CHAPTER 4
EMPIRICAL ANALYSIS AND QUANTIFICATION OF EXISTING PAVEMENT STRUCTURAL DESIGNS

4.1 Introduction

As a precursor to the mechanistic analysis of existing haul road structural designs it is necessary to determine the extent to which current empirical structural design and quantification techniques may be applied to haul road design. This chapter addresses the use of the Dynamic Cone Penetrometer in the context of haul road structural design investigations to analyse the location of various pavement layers, the California Bearing Ratio (CBR) values of these various layers and the overall balance of the structural design.

Whilst the DCP data affords an insight into the actual road structure, as opposed to the design structure and the strength of each layer actually achieved in the field, the extent to which each type of design fulfils the structural performance requirements can only be determined from analysis of the response of each layer to the applied loads. As a precursor to the analysis, the California Bearing Ratio design technique is introduced, in which CBR data generated from the DCP investigation is compared to actual cover requirements predicted from the CBR design method.

4.2 Dynamic Cone Penetrometer Analysis of Pavement Structures

Although the DCP instrument is ideally suited to the evaluation of existing pavements, the original research was used to establish a simplified design method for new pavements. This design approach is in principle similar to the CBR approach in that over-stressing of the lower layers are prevented through a balanced increase in layer strength. The DCP apparatus is used in the context of haul road structural design investigations to analyse;

(i) the location of various pavement layers
(ii) the California Bearing Ratio (CBR) values of these various layers
(iii) the overall balance of the structural design

Following Kleyn et al (1982) the "DCP Structure Number" (DSN) was postulated as being a function of the thickness (h) of a layer of material and its DCP Number (DN) (mm/blow) such that:

$$Layer\ DSN = \frac{h}{DN} \qquad [4.1]$$

Thus the DSN is equal to the number of hammer blows to penetrate a certain thickness. The DCP structure number for the total pavement is thus the sum of the separate layer DSN's:

$$Pavement\ DSN = \sum_{1}^{n} \frac{h_1}{DN_1} + \ldots + \frac{h_n}{DN_n} \qquad [4.2]$$

The pavement DSN is subscripted according to the total depth of analysis, ie. DSN_{800} or DSN_{1800} depending on the method used. The information thus obtained is usually presented graphically, showing the relationship between the number of blows (horizontal axis) to the penetration depth (vertical axis). From this information, a first attempt at layer interface recognition can be made by considering changes in the slope of the graph. Typical DCP curves for a mine test site are given in Figure 4.1 from which 4 structural layers can be discerned, a medium-strong wearing course to a depth of 100mm, a strong base to a depth of 750mm followed by a weaker lower base to 1460mm and sub-base to beyond 1800mm.

The layer-strength diagram for the corresponding site, shown in Figure 4.2, is derived from the DCP curve. It relates the depth of each layer (vertical axis) to the percentage CBR on the horizontal axis. The following formulae are used;
If average penetration rate (DN) > 2mm/blow then;

$$CBR = 410 \times DN^{(-1,27)} \qquad [4.3]$$

and if DN ≤ 2mm/blow then:

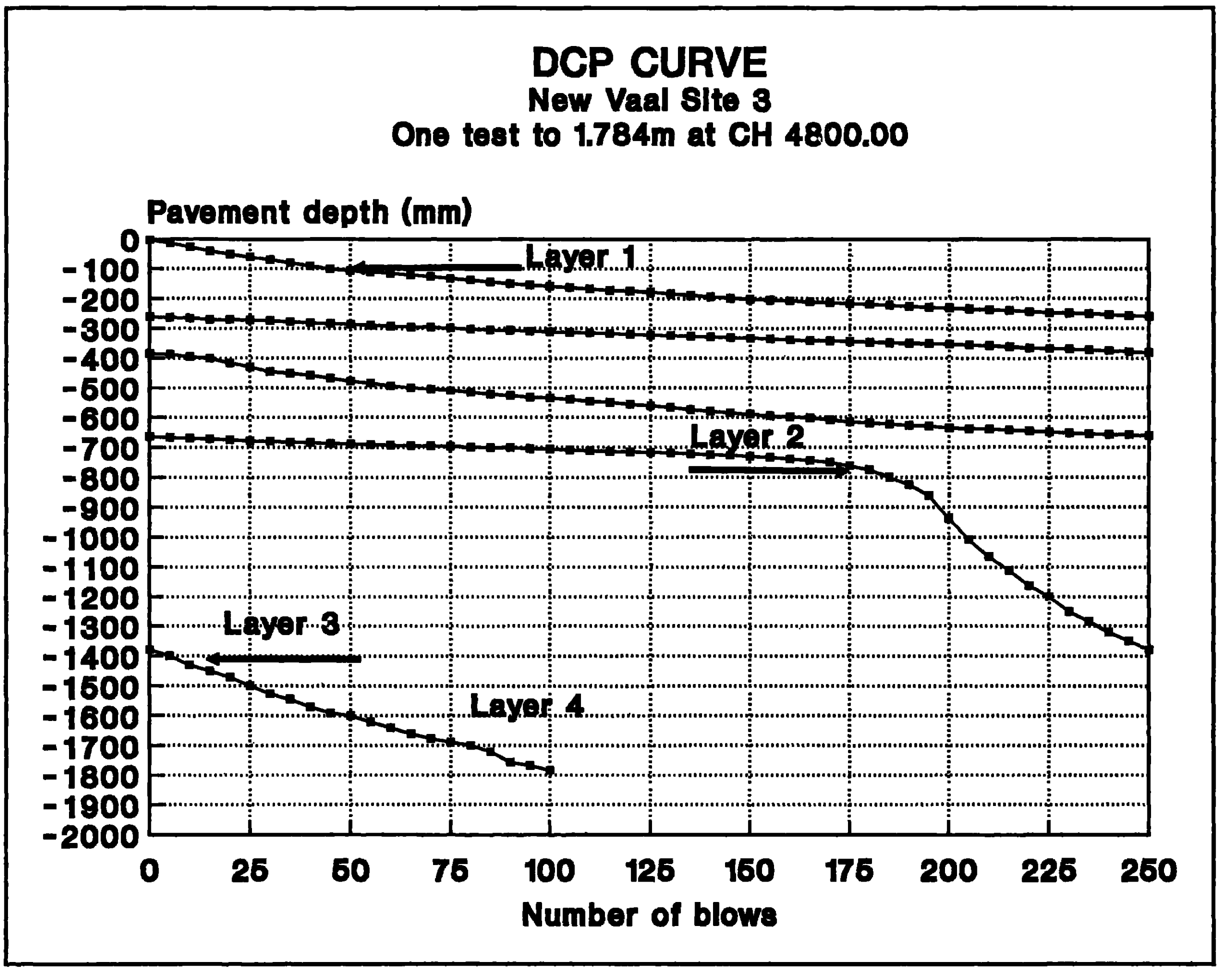


Figure 4.1 DCP Curves, New Vaal Colliery Site 3.

Figure 4.2 shows additionally the redefined CBR layer strength values from values averaged over the layer thickness. Kleyn and Van Heerden (1983) note that the correlation between CBR(%) and DN(mm/blow) is tentative above approximately 200 CBR or DN$\leq$2.

An analysis of pavement CBR values alone does not provide an objective base for interpretation and classification of road structures. The concept of strength balance is useful in providing comparative data and an insight into basic pavement behaviour. Fundamentally, the strength-balance of a pavement structure is defined as the change in strength of the pavement with depth (De Beer et al, 1988b). In general, the strength of the pavement decreases with depth and, in principle, if this decrease is smooth and without discontinuities, the pavement is regarded as being well balanced. The concept of pavement strength-balance is derived from consideration of the cumulative DSN at any point in the pavement, expressed

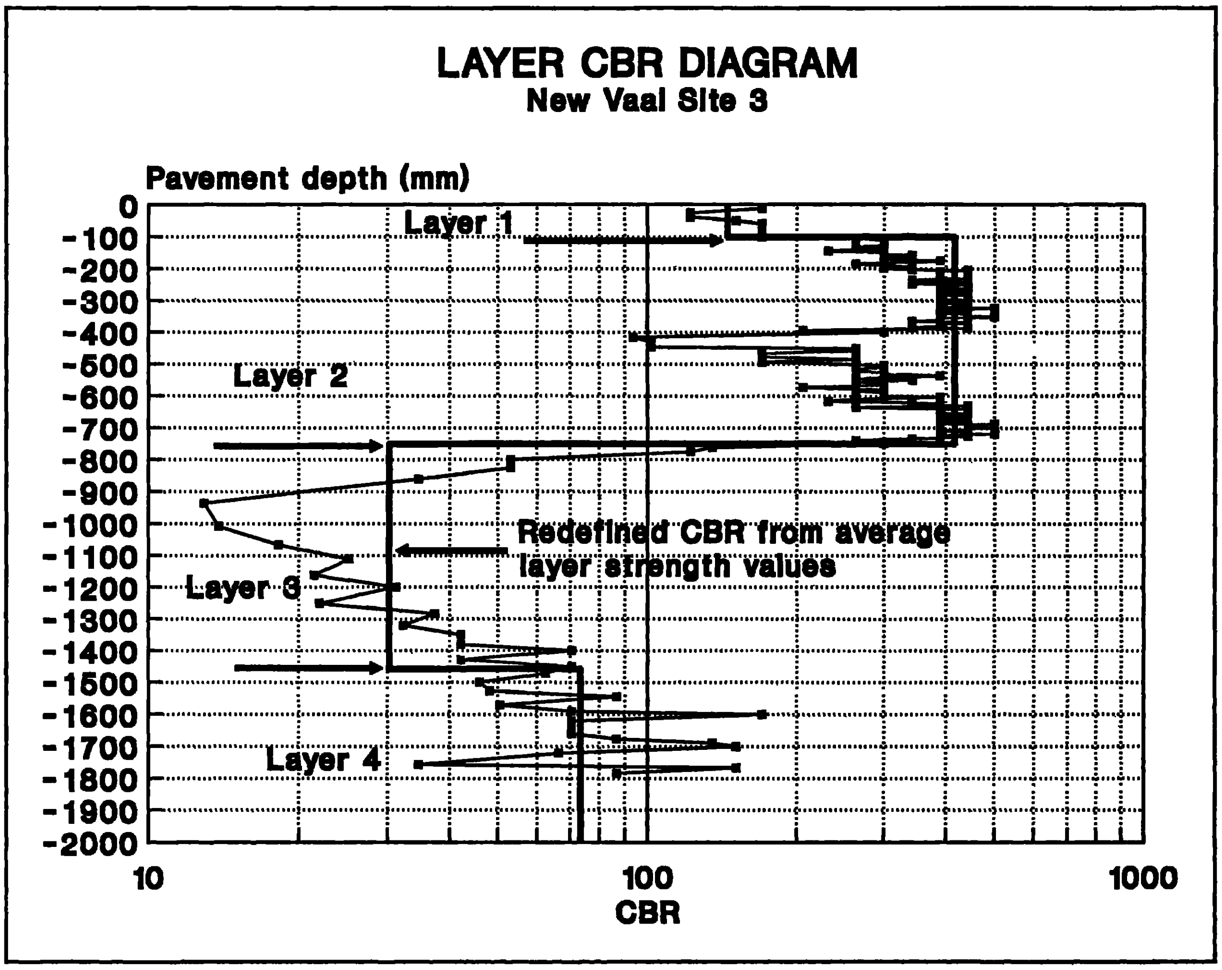


Figure 4.2 Layer Strength Diagram, New Vaal Site 3.

as a percentage of the total DSN over the full pavement depth. Standard pavement balance curves (SPBC) are used by which qualitative or quantitative assessments can be made of the deviation of the structure from a balanced design. SPBC are generated from the following formula;

$$DSN(\%) = \frac{D.[400.B + (100 - B)^2]}{4.B.D + (100 - B)^2} \qquad [4.5]$$

Where DSN = pavement structure number (%)

B = parameter describing the SPBC

D = pavement depth (%)

Figure 4.3 illustrates a number of SPBC from -60 to +60 which represent the extent that strength increases or decreases with depth respectively. The higher the SPBC number, the greater is the contribution to overall pavement strength from the upper (shallow) road layers. Deviation from a SPBC represents the state of imbalance in the structure and can be quantitatively assessed from consideration of the areal deviation (A) which represents a "goodness of fit" parameter for the pavement. For the purposes of this exercise however, a qualitative description of the strength-balance of the structure will suffice, based on the quantitative derivation of pavement strength balance categories (Kleyn et al, 1983). These are illustrated in Figure 4.4 and the corresponding descriptions given in Table 4.1.

The following ranges are recognised for SPBC and "goodness of fit" parameter A;

(i) SPBC in excess of 40 for shallow pavements, 0 to 40 for deep pavements and less than 0 for inverted structures.

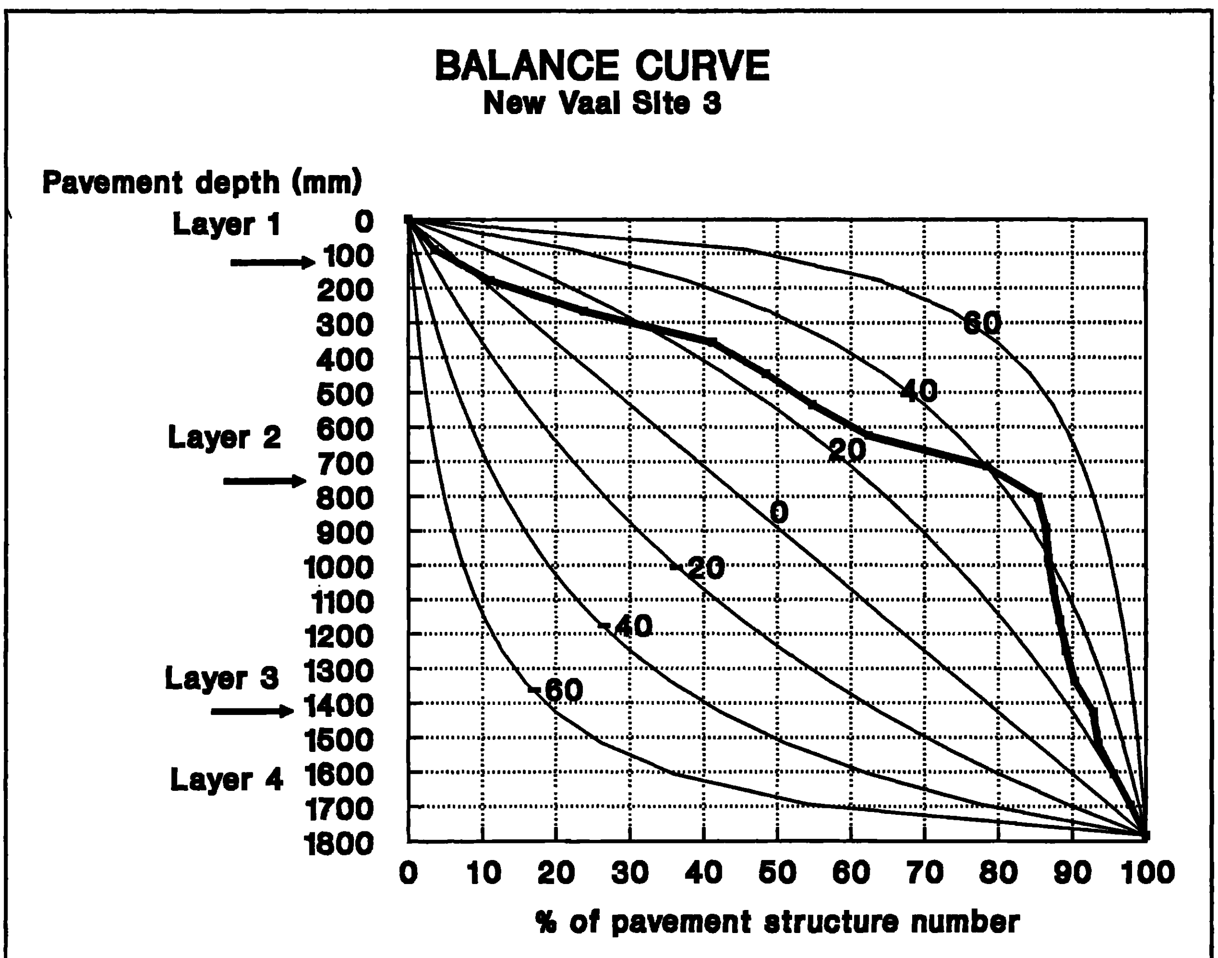


Figure 4.3 Pavement SPBC and Actual Balance Curve for New Vaal Colliery Site 3.

