CHAPTER 5

COMPRESSION FAILURE OF LIGHTLY CEMENTITIOUS MATERIALS

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5.1 INTRODUCTION

5.1.1 Background

The investigation described in Chapter 4, indicated that the actual failure mechanisms of deep and shallow pavements differ, in that the origin of permanent deformation differs. It was concluded that deep pavements fail predominantly in compression (crushing) in the upper portion of the cementitious base layer, while the shallow pavements fail predominantly in fatigue of the cementitous base layer.

This chapter concentrates on compression (crushing) failure of lightly cementitious pavement layers. It is believed that there is a need to establish a correct understanding of this failure mechanism, as till now this was relatively unknown and unquantified for pavements. This includes the development of applicable failure criteria for this particular failure, so that for example, effective rehabilitation may be implemented on these types of pavements (Maree et al., 1987).

5.1.2 The Problem

Evidence exists in South Africa that relatively thin surface seals on pavements with lightly cementitious base layers show fatigue failure and become loose relatively early in the roads life, resulting in potholes during the wet periods. This problem is accentuated on roads carrying heavy traffic loads and under high contact stresses near commercial sand- and gravel quarries and coalmines, for example, in the Eastern Transvaal. Inspection of failures on these pavements revealed the presence of loose (crushed) base material at the interface between the seal and the base.

Laboratory investigations of these base materials indicated that the inherent quality of the material was adequate and within current specifications for cementitious materials (TRH4, DRTT, 1985). Curing and construction practices were adequately controlled (Kleyn, 1988).

When designing a new road, the Transvaal Roads Department (TRD) considers it sound policy to invest in the supporting pavement layers by providing a relatively deep structure which reduces the structural demand on the upper layers including the wearing course (Kleyn et al., 1986, 1987).

5.1.3 Extent

Evidence of loose seals and surfacings also exists in the Orange Free State (Du Pisani, 1988) and the Cape area (De Villiers, 1988). In a study by Jordaan (1988) compression failure (crushing) was also noted near the top of a cementitious base pavement (MR 27) in the Cape area. This is a clear indication that compression failure of lightly cementitious pavement materials must also be evaluated in addition to the rather well - known fatigue failure analysis of these layers (Otte, 1972, 1978).

In these cases, however, it is not clear if the problem is material, construction and/or traffic associated as with many other pavements in South Africa (Netterberg et al.,1987), as well as with pavements containing weathered igneous aggregates in Botswana (Pinhard, 1987).

5.1.4 Research

Most of the research on lightly cementitious road building materials (C3 and C4) to date concentrates primarily on its fatigue characteristics when the material is used as structural pavement layers. This is probably because of the similarity between the fatigue behaviour of relatively strongly cementitious materials (C1, C2 and lean concrete) and portland cement concrete (PCC) pavement materials. In this chapter, however, an investigation into the behavioural characteristics of relatively lightly cementitious materials (C3 and C4) is described.

Extensive full scale testing with the Heavy Vehicle Simulator (HVS) was done on a relatively deep pavement structure, as defined with the Dynamic Cone Penetrometer (DCP) survey (De Beer et al., 1988). The testing was on Road 1932 near Rooiwal, north of Pretoria (De Beer, 1986a, 1986b, 1986c, 1986d). The base and subbase are of C3/C4 quality. In Chapter 4 the permanent deformation characteristics of this pavement are discussed.

The results indicated that compression failure near the top of the base layer predominantly governed the rate of permanent deformation ("plastic deformation") as measured on the surface of the pavement. In the case of this pavement, and to a lesser extent, on a relatively shallow Road 2212 (Bultfontein), compression failure of the base layer contributed largely towards the total permanent deformation (rutting) on the surface of the pavement after extensive HVS testing. It is therefore important to study this type of failure to quantify its effect and also to provide guidelines accounting for this occurrence during initial design stages.

