APPENDIX A

EVALUATION OF DEFLECTION BASIN CURVE FITTING PROCEDURES

1 MODELS USED FOR CURVE FITTING

Various mathematical and physical models available, were selected and tested using the prepared RSD data sets. The discussion of these models are divided into two subgroups namely; Linear and non-linear models, based on the procedures of regression analysis used. In both cases the SPSS Statistical Package for the Social Sciences (Nie et al., 1975; Robinson, 1984) was used to do linear and non-linear regression analyses. The reason for the choice of the SPSS package was the compatibility with the computer system where the deflection basin data sets are stored. Each of the models used are briefly discussed in terms of the results.

1.2 Linear curve fitting model

If a typical set of deflection basin results are plotted on a logarithmic versus linear scale, like in Figure A.1, some guiding observations can be made as to what types of models can be used. There is in general a tendency towards a straight line, although as shown, it is still slightly curved. Based on these observations, the following models as shown in Table A.1, were used. The values of the related parameters are also given as derived from the linear regression model in the SPSS package (Nie et al., 1975) for a typical data set.

The results of these models for typical data sets are shown in Figures A.2 and A.3. Before even considering the statistical evaluation, it can be seen that there are some shortcomings in some of these models.

Figure A.2 illustrates that models 1 and 2 (as defined in Table A.1) are only applicable to the area of positive curvature. Model 2 is however able to be accurate over a wider area (10 to 300 mm). Figure A.3 shows that if model 1 is used over a wider area than that covered by the positive curvature of the deflection basin, it can easily lead to an ill fit. For that reason models 1 and 2

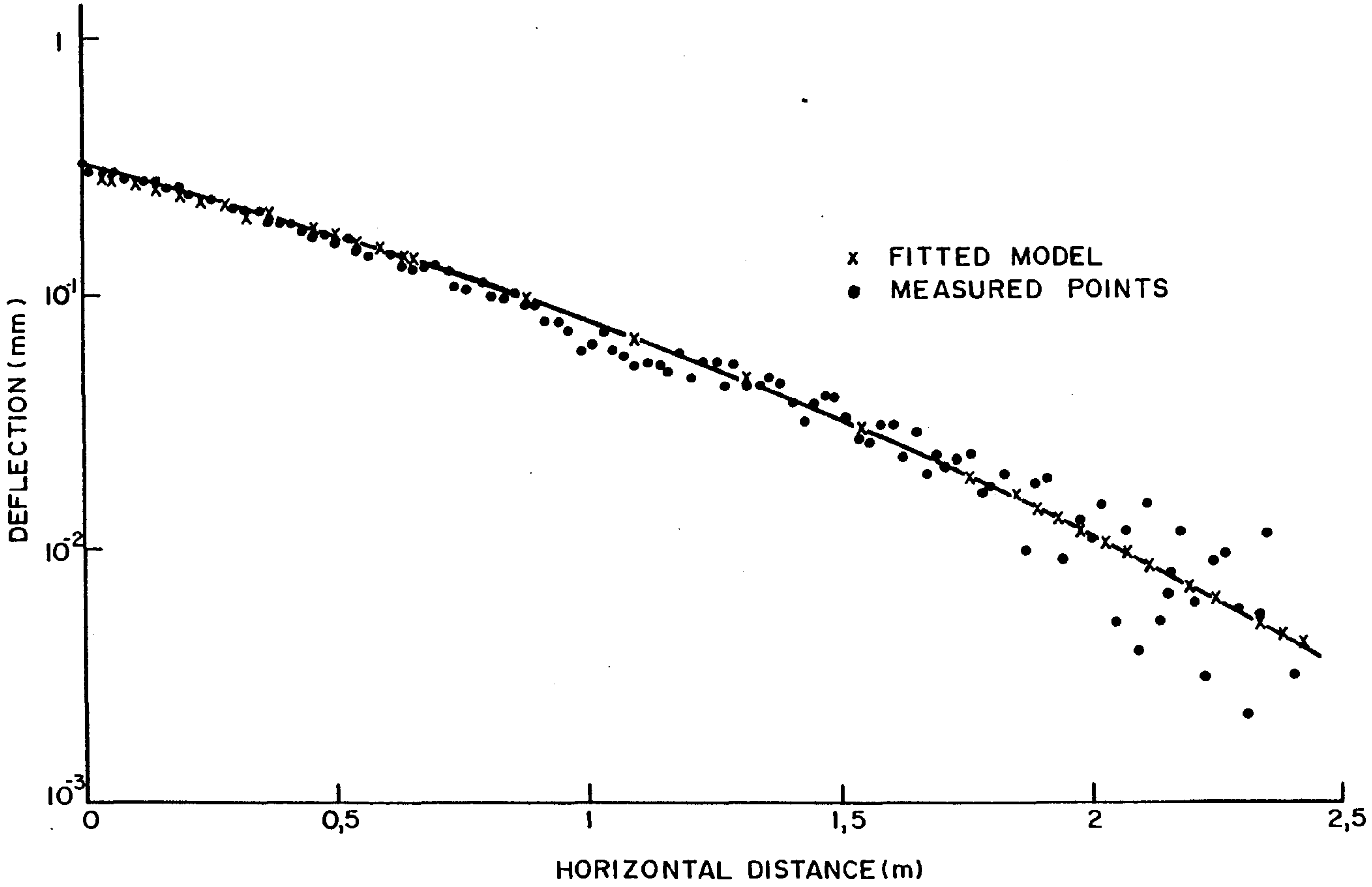

FIGURE A.1

TYPICAL LOG VERSUS LINEAR PLOT OF RSD DEFLECTION BASIN MEASUREMENTS

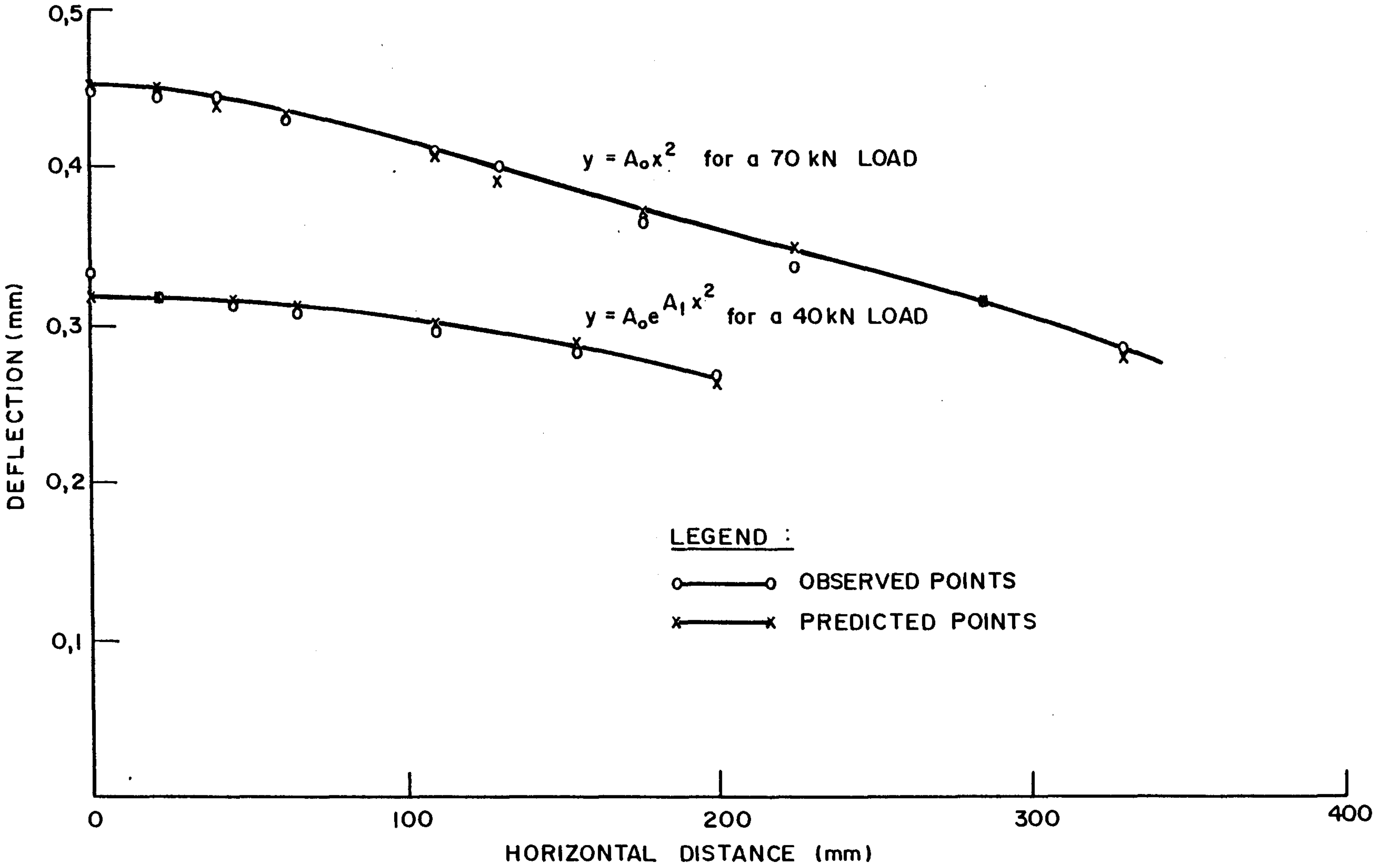

FIGURE A.2

TYPICAL PLOTS OF LINEAR REGRESSION MODELS FITTED TO THE POSITIVE CURVATURE

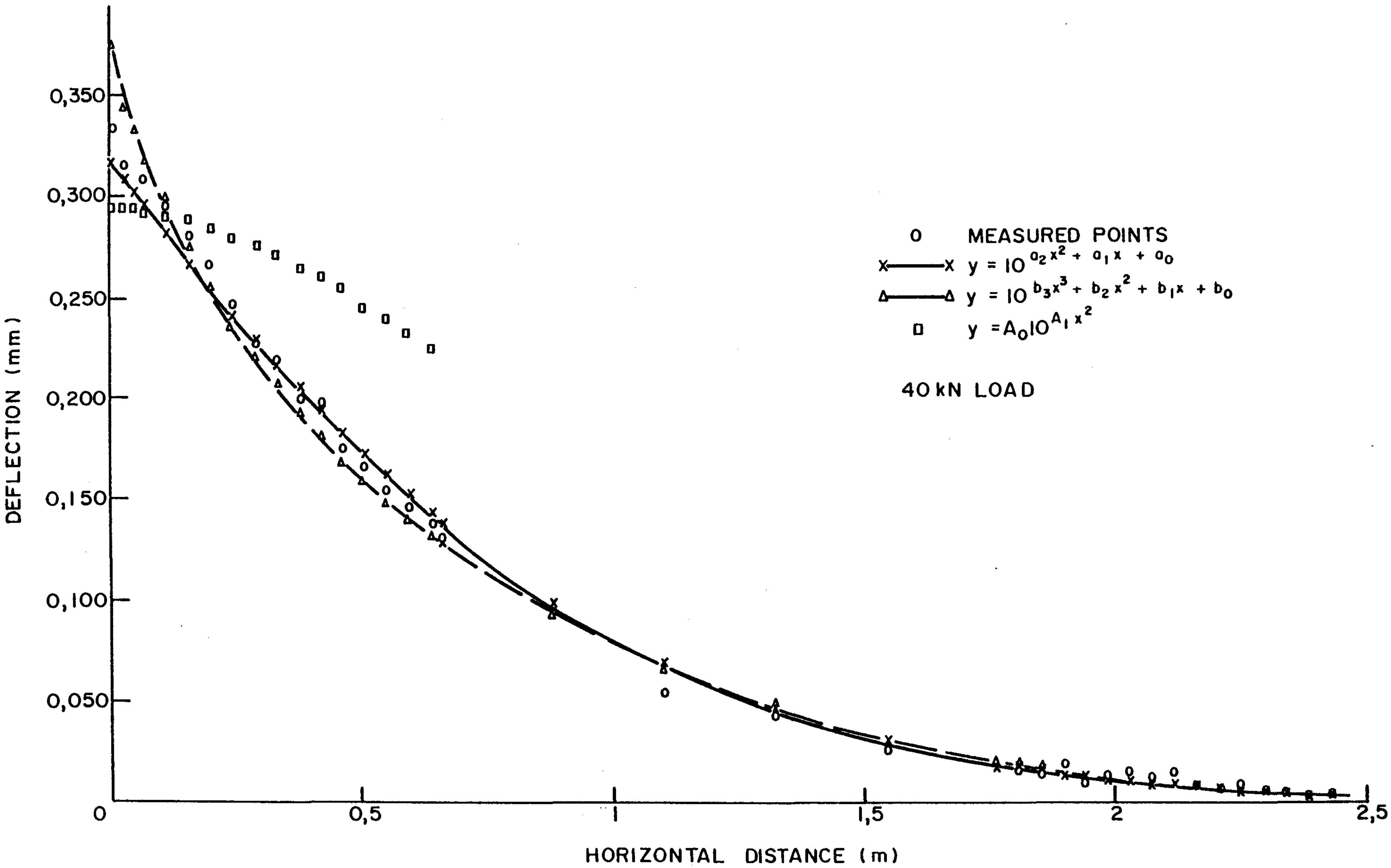

FIGURE A.3

TYPICAL PLOTS OF LINEAR REGRESSION MODELS FITTED TO THE WHOLE DEFLECTION BASIN

TABLE A.1 - Summary of linear model curve fittings

Model		Parameters for a typical data set	R-square
1.	$y = A_o \exp (A_1 x^2)$	$A_o = 0{,}2959$	0,99
	or $y = A_o 10^{(A_1 x^2)}$	$A_o = 0{,}991^{-6} \times 10$	
	or $\log y = \log A_o + A_1 x^2$		
2.	$y = A_o x^2$	$A_o = 0{,}4578$	0,99
3.	$y = \exp (a_2 x^2 = a_1 x + a_o)$	$a_o = -0{,}4990$	
	or $y = 10^{(a_2 x^2 + a_1 x + a_o)}$	$a_1 = -0{,}4513 \times 10^{-3}$	0,98
	or $\log y = a_2 x^2 + a_1 x + a_o$	$a_2 = -0{,}1330 \times 10^{-6}$	
4.	$y = \exp^{(a_3 x^3 + a_2 x^2 + a_1 x + a_o)}$	$a_o = 0{,}4467$	
	or $y = 10^{(a_3 x^3 + a_2 x^2 + a_1 x + a_o)}$	$a_1 = 0{,}7818 \times 10^{-3}$	0,98
	or $\log y = a_3 x^3 + a_2 x^2 + a_1 x + a_o$	$a_2 = 0{,}2309 \times 10^{-6}$	
		$a_3 = -0{,}1021 \times 10^{-9}$	

were only applied to the first 200 mm. This is covering an area wider than the normal width of positive curvature (Dehlen 1962) but it does not lead to ill fits of model 1 (see Figure A.2).

A distinctive feature of the deflection basin curves as shown in Figure A.3, is the peakedness of the area of the area of positive curvature. This area of positive curvature is on average less than 10 per cent of the horizontal distance of the whole deflection basin.

The R-square values of model 1 and 2 are an acceptable 0,99. The goodness of fit was also calculated as follows:

$$T = \sqrt{\frac{1 - \sum_{i=1}^{n} (Y_i - Y_i)^2}{1 - \sum_{i=1}^{n} (Y_i - A)^2}} * 100\ \%$$

where A $=$ Average of measured values

Y_i $=$ Measured data values

y_i $=$ Values from model fitted

The goodness of fit for a typical data set for models 1 and 2 is 95 per cent. Both therefore have acceptable values of R-square and goodness of fit for the positive curvature region.

Models 3 and 4 are polynomial functions of the order 2 and 3. There is no real advantage gained in accuracy when the order of the polynomial is increased above the third order. The computation takes longer and becomes costlier too. The goodness of fit of the second order polynomial (model 3) for a typical set of measurements is 99 per cent. It can be seen in Table A.1 that the R-square values for models 3 and 4 are also an acceptably high 0,98. This is very good, but visually it can be seen in Figure A.3 that the deviance from the observed values in the very small area near the origin (positive curvature) does lead to some concern as to the applicability thereof for the whole deflection basin. This tendency to give equal weight to all data points along the linear horizontal distance (x) is typical of this linear regression model used in the SPSS package. For that reason it was decided to investigate the non-linear regression analysis with available models that would tend to give a better description of the whole deflection basin.

Another point of interest in the vicinity of maximum deflection (x=0) is the gradient of the tangent at x=0. In Table A.2 the gradient of the curve described by any of the 4 models is given as first order differentials.

TABLE A.2 - First order differentials of curve fitting models

Models	First order differentials
1. $y = A_o \exp(A_1x^2)$	$\frac{dy}{dx} = A_o (\mathrm{Exp}(A_1x^2))\ 2A_1x$
2. $y = A_o\ x^2$	$\frac{dy}{dx} = 2A\ x_o$
3. $y = \exp(a_2x^2+a_1x + a_o)$	$\frac{dy}{dx} = a_1 + 2a_2\ x)\ \exp(a\ x_1{}^2+a\ x_o + a\ )$
4. $y = \exp(a_3x3+a_2x^2+a_1x + a_o)$	$\frac{dy}{dx} = (3a_3x^2+2a_2x + a_1)\ \exp^* (a_3x^3+a_2x^2+a_1x + a_o)$

At the point of maximum deflection ($x=0$) only models 1 and 2 have a horizontal gradient as the first order differentials are equal to zero. The gradient given by Models 3 and 4 are dependent on the values of the constants when $x=0$. This is another indication of ill fit of models 3 and 4 at the point of maximum deflection.

1.2 Non-linear curve fitting model

1.2.1 Mathematical models

As indicated earlier, the non-linear model of regression analysis used in the SPSS package (Robinson, 1984) tends to give equal weight to each measurement on the deflection basin. In using this non-linear regression analysis package the aim is then to minimize the sum of squares. It is the sum squares of the difference between the fitted model and the observed measured points. For this reason it is therefore important that the unnatural "spikes should be smoothed out before curve fitting is done.

There are two options in the non-linear regression analysis package of SPSS (Robinson, 1984) namely using the Gauss method or the Marquardt method. The latter was selected as superior due to its shorter computing time required. In Table A.3 the two models tested by the two options are shown with the calculated constants and sum of squares values.

TABLE A.3 - Results of non-linear curve fitting procedures

Model	Method	Sum of squares	Parameters for a typical data set
1. $y = A_o \exp(A_1 x^2)$	Gauss	$8{,}914 \times 10^{-1}$	$A_o = 3{,}321*10^{-1}$ $A_1 = -9{,}991*10^{-1}$
2. $y = \exp(a_2 x^2 + a_1 x + a_o)$	Marquardt	$3{,}768*10^{-3}$	$a_o = -1{,}066$ $a_1 = -1{,}134*10^{-3}$ $a_2 = -2{,}796*10^{-7}$

As can be seen both models had been tested for curve fitting by the linear regression analysis facility of SPSS before. Model 1 was again tested here on an area wider than the positive curvature by selecting the first 350 mm for curve fitting. As could be expected it did lead to a poor fit as reflected in the rather high value of sum of squares.

The residuals (difference between prediction and observation) are also plotted. Apart from giving a visual impression of the goodness of fit, it also serves as a monitor for specific patterns which indicate poor fitting models. In Figure A.4 the residual plot of the model 1 fitting is shown. The definite pattern confirms the ill fit. It re-emphasises the fact that model 1 can only be applied to the area of positive curvature (< 150 mm).

Model 2 (Table A.3) proved to be better suited for the fitting of the whole deflection basin and particularly the large area of

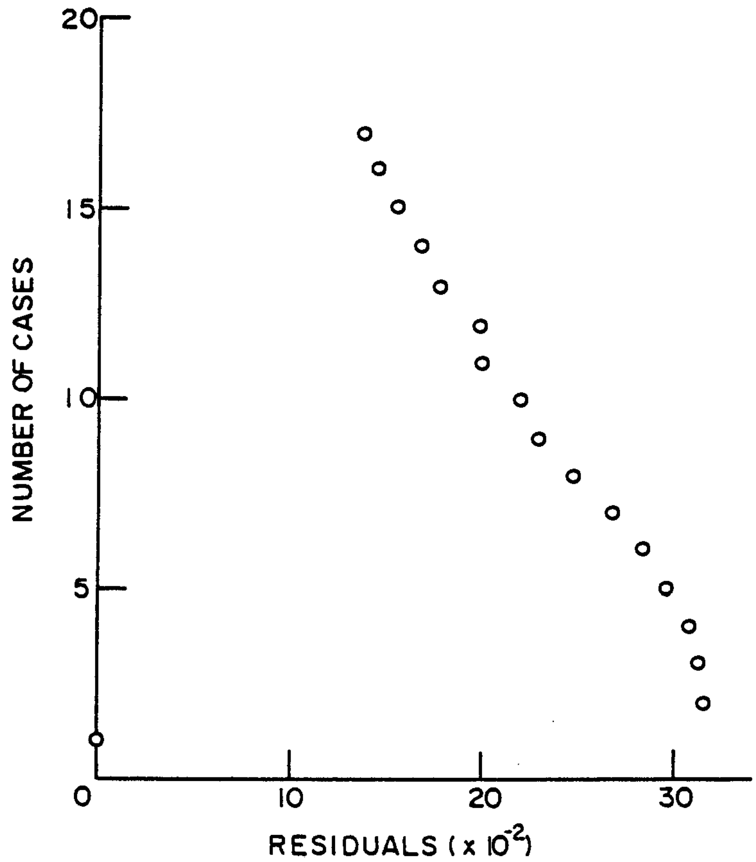

FIGURE A.4

RESIDUAL PLOT OF AN ILL FITTING MODEL

the reverse curvature of the deflection basin. The good fit thereof is reflected by the low value of the sum of squares in table A.3. The plot of residuals in Figure A.5 also reflects no specific patterns indicating a good fit. The lack in fitting the area of positive curvature in the vicinity of maximum deflection still persists as was indicated in the linear regression analysis too. It would be possible though to use these two models of table A.3 and limit the curve fitting of the positive curvature to that by model 1 and the curve fitting of the larger reverse curvature to that of model 2 and achieve satisfactory results.