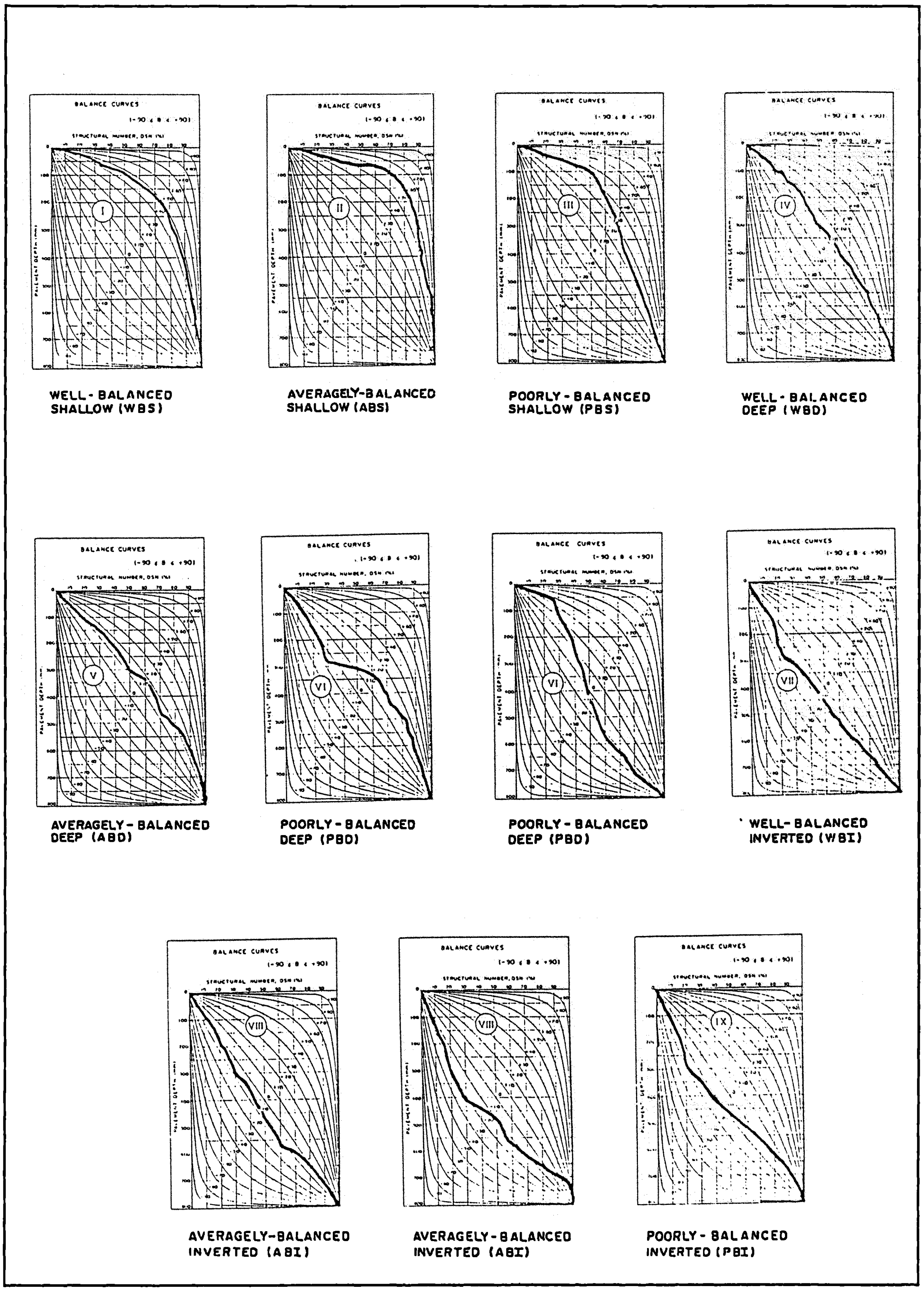


Figure 4.4 Examples of Different Structural Balance Categories (after De Beer et al, 1988b)

Table 4.1 Definition of the Nine Different Pavement Strength-balance categories (after De Beer et al, 1988b).

LIMITS FOR SPBC AND FITTING A	CATEGORY	DESCRIPTION OF CATEGORY
$B \geq 40,\ 0 \leq A \leq 1200$	I	Well balanced shallow structure (WBS)
$B \geq 40,\ 1200 \geq A \geq 3000$	II	Averagely balanced shallow structure (ABS)
$B \geq 40,\ A > 4000$	III	Poorly balanced shallow structure (PBS)
$0 \leq B < 40,\ 0 \leq A \leq 1200$	IV	Well balanced deep structure (WBD)
$0 \leq B < 40,\ 1200 \leq A \leq 3000$	VI	Averagely balanced deep structure (ABD)
$0 \leq B < 40,\ A > 3000$	VI	Poorly balanced deep structure (PBD)
$B < 0,\ 0 \leq A \leq 1200$	VII	Well balanced inverted structure (WBI)
$B < 0,\ 1200 < A \leq 3000$	VIII	Averagely balanced inverted structure (ABI)
$B < 0,\ A > 3000$	IX	Poorly balanced inverted structure (PBI)

(ii) A from 0 to 1200 for a well balanced pavement, 1200 to 3000 for average and in excess of 3000 for poorly balanced.

A typical strength balance curve is illustrated in Figure 4.3 for New Vaal Colliery site 3 from which it is seen the road corresponds to a poorly balanced deep structure. The inferences and implications of the above assessments are discussed for each mine site in the following section. DCP curves, layer CBR and strength balance curves for each site are given in Appendix B.

4.2.1 Discussion of DCP Analysis - Kriel Colliery

The pavement profiles and corresponding CBR values of each layer as determined from the DCP analysis and road construction plans are presented for each site in Figure 4.5 and discussed in the proceeding sub-sections.

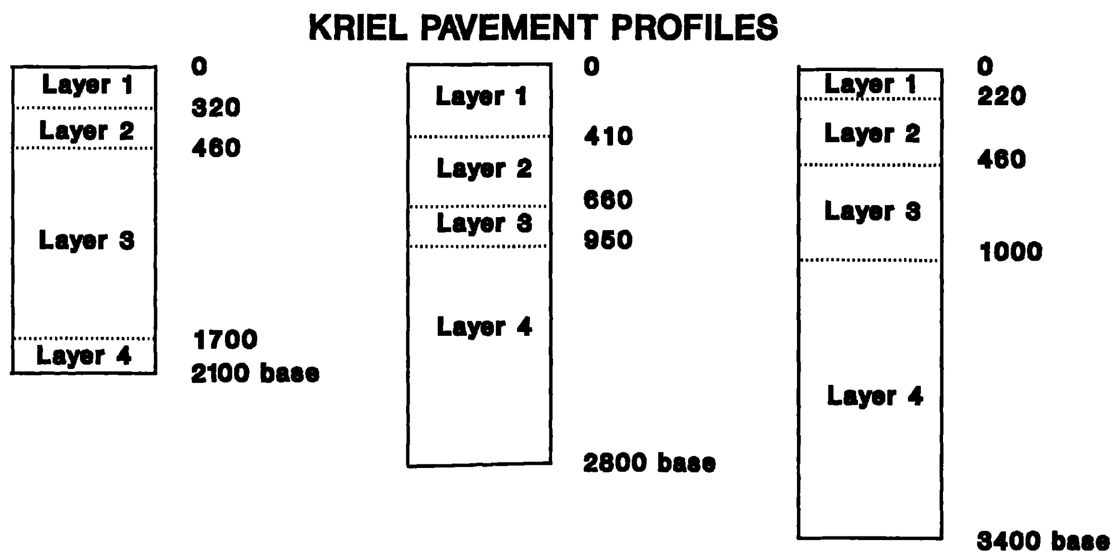
Site 1

Four structural layers were discerned at site 1; a medium-strong wearing course to a depth of 320mm, a medium-weak base 140mm in thickness, a very weak sub-base to a depth of 1700mm and medium-weak selected material layer beyond that. The average CBR values calculated for the layers correlated closely to recorded values, except in the case of the transition between layers 1 and 2 and some large CBR variations in layer 3, albeit over less than 100mm. This is indicative of isolated pockets of poor quality material in the construction.

The balance of the pavement may be described as an averagely balanced shallow structure, although the curve lies below the SPBC=40 curve, since the majority of the strength of the pavement lies in the upper 2 layers.

Site 2

Four layers are discerned at site 2; a medium-weak wearing course extending down to 410mm, followed by a strong base to a depth of 660mm, a very strong sub-base to 950mm



Site 1

LAYER	CBR
1	228
2	56
3	27
4	58

Site 2

LAYER	CBR
1	85
2	178
3	227
4	141

Site 3

LAYER	CBR
1	118
2	428
3	240
4	32

Figure 4.5 Kriel Colliery Pavement Profiles as Determined by DCP Analysis.

and a strong lower sub-base to beyond 1264mm. Layer 1 correlates well with the average CBR value found, but layers 2 and 3 reveal the existence of isolated strong and weak spots within the structure.

The balance of the pavement is thus described as an averagely balanced inverted structure which is borne out by the CBR values of the pavement layers, the strength increasing with depth. From layer 3 onwards, the structure reverts to a well balanced deep structure due to the decreasing layer strength values beyond this point.

<u>Site 3</u>

Four structural layers are discerned at this site consisting of a medium-strong wearing course extending to a depth of 220mm lain upon a stabilised (5% hydrated lime) base 240mm thick which exhibits particularly high CBR values. The sub-base is also of medium strength whilst the lower sub-base consists of weak material beyond a depth of 1939mm. Layers 2 and 3 correlate well with the average CBR calculated whilst layers 1 and 4 show some scatter. In the case of the former a hard compacted layer some 25mm deep is seen to form in the wearing course under the action of heavy traffic.

The balance of the pavement is thus described as a poorly balanced shallow structure, primarily due to the action of the stabilised base which moves the curve from an inverted structure above a depth of 220mm to a well balanced shallow structure below a depth coincidental with the end of the stabilised layer.

4.2.2 Discussion of DCP Analysis - Kromdraai Colliery

The pavement profiles and corresponding CBR values of each layer as determined from the DCP analysis are presented for each site in Figure 4.6 and discussed in the proceeding sub-sections.

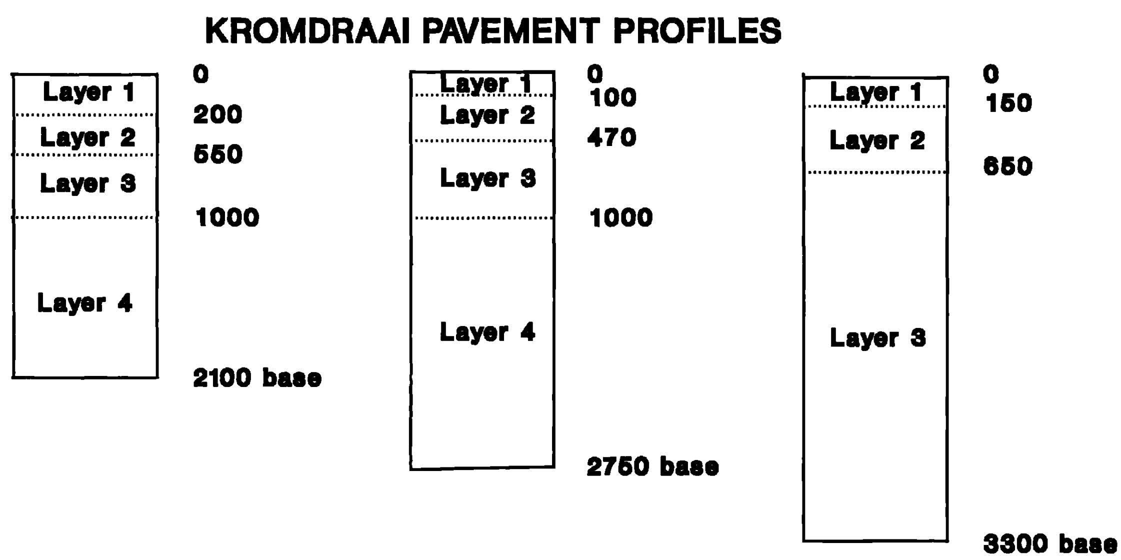
Site 1

Four structural layers are discerned at this site, a very strong wearing course 200mm thick lain upon a strong base 250mm thick, below this the sub-bases are weak down to a measured depth of 1906mm and beyond. CBR values redefined from average layer strength values correspond well with actual values recorded although the latter show a gradual decrease in strength with depth as opposed to the definite layer boundaries assumed for the layer locations.

The balance of the pavement is thus described as a well balanced shallow structure, the majority of the structural strength being seen in the upper two layers of the structure.

Site 2

Four structural layers were discerned at this site, the wearing course extending down to 100mm consists of very strong material lain upon a strong base extending down to 470mm. Below this the sub-bases are weak, with the lower sub-base particularly weak down to a depth of 1911mm and beyond. Redefined CBR values correspond closely to those measured



Site 1

LAYER	CBR
1	211
2	133
3	37
4	17

Site 2

LAYER	CBR
1	354
2	113
3	21
4	8

Site 3

LAYER	CBR
1	269
2	99
3	26

Figure 4.6 Kromdraai Colliery Pavement Profiles as Determined by DCP Analysis.

for each layer except for layer 3 where a gradual decrease in strength with depth is apparent. No significant strong or weak spots were seen in the structure.

The balance of the pavement is thus described as a well balanced shallow structure, in terms of balance one of the best sites investigated as shown in Figure 4.7. (Refer to Appendix B for the complete results).

Site 3

Three structural layers were discerned at this site consisting of a strong wearing course layer 150mm deep lain upon a medium strong base of 500mm thickness and a sub-base of weak material down to a measured depth of 1881mm and beyond. There is a gradual decrease in strength with depth over layers 1 and 2 whilst layer 3 exhibits isolated strong and weak spots about an average CBR value of 26.

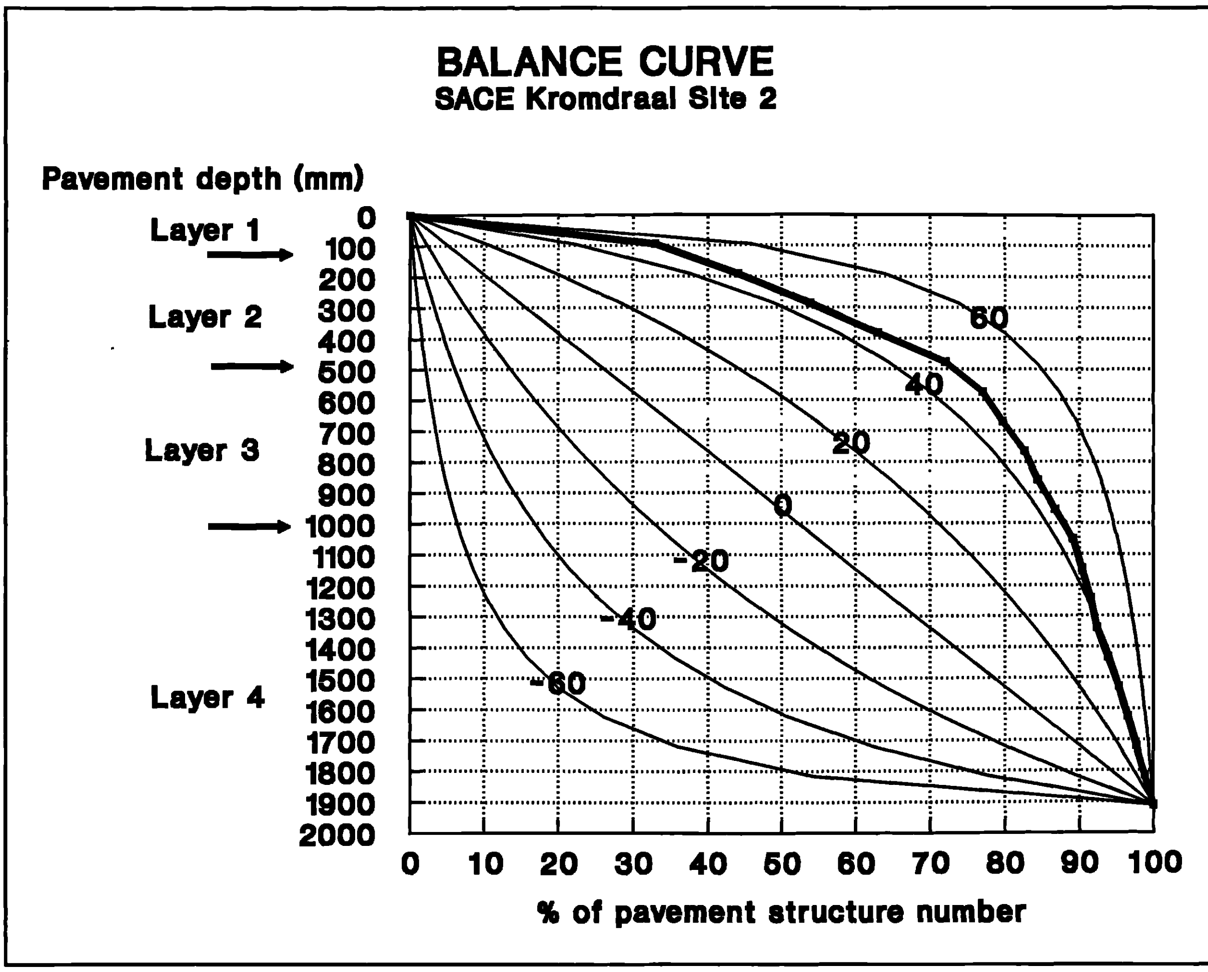


Figure 4.7 Particularly Well Balanced Shallow Structure at Kromdraai Mine Site 2.

