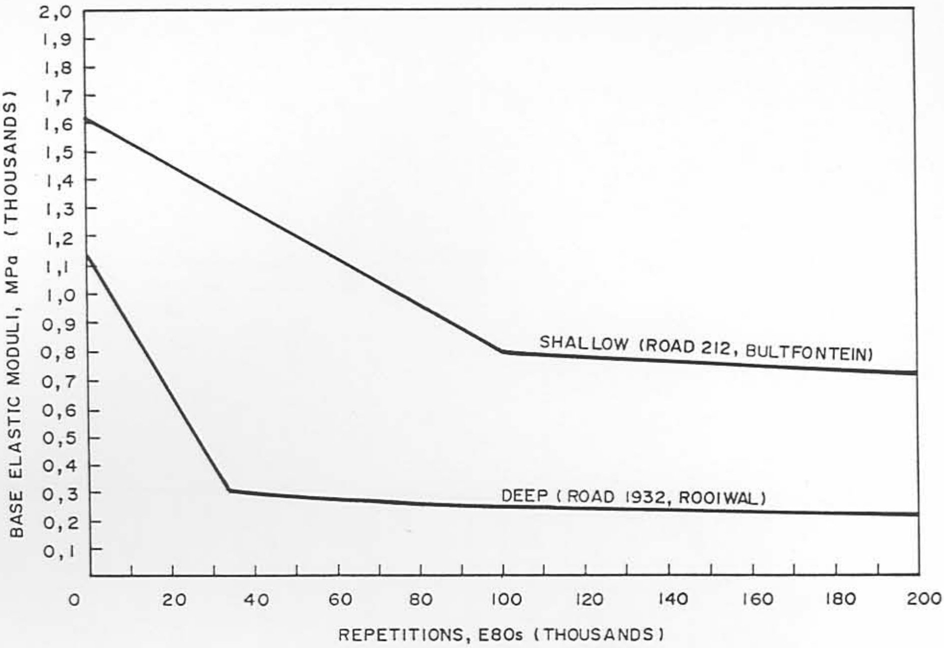


FIGURE 7.14

REDUCTION IN BASE MODULI IN THE TRANSITION ZONES FOR THE DEEP AND SHALLOW PAVEMENTS

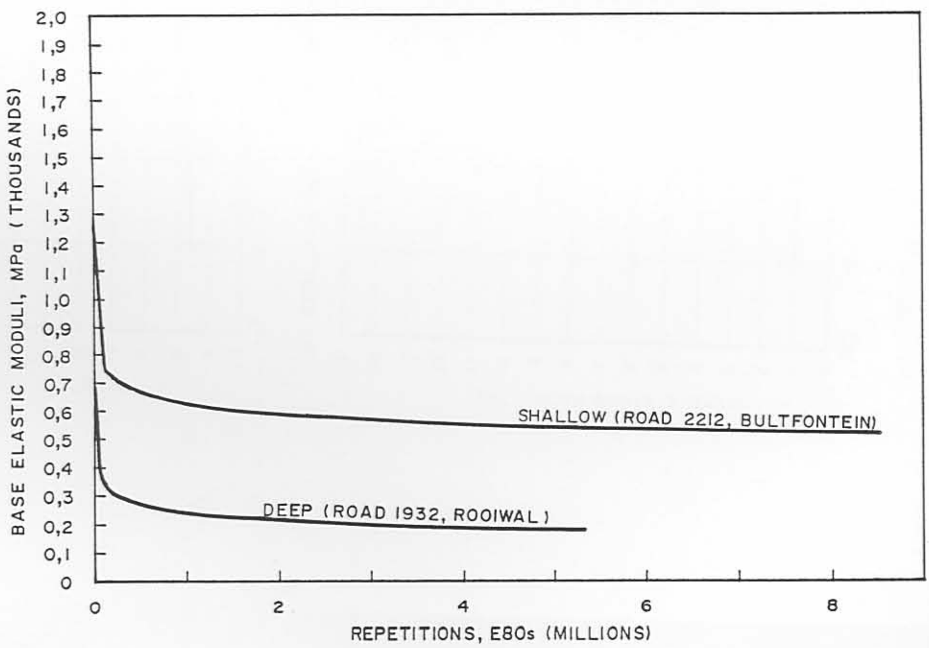


FIGURE 7.15

RELATIONSHIPS OF THE AVERAGE CHANGE IN THE EFFECTIVE ELASTIC MODULI OF THE BASE LAYERS OF THE RELATIVELY DEEP AND SHALLOW PAVEMENTS EVALUATED IN THIS STUDY

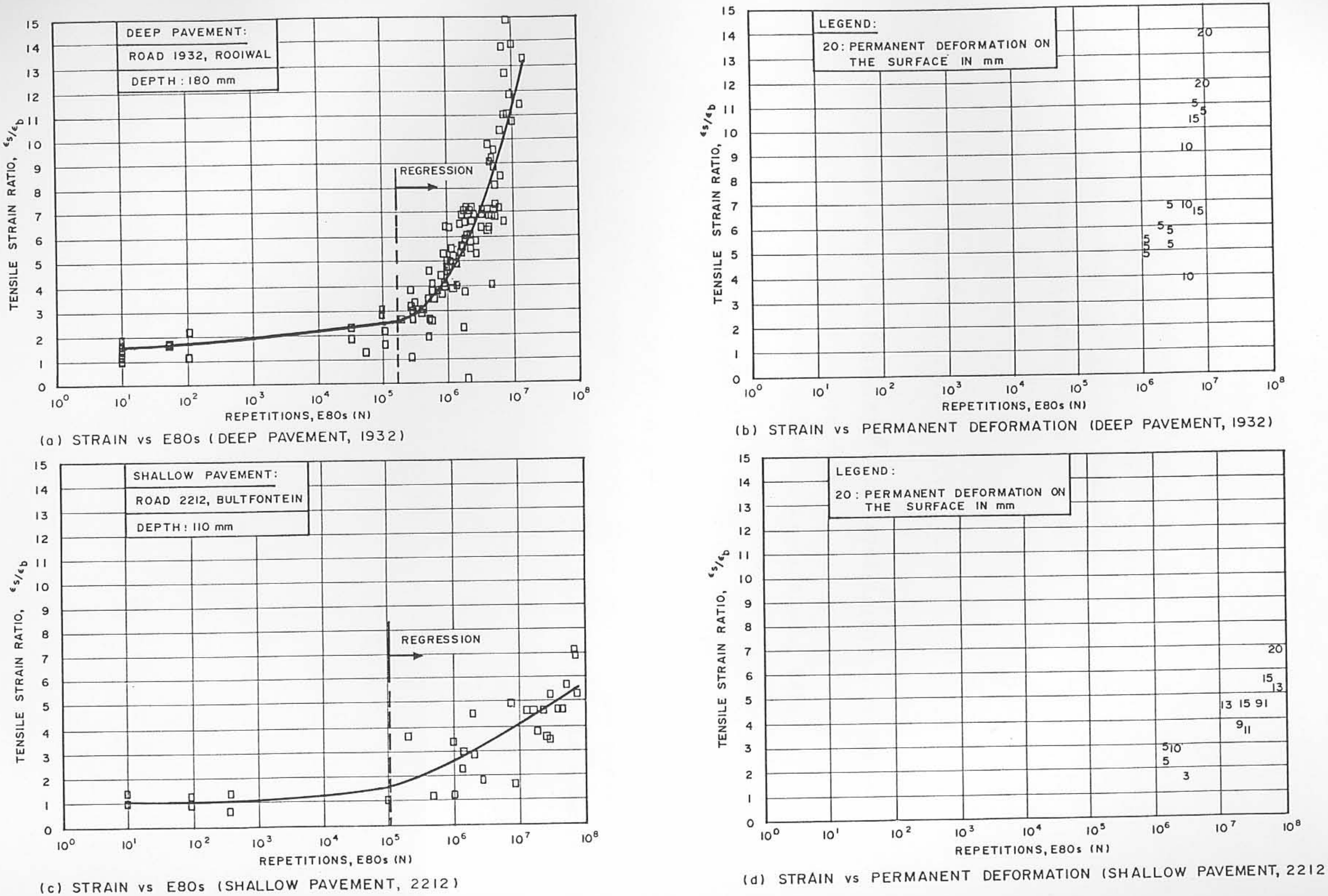


FIGURE 7.16

SUMMARY OF THE CHANGE IN MAXIMUM TENSILE STRAIN (ϵ_s) AT THE BOTTOM OF THE BASE LAYER AT VARIOUS STAGES OF HVS TRAFFICKING, TOGETHER WITH THE ASSOCIATED PERMANENT DEFORMATION (RUT) ON THE SURFACE OF BOTH THE DEEP AND SHALLOW PAVEMENT