1.2.2 Physical model

The non-linear package of SPSS makes it possible to use relatively complex models in the regression analysis. This gives the opportunity to look at physical models which can be adapted to the observed deflection basin. A promising model is that of beams of unlimited length on elastic foundations with concentrated loading. This model is described in great detail by Hetényi (1971) and Frýba (1967). The intention is not to give a detailed description of this theory here, but rather concentrate on the use and manipulation of the derived solutions in the curve fitting exercise. In the analysis of bending of beams on an elastic foundation Hètenyi (1971) states that the assumption is that reaction forces of the foundation are proportional at every point to the deflection of the beam at that point based on the Winkler theory. This theory also states that deformation exists only along the portion ~~directly~~ under loading and was verified in experiments for a variety of soils. Hetényi (1971) is quoted as follows; ".. that the Winkler theory, in spite of its simplicity may often more accurately represent the actual conditions existing in soil foundations than do some of the more complicated analysis..."

A short description of the model is as follows; Consider an infinite beam subjected to a single concentrated force P at the a point O, which is the origin of the axis system as shown in figure A.6. The general solution for the deflection curve of a

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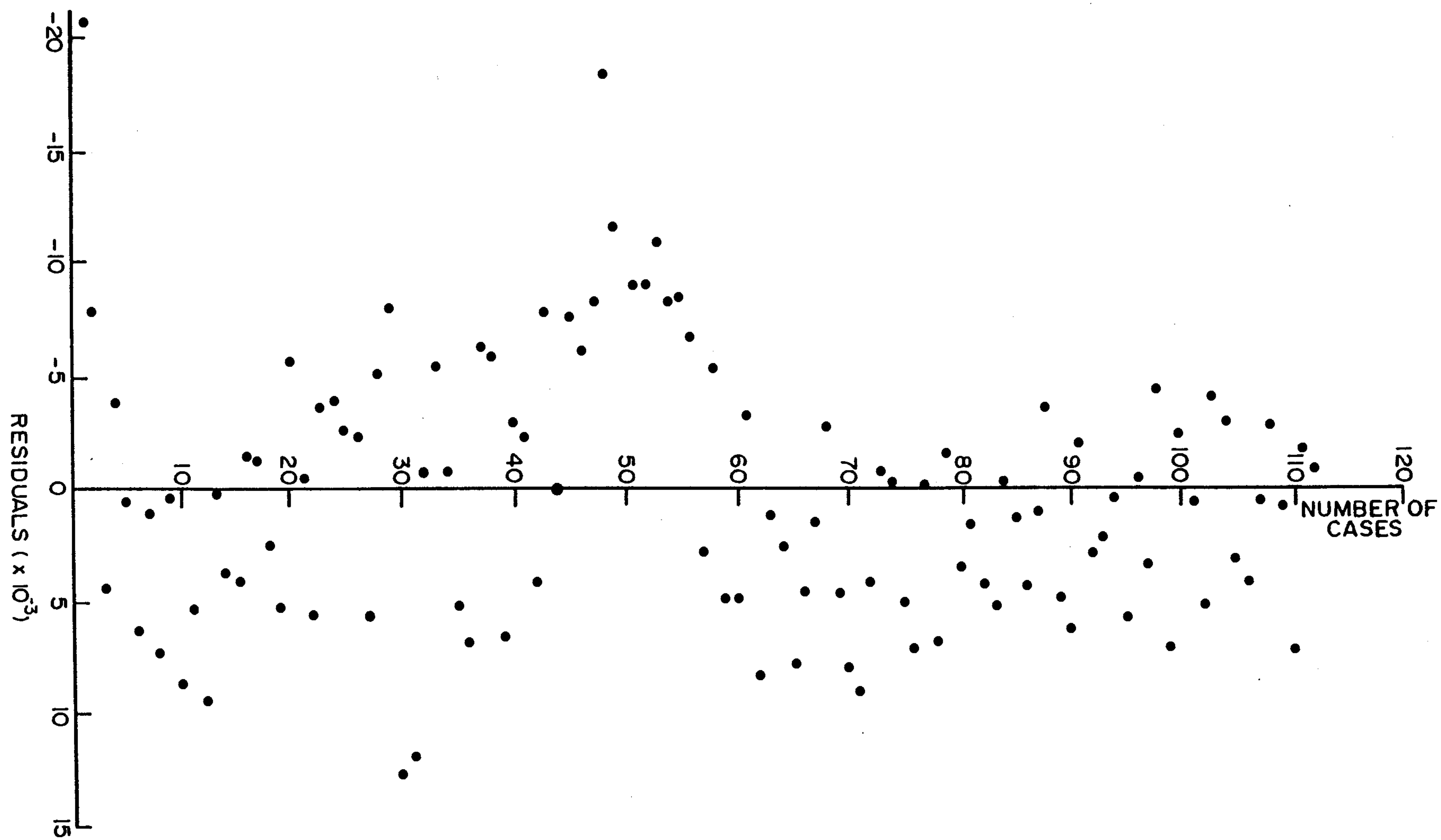

FIGURE A.5

RESIDUAL PLOT OF GOOD FITTING MODEL

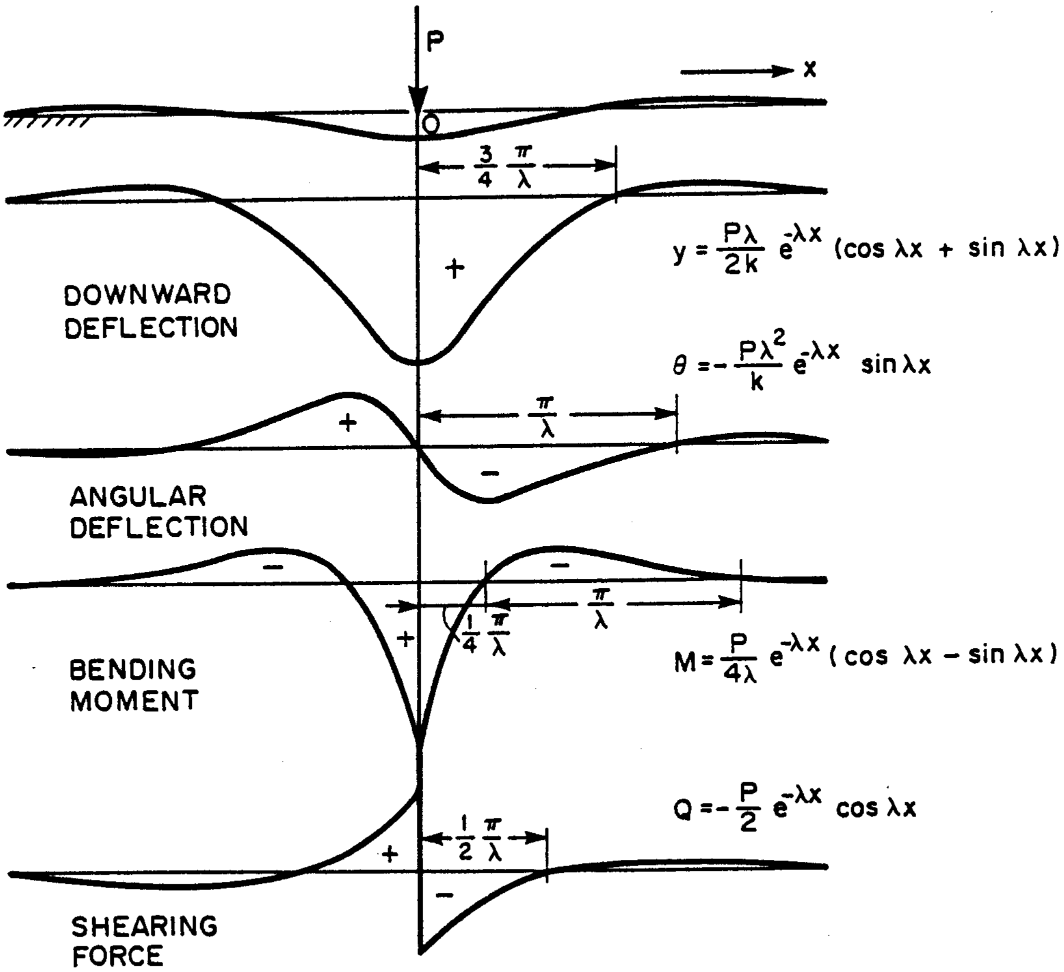

FIGURE A.6

POINT LOAD ON AN INFINITE BEAM ON AN ELASTIC FOUNDATION beam subjected to transverse loading is derived by Hetényi (1971) as;

$$y = e^{\lambda x} (C_1 \cos\lambda x + C_2 \sin\lambda x) + e^{-\lambda x} (C_3 \sin\lambda x + C_4 \sin\lambda x)$$

where y is the deflection taken as positive downwards
x is the horizontal distance from the origin
C_1, C_2, C_3, C_4 are constants
λ is the damping factor, and

$$\lambda = \sqrt[4]{\frac{k}{4EI}}$$

k is the modulus of the foundation expressed in $kg/m^2/m$
E is the modulus of elasticity in kg/mm
I is the second moment of area of the beam

Without even going into further detail of the simplification of this model it can be seen in Figure A.6 what the similarities between the modelled deflection curve and the measured deflection basin curves are. When the symmetry of the deflection curve and the equilibrium of the reaction forces are considered this general equation reduces to;
$y = \frac{P}{2k} e^{-\lambda x} (\cos\lambda x + \sin\lambda x)$, which gives the deflection curve for the right side ($x \geq o$) of the beam. This correlates well with the data preparations of the deflection basin described in the preceding sections. The form of the curve suggests that the case of no dampening can be considered for this part of the curve analyzed. Frýba (1967) gives the solution as follows for such a situation;

$$y = \frac{1}{ab} e^{-b|x|}(a \cos ax + b \sin a |x|)$$

This form is obviously similar to that derived by Hetényi (1971. In order to simplify the determination process of the constants in these equations, it was decided to use the following curve fitting model;

$$y = B_1 e^{B2|x|}(\cos B_3 x + \sin |B_3 x|)$$

In figure A.7 a plot of a typical data set and the curve fitting is shown. Visually it can be seen that this model succeeds in describing the observed deflection basin accurately. The sum of squares value is low ($4{,}15 \times 10^{-3}$ for this typical data set) and the plot of the residuals does not indicate any ill fit. The goodness of fit for such a typical data set is above 98 per cent. It was found that this mode is applicable over a wide range of variances in load, load repetitions and structural condition of pavements. It is obvious though that although this model gives the best fit of all models tested it still tends to give an ill fit in the positive curvature area. Although this area is very small and very peaked in relation to the rest of the deflection basin, it was decided to use the parabola of the linear models in this area of possible curvature in order to arrive at a true representation of the whole deflection basin.

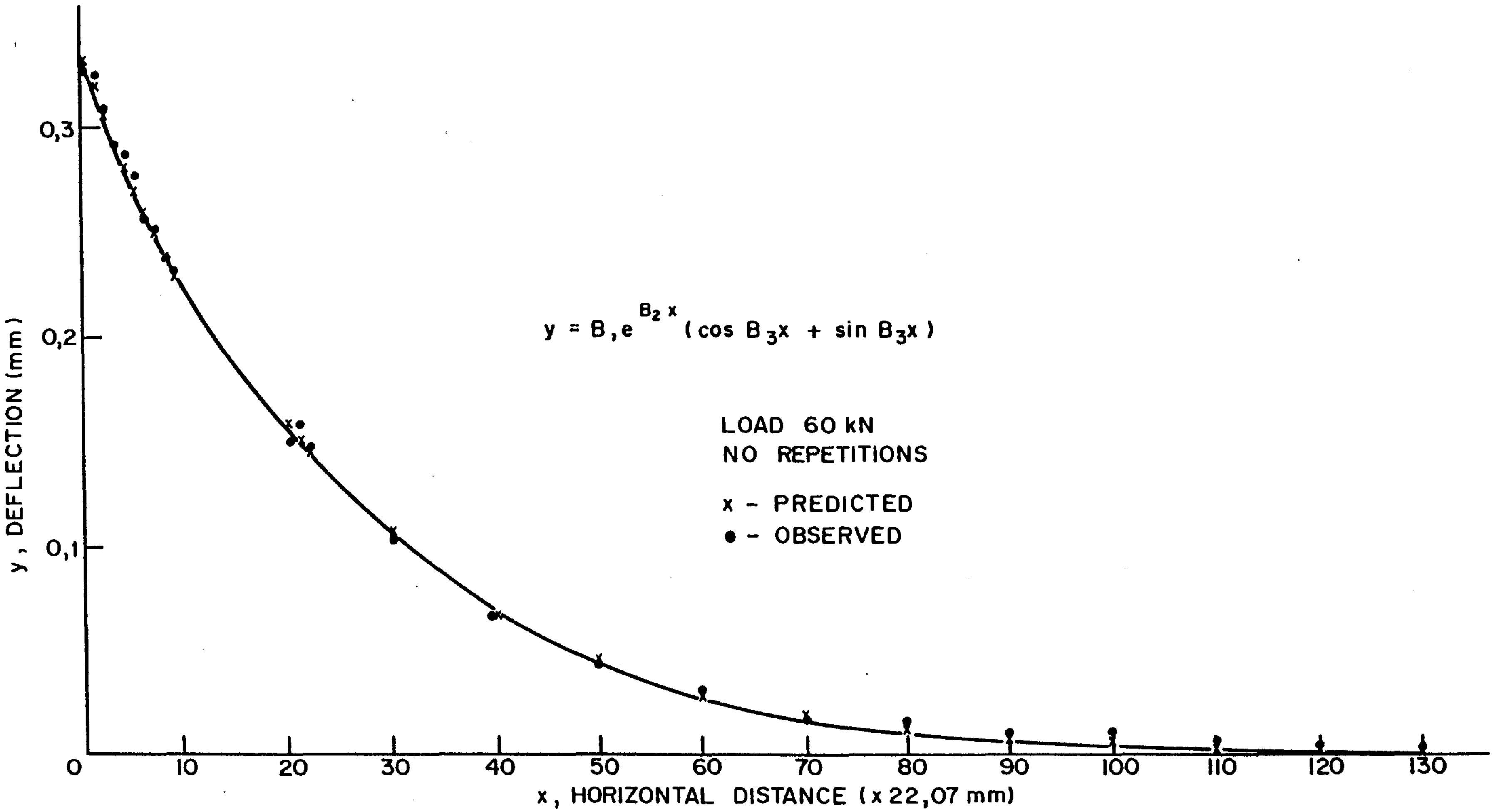

FIGURE A.7
TYPICAL CURVE FITTING OF PHYSICAL MODEL

APPENDIX B

SUMMARY ON CONDITION SURVEYS ENHANCING DEFLECTION BASIN ANALYSIS

1 INTRODUCTION

A condition survey is an important input in the non-destructive testing of an analysis procedure. In Figure B.1 it is shown how this kind of visual survey greatly enhance the understanding of the material characterization in a typical analysis procedure for overlay design. Condition surveys are non-destuctive testing procedures enhancing the other non-destuctive testing procedures such as deflection basin analysis. In Figure 4.1 it was explained how in the South African mechanistic rehabilitation design procedure (Freeme, 1983) it is important to identify the pavement layer state. The discussion on condition survey will therefore focus on crack and rut classification related to the deflection basin survey (see Figure B.1).

Ullidtz (1982) defines a functional and structural condition in his model on pavement rehabilitation. He states that; "The structural condition is of no immediate interest to the road user but is is extremely important to the highway agency because the future functional condition depends on the present structural condition." A visual condition survey can be seen as an aid to a proper structural evaluation.

The standard procedure for conducting a condition survey, as outlined in Draft TRH12 (NITRR, 1983), should be followed. Normally the visual assessment precede the deflection basin survey but it is suggested that the deflection basin survey and visual assessment may be done simultaneously on smaller scale projects. The results of both surveys should be plotted on the same scale. By this means the obvious weak spots can be identified when other relevant information such as drainage, cut or fill transition and soil changes is taken into consideration.

2 CRACK CLASSIFICATION

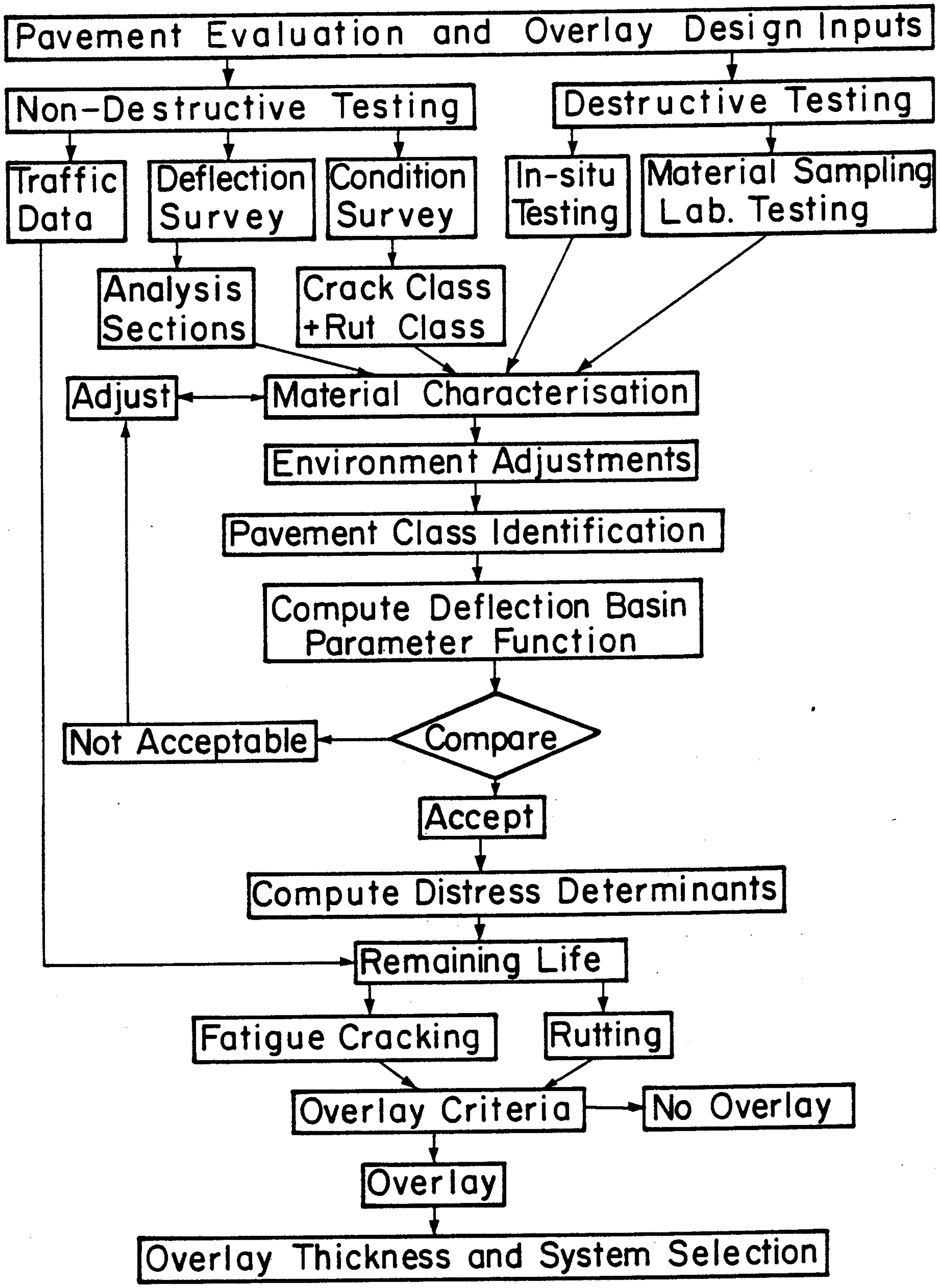

FIGURE B.1
Mechanistic overlay design flow diagram

Cracks in the existing asphalt concrete layer have a major influence on the overlay design procedure. Normally pavements are classified as either cracked or not cracked. However, the majority of pavements fall somewhere between these extremes. The aim of this section is therefore to establish quantitive procedures to classify cracked pavements in order to improve the rehabilitation design procedure.

Grant and Curtayne (1982) point out that fatigue is not necessarily the cause of cracking in asphaltic concrete layers in South Africa. Other factors, not necessarily traffic-related, should also be considered as being possible causes of premature cracking (Grant, et al (1979). Pronk and Buiter (1982) indicate that with full depth asphalt pavements, even in the Netherlands, cracking does not necessarily begin at the bottom of the asphalt layer. Grant and Curtayne (1982) therefore stress that a study of the past behaviour of the pavements can provide good clues in this respect. The preceding statements are to indicate the complexity of crack mechanisms of asphalt concrete layers. Different reasons for cracking are therefore discussed in order to arrive at a classification for cracked pavements.

The aim of this is purely to simplify the analysis of such a pavement by using the deflection basin parameters in the mechanistic approach.

First it is suggested that the difference in basic crack mechanism, due to the difference in pavement structure be considered. For the South African condition it is suggested that on the grounds of as-built plans or material sampling procedure (Figure B.1), a flexible pavement under survey be classified according to the basic TRH4 (NITRR, 1985a) catalogue, i.e.

(a) granular base pavements
(b) bitumen base pavements (tar as an alternative)
(c) cemented base pavements.

This basic classification is taken a step further by Freeme (1983) by relating the time-dependent behaviour of different pavement types to the concept of equivalent material state illustrated in Figure B.2. It is explained in Chapter 3 how the measured deflection basin parameters can be used to accurately identify any such pavement behaviour state.

Next, the degree of cracking should be defined. In the light of work done by Kilareski et al. (1982) and Treybig et al. (1978), it is suggested that the AASHO definition of cracking could be applied. Jordaan and Servas (1983) give a very clear description of three types of cracking and how it should be calculated. These types are crocodile or map or block cracking, longitudinal cracking and other crack patterns or combinations of the preceding types. For these types each 100 m of road length is classified as being in a sound, warning or severe condition. This depends on the percentage of 100 m being cracked and the road category.