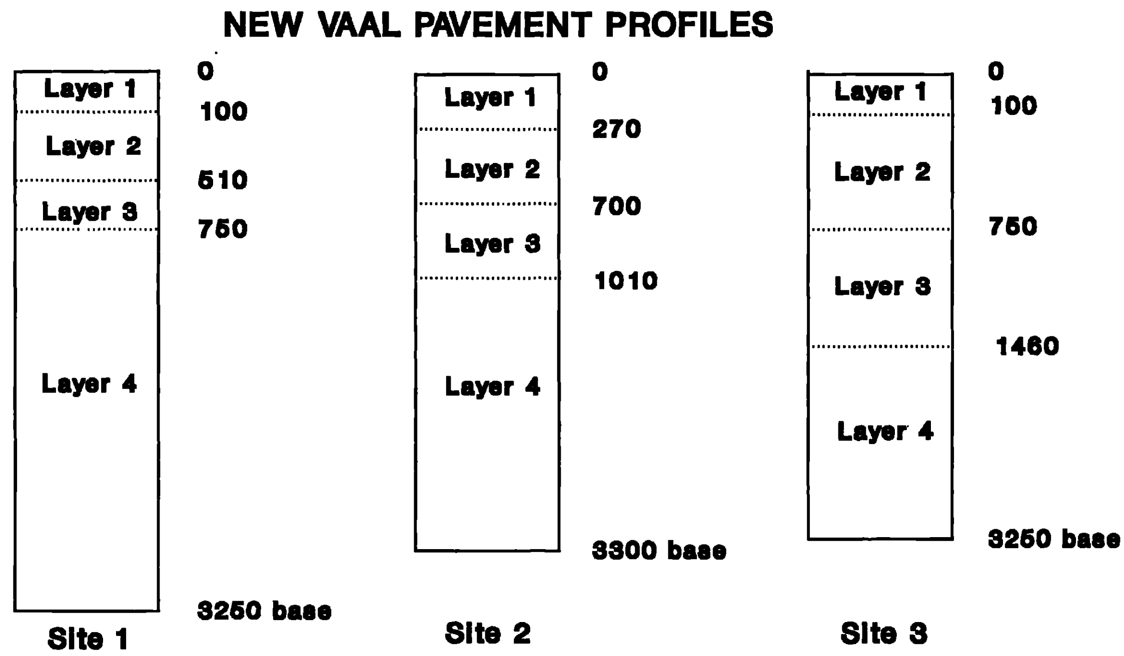
The balance of the pavement is described as an averagely balanced shallow structure, primarily arising from the rapid degradation of strength with depth in layer 3.

4.2.3 Discussion of DCP Analysis - New Vaal Colliery

The pavement profiles and corresponding CBR values of each layer as determined from the DCP analysis are presented for each site in Figure 4.8 and discussed in the proceeding sub-sections.

Site 1

Four structural layers are discerned at site 1 consisting of a strong wearing course 100mm



LAYER	CBR
1	237
2	370
3	103
4	34

LAYER	CBR
1	302
2	166
3	270
4	80

LAYER	CBR
1	154
2	408
3	30
4	73

Figure 4.8 New Vaal Colliery Pavement Profiles.

in thickness followed by a very strong base 410mm in thickness. These are lain upon a moderately strong and weak sub-base layers 240mm and in excess of 509mm (respectively) in thickness. CBR values calculated from actual values show some deviation about the mean value in layers 1 and 2 but remain approximately constant with depth. Layer 3 exhibits a gradual decrease of strength with depth over the layer whilst layer 4 is again relatively constant in strength. The refusal of the penetrometer at 1259mm depth gives rise to spuriously high values in the last 150mm depth of probing which are ignored for calculation purposes.

The balance of the pavement is thus described as a poorly balanced deep structure, primarily due to the strong base layer 2 lain upon the much weaker sub-base layers 3 and 4. A degree of imbalance in the lower levels of the pavement may be ascribed to the penetrometer refusal and apparent increase in strength in this region.

Site 2

Four structural layers are discerned at this site consisting of a very strong wearing course 270mm thick placed upon a strong base 430mm thick. The sub-base layers consist of a very strong upper layer 310mm thick and a moderately weak lower layer to a depth of 1620mm and beyond. The CBR values show considerable scatter for all layers but the general trend of layer strengths is evident from the redefined CBR layer values.

The balance of the pavement is thus described as an averagely balanced deep structure primarily due to the influence of layer 3 at 700mm - 1010mm depth providing structural strength at depth.

Site 3

Four layers are discerned at this site, the wearing course being moderately strong and 100mm in thickness. The base layer 2, extending from 100mm to 750mm is particularly strong and is lain upon a very weak upper sub-base 710mm in thickness. The lower sub-base extends to 1784mm and beyond and is moderately weak. CBR values calculated for each layer correlate well with actual values in layers 1 and 2 whilst layer 3 exhibit isolated weak spots and an overall trend of increasing strength with depth. Layer 4 shows isolated weak and strong spots but no specific trend of strength with depth.

The balance of the pavement is thus described as a poorly balanced deep structure, primarily as a result of the particularly strong base layer 2.

4.3 DCP Analysis Summary

Table 4.2 presents a summary of the DCP results and should be read in conjunction with Figures 4.5, 4.7 and 4.8. The results presented confirm the classification of test sites proposed in Table 3.6 for the site location matrix, envisaged in the experimental design.

In general, those sites showing a shallow structure, in which the majority of the pavement strength lies in the upper layers may be more sensitive to increased wheel loads and consequential failure of the upper layers. A deep structure, in contrast, would be less sensitive to any increase in wheel loads, but may well show signs of excessive deformation in the weaker upper layers. The extent to which these effects are seen in haul roads can only be determined from in-situ deflection measurements.

Table 4.2 Summary of DCP Results - Pavement Balance

MINE	TEST SITE	PAVEMENT DESCRIPTIVE SUMMARY
Kriel Colliery	1	Averagely balanced shallow structure
	2	Averagely balanced inverted structure
	3	Poorly balanced shallow structure
Kromdraai Colliery	1	Well balanced shallow structure
	2	Well balanced shallow structure
	3	Averagely balanced shallow structure
New Vaal Colliery	1	Poorly balanced deep structure
	2	Averagely balanced deep structure
	3	Poorly balanced deep structure

In addition, the pavement strength-balance concept focuses on the upper 1,8m of material, which, for most mine sites generally includes a portion of sub-grade. The strength-balance concept does not address whether the pavement as a whole is suited to the sub-grade strength. Thus, although the DCP data affords an insight into the actual road structure as opposed to the design structure and the strength of each layer actually achieved in the field, the extent to which each type of design fulfils the structural performance requirements can only be determined from analysis of the response of each layer to the applied loads. As a precursor to the analysis, the California Bearing Ratio design technique is investigated in which CBR data generated from the DCP investigation is compared to actual cover requirements predicted from the CBR design method.

4.4 California Bearing Ratio (CBR) Design Procedure

A survey conducted in 1928-1929 by the California Roads Department to determine the extent and cause of road pavement failures concluded that failure was caused by either inadequate compaction of materials forming the road layers and/or insufficient cover over weak in-situ material. These conclusions indicated the importance of material compaction and shear strength considerations in road building, both in terms of a suitable design procedure and an associated materials testing method (Yoder and Witczak, 1975). The notion of the California Bearing Ratio (CBR) value for a specific material was thus developed from a laboratory penetration test of a soaked sample of pavement material as an inference of its shear strength. The CBR value for a material is thus the relationship between the force necessary to drive a piston into the sample and the force to likewise drive the piston into a standard gravel sample upto a given depth, usually 2,54mm, results being reported as a percentage of the standard (gravel) test.

The first indictions of cover requirements over in-situ materials of specific CBR (%) values was reported by the California Division of Highways during the years 1928-1929 (American Society for Civil Engineers, 1950). Later modifications included consideration of (air) traffic volumes, single wheel loads and increased wheel loads based on an estimated maximum allowable shear stress for specific materials. The problem of dual wheel assemblies was addressed by Boyd and Foster (1949) through consideration of the Equivalent Single Wheel Load (ESWL), where a load is calculated which generates the same tyre contact area and maximum deflection as would the group of wheels. The concept of equivalent deflection is used to equate an equivalent single wheel to the multiple wheel group.

Traffic volume and its effect on the structural design of pavements was considered by Ahlvin et al (1971) in which a repetition factor was determined according to load repetitions and the total number of wheels used to determine the ESWL. In this manner, the resulting thickness of cover could be modified to accommodate air traffic volumes.

Despite the empirical origins of the technique, Turnbull and Ahlvin (1957) derived a mathematical approach to the calculation of cover requirements using the CBR method. This approach is adopted for the calculation of cover requirements over in-situ material, as predicted by the CBR design method for ultra-heavy axles.

4.4.1 Mathematical Correlation

The contact pressure and distribution between any tyre and the pavement depends on tyre pressure, wheel load and tyre construction. The contact area is generally approximated as circular, although an analysis of heavy vehicle tyre loads has shown this only to be true when the ratio between applied load and maximum rated load is small (Marshek, 1978, Tielking and Roberts,1987). Mine haul trucks fully loaded exhibit ratios of between 0,7 to 0,9 of manufacturers recommended payload (Goodyear, 1990) and the contact area approximates more to a rectangle. Additionally, contact pressure is assumed to equate to tyre inflation pressure which is an over simplification; upto 10% variation may occur between tyre and contact pressure under ideal inflation conditions, greater variations with increased deviation from recommended inflation pressures. For the purposes of the CBR analysis, Goswami and Bhasin (1986) illustrated that more refined modelling of these two parameters is not warranted and the only deviations noted (from cover requirements calculated with and without these simplifying assumptions) were with particularly weak materials (CBR<2%) covered by a thin wearing course only. Bearing in mind the empirical nature of the technique and the relatively thicker wearing course layers encountered in mine haul roads, the adoption of the contact area and pressure assumptions are valid.

Contact area (*A*) of a tyre is given by;

$$A = \frac{LOAD}{TYRE\ PRESSURE} \qquad (m^2) \qquad [4.6]$$

from which follows the expression for contact radius.

The contact radius (r) is then;

$$r = \sqrt{\frac{A}{\pi}} \quad \text{(m)} \quad [4.7]$$

The relationship developed by Turnbull and Ahlvin (1957) to describe the cover required (t) over a material of strength (CBR) subjected to a wheel load (P) is;

$$t = \sqrt{\frac{P}{55.8 \times CBR} - \frac{A}{\pi}} \quad \text{(m)} \quad [4.8]$$

Equation [4.8] should only be applied where CBR values less that 12 are encountered, more recent research by Ahlvin et al (1971) in conjunction with work on multiple wheel groups, proposes the following relationship for cover thickness;

$$t = \sqrt{A}\left(-0.0481 - 1.1562\left(\log\frac{CBR}{p_e}\right) - 0.6414\left(\log\frac{CBR}{p_e}\right)^2 - 0.4730\left(\log\frac{CBR}{p_e}\right)^3\right) \quad [4.9]$$

where p_e is defined as the equivalent tyre pressure at depth t given by;

$$p_e = \frac{ESWL}{A} \quad [4.10]$$

When considering traffic volume, equations [4.8] and [4.9] can be rewritten to incorporate the repetition factor developed by Ahlvin et al (1971), a graphical relationship between repetition factor, number of repetitions and wheels in multiple configuration as given in Figure 4.9. This relationship is derived from consideration of aircraft wheel loads on asphalt surfaced pavements and the validity of its adoption for the design of gravel-surfaced mine haul roads for large haul trucks has not been ascertained. If a repetition factor is included, equation [4.11] represents the required CBR at a given depth (t) for the specific ESWL used whilst the revised equation [4.12] is solved iteratively.

$$CBR = \frac{ESWL}{55.8\left[\left(\frac{t}{\alpha}\right)^2 + \frac{A}{\pi}\right]} \qquad (\%) \qquad [4.11]$$

$$t = \alpha\sqrt{A}\left(-0.0481 - 1.1562\left(\log\frac{CBR}{p_e}\right) - 0.6414\left(\log\frac{CBR}{p_e}\right)^2 - 0.4730\left(\log\frac{CBR}{p_e}\right)^3\right) \qquad [4.12]$$

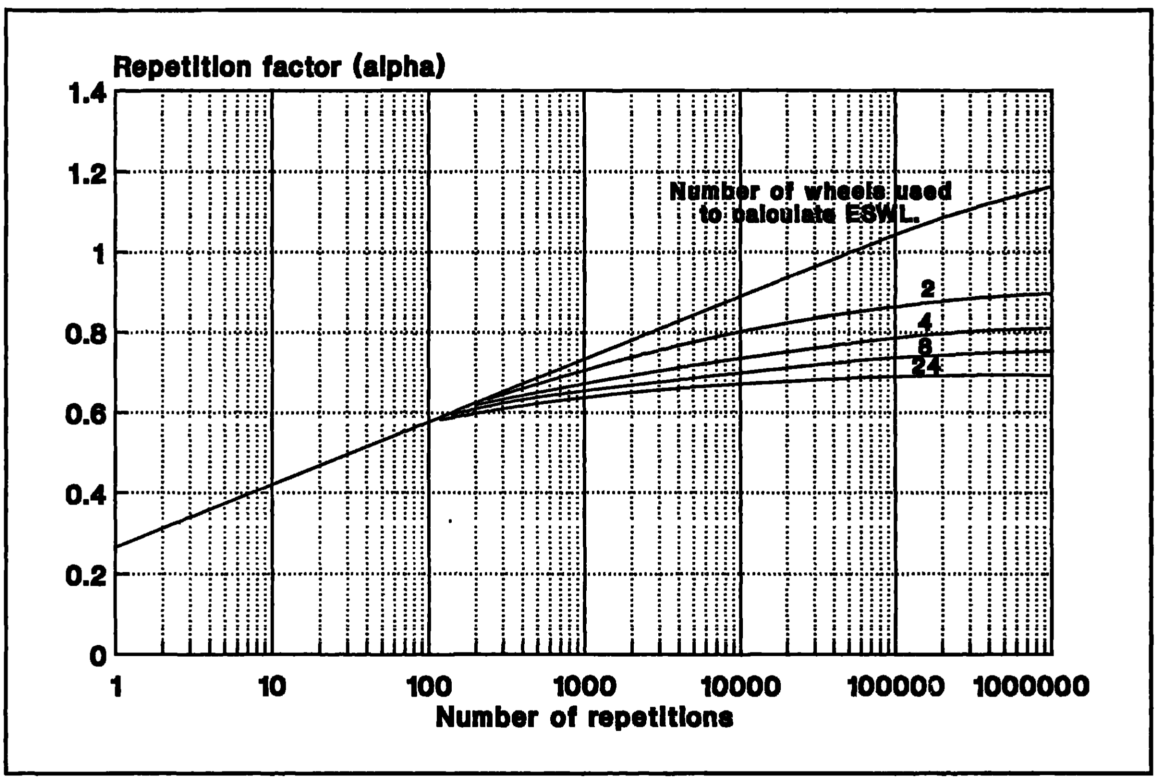


Figure 4.9 Load Repetition Factor (after Ahlvin et al, 1971).

The calculation of the ESWL for multiple wheels incorporates two conditions;

- the ESWL will have the same contact area (A) as the other wheels in the group
- and the maximum deflection generated by the ESWL will be equivalent to the maximum deflection generated by the group of wheels.

Following Foster and Ahlvin (1954) the deflection under a single wheel (W_s) is given by;

$$W_s = \frac{r_e}{E} b_s F_s \qquad \text{(m)} \quad [4.13]$$

Where	r_s	=	contact radius for single wheel (m)
	E	=	elastic modulus of pavement (MPa)
	b_s	=	tyre pressure for single wheel (MPa)
	F_s	=	deflection factor for single wheel

and for a group of wheels (W_d) similarly by;

$$W_d = \frac{r_d}{E} b_d F_d \qquad \text{(m)} \quad [4.14]$$

Following the conditions described above, equations [4.13] and [4.14] can be rewritten as;

$$W_s = W_d \quad and \quad r_s = r_d \qquad [4.15]$$

Equations [4.6] and [4.7] relate load and tyre pressure to contact area and combining with equation [4.13] gives;

$$P_s = \pi r_s^2 b_s \quad and \quad P_d = \pi r_d^2 b_d \qquad [4.16]$$

Equation [4.17] below represents this in terms of wheel loads and deflection factors for single and groups of wheels;

$$\frac{P_s}{P_d} = \frac{F_d}{F_s} \qquad [4.17]$$

Equivalent deflection values at specific depths for various horizontal locations are found graphically as shown in Figure 4.10 and these are used to determine the ESWL at various pavement depths.