The strain ratio is defined as follow:

Let ϵ_i = the maximum calculated induced tensile strain
d = a factor to allow for the effect of shrinkage cracking in cemented layers (1,15 in this case)
ϵ_s = modified induced maximum tensile strain
$= d \times \epsilon_i$
Further if ϵ_b = tensile strain at break, then
ϵ_s/ϵ_b = the strain ratio

Detail of this argument is given elsewhere (Otte, 1978, Freeme et al, 1984, De Beer, 1985).

The ϵ_b for the cemented layers of the deep pavement was assumed to be 125 $\mu\epsilon$, because it was impossible to obtain intact beam specimens from the test sections before testing (Freeme et al, 1984). The figure indicates that the tensile strain ratio (ϵ_s/ϵ_b) increases with trafficking, ie ϵ_s increases. This increase, as with the decrease in modulus, is directly related to the breakdown of the base layer, and is also related to the subsequent permanent deformation associated with the pavement. The associated deformation levels are illustrated in Figure 7.16 (b). A regression analysis was performed on the strain/repetitions data from approximately 160 000 repetitions, and a relatively good (R^2 = 75 per cent) parabolic regression function was obtained. The relationship is given below:

$$(\epsilon_s/\epsilon_b) = 10^x \qquad \text{.......................................7.1}$$

where $x = 2{,}640222(Lg(N))^2 - 27{,}4764(Lg(N)) + 74{,}08693$
(ϵ_s/ϵ_b) = induced strain ratio
N = number of equivalent repetitions, E80s

7.5.2 Shallow pavement

Similar behaviour in the strain ratio versus repetitions were obtained for the shallow pavement, but in this case it was possible to measure the breaking strain from intact field specimens. The measured breaking strain was 152 $\mu\epsilon$, with a standard deviation of 53 $\mu\epsilon$. The change in strain ratio with trafficking on the shallow pavement is illustrated in Figure 7.16 (c). The subsequent permanent deformation is illustrated in Figure 7.16 (d). Although the strain ratio on the shallower pavement appears to be lower than that for the deep pavement, similar permanent deformation resulted. The strain ratio, however, is dependent on the ϵ_b, and, therefore variations in ϵ_b could result in variations in the strain ratio, without the permanent deformation being notably effected.

The regression (also parabolic, R^2 = 74 per cent) was performed on the data from approximately 100 000 repetitions, and is given below:

$$(\epsilon_s/\epsilon_b) = 10^x \quad \ldots\ldots 7.2$$

where $x = 0{,}208100(Lg(N))^2 - 1{,}24812(Lg(N)) + 2{,}56910$

(ϵ_s/ϵ_b) = induced strain ratio

N = number of equivalent repetitions, E80s

7.5.3 Fatigue life determination

The current method for calculating the fatigue life of cementitious layers in the South African mechanistic design method was proposed by Otte (1978). The calculation is based on the tensile strain ratio and is given below:

$$N_f = 10^{9{,}1(1 - \epsilon_s/\epsilon_b)} \quad \ldots\ldots 7.3$$

where N_f = fatigue life during the pre - cracked (intact) state.

and ϵ_s/ϵ_b = initial induced strain ratio

The above relation was obtained from a literature study based on a variety of laboratory tests, and was unverified for actual field conditions for lightly-cementitious layers until this study. Over the past few years experience with normal roads as well as with the HVS testing programme indicated that the above relation underestimates the fatigue life of lightly-cementitious layers. Therefore, in this study, research was also aimed to verify this relation for normal field conditions. Data were accumulated on the fatigue life of both the deep and shallow pavements, and it was found that the effective fatigue in the cementitious base layers occurs at a deflection level between 0,5 mm to 0,75 mm, and an associated permanent deformation level of approximately 2 mm. Load tests at the start of the HVS testing on a number of sections were done at various loads, viz 40 kN, 70 kN and 100 kN, from which moduli where back-calculated and initial strain ratios determined. The same loading was then used to traffic the test sections and the effective fatigue life determined, using the above limiting conditions. The test sections and results used for this analysis are summarised in Table 7.1.

Table 7.1 Summary of the effective fatigue life information

HVS Section	Test load (kN)	Initial strain ϵ_i ($\mu\epsilon$)	Number of repetitions to real fatigue*
260A4	40	170	$1,0 \times 10^6$
274A4	40	175	$1,0 \times 10^6$
275A4	40	142	$1,25 \times 10^6$
289A4	40	128	$1,0 \times 10^6$
294A4	40	121	$1,0 \times 10^6$
275A4	40	175	$1,0 \times 10^6$
289A4	70	251	$0,2 \times 10^6$
294A4	100	243	$0,1 \times 10^6$
306A4	40	159	$1,1 \times 10^6$
307A4	40	147	$1,1 \times 10^6$
308A4	40	138	$1,1 \times 10^6$
308A4	100	446	30 000
337A4	150	434	5 000
338A4	150	382	5 000

* Number of repetitions to achieve a permanent deformation of 2 mm on the surface of the pavement, and a total standard surface deflection level of approximately 0,5 to 0,75 mm.

In Figure 7.17 the relationship between the strain ratio and number of repetitions to reach the defined fatigue failure condition is illustrated. The current fatigue relationship proposed by Otte (1978) is also shown. The figure shows that much higher strain ratios must be mobilised to reach effective fatigue failure than those of Otte. The relationship (R^2 = 94 per cent) between strain ratio and effective fatigue life, N_{ef}, is given below:

$$N_{ef} = 10^{7,19(1 - \epsilon_s/8\epsilon_b)} \qquad \text{.......................7.4}$$