Crack conditions can be improved by crack filling and repair of the low percentage of severe cracking of the defined categories or crack types. The decision to repair cracks should be based on economic comparisons, but it is obvious that crack repair will improve possible crack attenuation behaviour in general. Koole (1979) suggests though that an overlay design based on the severe condition of cracking might in some cases be the most economical in the final analysis owing to the expensive nature of procedures to upgrade the pavement in regard to the crack condition outlined above.

In the final level of crack classification more sophisticated methods should be considered. This normally consists of analysis procedures associated with the overlay design analyses. Various researchers, such as Molenaar (1983) and Coetzee and Monismith (1979) suggest the use of fracture mechanics principles and finite element computer programs in the analysis stage. In order to use these procedures though, the previous crack classification would have to be elaborated in order to establish average values of crack width too. Molenaar (1983) states the following:

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STATE →	VERY STIFF	STIFF	FLEXIBLE	VERY FLEXIBLE
APPROXIMATE DEFLECTION LEVEL (mm)	0,2	0,4	0,6	
EXAMPLES OF CHANGE OF STATE OF DIFFERENT PAVEMENT TYPES	CONCRETE →			
	CEMENT TREATED BASE PAVEMENT →			
		CRUSHED-STONE OR BITUMEN TREATED BASE OVER CEMENT TREATED SUBBASE		
			BITUMEN TREATED BASE OVER GRANULAR SUBBASE	
			GRANULAR BASE/GRANULAR SUBBASE. LOW SUBGRADE CBR VALUE	
TYPICAL EXAMPLES IN PRACTICE	CONCRETE ROAD N2			
	HOORNSNEK (TVL); BRONKHORSTSPRUIT P67/1 (TVL); N3 SWINBURNE (OFS)			
		P157/1 & P157/2 JAN SMUTS AIRPORT (TVL); PARADISE VALLEY (NTL) N3 MARIANNHILL (NTL)		
			P205/2 GILLOOLY (TVL)	
			KOEBERG (CAPE) P123 MAGALIESBURG (TVL) P6/1 BABSFONTEIN (TVL)	

FIGURE B.2

DIAGRAMMATIC REPRESENTATION OF THE TIME DEPENDENT BEHAVIOUR OF DIFFERENT PAVEMENT TYPES (Freeme, 1983)

"Although the fracture mechanic's approach has the potential to be an excellent tool in solving the reflection crack problem, it has not gained very much popularity. In fact it can be stated that it is still a research tool and that its practical application is limited to only a few cases."

Recently the monitoring of crack movements, as described by Rust (1984), has become another viable method that may be associated with the classifications outlined above. The Crack Activity Meter (CAM) that was developed can measure amongst others the defined total crack movement. Crack activity or total crack movement normally has the typical peaking behaviour with axle repetitions as shown in Figure B.3. Rust (1985) was able to determine that for a flexible pavement with a cemented base and under specific conditions, there is a good correlation between block size and crack movement. The data indicated that there is a critical block size below which the crack movement increases markedly with further decrease in block size. Typical results are shown in Figure B.4. These concepts can be used effectively to enhance the crack classification as given above.

In Appendix E the good correlation between the measured crack-activity and measured deflection basin parameters is illustrated by means of an example. As will be shown there this greatly enhances the rehabilitation analysis procedure.

3 RUT CLASSIFICATION

One of the major aims of a rut survey is to determine the amount of material needed for the levelling of the existing rut before an overlay is applied. This is all related to ride comfort (PSI values) and, in wet conditions in particular, to rider safety. The extent of rutting is generally used in overlay design as a major criterion of permanent deformation and the structural state of the pavement. The general procedure is to limit rutting in overlay designs by limiting the vertical subgrade strain (ε_{vs}). This approach was originally developed by Dorman and Metcalf

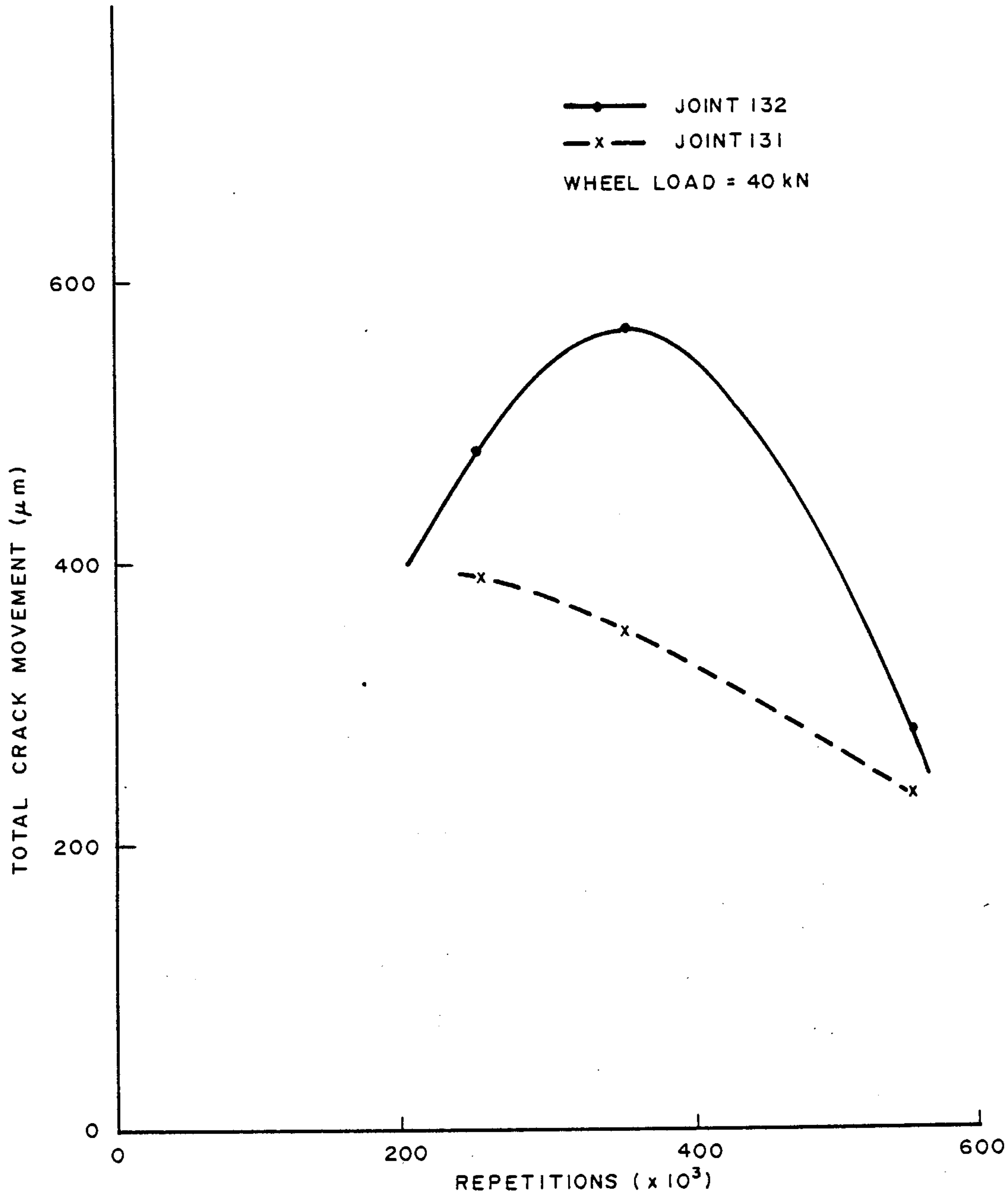


FIGURE B.3

CHANGE IN TOTAL CRACK MOVEMENT DURING HVS TESTING ON THE N2 CONCRETE ROAD–SECTION 258A2
(Rust, 1984)

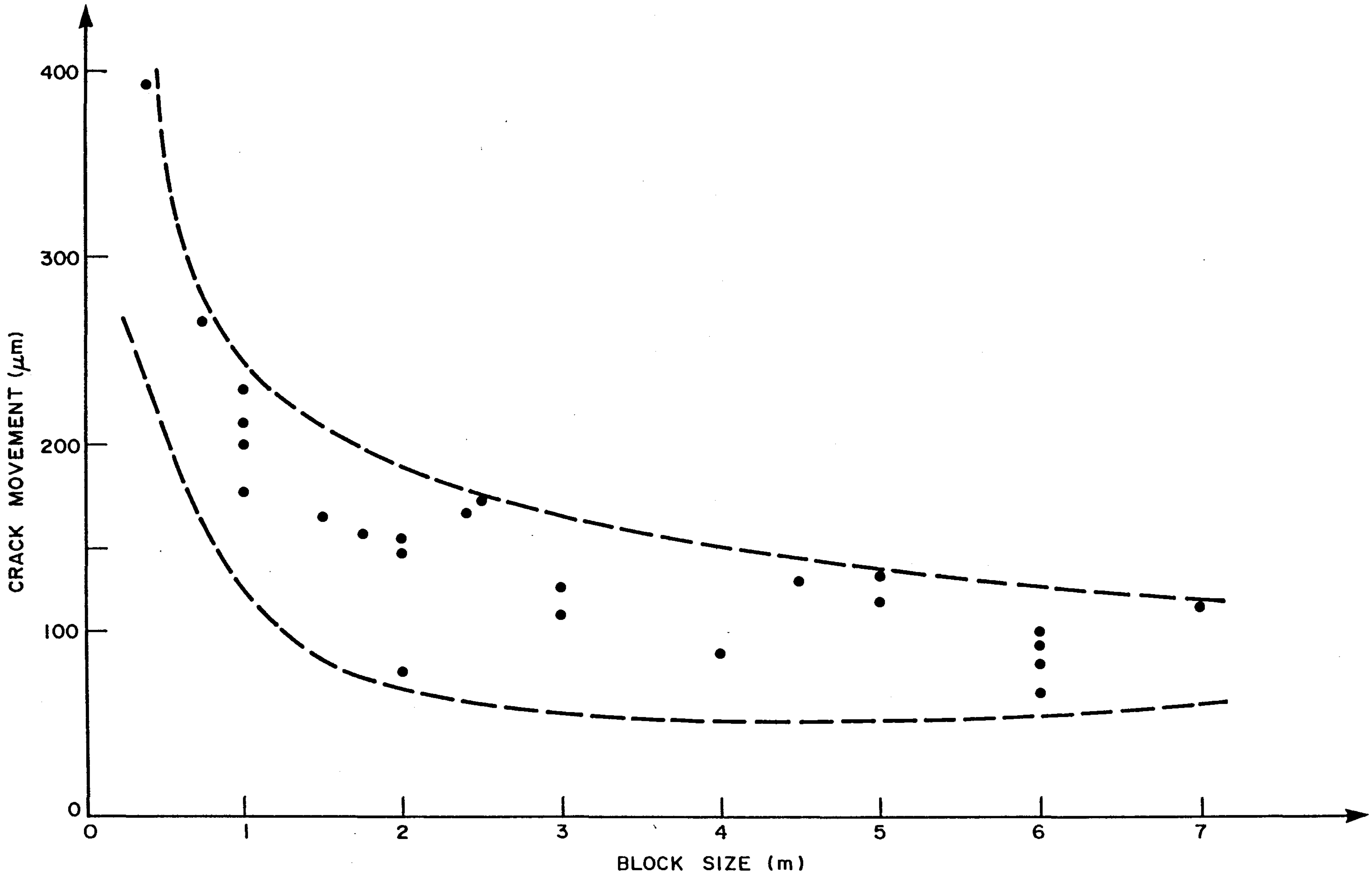


FIGURE B.4

CORRELATION BETWEEN CRACK MOVEMENT AND BLOCK SIZE ON THE MR 27 (Rust, 1985)

(1963) in their analysis of the behaviour of the test sections of the AASHO road test.

It is should be noted that rutting is not only related to the subgrade but that contributions also come from the various layers in the pavement. For this reason Koole (1979) mentions that the rut in the asphalt concrete layer should be treated separately. The South African experience also indicates that such a direction should be followed. Freeme et al. (1982a) indicate that better characterization of the bitumen layer in terms of volumetric and shear properties is necessary to accommodate this deformation phenomenon in the bitumen layer. Maree et al. (1982) show that for granular base pavements tested with the Heavy Vehicle Simulator (HVS), most of the permanent deformation took place within the granular base and subbase. The subgrades never meaningfully contributed towards the total deformation and were always well protected. In the same report Maree et al. (1982) illustrate the strong correlation between cracking, excessive rain, moisture intrusion and deformation for typical granular base pavements; this is shown Figure B.5.

The preceding statements make it obvious that a more qualitative classification of rutting is needed than just a report of the average rut. In line with the classification outlined in the previous section on cracking, it is suggested that the pavement structure classification as defined be used also. In fact it is re-emphasized that no indicator like rutting or deflection should be used in isolation. The concept is clearly illustrated by Freeme (1983) in Figure B.6 where various indicators of the behaviour of typical granular layers are shown.

A further practical classification is needed to discriminate between various mechanisms of permanent deformation. Molenaar (1983) classifies two types of rutting (see Figure B.7). The first type is that without lateral displacement due to densification. The second type is that with lateral displacement due to Prandtl type of shear deformation. This ties in with the previous discussion on the South African experience. Grant and

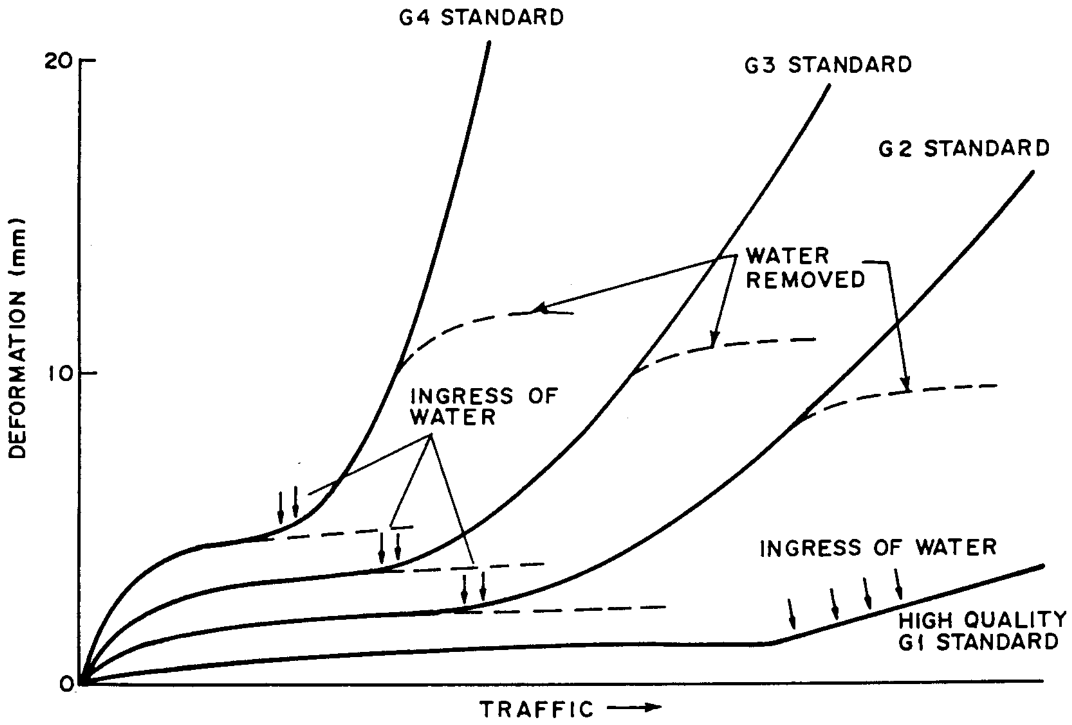


FIGURE B.5

SCHEMATIC DIAGRAM OF THE RELATIVE BEHAVIOUR OF GRANULAR MATERIAL OF DIFFERENT QUALITIES

(Maree, at al., 1982)

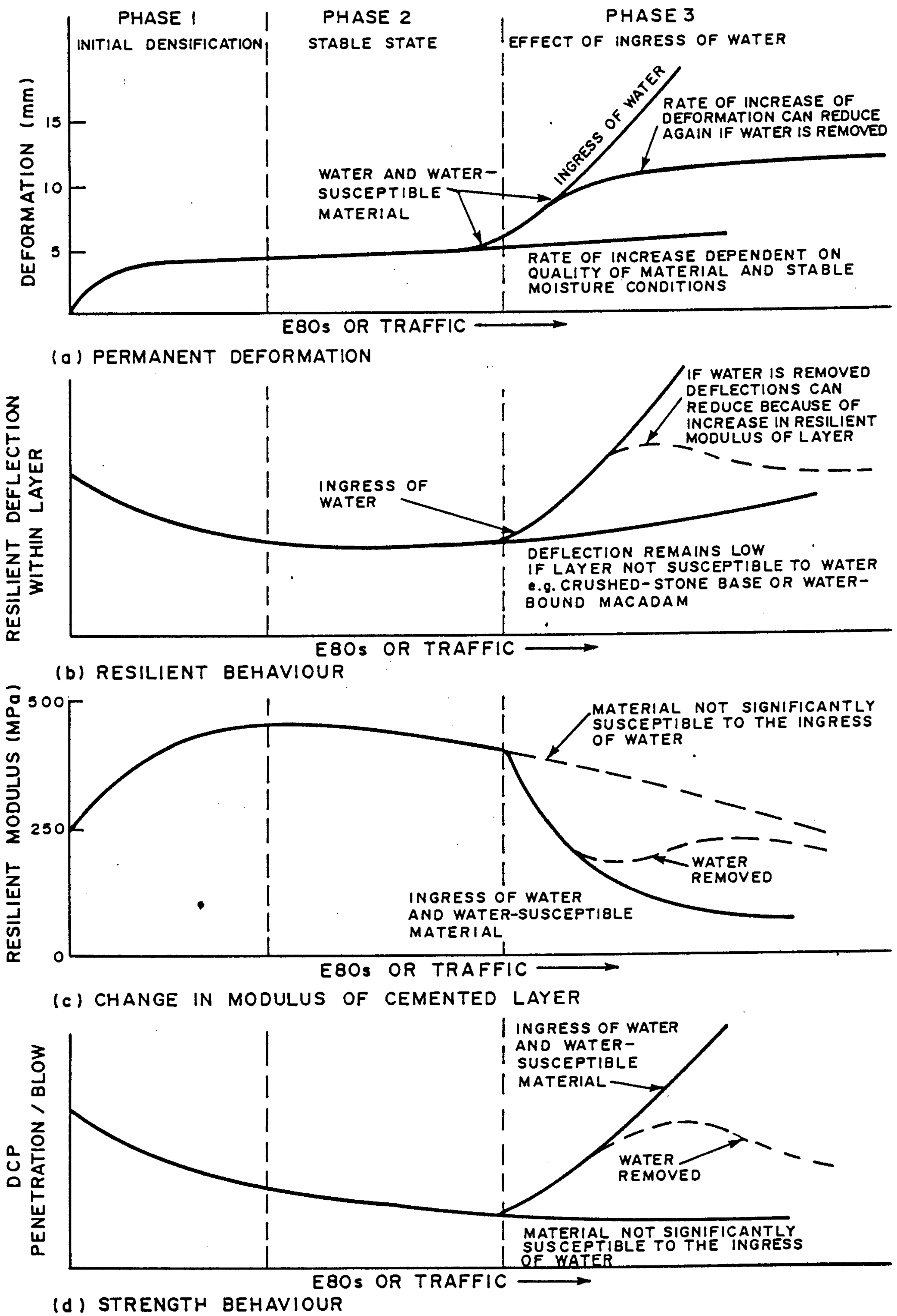
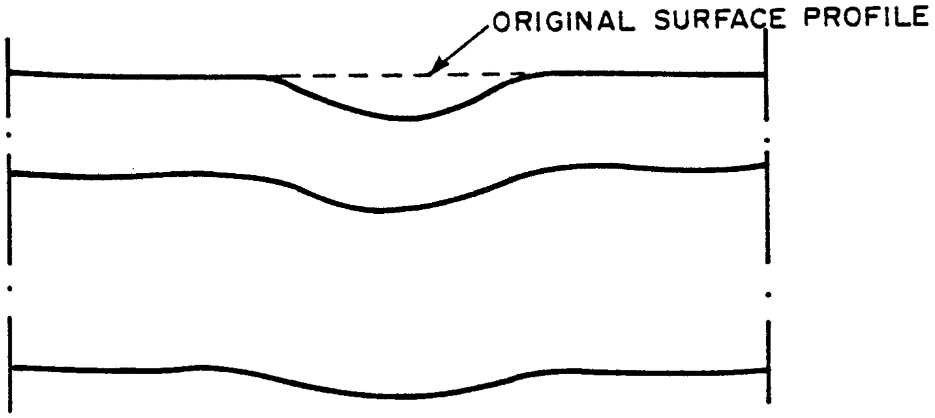


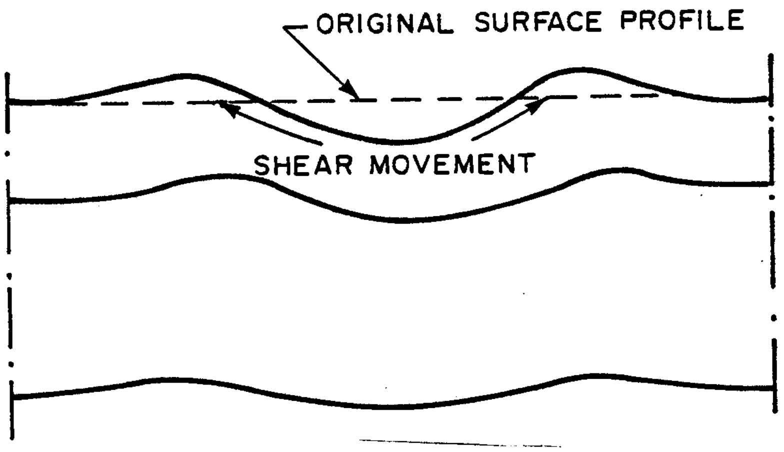
FIGURE B.6

INDICATORS OF THE BEHAVIOUR OF GRANULAR LAYERS
(Freeme, 1983)



Type A

Rutting without lateral displacement of the material. This type of rutting is due to densification of the material.



Type B

Rutting with lateral displacement of the material. Rutting can be judged to be a Prandtl type of shear deformation.

