to the calculation procedure of the non-linear approach further away from the centre of loading. It also illustrates some stress dependency as the calculation point is deeper into the structure in the non-linear approach. In the recalculation with BSFT of the effective elastic moduli of the other structural layers, the subbase did increase to a more realistic figure for a cemented subbase.

TABLE 5.3 - Effective elastic moduli with a linear and non-linear approach

Test	Actual repetitions	Layer description	Effective elastic moduli (MPa)	
			Linear	Non-linear
Erasmia	10	Base	400	400
		Subbase	499	912
		Selected	210	210
		Subgrade	230	359
		Subgrade	-	349
		Subgrade	-	353
		Subgrade	-	353
		Subgrade	-	353
Erasmia	1 009 000	Base	200	200
		Subbase	155	155
		Selected	37	19
		Subgrade	270	466
		Subgrade	-	627
		Subgrade	-	638
		Subgrade	-	640
		Subgrade	-	648
Malmesbury	591 000	Base	40	40
		Subbase	143	177
		Selected	30	25
		Subgrade	521	760
		Subgrade	-	1 004
		Subgrade	-	1 003
		Subgrade	-	1 002
		Subgrade	-	1 001

Towards the end of the test at Erasmia (1 009 000 actual repetitions) stronger evidence of non-linearity was shown by the subgrade. This tendency of non-linearity is true in depth of the subgrade too. In the recalculation with BSFT the selected layer showed a tendency to lower also as for the linear case. It is believed that the structural strength of the selected layers is not reflected correctly. Although there is evidence of stress-softening behaviour this layer is forged into balance with the rest of the pavement structure. This means that the subgrade underneath does not suddenly increase to a value of 466 MPa as a discrete event. It is rather a gradual change from the value of 466 MPa to a lower value, but not necessarily as low as 19 MPa, when moving up towards the subbase.

In the case of the test at Malmesbury there is also definite indications of non-linearity of the subgrade. The effect on the other structural layers when recalculation with BSFT is done, is however minimal.

4 SUMMARY AND CONCLUSIONS

(a) Effective elastic moduli are calculated very effectively using the Multi-depth Deflectometer (MDD) measurements. The effective elastic moduli calculated in this manner have been correlated very effectively with the mechanistic analysis procedure.

(b) The procedure to calculate effective elastic moduli from surface deflections was described using the BISAR linear-elastic computer program.

(c) Effective elastic moduli of the various pavement layers in the stiff behaviour state are correlating well when values determined from surface deflection basin measurements are compared with those determined from depth deflections (MDD).

(d) In the flexible behaviour states the effective elastic moduli of the subgrade, as determined from surface deflections, are determined 2 to 5 times larger than with the MDD deflections.

(e) The BSFT back-analysis procedure needs considerable assistance in the calculation of particularly the upper layers, in order to ensure unique results.

(f) The selected layer in a four or five-layer pavement system is constantly undercalculated compared to the upper layers, in order to ensure unique results.

(g) Non-linearity of the subgrade can be modelled by the BSFT back-analysis procedure. Non-linearity does feature more strongly in the subgrade effective elastic moduli when the pavement is in the very flexible behaviour state. This is probably due to the stress-softening behaviour accompanying the normal change in balance towards a deeper pavement structure.

(h) The number of pavement structures that were back-analysed with the aid of the BSFT procedure are limited and results can be used only as mere indicators.

CHAPTER 6

RELATIONSHIPS BETWEEN DISTRESS DETERMINANTS AND DEFLECTION BASIN PARAMETERS : A LITERATURE SURVEY

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1 INTRODUCTION

Material characterization is greatly enhanced by the use of deflection basin measurements. This has been described in the preceding chapters. This forms the basis of detailed analyses with multi-layer linear elastic computer models in order to calculate typical distress determinants. These distress determinants must normally reflect the two distress states, fatigue and rutting of the pavement. In the analysis of flexible pavements the distress determinant for rut is vertical subgrade strain and for fatigue the distress determinant is that of the maximum horizontal asphalt strain. Jordaan (1986) defines the analysis approach outlined above as the non-simplified approach. It is possible to make use of empirical-theoretical relationships between these distress determinants and deflection basin parameters. Jordaan (1986) classifies the latter approach as a design curve approach. These relationships between the distress determinants and deflection basin parameters are pre-calculated or established by empirical relationships. In the discussion in this chapter such relationships between deflection basin parameters and the distress determinants are highlighted as found in literature. A brief discussion on the validity of the use of these distres determinants are given too.

2 FATIGUE CRACKING

Fatigue cracking is a major type of distress in bituminous pavements. The three basic types of cracking are: longitudinal, transverse and alligator or crocodile cracking. Longitudinal cracking is mainly due to environmental effects. Transverse and crocodile cracking are usually due to the effect of traffic loading. Shrinkage cracking (transverse) is mostly kerbed by proper material specification and construction control.

The traditional distress parameter relating to the fatigue life of the asphalt layer is described by Snaith et al. (1980) as follows:

$$N = c \left(\frac{1}{\varepsilon_{HA}}\right)^m$$

where:

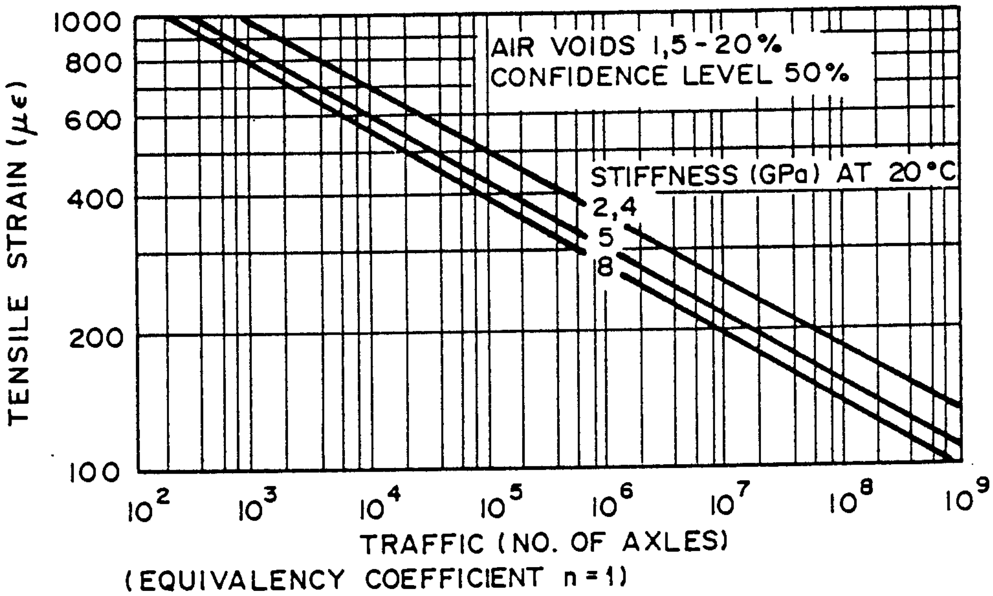
N = number of applications to failure

ε_{HA} = tensile strain repeatedly applied

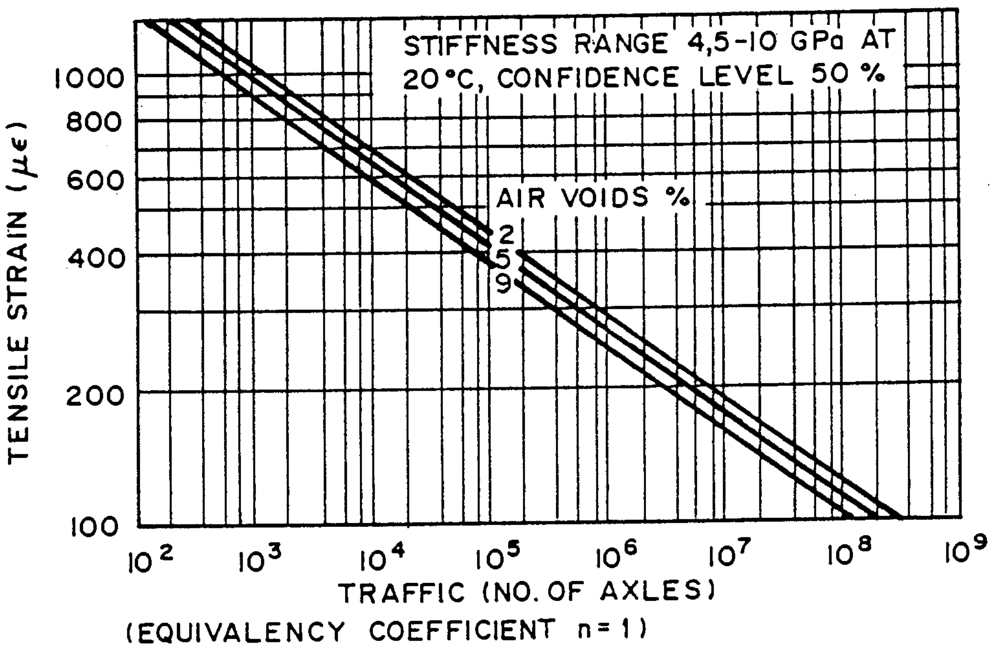
c, m = material coefficients.

Normally researchers use the radial or horizontal strain (ε_{HA}) at the bottom of the asphalt concrete layer as the criterion. This is also used for the mechanistic design procedure in South Africa. Freeme et al. (1982a) however, refer to recent research indicating that the maximum tensile strain does not necessarily occur at the bottom of the asphalt layer. Under certain conditions of low stiffness in the asphalt layer, the principle tensile strain at depths 0,8 to 1 times the radius of the load area exceeds the magnitude of the strain at the bottom of the layer. Typical relationships determined for horizontal strain (ε_{HA}) are shown for thin and thick asphalt layers in Figures 6.1 and 6.2 for typical South African asphalt mixes. Shift factors are used to compensate for the crack growth until the cracking is clearly visible. For the South African condition these shift factors range from 2 to 10.

Although the maximum horizontal strain is normally calculated at the bottom of the asphalt concrete layer, as described above, the effect of the thickness of this layer is also noted by researchers. Anderson (1977) shows that asphalt concrete layers of 25 to 75 mm reached maximum tensile strain values at thicknesses of 50 to 75 mm for various pavement types as shown in Figure 6.3. Dehlen (1962a) also noted that theory indicates that there may be a critical thickness of surfacing, in the range of 50 to 100 mm, for which flexural stresses are a maximum. This same tendency is illustrated by Grant and Walker (1972) for various combinations of pavement structural strength and radius of curvature (R) (See Figure 6.4). They also note: "For the majority of pavements where the asphalt thickness was less than 50 mm the maximum tensile strain in the asphalt layer was at the surface, while invariably being at the bottom of the layer for greater thicknesses." Freeme et al. (1982a) and Patterson (1985) indicate that for such thin asphalt surfacing layers the total number of axles, irrespective



(a) GAP-GRADED ASPHALTS



(b) CONTINUOUSLY GRADED ASPHALTS

FIGURE 6.1

RECOMMENDED FATIGUE CRITERIA FOR THIN BITUMEN SURFACINGS

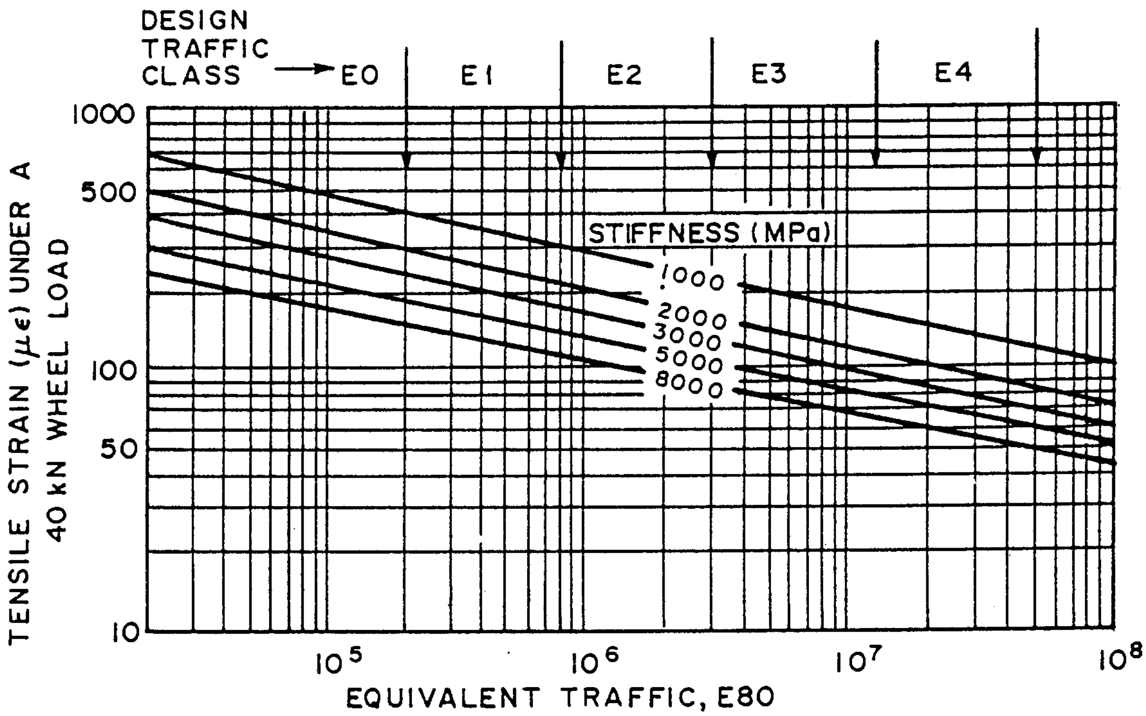


FIGURE 6.2

RECOMMENDED FATIGUE CRITERIA FOR THICK BITUMEN BASES

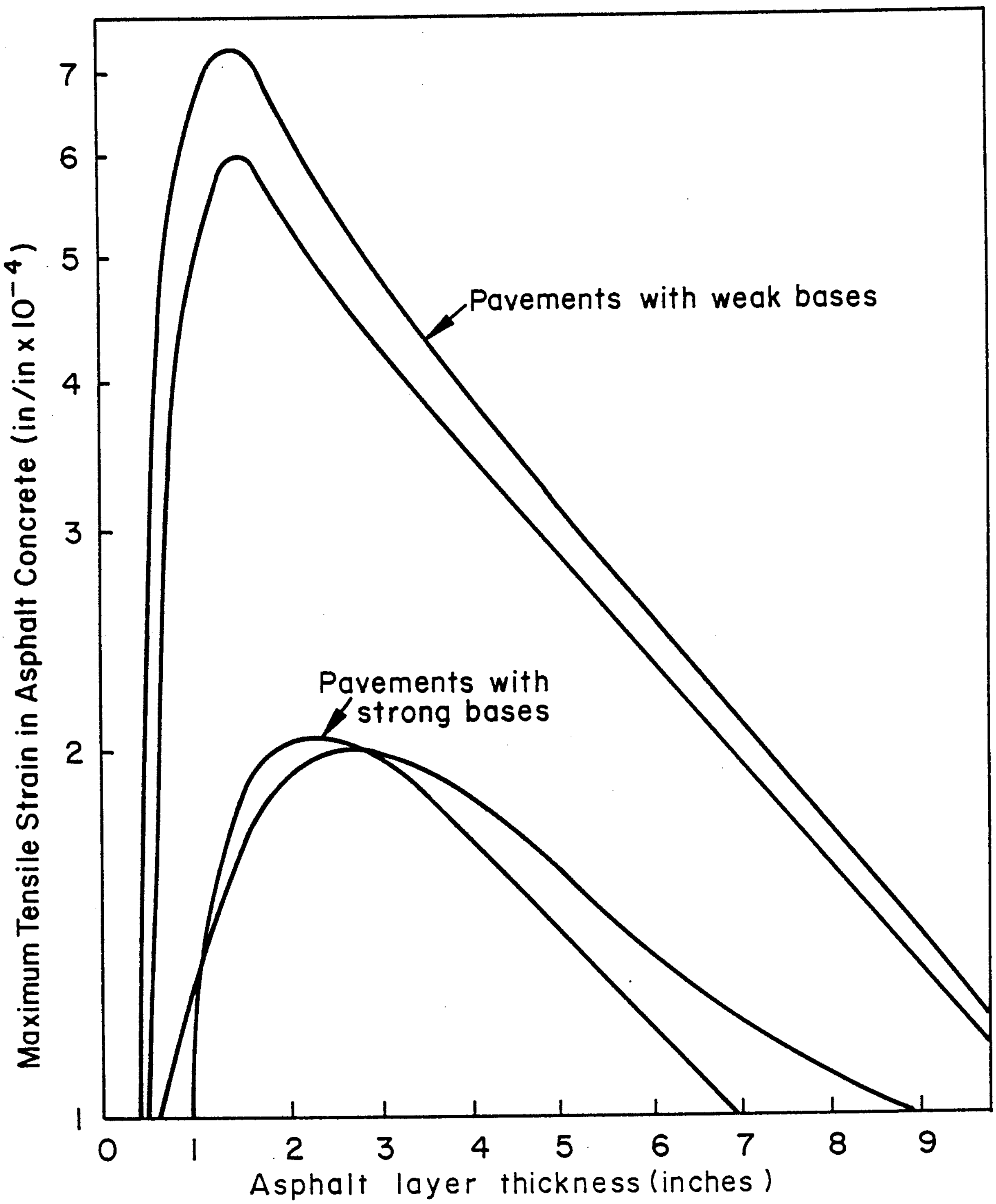


FIGURE 6.3
RELATIONSHIP BETWEEN A.C. THICKNESS AND TENSILE STRAIN (Anderson, 1977)

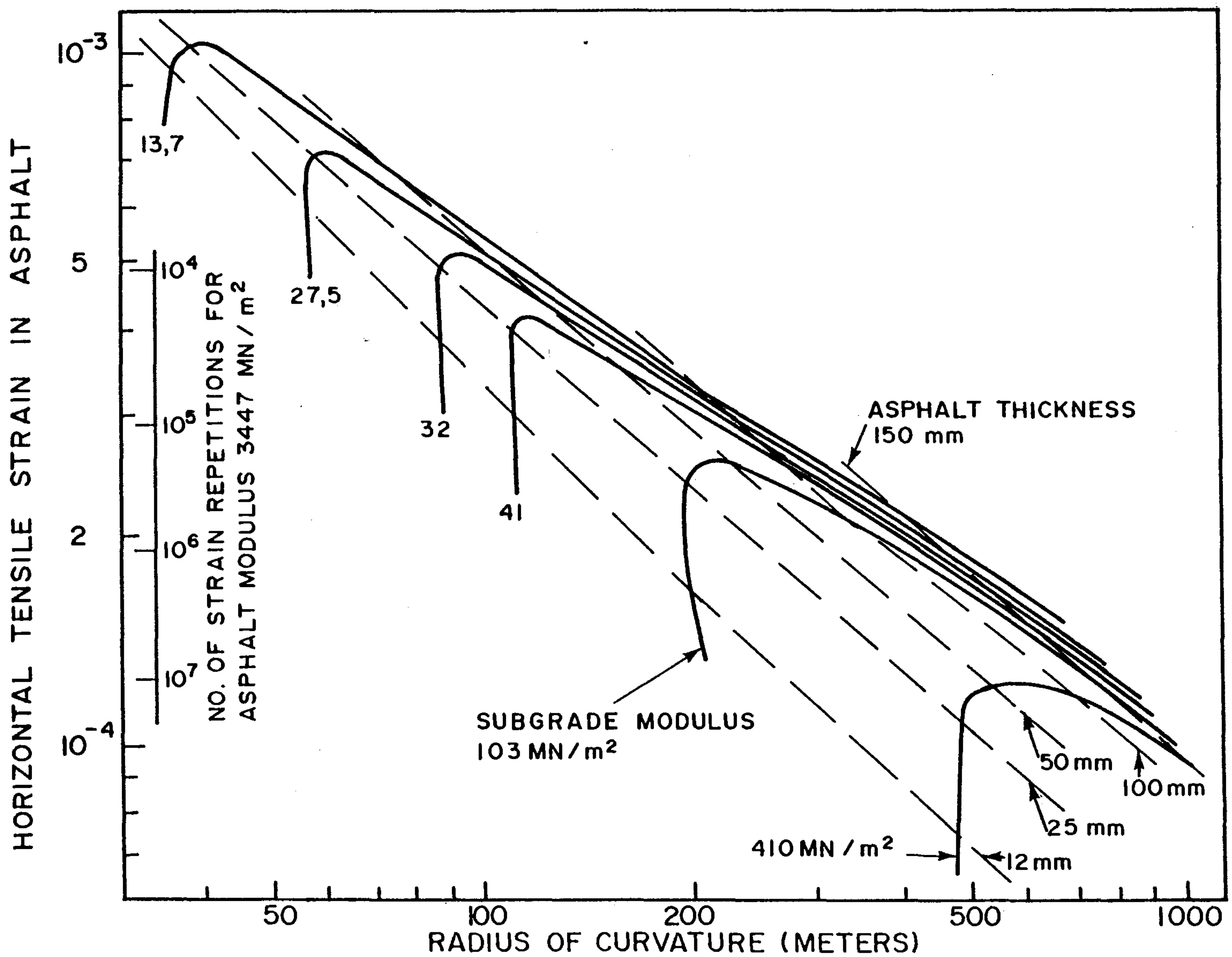


FIGURE 6.4
CHART FOR DESIGNING AGAINST EXCESSIVE TENSILE STRAIN IN THE ASPHALT (Grant and Walker, 1972)

of load, should be considered. Molenaar (1983) refers to various researchers in order to make practical conclusions on cracking behaviour (for example cracking from bottom to top or vice versa), and, in keeping with the recent research trend, also refers to fracture mechanics principles.