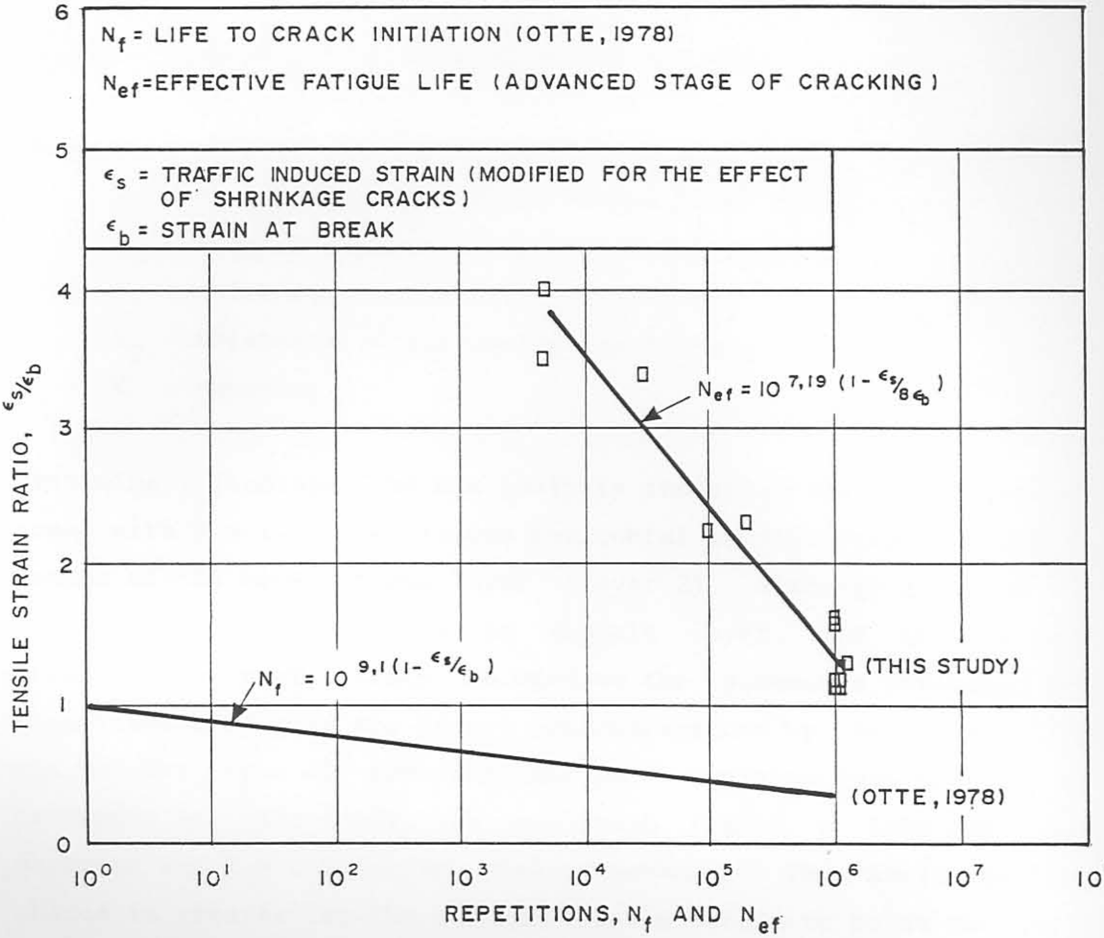


FIGURE 7.17

COMPARISON OF THE RELATIONSHIPS BETWEEN MAXIMUM TENSILE STRAIN RATIO (ϵ_s/ϵ_b) AND NUMBER OF STRAIN REPETITIONS TO INITIATE AND EFFECTIVE FATIGUE CRACKING IN CEMENTED LAYERS

This relationship (Equation 7.4) is proposed for pavements with lightly cementitious layers instead of the Otte relationship (Equation 7.3), as the latter grossly underestimates the actual (effective) fatigue life by factors of more than one million!. This was also observed by Jordaan (1988) on a similar pavement structure, but with a 35 mm asphalt surfacing on the cemented base. Based on the results of this HVS test, Jordaan suggested increasing the laboratory determined strain at break by a factor of 4,7 in order to determine the life to visible cracking on the surface of this pavement. This case is a further indication that the existing fatigue relationship (Equation 7.3) needs amendment if used on lightly cementitious base pavements.

7.5.4 Maximum tensile strain

As an aid to establish the position of the maximum horizontal tensile strain in cementitious layers, Jordaan (1988) suggested using the following relationship:

$$(E_3/E_2)^2(h_1(E_1/E_3)^{1/3} + h_2(E_2/E_3)^{1/3}) < K \quad (\mathrm{mm}) \; \ldots\ldots\ldots\ldots 7.5$$

where E_1 = elastic modulus of the first layer (MPa)
E_2 = elastic modulus of the second layer (MPa)
E_3 = elastic modulus of the third layer (MPa)
h_1 = thickness of the first layer (mm)
h_2 = thickness of the second layer (mm)
K = constant

Preliminary findings from his analysis indicated that if Equation 7.5 is true, with $K \approx 128$, the maximum horizontal tensile strain occurs at the bottom of the cementitious layer (layer 2). Although Equation 7.5 was derived for layer 1, as an asphalt layer, and the second the cementitious layer, it may be used on the pavements evaluated in this dissertation because the layers are represented by their elastic moduli and not the type of material, per se. Applying Equation 7.5 to the pavements in this study, it was found that $K \approx 1500$ for the deep pavement and $K \approx 250$ for the shallow pavement. This indicates that the chance is greater for the maximum tensile strain to be at the bottom of the cementitious layer (smaller K) in the shallow pavement, than the deep pavement (larger K). The HVS test results indicated that the cementitious base layers of the deep pavement failed in compression, whilst that of the shallow pavement failed in fatigue, which confirms this conclusion.

The value of K ≈ 128 for the maximum strain to occur at the bottom of the cementitious layer suggested by Jordaan (1988) was based on a limited number of results. However, more research is needed to establish a general value for K, or different K values for different pavement compositions.

Future research, however, should also be guided towards the development of a general fatigue relationship (similar to Equation 7.4). This should be done for pavements with lightly-cementitious as well as bitumin-emulsion-treated bases if varying thickness of asphalt surfacings are to be included because the research discussed in this dissertation is related to pavements with relatively thin asphalt surfacings only.