FIGURE B.7
TYPES OF RUTTING WHICH CAN BE DISCERNED
(Molenaar, 1983)

Curtayne (1982) note that shear in the subgrade is characterized by wide rutting. Shear in the base layer is characterized by narrow ruts with displaced material appearing like a mound adjacent to the rut.

From the visual survey therefore a classification of the type of rutting that exists may be made, which will strongly influence the overlay design analysis. The Draft TRH12 (NITRR, 1983) give clear indications for rut criteria related to pavement class and pavement type. It is suggested that those criteria and those suggested by Jordaan and Servas(1983) be followed.

APPENDIX C

REMAINING LIFE DETERMINATION

1 INTRODUCTION

At any specific moment, an existing pavement has a certain amount of accumulated damage done to it by repeated traffic loading. There is normally also a certain amount of remaining damage which the existing pavement can undergo before failure. The severity of the damage caused by each repeated traffic loading depends on the structural strength of the existing pavement. This is usually expressed in terms of the equivalent number of standard axles (E80s). If the magnitude of the critical strains is reduced, then the existing pavement can carry a larger number of standard axle loads (E80s). The function of an overlay is therefore to reduce the magnitude of these critical load-induced strains or stresses, depending on the distress determinants being used.

Remaining life has two meanings. Without an overlay a pavement normally has remaining life and with an overlay a pavement has a remaining life, that is usually lengthened or prolonged. For this reason "remaining life" will refer to the remaining life of a pavement without an overlay.

In the literature remaining life is usually analysed on the basis of the phenomenological theory of cumulative damage. Attempts to relate the structural condition, based on deflection basin measurements, in a different way to the life of the pavement, will also be discussed.

2 THEORY OF CUMULATIVE DAMAGE

The phenomenological theory of cumulative damage is also referred to as the linear summation of cycle ratios. This was advanced by Miner (1945) to predict the fatigue life of metals subjected to fluctuating stress amplitudes. Monismith et al. (1966, 1969) used it to estimate fatigue life of bituminous layers in pavement structures and established it as an acceptable and useful relationship.

This theory is described by Snaith et al. (1980) as follows:

Let n_i = number of applications at stress or strain level

N_i = number of applications to failure at stress or strain level

D_i = damage due to N_i number of applications at stress or strain level

Then the damage, D_i, is defined as the stress or strain cycle ratio, i.e.

$$D_i = \frac{n_i}{N_i}$$

Failure will occur when $D_i = 1$.

Let r = number of different stress or strain levels involved

D = cumulative damage due to number of applications at different stress or strain levels

Then the cumulative damage, D, is stated as the linear summation of cycle ratios, i.e.

$$\sum_{i=1}^{r} D_i = \sum_{i=1}^{r} \frac{n_i}{N_i}$$

Failure occurs when

$$D = 1 \quad \text{or} \quad \sum_{i=1}^{r} \frac{n_i}{N_i} = 1$$

Snaith et al. (1980) use the distress determinants vertical subgrade strain (ε_{vs}), and maximum horizontal aspahlt strain (ε_{HA}), as discussed in chapter 4 and 6 , to determine damage due to rutting deformation and fatigue cracking respectively. For both forms of damage the strain-life relationship is given by the general equation:

$$N = A\left(\frac{1}{\varepsilon}\right)^b$$

where A and b are material constants.

It is therefore possible to apply the cumulative damage theory to both forms of damage. The accumulation of damage from repeated applications at various strain levels is illustrated diagrammatically in Figure C.1. In Figure C.1(a) the strain-life diagram is shown with a typical strain-life curve, l-k. On this curve a strain level ε_1, for example, corresponds to a life N_1. Lines a-b, c-d, etc., represent n_1 applications at strain level ε_1, and n_2 applications at strain level ε_2, etc. These lines, represented by arrows, are called damage paths. The dashed lines, b-c, d-e etc. are called iso-damage lines. If the amounts of damage at b and c are the same, then

$$\frac{n'}{N_2} = \frac{n_1}{N_1} \text{ and thus } n' = N_2 \frac{n_1}{N_1}, \text{ and}$$

similarly n'' and n''' can be found.

This is represented more simply in a damage-life diagram (see Figure C.1(b)). The damage scale ranges from 0 to 1. The damage paths can be plotted continuously as shown in Figure C.1(c). In this way, the cumulative damage arising from repeated applications is determined in diagrammatic form.

In practice the number of repeated applications (n_i) is expressed in terms of the equivalent number of standard axles (E80s). This reduces the analysis to only one strain level to determine remaining life. In Figure C.1(a), therefore, at strain level ε_1 the damaged or consumed life is n_1 and total life is N_1. Remaining life at this strain level is equal to:

$$N_1 - n_1$$

Alternatively, damage (D_1) is often expressed as previously defined and remaining life (R_1) is then:

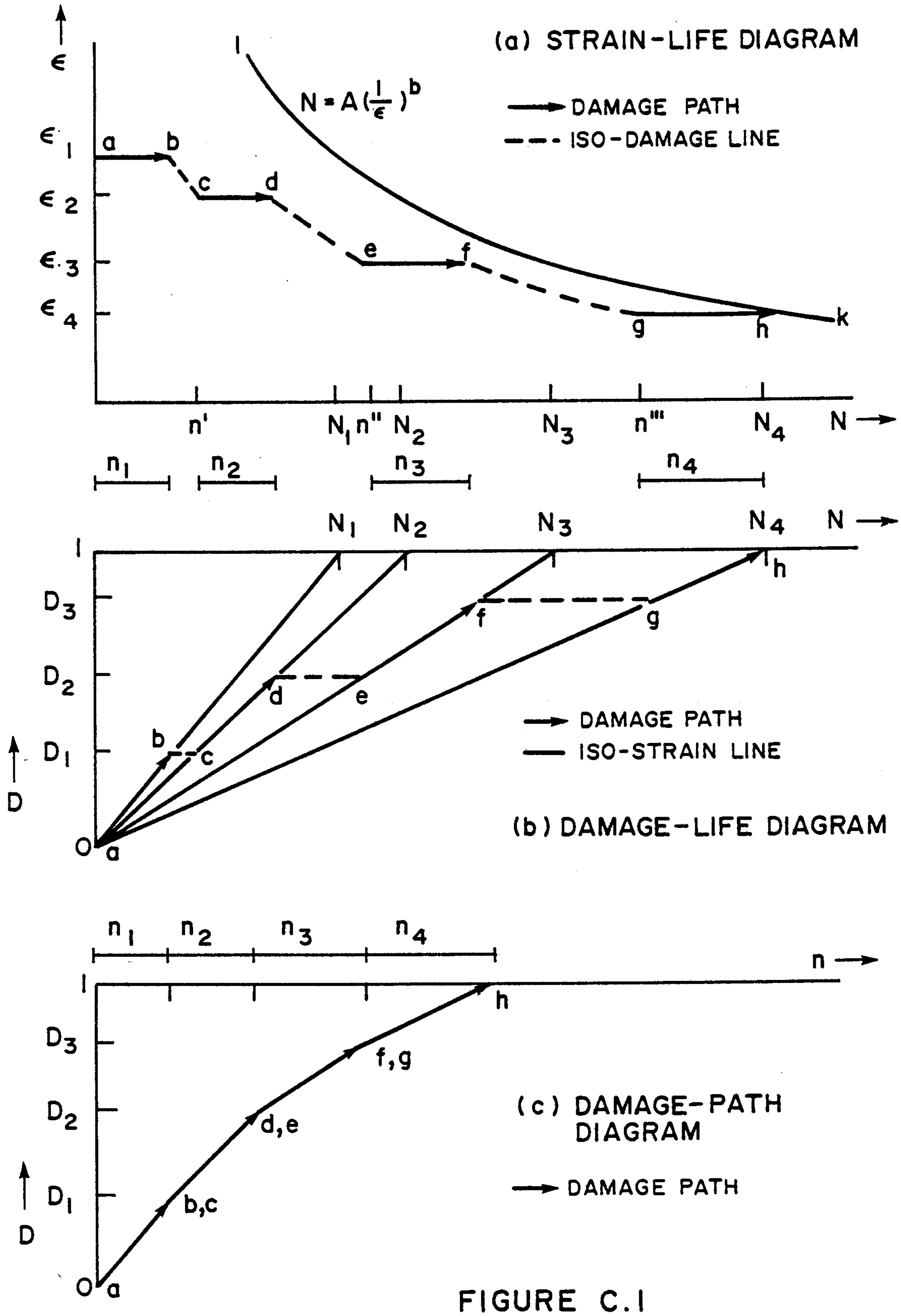


FIGURE C.1

GRAPHICAL PRESENTATION OF CUMULATIVE DAMAGE THEORY (Snaith, et al., 1980)

$$R_1 = 1-D_1$$

$$= 1 - \frac{n_1}{N_1}$$

Using the same distress criteria, vertical subgrade strain (ε_{vs}) and maximum horizontal asphalt strain and (ε_{HA}), Anderson (1977) also used this theory to determine remaining life. It is pointed out, though, that future environmental or traffic changes cannot usually be foreseen and therefore such a procedure should be seen as a guide only. In considering the remaining life of a pavement with rutting due to permanent deformation, Anderson (1977) reasons that the damaged life or consumed life will be nullified when the surface deformation is removed by an overlay. Koole (1979) supports this view by Anderson (1977) in his description of the Shell overlay design method. The remaining life of a pavement with fatigue cracking is determined in accordance with the description by Snaith et al. (1980).

Treybig et al. (1978) also use the theory of cumulative damage in order to determine remaining life for a pavement with fatigue cracking and rutting due to permanent deformation. As mentioned in Appendix B, however, the cracked state of the existing pavement is taken into consideration in determining the material parameters (E asphalt). Chapter 4 described how these material parameters are used to determine the distress determinants. In a pavement with fatigue cracking the maximum horizontal asphalt strain (ε_{HA}) is calculated and used to determine the remaining life in terms of standard axle (E80) repetitions, as described by Snaith et al. (1980). It was shown in Chapter 6, Treybig et al. (1978) consider the contribution of all the structural layers to rutting due to permanent deformation by determining the various stresses and strains of each layer.

It is obvious that Kilareski et al. (1982) only considered fatigue cracking when determining remaining life. The strain (ε_{HA}) need not necessarily be determined, but as shown in Figure C.2 the deflection basin parameter, surface curvature index (SCI), is related to the number of equivalent single-axle loads (EAL). In this case the structural number has also been determined, based on the AASHO Design Procedure (for the various test sections). The 10 per cent fatigue cracking line is the same form as described above for the general relationship, $N = A(\frac{1}{\varepsilon})^b$. The equation for remaining life is as described above, namely $(N_1 - n_1)$. Kilareski et al. (1982) advance this one step further by relating remaining life (in terms of equivalent axle loads) to the SCI for various structural numbers (pavement strengths), as shown in Figure C.3.

3 STRUCTURAL PERFORMANCE MODEL

Residual life determined from deflection measurements alone does not lead to satisfactory results. Koole (1979) states: "It is not possible to determine the residual life of a pavement solely from deflection measurements". The reason lies in the fact that the change in a structural parameter, for example elastic modulus (E), with an increase in load repetitions shows a sharp decrease in value initially but thereafter there is a long period during which virtually no change occurs and only at the end of the structural life is there a definite sharp decrease to distress. Deflection measurements also reflect this typical behaviour. However, it is possible to relate early life deflections empirically to the critical life of particular types of pavement structures, as shown in Figure C.4 using work done by Lister and Kennedy (1977). Koole (1979) also mentions that original design life can be determined from FWD deflections. A "crude" test on consumed life is to take FWD deflections between the wheel tracks. If the deflections measured in the wheel tracks, are significantly greater than those measured between the wheel tracks the pavement is approaching the end of its service life.

Pronk and Buiter (1982) mention the procedure in which the decline in effective layer thickness is related to the structural strength. This forms the basis of the structural performance model developed by Molenaar (1983). This principle is shown schematically in Figure C.5 where equivalent layer thickness (H_e) decreases in

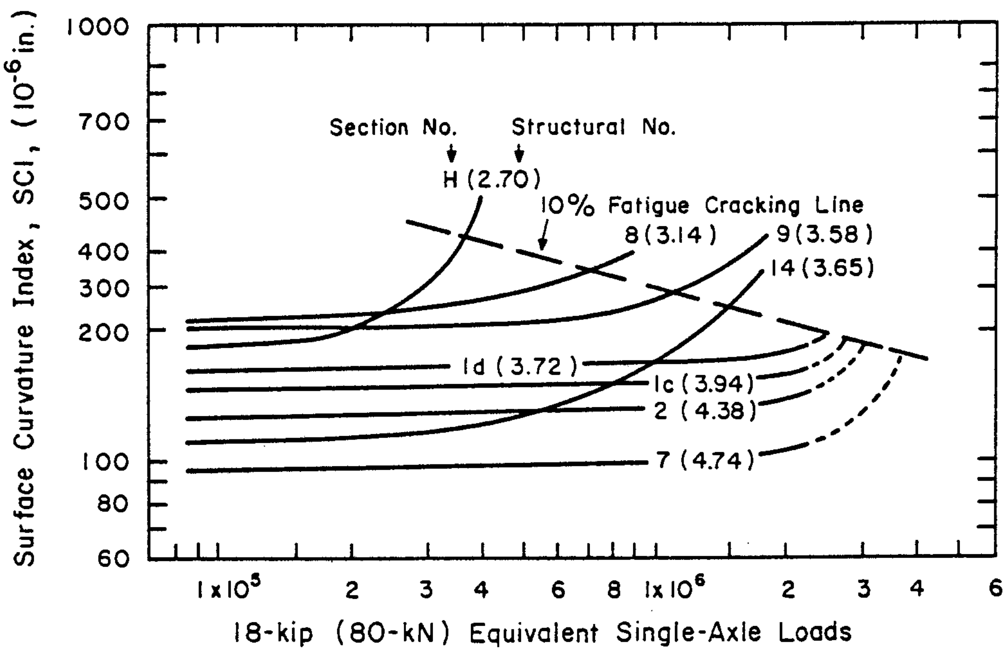


FIGURE C.2
VARIATION OF SURFACE CURVATURE INDEX WITH EAL. (Kilareski, et al., 1982)

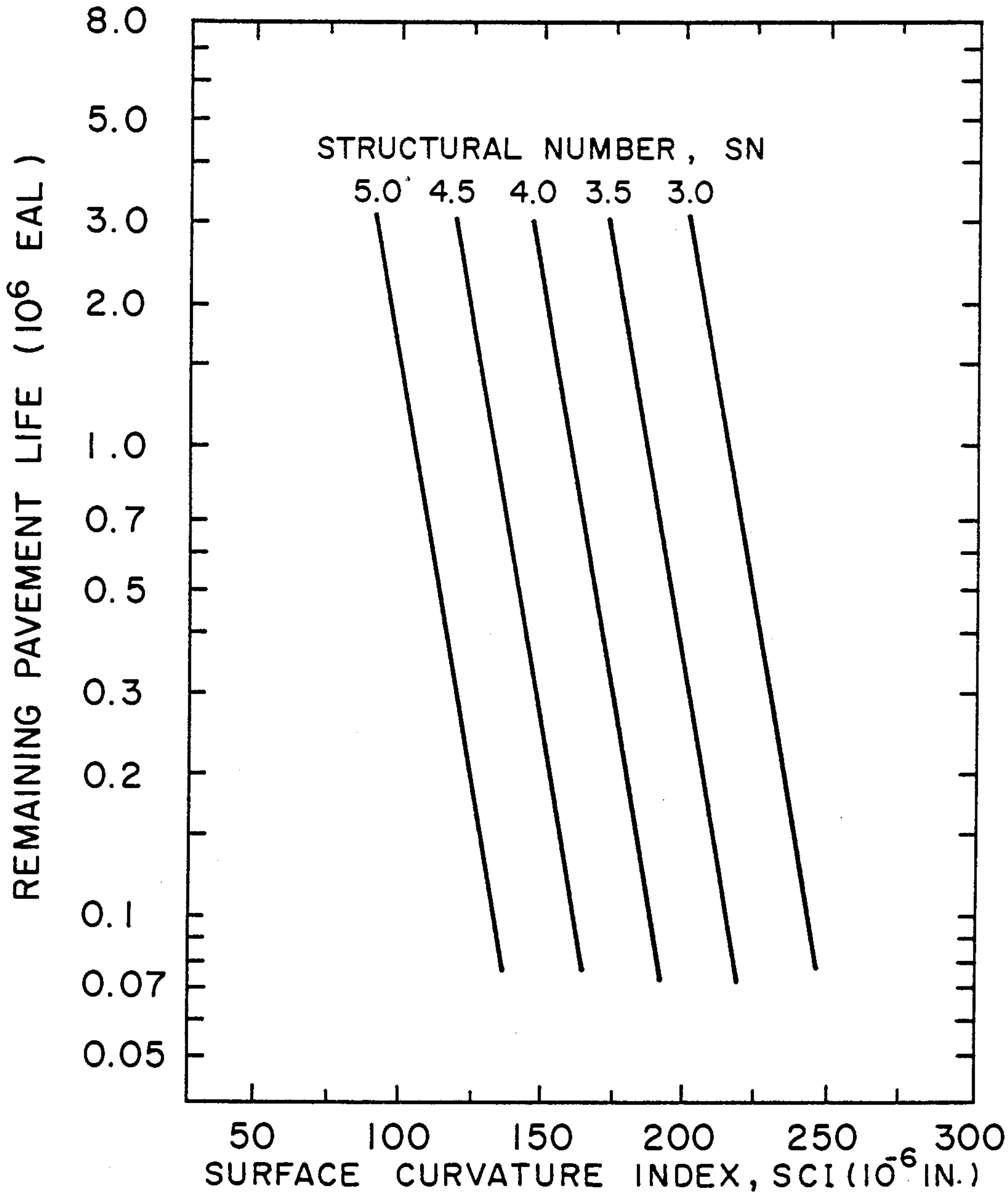


FIGURE C.3
REMAINING PAVEMENT LIFE BASED ON FATIGUE CRACKING FOR BITUMINOUS CONCRETE PAVEMENTS WITH SUBBASE.
(Kilareski, et al., 1982)

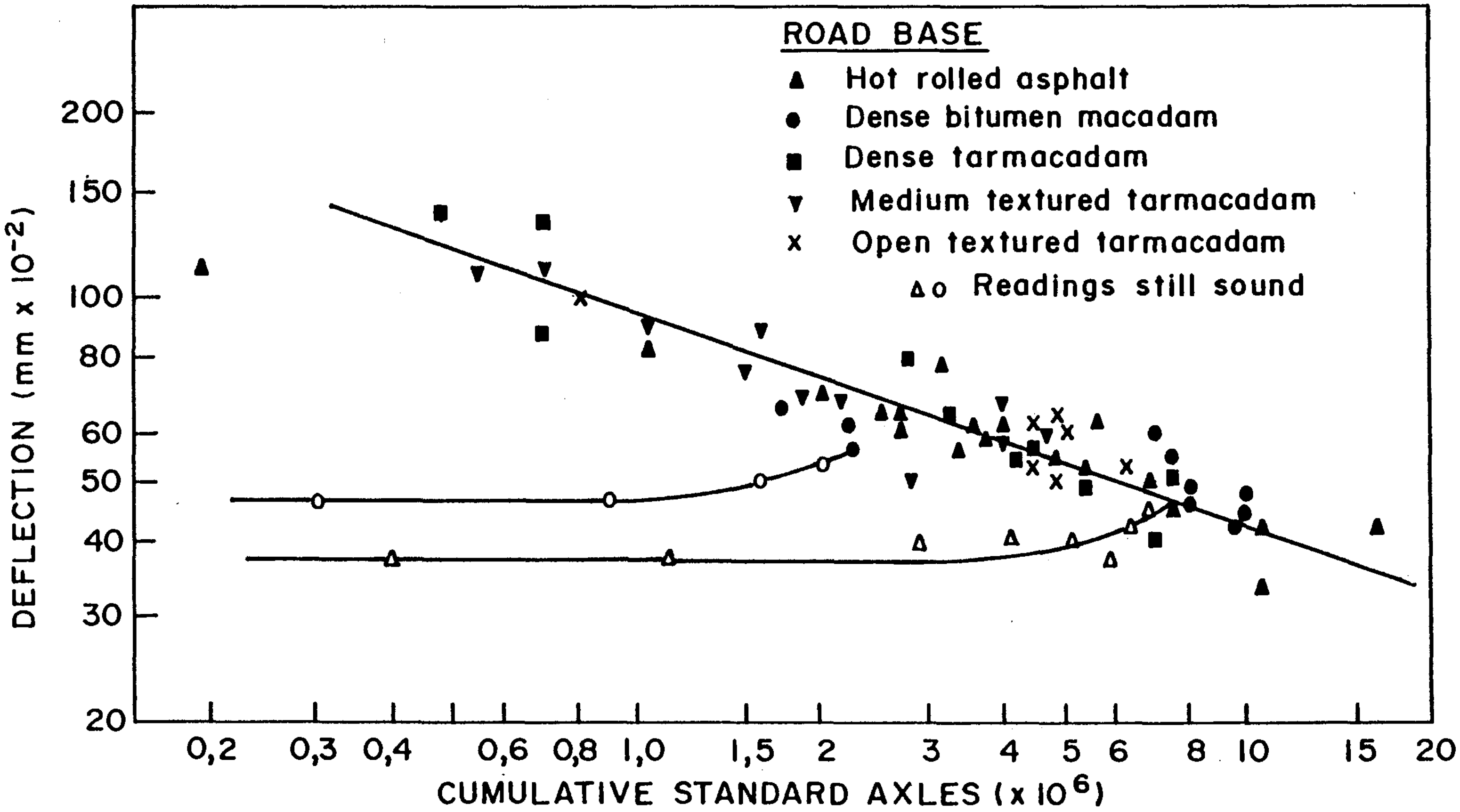


FIGURE C.4

RELATION BETWEEN DEFLECTION AND CRITICAL LIFE OF PAVEMENTS WITH BITUMINOUS AND TAR BOUND BASES

(Lister and Kennedy, 1978)

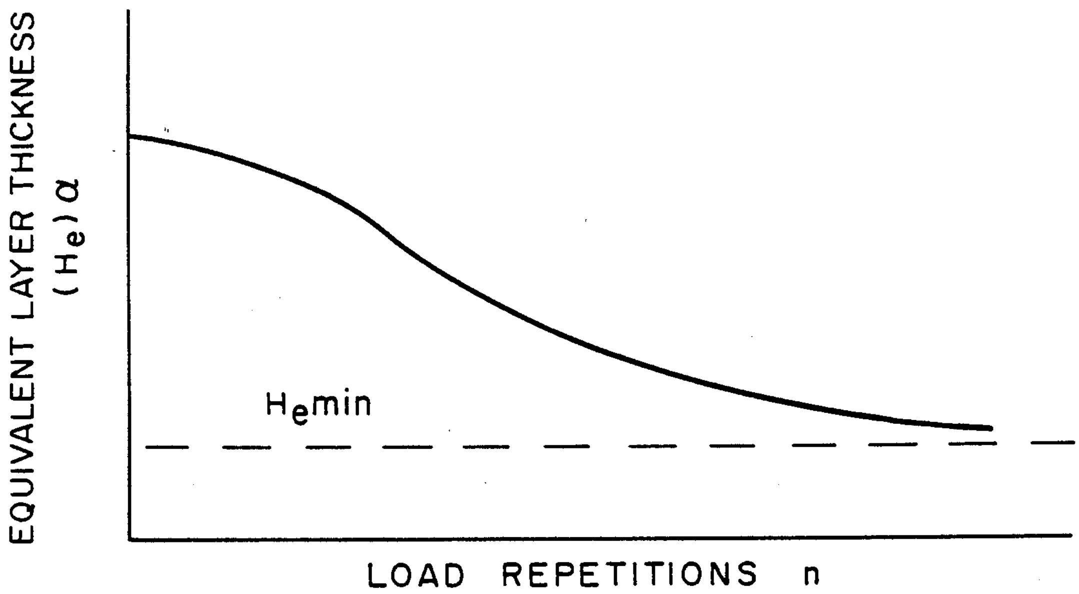


FIGURE C.5
HYPOTHESIZED DECREASE OF THE EQUIVALENT LAYER THICKNESS (H_e) WITH RESPECT TO THE NUMBER OF LOAD APPLICATIONS (n).

relation to the number of load applications(n). The structural condition of the pavement can be characterized by means of the structural condition index P, which is defined as:

$$P = H_{ecn}/H_{eco}$$

where H_{ecn} = equivalent layer thickness after n load applications corrected for temperature and environmental fluctuations

H_{eco} = equivalent layer thickness just after construction corrected for temperature and environmental fluctuations

In order to determine H_{eco}, deflection values between the wheel paths are measured as described above. Molenaar (1983) defines H_{eco} values determined in this way as "candidate" H_{eco} values since they would have been subjected to some loading between the wheel paths. The amount of future deterioration depends on the expected number of load applications, the structural condition index P and the shape of the deterioration function characterized by $S_{\log N}$.