Various researchers have done extensive analyses on different pavement structures in order to arrive at regression functions of the deflection basin parameter and the distress determinant for fatigue cracking. The typical relationships between asphalt tensile strain (ε_{HA}) and surface curvature index (SCI) for three subgrade conditions are shown in Figure 6.5 . This is true for typical three-layered pavements.

Anderson (1977) analysed the typical pavement structures in the State of Victoria, Australia, in order to arrive at a delationship between surface curvature index (SCI), as defined by him versus the asphalt tensile strain (ε_{HA}) as shown in Figure 6.6. All these pavement structures have asphalt concrete layers of 50 mm or less. In this figure values are recorded for strains up to 2×10^{-4} (inch/inch). Anderson (1977) concludes that this function can be used to estimate asphalt tensile strains in an overlay design method. In general these relationships have been determined for a granular based, three-layered pavement structure. Monismith and Markevich (1983) expanded this work by analysing 300 different cases of this typical pavement structure. Their resulting relationship was:

$$\varepsilon_t = 1{,}794 \times 10^{-2} (D_o - D_{12})^{0{,}828}$$

where:

ε_t = maximum tensile strain

$D_o - D_{12}$ = curvature function or surface curvature index (SCI) (see Table 1.1)

In essence, this is also what was done by Grant and Walker (1972) as shown in Figure 6.4. In this case, though, a specific three-layered pavement structure was analysed (granular base) and the radius of curvature (R) was correlated with strain (ε_{HA}).

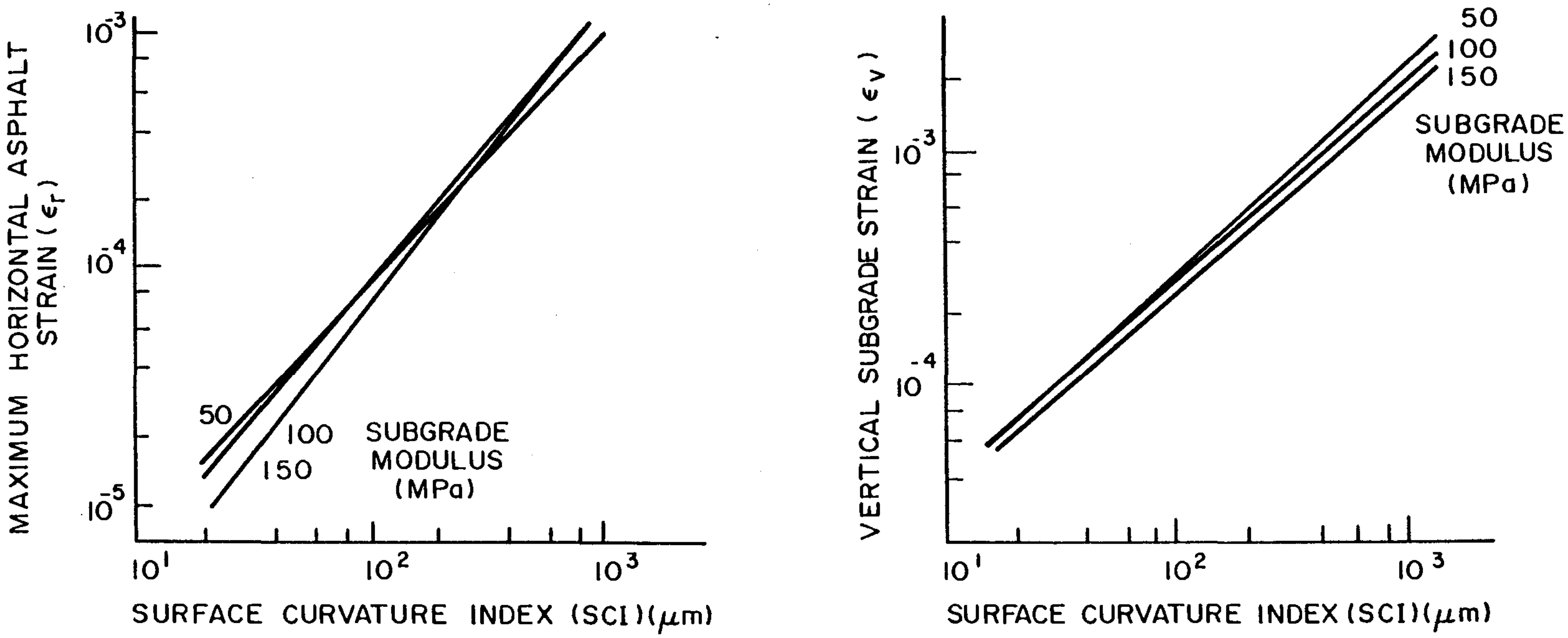


FIGURE 6.5
RELATIONS BETWEEN SURFACE CURVATURE INDEX AND HORIZONTAL ASPHALT STRAIN OR VERTICAL SUBGRADE STRAIN. (Molenaar, 1983)

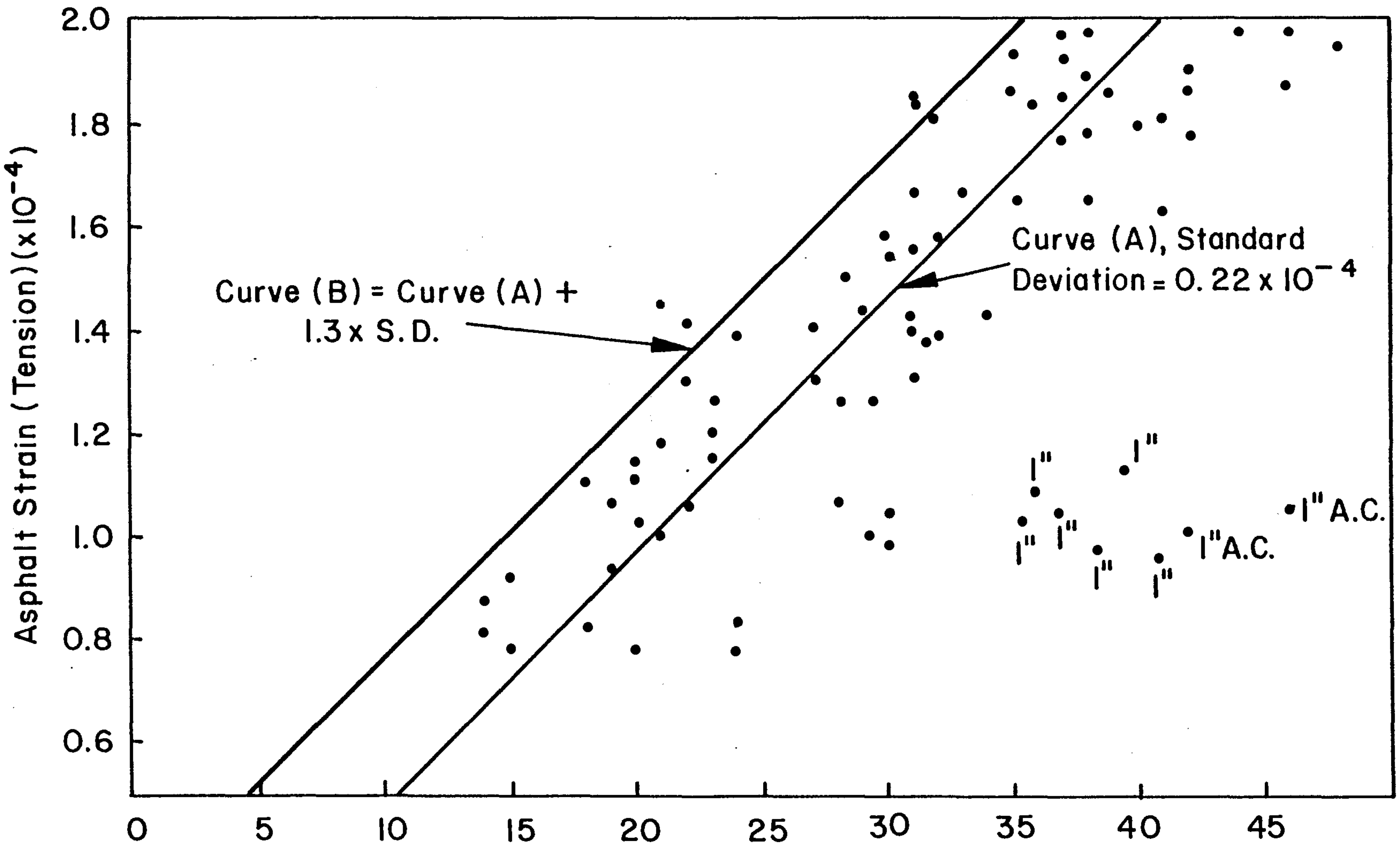


FIGURE 6.6

SURFACE CURVATURE INDEX vs ASPHALT TENSILE STRAIN

(Anderson, 1977)

Kilareski, et al 1982) conclude from their analysis that the deflection basin parameters surface curvature index (SCI) and the base damage index (BDI) give better defined relationships with ε_{HA} than the parameter base curvature index (BCI). The function of surface curvature index (SCI) versus maximum asphalt strain (ε_{HA}), as shown in Figure 6.7, is the preferred relationship. In this Figure it is also indicated that the relationship is not linear, but at low strain levels a linear relationship may be used.

3 RUTTING

Permanent deformation of the individual layers resulting in permanent deformation of the total pavement structure is a major distress state to be considered in overlay design. The nature and classification of rutting in asphalt concrete pavements are discussed in Appendix B. As mentioned there, the distress determinant most often used is vertical subgrade strain (ε_{vs}). As was noted this approach is more historical in the sense that it does not focus on the prevention of deformation, but on the results thereof.

The emphasis in this section is on the analysis and prevention of rutting. The basic relationship used as a first approach is thus restated as follows:

$$N = k\left(\frac{1}{\varepsilon_{vs}}\right)^n$$

where

N = number of applications to failure

k, n = material coefficients

ε_{vs} = vertical subgrade strain as determined by, for example multi-layer linear elastic computer materials.

Various researchers have analysed different pavement structures to arrive at regression functions of ε_{vs} and the deflection basin parameters. Such typical relationships are shown in Figure 6.8. Anderson (1977) established a relationship between maximum deflection (δ_0) and vertical subgrade strain (ε_{vs}). This is shown in

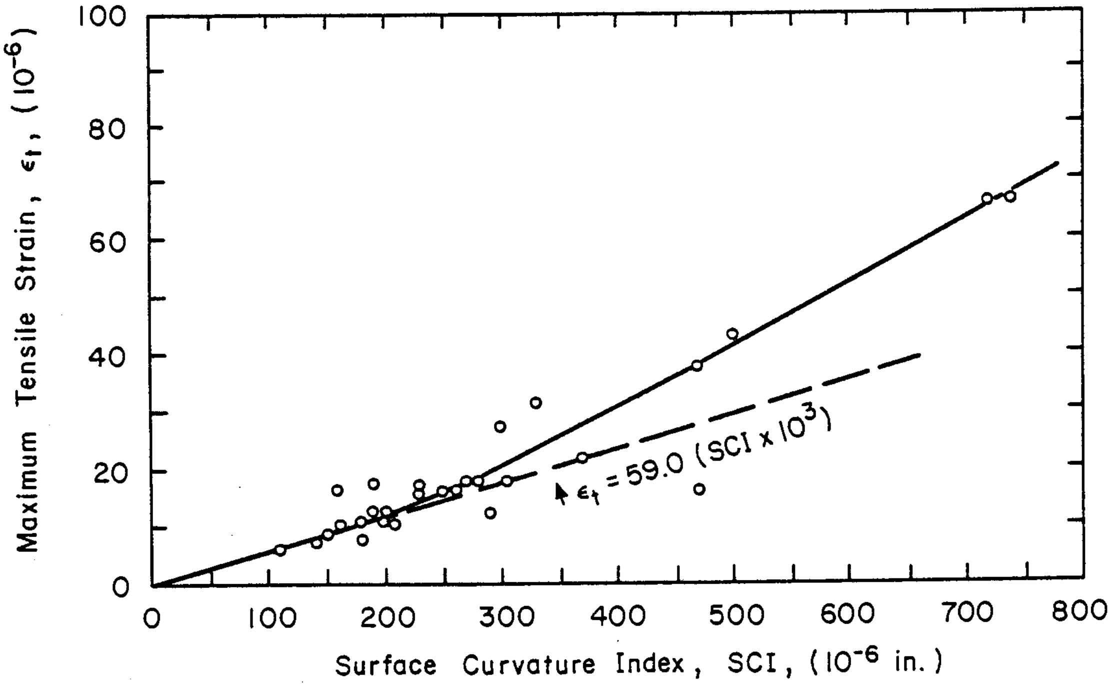


FIGURE 6.7
RELATIONSHIP BETWEEN MAXIMUM TENSILE STRAIN AND SURFACE CURVATURE INDEX.
(Kilareski, et al., 1982)

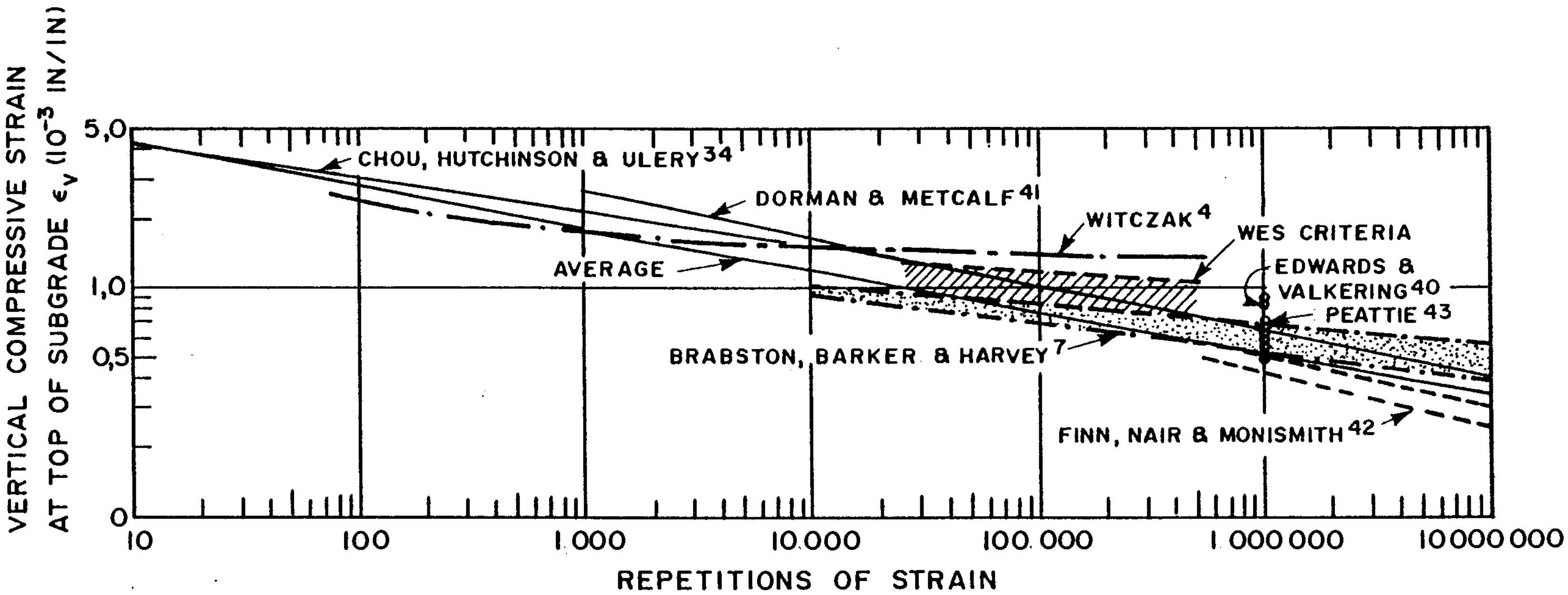


FIGURE 6.8
COMPARISON OF SUBGRADE STRAIN CRITERIA
(1 in. = 2,54 cm (Yoder and Witzak , 1975)

Figure 6.9 with two possible curves fitting the results of granular based three layered pavements analysed. Monismith and Markevich (1983) also used δ_0 as the deflection basin parameter in their analysis of similar pavement structures. That relationship is:

$$\varepsilon_{vs} = 6{,}3\times10^{-4}\ (\delta_0)^2(E_s)^{0{,}8}$$

where:

δ_0 is measured in inches

E_s = subgrade modulus expressed in psi.

A high correlation coefficient was established, for the pavement structures analysed, allowing confidence in this method of estimating vertical subgrade strain (ε_{vs}) for this type of pavement structure.

In the light of what was discussed in chapter 4 it is clear that maximum deflection (δ_0) is not necessarily the most ideal deflection basin parameter to correlate with vertical subgrade strain (ε_{vs}). Wang et al. (1978) established a relationship incorporating maximum deflection (δ_0) and base damage index BCI for measurements with the Road Rater Model 400. This is shown in Figure 6.10.

On the use of vertical subgrade strain (ε_{vs}) as the distress determinant, Molenaar (1983) notes: "Although these simple subgrade strain criteria are very easy to use and therefore very attractive, they are nevertheless thought to cover only a part of the permanent deformation of the pavement structure because each layer can exhibit some permanent deformation itself". In Figure 6.5 it was shown how he relates subgrade strain to surface curvature index (SCI) as he did for maximum asphalt strain. This should be kept in mind, since Grant and Curtayne (1982) also indicated that owing to densification of some layers the pavement is often structurally stronger at the time of overlay analysis. This has been repeatedly confirmed by HVS tests.