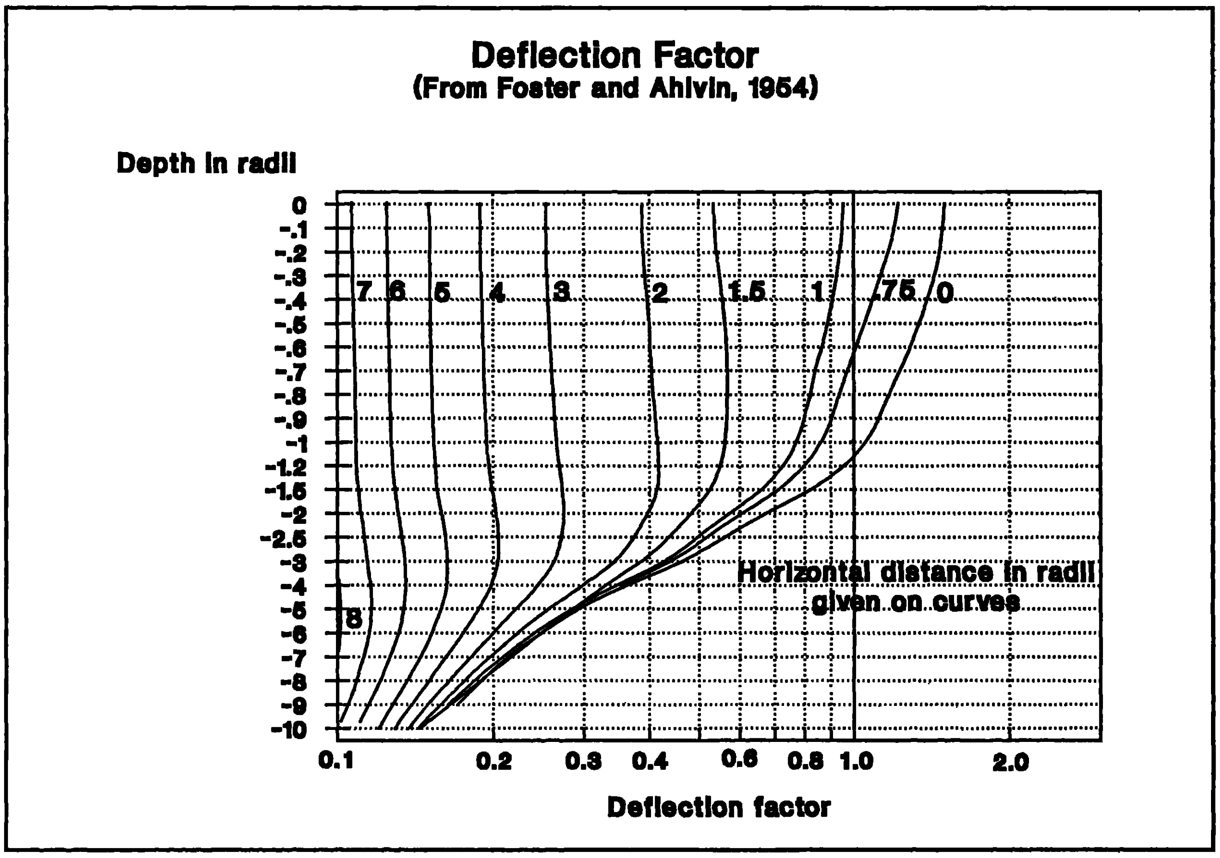


Figure 4.10 Deflection Factors for ESWL Determination (after Foster and Ahlvin, 1954).

The ESWL is calculated at a range of pavement depths from which the required CBR cover curve is constructed. The specific wheel grouping of a haul truck is reduced to four wheels by means of an equivalent single wheel load representing dual assemblies or axles and the deflections under four characteristic points recorded. These characteristic points are derived from consideration of the stresses generated in a uniform homogeneous pavement under the action of two sets of two wheels, specifically the increase in stress (and thus deflection) where stress fields overlap. With an equivalent single wheel load representing the dual assembly, the critical points (following Yoder and Witczak, 1975) occur either under the centre of one rear load (D) or at the centre of the rear axle (C). When the front axle interaction is considered, two additional critical points (A and B) are analysed in a position calculated in proportion to the fully laden axle weight distribution. This is represented schematically in Figure 4.11(a) whilst Figure 4.11(b) illustrates the corresponding layout of the wheel group of a Haulpak R170 truck with all dimensions normalised in terms of the tyre contact radius. The influence of each wheel in terms of deflection factor upon the characteristic point (A-D) chosen is summed and the maximum ESWL at that depth found

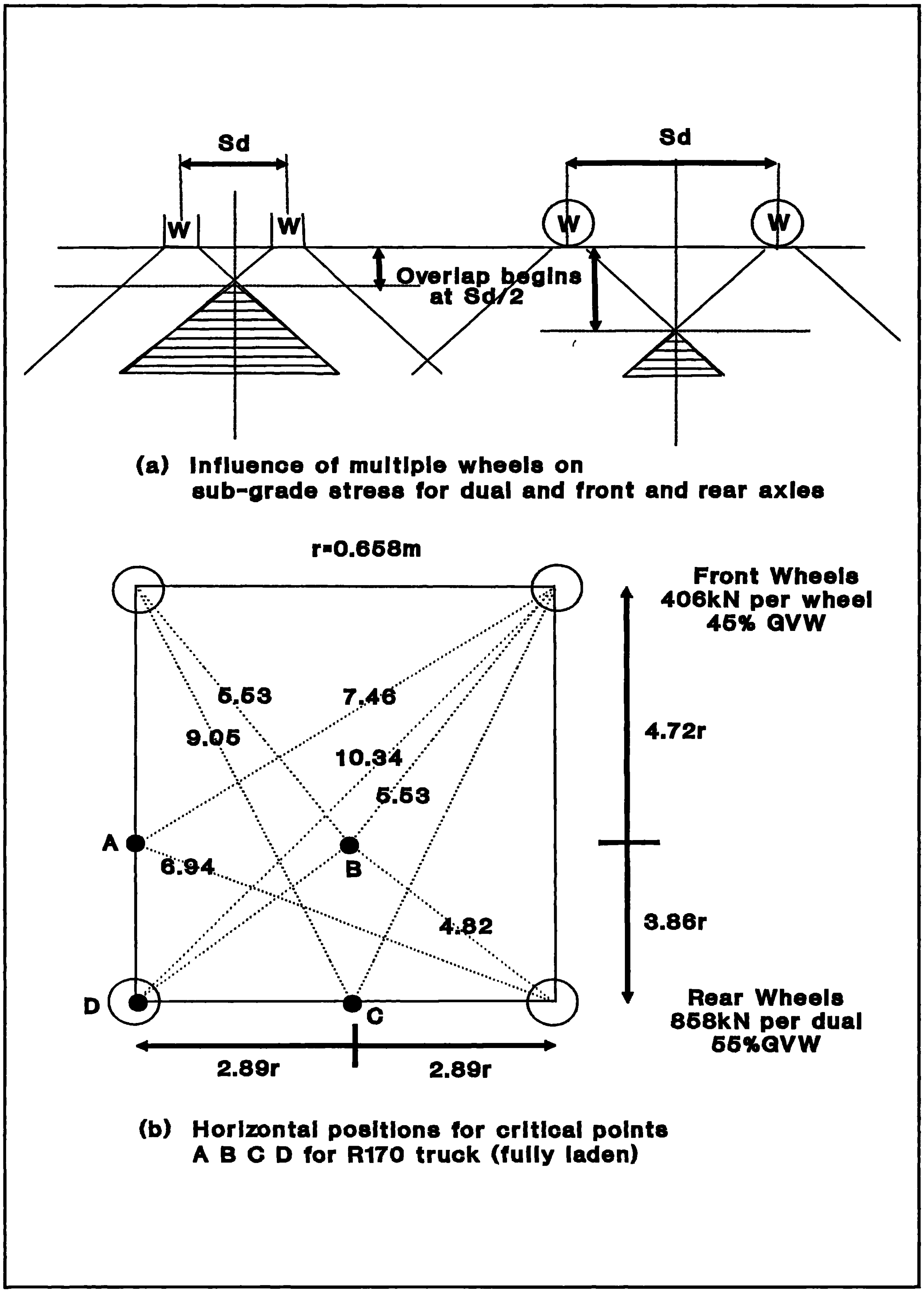


Figure 4.11 (a & b) Vertical Sub-grade Stress generated under a Group of 4 Wheels and (b) the Corresponding Critical Point Locations in Terms of Contact Radius (r).

from Equation [4.17]. The required cover for the maximum ESWL calculated at depth increment *r* is then calculated from equation [4.12]. As a basis for comparative analysis of the utility of the CBR method for haul road design, the CBR cover curve is calculated for the largest vehicle used on a particular mine site and compared with the actual design as determined by DCP analysis. Results for each mine site are summarised in the following sub-sections and given in full in Appendix C.

4.4.2 CBR Cover Curve Design - Kriel Colliery

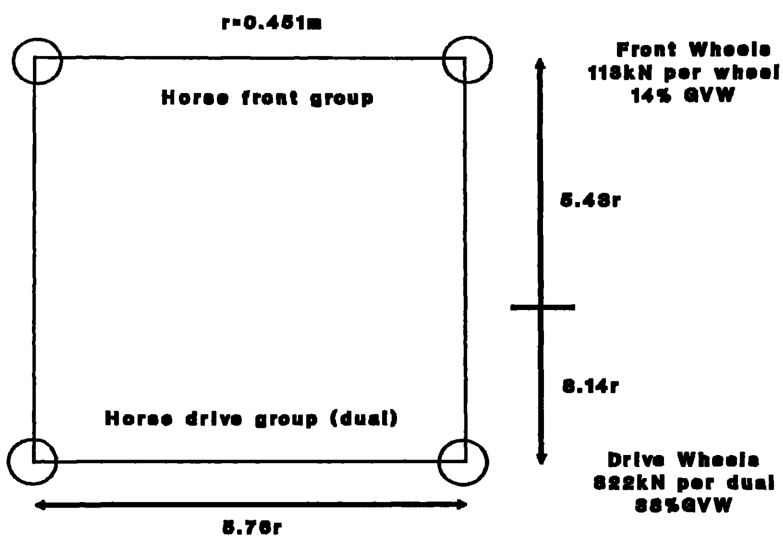
Mine roads were historically designed for the 3-axle Cat 772 bottom dump truck and only comparatively recently have the larger 2-axle Euclid R170 rear dump trucks been introduced. It is thus instructive to generate separate cover curves for each vehicle to qualify any pavement under-design apparent with the use of R170 trucks.

<u>Cat 772 Cover Curve</u>
Two distinct drive groups for fully laden conditions must be considered;

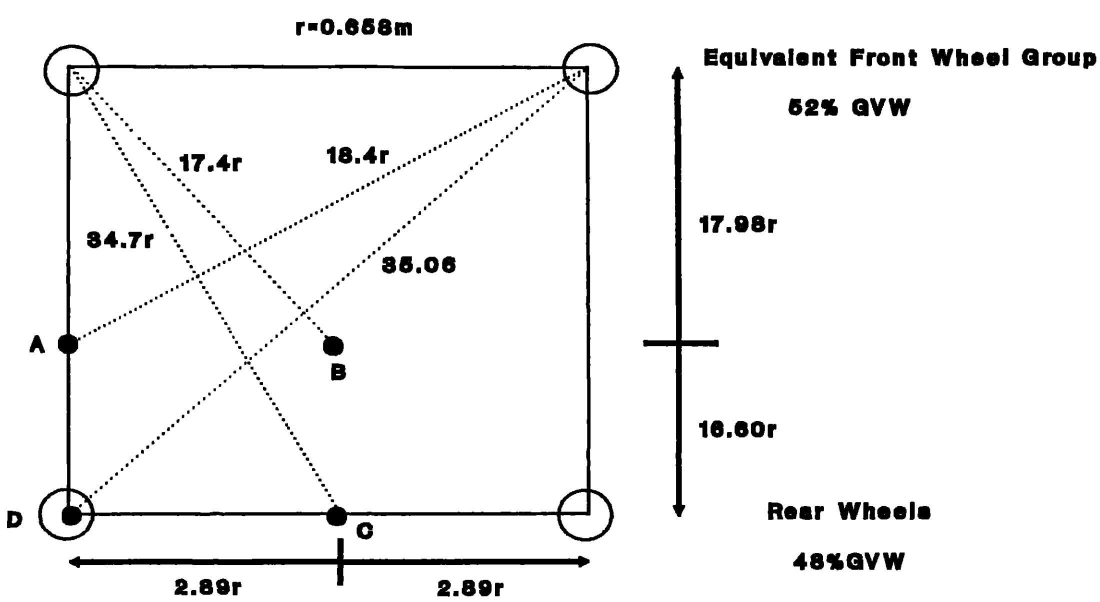
(i) front wheel group (horse front and drive axles)
(ii) drive wheel group (horse drive and trailer rear axles)

Results of these two analyses are given in Appendix C. The possibility also exists of the horse group of wheels influencing the rear group of wheels in terms of deflection generated by the vehicle. Reducing the front group to an approximate equivalent and accepting the maximum contact radius to be associated with the rear group, the horizontal radii for the critical locations A-D normalised in terms of the contact radius are found to be large. With reference to Figure 4.10, it may be assumed that no influence is seen on deflections generated by the vehicle from combined front and rear groups. Figure 4.12 relates the horizontal radii for the combined front and rear groups.

<u>Euclid R170 Cover Curve</u>
The cover curve for the fully laden R170 truck is constructed for the front and rear wheel groups as outlined in the foregoing section, using the horizontal radius (r) associated with the



(a) The front wheel group reduced to an equivalent front wheel group



(b) Horizontal positions for critical points A B C D for Cat 772 truck fully laden with horse reduced to equivalent group

Figure 4.12 Horizontal Radii for Combined Front and Rear Wheel Groups, Cat 772 Truck.

rear group of wheels, as shown in Figure 4.11(b).

The DCP generated redefined layer strengths for each test site are given in Figure 4.13 together with the CBR cover curve for both vehicles considered. In all cases the design structural strengths exceed the minimum cover requirements predicted by the CBR calculation method. Site 1 layer 3 approaches the minimum cover requirements and the possibility exists that excessive vertical strains may develop in the layer due to overstressing. The extent to which this may lead to deformation in the sub-grade is not easily determined from the CBR data alone. The situation is ameliorated when the cover curve for the Cat 772 truck is considered.

4.4.3 CBR Cover Curve Design - Kromdraai Colliery

Mine roads were designed for the 2-axle Haulpak 630E rear dump truck and the required cover according to the CBR design method is calculated for fully laden conditions. The DCP generated redefined layer strengths for each test site are given in Figure 4.14 together with the CBR cover curve for the vehicle under consideration. The design structural strengths exceed the minimum cover requirements predicted by the CBR calculation method for sites 1 and 3. Site 2 layers 2 and 3 exhibit strengths below the CBR predicted minimum cover requirements and the possibility exists that excessive vertical strains may develop in these layers due to overstressing. Since road construction was not complete at the time of testing (September 1993), the placement of the final wearing course layer to design depth will have the effect of moving the redefined DCP layer strength profile down, thus effectively ensuring all layer strengths eventually exceed those predicted by the CBR design method.