Values for $S_{\log N}$ should also be determined by means of deflection measurements. $S_{\log N}$ can be calculated as follows:

$$S^2_{\log N} = a_1^2 b_1^2 S^2_{\log He} + S^2_{l.o.f(\log N - \log\varepsilon)}$$

where a_1 = slope of fatigue relation

b_1 = slope of H_e versus log ε relation (=2)

$S^2_{l.o.f.}$ = lack of fit of the equation used to describe the fatigue relation (=0,16)

In Figure 4.10 the typical relationship between H_e and surface curvature index (SCI) is shown from results of deflection basin measurements. In Figure C.6 the structural condition index P is

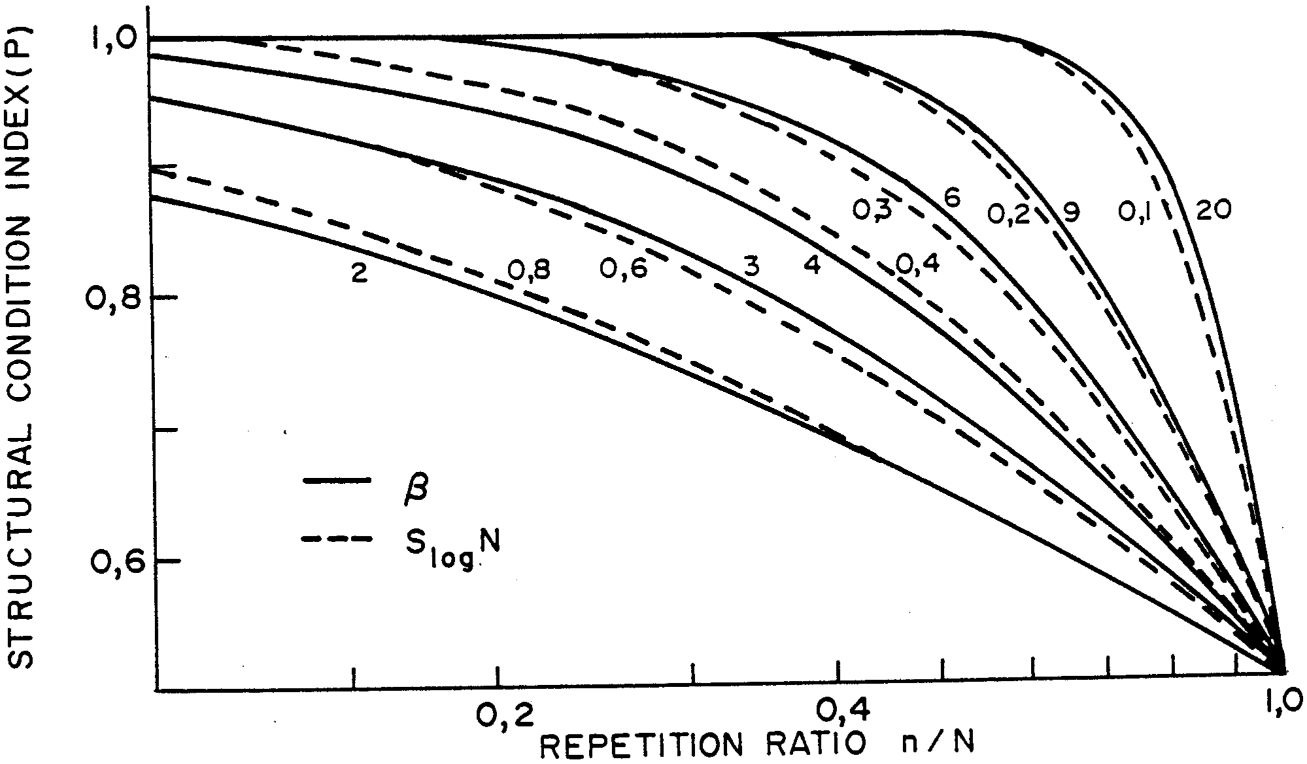


FIGURE C.6
COMPARISON OF THE THEORETICALLY DERIVED STRUCTURAL PERFORMANCE MODEL AND THE EQUIVALENT LAYER THICKNESS DETERIORATION MODEL
(Molenaar, 1983)

plotted in relation to deterioration (n/N) and the influence of $S_{\log}N$ can also be seen.

Molenaar (1983) takes this even further by calculating P and $S_{\log}N$ directly from deflection basin parameter values such as surface curvature index (SCI) as follows:

$$P = (SCI_o/SCI_n)^d$$

and $$S^2_{\log}N = d_1^2\, c_1^2\, S^2_{\log}SCI + S^2_{l.o.f}\,(\log N - \log \varepsilon)$$

where SCI_o = SCI at time of construction

SCI_n = SCI after n load applications

d_1 = absolute value of the slope of the SCI versus H_e relation (a reasonable value is 0,53)

c_1 = slope of the log(SCI) in relation to log N (=0,943)

All other variables have been defined before. However, Molenaar (1983) warns as follows:

"Although the procedure to calculate P seems very simple, one should be aware of the fact that in a number of cases the ratios H_{ecn}/H_{eco} and SCI_o/SCI_n might be larger than one.

Remaining life is determined by this procedure as illustrated in Figure C.7.

Molenaar (1983) modified the work done by the Belgian Road Research Centre. He uses the following equation for permanent deformation model:

$$u_p = b_o u_e n^{b_1}$$

where u_p = permanent deformation (m)

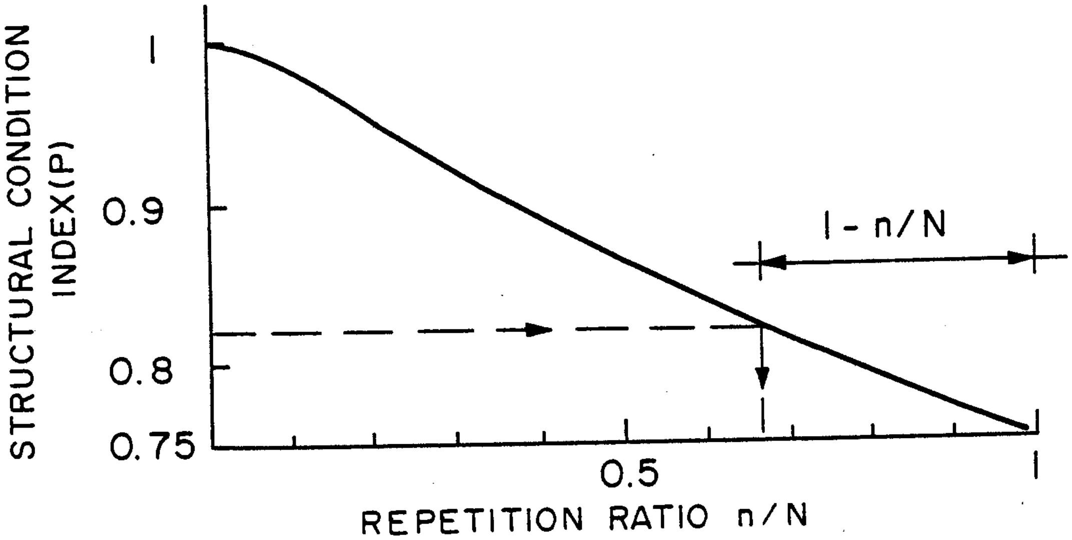


FIGURE C.7
PROCEDURE TO ASSESS THE REMAINING LIFE (Molenaar,1983)

u_e = elastic deformation (m)

n = number of load repetitions

b_o, b_1 = constants

By means of regression analyses of a typical three-layered pavement system Molenaar (1983) used the BISAR computer program to arrive at values of b_o, b_1 and n for the various interfaces between the layers. The elastic deformation at the pavement surface must be known in order to be able to determine the elastic deformation of the top layer. This deformation due to dual wheel loading can be estimated from the maximum deflection (δ_o) value of the falling weight deflectometer (FWD) by using the following equation:

$$\log U_e \text{ surface} = 0{,}09+0{,}948 \log \delta_{o\,FWD}$$

The elastic deformation of each layer can be calculated by subtracting the deformation at the lower interface from the deformation at the upper interface. The permanent deformation can then be calculated by means of the permanent deformation model with constants shown in Table C.1.

A correction factor is applied to relate observed rut depth to these calculated values. By these means rut depth can be related to load applications (n), and consumed rut life can be determined by defining a terminal rut depth of for example 20 mm.

4 CONCLUSIONS AND RECOMMENDATIONS

In chapter 4 and 6 it was concluded that the current mechanistic design procedure in South Africa using distress determinants vertical subgrade strain (ε_{vs}) and horizontal asphalt strain (ε_{ha}) is a sound one. Proper fatigue relationships have been established for these parameters. This makes the use of the linear summation of cycle ratios applicable to both distress criteria: fatigue cracking and deformation rutting. The generalized relationship, ($N = (\frac{1}{\varepsilon})^b$), described by Snaith et al. (1980) can thus be used to

TABLE C.1. Values for $_ob$ and $_1b$ to be used in the calculation of the permanent deformation (Molenaar, 1983)

Materials	$U_p = U_e{*}b_o n^{b_1}$	Modulus (MPa)
Bituminous layers	$U_e{*}4{,}49n^{0,25}$	5 000 (summer)
Stone base	$U_e{*}2n^{0,3}$ if $n<0{,}12$ m $U_e{*}2n^{0,2}$ if $n<0{,}12$ m	500
Lean concrete base	-	1 500
Granular subbase	$U_e{*}2n^{0,3}$	200
Subgrade	$U_e{*}(1+0{,}7 \log n)$	5, 10, 20, 40

determine remaining life for both distress criteria. The more critical value can then be used in the selection of an overlay, as described in Appendix D. Although Freeme et al. (1982a) give a fundamental basis to rehabilitation design in their description of the mechanistic design procedure, it is felt that, particularly in the case of establishing criteria to determine consumed life due to rutting, some advances can be made. This would again be possible with the information available from HVS tests and observed field data. In this regard the approach by Treybig et al. (1978), where the deformation contributions of each layer is better represented by the computed stresses and strains of each layer, should be pursued with the available data.

It is clear too that the approach to relate the remaining life of the pavement to other structural indicators such as the equivalent layer approach shows much promise. The structural performance model suggested by Molenaar (1983) was developed specifically for a three-layered pavement structure and therefore it is obvious that it would not be possible to use this approach in all cases. Instead it is suggested that, with the previously mentioned information available on pavement performance in South Africa, the performance model be established with values determined from regression analyses. This approach would then take into consideration factors such as the deflection basin measuring device, deflection basin parameter selected and pavement structure classification described previously.

The model relating permanent deformation and elastic deformation to the number of load applications seems a sound approach. It would also be possible to establish these relationships with the regression analysis of the information available for the South African condition.

APPENDIX D

CRITERIA FOR OVERLAY

1 INTRODUCTION

This section is a logical continuation of the discussion in Appendix C. In general, the decision to overlay a pavement under analysis will be based on criteria related to the remaining life or consumed life. The distress criteria, fatigue cracking and permanent deformation rutting, are considered separately to determine the remaining life of the pavement. The decision to overlay the pavement is based on the more conservative of the two criteria, but both criteria are checked again to ensure that the prolonged life (remaining life after the overlay) would indeed be achieved. As in any situation where various possible alternatives are generated, sound engineering judgement is influenced by economic considerations. The latter type of decision strongly indicates the typical considerations of a maintenance or pavement management programme and should be viewed against that broader background although the focus here is on a project level based on deflection basin related criteria.

2 RUTTING CRITERIA

Snaith et al. (1980) describe how on the basis of the theory of cumulative damage, the remaining life can be determined. In general this remaining life, as described in Appendix C, would be expressed as: $R_1 = N_1 - n_1$ or $R_1 = 1 - n_1/N_1$. Snaith et al. (1980) do not mention any specific criteria related to this remaining life for decisions to overlay or not. Anderson (1977) bases the decision to overlay or not on the length of the remaining life. If the anticipated or estimated future traffic is more than the remaining life, an overlay is needed. If the remaining life is more than the anticipated traffic over the functional life of the pavement, no overlay is needed. An overlay may be required for other functional reasons such as improving the skid resistance of the riding surface. It is in this regard that Anderson (1977) states that even a nominal thickness of asphalt concrete placed on an existing pavement gives the pavement a new "life" by removing the surface deformation. "There is no theoretical or practical evidence which suggests that the permanent deformation which existed before rehabilitation will affect the future performance of the pavement."

In general Anderson (1977) does support the analysis procedure described by Snaith et al. (1980). For the generalized fatigue relationship ($N=(\frac{1}{\varepsilon})^b$) the aim of an overlay would be to reduce the strain level (ε_{vs}) to the level where the anticipated traffic would meet the prolonged life or remaining life after overlay. This process is shown in Figure D.1 and in a more general form in Figure D.2. The formulation of the fatigue relationship considered by Treybig et al. (1978) (as discussed in chapter 6 and Appendix C) is obviously more complicated. Although no specific mention is made of any criteria for overlays related to remaining life the reasoning above was evidently followed.

Molenaar (1983) does not use his permanent deformation model (see Appendix C) in his proposed overlay design. It is obvious though that this model, if properly calibrated to field performances, would also be able to provide the same criteria based on remaining life as described in Appendix C. If an overlay is needed, the aim would be to reduce the elastic deformation (U_e) and resulting permanent deformation (U_p) of each layer in order to meet the required prolonged life.

3 FATIGUE CRACKING CRITERIA

Remaining life (N_1-n_1) compared with the anticipated or future traffic is the general criterion for overlays, based on analysis using the cumulative damage (linear summation ratio) theory. This has already been briefly described on the basis of the discussion by Snaith et al. (1980) (see Appendix C and sections 2).

In considering the previously defined remaining life, Anderson (1977) also considers the cracked state of the existing asphalt concrete layer and whether the pavement has an asphalt concrete layer when establishing criteria for considering an overlay. The remaining life is automatically zero if the pavement is cracked

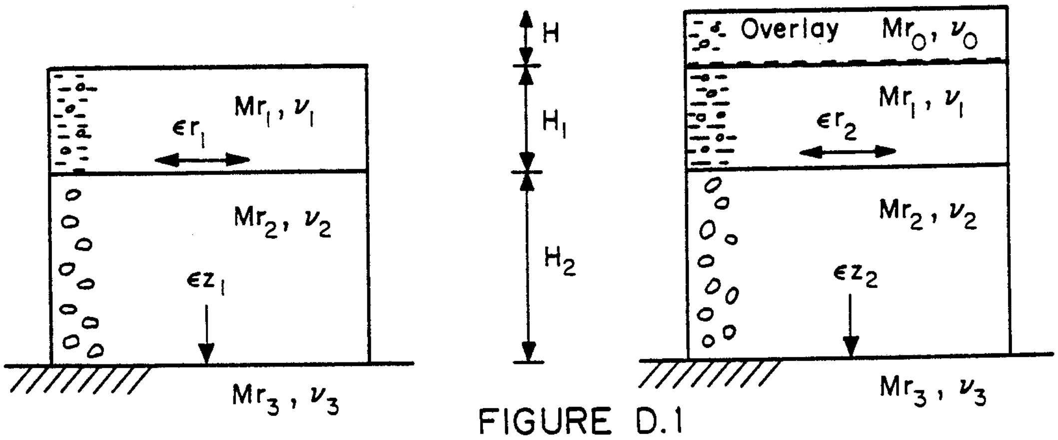
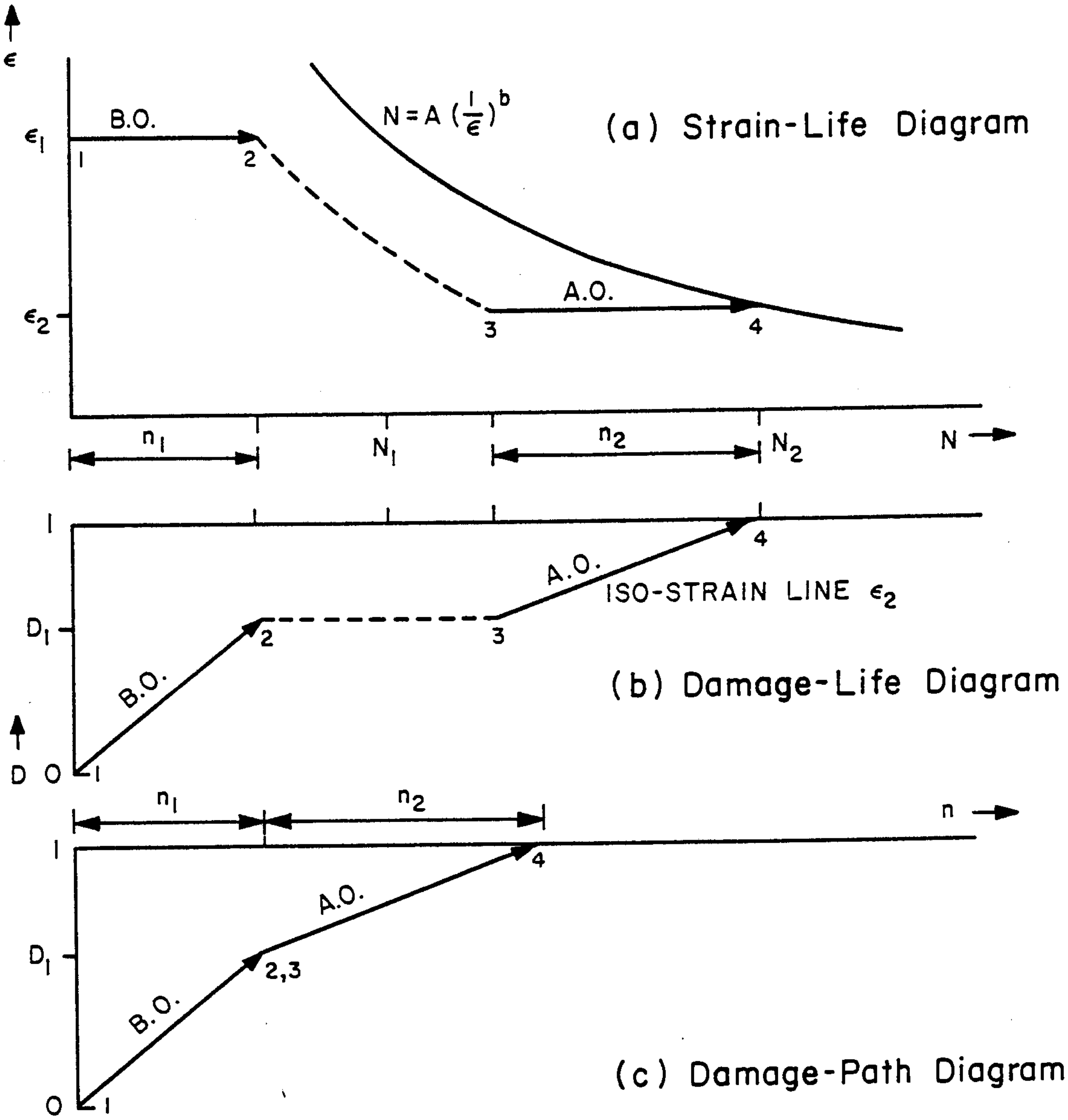


FIGURE D.1
CHANGES IN STRAIN LEVELS DUE TO OVERLAY.