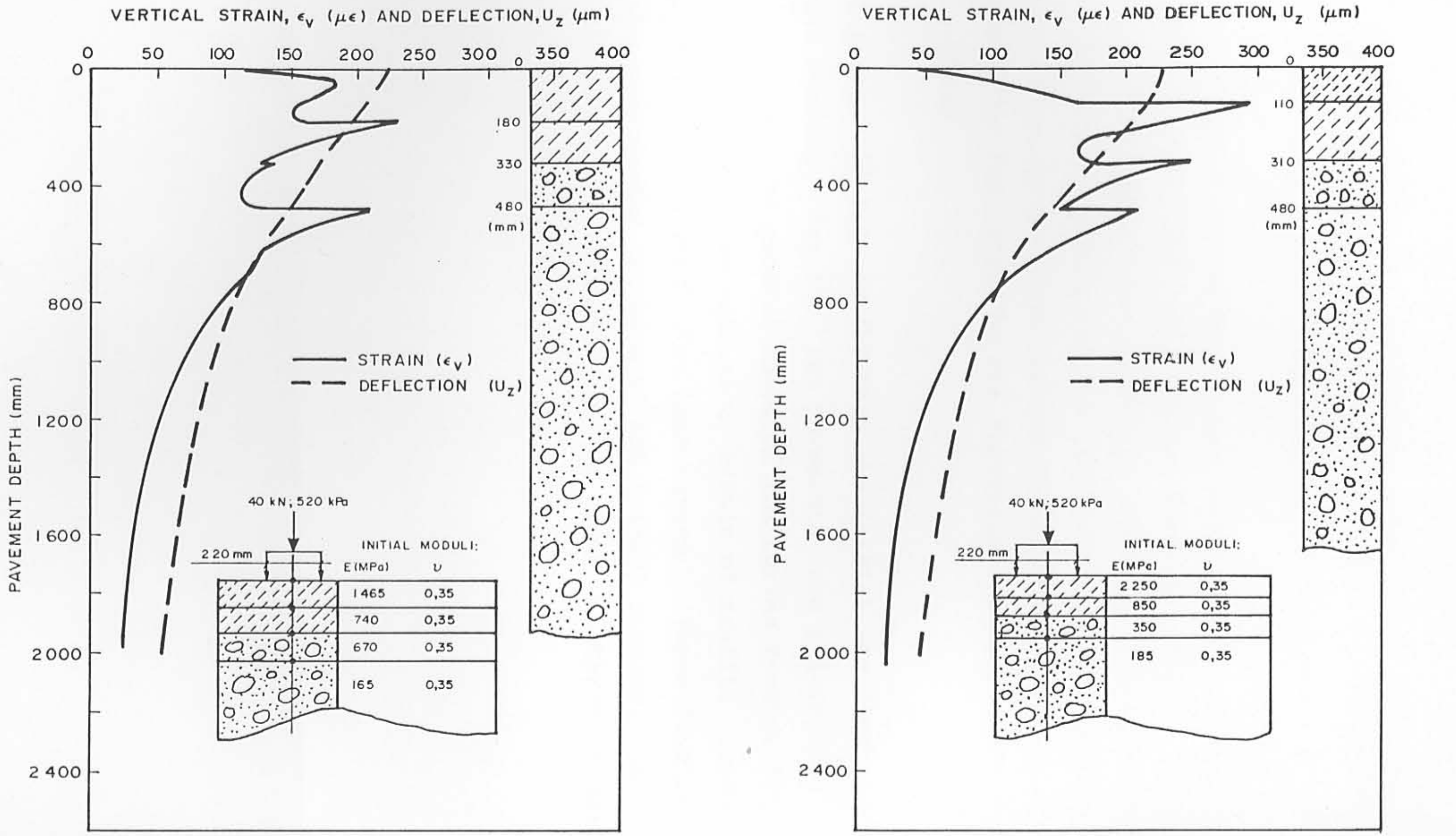
7.6 VERTICAL COMPRESSIVE ELASTIC STRAIN

Similar to cracking, another primary indicator of pavement behaviour is permanent deformation. Since the early 1960s the vertical elastic strain, ϵ_v (based on the linear elastic theory) has been used to develop limiting strain criteria for subgrade soils in order to control permanent deformation in these layers and eventual serviceability of pavements (Dorman, 1962; Dorman et al, 1964; Finn et al, 1966; Edwards et al, 1974; ; Paterson et al, 1978, Visser, 1981). Although ϵ_v is an elastic parameter, it appears that it may be used readily as a predictor of permanent deformation and as a limiting criterion (Wang et al, 1980).

As in the case of tensile strain, one of the aims of this study was to verify the current deformation criteria used in the South African mechanistic design method, for the pavements evaluated in this study. The attempts to achieve this end follow in the next paragraph.

7.6.1 Vertical compressive strain profiles

Figures 7.18 (a) and (b) illustrate examples of the typical initial vertical strain profiles on the centre line through the deep and shallow pavements. The associated deflection profiles (measured with



(a) DEEP PAVEMENT (MDD8; HVS SECTION 275A4, ROAD 1932; ROOIWAL)

(b) SHALLOW PAVEMENT (MDD4; HVS SECTION 306A4, ROAD 2212; BULTFONTEIN)

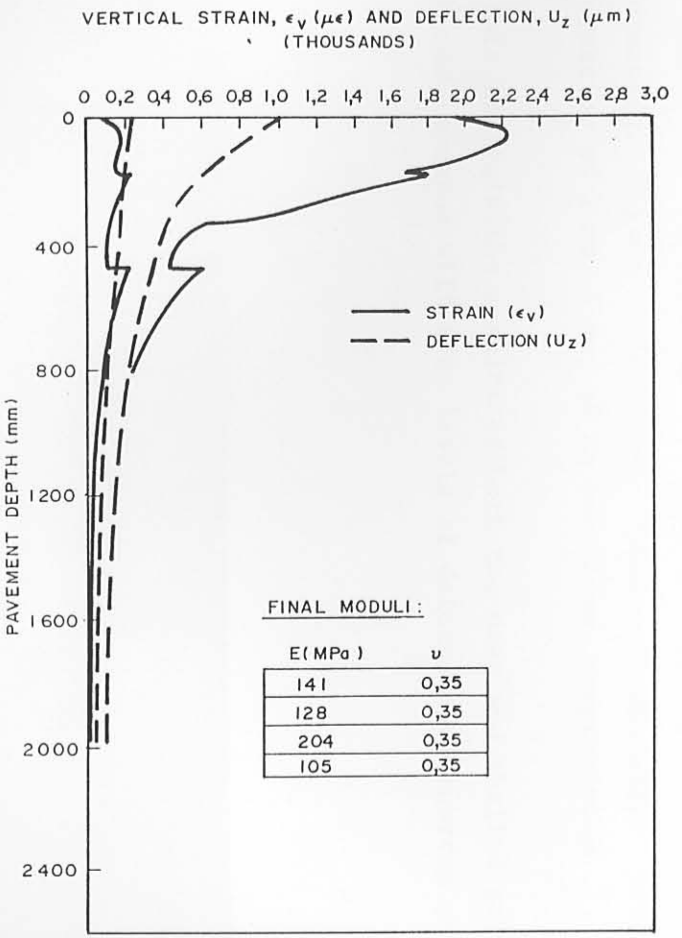
FIGURE 7.18

EXAMPLES OF THE INITIAL (N=10) VERTICAL STRAIN (ϵ_v) AND DEFLECTION (U_z) PROFILES FOR THE DEEP AND SHALLOW PAVEMENTS ANALIZED IN THIS STUDY the MDD and back-calculated effective elastic moduli) are also shown on the figures, since the surface deflection is the integration of the vertical strain with depth. The area under the strain curves gives the central deflection. The effect of the different layers, including the dominant effect of the subgrade on the central deflection, is therefore evident for both pavements.

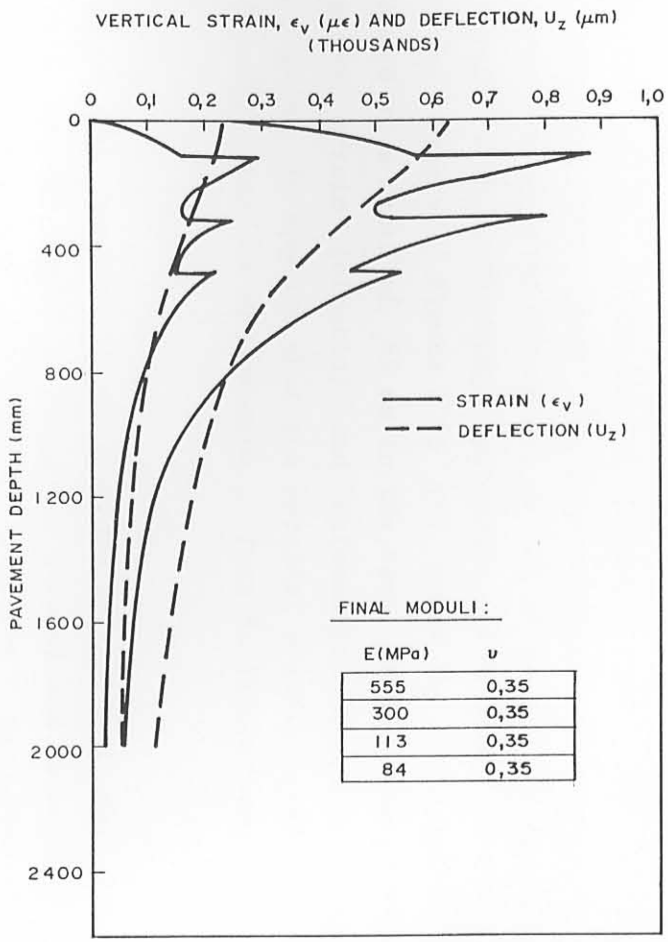
Not only is the area (deflection) important, but also the shape of the strain curve with depth. Inspection of the strain curve of the deep pavement in Figure 7.18 (a) indicates that peak strains occurred at three depths, viz 65 mm (within the base); 181 mm (top of the subbase) and 481 mm (top of the subgrade). For the shallow pavement, the peak strains also occurred at three but different depths, at 111 mm (top of the subbase); 311 mm (top of the selected layer) and 481 mm (top of the subgrade).