4.4.4 CBR Cover Curve Design - New Vaal Colliery

Mine roads were designed for the 2-axle Komatsu HD 1600 M1 and Euclid R170 rear dump trucks, the former truck being used under fully laden conditions to derive the required cover according to the CBR design method. The DCP generated redefined layer strengths for each

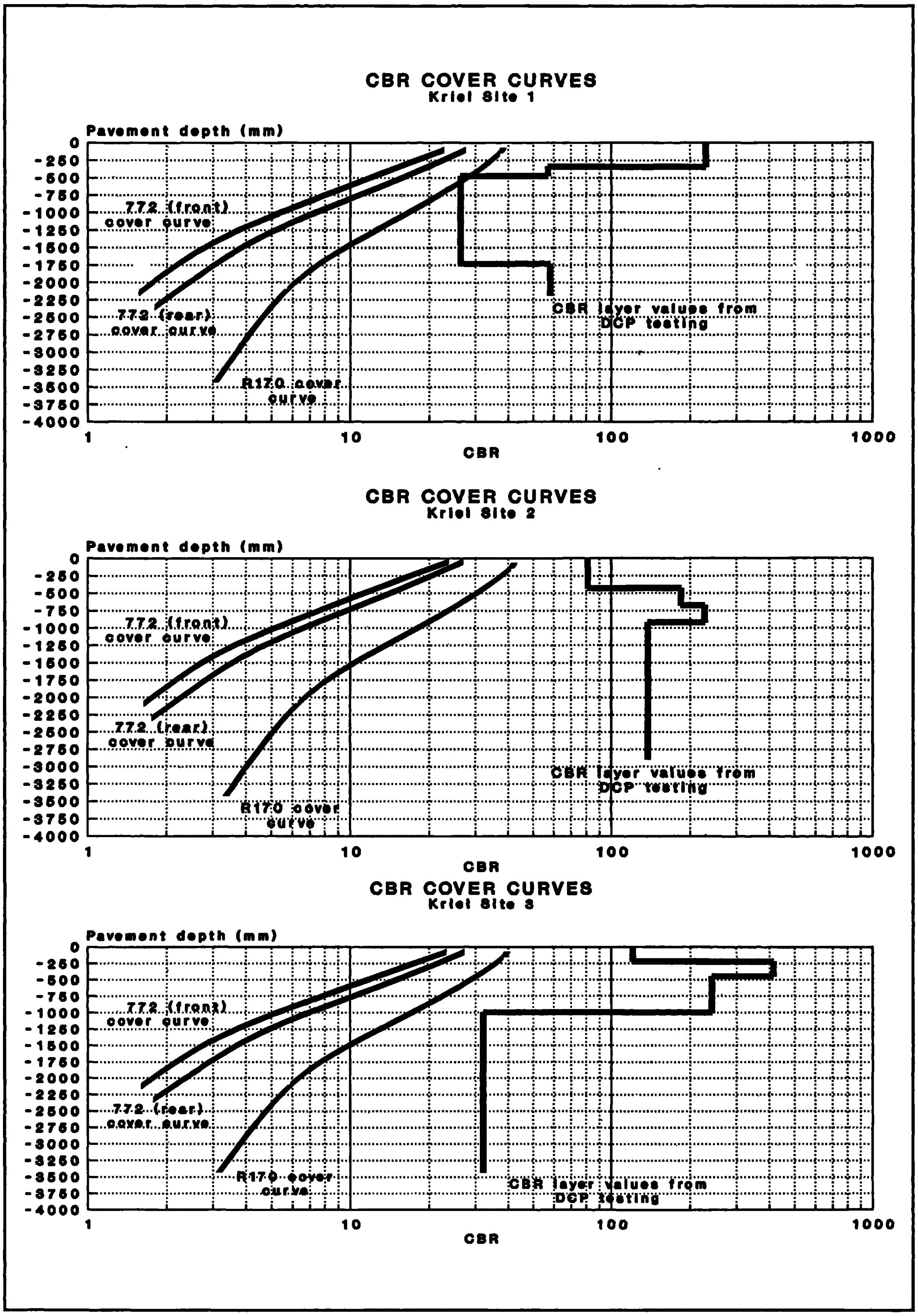


Figure 4.13 CBR Cover Curves for Kriel Colliery Sites 1, 2 and 3.

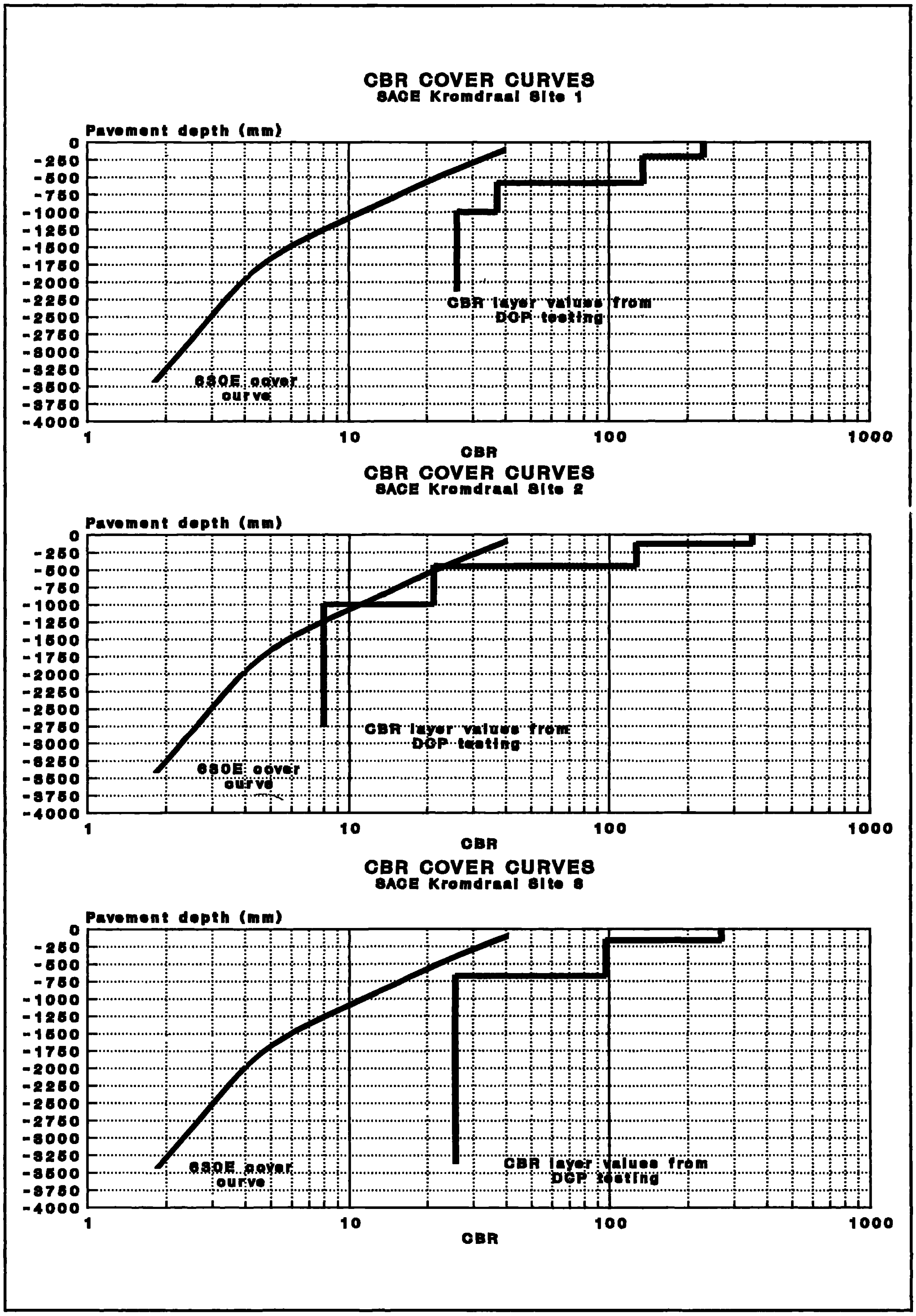


Figure 4.14 CBR Cover Curves for SACE Kromdraai Colliery Sites 1, 2 and 3

test site are given in Figure 4.15 together with the CBR cover curve for the vehicle considered. The design structural strengths are not exceeded by the CBR predicted minimum cover requirements at any site. Site 1 layer 4 and site 3 layer 3 exhibit strengths close to the CBR predicted minimum cover requirements and the possibility exists that excessive vertical strains may develop in these layers due to overstressing. However, the extent to which this may lead to deformation of the sub-grade is not easily determined from the CBR data alone.

4.5 Summary of Results for CBR Cover Curve Design

The CBR method has been widely applied to the design of surface mine haul roads in which untreated materials are used. In essence, it relates sub-base thickness requirements to sub-grade bearing capacity, thereby eliminating overstressing and consequent deformation of the sub-grade due to axle loading.

Although the CBR method is a simple and straight forward design method based on and improved by considerable practical experience, numerous disadvantages exist when applying the method to mine haul road design problems: The method has its base in Boussinesq's single semi-infinite layer theory which assumes a constant elastic modulus for the material (sub-base). Mine haul road structures consist of numerous layers of differing material each with its own specific elastic and other properties. More specifically, the CBR method was based on empirical results relating to the design of asphalt-surfaced airfield pavements for wheel-gear loads upto 4 400kN for a C5A aircraft. When aggregate-surfaced mine haul roads are considered in conjunction with stabilised bases, albeit at similar load levels, the same approach is of questionable validity. The graphical relationship proposed by Ahlvin et al (1971) in conjunction with the modified CBR design technique to accomodate the effect of traffic repetitions may also therefore not be applicable to haul road structural design. Simple extrapolation of these empirical design criteria to accommodate higher axle loads upon very different pavement construction materials can lead to serious errors of under- or over-design.

The deficiencies inherent in the development of the CBR design method militate against using

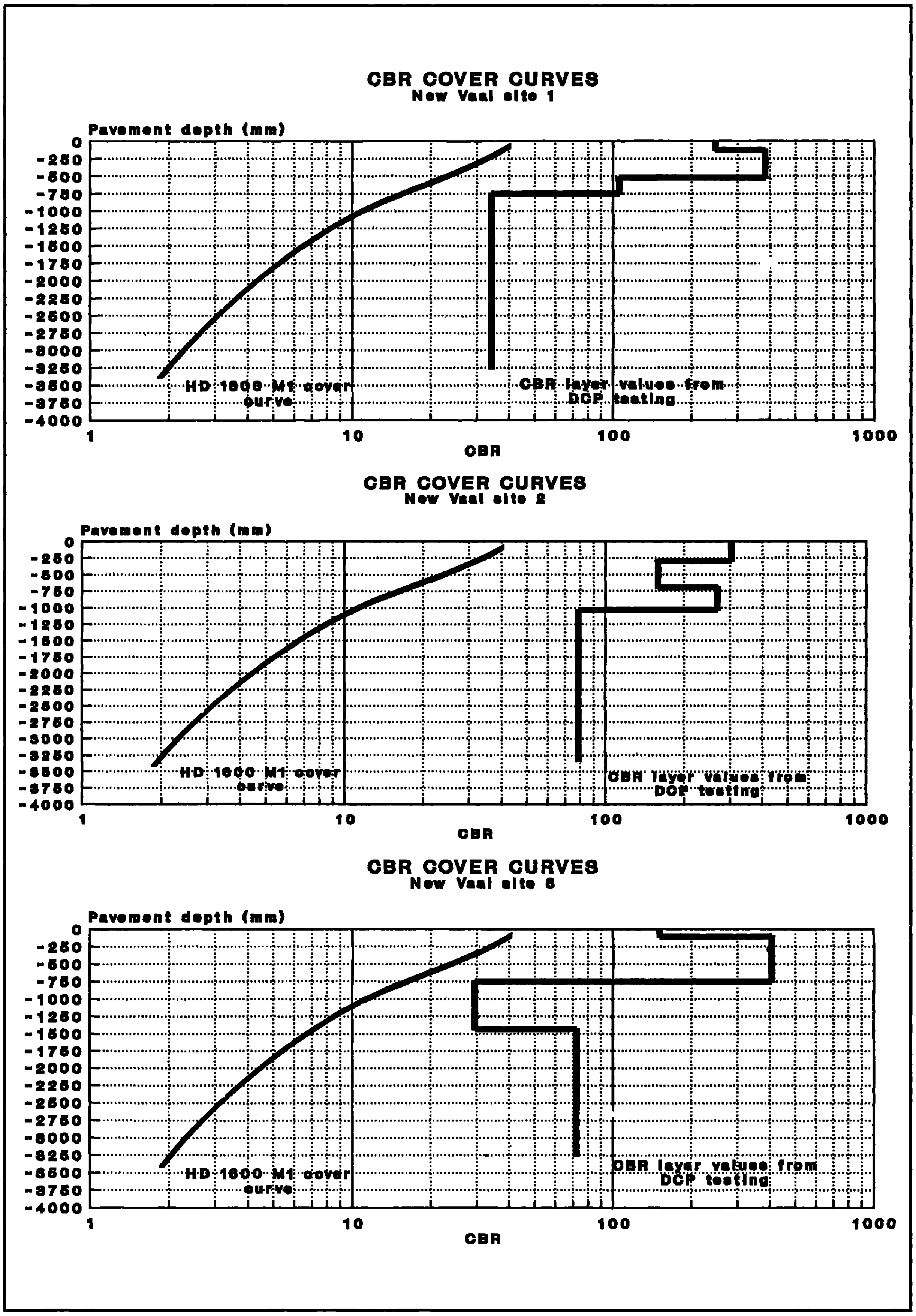


Figure 4.15 CBR Cover Curves for New Vaal Colliery Sites 1, 2 and 3.

the techniques for the structural design of mine haul roads. When the results of the DCP redefined layer strengths are analysed in conjunction with the CBR cover curves generated, it would appear that the method, when applied judiciously, can be used to determine safe (total) cover over in-situ materials, although the extent of over or under design associated with the method cannot be qualified. The method is thus exclusively recommended to design cases where no surface layers exist above standard gravel bases. Where cemented or stabilised layers are included in the design, or where the optimal structural design is sought, due to the very different properties of the layer in comparison to normal roadbuilding gravels, other design techniques should be employed which can account for the different material properties and more accurately predict their performance.

CHAPTER 5
MECHANISTIC ANALYSIS AND QUANTIFICATION OF EXISTING PAVEMENT STRUCTURAL DESIGNS

5.1 Introduction

This chapter addresses the mechanistic analysis and quantification of existing haul road structural designs. The fundamentals of the mechanistic design procedure are introduced and the benefits the method accrues over purely empirical approaches are discussed. Pavement deflection profiles generated from Multi-depth Deflectometer installations in mine pavements are then analysed with the aid of multi-layer pavement elastic models and the ELSYM5A (1985) computer program. The mechanistic-empirical design process is then introduced, by means of which the results of the multi-layer elastic analyses are used to deduce acceptable design criteria for haul road structural design.

5.2 Fundamentals of Mechanistic Design

The mechanistic approach to pavement engineering involves the application of physics to determine the reaction of the pavement structure to loading. Of prime importance is the extent to which the structure distributes vehicle loads to the underlying in-situ material. Weak pavement structures concentrate the load over a smaller area of the sub-grade than strong pavements as shown schematically in Figure 5.1, resulting in higher stresses in the sub-grade. In order to quantify how the load is being distributed, certain fundamental properties of the materials comprising the structure must be determined along with layer thickness and load characteristics. The mechanistic component relates to the determination of stresses, strains and deflections within the pavement layers through the use of layered elastic analysis.

Empirical design criteria are to some extent a requirement of all structural design techniques. Whilst the CBR based approach is entirely empirical and therefore subject to data characteristic limitations, the mechanistic approach, although including an empirical

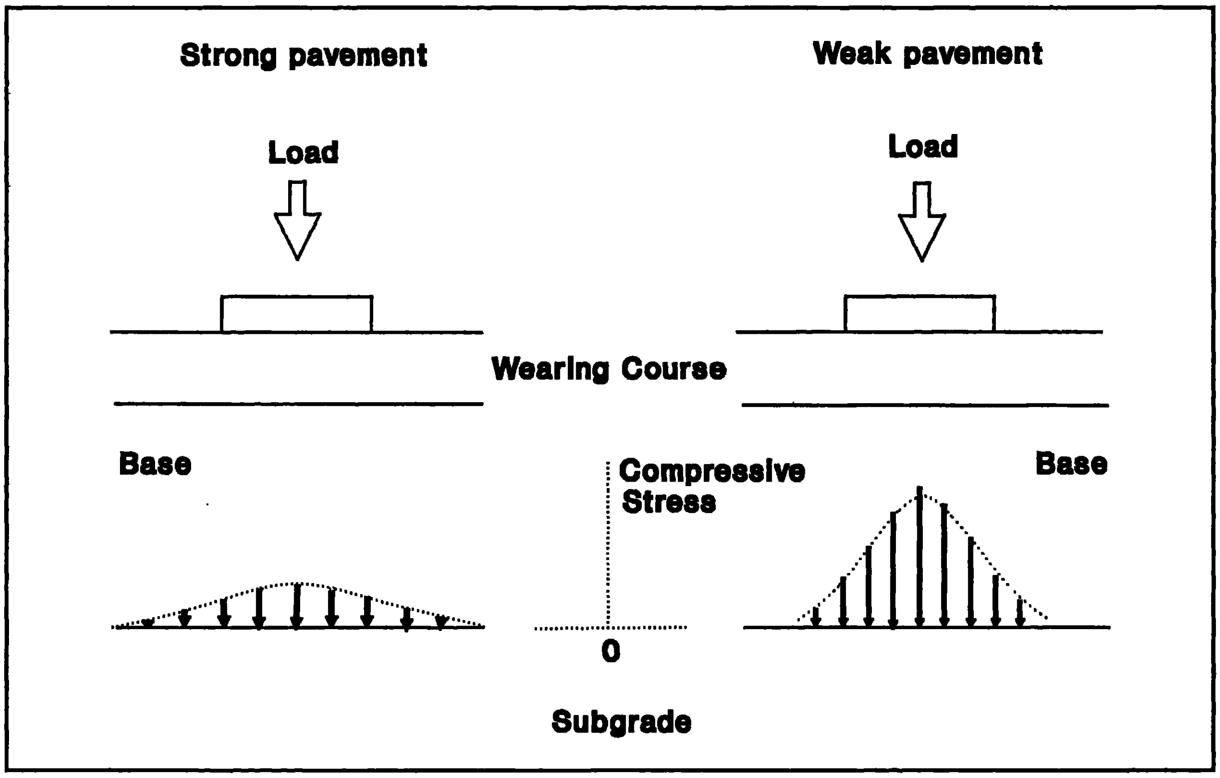


Figure 5.1 Load Distribution Characteristics of a Strong versus Weak Pavement

component, relies largely on mechanistically derived data to which empirical procedures are applied, therefore extending the functionality of the technique. Typical benefits of the mechanistic-empirical approach when applied to haul road design are;

(i) The ability to accommodate changing loads and analyse their impact on pavement structural performance in terms of the strains developed in each layer

(ii) The ability to utilise available construction materials in a more efficient manner by evaluating excessive vertical strains that be be induced in poorer quality materials

(iii) The ability to analyse the effect of alternative construction materials on the pavement structure and modify the design to accommodate these materials within the design limiting criteria

(iv) More reliable performance predictions for multi-layer structures incorporating various material qualities

(v) Use of material properties in the design process which are more closely related to field performance of the structure, particularly the elastic reponse of the pavement

(vi) An improved definition of existing pavement layer properties.