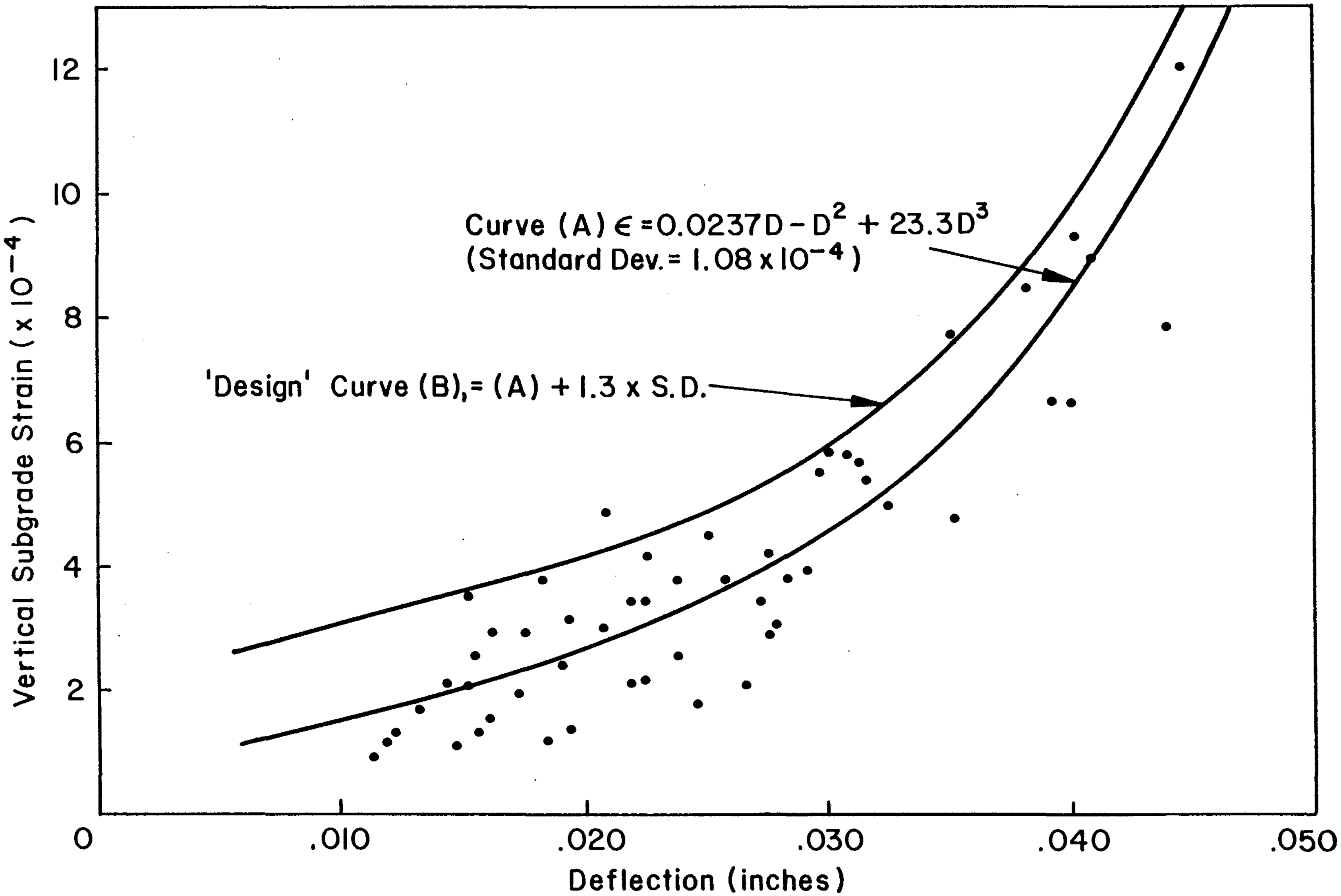


FIGURE 6.9

DEFLECTION vs SUBGRADE STRAIN (Anderson, 1977)

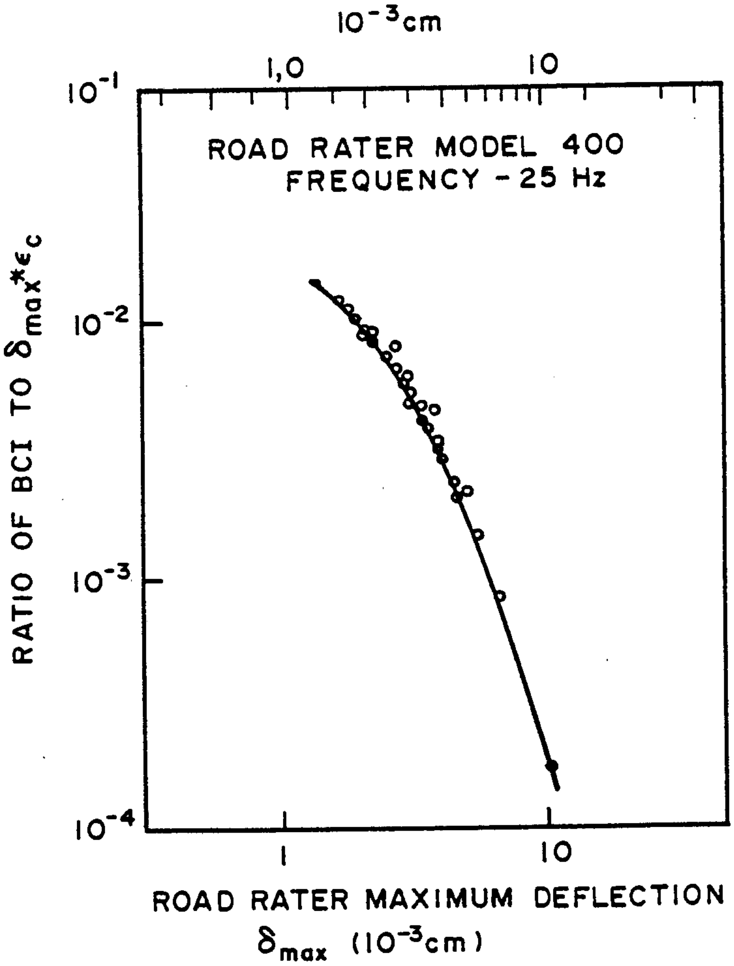


FIGURE 6.10

RELATIONSHIP BETWEEN BASE CURVATURE INDEX, MAXIMUM DEFLECTION AND SUBGRADE COMPRESSIVE STRAIN
(Wang, et al., 1978)

In an effort to assess the accumulated permanent deformation in all the pavement layers, Treybig et al. (1978) calculate the following stresses and strains:

(a) vertical strain at the bottom of the top layer
(b) vertical stress at the bottom of the top layer
(c) vertical stress at the bottom of the second layer
(d) horizontal stress, parallel to the load axle, at the bottom of the second layer
(e) vertical stress at the bottom of the third layer
(f) vertical strain at the bottom of the fourth layer
(g) vertical stress at the top of the fifth layer
(h) vertical strain at the top of the fifth subgrade layer.

These stresses and strains are the largest values, irrespective of their horizontal position under the loaded wheels. The values are correlated to the number of repetitions (N). These correlated functions are derived from the analysis of pavement structures' bound and unbound bases.

The deformation in the bitumen or asphalt concrete layer was referred to in Appendix B. The Shell method of estimating deformation in this layer, as described by Van de Loo (1976) is well established. Work by Viljoen and Meadows (1981) on the creep test provides practical guidelines for South African conditions on the prevention of permanent deformation in the asphalt concrete layer.

4 CONCLUSIONS AND RECOMMENDATIONS

In conclusion it may be stated that the maximum tensile strain in the asphalt layer (ε_{HA}) is the distress determinant that is most associated with fatigue cracking due to the ease of calculation. Various research efforts have been reported that characterize and explain the actual cracking behaviour.

It is concluded that, in the South African mechanistic design procedure, the method currently used to determine maximum tensile strain (ε_{HA}) at the bottom of the asphalt layer is acceptable. Regression analyses relating ε_{HA} to the desired deflection basin parameters indicate that this is an acceptable method to be used in overlay design. The majority of analyses though, were done for a basic granular-based three-layered pavement structure. In the light of the previously proposed pavement categories and the behaviour states, (see chapter 4), similar analyses should be done in order to establish possible relationships, for example on bitumen-based and cement-treated base pavements. This will be possible with the available information from the recorded HVS tests and other field observations. The interactions of cement-treated layers and cracks reflected through the upper layers and asphalt concrete layers are well recorded and should also be analysed in this way as they are the cause of cracking in a high percentage of pavements in South Africa.

It can be stated that vertical subgrade strain (ε_{vs}) is the distress determinant most often calculated in order to predict rutting. This is the practice in South Africa and multi-layer linear elastic computer programs are usually used to calculate ε_{vs}. Various deflection basin parameters have been related to ε_{vs} by regression analysis in order to estimate ε_{vs} directly from deflection basin measurements. A clear distinction is not always made in regard to the pavement type analysed, although the typical three-layered granular base pavement prevails. As was suggested in the previous section, analyses should be done with the available South African data in order to classify or do regression analyses for each of the aforementioned pavement classes or behaviour states.

From this discussion, HVS tests results and field results it is clear that all the layers contribute to the total permanent deformation. Various other distress determinants are calculated by methods incorporating the stress and strain states in the various layers. It is suggested that on this line, regression analysis be done too. This in turn should be related to the various pavement categories.

CHAPTER 7

RELATIONSHIPS BETWEEN DEFLECTION BASIN PARAMETERS AND DISTRESS DETERMINANTS FOR TYPICAL SOUTH AFRICAN PAVEMENTS

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1. INTRODUCTION

In the analysis of flexible pavements for rehabilitation design the mechanistic rehabilitation design procedure can be used with confidence (Freeme, 1983 and Freeme et al, 1982). It has been explained in chapter 4 how proper material characterization is an important step in this analysis procedure. The normal linear elastic computer programmes are used in such an analysis in order to calculate the distress determinants and relate them to structural life. In chapter 4 and Appendix C and D it was explained how this non-simplified or fundamental approach (Jordaan, 1986) should function.

In the previous chapter it was shown how it is possible to develop design curves for specific pavement types whereby deflection basin parameters are related to distress determinants. In this chapter it will be shown how this author analysed typical South African flexible pavements in order to relate selected pavement deflection basin parameters to the distress derminants. The broad spectrum of flexible pavements is properly defined and the analysis is further restricted to bitumen, granular and cemented base pavements. The latter type is analysed in the equivalent granular state which means that their analysis is similar to that of the granular base pavements. They have a further common factor in the asphalt surfacing (40 mm average or less).

In the discussion reference is also made to the findings of overseas researchers. Overlaying flexible pavements is an important rehabilitation alternative. For that reason the effect of various overlay thicknesses were also investigated in relation to the effect on the deflection basin parameters.

2. ANALYSIS OF FLEXIBLE PAVEMENT STRUCTURES

The pavement structures in the Catalogue of designs (NITRR, 1985a) reflect the typical pavement structures used in South Africa. The pavement types can be subdivided into pavements having granular, bituminous, cemented and concrete bases. Typical granular base pavement structures analysed, are shown in Figure 7.1. Only the subbase, base and surfacing layers are shown; the selected layer and subgrade are excluded. The latter two layers are, however, prepared in the standard prescribed way (NITRR, 1985a) and are common to all these pavement structures.

The different pavement types behave differently. Their behaviour is time-dependent; generally pavements become more flexible with time. This necessitates the proper definition of the current state of the pavement in order to determine the most appropriate analysis or design method of predicting future behaviour. The definition of the states of pavement behaviour is given by Freeme (1983); these states range from very stiff to very flexible (See chapter 3 and 4). A flexible pavement is defined as a pavement structure of which the behaviour is controlled by material in the granular state. In chapter 3 it was shown how the various deflection basin parameters can enhance such accurate definition of behaviour state.

The analysis of flexible pavements therefore includes granular, bitumen and cemented base pavements. Pavements with cemented bases or subbases were considered to be in the cracked phase, exhibiting equivalent granular behaviour (Freeme, 1983). All such standard TRH4 (NITRR, 1985a) flexible pavements were analysed first by means of the multi-layered linear elastic program, MECD3 (Maree and Freeme, 1981a). A second level of analysis was done on granular base pavements or bases in the equivalent granular state with the computer program ELSYM5 (University of California, 1972). The ELSYM 5 program (University of California, 1972) tends to be inaccurate in calculating surface deflections between 100 mm and 250 mm from the point of maximum deflection. This is true for bituminous base pavements (Tam, 1985). This tendency is more pronounced when five-layered pavement structures are analysed and when the pavement structure is more rigid, for example a concrete base pavement (Taute, McCullough and Hudson, 1981). For that reason the detailed analysis of the bituminous base pavements were done with the BISAR program (Horak, 1985).

In the analysis of the typical flexible pavement structures of the

GRANULAR BASES

ROAD CATEGORY	DESIGN TRAFFIC CLASS E80/LANE OVER STRUCTURAL DESIGN PERIOD				
	E0 $<0{,}2 \times 10^6$	E1 $0{,}2-0{,}8 \times 10^6$	E2 $0{,}8-3 \times 10^6$	E3 $3-12 \times 10^6$	E4 $12-50 \times 10^6$
A			30-40A† 125 G1 150 C3	40A† 150 G1 125 C3 125 C4	50A† 150 G1 150 C3 150 C3
B		S† 125 G2 150 C4 S† 150 G2 150 G5	30A† OR S† 125 G1 150 C4	40A† 150 G1 250 C4	
C	S† 100 G4 100 C4 S† 100 G4 125 G5	S† 100 G2 125 C4 S† 100 G2 150 G5	S† 125 G2 150 C4 S† 150 G2 150 G5		

†SYMBOL A DENOTES AG, AC OR AS. SYMBOL S DENOTES S2 OR S4

FIGURE 7.1
Typical granular base pavements from the Catalogue of designs

Catalogue of designs (NITRR, 1985a), surface deflections were calculated at various offsets perpendicular to the axle of the wheels.

The structural integrity of a pavement is reflected in varying degrees of accuracy by various deflection basin parameters. The deflection basin parameters that were investigated in this study are listed in Table 1.1 (for ease of reference use the fold out at the back). The formulae and measuring devices normally associated with them are also listed (Horak, 1984). The deflection parameters vary from the traditional singular point, maximum deflection, to deflection basin parameters such as spreadability and area, which are highly descriptive of the deflection basin as a whole. In the ELSYM5 analysis of the granular base pavements, deflection basin pavements calculated in this region carried less weight in the analysis.

In all of these analyses the wheel load used was a 40 kN dual wheel load with a standard tyre pressure of 520 kPa. A Poisson ratio of 0,44 was assumed for bituminous materials and 0,35 for all other types of material. The elastic moduli values of the materials used in the MECD3 analysis were the average values as defined and verified with HVS test results (Freeme et al., 1982a and Freeme, 1983). The value for thin asphalt surfacings (mostly granular base pavements) was set at 2 000 MPa. The selected subgrade's elastic modulus was set at 120 MPa and the subgrade modulus was varied with 50,70 and 150 MPa.

In the ELSYM5 analysis of granular base pavements (and equivalent granular states) the base elastic modulus and thickness were identified as the most influential structural components in terms of the deflection basin parameters (Tam, 1985). These values were varied over a range commonly found in practice (Freeme, 1983). The base thicknesses were varied in steps of 25 mm between 100 and 250 mm. The effective elastic modulus of the granular base was varied in steps of 15 MPa between 100 and 450 MPa. Also, in order to determine the effect of overlays on granular base pavements, the surfacing thickness was increased in steps of 10 mm (up to 100mm thick) over a basic surfacing thickness of either 20 mm or 40 mm.

In the detail analysis (Horak, 1985) of bitumen base pavements the BISAR computer program was used. In Table 7.1 the various structural components are listed.

TABLE 7.1 - BISAR analysis of bitumen base pavements

Parameter	Range
Overlay elastic modulus (MPa)	3 000*
Overlay thickness (mm)	5, 10, 40, 60
Surfacing elastic modulus (MPa)	2 000, 4 000
Surfacing thickness (mm)	10, 40
Base elastic modulus (MPa)	3 000, 5 000, 7 000
Base thickness (mm)	80, 150, 220
Subbase elastic modulus (MPa)	150, 500
Subbase thickness (mm)	300*
Selected subgrade modulus (MPa)	120*
Selected subgrade thickness (mm)	150*
Subgrade elastic modulus (MPa)	50, 70, 150

* These values were typical representative values and their variance has a minor influence (Tam, 1985) on deflection basins.

3. DEFLECTION BASIN PARAMETERS VERSUS DISTRESS DETERMINANTS

3.1 General

The deflection basin parameters listed in Table 1.1 (see the fold out for ease of reference) were calculated for each flexible pavement structure in these analyses. As pointed out in the introduction the intention is to determine which parameters can be related to the typical distress determinants (ε_{HA} and ε_{VS}) accurately. Only those deflection basin parameters that have a clear and meaningful relationship with the distress determinants are reported here.

3.2 Vertical subgrade strain

The calculated vertical subgrade strain (ε_{VS}) was correlated with the various deflection basin parameters of bitumen bases and granular bases separately. This relationship is not possible for flexibile pavements in general. The reason for this is shown in Figure 7.2. Both base types show a relationship between SCI and ε_{VS} and discern the variance in subgrade elastic modulus. As in the relationships between equivalent layer thickness (H_e) and SCI (see Appendix F), there is a difference in gradient between the relationships for bitumen and granular base pavements. Additionally, in the case of the granular base pavements these relationships shown in Figure 7.2, are only valid for average values of elastic moduli and layer thickness of the base. In Figure 7.3 the relationship between BDI and ε_{VS} for granular bases, shows that no discernment is made between subgrade elastic moduli being varied. This is however also true only for average values of base elastic moduli and layer thicknesses. In Figure 7.4 the relationships between R, F1, Q and SD versus ε_{VS} are shown for bitumen base pavements. It seems that these relationships are not influenced by the variance in the structural input values, as granular base pavements seem to show a tendency for. Of all these ralationships shown in Figure 7.4 the deflection basin parameter SD has the most significant relationship.