According to the observed and measured failure mechanisms for these two types of pavements, viz crushing in the base of the deep pavement, and fatigue failure of the base for the shallower pavement, it appears that these peak strains may assist in explaining these behaviours. For the deep pavement (Figure 7.18 (a)) crushing of the base layer occurred within the upper 50 mm to 75 mm, in the region where a peak vertical strain occurred. For the shallow pavement (Figure 7.18 (b)) fatigue failure of the base layer occurred, which may be explained by the peak vertical strain on top of the subbase directly underneath the base. Deterioration in the base reduces the effective protection of the lower layers as is indicated by the increased vertical strain in the top of the subbase.

In Figures 7.19 (a) and (b) both the initial and final (end of HVS test) strain profiles for the two pavements are illustrated. The figures indicate that the general shape of the curves remains constant and that the peak strains increase as a result of traffic loading, because of reduced effective moduli for most layers. These final peak strains are at the same depths as those of the initial strains, and tend to explain the observed failure mechanisms for these two types of pavement.



(a) DEEP PAVEMENT (MDD8; HVS SECTION 275A4; ROAD 1932, ROOIWAL)



(b) SHALLOW PAVEMENT (MDD4; HVS SECTION 306A4; ROAD 2212, BULTFONTEIN)

FIGURE 7.19

INITIAL AND FINAL VERTICAL STRAIN AND DEFLECTION PROFILES OF THE DEEP AND SHALLOW PAVEMENTS ANALIZED IN THIS STUDY

7.6.2 Vertical strain behaviour

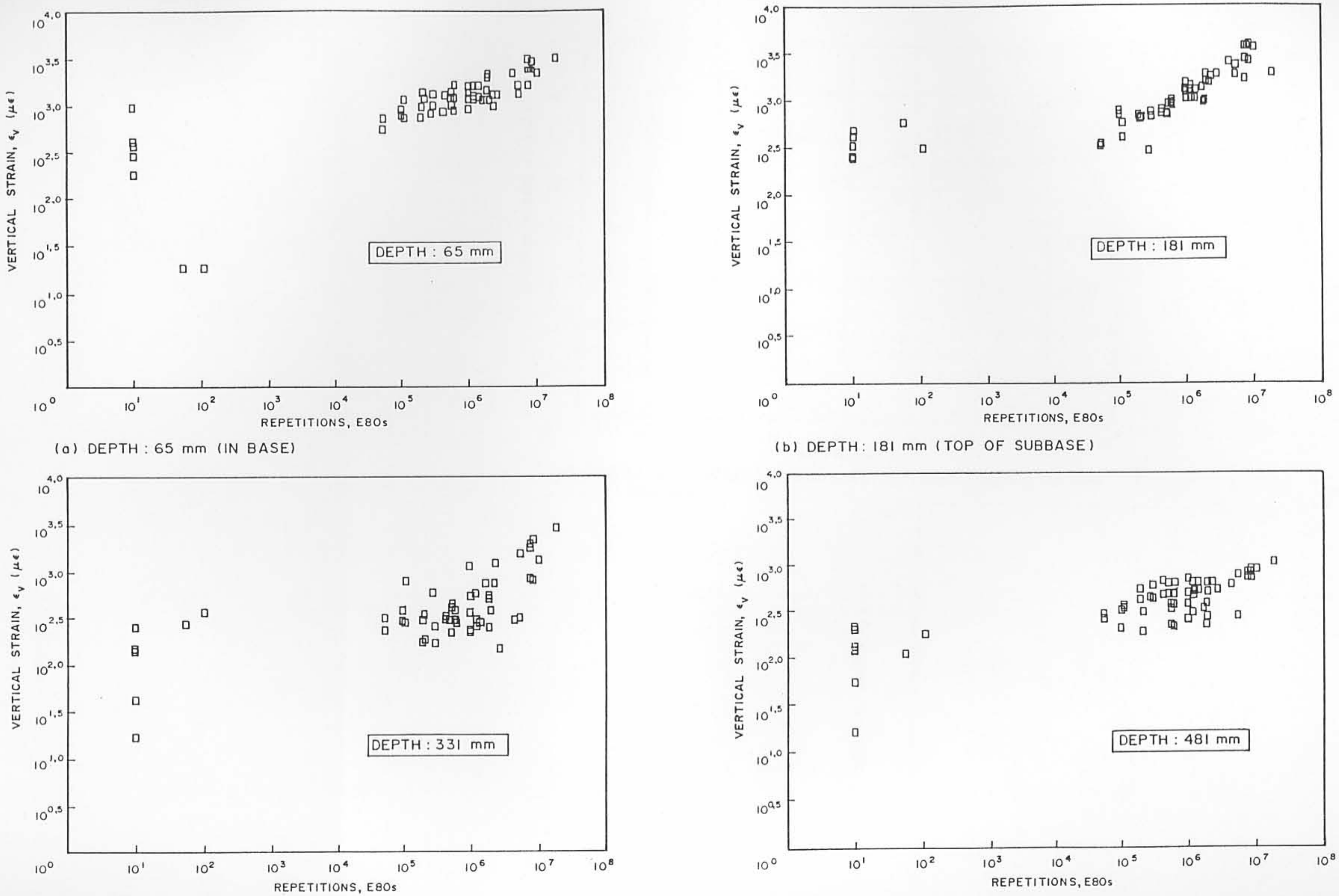
Based on the above, the changes in peak vertical strains at various depths for the deep pavement with trafficking were investigated further. As with the tensile strain behaviour discussed in Paragraph 7.5, the vertical strains also increase with trafficking. Figure 7.20 illustrates the increase at various depths with trafficking. According to these results there appears to be a linear relationship, on a log-log plot between vertical strain and traffic. The increase in strain is again a manifestation of the change in the pavement system owing to traffic loading, and as with tensile strain, it confirms that these pavements are subjected to <u>variable (increasing)</u> strain repetitions throughout their life, even under <u>constant loading</u> conditions.

7.6.3 Vertical strain and permanent deformation

In Figure 7.21 (a) a comparison of the strain/traffic relationship at two depths is illustrated. The figure clearly shows that the strain at 65 mm is greater than the strain at 481 mm depth. During this increase in strain with trafficking, permanent deformation is also associated with the change. Figure 7.21 (b) shows the associated deformation measured at a depth of 65 mm in the base of the deep pavement. Since both the strain (elastic) and deformation (plastic) increase with trafficking, it appears that the vertical elastic strain is adequate for evaluating permanent deformation. This is illustrated in Figure 7.21 (c).

Based on this result, it was decided to evaluate the current failure criteria for the subgrade, using the strains and deformations at various depths in both deep and shallow pavements. In order to verify the current subgrade failure criteria used in South Africa (Freeme et al, 1984), the initial induced vertical elastic strains were plotted against the number of repetitions to produce various levels of permanent deformation. Figure 7.22 shows these relationships for levels of deformation of 1 mm, 5 mm, 10 mm and 15 mm, respectively.