The basic theory from which the procedure is developed is attributable to Boussinesq. Equations were presented from which stress, deformation and deflections could be calculated in a homogeneous, isotropic linear elastic half space, subject to point loading. In the early 1900's Love developed extensions to the basic equations to account for circular loads whilst Westergaard later modified the equations for no deflection in the uppermost layer. Latterly Timoshenko developed the general theory for a linear elastic system upon which Burmister advanced a solution for 2 and 3 layer systems using numerical integration. The equal deflection criteria for the determination of ESWL described in section 4.3.1 and developed by Foster and Ahlvin is derived from Boussinesq's solution, but with the application of circular loads. Numerical integration techniques for the direct solution of multi-layer multi-load problems are now widely available with the advent of microcomputers.

A simple and convenient method to assess the structural integrity of pavements is to apply a load and measure the resulting depth deflection profile. The Multi-depth Deflectometer as described in section 3.2.1.3 can be used for this purpose, in which an array of 6 Linear voltage differential transducers (LVDTs) are used to determine the pavement layer deflections resulting from an applied load. A typical depth-deflection profile is given in Figure 5.2 from which it is seen that the larger deflections occur towards the top of the structure. These deflections are used together with a multi-layer analysis program to determine the layer effective elastic moduli, stresses and strains by means of which the response of the structure may be characterised. These stresses and strains are used in conjunction with empirical limiting fatigue or distress values and relationships to evaluate structural performance of the pavement and, if necessary, to evaluate the efficacy of corrective measures.

5.2.1 Layered Elastic Systems

Much of the structural deterioration of a pavement is attributable to the stresses or strains developed in individual pavement layers. Vertical strains in the top of sub-base and sub-grade layers are associated with rutting and deformation whilst strains in upper stabilised

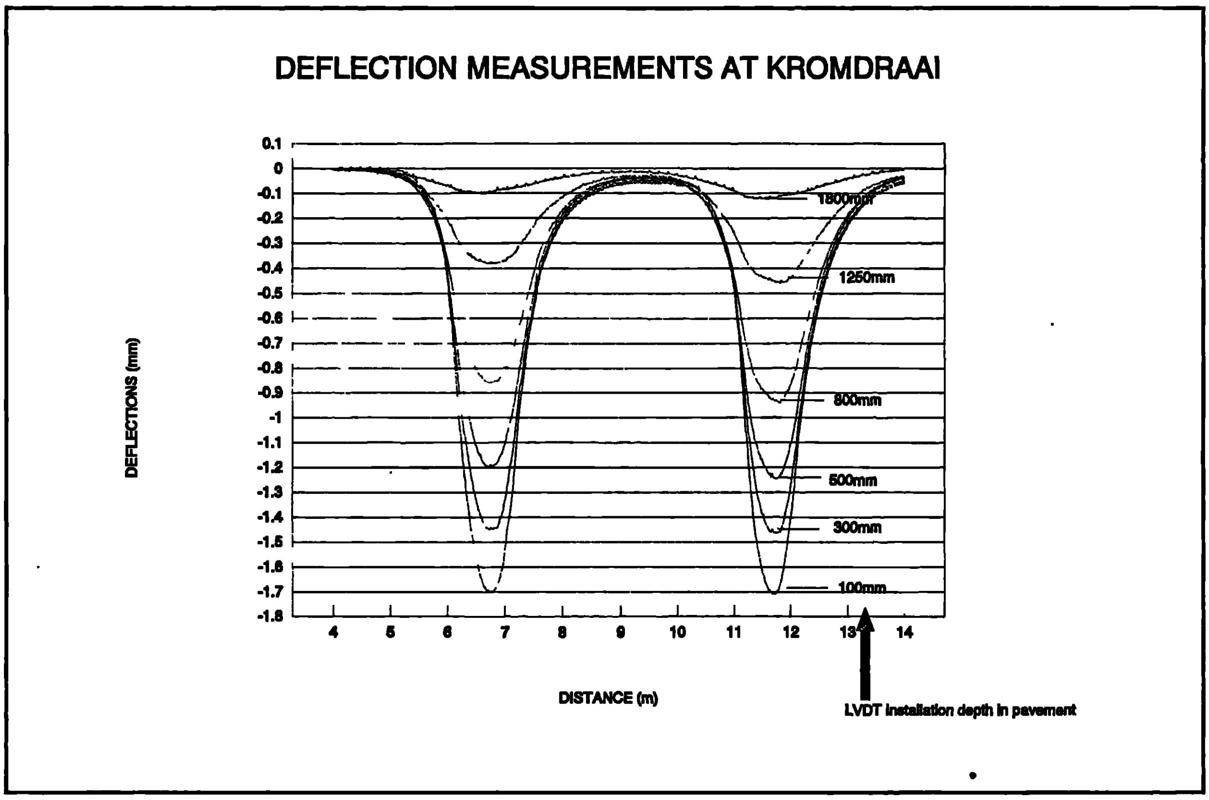


Figure 5.2 Typical Depth-deflection Profile Generated From a MDD Array.

layers with cracking. To determine the layer response to an applied load, it was necessary to use layered elastic models and a back-calculation approach in which actual pavement deflections were compared with those calculated in the model pavement structure for a particular modulus of elasticity. The ELSYM5A (1985) program is used for this purpose.

The effective modulus of elasticity (E_{eff}) and Poisson's ratio (μ) define the material properties required for computing the stresses (deviator σ_d and sum of principals Θ) and strains (vertical ϵ_v and horizontal ϵ_h) in a pavement structure. In addition to the material properties, layer thickness is also specified, in this case with reference to the DCP derived redefined layer structural thickness data given in Appendix B. For computational purposes, the layers are assumed to extend infinitely in the horizontal direction and the lowest pavement layer, extending from a point where the MDD recorded deflections are extrapolated to zero, to be infinite in depth and assigned a high elastic modulus to account for the observed stiff layer with zero deflection. It is also assumed that material behaviour is perfectly linearly elastic, homogeneous and isotropic.

The applied load is calculated according to the mass of the vehicle and the axle load distributions given in Tables 3.3, 4 and 5, together with tyre pressure, to calculate the contact stress. The assumption is made that the area of the applied load approximates to a circle which is valid except at very shallow pavement depths. Figure 5.3 summarises the layered elastic model and data requirements.

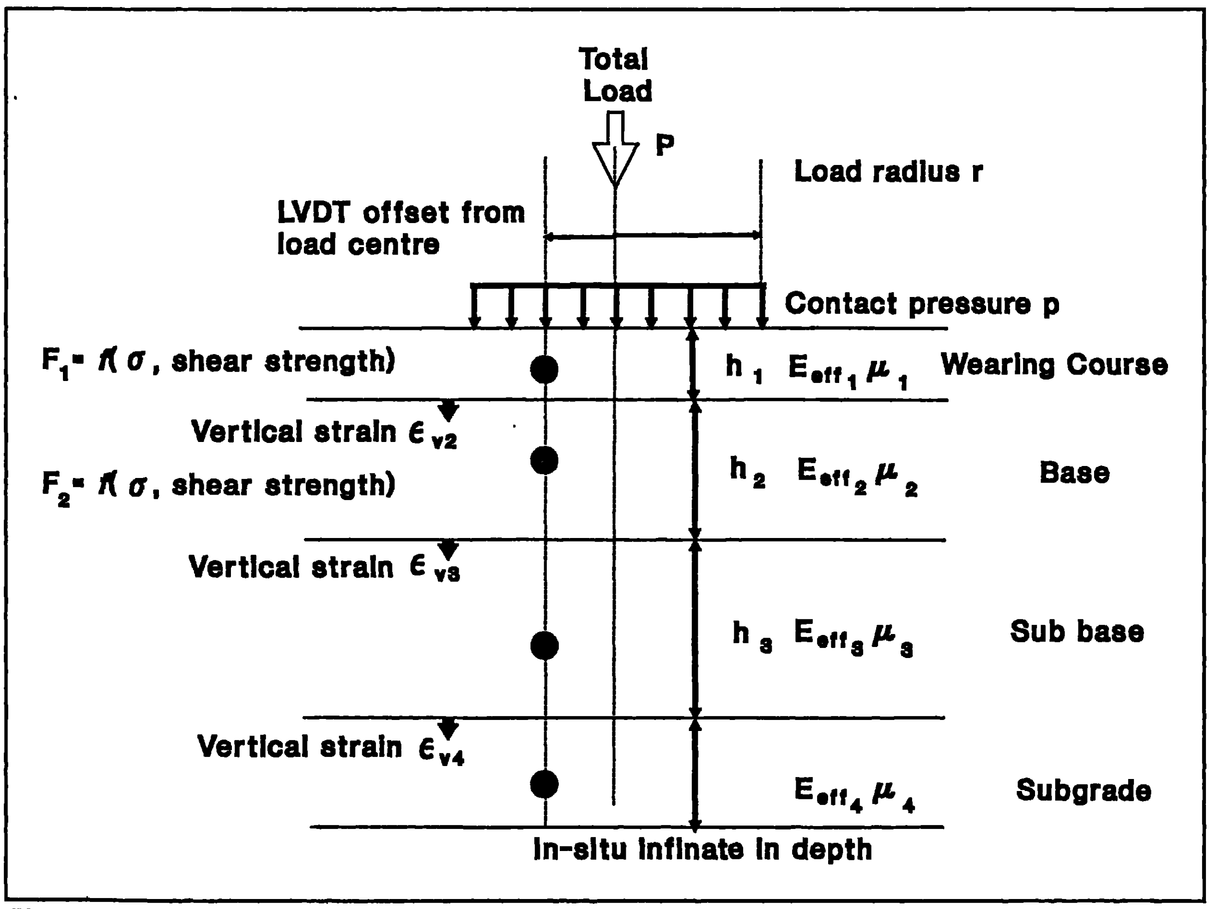


Figure 5.3 Layered Elastic Pavement Model for Use with MDD Data.

5.3 Mechanistic-Empirical Design Process

The deflections generated in each pavement layer due to the applied load are used to back-calculate an effective elastic modulus which satisfies the measured deflections recorded in each layer. The elastic modulus represents a constant ratio of stress and strain as presented in equation [5.1].

$$E_{eff} = \frac{\sigma}{\varepsilon} \quad \text{and} \quad \varepsilon = \frac{\Delta l}{l}$$

where σ = stress (MPa) in layer [5.1]
ε = strain in layer
Δl = deflection in layer
l = layer thickness

Poisson's ratio relates the ratio of transverse to vertical strains and is required for elastic layer modelling with the ELSYM5A program. Since it is particularly difficult to generate reliable values for Poisson's ratio from laboratory tests, an assumed value is used. It is believed that multi-layer elastic analyses are relatively insensitive to small variations in this value. A value of 0,35 is suggested by Maree and Freeme (1981) to be representative of most granular materials, although saturated materials can exhibit a value of 0,5.

The solution technique adopted with the ELSYM5A program involves the manual iteration from some assumed seed moduli for each layer until calculated deflections match those measured by the LVDTs. The seed value used for the effective elastic moduli is derived from the DCP testing data reported in Chapter 4.2. Little work has been done to correlate DCP results with effective elastic moduli although De Beer (1991, 1992) has proposed a tentative empirical relationship based on a 40kN dual wheel load (520kPa contact stress) which is illustrated in Figure 5.4. The relationship may be expressed mathematically as given in equation [5.2] with associated standard error of estimate of 0,209 and R^2=76% for a penetration rate of 0,63 to 25mm per blow (CBR from 7% to 380%).

$$\log(E_{eff}) = 3{,}04758 - 1.06166(\log(DN)) \qquad [5.2]$$

This relationship is used initially as a seed value from which to commence the manual iteration. It remains to be seen whether this relationship holds true at the load and stress levels encountered on mine haul roads. Other empirical relationships exist by which the seed moduli may be sought, including the Shell, WES, TRRL and Danish Road Laboratory methods (Federal Highway Administration, 1994). These are limited in their applicability to CBR ranges of 1% to 20% only.

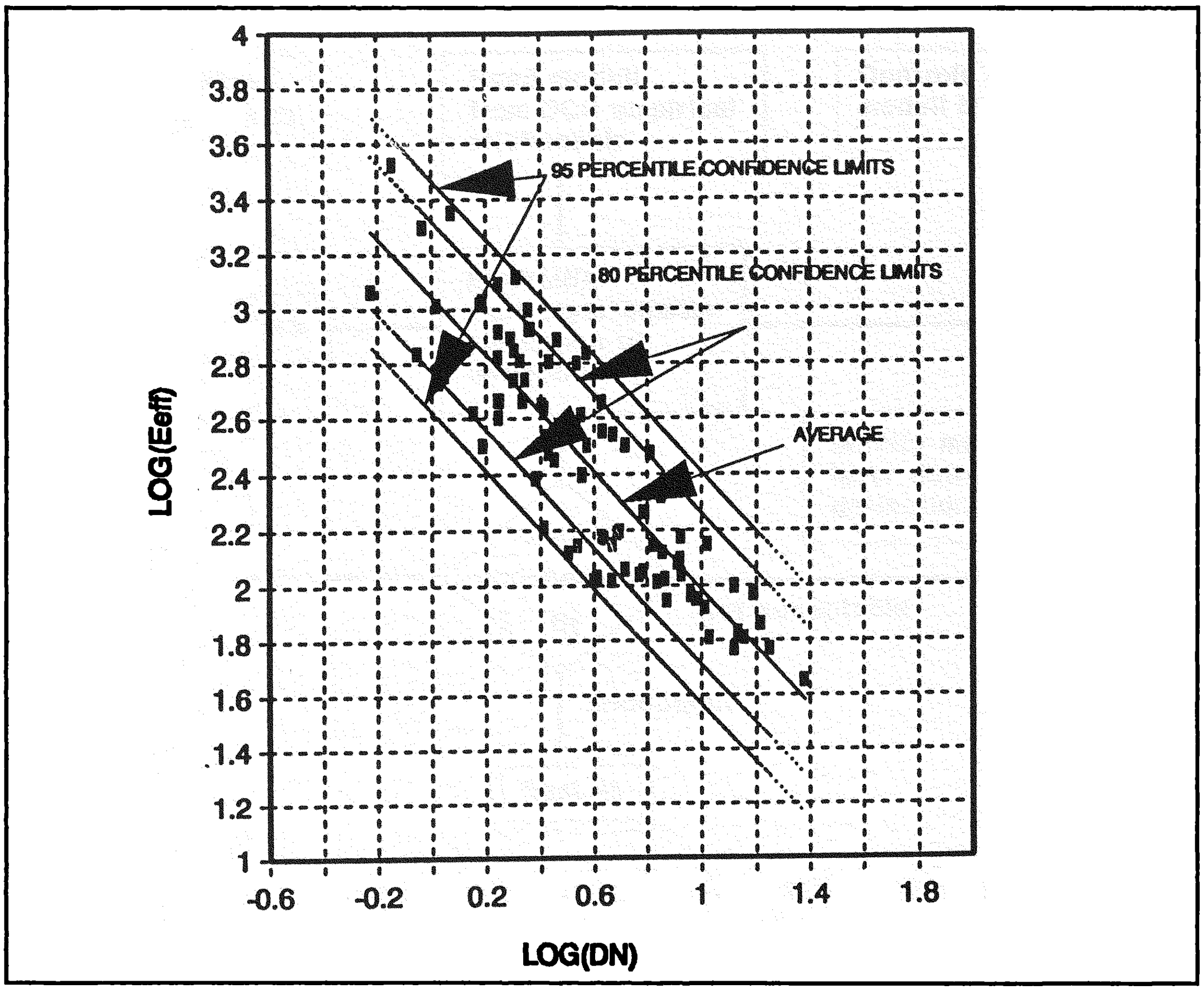


Figure 5.4 Tentative Empirical Relationship Between Effective Elastic Modulus (E_{eff}) and DCP Penetration Rate (DN) for a 40kN Dual Wheel Load(after De Beer, 1991)

The goodness of fit is determined for a particular LVDT (usually located centrally within the layer) to within 2% of the actual recorded deflection. The procedure is illustrated in Figure 5.5. Once individual layer moduli are determined, stresses and strains are then determined with ELSYM5A and compared to established design criteria to verify whether critical stresses or strains have been exceeded. Little published data exists concerning established design criteria for haul roads. The most tractable approach is thus to identify those damage parameters applied in the design of pavements subject to standard axle loadings and by means of categorising haul road test section structural performance, deduce acceptable design criteria for haul road structural design. In the case of haul road structural design and analysis, three distinct design criteria may be adopted from conventional pavement design. Table 5.1 appertains to the criteria associated with the pavement structural layers.