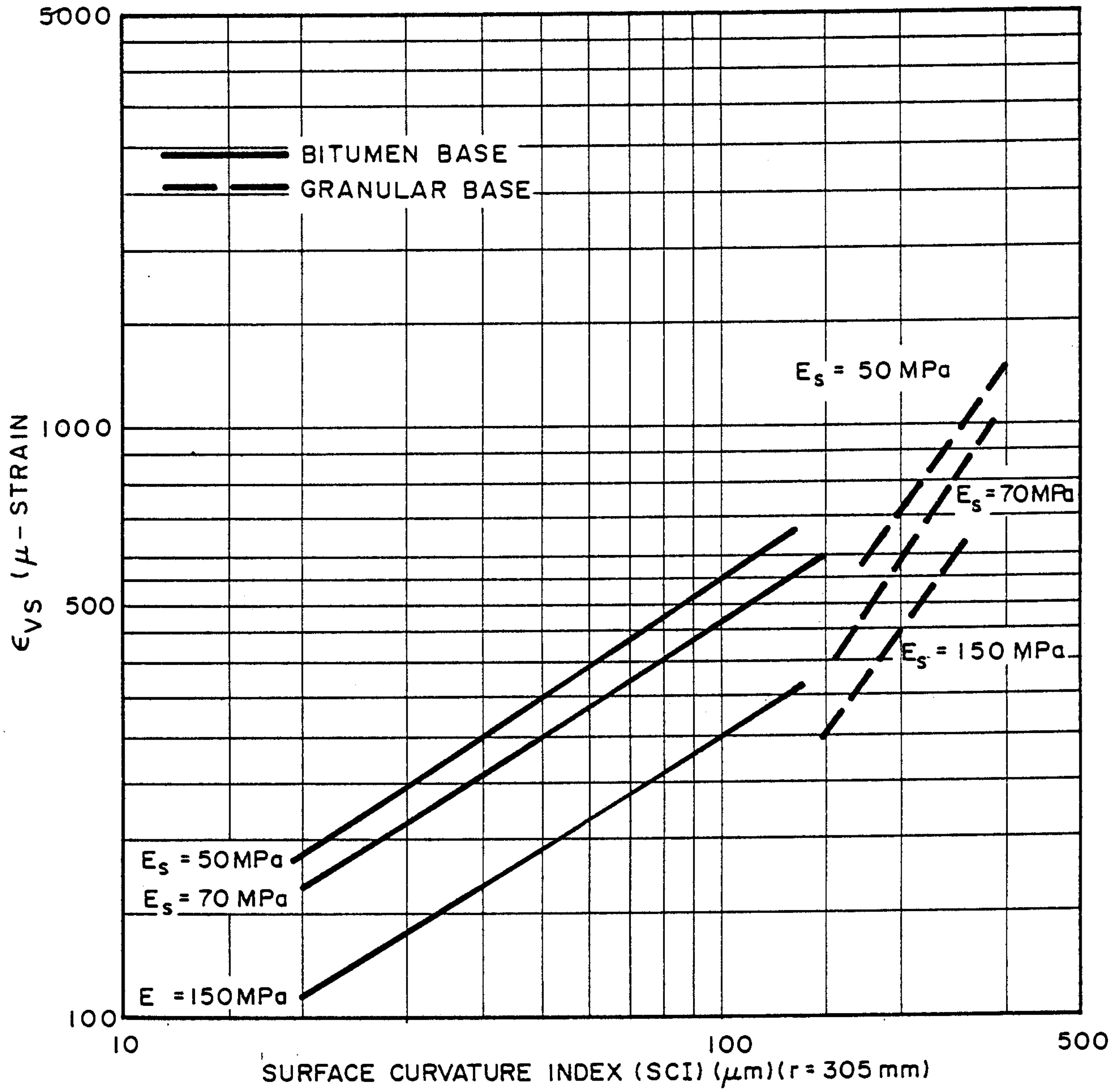

FIGURE 7.2
Surface curvature index versus vertical subgrade strain

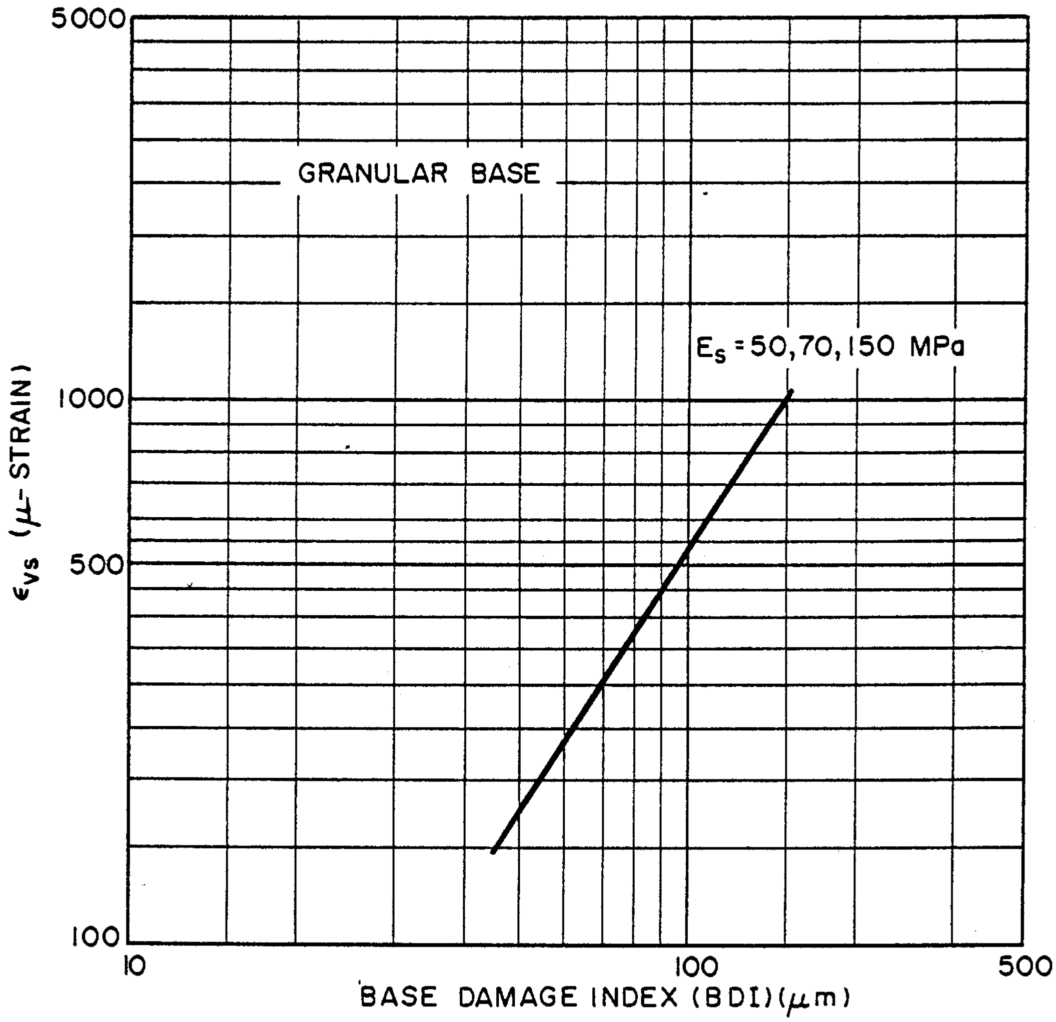

FIGURE 7.3
BASE DAMAGE INDEX VERSUS VERTICAL SUBGRADE STRAIN

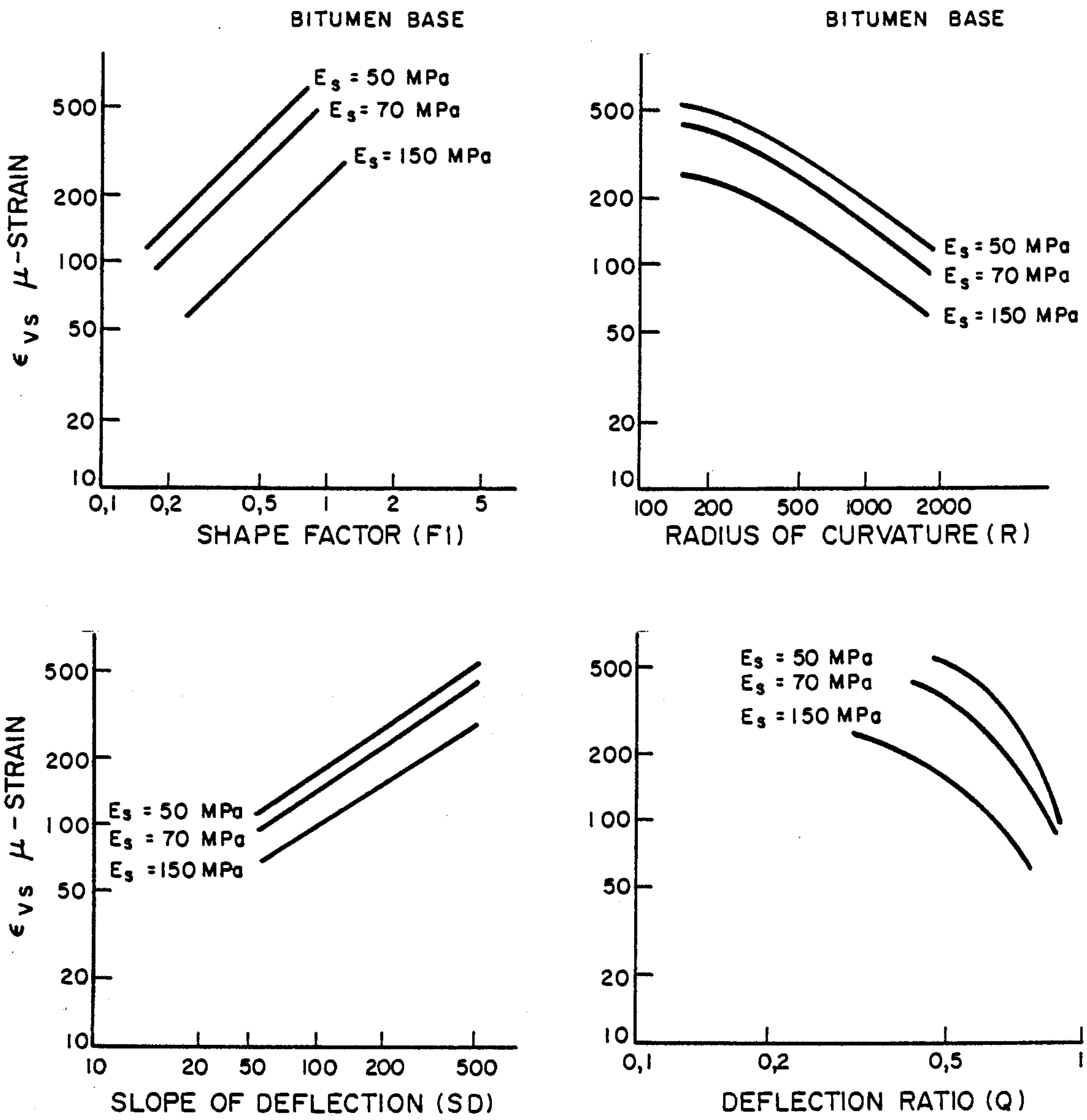

FIGURE 7.4
BITUMEN BASE DEFLECTION BASIN PARAMETERS VERSUS VERTICAL SUBGRADE STRAIN

3.3 Maximum asphalt strain

The calculated maximum asphalt strains, (ε_{HA}, at the bottom of the base) of typical bitumen base pavements were related to the various deflection basin parameters separately. In Figure 7.5 it can be seen that a typical value such as SCI (r = 500 mm) correlated well with ε_{HA}, but, as Molenaar and Van Gurp (1980) indicated, there is no clear discernment between the various subgrade elastic moduli. In Figure 7.6, deflection basin parameters S, A, F1 and BCI proved to be better in that respect by discerning the variance in subgrade elastic modulus. In this figure the relationship with BCI is the most significant and usefull. When the maximum asphalt strains (ε_{HA}, under the surfacing) of bitumen base pavements were related to deflection basin parameters, clear, simple relationships became limited to those as shown in Figure 7.7. Area (A) and spreadability (S) can discern the effect of variance in subgrade elastic modulus, but both seem to be rather insensitive to changes in their respective values. As the ε_{HA} values are compressive, the significance for fatigue calculation of the surfacing is further nullified.

The thickness of the asphalt surfacing layer of a typical granular base pavement has a marked effect on the calculated value of the maximum asphalt strain (ε_{HA}). Grant and Walker (1972) showed that for three-layered pavement structures there is a sharp increase in maximum asphalt strain (ε_{HA}) values when surfacing thickness is increased to 25 mm . Thereafter the strain increased slowly to reach a maximum strain value between surfaces with 40 and 100 mm thickness. In Figure 7.8 this effect is verified when radius of curvature (R) is correlated with ε_{HA} for various thicknesses of the surfacing. The subgrade elastic modulus was kept at a standard 70 MPa, but the base elastic modulus and thickness were varied for the granular base pavements. For bitumen base pavements the subgrade elastic modulus was varied though. Figure 7.8 clearly shows that for a normal granular base there is a definite difference in the relationships between R and ε_{HA} for various thicknesses of asphalt surfacing. At a thickness of 80 mm and above, such a pavement is according to definition (Freeme et

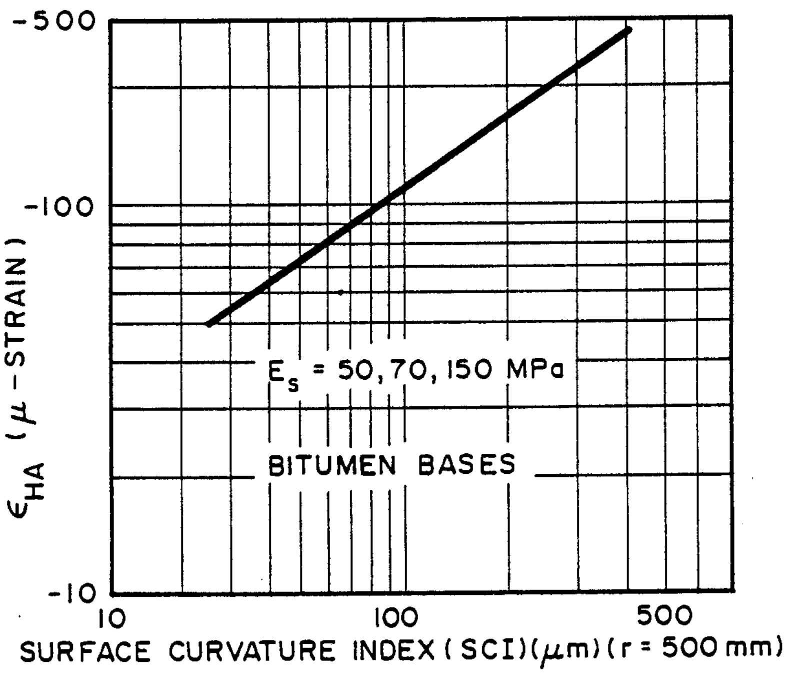

FIGURE 7.5
Surface curvature index versus maximum asphalt strain for bituminous base pavement structures

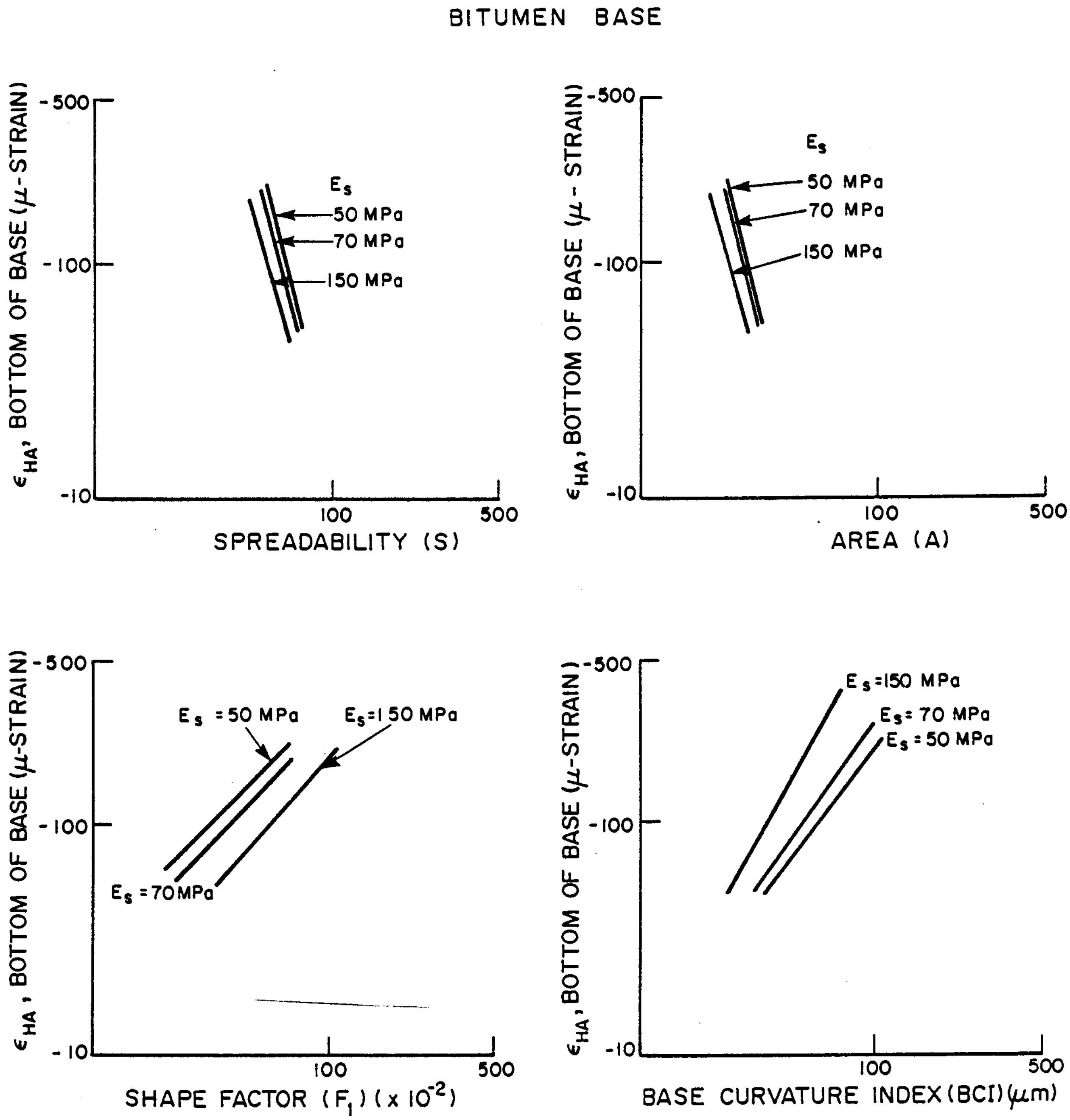

FIGURE 7.6

BITUMEN BASE DEFLECTION BASIN PARAMETERS VERSUS MAXIMUM ASPHALT STRAIN UNDER THE BASE

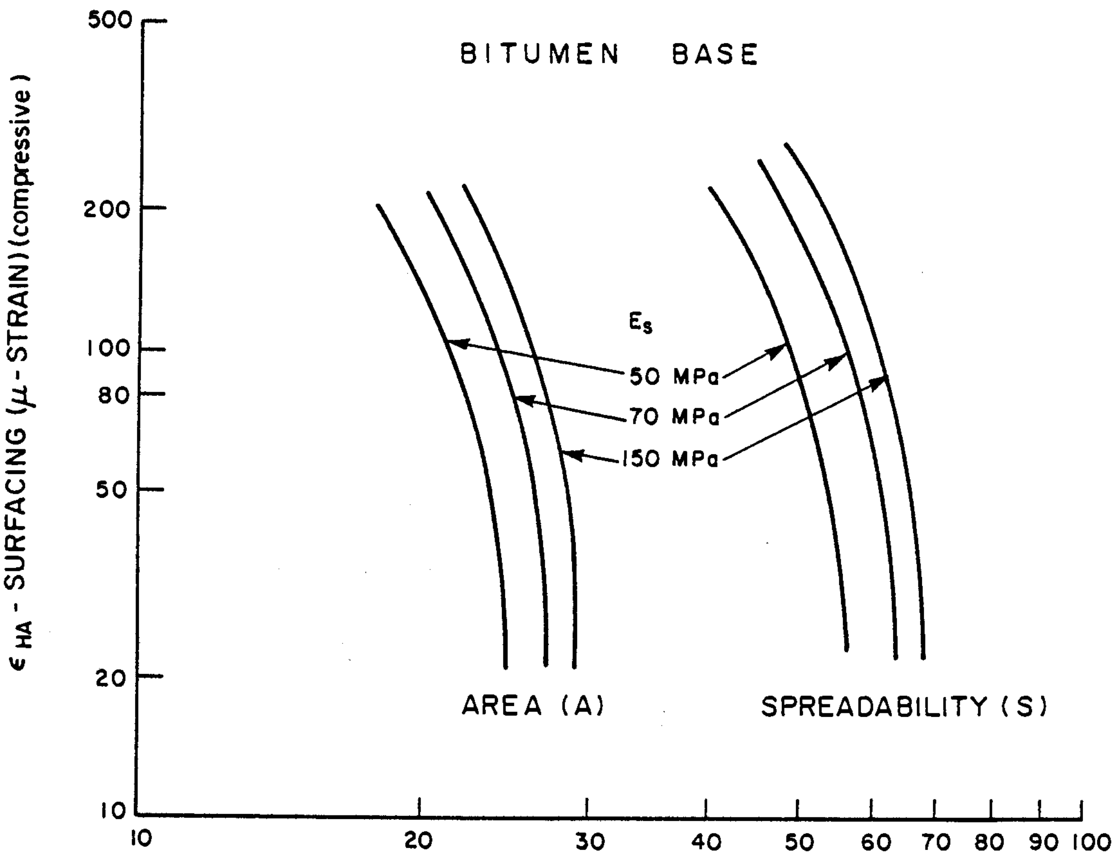

FIGURE 7.7

MAXIMUM ASPHALT STRAIN UNDER SURFACING VERSUS AREA AND SPREADABILITY

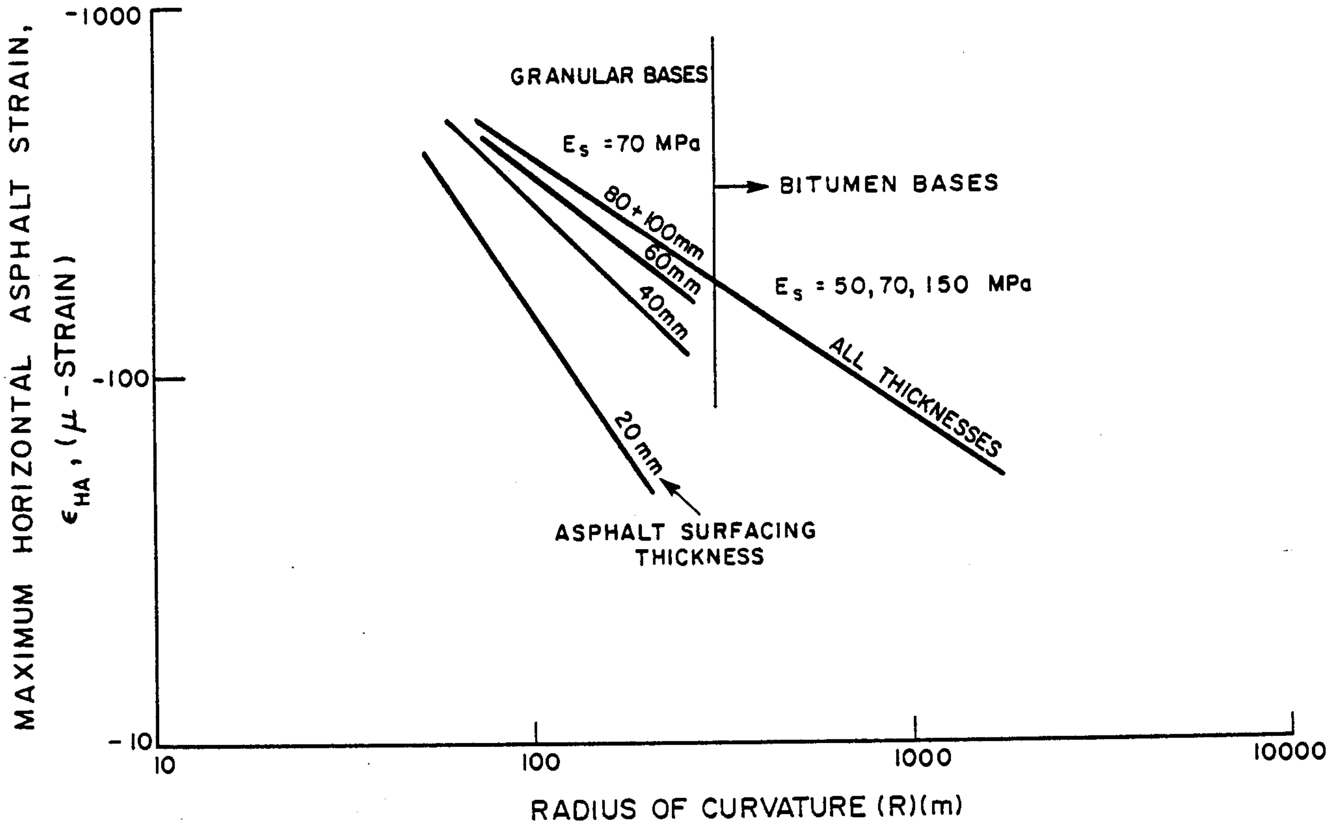

FIGURE 7.8

RADIUS OF CURVATURE VERSUS MAXIMUM ASPHALT STRAIN al., 1982a) and (NITRR, 1985a) rather classified as a bitumen base pavement. Proof of the latter is that this relationship (80 mm and 100 mm surfacing) extrapolates to the relationship established for bitumen base pavements. In Figure 7.9 it can be seen that other deflection basin parameters such as S, SCI, F1 and Q tend to group surfacing thicknesses of 40 mm to 100 mm together, while those with thicknesses of up to 20 mm is apart. This reaffirms the observation by Grant and Walker (1972) discussed earlier. Of these relationships that with SCI can be termed the most significant.