In this analysis the results of both the deep and shallow pavement were combined, because different levels of deformation occurred at different



(a) DEPTH : 65 mm (IN BASE)

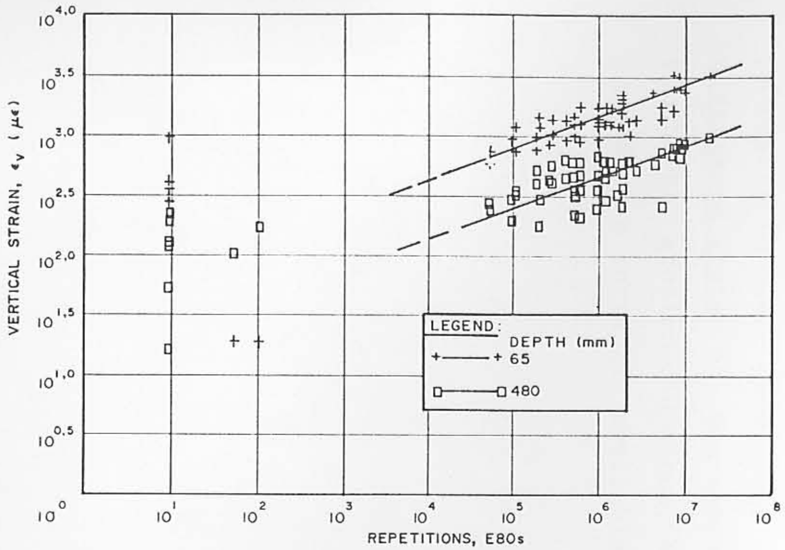
(b) DEPTH : 181 mm (TOP OF SUBBASE)

(c) DEPTH : 331 mm (TOP OF SELECTED LAYER)

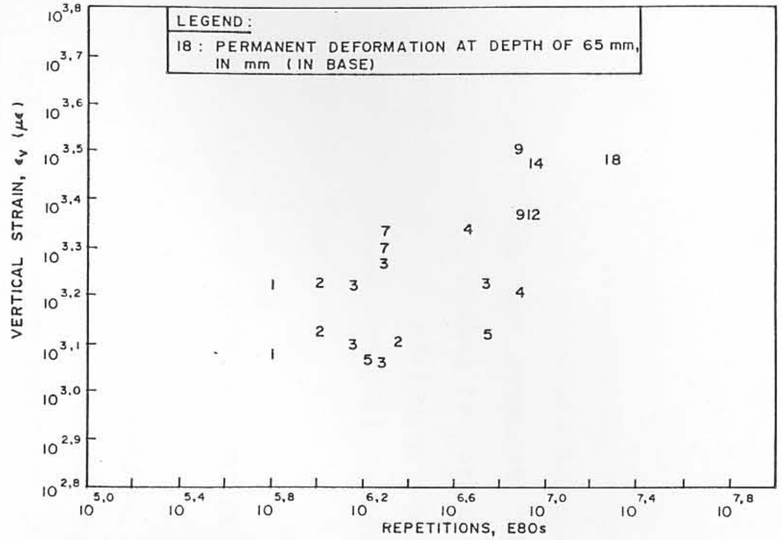
(d) DEPTH : 481 mm (TOP OF SUBGRADE)

FIGURE 7.20

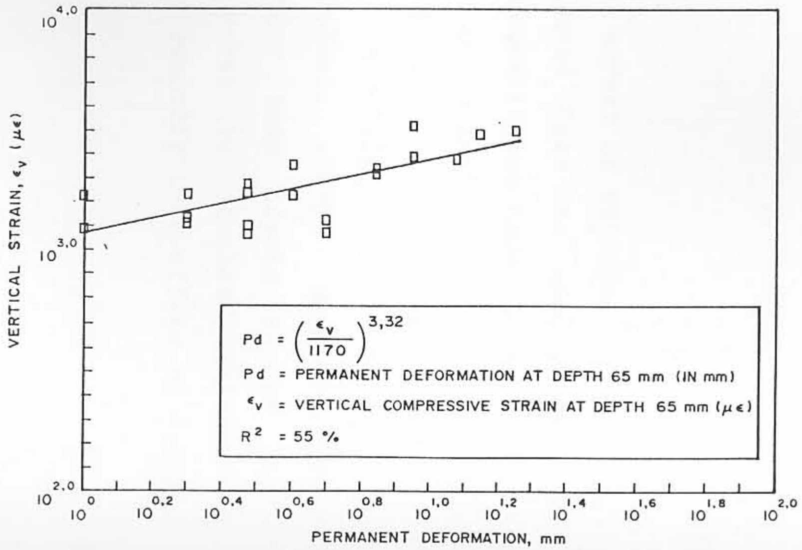
SUMMARY OF THE CHANGE IN VERTICAL COMPRESSIVE STRAIN (ϵ_v) AT VARIOUS STAGES OF HVS TRAFFICKING ON THE VARIOUS TEST SECTIONS ON THE DEEP PAVEMENT (ROAD 1932, ROOIWAL)



(a) COMPARISON BETWEEN STRAINS AT DIFFERENT DEPTHS (65 mm AND 480 mm)



(b) ϵ_v AND PERMANENT DEFORMATION



(c) RELATIONSHIP BETWEEN VERTICAL COMPRESSIVE STRAIN AND PERMANENT DEFORMATION AT A DEPTH OF 65 mm

FIGURE 7.21

SUMMARY OF THE CHANGE IN VERTICAL COMPRESSIVE STRAIN (ϵ_v) WITH DEPTH, REPETITIONS AND ASSOCIATED PERMANENT DEFORMATION ON THE DEEP PAVEMENT (ROAD 1932, ROOIWAL) depths in these pavements and provided an adequate data base. According to Figure 7.22, the slope of the relationships appears to be independent of the magnitude of deformation and its depth of measurement. Although the accuracy of regressions was not very high, especially for the upper levels of deformation, the general trend of the relationships appears to be the same.

In Figure 7.23 the relationships for different levels of deformation is given, and also compared to failure criteria suggested by Paterson et al (1978) and Shell (Claessen et al, 1977).

In general, there is a high correlation between the results of this study and other criteria. The current criteria may therefore be used with greater confidence on the pavement types investigated in this study, particularly those of Paterson et al (1978) for a deformation of 18 mm.

A further interesting fact is that, according to the results of this study, the relationships indicated here appear to be independent of depth, and may therefore be used to evaluate strains and deformations at any depth in the pavement. More research, however, is necessary to extent this finding to other pavement types, and also to study the possibility of including variable strain (increasing strain with trafficking) in the failure criteria, rather than constant strain, as is normally done.

7.7 EFFECT OF MOISTURE INGRESS

As with permanent deformation, the effect of moisture into the base and sub-layers of the pavements evaluated here was negligible during the intact state, especially for the cementitious base layers. This finding is different from that for cementitious subbase layers in bitumin-base pavements, where erodibility of these layers was identified as one of the most important parameters controlling the behaviour of these pavements in wet conditions (De Beer, 1985). However, during the distressed state of the cementitous base layers (either fatigue or crushing), erosion did occur, and excessive potholes resulted. Limited methods are currently available to quantify this effect of erosion