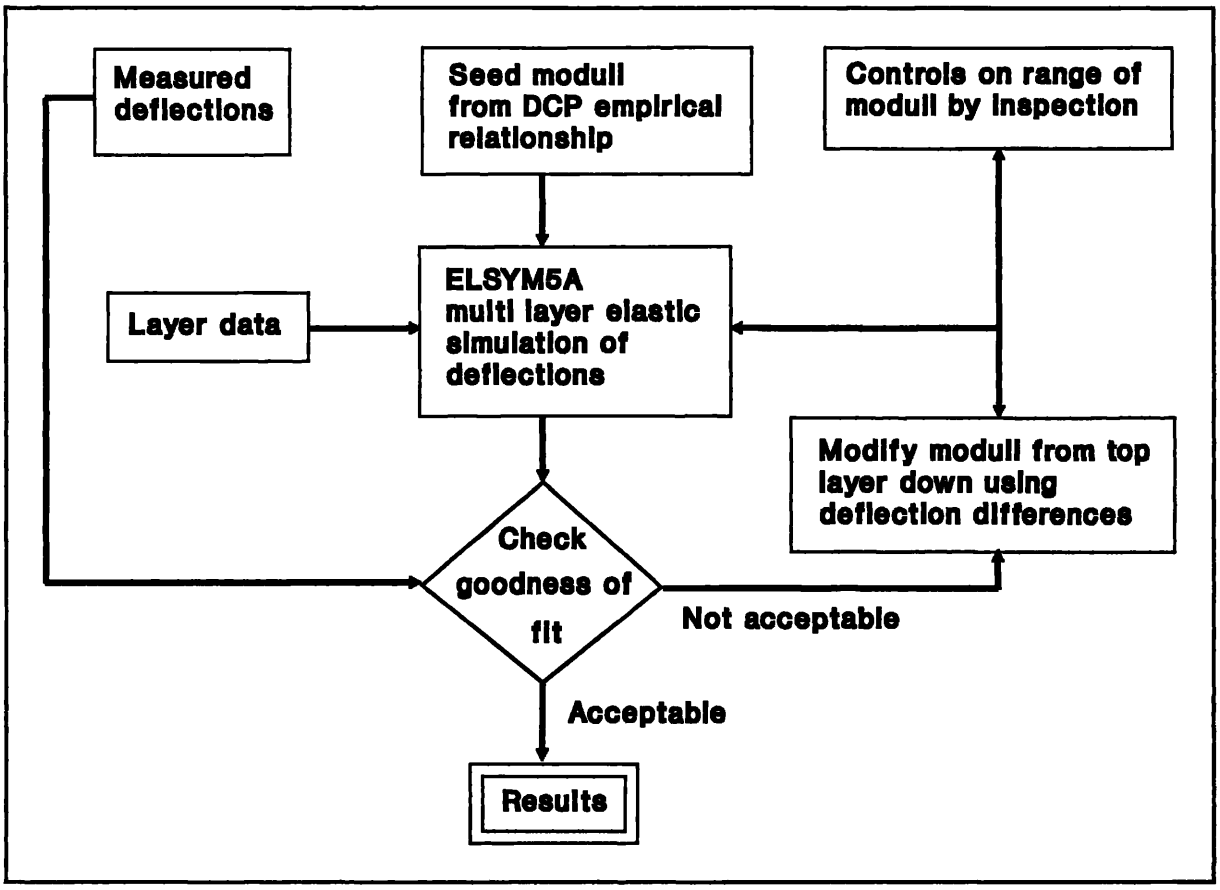


Figure 5.5 Manual Iteration Procedure used with ELSYM5A Program (after Lytton, 1989)

Table 5.1 Design Criteria Applied to Haul Road Pavement Layers (after De Beer, 1992)

LAYER	DAMAGE PARAMETER	DESIGN CRITERION
1	Stress state (σ_1, σ_3)	Safety factor (F) where F = $f(\sigma$, shear strength) Stress sensitivity
2	Stress state (σ_1, σ_3) Vertical compressive strain	Safety factor (F) where F = $f(\sigma$, shear strength) ϵ_v Stress sensitivity
3	Vertical compressive strain	ϵ_v Stress sensitivity
4	Vertical compressive strain	ϵ_v Stress sensitivity

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Safety Factor

Granular materials exhibit distress through cumulative permanent deformation or inadequate stability. Both forms of distress are related to the ultimate shear strength of the material and to prohibit shear failure or excessive gradual shear deformation in the layer, traffic generated shear stresses must be limited. The ultimate shear strength of the layer can be calculated from the maximum single load shear strength, expressed in terms of the Mohr-Coulomb strength parameters c (cohesion) and Φ (angle of internal friction).

The safety factor at any point in the layer can be defined following Maree (1978) as;

$$F = \frac{\text{Maximum safe shear stress}}{\text{working shear stress}} \qquad [5.3]$$

from which equation [5.4] follows;

$$F = \frac{2Kc\tan\left(45+\frac{\Phi}{2}\right)+\sigma_3\left|K\tan^2\left(45+\frac{\Phi}{2}\right)-1\right|}{\left|\sigma_1-\sigma_3\right|} \qquad [5.4]$$

where σ_1 and σ_3 = calculated major and minor principal stresses acting at a point in the layer. (Geomechanics sign convention adopted)

c and Φ = Mohr-Coulomb strength parameters cohesion (kPa) and angle of internal friction (degrees)

K = Constant; 0,6 for highly saturated materials, 0,95 for normal conditions

σ_3 = Minor principal stresses

Equation [5.4] can be rewritten to accommodate published values for friction and cohesion term components applicable to the particular granular material. In this case;

$$F = \frac{(\text{c-term}) + \sigma_3(\phi\text{-term})}{\sigma_d} \qquad [5.5]$$

Published c and Φ term components (Maree and Freeme, 1981) for dry base quality gravel (G4) are generally applicable for materials used in haul road construction, in this case 223 and 5,50 respectively. The shear strength c and Φ terms increase with increasing quality of the road building material and a reduction in moisture component thereof. Safety factors vary widely with depth within a layer, typically between compressive and tensile conditions. In this analysis, safety factors are calculated under single or at the centre of dual wheel assemblies at the mid-depth of the layer, following Maree (1978). Minimum safety factor values are available for various levels of equivalent traffic (E80 axle repetitions) for various categories of public roads. Owing to the uncertainty surrounding the load equivalency factor and equivalent damage attributable to ultra-heavy axle loads (between 600 and 300 000 passes of a standard E80 axle), extrapolation of these recommended safety factor values is unreliable and recourse must be made to categorisation of performance to deduce limiting safety factor values.

Elastic Vertical (Compressive) Strain

For paved roads, limitations are placed on the permissible compressive vertical elastic strains at the top of subgrade layers to prevent rutting and subsequent deformation of the road surface. Limiting the rut depth for unpaved roads is valid and in addition penetration of the upper construction layers into the subgrade should be avoided. In a similar manner, the Asphalt Institute subgrade design criteria for flexible airport pavements establishes permissible subgrade strain values for different load repetitions and subgrade moduli (Asphalt Institute, 1973), as does the Federal Aviation Administration (Brown and Rice, 1971).

Four characteristics of an unpaved road that influence the magnitude of vertical subgrade strains under the application of a constant wheel load may be identified;

(i) Resilient modulus of wearing course material

(ii) Wearing course layer thickness

(iii) Subgrade resilient modulus

(iv) Wander width

In the case of lateral wander width, although the depth of rutting that results from a given level of vertical strain is influenced by wander width, the operational practice observed on strip mine haul roads tends more to channelised (haul truck) traffic. This arises primarily from the left-hand drive configuration of the trucks and the various traffic speeds on the road, predictability being important for safe overtaking maneuvers.

The design criteria for the layers below the wearing course is that of horizontal tensile or vertical compressive strain depending on whether stabilised layers are used or not. These relate to the failure criteria of fatigue cracking of stabilised layers and rut initiation in the subgrade (respectively). Analogous to the safety factor design criteria, no published data exist relating the limiting values for strains in a haul road associated with adequate structural performance. Recourse must be made to categorisation of performance to deduce limiting vertical strain values, taking cognisance of the characteristics and limiting strain values suggested above.

The performance of stabilised layers included in the structural design of haul roads is not considered here since similar structural performance levels may be obtained without the use of (relatively expensive) stabilisation techniques. Additionally, only one mine site incorporated a stabilised layer in the design and thus no comparative conclusions may be drawn concerning the relative efficacy of the various design options available with stabilised layers.

Stress Sensitivity

Many unbound granular materials are stress sensitive with moduli significantly affected by stress level. Granular materials will often exhibit stress-stiffening behaviour with the modulus increasing with increased stress level according to the general relationship given in equation [5.6]. Fine grained materials exhibit the converse, where modulus decreases with increasing stress level. Equation [5.7] represents this behaviour. When considering the structural design of a pavement it is important to categorise this stress dependant behaviour since departures from anticipated behaviour (especially predicted stress-hardening in granular materials) may lead to under design.

Generally for granular materials;

$$E = k_1 \theta^{k_2}$$

$$\text{where } k_1, k_2 = \text{constants} \qquad [5.6]$$

$$\theta = \text{sum of principle stresses}$$

Generally for cohesive materials;

$$E = k_3 \sigma_d^{\ k_4}$$

$$\text{where } k_3, k_4 = \text{constants} \qquad [5.7]$$

$$\sigma_d = \text{deviator stress}$$

The stress sensitivity of the materials comprising each mine test site pavement layer are determined graphically from the results of the ELSYM5A analysis for the various loads applied. By plotting the variation of either the deviator stress (σ_d) or the sum of principal stresses (Θ) against the effective elastic modulus of each pavement layer material, any stress-stiffening or -softening can be identified.

5.4 Multi-depth-deflectometer Results

Results of the MDD installations at each mine site are given in Appendix D. A typical result is given in Figure 5.6 showing the average pavement deflection associated with a particular vehicle. For each vehicle axle test, average deflection values were calculated by inspection on the basis of load offset from the MDD array vertical axis.

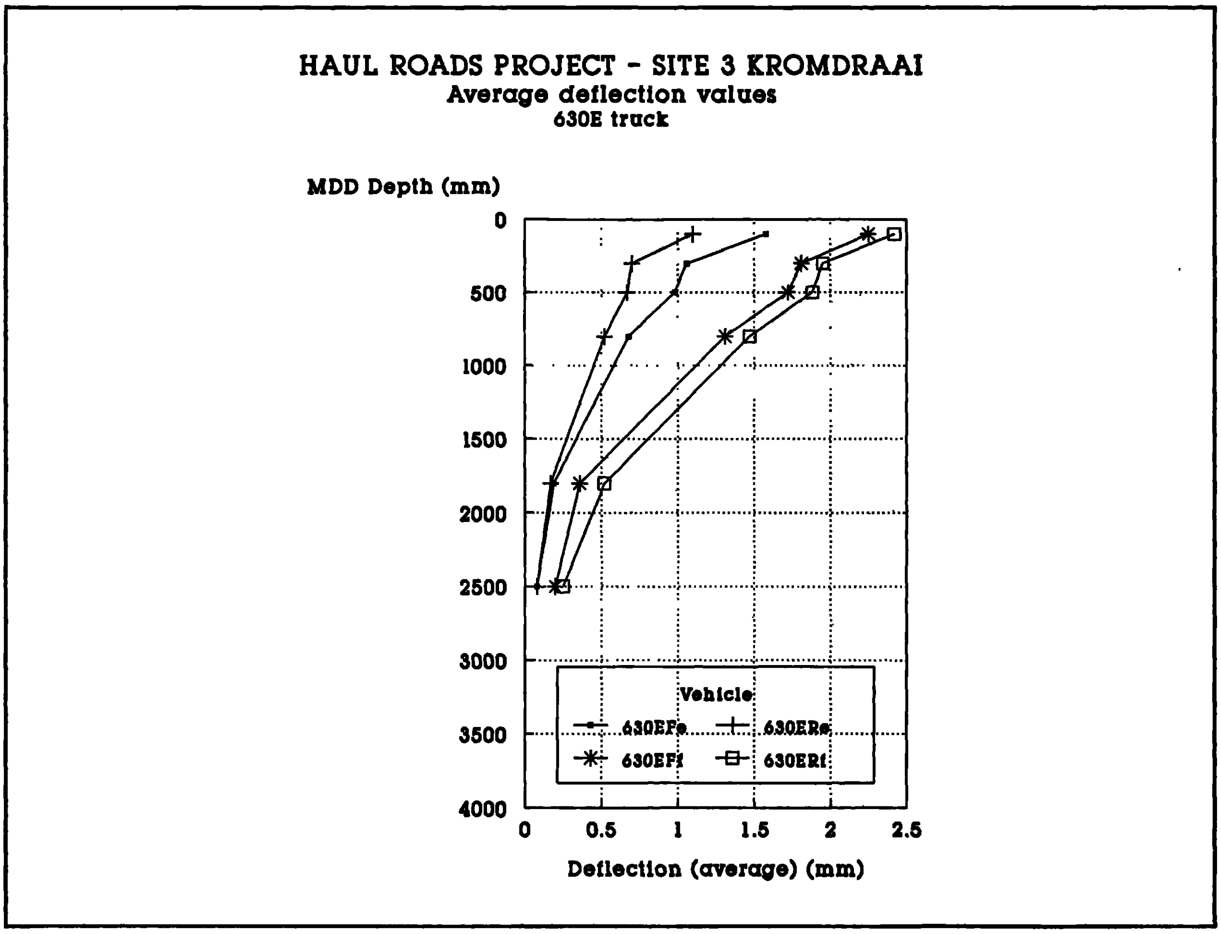

Figure 5.6 Typical MDD Derived Vehicle Deflection Profile.

The top of the model semi-infinite lowest layer is determined by extrapolating deflections to zero. This indicates the depth at which no load induced stress or strain is felt in the subgrade. With reference to Figures 4.5, 6 and 8, it may be seen that this depth varies according to the structural design used. Additionally, the top of the layer is defined for the largest applied load encountered at the particular mine site. Since no deflections are felt below this level, a high modulus value is ascribed to this material to simulate a rigid base. Additionally, high water tables (as evidenced during MDD installation and cone penetrometer probing) can also be reconciled with the inclusion of a stiff layer as a result of pore pressure increases in response to an applied (transient) load. For the mine sites investigated several exhibited saturated material deeper in the road stucture. Whilst this may appear favourable in terms of subgrade deformation, fluctuations in water table levels and the application of slow moving heavy loads may give rise to incidences of deformation and road structural failure. Nevertheless, the analysis serves to provide original data confirming the depths at which no load induced strains are felt for the ultra-heavy loads applied by mine haul trucks.

5.5 Haul Road Structural Performance Classification

As a precursor to the analysis of the structural performance of haul roads and the derivation of limiting design criteria for safety factor and vertical compressive subgrade strain, a classification of haul road structural performance was required to indicate in broad terms the adequacy of the various designs encountered. This was achieved by assigning each mine test site an index on a scale of 1-10 representing poor to excellent structural performance, together with a short summary of the structural defects observed or reported by mine personnel. In addition, the maximum deflection recorded in the structure was depicted for each site for dual rear wheel loads ranging from 429-439kN as an aid to classification. Figure 5.7 illustrates these data.

5.5.1 Results of Mechanistic Analysis - Kriel Colliery

Results are presented in Appendix D1 for all Kriel Colliery site mechanistic analyses.

<u>Site 1</u>

The pavement construction at this site consists of approximately 5m of fill under the pavement. Structural thickness is 1700mm, consisting of 320mm wearing course, 140mm base and 1240mm sub-base. The fill material is modelled to a depth of 2100mm, below which no deflections were measured. Structural performance according to Figure 5.7 is very poor and mine personnel report that excessive maintenance is required in this area. The pavement is seen to deform both vertically and horizontally due to the combined failure of the road shoulders and running surface under the action of high axle loads. The DCP analysis provides some insight into structural performance in terms of the pavement balance (Table 4.2) in which it is seen that this site is an averagely balanced shallow structure, susceptible to failure in the upper layers. CBR values for the layers range from 228% in layer 1 to 27% in layer 3.