NOTE : B.O. – BEFORE OVERLAY A.O. – AFTER OVERLAY

FIGURE D.2
DAMAGE PROCESS IN A PAVEMENT STRUCTURE WITH A SINGLE OVERLAY.
(Snaith, et al., 1980)

and warrants consideration for an overlay, or if there is no asphalt concrete layer yet. If the remaining life is less than the anticipated traffic an overlay may be considered. If the remaining life is more than the anticipated traffic no overlay is needed. When an overlay is considered as was discussed in section 2 (referring to Figures D.1 and D.2), the aim would again be to reduce the strain level (ε_{HA}) to accommodate a prolonged life or remaining life after the overlay, which would meet the required anticipated traffic life. Analysing various pavements in this way, Anderson (1977) arrived at characteristic curves as shown in Figure D.3. In this figure remaining life is expressed as a percentage of the overlay thickness. The latter value of overlay thickness corresponds to the reduction in strain level (ε_{HA}). Comparing these results with those of a fully cracked asphalt concrete layer with no remaining life, Anderson (1977) concludes that it will always be more economical to neglect any existing asphalt when the remaining life is below 75 per cent. In this overlay design procedure, a "critical" remaining life of 50 per cent was adopted, this being the point at which the existing life is disregarded in designing an overlay. This approach, based on the fatigue relationships described in chapter 6, was also followed by Monismith and Markevich (1983).

The approach by Molenaar (1983), using the structural performance model, obviously differs from the one described above. Molenaar (1983) is quoted as follows:

"Although Miner's law is applicable to the development of one crack, further extension of cracks is dependent on the redistribution of the stresses, and in this case Miner's law may not be fully applicable. Furthermore Miner's law defines a clear failure condition which occurs at e.g. the fracture of a test specimen. Such a failure point does not exist in the case of pavements. A 100 per cent cracked pavement surface can still be used as a reasonable driving surface unless large deformations and/or pot-holes occur. Therefore a straightforward use of Miner's law in the estimation of overlay thicknesses is not considered to be a proper approach, since this will result in an unrealistic overlay design especially in those cases where Miner's ratio comes close to

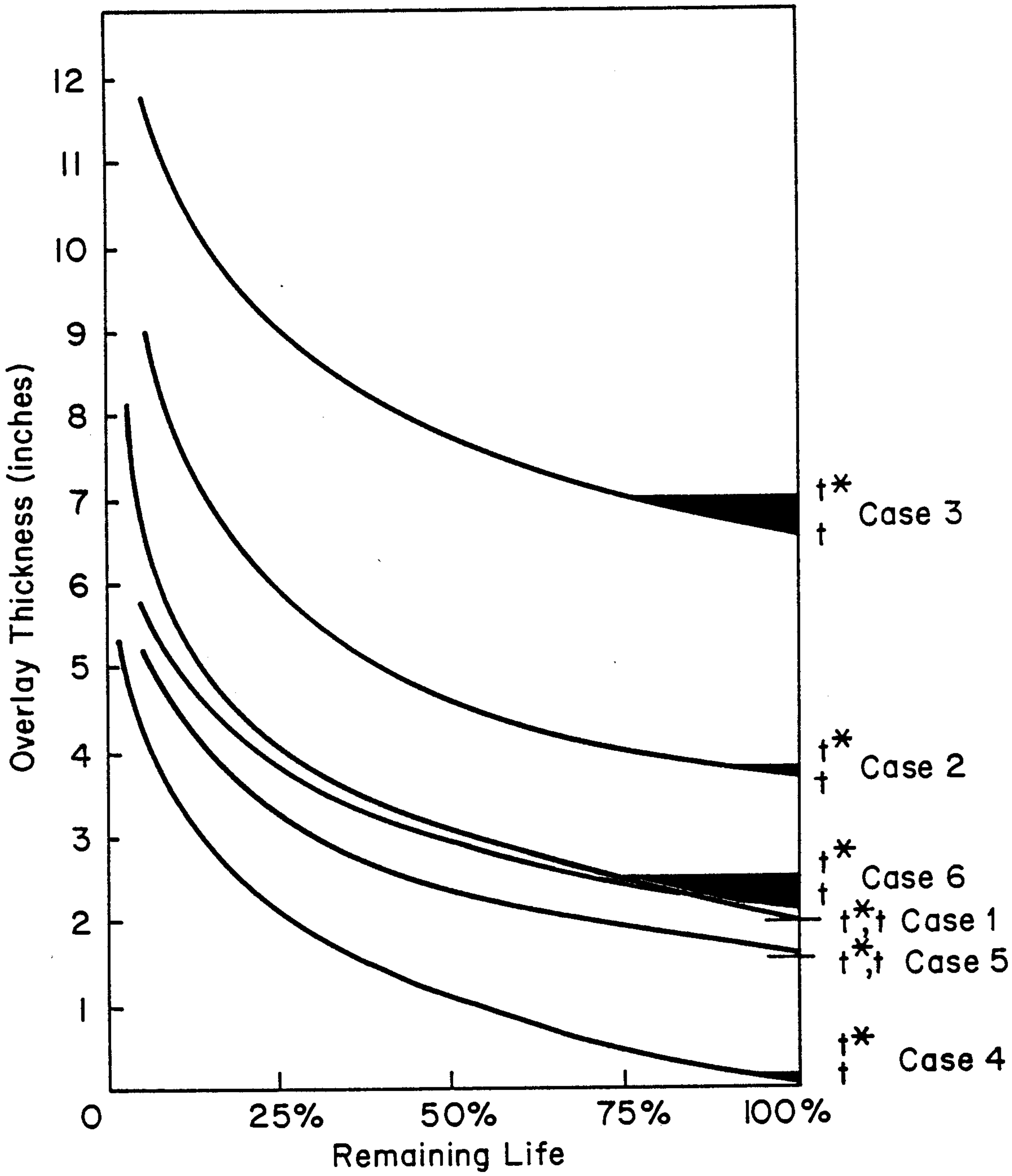


FIGURE D.3
EFFECT OF REMAINING LIFE ON OVERLAY THICKNESS (Anderson, 1977)

one."

This supports the reasoning of Anderson (1977), but also points to the possibility of the structural performance model being used to give a more realistic estimate of the structural life of a bitumen pavement. Molenaar (1983) does not give any specific indication of criteria for decisions on overlays. It is evident from the reasoning, however, that the remaining life determined in this way, would also be used, but with different preconditions.

4 DETERMINATION OF OVERLAY THICKNESS

The two distress criteria, fatigue cracking and permanent deformation rutting have deliberately been considered separately. The reasoning behind this is explained by Koole (1979):

"In determining the thickness required for an overlay, the subgrade-strain and asphalt-strain criteria should be considered separately; it is quite possible that the design criterion that did not govern the original pavement design will become limiting for the overlay thickness."

In this section an overlay thickness is thus decided upon by means of the limiting life of the two defined criteria described in section 2 and 3. The resulting lower distress criteria parameters (ε_{HA} and ε_{vs}) are usually calculated for the possible thicknesses considered. Anderson (1977) calculates these relationships for the various thicknesses of overlays by means of the techniques described in chapter 4. This is shown in Figures D.4 and D.5 for reduction in subgrade strain (ε_{vs}) and asphalt tensile strain (ε_{HA}). In Figure D.3 only pavements with more than 50 mm of asphalt concrete prior to overlaying are considered. The reason was discussed in chapter 6 and in Figure 6.2 what the effect of relatively thin asphalt concrete layers (50 to 75 mm) on tensile strain in asphalt concrete was shown. From the quotation by Koole (1979) above it is obvious that an overlay of for example 25 mm on the existing 25 to 40 mm of asphalt for rut requirements, could in fact shorten the remaining life of the fatigue cracking requirements. The desired

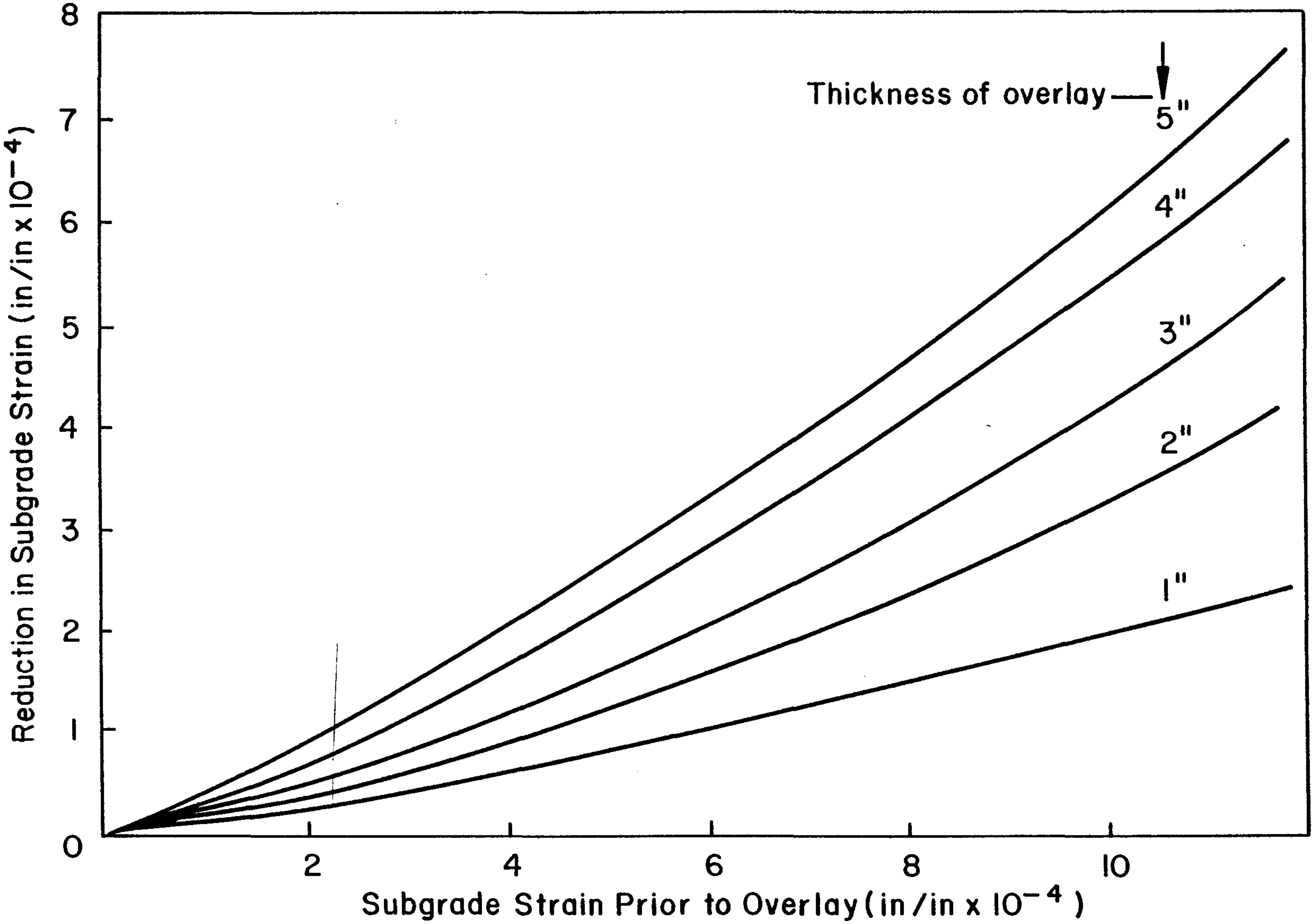


FIGURE D.4

REDUCTION IN SUBGRADE STRAIN DUE TO OVERLAY (Anderson, 1977)

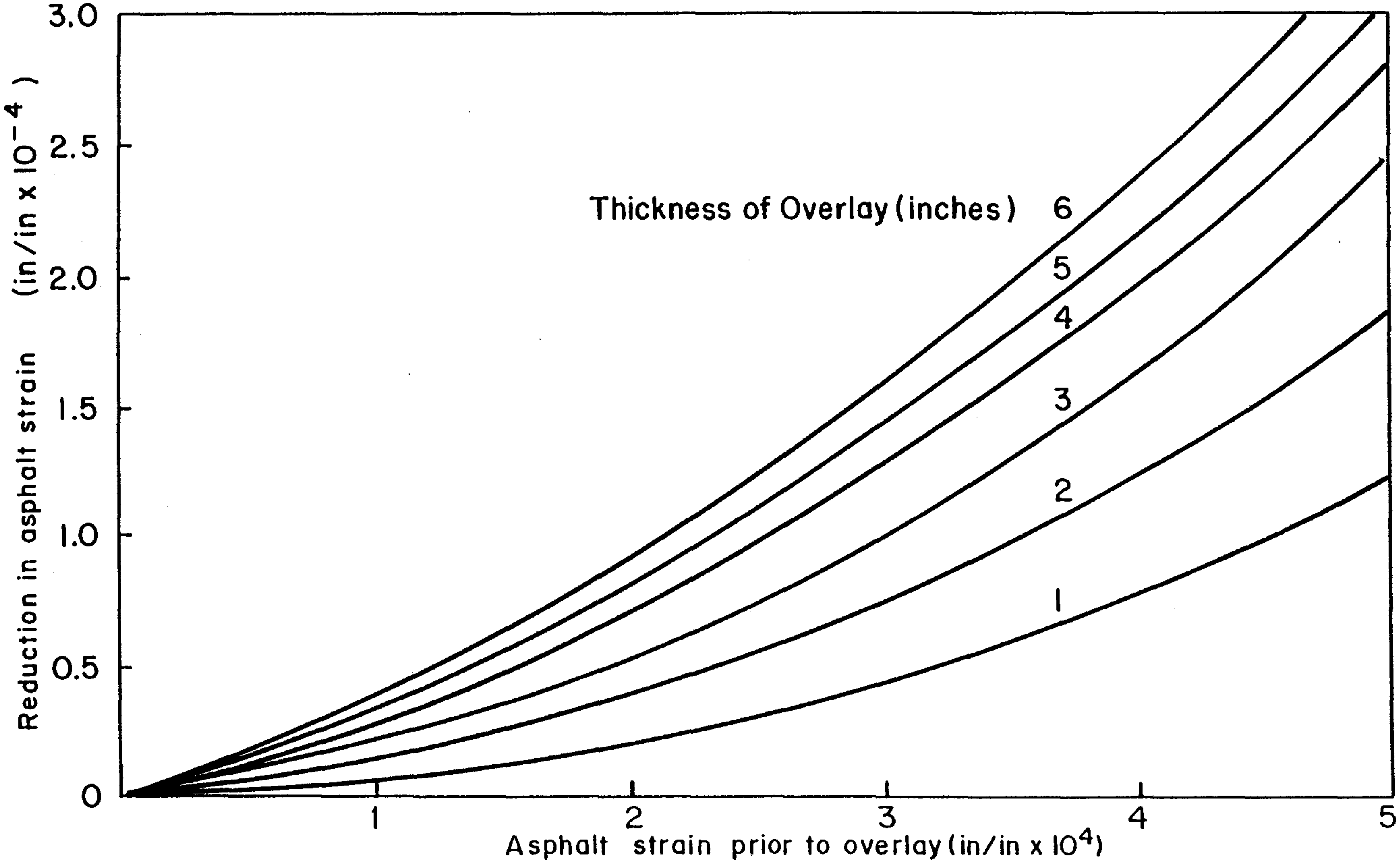


FIGURE D.5
REDUCTION IN ASPHALT TENSILE STRAIN DUE TO OVERLAY
(Pavements with more than 2" of asphalt concrete prior to overlaying)
(Anderson, 1977)

reduction in strain level can also be expressed in terms of the selected deflection basin parameters, as shown in Figure D.6, according to Anderson (1977). Similarly the desired lower deflection basin parameter such as surface curvature index (SCI) can be related to a higher equivalent layer thickness (H_e) (see Figure 4.10) according to the analyses of Molenaar (1983).

Treybig et al. (1978) established the most comprehensive procedure for considering the effect of fatigue cracking and rutting simultaneously. This is shown in Figure D.7 where the existing asphalt concrete layer is regarded as uncracked. The overlay thickness required, is determined by selecting the thicker of the two thicknesses related to the various criteria for the desired load repetitions.

Koole (1979) also describes how three separate overlay thicknesses are determined. This includes the previously discussed criteria for fatigue cracking and rutting, and also a method of determining thickness based on the assumption that the existing pavement has deteriorated to such an extent that the asphalt concrete layer is treated as a granular layer and the overlay as a "new" asphalt concrete layer.

The latter approach is also suggested by Thompson and Hoffman (1983) when the asphalt concrete layer displays interconnected Class 2 cracking.

5 CONCLUSIONS AND RECOMMENDATIONS

Remaining life in relation to the distress criteria, rutting and fatigue cracking, is the main criterion in the consideration of overlays. The remaining life determined by methods described in chapter 6 was determined for each of the distress criteria separately. For the rutting criterion the views on remaining life vary considerably. The view that remaining life is completely restored by an overlay removing the deformations is widely accepted in overlay design. Using the various models discussed in Appendix C it is possible to determine the prolonged life by lowering the

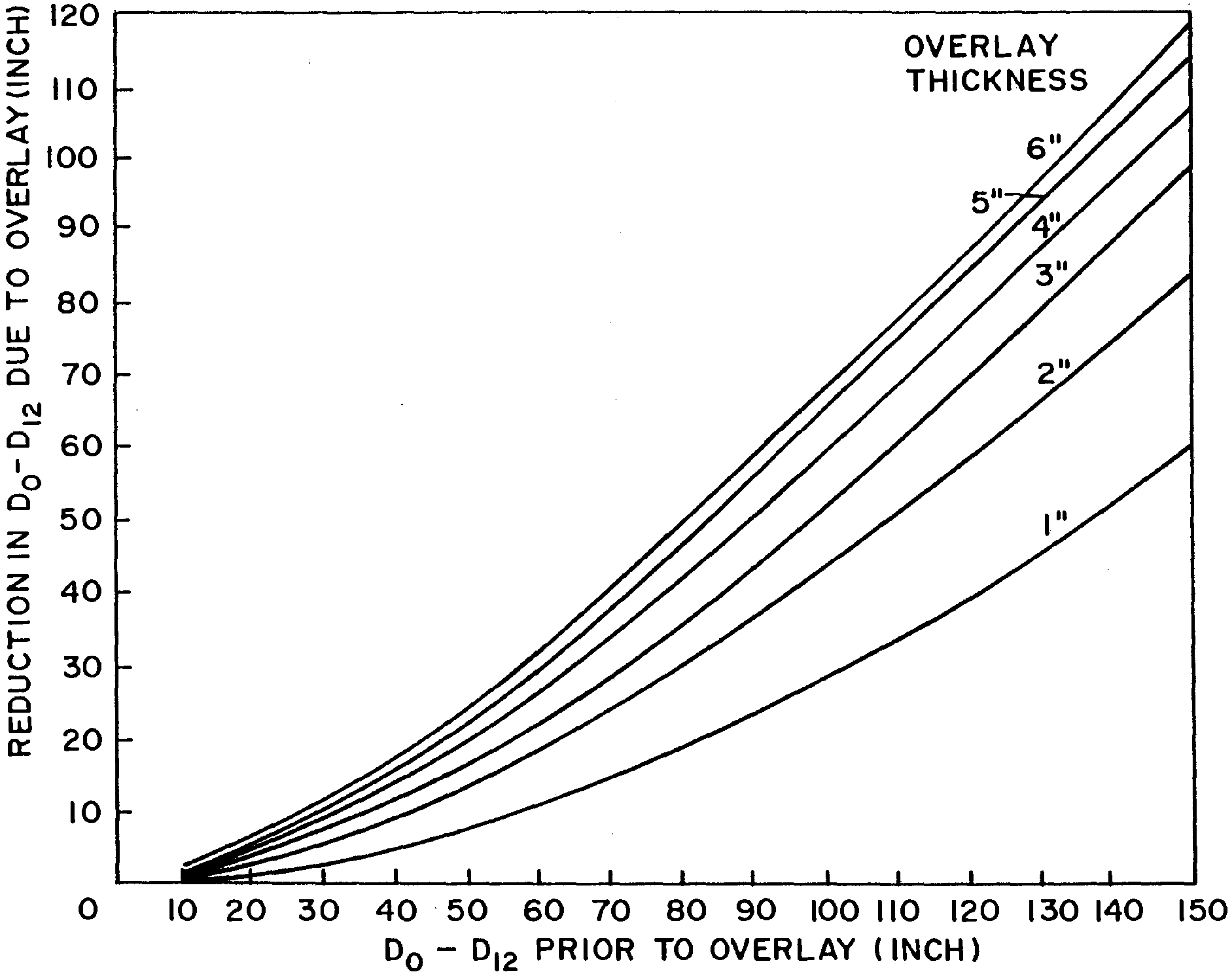


FIGURE D.6
REDUCTION IN $D_0 - D_{12}$ DUE TO OVERLAY
(Anderson, 1977)

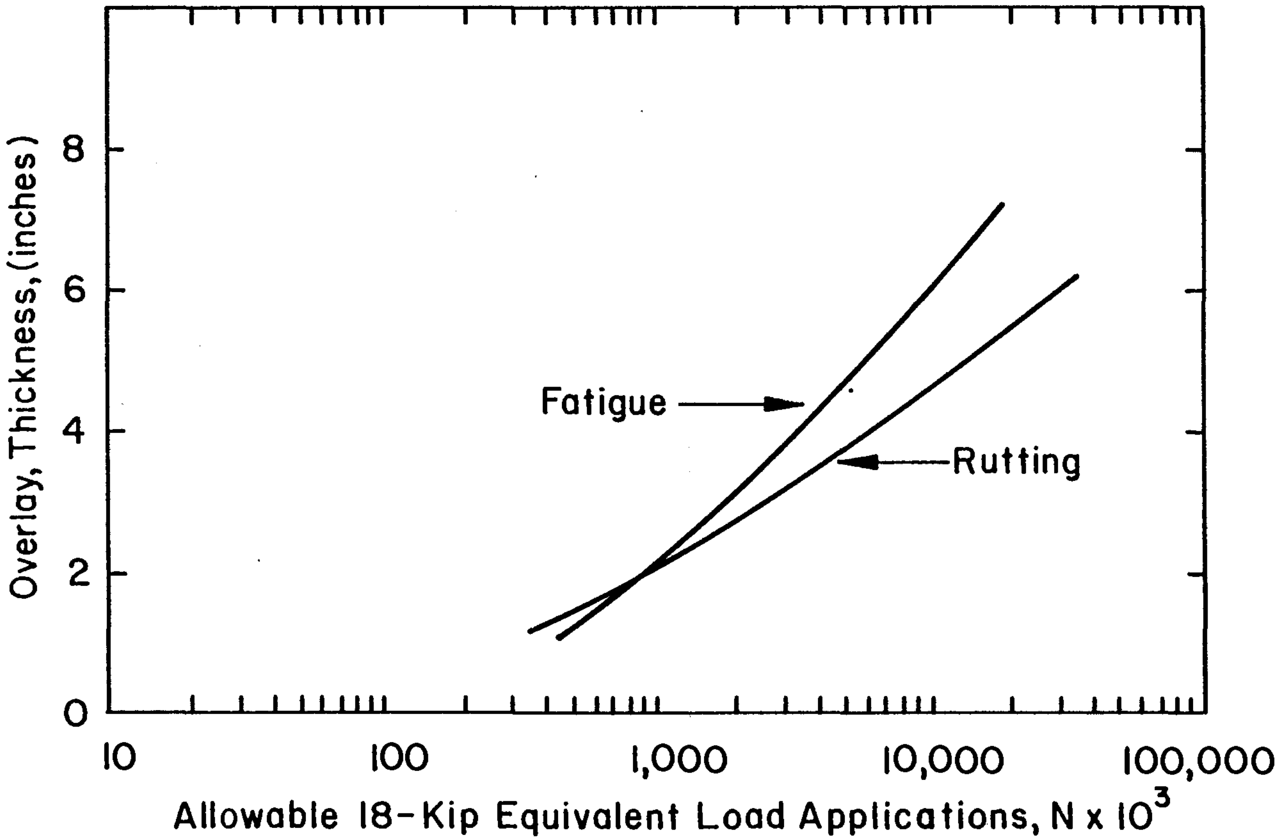


FIGURE D.7
SAMPLE OVERLAY THICKNESS DESIGN CURVES

vertical subgrade strain (ε_{vs}) for example. The more comprehensive model proposed by Treybig et al. (1978) warrants a closer look if it were to be related to the South African situation as were the other proposals and recommendations mentioned before. This may all be incorporated in a proposed catalogue of overlay designs, which would be similar to the existing TRH4 (NITRR, 1985a).