4 RELATIONSHIPS WITH OTHER STRUCTURAL PARAMETERS

4.1 General

It is clear from the results of the previous section, that it is not always possible to form clear, simple relationships between distress determinants and deflection basin parameters. The reason for this seems to be the conflicting influences that other structural input parameters being varied, have on the calculated distress determinants. This is more pronounced in the case of the granular base pavements than with bitumen base pavements. The literature survey in the previous chapter and chapter 4 already indicated there are peculiarities with the linear elastic layered programmes used (in the vicinity of the loaded wheel).

4.2 Granular base pavements

In the case of granular bases in particular the variance of elastic modulus of the base and asphalt surfacing thickness had a significant effect on the calculated deflection basin parameters. The effect of the variance of the elastic modulus of the granular base is best illustrated in Figure 7.10 where SCI, BCI, Q and R are related to the base elastic modulus. Surfacing thickness (H_{SF}) is however not being discerned by parameters BCI and Q.

A very useful relationship was developed between maximum horizontal asphalt strain (ε_{HA}) and easily measurable parameters

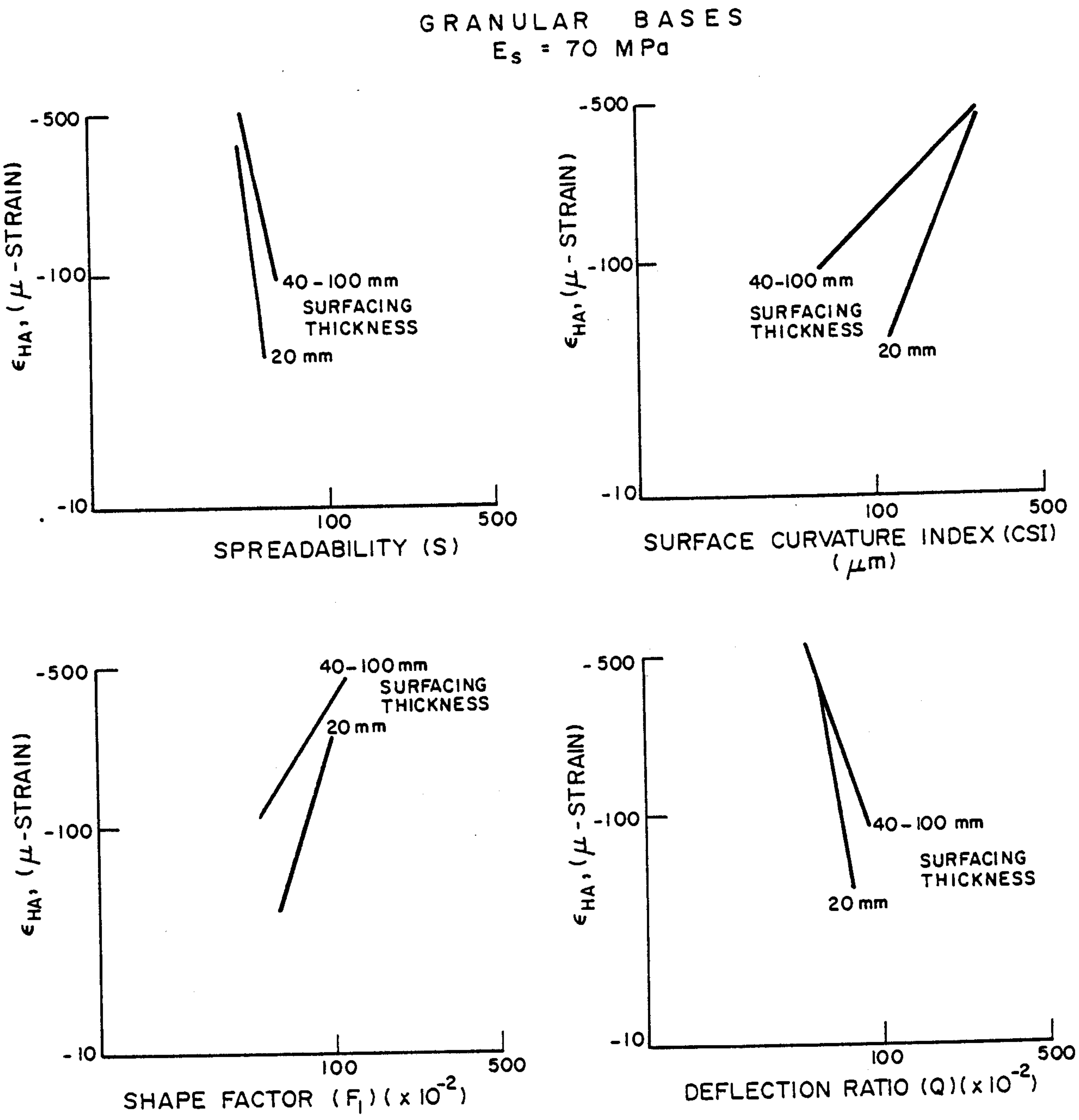

FIGURE 7.9

GRANULAR BASE DEFLECTION BASIN PARAMETERS VERSUS MAXIMUM ASPHALT STRAIN

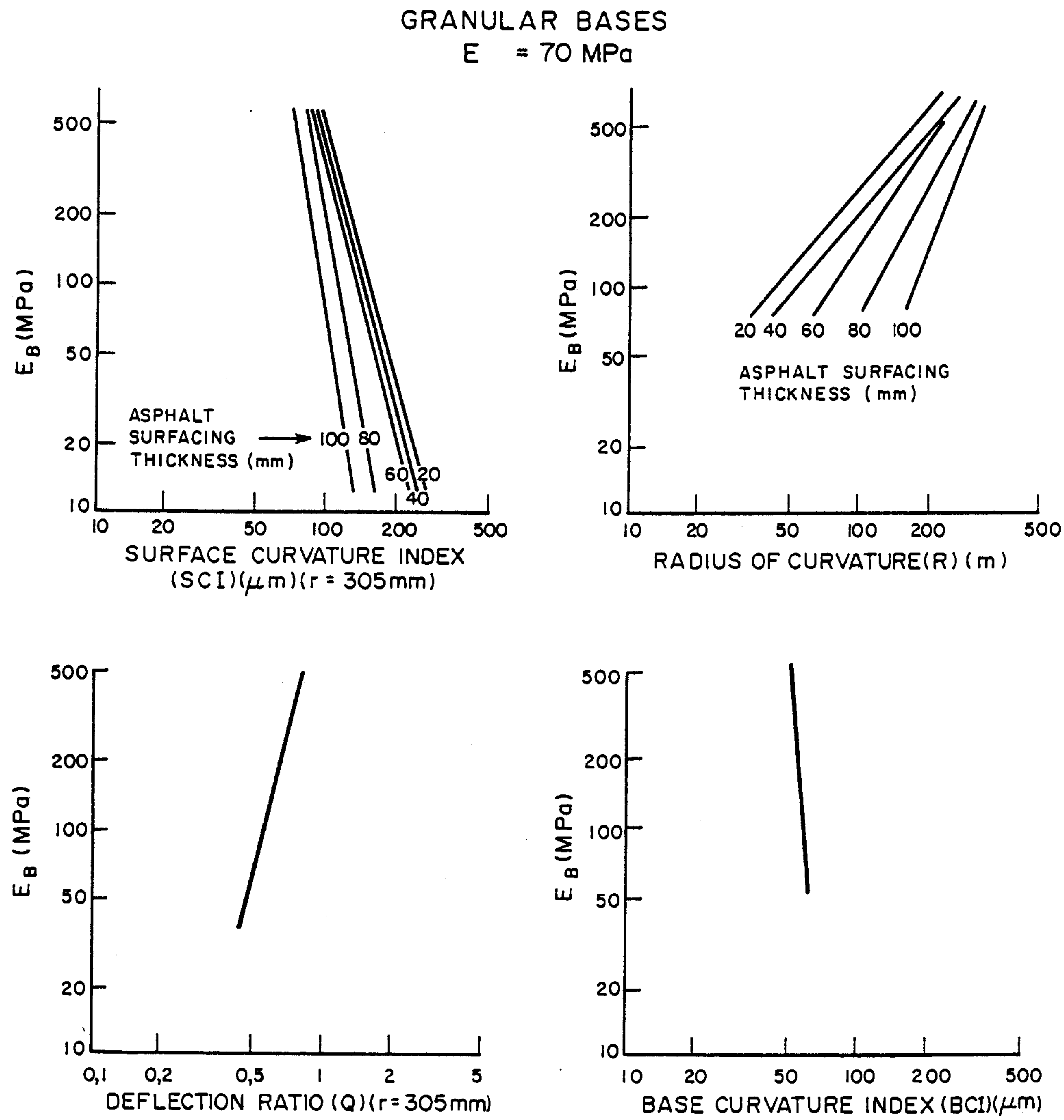

FIGURE 7.10

GRANULAR BASE ELASTIC MODULUS VERSUS DEFLECTION BASIN PARAMETERS for granular base pavements, by means of stepwise multiple regression. The relationship is as follows:

$$\varepsilon_{HA} = 1360{,}03\ F_1 + 4{,}54\ H_{SF} - 2{,}10\ R + 3806{,}29\ Q \quad -2563{,}82$$

where: ε_{HA} = maximum asphalt strain (μ-strain)

F_1 = shape factor

H_{SF} = asphalt surfacing thickness (mm)

R = radius of curvature (m)

Q = deflection ration (r=610 mm)

This relationship has a R-square value of 92,4 per cent and a coefficient of variability of 9,7 per cent.

Maree et al. (1982a) have shown that for granular base pavements, permanent deformation originates mostly within the base itself. In order to limit or indicate the potential for permanent deformation in a granular layer, Maree (1978) suggested the use of a safety factor,

$$f = \frac{k\{\sigma_3 \tan^2(45 \quad \emptyset/2)-1 \quad 2C \tan(45 + \emptyset/2)\}}{\sigma_1 - \sigma_3}$$

where C = Apparent cohesion

$\emptyset$ = angle of internal friction

σ_1; σ_3 = major and minor principle stresses

K = constant, depending on the moisture condition.

The safety factor will therefore vary depending on the typical shear parameters (C and $\emptyset$) and the moisture condition. Typical shear parameter values and moisture conditions (wet or dry) as suggested by Freeme (1983) were used to calculate safety factors for the wet condition (F wet) and for the dry condition (F dry). These calculations were done for typical granular base material (G1 to G4). Additionally the deviator stress (σd) for each situation was calculated too.

The safety factors (F wet and F dry) and the deviator stress (σd) were correlated with the calculated deflection base parameters by means of a stepwise multiple regression analysis. The number of variables in the final equation was limited to three or the R-squared value had to exceed 0,75. The results of these regression analysis are shown in Table 7.2.

The deflection basin parameters that correlated the best with F wet and F dry, were the shape factor F2 and the index parameter BCI. The deviator stress σd correlated very well with the surfacing thickness and the index parameter BCI. The other deflection basin parameters that occurred in the relationships were; F1, Q_{610}, SD, BDI and S. (See Table 1.1 for definitions.)

The significance of these relationships are that for a granular base type, the potential for permanent deformation can be determined by calculating either F wet, F dry or σd from deflection basin parameters.

4.3 Bitumen base pavements

The same type of relationship was developed for bitumen base pavements by means of a stepwise multiple regression analysis. The relationship before overlay is as follows:

TABLE 7.2 REGRESSION FUNCTIONS FOR GRANULAR BASES

MATERIAL CODE	DEPENDENT VARIABLE	REGRESSION FUNCTION	R^2
G1	F_{WET}	$-5{,}80 + 15{,}84\,(F_2) - 45{,}25\,(BCI) + 4{,}85 \times 10^{-3}\,(H_{SF})$	0,77
	F_{DRY}	$-8{,}07 + 28{,}06\,(F_2) - 116{,}30\,(BCI) + 9{,}56 \times 10^{-3}\,(H_{SF})$	0,78
	σ_d	$240{,}73 - 1{,}93\,(H_{SF}) + 3704{,}43\,(F_1) - 197{,}51\,(BCI)$	0,97
G2	F_{WET}	$10{,}04 + 3{,}77\,(F_2) - 19{,}01\,(Q_{610}) - 4{,}00\,(F_1)$	0,77
	F_{DRY}	$-6{,}03 + 13{,}74\,(F_2) + 2{,}48\,(BCI) - 55{,}79\,(SD)$	0,80
	σ_d	$-72{,}58 - 2{,}40\,(H_{SF}) + 6848{,}40\,(BCI)$	0,95
G3	F_{WET}	$-3{,}68 + 6{,}23\,(F_2)$	0,80
	F_{DRY}	$-7{,}41 + 12{,}67\,(F_2)$	0,81
	σ_d	$-43{,}38 - 2{,}55\,(H_{SF}) + 6329{,}04\,(BCI)$	0,95
G4	F_{WET}	$-0{,}14 + 21{,}19\,(BDI) - 3{,}08\,(Q_{610})$	0,77
	F_{DRY}	$-2{,}46 + 8{,}09\,(F_2) - 4{,}50\,(Q_{610})$	0,77
	σ_d	$-1{,}18 - 2{,}45\,(H_{SF}) + 5447{,}25\,(BCI)$	0,84
G1 TO G3	F_{WET}	$-0{,}971 - 0{,}048\,(S) + 38{,}96\,(BDI) + 9{,}76 \times 10^{-3}\,(F_2)$	0,77
	F_{DRY}	$-1{,}78 + 2{,}25 \times 10^{-2}\,(F_2) + 66{,}31\,(BDI) - 0{,}07\,(S)$	0,78
	σ_d	$-85{,}09 + 12703{,}27\,(BCI) - 4003{,}98\,(BDI) - 1{,}16\,(F_2)$	0,81

ELASTIC MODULUS OF SUBGRADE = 70 MPa
DEFLECTION BASIN PARAMETERS AS DEFINED IN TABLE 1.1

$$\log_{10} \varepsilon_{HA} = -0{,}996 (\log_{10} SCI) - 0{,}352 (\log_{10} S) - 0{,}298 (\log_{10} Q) + 2{,}521$$

where: ε_{HA} = maximum (tensile) asphalt strain (μ-strain)

SCI = surface curvature index (mm)

S = spreadability

Q = deflection ration (r=305 mm)

This relationship has an R-square value of 0,99 and a coefficient of variability of 16,5 per cent.

When this relationship is determined for bitumen bases with overlays it looks as follows:

$$\log_{10} \varepsilon_{HA} = -0{,}623 (\log_{10} A) - 0{,}619 (\log_{10} Q) - 0{,}365 (\log_{10} SCI) + 3{,}470$$

where: A = area and the rest are as defined earlier.

This relationship has an R-square value of 0,61 and a coefficient of variability of 80,1 per cent. The effect of the overlay is such that this relationship with only three independent variables reduces the R-square value considerably and increases the coefficient of variability to unacceptable limits and the use thereof is not recommended.

4.4 Subgrade elastic modulus

A relationship between deflection at 2 m and subgrade elastic modulus was derived similar to that derived by Molenaar and Van Gurp (1980). In this case though, it is applicable to all types of flexible pavements with four or five layers. This relationship is:

$$\log E_s = 9{,}727 - 0{,}989 \log \delta_{2000}$$

where:

E_s = subgrade elastic modulus (Pa)

δ_{2000} = deflection at a distance of 2 m from the centre of loading (μm).

This relationship is shown in Figure 7.11 where the subgrade moduli had been correlated with deflections at 500, 610 and 915 mm, too. It is obvious that the gradient of the relationships change from deflection at 500 mm (δ_{500}) to deflection at 915 mm (δ_{915}), but from the latter deflection up to deflection at 2 000 mm ($\delta_{2\ 000}$) the gradient of the relationships stays virtually the same. It is the general feeling (Tam, 1985) that deflections nearer than 500 mm (δ_{500}) to the centre of loading, would increasingly reflect the influence of the other structural layers. nearer to the surface (base and subbase).

5 THE EFFECT OF OVERLAYS

5.1 Granular base pavements

Granular base pavements in the dry regions of South Africa have typical surfacing thicknesses of 30 to 50 mm (40 mm average) thick. Alternatively surface treatments typically have thicknesses of 20 mm for double surface treatments. As shown in Appendix F, these thin asphalt surfacings have relatively little influence on the value of equivalent layer thickness (H_e) and therefore also on vertical subgrade strain (ε_{VS}), as the thickness of the overlay is increased. In the wetter regions of South Africa thicker asphalt surfacings and overlays may be applicable.

The effect of overlay thickness on ε_{VS} is illustrated; in Figure 7.12. The greater the overlay thickness, the more ε_{VS} is reduced. An elastic modulus of 3000 MPa was assumed typical for the asphalt overlay and an elastic modulus of