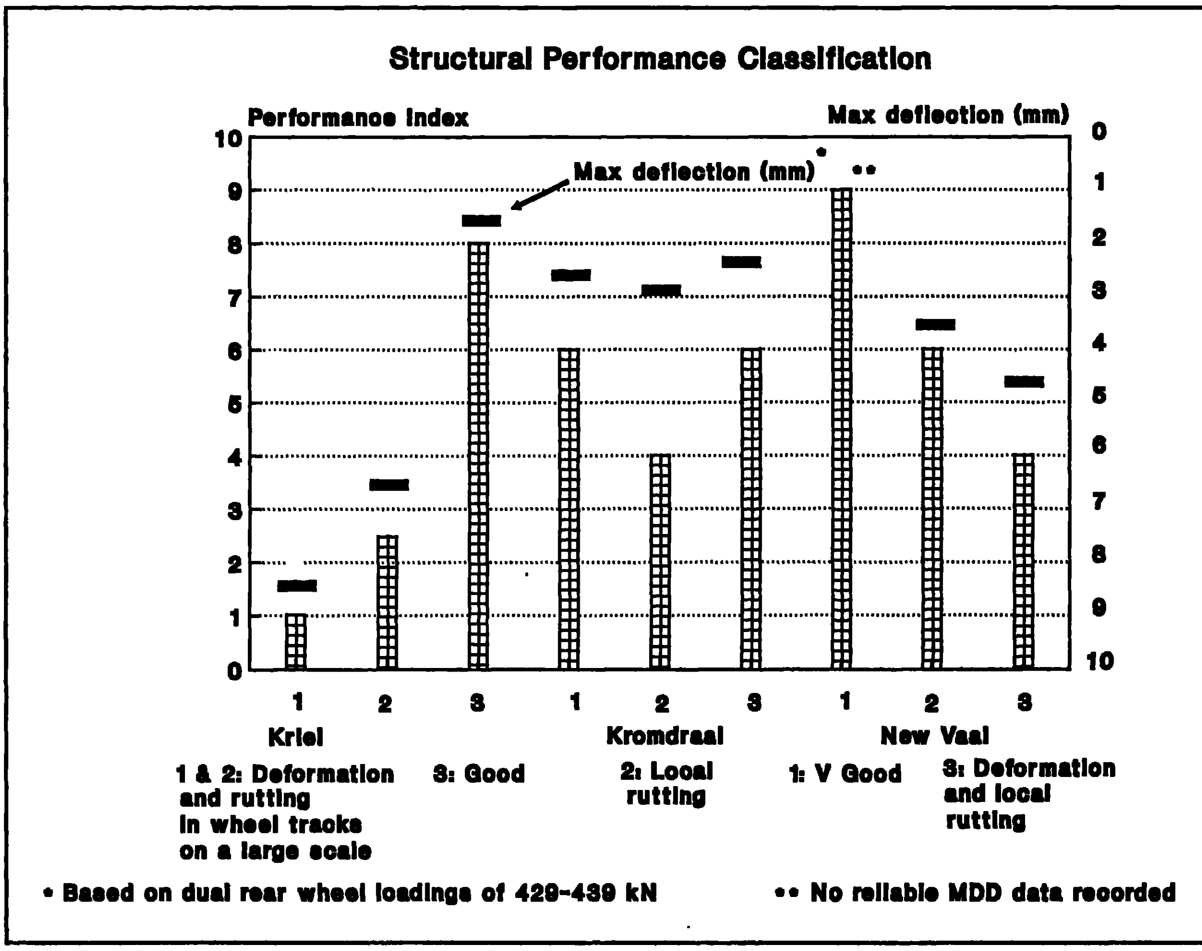

Figure 5.7 Structural Classification of Mine Haul Road Test Sites

With regard to the modelling of deflections generated by the MDD installation, a good fit was seen for most layers except layer 1 uppermost module. The effective elastic moduli for layer 1 could be questioned, especially if non-linear behaviour is present as a result of an impure granular material. This was evidenced at the site by cracking of the wearing course under the shear action of the wheel loads. This implies the material has a high plasticity index and thus deviates from a true granular material. The effective elastic modulus adopted for this layer was highly variable (200-550MPa) but accomodated repeatable modulus values and closer agreement between measured and calculated deflections lower in the pavement.

The factor of safety (FOS) design criterion reveals that the wearing course has a minimum FOS of 5 whilst the base and FOS of approximately -3 at the midth depth positions chosen. These values are indicative of both layers bending and hence a stress reversal from compressive to tensile deeper in the structure. In layers 2, 3 and 4 the design criterion is that of vertical strain. It is clear that layer 3 is subjected to excessive strain whilst layers 2 and 4 proportionately less. The excessive vertical displacement experienced at this site may then be attributable to deformation in these layers.

Regarding the propensity of the various pavement layers to stress harden or soften, plots are presented of effective elastic modulus against deviator and sum of principal stresses (respectively) to ascertain if such effects are seen. Stress softening of layer 2 material is evident, reducing its strength under the action of increased loads. A granular pavement generally exhibits stress hardening and thus the material comprising layer 2 does not have a pure granular structure. A plasticity index value for the layer of 12-15 as opposed to less than 8 for granular materials confirms the observed behaviour. No stress hardening effects are seen.

If the CBR cover curve design criteria (section 4.4.2) are compared with the results of the mechanistic analysis it is seen that the outcome of the CBR technique is under-design, specifically in terms of the excessive vertical strains seen in layers 3 and 4. This is seen by the approach of the top of layer 3 to the minimum cover requirements, but not evidenced by layer 4.

<u>Site 2</u>

Site 2 is also problematic as regards structural performance and excessive maintenance and remedial work. This is typified by the large deflections seen, typically in excess of 7mm. The road was built 950mm in thickness over a vlei area by placing a rock base (layer 3) followed by the construction layers 2 and 1. The pavement was modelled to a depth of 2800mm including the in-situ layer 4 material. Below this depth no deflections were observed.

The DCP analysis classifies the profile as averagely balanced inverted with strength increasing from layers 1 to 3 and decreasing again in the lowest layer. Layer 3 corresponds to the rock layer and the MDD results, although yielding a modulus value may be regarded as unreliable in this layer.

The mechanistic analysis reveals very high vertical compressive strains in layer 2, exceeding

8000 microstrain. Layer 3 exhibits low values typical of a rock layer whilst layer 4 maximum strains of 2700 microstrains. From the stress sensitivity graphs it is seen that layers 1 and 2 may be regarded as stress softening due to the inclusion of clay materials and this exacerbates the performance problems at this site.

If the CBR cover curve design criteria for the site are compared to the results of the mechanistic analysis it is seen that under-design is apparent. This is due in part to the very different CBR profile to that of the MDD generated effective elastic moduli profile. This aspect will be discussed in more detail later. The inclusion of a rock layer at depth does not appear to improve performance of the road as predicted by the CBR design criteria.

Site 3

The road at site 3 is an old section of haul road constructed early in the life of the mine by contractors. It comprises material common to other sites but also a lime stabilised layer from 220mm to 460mm. Total structural thickness is 1000mm with in-situ material extending to a depth of 3400mm below which no deflections were observed. The structural performance of the road is excellent as evidenced in Figure 5.7, the small deflection measured being due in most part to the resilience of the stabilised layer. If the performance of the remaining layers are assessed without the stabilised layer, it is likely the lower layers in the road would not perform as adequately.

The DCP generated profile records it as a poorly balanced shallow structure, due to the effect of the lime stabilised layer. However, the mechanistically derived performance data tend to correspond well with the field observations of performance

The wearing course layer has a FOS of under 2 although there was no field evidence to support this result. The vertical strains recorded in layers 3 and 4 were particularly low, due primarily to the action of the stabilised layer. Referring to the stress sensitivity plots it may be seen that there is no clear evidence to support either stress softening or hardening from the data analysed.

When comparing the CBR cover curve design criteria with the results of the mechanistic analysis it is seen that despite the poorly balanced shallow profile, the road performs well and is not susceptible to the effects of high axle loads in the upper layers, primarily due to the load carrying capacity of the stabilised layer. This result is considered to have important implications in terms of the optimal structural design of a haul road.

5.5.2 Results of Mechanistic Analysis - Kromdraai Colliery

Results are presented in Appendix D2 for all Kromdraai Colliery site mechanistic analyses.

Site 1

The pavement at this site was constructed by contractors to design specifications. Pavement depth is the shallowest of the three sites considered; three layers extending down to 1000mm and the in-situ material modelled to a total depth of 2100mm below which no load induced deflections were seen. The road structural performance is good as reflected in Figure 5.7, with maximum deflections of approximately 2,5mm recorded. Mine personnel do not report any specific under performance of the road at this site.

The DCP generated profile for the site is that of a well balanced shallow structure with CBR values ranging from 211% in the top layer, decreasing to 17% in the in-situ material. The mechanistic analysis in terms of vertical strain reveals maximum values of approximately 1 000 microstrain in layers 2 and 4 whilst layer 3 exhibits a maximum of 2 500 microstrains. From the stress sensitivity plots no evidence of stress sensitivity is seen in any of the pavement layers.

The CBR cover curve design criteria for site 1 anticipates less cover than that actually placed and would appear to provide a reasonable, slightly conservative, base for design in this case. The larger vertical strains in the top of layer 3 are seen to coincide with the approach of the actual cover curve to the predicted cover requirements in the vicinity of layer 3. Layer 4 departs from the predicted curve and vertical strains are seen to reduce.

Site 2

The pavement was constructed by the mine and localised problems are experienced with deformation due to clay within and underlying the pavement construction. The structure consists of 3 structural layers to a depth of 1000mm placed on in-situ material (layer 4) which extends to a depth of 2750mm below which no deflections are observed. Construction in this area was not complete at the time of testing and an additional wearing course layer will be added to a total depth of approximately 200mm.

The road structural performance is adequate and a maximum deflection of 3,0mm recorded. Mine personnel report localised deformation as a result of clay in the construction and the classification (Figure 5.7) is accordingly lower than that suggested by the maximum deflection. The DCP generated profile is that of a well balanced shallow structure with corresponding CBR values from (a high) 354% in the top layer to 8% in layer 4.

The mechanistic analysis in terms of vertical strain reveals maximum values of approximately 1000 microstrain in layer 2, 3000 in layer 3 and 1500 in layer 4. With reference to the stress sensitivity plots, layer 1 is omitted since an assumed effective elastic modulus was adopted for the layer in the absence of any MDD generated deflections at this depth. Layers 3 and (particularly) layer 4 exhibit stress softening tendencie., a fact which may be attributable to clay material in the pavement layers. Loads are carried in decreasing proportions as predicted by the DCP generated data.

The CBR cover curve design criteria for site 2 anticipates more cover required than is actually placed, especially in the vicinity of layers 3 and 4. The larger vertical strains seen in layer 3 appear to coincide with the CBR predicted localised under-design at this depth. The FOS values for layers 1 and 2 of approximately 12 and 6 correspond to the (assumed) load carrying capacity of each layer (due to the adopted of an assumed modulus for layer 1) and the reduced thickness of the top layer.

Site 3

The pavement at this site was constructed upon 4000mm of fill material by the mine. The construction was not complete at the time of testing and a wearing course layer of ±300mm was due to added. Three layers are identified, the top 2 layers extending to a depth of 650mm and the fill material (layer 3) modelled to a depth of 3300mm below which no deflections were observed. The road structural performance is classified as good with a maximum deflection of 2,4mm recorded. The DCP profile is that of an averagely balanced shallow structure, CBR values ranging from 269% in the top layer to 26% in the fill material.

The mechanistic analysis revealed maximum vertical compressive strains of 1300 and 1900 microstrains in layers 2 and 3 respectively. Some evidence of stress softening is seen in layers 2 and 3 due most probably to the presence of clay material in these layers. Again, the load carrying is fairly well predicted by the DCP generated balance curve.

The CBR cover curve design criteria for site 3 anticipates less cover than that actually constructed except in the vicinity of the top of layer 3 where the actual cover approaches predicted cover. The technique appears to give reasonable, if not slightly conservative results. Vertical strain at this point is approximately 1900 microstrains and seems to correspond well with localised cover reductions at this point. FOS values of 9 and 6 indicate that the applied stresses are much lower than the ultimate strength of the layers.

5.5.3 Results of Mechanistic Analysis - New Vaal Colliery

Results are presented in Appendix D3 for all New Vaal Colliery site mechanistic analyses.

<u>Site 1</u>

No meaningful data could be deduced from the recorded MDD deflections at this site due to inferred anchor movement during testing.

<u>Site 2</u>

The pavement at site 2 was constructed by the mine and consists of three layers to a depth of 1010mm. The road is located in a vlei area with soft material in-situ below the road.

Mine personnel report good/adequate performance over this section of road with only localised sections showing signs of distress. The classification reflects this and the associated maximum deflection of approximately 3,5mm. The DCP generated profile classifies the road as an averagely balanced deep structure, primarily due to the presence of a particularly weak layer 2.

The results of the mechanistic analysis show that in terms of vertical compressive strain at the top of layers 2 and 3, maximum values of 1800 and 1000 microstrain (respectively) are recorded. From the stress sensitivity plots, layers 1 and 2 are seen to be stress hardening, their strength increasing with increased trafficking.

The CBR cover curve design criteria again provide a conservative estimation of cover requirements. Whilst vertical strains in layers 2 and 3 are low, any reduction in cover thickness or layer strength may result in unacceptably high deflections in the pavement.

The FOS values calculated for layers 1 and 2 of approximately 6 and 9 (respectively) correlate with applied stresses in each layer but not with the DCP generated profile of increasing strength with depth.

Site 3

The pavement at site 3 was constructed by mine personnel and consists of three layers over a total thickness of 750mm. Road performance is adequate for the low level of traffic seen, but maximum deflections of 4,5mm and local deformation in wheel tracks result in some remedial work being required to maintain structural performance. The DCP generated profile is that of a poorly balanced deep structure, primarily due to the occurrence of a weak layer (CBR 30%) in the structure. This is borne out from the results of mechanistic modelling.

The mechanistic analysis reveals that this weak layer is subjected to maximum vertical strains of approximately 4500 microstrains and as such is the cause of much of the deformation seen in the road. This layer also exhibits stress softening which exacerbates the problem of excessive strains in the layer. In addition, the propensity of strain softening also explains the poor fit between MDD derived deflections and those obtained by mechanistic modelling. This may be alleviated by the addition of an extra (weaker) layer in the model.

5.6 Summary and Conclusions

The mechanistic analysis and quantification of existing pavement structural designs, incorporating a categorisation of structural performance and the assessment of damage criteria applied in the design of flexible roads and airfields revealed that the vertical compressive strain criterion is an important design parameter linking rut initiation in lower pavement layers with surface deformation. Table 5.2 summarises the results of the mechanistic analysis and quantification for mine haul roads from which it seen that the vertical compressive strain criterion correlates well with observed performance and maximum surface deflection.

When analysing the proposed FOS design criterion, it was found that since the applied stresses were much lower than the ultimate strength of the pavement layer materials and, since the FOS is dependant on the particular depth chosen in the analysis, the combination of high wheel loads and stress reversal in softer materials implies that the FOS criterion was not applicable to haul road design.

Regarding the propensity of the various pavement layers to stress-stiffen or -soften, some localised evidence of stress stiffening and softening was seen. This is however, more a function of the specific construction material used at each site rather than a universal phenomenon. Irrespective of the extent of over- or under-design apparent at each site, the analysis of deflection profiles generated from the MDD installations revealed that no induced vertical strains were seen in the pavement below a depth of approximately 3000mm.

By using the vertical compressive strain criterion in conjunction with the qualitative performance classification and maximum recorded surface deflection, an insight was afforded into the utility of the CBR- and DCP-based structural design techniques. The balance profile approach has limited application in the design of mine haul roads since one

Table 5.2 Summary of Structural Analysis - Mechanistic Evaluation Results

MINE	SITE	TOTAL LAYER	PERFORMANCE CLASSIFICATION			MECHANISTIC EVALUATION			
			Structural defect description	Perf. index. (1=poor)	Max deflection (mm)	Layer	Max vertical μstrain	FOS layer 1	FOS layer 2
Kriel	1	4	Deformation and rutting on a large scale	1	8,7	2	1950	5,0	2,9
						3	5450		
						4	4300		
	2	4	Deformation and rutting	2.5	6,4	2	8400	3,1	5,2
						3	Rock layer		
						4	2750		
	3	4	Excellent structural performance	8	1,6	2	Stabilised	1,7	No data
						3	1350		
						4	900		
Krom-draai	1	4	Localised deformation and rutting	6	2,5	2	950	9,0	4,3
						3	2600		
						4	1200		
	2	4	Localised excessive rutting	4	2,9	2	1350	11,9	6,3
						3	3000		
						4	1500		
	3	3	Localised deformation and rutting	6	2,3	2	1400	9,1	6,1
						3	1900		
New Vaal	1	4	Excellent structural performance	9	No data	No data	No data	No data	No data
	2	3	Localised deformation and rutting	6	3,4	2	1800	6,3	8,7
						3	1050		
	3	3	Localised excessive rutting	4	4,6	2	4650	6,3	8,5
						3	950		

of the most efficient and structurally sound designs incorporates a rock layer at a shallow depth, resulting in a poorly balanced shallow strength profile. An evaluation of the vertical strains generated within the pavement due to the applied load indicates that the strength balances to be avoided are those of inverted structures and, to a lesser extent, poorly balanced deep. Both are associated with excessive vertical strains in the pavement and poor structural performance.

With regard to the CBR cover curve empirical design approach, excessive vertical strains were generally associated with under-design of the pavement where less cover was placed than that predicted by the cover-curve method. The deficiencies inherent in the method, together with the potential for under-design associated with multi-layer structures limit the utility of the method when applied to mine haul road structural design.