In consideration of remaining life as a criterion for fatigue cracking, the consideration of cracking only leads to some uneconomical overlay proposals. Anderson (1977) indicates that a 50 per cent remaining life for fatigue cracking should be a critical value. The structural performance model by Molenaar (1983) attempts to be more economical by considering the structural value of the cracked asphalt layer. It also offers better consideration of the use of other new materials like bitumen-rubber.

In the final selection of the thickness of the proposed overlay for the critical strain parameter, the emphasis is on checking the other parameter again in order to ensure that the overlay does not shorten remaining life after overlay for the previously non-critical parameter value. From Anderson's (1977) work it is obvious that this would be of particular importance with thin overlays on thin asphalt concrete layers. The approach by Treybig et al. (1978) to plot overlay thickness for both criteria simultaneously in relation to remaining life gives a good graphical indication of such trends.

It has been stated that the selection of an overlay must be seen against the background of maintenance or rehabilitation management systems. The models discussed for analysis are not always applicable to the South African situation. It is therefore suggested that the recommendations in regard to pavement performance and structure were made in previous sections be extended to this area of overlay design in order to make the whole design procedure mechanistically sound. This could easily be incorporated in the suggested catalogue of overlay designs, mentioned above.

APPENDIX E

CRACK MOVEMENT CALCULATIONS WITH DEFLECTION BASIN MEASUREMENTS ON N4/3

1 INTRODUCTION

In Appendix B it was mentioned that there exists a good correlation between the measurements of the Crack-Activity-Meter (CAM) and deflection basin parameters as measured with the Road Surface Deflectometer (RSD). The normal procedure of initial assessment according to the draft TRH12 (1983) guidelines are carried out on a typical cemented base pavement. In the detailed assessment stage the question whether the cracks recorded on specific sections are active or not must then be addressed with confidence. The new service of the NITRR where the CAM and the RSD are combined can then give the required information to make a sound decision in regards to the rehabilitation option. Various cracks with related block sizes and degrees of severity are selected on such a section under investigation. At the same point (crack) the CAM and RSD are set up and measurements are taken with the Benkelmanbeam truck travelling over the crack following the WASHO procedure. The crack activity measurements are then correlated with various other parameters such as block size and deflection basin parameters (Rust, 1984). This appendix therefore describes how such an analysis on the N4/3 was used to verify the rehabilitation option selected in terms of its crack attenuation.

2 PROBLEM DESCRIPTION

The cemented base of N4/3 is cracked and urgently needs rehabilitation. Crack movement measurements were taken in October (Rust, 1986). It was found that there exists sections of road where the crack movements are very high. The block sizes were found to be relatively large. This means that the crack movements are likely to increase as the block sizes break down to a smaller size. The rehabilitation option that was selected is to overlay the existing pavement with a 100 mm G1 crushed stone base and 40 mm asphalt surfacing. The analysis described in this technical note is to determine what the effectiveness of the overlay is to reduce crack movement. In the analysis use was made of measured deflection basins and the correlation thereof with crack movement measurements. This was followed with a mechanistic analysis of the rehabilitation option to calculate the deflection basin and predicted crack movements.

3 CRACK MOVEMENT AND DEFLECTION BASIN CORRELATION

On each of the measuring points of the CAM the deflection basin was also measured with the RSD. The measurement of the whole deflection basin with the RSD makes it possible to determine various deflection basin parameters. The most common deflection basin parameters (Rust, 1986) that can be calculated from RSD measurements are listed in Table 1.1 with their respective formula. The maximum horizontal crack movements (HMAX) and the maximum vertical crack movements (VETOT) in micro-meters were correlated with various deflection basin parameters. The results were as follows:

$$\begin{aligned} \text{HMAX} = {} & 904{,}271*(\text{MAX. DEFL})^{2,6} - 9{,}483\text{E-}6*(\text{SCI}_{915})^{2,5} + \\ & 3{,}086\text{E-}3*(\text{SCI}_{610})^{1,5} - 2{,}538\text{E-}2*(\text{DI}_1)1{,}3 + \\ & 9{,}81*(\text{SCI}_{305})^{1,4} + 71{,}765 \end{aligned}$$

R-squared = 0,69

$$\begin{aligned} \text{VETOT} = {} & 4931{,}765*(\text{MAX. DEFL})^{5,2} - 1{,}813\text{E-}12*(\text{SCI}_{915})^{5,1} + \\ & 4{,}312\text{E-}8*(\text{SCI}_{610})^{3,6} + 1{,}65\text{E-}3*(\text{DI}_1)^{1,9} - \\ & 1{,}887\text{E-}3*(\text{SCI}_{305})^{1,9} + 49{,}713 \end{aligned}$$

R-squared = 0,83

Where: SCI = Surface curvature index with the subscripts indicating the offset for deflection in mm.

DI_1 = Deflection Index which is the difference in deflection at 127 mm and 305 mm.

MAX. DEFL = Maximum deflection in mm.

VETOT = Total vertical movement in micrometer.

The regression analysis indicate that VETOT correlated better with the deflection basin parameters than HMAX. The reason for that can clearly be related to the relatively large block sizes (Rust, 1986).

4 CALCULATED DEFLECTION BASIN PARAMETERS

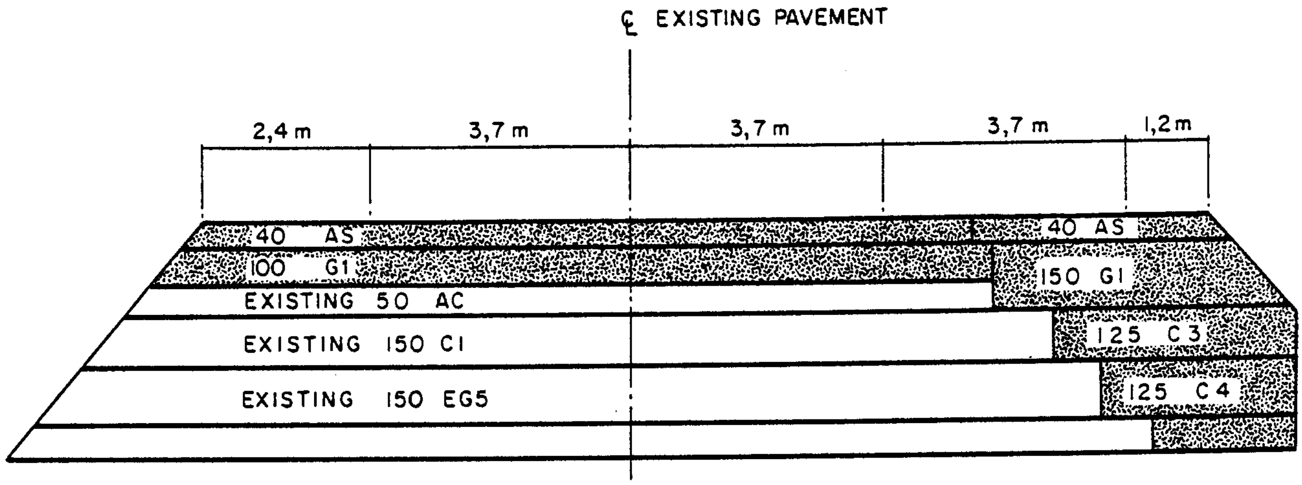
The pavement structures as shown in Figure E.1 for the existing pavement and the rehabilitated pavement were analysed mechanistically with the computer program ELSYM5. The input values are as indicated in the figure. The stress directly on top of the cemented base was calculated before and after the G1 crushed stone base overlay. The calculated vertical stress was 374 kPa and after the overlay it was reduced to 111 kPa. This is a drastic reduction in the calculated stress values and clearly indicates that the overlay did indirectly reduce the possible crack movements.

During the mechanistic analysis the deflection basin was calculated for the two pavement structures. In Table E.1 the relevant deflection basin parameters are indicated. The deflection basin parameters on top of the old cemented base (now sub-base) and on top of the overlayed pavement are shown.

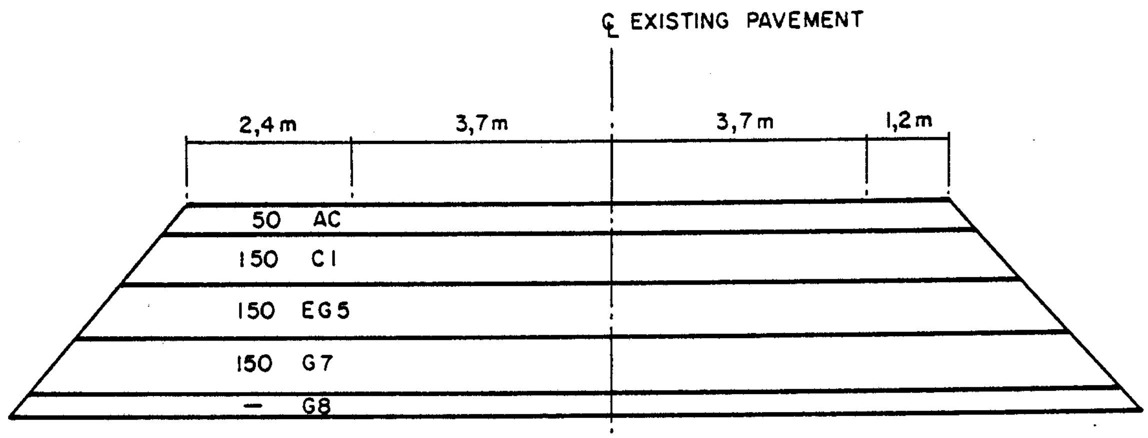
TABLE E.1 Calculated deflection basins

Pavement	Depth (mm)	Deflection basin parameters (mm)				
		MAX. DEFL	SCI_{915}	SCI_{610}	SCI_{305}	DI_1
Existing	0	0,434	0,300	0,245	0,144	0,098
Overlayed	0	0,373	0,253	0,209	0,135	0,084
	140	0,351	0,230	0,186	0,108	0,075

When comparing the deflection basin parameters calculated on the surface directly it is shown in Table E.1 that the overlay reduces the deflection basin parameters values drastically. This reduction in the respective deflection basin parameter values are even more when the values calculated on top of the cemented base are compared.



OPTION 1



EXISTING PAVEMENT

FIGURE E.1

REHABILITATION OPTION ON N4/3

The regression analysis described earlier were also used to calculate the respective predicted crack movements. These results are shown in Table E.2.

TABLE E.2 Calculated crack movements (micro-meter)

Pavement	Depth (mm)	HMAX	VETOT
Existing	-	175	144
Overlayed	140	131	71

The results in Table E.2 show that there is a drastic reduction in the vertical and the horizontal crack movements. In the case of the vertical crack movements the reduction was more. This vertical movement was the more severe case for crack movement due to the relatively large block sizes.

5 CONCLUSIONS

(a) The crack movements (HMAX and VETOT) were correlated with various deflection basin parameters as measured with the CAM and the RSD.

(b) The vertical stress calculated on top of the cemented base show a drastic reduction in values when compared with the vertical stress values calculated on top of the cemented sub-base of the overlayed pavement. This reduction indicates that there should be a reduction in crack movements too.

(c) The deflection basin parameters were calculated for the existing pavement and the overlayed pavement. These calculated deflection basin parameters were used in the correlation relationships to determine the calculated crack movements. There is a drastic reduction in the crack movements on top of the cemented base due to the overlay.

APPENDIX F

APPLICATION OF EQUIVALENT LAYER THICKNESS CONCEPT

1 INTRODUCTION

Odemark's (1949) equivalent layer thickness concept is used as a simple method of approximation. It enables the transformation of a multi-layered system into a single layer with equivalent thickness. The principle is that the equivalent layer has the same stiffness as the original layer, so as to give the same pressure distribution underneath the layer. This concept of classifying a pavement with one number that represents more or less the bearing capacity of that pavement is clearly illustrated by Molenaar and Van Gurp (1980) and Molenaar (1983). The typical South African pavement structures that were analysed in chapter 7 were also converted to the equivalent layer thickness. The equivqlent layer thickness values calculated were then rlated to various distress determinants and fatigue life in order to evaluate this concept as a possible aid in the mechanistic rehabilitation design procedure.

The equivalent layer thickness is calculated as follows:

$$H_e = a\Sigma^{L-1} h_i \left| \frac{E_i(1-\nu_s^2)}{E_s(1-\nu_i^2)} \right|^{1/3}$$

where

a = 0,9 for flexible pavements
h_i = thickness of layer i in m
E_i = elastic modulus of layer i in N/m^2
E_s = elastic modulus of subgrade in N/m^2
ν_i = Poisson ratio of layer i
ν_s = Poisson ratio of subgrade layer with value equal to 0,35
L = Number of layers

2 EQUIVALENT LAYER THICKNESS RELATIONSHIPS

Molenaar (1983) and Molenaar and Van Gurp (1980) analysed a typical three-layered pavement structure. The typical flexible pavement structures referred to in this Appendix differ from this three-layered system in the sense that the pavement structures are either four layered or five-layered systems with a different standard wheel load and tyre pressure. The bitumen base pavements analysed resemble these three-layered pavements most closely in terms of thickness of the bitumen bases. Most of the typical flexible pavement structures analysed, though, have thin asphalt surfacings (≤ 40 mm).

In Figures F.1 and F.2 typical relationships of H_e versus deflection basin parameters, shape factor (F1) and slope of deflection (SD) are shown as calculated for bituminous base pavements. The purpose is to show that some deflection basin parameters like SD, R, SCI, BCI and BDI can discern between the various subgrade elastic moduli while others, such as F1, F2, S, A and Q cannot. In Figure F.3 surface curvature index (SCI) is shown for bituminous and granular bases versus H_e. In both cases SCI can discern between the various subgrade effective elastic moduli. The gradients for these functions of the bitumen base pavements correlate well with bitumen base pavements with three layers (SCI with r = 500 mm) (Molenaar and Van Gurp, 1980). The gradients for the relationships of the granular base pavements though, are shallow and reflect a greater sensitivity to changes in H_e.

Flexible pavements in general were grouped together in Figure F.4 to show that H_e correlates well with vertical subgrade strain (ε_{vs}). The various values of effective elastic moduli of the subgrade lead to different relationships as shown in Figure F.4. In Figure F.5 the other distress determinant, horizontal asphalt tensile strain (ε_{HA}), at the bottom of the bituminous base, is shown versus H_e. Here again, there is a clear discernment between the elastic moduli of the subgrade. It is however not possible to develop the same relationship between ε_{HA} and H_e for the thin surfacings of granular base pavements. One of the reasons for the latter situation seems to be that the thickness, Poisson ratio and elastic modulus ratio of the thin surfacing, compared to that of the base and even subgrade, differ markedly from that of a bituminous base pavement. This is