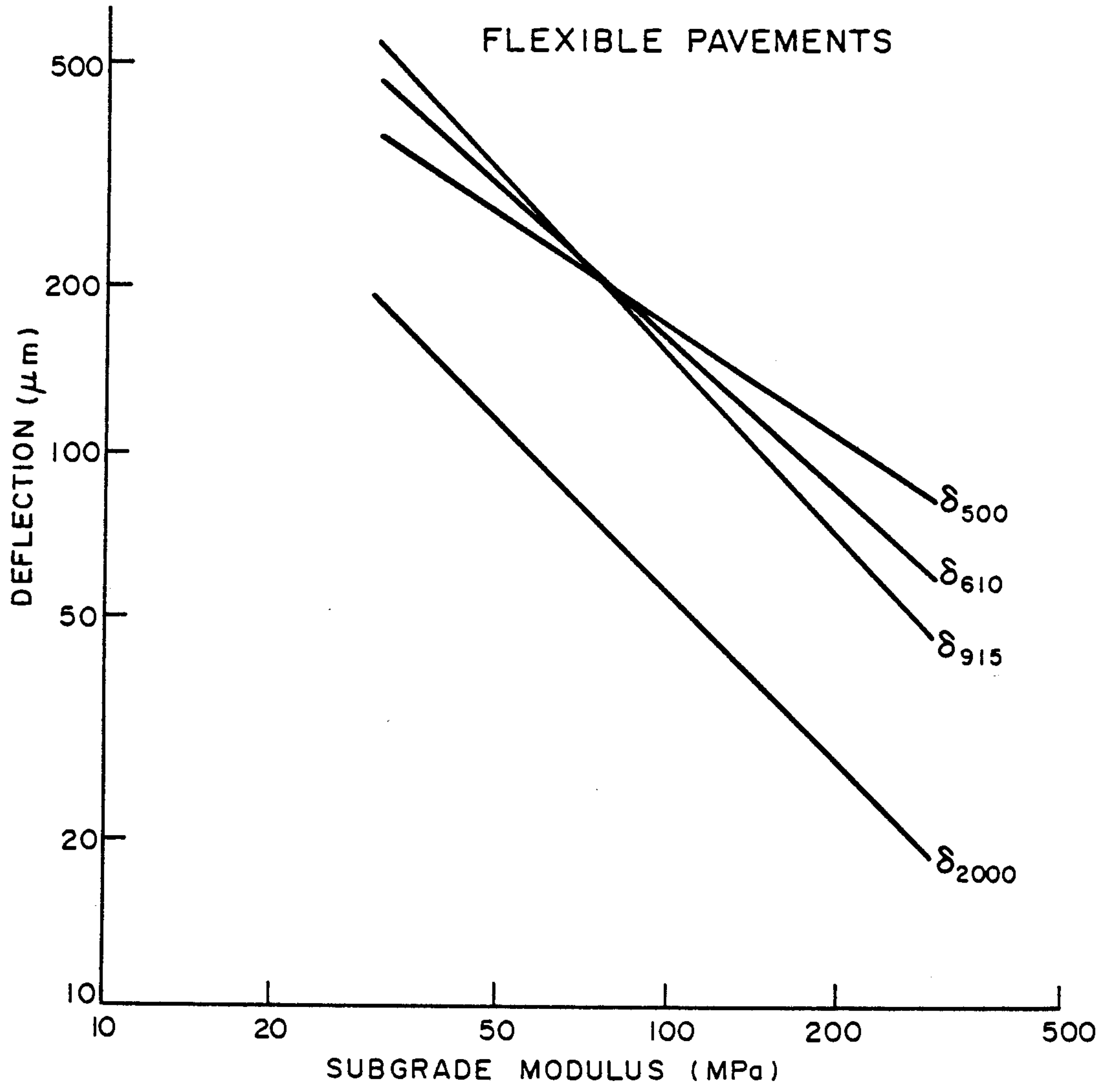

FIGURE 7.11

SUBGRADE ELASTIC MODULUS VERSUS DEFLECTION

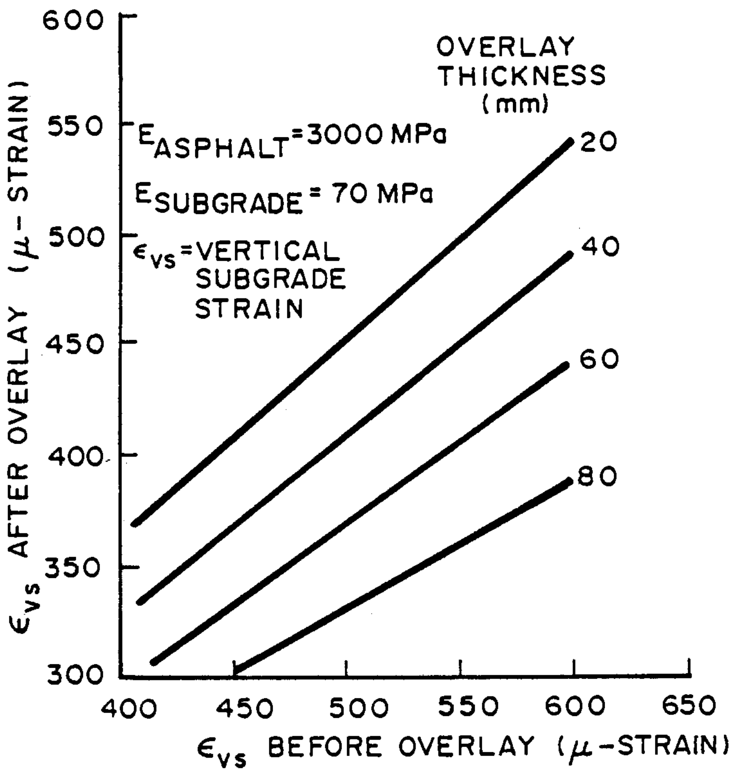

FIGURE 7.12
Effect of overlay thickness on subgrade strain of granular base pavements

70 MPa for the subgrade layer. Overlay thicknesses of 60 mm or more would substantially reduce ε_{vs}, but asphalt layers of such total thickness (surfacing plus overlay) are classified as bitumen base pavements (Freeme et al., 1982a). Such overlay thickness would therefore rarely be applicable. Possibly as levelling courses to correct excessive rut irregularities.

In the discussion of the selection of overlay thicknesses, it is assumed that granular base pavements have either an existing surface treatment with maximum thickness of 20 mm or a premix asphalt surfacing layer with a typical thickness of 40 mm. This original asphalt surfacing thickness has a deciding influence on the deflection basin in the vicinity of the wheel load. This is best reflected in Figure 7.13 which shows surface curvature index (SCI) values before and after overlays of various thicknesses. It can be seen that for granular base pavements with a maximum original surfacing thickness of 20 mm, an overlay of at least 40 mm will be needed to reduce SCI values. For a granular base pavement with an original surfacing thickness of 40 mm, a 20 mm overlay will lead to a reduction in SCI values. This means that there is a reduction in SCI values, only after a total thickness of 60 mm.

The influence of the original asphalt thickness is even more pronounced when the effect of overlays of various thicknesses on the reduction in maximum horizontal asphalt strain (ε_{HA}) is considered. In Figure 7.14 the values of ε_{HA} before and after overlays are shown for original surfacing thicknesses of both 20 mm and 40 mm and for various overlay thicknesses. For a granular base pavement with an original surfacing thickness of 20 mm, an overlay of over 80 mm would clearly be needed to reduce ε_{HA} values, and this is seen as impractical and costly. A granular base pavement with an original asphalt thickness of 40 mm may only require an overlay of 20 mm to reduce ε_{HA} if the value of ε_{HA} before overlay was more than 250 μ-strain.

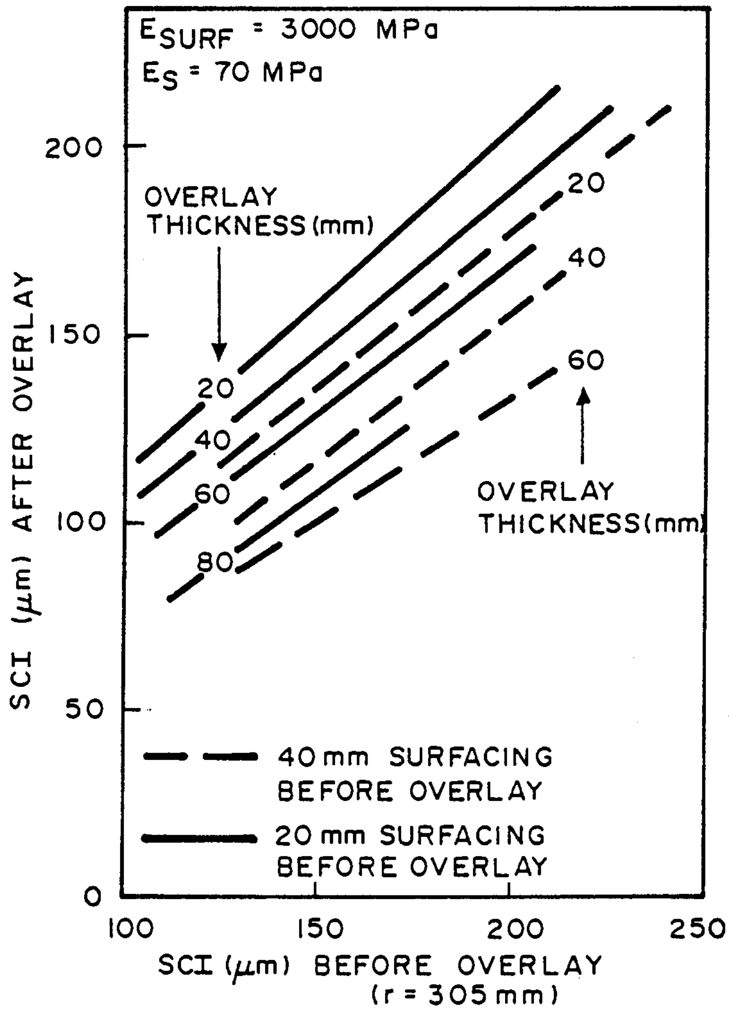

FIGURE 7.13

EFFECT OF OVERLAY THICKNESS ON SCI (r = 305 mm) OF GRANULAR BASE PAVEMENTS

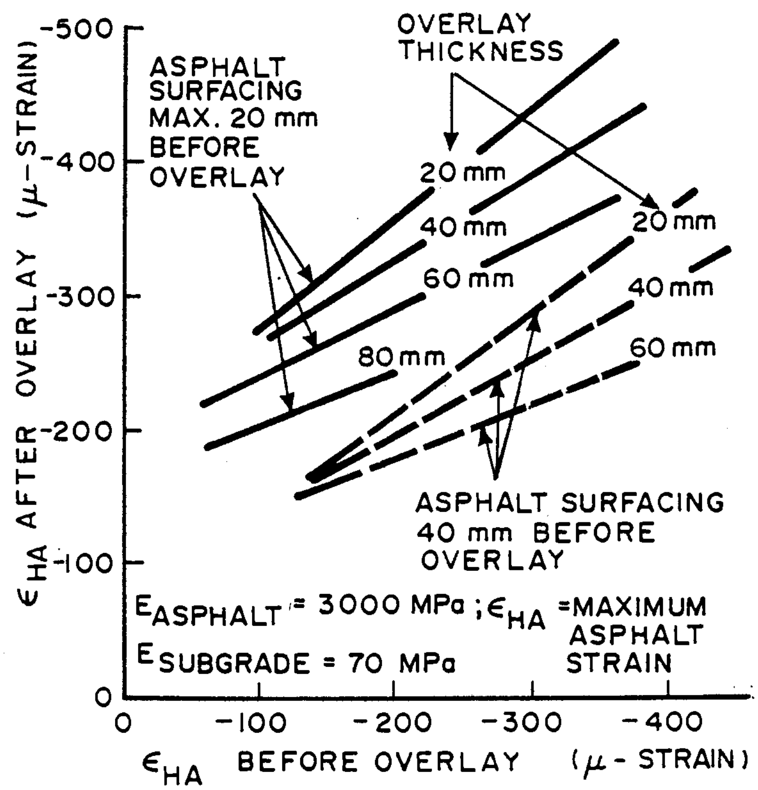

FIGURE 7.14
Effect of overlay thickness on maximum asphalt strain of granular base pavements

5.2 Bitumen base pavements

Overlay thicknesses have the same effect on bitumen base pavements as on granular base pavements in general, concerning the effect of reducing subgrade strain (ε_{VS}). In Figure 7.15 it can be seen that the thicker the overlay, the more the reduction in ε_{VS}.

The subgrade, as varied in terms of elastic moduli, also has an influence in terms of a difference in gradient for the thicker overlays (40 and 60 mm). Each subgrade condition has a definite range over which they are applicable too. For the overlay of 10 mm, subgrade moduli variance does not have an influence. This thickness would normally not be considered for such a purpose.

Deflection basin parameters calculated before and after an overlay reflect a change with various degrees of significance. In Figure 7.16, deflection basin parameters, S, F1, SCI and SD clearly show the effect of overlay thicknesses on values before and after overlays. In general F1, SCI and SD show a reduction in values while S shows an increase in values when overlaid. The indications are not the same though in terms of a structural improvement as SCI and SD are insensitive to changes in elastic moduli of the subgrade (50, 70 and 150 MPa). The relationships determined for S and F1 are true only for a subgrade modulus of 70 MPa.

Overlays reduce the asphalt strain under the base (ε_{HA}) as can be expected. The thicker the overlay, the more the reduction. In Figure 7.17 it is shown how this is true for bitumen base pavements of which the subgrade elastic moduli were varied from 50 to 150 MPa. In the whole analysis it was assumed that no existing cracks occurred in the base, as this would definitely need a different analysis procedure.

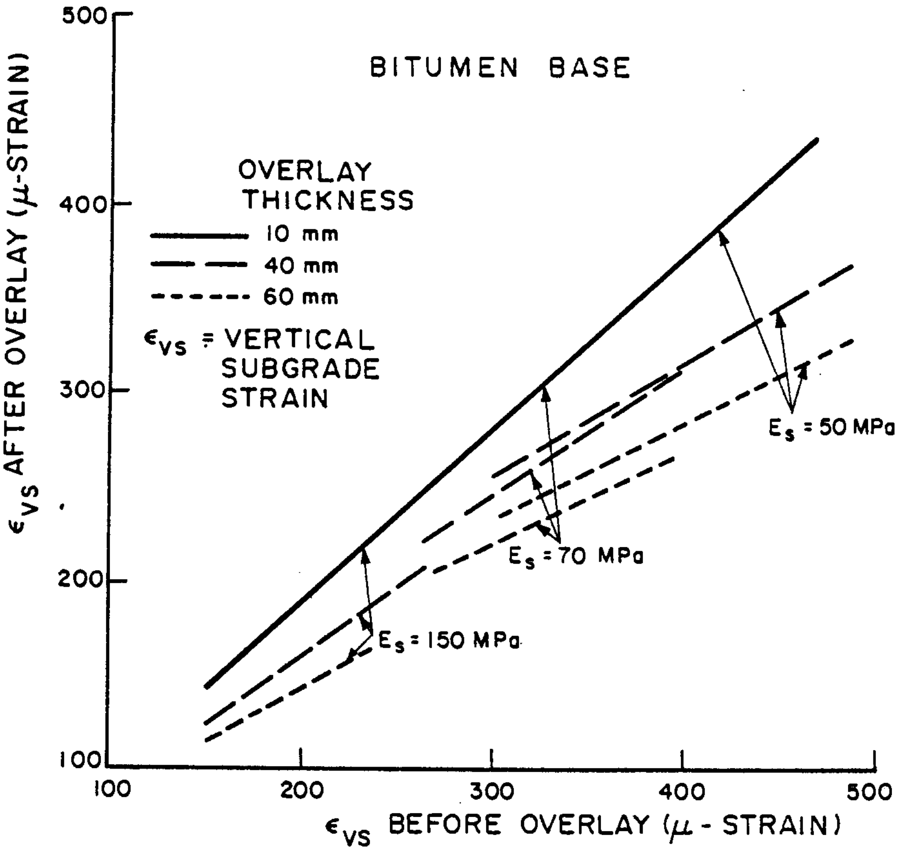

FIGURE 7.15

EFFECT OF OVERLAY THICKNESS ON SUBGRADE STRAIN FOR BITUMEN BASE PAVEMENTS

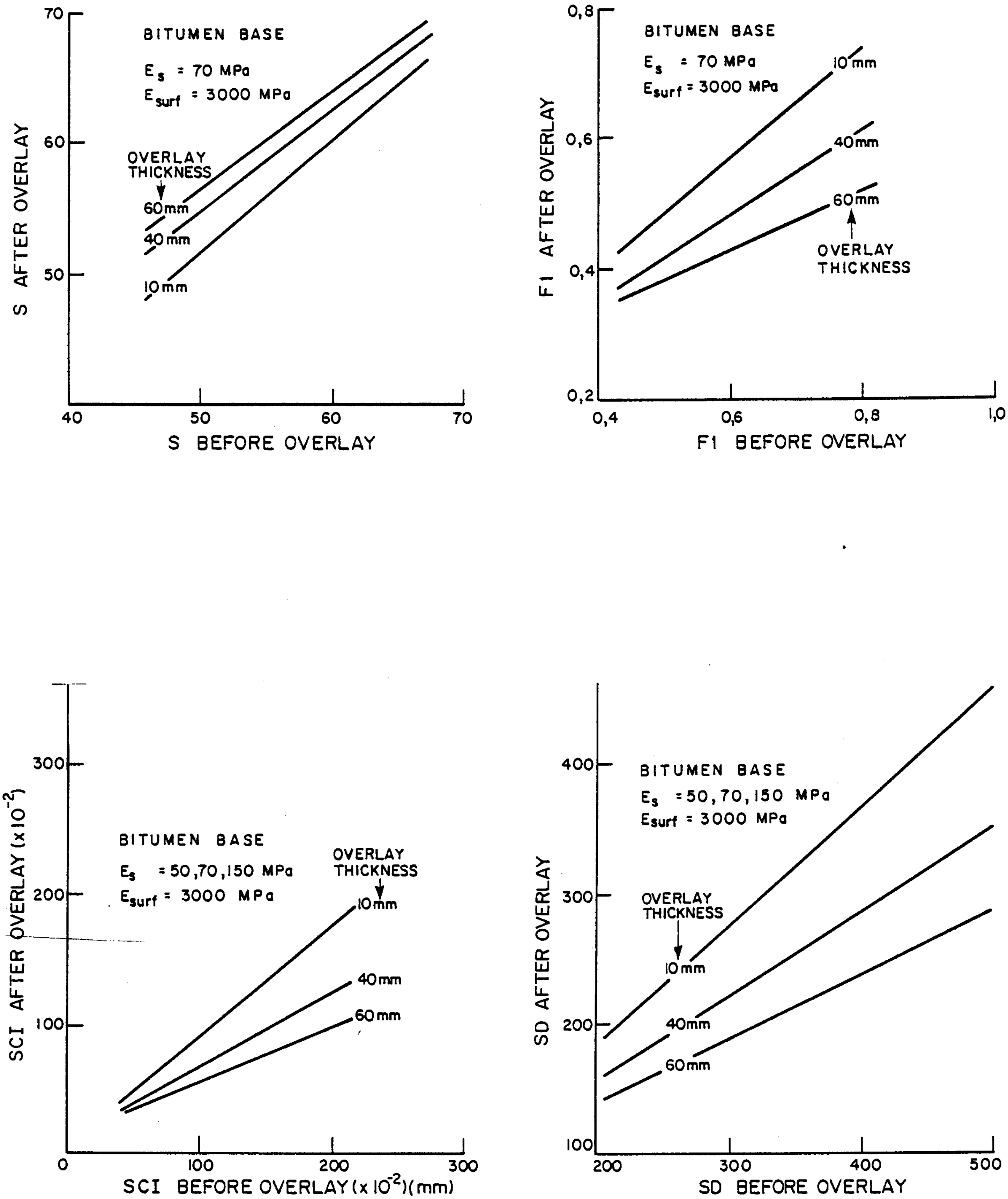

FIGURE 7.16

INFLUENCE OF OVERLAYS OF BITUMEN BASES ON DEFLECTION BASIN PARAMETERS

940-4-5260/9 BS

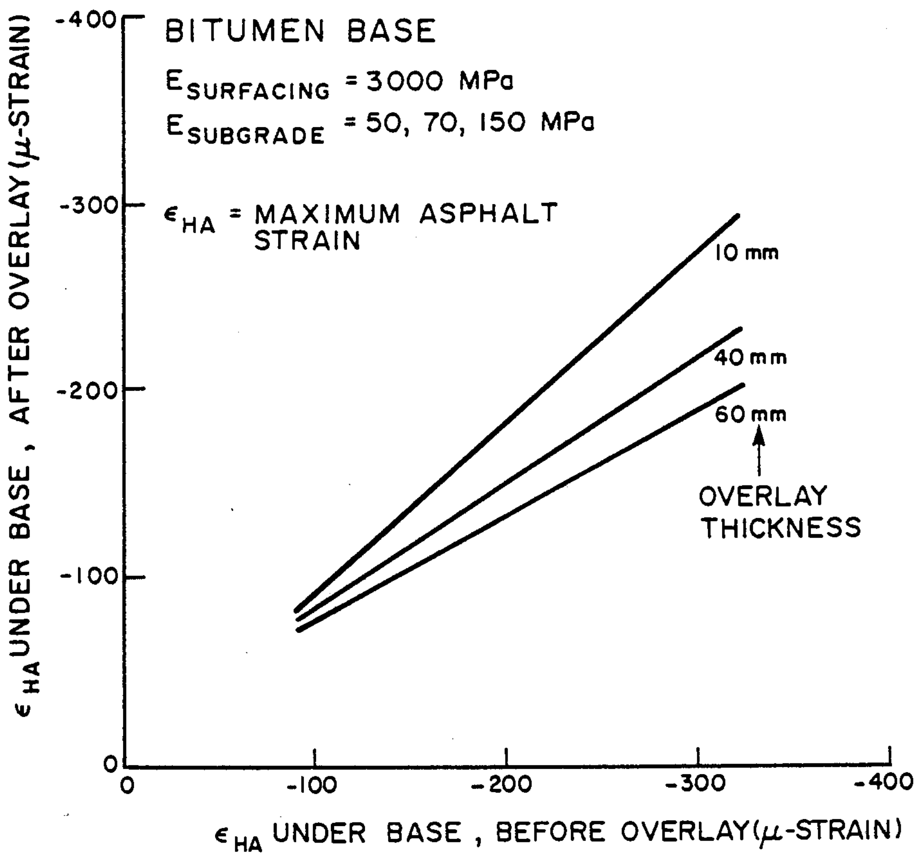

FIGURE 7.17

EFFECT OF OVERLAY THICKNESS ON MAXIMUM ASPHALT TENSILE STRAIN OF BITUMEN BASE PAVEMENTS

940-4-5260/8 BS

6 CONCLUSIONS

Some deflection basin parameters can be related to distress determinants (ε_{VS} and ε_{HA}) in meaningful and clear relationships. In some cases though no discernment can be made with a variance in subgrade elastic moduli. This is not a great concern as simple and clear relationships are still possible. The simultaneous variance of other structural parameters such as surfacing thickness and base elastic modulus and thickness, lead to a lack of simple meaningful relationships for granular base pavements in particular.