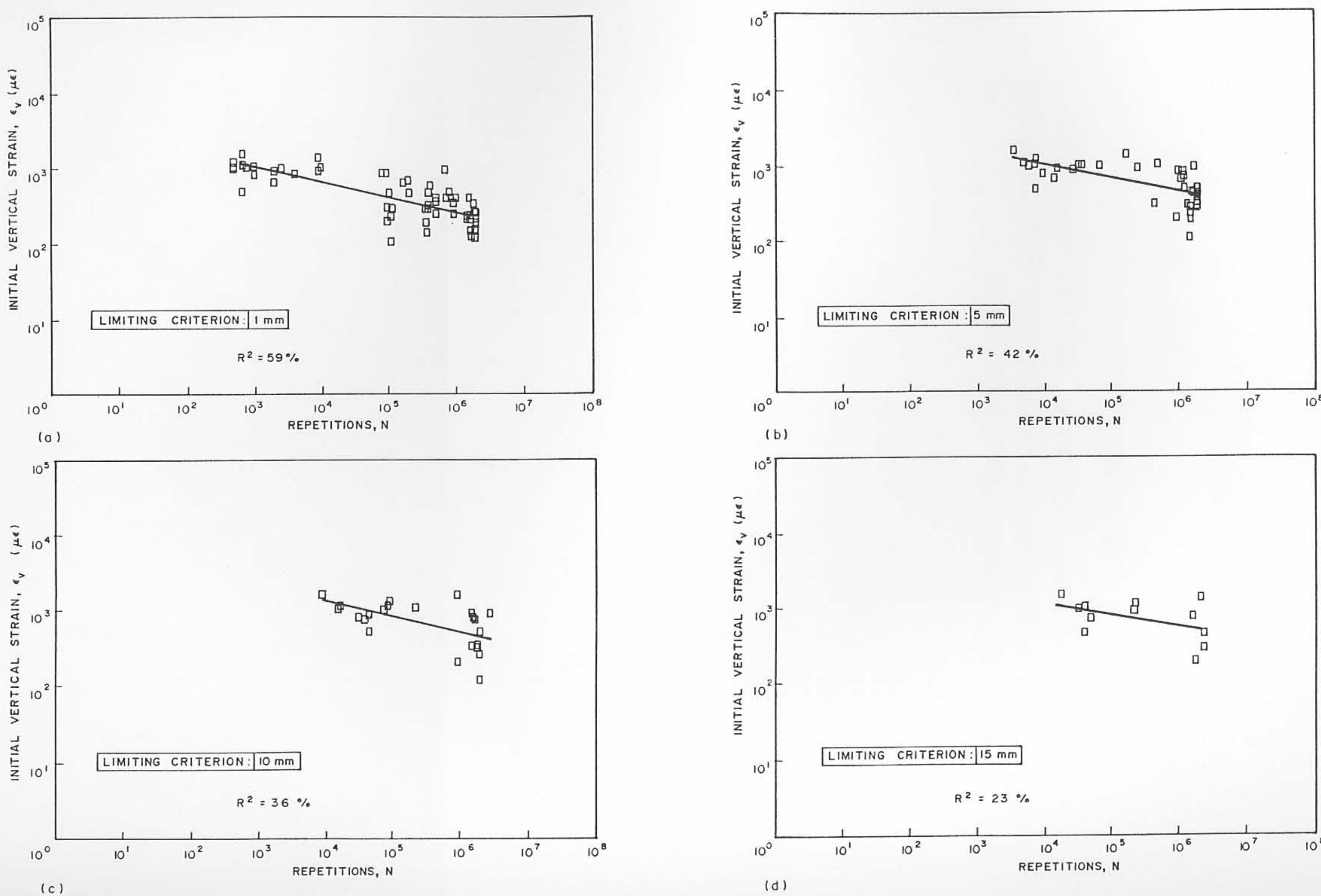


FIGURE 7.22

SUMMARY OF THE INITIAL MAXIMUM VERTICAL COMPRESSIVE STRAIN AT DIFFERENT LIMITING LEVELS OF PERMANENT DEFORMATION FOR BOTH THE SHALLOW AND DEEP PAVEMENT TESTED IN THIS STUDY

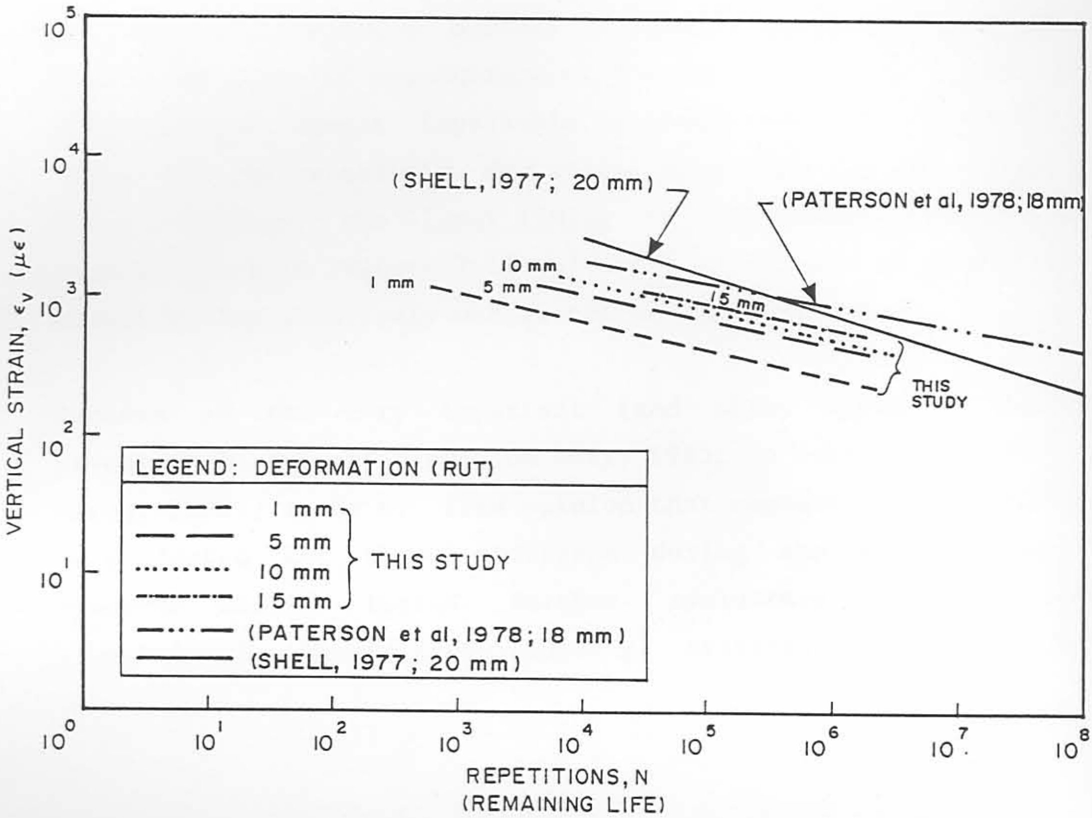


FIGURE 7.23

COMPARISON OF THE VERTICAL COMPRESSIVE STRAIN RESULTS FOR DIFFERENT DEFORMATION LEVELS TOGETHER WITH OTHER PUBLISHED CRITERIA in the laboratory, and the related aspects thereof. As a result of these earlier findings on erodibility of cementitious subbase layers (De Beer, 1985), and the HVS results discussed in this dissertation, an erodibility test was developed by the author. Detail of this, together with a detailed laboratory evaluation on a series of different materials/stabiliser systems, are discussed elsewhere (De Beer, 1989).

With this new method erodibility tests on the cementitious base material for both the deep and shallow pavements investigated here indicate that during the intact state the in situ material is non-erodible and very durable, although it is highly carbonated, according to the carbonation tests (Netterberg et al, 1984).

Where the resilient response of these pavements is concerned, it was found that the deflections on the surface increased during the intact and relatively dry states because of the crushing and fatigue of the base layer. With moisture ingress very rapid moisture accelerated distress (MAD) followed, resulting in pumping, loss of the surfacing and subsequent potholes (see Photo Plates D.6(c), D.9(d) and (c),and D.12(b) in Appendix D). During these states it was very difficult to monitor the resilient responses because of damage to the MDDs as well as the distressed state of the surface of the test sections. Therefore it was difficult and almost impossible to differentiate accurately between moduli in the relatively dry state from that in the relatively wet state. However, the lower limits of the moduli versus repetitions relationships in Figures 7.11 (b) and 7.12 (b) are an indication of the moduli during relatively wet states of these pavements.