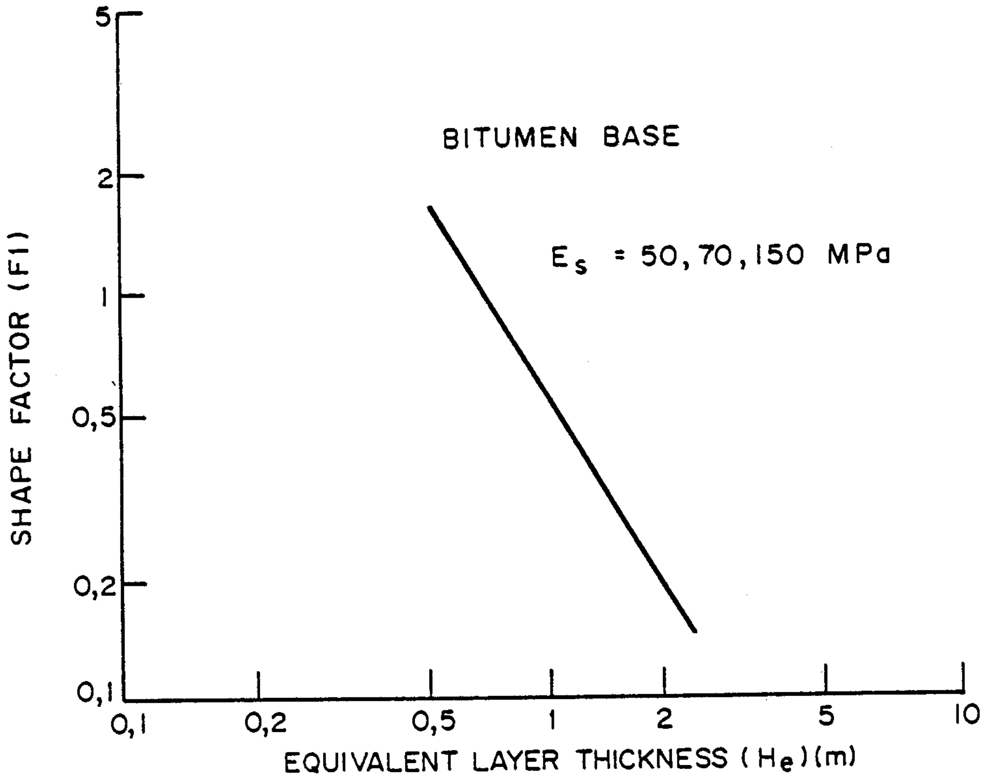


FIGURE F.1

EQUIVALENT LAYER THICKNESS VERSUS SHAPE FACTOR F1

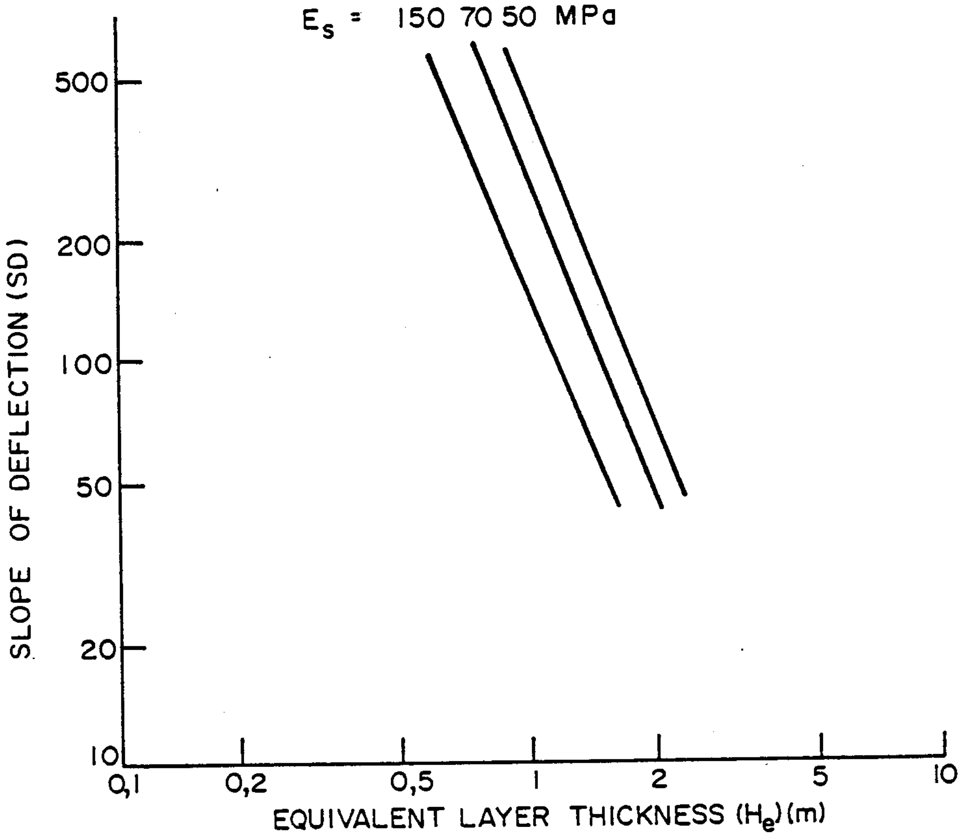


FIGURE F.2

EQUIVALENT LAYER THICKNESS VERSUS SLOPE OF DEFLECTION

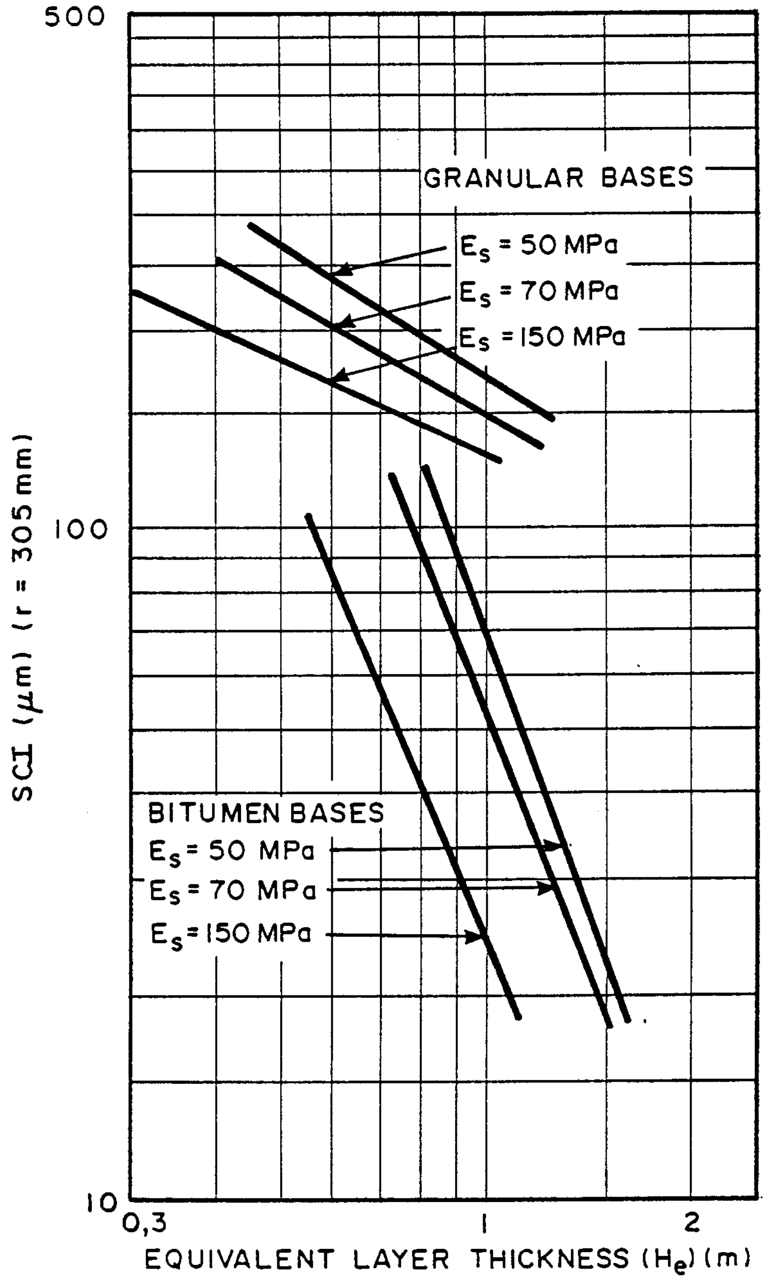


FIGURE F.3
Equivalent layer thickness versus surface curvature index

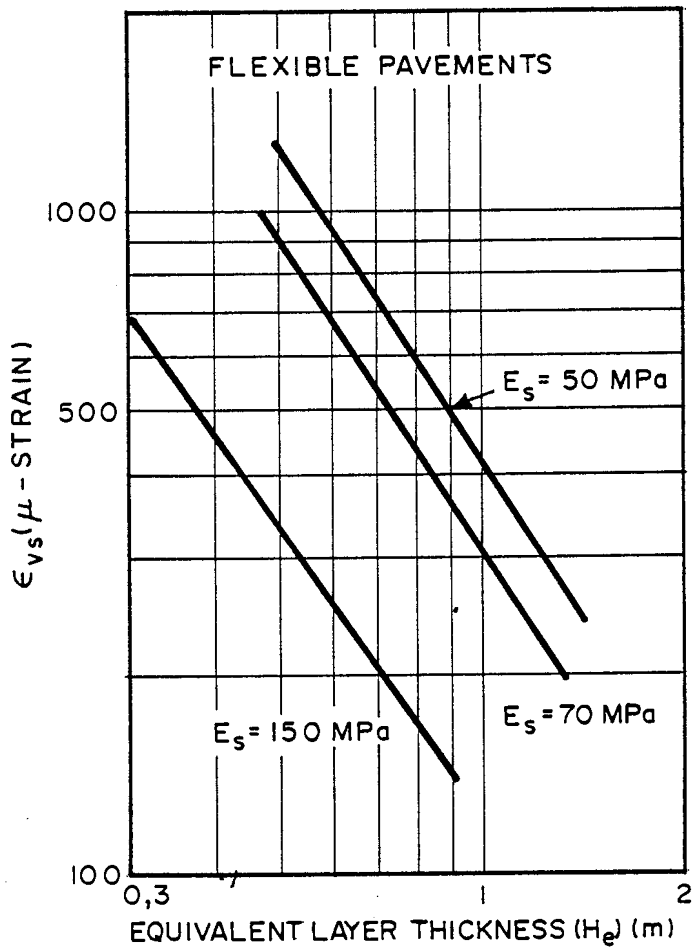


FIGURE F.4
VERTICAL SUBGRADE STRAIN VERSUS EQUIVALENT LAYER THICKNESS FOR FLEXIBLE PAVEMENTS

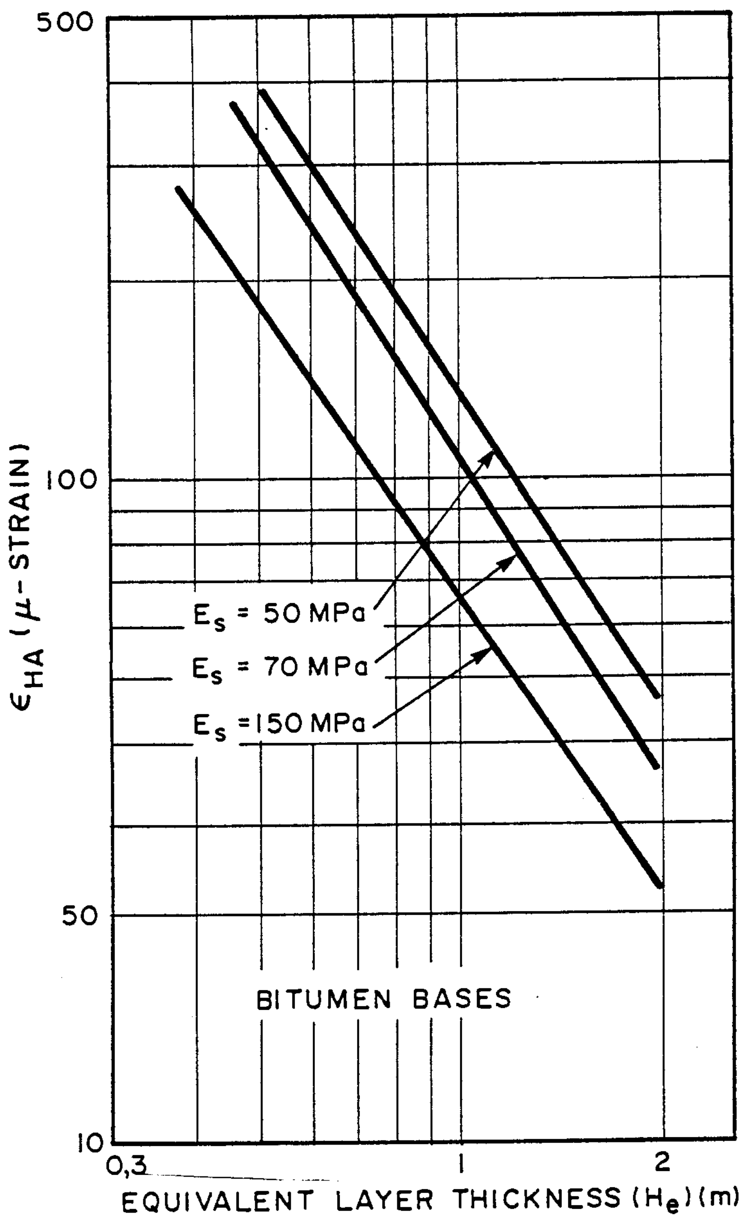


FIGURE F.5

EQUIVALENT LAYER THICKNESS VERSUS MAXIMUM ASPHALT STRAIN FOR BITUMEN BASE PAVEMENTS clear when one looks at the formula for the calculation of H_e, given earlier.

Equivalent layer thickness (H_e) can be used to indicate whether a pavement structure with cemented subbases or bases is in the flexible state, with the cemented layers in the cracked phase exhibiting equivalent granular behaviour according to the definition given by Freeme (1983). In Figure F.6, H_e for the pre-cracked life of pavements with cemented subbases and bases is shown in terms of standard 80 kN axle repetitions (E80s) determined as prescribed by Freeme et al. (1982a). A distinction can be made based on the variance of the elastic modulus of the subgrade. It can be seen, however, that an H_e value of at least 1,1 m is required for a subgrade modulus of 70 MPa to have any significant pre-cracked life of cemented layers. This is rather high and reaffirms that the major portion of the structural life of typical TRH4 (NITRR, 1985a) pavement structures with cemented layers is in the cracked phase or flexible behaviour state.

The recommended vertical subgrade strain (ε_{vs}) criteria for different road categories (Freeme et al., 1982a) were used to calculate the standard 80 kN axle repetitions for all the flexible pavement structures for their respective values of H_e. This relationship between H_e and E80s is shown in Figure F.7 for all flexible pavement structures. In this figure the fatigue life of bitumen base pavements was also calculated with respect to maximum asphalt strain (ε_{HA}) and correlated with the respective H_e value. The recommended fatigue life criteria for thick bitumen base pavements were used in the calculation (Freeme et al., 1982a). The recommended shift factors shown in Table F.1 were applied to the calculated fatigue lives.

TABLE F.1 - Shift factors for bituminous bases

Road category		
A	B	C
2	5	10

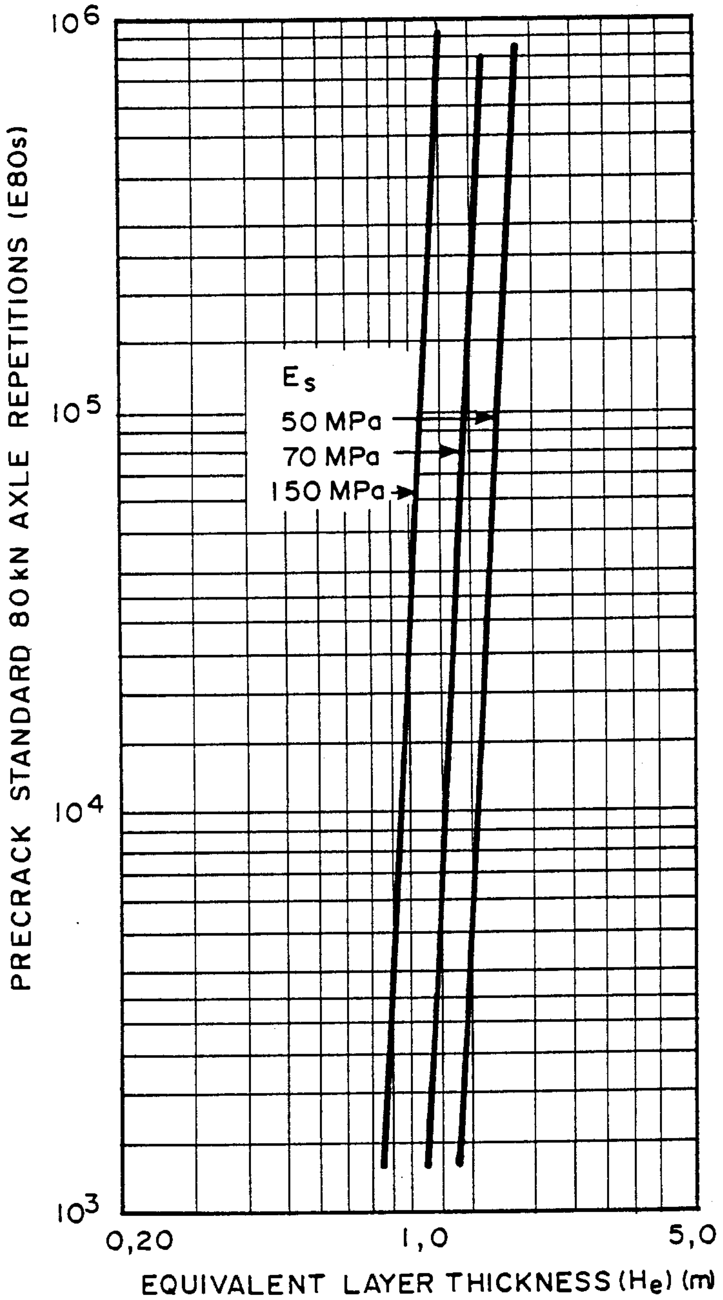


FIGURE F.6

Initiation of cracking of cemented bases and subbases in terms of equivalent layer thickness

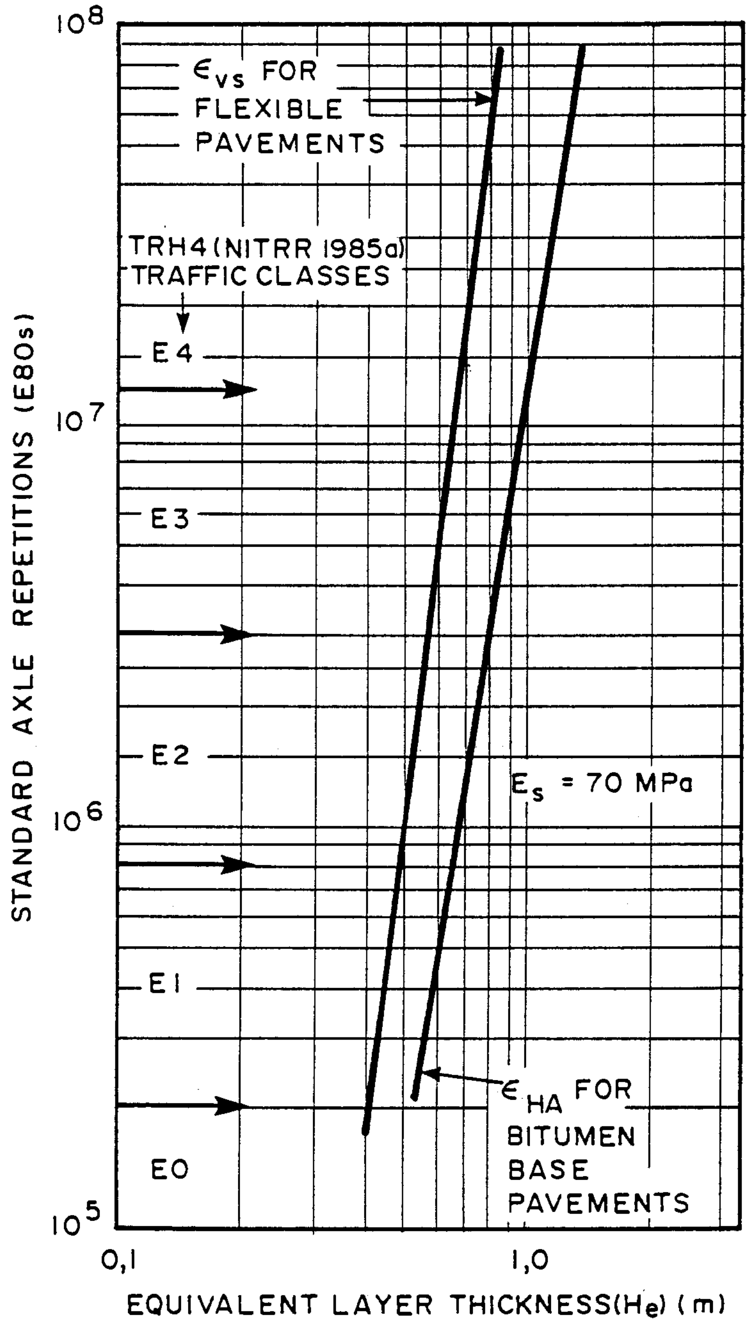


FIGURE F.7

Pavement life for maximum asphalt strain and vertical subgrade strain criteria in terms of equivalent layer thickness

3 CONCLUSIONS AND RECOMMENDATIONS

The equivalent layer thickness (H_e) concept proved to be a concept that more or less represents the structural capacity of flexible pavements. Deflection basin parameters correlate well with a value such as H_e in general, as calculated for flexible pavements. It is however only such deflection basin parameters, that normally use points of deflection near each other in the calculation procedure (e.g. SCI, R, BCI, BDI and SD), that can discern the effect of variance in subgrade elastic moduli. Such relationships however do not have much value except as for an interim step towards establishing relationships between the distress determinants (ε_{HA} and ε_{VS}) and H_e.

H_e correlates well with subgrade vertical strain (ε_{VS}) for flexible pavement structures and discerns the effect of variance of subgrade elastic moduli. Granular bases on the other hand do not give any clear relationships between maximum asphalt strain (ε_{HA}) and H_e as is the case with bitumen base pavements. The reason seems to be the ratios of the thickness, elastic modulus and Poisson ratio of the surfacing and the base as well as that of the subgrade, in the calculation of H_e, which leads to this marked difference between granular and bitumen bases.

The value of H_e can be used in a mechanistic design or analysis procedure to establish the structural life of a flexible pavement with regard to the distress determinants (ε_{VS} and ε_{HA}). The pre-cracked life of a cemented base and subbase layer can be determined. It must be remembered though, that in order to use H_e in such a way, the effective elastic moduli of all the layers as well as the thicknesses have to be known. This restricts H_e to an interim value in the determination of the distress determinants (ε_{VS} and ε_{HA}) in the mechanistic analysis of a pavement.

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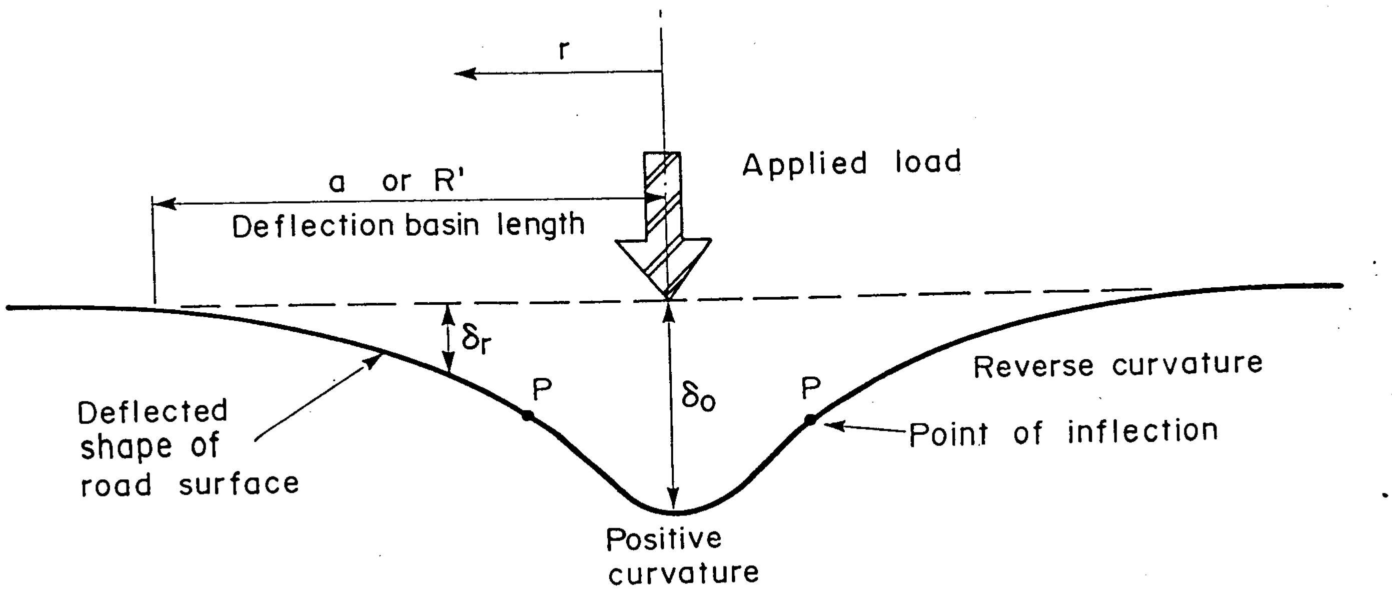
TABLE 1.1 : SUMMARY OF DEFLECTION BASIN PARAMETERS.

Parameter	Formula
1. Maximum deflection	δ_0
2. Radius of curvature	$R = \dfrac{r^2}{2\delta_0(\delta_0/\delta_r - 1)}$; $r = 127$ mm
3. Spreadability	$S = \dfrac{[(\delta_0+\delta_1+\delta_2+\delta_3)/5]100}{\delta_0}$; $\delta_1 \cdots \delta_3$ spaced 305 mm
4. Area	$A = 6[1+2(\delta_1/\delta_0)+2(\delta_2/\delta_0+\delta_3/\delta_0]$
5. Shape factors	$F = (\delta_0-\delta_2)/\delta_1$; $F_2 = (\delta_1-\delta_3)/\delta_2$
6. Surface curvature index	$SCI = \delta_0 - \delta_r$; $r = 305$ or 500 mm
7. Base curvature index	$BCI = \delta_{610} - \delta_{915}$
8. Base damage index	$BDI = \delta_{305} - \delta_{610}$
9. Deflection ratio	$Q_r = \delta_r/\delta_0$; where $\delta_r \simeq \delta_0/2$
10. Bending index	$BI = \delta_0/a$; where a = deflection basin length
11. Slope of deflection	$SD = \tan^{-1}(\delta_0-\delta_r)/r$; where $r = 610$ mm
12. Tangent slope	$ST = (\delta_0-\delta_r)/r$; where r = distance to inflection point
13. Radius of influence	$RI = R'/\delta_0$; where R' is the distance from δ_0 to where basin is tangent to horizontal.

TYPICAL DEFLECTION BASIN



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