The calculated value of surface curvature index (SCI) versus vertical subgrade strain (ε_{VS}) is a useful relationship for granular and bitumen base pavements. There is a clear discernment when subgrade elastic moduli are varied. The gradients of the relationships also differentiate between the two base types (granular and bitumen). Bitumen base pavements seem to be less influenced by the simultaneously varied structural input parameters referred to earlier. Clear relationships between deflection basin parameters F1, R, SD and Q versus vertical subgrade strain (ε_{VS}) are examples.

The maximum asphalt strain (ε_{HA}) calculated under the base of bitumen base pavements, correlates well with deflection basin parameters such as S, A, F1, and BCI. When maximum horizontal asphalt strain (ε_{HA}) is calculated under the surfacing of such bitumen base pavements, the values are compressive which reduces its significance for fatigue calculation. The effect of varying the subgrade elastic modulus was discerned by the above-mentioned deflection basin parameters. The thickness of surfacing of granular bases has a definite effect on relationships between deflection basin parameters and ε_{HA}. The difference in the effect of asphalt thickness is clearly illustrated in the relationships between radius of curvature (R) and maximum horizontal asphalt strain (ε_{HA}) for granular and bitumen bases. The tendency for deflection basin parameters, such as S, SCI, F1 and Q, to group together granular base surfacing thicknesses, of 40 mm to 100 mm differing from the effect of thicknesses of up to 20 mm, is in line with work previously done in this regard.

Overlays of flexible pavements reduce subgrade strain (ε_{VS}) increasingly as thicknesses of overlays increase. This does not mean though that it is a viable and economic procedure to reduce subgrade strain (ε_{VS}). For granular base pavements the overlay thicknesses required to reduce ε_{VS} is more than that normally applied in practice. Such thick overlays could in some cases be applicable to levelling courses on granular base pavements in order to correct excessive rut irregularities.

Overlay thickness of granular bases does have an effect on the calculated values of deflection basin parameters. Surface curvature index (SCI) values reflect a reduction when granular base pavements are overlaid. The thickness of the original surfacing does have a major effect on SCI values. Even better relationships were found for bitumen base pavements for S, F1, SCI and SD. It is even possible to show that SCI and SD reflect changes due to overlay thicknesses, irrespective of subgrade elastic moduli.

The original thickness of a granular base surfacing influences the required thickness of overlay for a reduction in asphalt strain (ε_{HA}). Thin original surfacing layers (less than 20 mm) require impractical thick overlays (80 mm and more) to reduce (ε_{HA}). Original surfacing thicknesses of 40 mm require overlays as thin as 20 mm to reduce ε_{HA}. The base thickness of bitumen bases (more than 80 mm) is sufficient to require normal practice overlay thicknesses to reduce asphalt strain (ε_{HA}). Overlay thicknesses as low as 10 mm reduce ε_{HA} at the underside of the base. Such thin overlays are however also impractical.

CHAPTER 8

PROPOSED USE OF DEFLECTION BASIN MEASUREMENTS IN THE MECHANISTIC ANALYSIS AND REHABILITATION OF FLEXIBLE PAVEMENTS

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1 INTRODUCTION

In this chapter this author endeavours to summarise this study on deflection basin measurements by presenting it in a format that would enhance the present mechanistic rehabilitation design procedure. The discussion summarises the proposed measurement of deflection basins on a network and a project level. Deflection basin parameters that describes the full deflection basin, are suggested for general use. It is shown how the selected deflection basin parameters can enhance the accurate description of the pavement behaviour states. Better material characterization is facilitated by using the relationships established between material properties and deflection basin parameters. In the analysis of typical flexible pavements the relationships between deflection basin parameters and distress determinants are discussed to show how such a design curve approach can enhance the non-simplified approach. Although mention is made of rehabilitation design, the reference only applies to the mechanistic rehabilitation design procedure. Overlay design as an option is high lighted in this chapter. Finally this author indicates what future research needs to be done on deflection basins.

2. DEFLECTION BASIN MEASUREMENTS

2.1 Deflection basin parameters

In chapter 1 the deflection basin was discussed in great detail. Various deflection basin parameters were defined as listed in Table 1.1. In summary it can be stated that deflection basin parameters must:

(a) Represent the full range of characteristics of the whole deflection basin (not only maximum deflection, but rather a combination of parameters covering the whole deflection basin)

(b) Be simple to calculate and interpret, and

(c) Be able to relate to the structural characteristics of the full depth of pavement structures.

In chapter 2 it was shown that irrespective of the measuring equipment the majority of these deflection basin parameters can be calculated fairly easily. It was shown that the full deflection basin as such does not have to be measured. It was suggested that only deflections on the deflection basin be measured at selected off-sets. The off-sets that are suggested are at 0, 127, 305, 610 and 915mm (the off-set at 127mm is seen as optional as it is only related to radius of curvature). As can be seen from Table 1.1 the majority of the deflection basin parameters can be calculated using the deflections measured at these points. In the discussion in chapter 3 it was shown that the deflection basin parameters that are the most significant in terms of their description of pavement behaviour state and material identification are the index parameters. They are ;

Surface curvature index (SCI)
Base damage index (BDI)
Base curvature index (BCI).

These deflection basin parameters describe the deflection basin in full. Surface curvature index (SCI) desribe the area of positive curvature in the immediate vicinity of the load (see Figure 2.7). This reflects on the structural capacity of the base and surfacing layer. Base damage index (BDI) describes the transitional zone of the deflection basin where the positive curvature changes over to that of the reverse curvature. This reflects on the structural capacity of the base and subbase layers. Base curvature index (BCI) describes the area of the reverse curvature. This reflects strongly on the structural capacity of the lower layers such as the selected layer or subgrade.

Other deflection basin parameters that can be used as assistance in the analysis procedure is that of slope of deflection (SD) or maximum deflection or radius of curvature (R). The latter two are included purely to ensure the change over from the traditional deflection basin parameters and their related relationships with pavement analysis can be used as an enhancement with the proposed index parameters.

2.2 Equipment and measurement

The various measuring devices are discussed in detail in Chapter 1. In general the ideal measuring device should:

(a) Realistically simulate moving traffic loads in terms of magnitude of load, shape and equivalent time of loading

(b) Accurately measure the whole deflection basin with high levels of reproducibility

(c) Be simple to operate, so that it could be used with confidence in the field, but it should also be applicable to research

(d) Be capable of attaining high levels of productivity, which must reduce the cost of testing.

2.2.1 Network level

The NITRR Deflectograph (previously the Lacroix deflectograph) is used on a network basis in South Africa by the road authorities, with a proven level of efficiency. It is believed that this will still be the primary function of this measuring device. In the discussion in chapter 1 it was shown that no other non-destructive measuring equipment can compete with this apparatus in terms of productivity and cost per measuring point. With the old Lacroix deflectograph only the maximum deflection was used to distinguish between various uniform sections of road. This author instigated the change with the new NITRR deflectograph whereby amongst others deflections are measured on the full deflection basin at the off-sets as suggested in the preceding section. This means that the most significant deflection basin parameters can be calculated (eg. index parameters). The IBM computer on board the NITRR deflectograph accumulates the data and from that level data manipulation becomes a standard procedure with other personal computers in the data processing unit. The ease of the data manipulation of the NITRR deflectograph enables the engineer to give a more fundamental evaluation of the pavement condition even at the network level which previously was not possible. This will be explained in more detail later in this chapter.

2.2.2 Project level

On a project level detailed assessment of a pavement section is required. The requirements listed above narrows the list of possible non-destructive deflection basin measuring equipment down to only two. The falling weight deflectometer (FWD) seems to be the most appropriate device when measured against these prerequisites. The FWD is a system which has proven its reliability in environments overseas. The ease and speed of operation and transportability gives it the edge over the road surface deflectometer (RSD). It will take a considerable time before such an apparatus can be commisioned for general use in South Africa. It is suggested that, should such a system become available in South Africa, it be properly calibrated against the road surface deflectometer (RSD). This should preferably be done on various pavement types and behaviour states. Accelerated tests with the Heavy Vehicle Simulator (HVS) would provide the ideal situation for such calibration.

The RSD, as used during accelerated tests under the HVS, is however, still the main reference device for non-destructive deflection measurements of pavements in South Africa. The vast extent of the information recorded on various pavement types under various testing conditions, behaviour states and material states, makes it possible to do extensive modelling of pavement behaviour. The deflection basins of the pavement, as measured with the Multi-Depth-Deflectometer (MDD), are also available for further comparative studies on the same pavement structures and loading conditions. The RSD measures the full deflection basin and therefore does not confine the analysis to specific deflection basin parameters. The WASHO procedure of measuring, as is used under the HVS, limits the effects of plastic deformation. The wheel approaches the measuring point while the typical rebound procedure, as is still used with the standard Benkelman beam truck, measures plastic deformation too. The RSD can therefore be used with the same standard Benkelman beam truck, but with a different measuring procedure.

The set-up for measuring the deflection basin with the RSD already exists, as used in combination with the Crack Activity Meter (CAM) (see Appendix E). The procedure of data management and analysis for the RSD is well established and is described in detail in chapters 2 and 3. The relationships developed during accelerated tests with the RSD will make it a powerfull tool in the hands of the engineer during the detailed assessment phase. In the rest of the chapter this will be discussed in more detail.

3 ENHANCEMENT OF THE MECHANISTIC REHABILITATION DESIGN PROCEDURE WITH DEFLECTION BASIN MEASUREMENTS

3.1 General

In chapter 4 and 6 reference are made to the mechanistic design procedure.The mechanistic rehabilitation design procedure as used in South Africa is well established and verified with the fleet of accelerated testing facilities, the Heavy Vehicle Simulators (HVSs) (Freeme, et al., 1982b and Freeme, 1983). However, where this rehabilitation design method is lacking, is in making better use of non-destructive measurements to make this rehabilitation and design process more effective. Deflection basin measurements have been under-utilized in South Africa as they were mostly limited to the use of maximum deflection and radius of curvature measurements. This has resulted in empirical relationships which reflected little information on fundamental material and pavement type behaviour.

The use of deflection basin parameters in the rehabilitation and analysis of flexible pavements was investigated and the suggested incorporation thereof in the existing mechanistic rehabilitation design procedure is described in the sections to follow.

3.2 Identification of pavement behaviour states

The behaviour catalogue is an extension of the existing design method, the TRH4 (NITRR, 1985a) which also has a catalogue of designs. The basic philosophy followed in the catalogue of rehabilitation designs and that of the TRH4 (NITRR, 1985a) is therefore similar. This is illustrated in the flow diagram of the mechanistic pavement rehabilitation design method in Figure 4.1. For the reader's convenience this figure is repeated here as Figure 8.1.

The First step is always to identify the pavement class. An in depth discussion of pavement class distinction or selection is given in TRH4 (NITRRa, 1985) and will not be repeated here.

The second phase illustrated in Figure 8.1 is to correctly identify the pavement type. This identification is done in terms of the base layer type. The base types are defined as: Granular, bituminous, cemented or concrete. This subdivision of pavements, based on base types, is clearly described by Freeme (1983) and Freeme et al. (1986) in terms of the difference in the performance or behaviour of these basic pavement types and the resultant difference in the behaviour states and catalogue selection.

Deflection measurements are the basis of the definition of the pavement behaviour states. Approximate ranges of maximum deflection (δ_o) are used to subdivide the pavement behaviour types into very stiff, stiff, flexible and very flexible behaviour states. These ranges of maximum deflection (δ_o) and an abbreviated description of the general behaviour are shown in Table 3.1.

Deflection basin parameters that were measured on various pavement types during accelerated tests (see chapter 3) were used to give a better definition of the various pavement states. These deflection basin parameter ranges are listed in Table 3.2 and repeated in

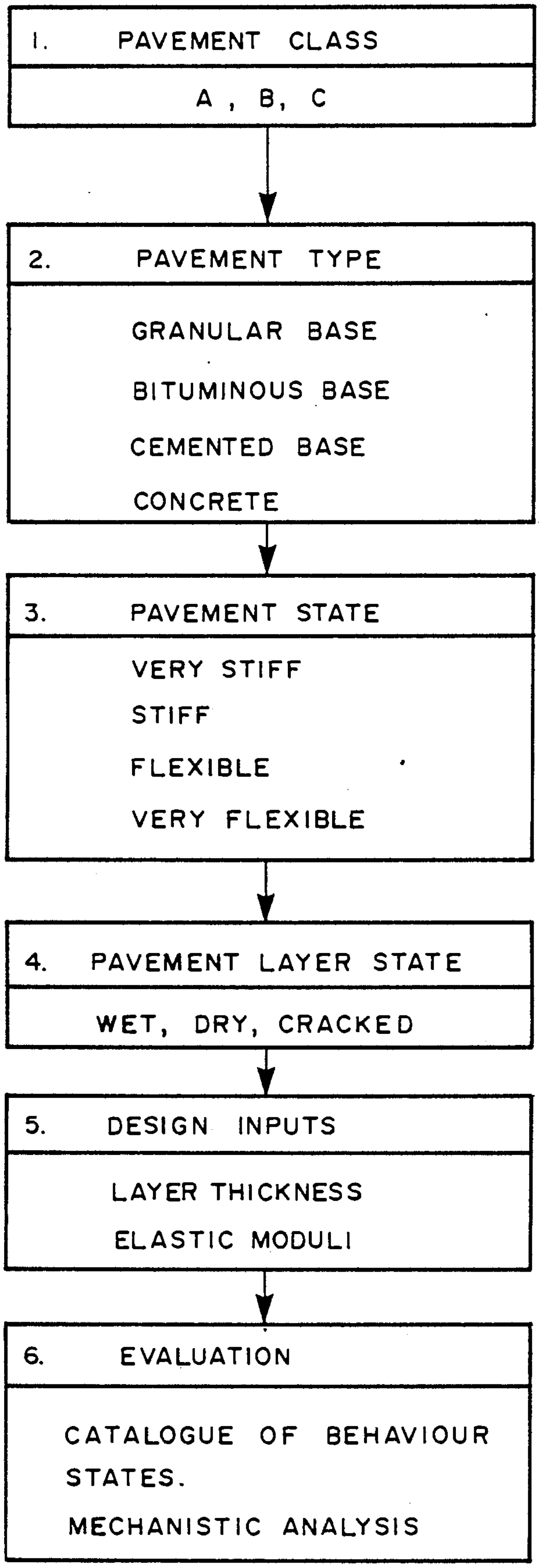


FIGURE 8.1

FLOW DIAGRAM OF THE MECHANISTIC PAVEMENT REHABILITATION DESIGN METHOD (FREEME, 1983)

Table 8.1 for the various pavement behaviour states. It is suggested therefore that the index parameters rather than maximum deflection be used in the identification of the pavement behaviour state.

The pavement behaviour states of particular interest in this discussion are the flexible and very flexible behaviour states. In general it will also mean that the pavement types as shown in Figure 8.1 will be narrowed down to granular bases, bitumen bases and cemented bases. As the latter type will mostly be in a cracked layer state, the flexible cemented bases will be analyzed as a granular or equivalent granular base type.

3.3 Material characterization

3.3.1 General

For a correct analysis of a pavement in the mechanistic rehabilitation design procedure it is very important that the material of the various layers are identified correctly. In Figure 8.1 it is shown how the class, type, the behaviour state and state of the layer materials are used to characterize the pavement layer materials. This normal approach can be enhanced by the incorporation of deflection basin parameters in the analysis procedure, but such deflection basin parameters can also be used to describe the pavement materials more directly.

3.3.2 Normal material characterization

The normal approach whereby material is characterized is where the principle input for the material properties (layer thickness, effective elastic moduli and Poisson's ratio) is determined from basic pavement and material information. This is best illustrated by referring to Figure B.1 (see Appendix B) where it is shown in the flow diagram that both destructive and non-destructive testing results are used to do effective material characterization. This ties in with the approach outlined earlier for the mechanistic rehabilitation design procedure (Freeme, 1983). Information from

TABLE 8.1 Behaviour states defined by deflection basin parameters

Behaviour state	Deflection basin parameter ranges				
	Max. defl. (mm)	SD ($x\ 10^{-6}$)	SCI (mm)	BDI (mm)	BCI (mm)
Very stiff	< 0,2	< 50	< 0,01	< 0,01	< 0,01
Stiff	0,2 - 0,4	50 - 400	0,01 - 0,2	0,01 - 0,1	0,01 - 0,05
Flexible	0,4 - 0,6	400 - 750	0,2 - 0,4	0,1 - 0,15	0,05 - 0,08
Very flexible	> 0,6	> 750	> 0,4	> 0,15	> 0,08

8.10

as-built information, condition surveys, traffic surveys and deflection surveys form the pool of information of the non-destructive testing. Destructive testing like in situ Dynamic Cone Penetrator (DCP) surveys or coring of cemented or treated materials for material sampling and laboratory testing, are normally done during a second phase detailed analysis. Such information is very valuable as even standard laboratory tests can be used to characterize layer material into various material classes as specified in TRH14 (NITRR, 1985b) and abbreviated in TRH4 (NITRR, 1985a). Freeme et al. (1982a) and Freeme (1983) also give a detailed description of the various material classes.

The use of more sophisticated laboratory tests like triaxial tests, repeated loads triaxial tests, repeated loads indirect tensile tests, beam tests, creep tests, etc. can only enhance the correct description of materials. The mechanistic design method in general encourages the use of such sophisticated tests. The effective elastic moduli relationships used in the design method are based on extensive tests of this kind to determine the basic material characteristics. Freeme et al. (1982a) and Freeme (1983) do provide transfer factors for the various pavement classes and pavement layers to relate such sophisticated laboratory results to actual behaviour as measured under accelerated testing. The DCP results enhances not only the proper material characterization,but with the balance curves concept (de Beer, 1986) the analysis procedure.

Finally it is therefore possible to accurately determine the basic input parameters for the mechanistic analysis, if adequate knowledge and experience concerning such a characterization are available. Freeme, et al. (1982) and Freeme (1983) give a detailed description of each material type, material states and possible configuration in a pavement structure. The effective elastic moduli can therefore be read off such tabulated values. It is believed that, as considerable time and engineering judgement have already gone into the correct material identification up to this stage, any uncertainty can be overcome by doing a sensitivity analysis on such pavement structures during the analysis phase.

3.3.3 Material characterization with deflection basin measurements

The subgrade is an important layer in the analysis of a pavement. The effect that this layer has on the general shape of the deflection basin was discussed in detail in chapter 4. In chapter 7, it was shown that there exists a very good correlation between subgrade elastic modulus and deflection measured on the reverse curvature of the deflection basin. These relationships are true for all flexible pavements with up to five structural layers. These relationships are as shown in Figure 7.11. It is suggested that more than one deflection be used in the determination of the subgrade elastic modulus and an average value can be determined. A further refinement can also be added by only using deflection values greater than 0,01 mm to avoid problems with measurement inaccuracies.

The elastic modulus of the base of a granular base pavement can also be determined from Figure 7.10 if the subgrade modulus is 70 MPa. As shown in this figure, various deflection basin parameters can be used to determine the subgrade elastic modulus. The effect of the surfacing thickness is also taken into consideration. Although the subgrade effective elastic modulus is not varied, this figure can be used to determine a first approach to the base elastic modulus value in analyses.

3.3.4 Back-analysis of effective elastic moduli

The effective elastic moduli as determined with depth deflections from Multi-depth-deflectometer (MDD) measurements, are the reference values as determined under accelerated testing. These effective elastic moduli were used extensively to verify the tables of effective elastic moduli as proposed by Freeme (1983) and Freeme et al. (1982a).

In Chapter 5, it was described and illustrated how surface deflection basin measurements can be used to back-calculate effective elastic moduli. The linear elastic computer program BISAR was used to back-calculate effective elastic moduli. The basic procedure is to use deflections on the outer edge of the deflection basin to calculate the effective elastic modulus of the lowest layer (subgrade). Hereafter deflections nearer to the point of loading are used to calculate the effective elastic modulus of the next layer nearer to the surface. By thus each time using deflections nearer to the point of loading, the effective elastic moduli of layers increasingly nearer to the surface are calculated. Although the back-analysis procedure as outlined above can be defined as a standard procedure, considerable manual interference is still needed to arrive at unique layer effective elastic moduli. In conclusion, it can be stated that the confidence in the indirect material classification, as outlined earlier, rates the necessity for such direct approach back-calculation procedure as a confirmative bonus.

3.4 Analysis of flexible pavement structures

3.4.1 General

In Figure 8.1 it can be seen that once the material characterization, also called design inputs, is completed, the last phase, namely the evaluation, can be tackled. As shown in Figure 8.1 the catalogue of behaviour states can be used in the analysis, as described by Freeme (1983). The non-simplified approach (Jordaan, 1986) on the other hand is still the standard procedure where the linear elastic computer programmes are used to calculate the various distress determinants. This approach is also described in great detail by Freeme (1983) and will not be repeated here. The deflection basin parameters can however be used in a design curve approach (Jordaan, 1986) whereby the selected deflection basin parameters are related to the various distress parameters. This approach was investigated in detail in chapter 7. The main distress determinants relating to rutting and fatigue cracking are discussed here in the analysis proses.