Because of the very important (and often dramatic) behaviour of pavements in the wet state (De Beer, 1985; De Beer et al, 1987; Van Der Merwe, 1988), it is my firm opinion that cementitious materials should be evaluated for these conditions during the design phase. Future research should include further evaluation of the erodibility (durability) aspects and limiting criteria for these materials.

7.8 SUMMARY AND CONCLUSIONS

In this chapter the resilient responses of the two pavements evaluated are discussed. These responses include the measurement of surface and depth deflection and the calculation of radii of curvature. The pavements were modelled using the linear elastic theory, and the effective elastic moduli of the different layers were back-calculated from multi-depth deflection results. The moduli were determined initially and at various stages of trafficking, and were used to determine stresses and strains in the pavement layers.

Both maximum tensile strain and vertical compressive strain analyses were used to describe the behaviour of these pavements, together with associated permanent deformation. Both the current fatigue and subgrade limiting strain criteria were also evaluated and verified.

The following conclusions resulted from this study:

(a) The resilient response behaviour of both the deep and shallow pavements investigated here is mechanistically adequately described using the surface and depth deflection results as input to linear elastic modelling.

(b) The damaging effect of traffic loading is well illustrated by surface deflection, depth deflection profiles and calculated radii of curvature. Both crushing and fatigue failure of the cementitious base layers result in drastic changes in these indicators.

(c) Local failures around the MDD hole adversely influence the back - calculated effective elastic moduli. Deflections on the MDD and regular monitoring of the pavement at the MDD position should be done to detect faulty deflection measurements before moduli back - calculation.

(d) Inspection of the depth deflection (MDD) at various stages of trafficking assists satisfactorily in determining which layers contributed largely to the majority of the total surface deflection. In this way the effect of the subgrade on total surface deflection is also well illustrated.

(e) Weakening of the cementitious base layer as a result of traffic loading not only increases the total surface deflection, but also reduces the effective protection of the lower layers, and hence increases the contribution of the lower layers to the total surface deflection.

(f) The average reduction in the effective elastic modulus of the cementitious base of the **deep pavement** is described by a hyperbolic function when the layer changes from the **intact state** to the **crushed state**.

(g) The average reduction in the effective elastic modulus of the cementitious base of the **shallow pavement** is also described by a hyperbolic function when the layer changes from the **intact state** to the **cracked state**.

(h) The average effective elastic modulus after approximately one million E80s of the base of the deep pavement is 200 MPa to 300 MPa, while that of the shallow pavement is approximately 500 MPa.

(i) The effective elastic moduli of the lower layers of both pavements also decrease with an increase in trafficking.

(j) The maximum tensile strain at the underside of the base of both pavements increased as a result of traffic loading and is associated with total permanent deformation on the surface of up to 20 mm.

(k) The concept of **effective fatigue life** for lightly-cementitious layers is introduced (Equation 7.4), and appears to describe the real fatigue life of these layers better than the current practise done by an unverified relation (Equation 7.3). The effective fatigue life of lightly cementitious base layers corresponds to a permanent deformation of approximately 2 mm and a total standard surface deflection between 0,5 mm and 0,75 mm for thin-surfaced (single and double seals) pavements.

(l) Initial and final vertical compressive strain profiles from the pavements evaluated assist in explaining the difference in measured behaviour of these pavements. For the deep pavement, maximum compressive strains occurred at a depth of approximately 65 mm in the base where crushing failure was noted, while for the shallow pavement, maximum strains occurred in the top of the subbase where large permanent deformations occurred because of the fatigue failure and "punch in" of the base layer into the subbase.

(m) Empirically, the vertical (elastic) compressive strain appears to be a valid indicator of permanent (plastic) deformation behaviour at any depth in the pavements evaluated here.

(n) The current limiting subgrade strain criterion used in the South African mechanistic design method appears to be accurate, and may also be used to evaluate permanent deformations at any depth in the pavements evaluated in this study.

7.9 REFERENCES

Basson, J E B (1985). The measurement of deflection and curvature of a road surface when loaded by a standard axle. Unpublished internal Technical Note TP/100/85, New Test Methods, NITRR, July 1985.

Brown, S F, Tam, W S and Brunton, J M (1987). Structural evaluation and overlay design: Analysis and Implementation. Proceedings of the Sixth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, July 1987.

Claessen, A I M, Edwards, J M, Sommer, P and Ugé (1977). Asphalt Pavement Design: The Shell Method. Fourth International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, August 1977.

De Beer, M (1985). Behaviour of Cementitious Subbase Layers in Bitumen Base Road Structures. M(Eng), Faculty of Engineering, University of Pretoria, Pretoria, 1985.

De Beer M and Horak E (1987). The effect of poor drainage on pavement structures studied under accelerated testing. Proceedings of the Annual Transportation Convention (ATC 1987), Pretoria, August 1987, Vol. 5B, Paper 5B/1, 26 pp, 1987.

De Beer, M (1989). Aspects of erodibility of lightly cementitious materials. Research Report DVPT 39, DRTT, CSIR, Pretoria, April 1989.

De Beer M, Horak E and Visser A T (1988). The Multi - Depth Deflectometer (MDD) System for determining the Effective Elastic Moduli of Pavement Layers. Unpublished paper accepted for publication in a ASTM Special Technical Publication (STP) of the First International Symposium on Nondestructive Testing of Pavements and Backcalculation of Moduli, Baltimore, USA, 1988.

Dorman, G M (1962). The extension to practise of a fundamental procedure for the design of flexible pavements. Proceedings of the International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, August, 1962.

Dorman, G M and Metcalf, C T (1964). Design curves for Flexible Pavements based on Layered Theory. Highway Research Record, No. 71, 1964.

Edwards, J M and Valkering, C P (1974). Structural Design of Asphalt Pavements for Road Vehicles - the Influence of High Temperatures. Highways and Road Construction, February 1974.

Freeme, et al (1984). Symposium on: Recent findings of Heavy Vehicle Simulator Testing. Proceedings of the Annual Transportation Convention (ATC 1984), Pretoria, August 1984.

Finn et al (1977). Development of Pavement Structural Subsystems. Final Report, Project 1-10B, NCHRP, February 1977.

Jordaan, G J (1988). Analysis and development of some pavement rehabilitation design methods. Ph.D Dissertation, Department of Civil Engineering, Faculty of Engineering, University of Pretoria, Pretoria, 1988.

Kadar, P (1988). Findings and results of three years accelerated testing of typical heavy duty pavements - an overview. Proceedings of the 14th ARRB Conference, Volume 14, Part 8, 28 August - 2 September 1988, Canberra, Australia, 1988.

Netterberg, F and Paige - Green, P (1984). Carbonation of lime and cement stabilised layers in road construction. Technical Report R5/3/84, NITRR, CSIR, Pretoria, 1984.

Otte, E (1978). A Structural Design Procedure for Cement-Treated Layers in Pavements. DSc Dissertation, Faculty of Engineering, University of Pretoria, Pretoria, 1978.

Paterson, W D O and Maree, J H (1978). An interim mechanistic procedure for the structural design of asphalt pavements. Technical Report RP/5/78, NITRR, 1978.

Van Der Merwe, C J (1988). The Need for Effective Drainage. Technical note I/FP/26/88 (Unpublished), Division for Roads and Transport Technology, DRTT, CSIR, Pretoria, 1988.

Visser, A T (1981). An evaluation of unpaved road performance and maintenance. Ph.D Dissertation, Faculty of the Graduate School, University of Texas, Austin, May 1981.

Wang, M C and Gramling, W L (1980). Distress behaviour of flexible pavements that contain stabilized base courses. Transportation Research Record No. 755, Washington, 1980.