3.4.2 Rutting

Vertical subgrade strain (ε_{vs}) is directly correlated to various deflection basin parameters as described in chapter 7. Only the preferred deflection basin parameters outlined earlier, are used here, but the relationships described elsewhere for these parameters can also be used.

The vertical subgrade strain (ε_{vs}) was correlated with the various deflection basin parameters for the basic pavement types, bitumen and granular bases, separately. In Figure 7.2 Surface Curvature Index (SCI) is shown versus vertical subgrade strain (ε_{vs}) for granular and bitumen base pavements. In the case of granular bases a relationship between base damage index (BDI) and subgrade vertical strain (ε_{vs}) can be used without regard to the subgrade effective elastic moduli. This relationship is shown in Figure 7.3.

In the mechanistic rehabilitation design procedure deformation or rutting is kerbed by the limitation of vertical subgrade strain (ε_{vs}). In Figure 8.2 these fatigue life criteria as verified by accelerated testing are shown. The various pavement classes are differentiated as shown. By first determining the vertical subgrade strain with the deflection basin parameters ,the remaining life can be determined as outlined in Appendix C.

Although deformation of the whole pavement structure cannot be assessed, the mechanistic rehabilitation design procedure as described by Freeme (1983) does allow for indirect design. In the case of granular bases the calculation of safety factors, based on the basic shear parameters (C and Ø) and the moisture condition of the pavement layers, limit deformation in such a granular layer by set criteria (Maree et al., 1982). Freeme (1983) expanded these criteria for the equivalent granular states of cracked cemented layers and thus limiting deformation in the cemented layers, too. If the quality of the granular base material is known, the safety factor can be determined using the regression functions as determined in Table 7.2 for that specific subgrade condition. The fatigue life criteria as discussed by Freeme (1983) can then be used to determine the remaining life of such a granular base layer.

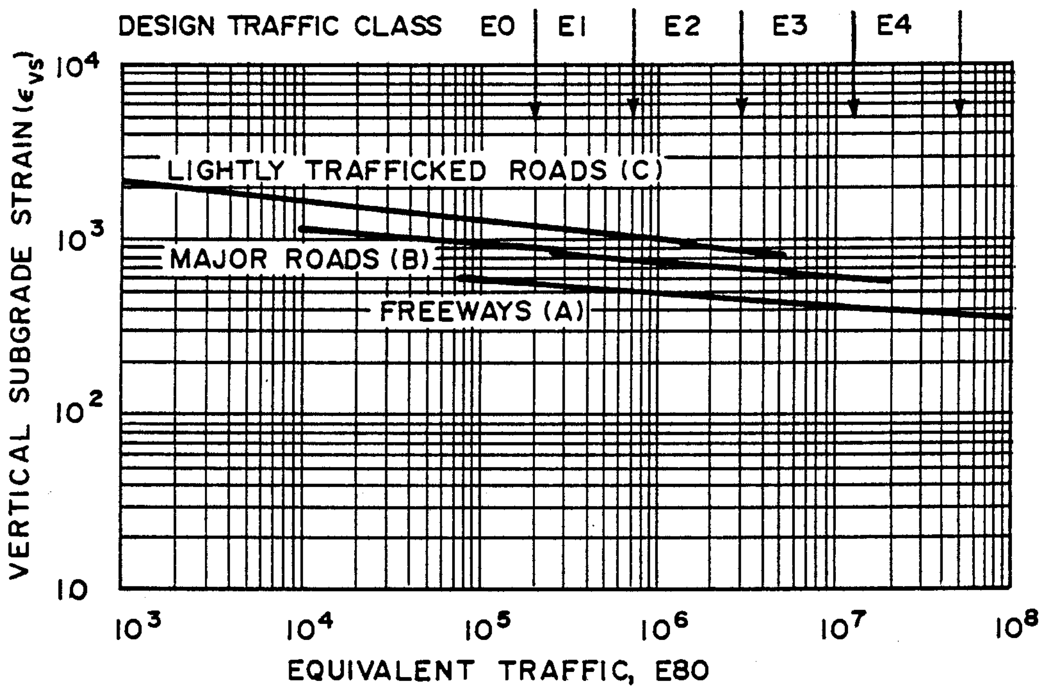


FIGURE 8.2
Recommended vertical subgrade strain criteria for different road categories

Deformation in the bituminous layers are also limited indirectly in mechanistic analysis and rehabilitation design procedures. Thompson et al, (1986) indicate that rut development in such layers are controlled by the proper selection of materials, mix design, and construction control. Freeme et al. (1982a) clearly indicated how this same approach was followed and verified with accelerated testing. Additional criteria developed to kerb creep (Viljoen and Meadows, 1981) indicate the continued development in this direction.

3.4.3 Cracking

The critical parameter most commonly used to predict or govern fatigue cracking of bituminous layers, is the maximum horizontal asphalt strain (ε_{HA}) at the bottom of the layer. In chapter 6 it was indicated that cognizance is given to the fact that the maximum asphalt strain does not always occur at the bottom of the asphalt layer, but that the criteria as used in the mechanistic rehabilitation design procedure does lead to satisfactory results in the majority of cases.

In Figure 7.8 radius of curvature (R) is related directly to maximum horizontal asphalt strain (ε_{HA}). This is true for bitumen base pavements without regard to the subgrade support condition. For the granular base pavements the relationships shown are true only for the effective elastic modulus of 70kN.

The surfacing thickness of granular base pavements has a definite influence on the relationship between maximum horizontal asphalt strain (ε_{HA}) and other deflection basin parameters. Other deflection basin parameters, like surface curvature index (SCI), is less sensitive to various surfacing thicknesses, but still sensitive to discern between thin (< 20 mm) and thicker (4) to 100 mm) surfacings. These relationships are shown in Figure 7.9 for a subgrade effective elastic modulus of 70 MPa.

A combination of deflection basin parameters and basic layer thickness information can also be used to determine maximum horizontal asphalt strain (ε_{HA}) by means of the regression relationships developed in chapter 7. Design curves and relationships as discussed above can therefore be used to determine maximum horizontal asphalt strain values for bitumen bases or asphalt surfacings of granular bases.

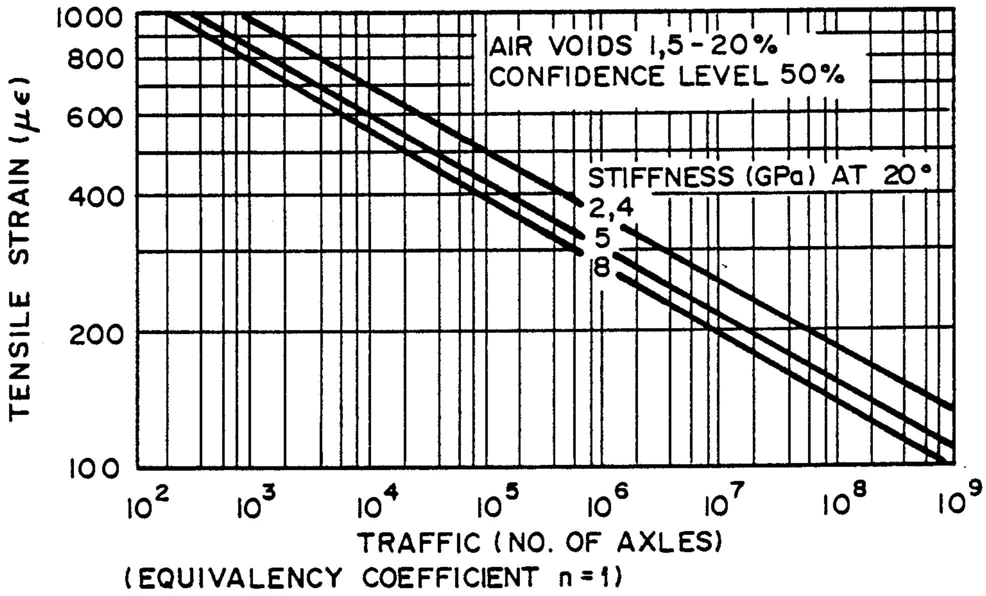
The mechanistic rehabilitation design procedure is unique in the sense that fatigue relationships were developed for thin asphalt layers (< 50 mm) and for thick bituminous base layers (> 80 mm). These relationships are shown in Figures 8.3 and 8.4. In the case of thin asphalt layers, not equivalent 80kN axles (E80's), but actual axle repetitions independent of loading are considered. Remaining life can then be determined as outlined in Appendix D.

3.5 Rehabilitation design

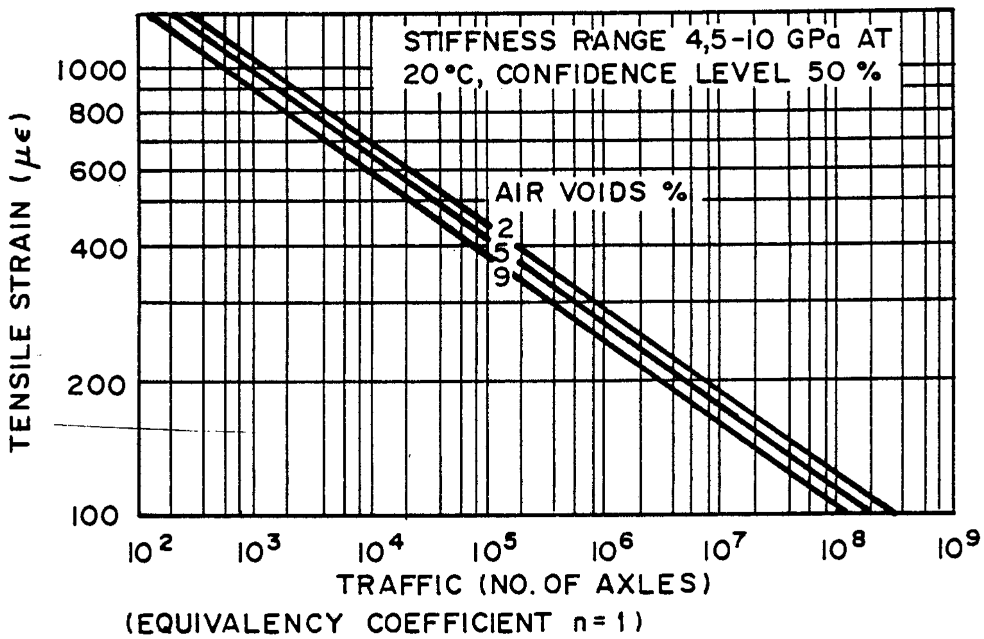
3.5.1 General

The South African mechanistic rehabilitation design procedure (Freeme, 1983) is ideally suited for rehabilitation design. The general tendancy would be to determine which distress determinant of which layer does not make the required structural design life. Remedial measures are then applied to lengthen the structural life of such a pavement under consideration. This may constitute various options of which overlays are but one option . As the discussion in this chapter is geared towards the distress criteria, maximum asphalt strain and vertical subgrade strain, overlay design will also be the focus of the deflection basin related rehabilitation design. Deflection basin parameters were used to determine these distress determinants. Overlays of various thicknesses were analyzed for the typical pavement types, granular and bitumen base pavements, in the flexible behaviour state. The results as discussed in Chapter 7 are summarized here.

3.5.2 Granular base pavements



(a) Gap-graded asphalts



(b) Continuously graded asphalts

FIGURE 8.3

Recommended fatigue criteria for thin bitumen surfacings

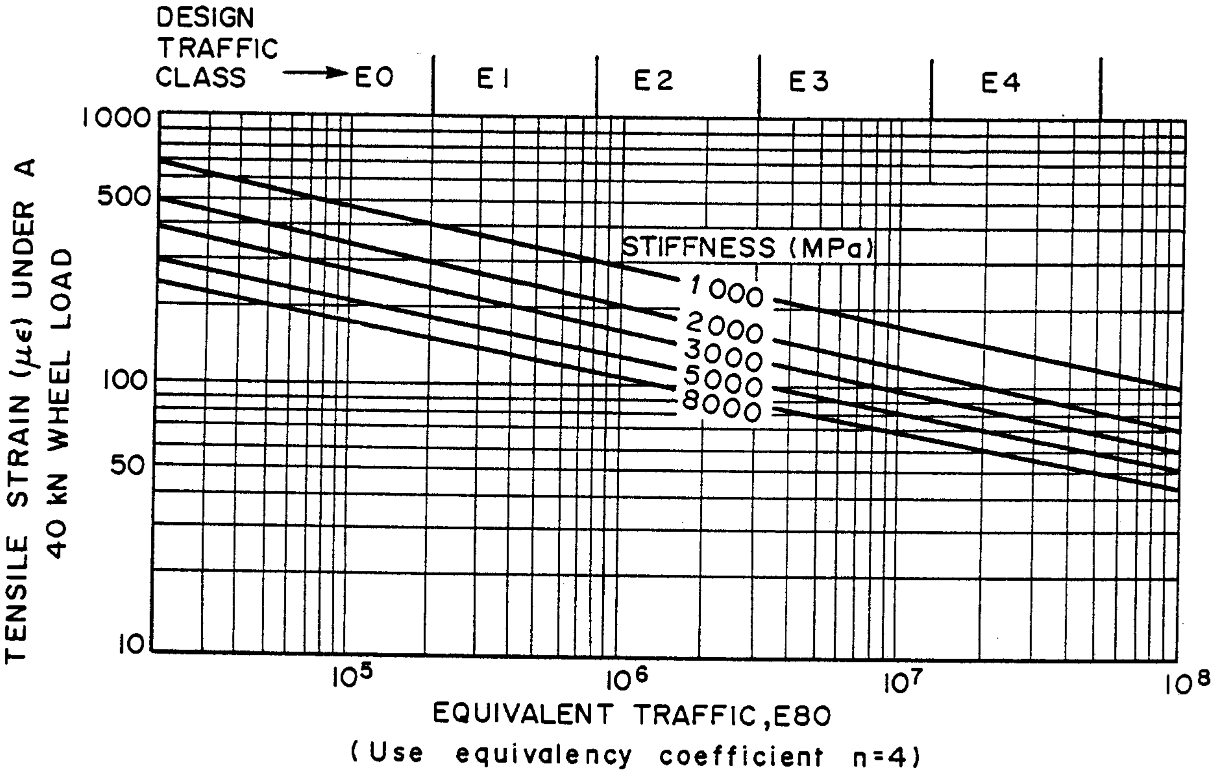


FIGURE 8.4

RECOMMENDED FATIGUE LIFE CRITERIA FOR THICK BITUMEN BASES

Granular base pavements in the larger dry regions of South Africa have surfacing thicknesses in the order of 30 to 50 mm. Thin overlay surfacings (20 mm) cannot be expected to have a significant influence on the reduction of subgrade strain. In Figure 7.12 it is shown that for a subgrade effective elastic modulus of 70 MPa an overlay thickness of 60 mm and more would substantially reduce subgrade strain, but asphalt layers of such total thicknesses are classified as bitumen base pavements. Overlays of such thicknesses would therefore seldom be applied to granular base pavements except in the wetter regions of South Africa.

Existing asphalt surfacings were classified as surface treatments (20 mm and less) or normal thickness surfacings (40 mm average) for overlay purposes to curb fatigue cracking. The effect of this original surface thickness is clearly shown in Figure 7.13 where various thicknesses of overlays (20, 40, 60 and even 80 mm) are shown related to the pre-defined existing surfacing thicknesses. The normal practice with existing surface treatments requiring an overlay is to ignore the thickness of asphalt surface treatment, equate it to the granular base and apply another surface treatment. In normal practice a pavement with an original thickness of 40 mm, however, can recieve an overlay of any required thickness in order to reduce the calculated strain values.

3.5.3 Bitumen base pavements

Subgrade strain (ε_{vs}) can be reduced by overlaying. In Figure 7.15 it is shown that knowledge on the effective elastic moduli of the subgrade is a prerequisite as the variance in subgrade effective elastic moduli lead to different gradients for the various overlay thickness relationships.

As overlay thicknesses are increased, maximum asphalt strain calculated under the base is reduced. In Figure 7.16 it can be seen that this reduction in maximum asphalt strain, by means of overlaying, is true even if the effective elastic modulus of the subgrade is varied as indicated.

3.5.4 Practical considerations

In the design and selection of overlays of flexible pavements, the standard practice should also be considered. Granular base pavements with a double surface treatment (20 mm) would normally receive another surface treatment of about 20 mm. Granular bases with surfacing thicknesses of 30 to 50 mm, can also receive a surface treatment. Overlays of 30 to 40 mm are more typical in the dry regions of South Africa, while thicker overlays may be considered in the wetter regions of South Africa.

As soon as it has been established which of the distress determinants, namely maximum asphalt strain or subgrade strain, would determine the overlay thickness, the situation with the overlay should be checked with the remaining distress determinant. This situation is of particular importance for granular base pavements where a total asphalt surfacing of 40 to 80 mm, in order to curb deformation, may in fact reduce fatigue life drastically (Freeme, et al., 1982a).

4 FUTURE RESEARCH

4.1 <u>General</u>

This author sees this thesis only as an exploratory excercise on deflection basin measurement and use in the mechanistic rehabilitation design procedure. There is an awareness that in a sense more questions are now asked than answers given to the initial questions asked at the outset of this study. The author nevertheless endeavoured to steer his investigation along the path whereby the new information gained on deflection basins and their use in analysis procedures can be of practical use. For that reason the well proven South African rehabilitation design procedure was used as a main reference in the discussions throughout. This approach also led to the identification of a number of areas in this field that urgently need further attention in future research efforts. The author therefore attempts to identify these needs and motivate the research in the various areas separately.

4.2 Measuring equipment

The RSD as being used in South Africa was specifically developed for the HVS testing system. This apparatus is a natural extention of the Benkelman beam. This is in effect also the problem with this apparatus in the sense that, just like with the standard Benkelman beam, a truck is needed to do the measurements with the equipment and at least 3 persons to operate it. A need for equipment like the FWD which bridges these problems is therefore obvious with the advantages as listed elsewhere in this thesis. As mentioned elsewhere in this thesis, this type of equipment however needs to be properly calibrated and compared to the RSD. The accelerated testing with the HVS provides the unique opporunity to do just that.

The new NITRR deflectograph can measure the deflection at suggested off-sets. This makes this equipment a more effective tool of evaluation on the network level. Considerable work still need to be done in order to relate such measured deflection basin parameters to the specific material and road conditions. The relationship between RSD deflection basins and that of the deflectograph needs proper investigation in order to enable the transfer of relationships developed for the former to be used with the latter.

4.3 Material Characterization

The limited investigation of measured deflection basins with the RSD during accelerated tests enhanced the mechanistic rehabilitation design procedure by enabling more accurate identification of pavement behaviour states. Seen at the back ground of the vast extent of similar information on measured deflection basins being available ,the need for an in depth study in this field is glaringly obvious. A larger sample would facilitate the possibility of establishing relationships between deflection basin parameters and specific layer materials in specific behaviour states and for specific pavement types to that of the distress determinants. This semi-empirical approach can greatly enhance the current mechanistic rehabilitation design procedure.

The possibility of back-calculating effective elastic moduli from RSD measured deflection basins is an area that need considerable attention. The procedure as outlined in this thesis, making use of linear elastic computer programmes can be developed much further in spite of the obvious short comings. The possible use of finite element programmes or elasto-plastic computer programmes like the VESYS program warrents a closer look. The possiblity of establishing regression relationships between deflection basin parameters as measured with the RSD and MDD may be an avenue to explore too. The effective elastic moduli as calculated with the help of MDD measured deflections are still considered as the most reliable values.

The possibility of establishing relationships between deflection basin parameters and other parameters of other design methods should also be investigated. A typical possible relationship could be between the area (A) parameter and the DSN_{800} number of the DCP-model.

The analysis of the standard pavement structures being used in South Africa can be expanded considerably to include other typical pavement types and behaviour states. The most influential parameters were identified in the initial analysis, but the small size of the analysis limited the application of the design curves that were established to very specific conditions. It is also believed that other distress determinants relating to fatigue cracking and deformation need to be considered in such an analysis.

4.4 Analysis procedure

The possibility of using the emperical theoretical relationships established between deflection basin parameters and distress determinants were explored. The limited relationships established however has consideralble constraints in their application. This confirms the need for a much broader analysis of various pavement types to be analysed as expressed in the previous section. These design curves can then be incorporated in a computer programme which automatically relates the measured deflection basin parameters to these relationships and calculate the life of a pavement type or the rehabilitation design.