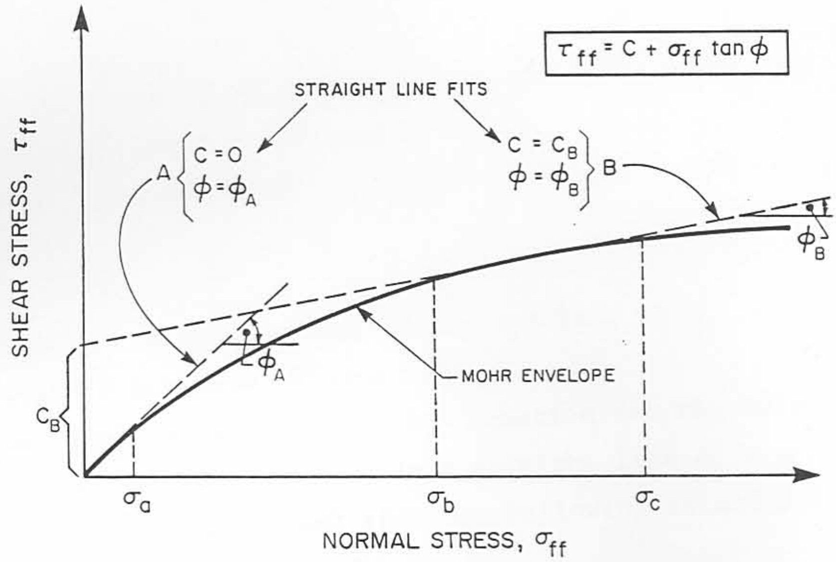
In this chapter these guidelines are given as tentative design and analysis curves from which the number of load repetitions needed to initiate compression failure (Nc) in lightly cementitious layers is obtained for a given tyre contact stress (pressure) on the pavement and a given in situ Unconfined Compressive Strength (UCS) of the material. It is proposed that these curves be used in conjunction with the mechanistic design method and maintenance planning for these types of pavements.

5.2 COMPRESSION STRENGTH PARAMETERS OF CEMENTITIOUS MATERIALS

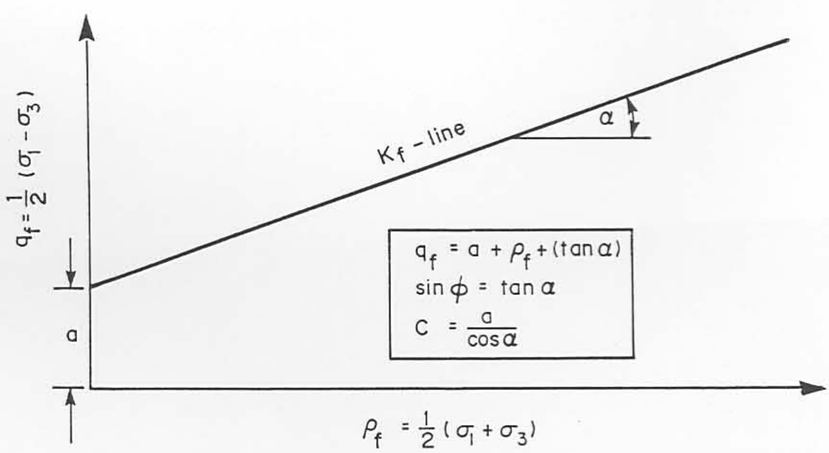
5.2.1 Cohesion c and angle of internal friction ϕ

Several investigators presented the strength properties of cementitious materials according to the Mohr - Coulomb theory (Wissa et al., 1965; Balmer, 1958; Rocha, et al., 1961; Nash et al., 1965; Ferguson et al., 1968; Christensen, 1969; Mitchell, 1976; Clough et al., 1981; Akinmusuru, 1987; and Dupas et al., 1979).

According to the Mohr - Coulomb theory, the cohesion intercept c and the angle of internal friction ϕ are the most important strength parameters of untreated (natural) soil. It was, however, found by the above - mentioned investigators that c and ϕ adequately describe the compression strength of relatively lightly cementitious materials. In Figure 5.1 two methods for determining the c and ϕ are illustrated, based on the Mohr-Coulomb theory. In Figure 5.1(a) the direct method is illustrated whereby c and ϕ are determined directly from the



(a) Mohr envelope from Mohr circles (direct method)



(b) K_f-line from $p-q$ diagram (indirect method)

FIGURE 5.1

THE METHODS OF DETERMINING C AND ϕ FROM A SERIES OF TRIAXIAL TESTS USING THE MOHR-COULOMB THEORY

Mohr-Coulomb circles, utilising the shear strength at failure, τ_{ff}, and the normal stress at failure, σ_{ff}. From straight line approximations, c and ϕ are determined for the relevant normal stress range. In Figure 5.1(b), the indirect method is illustrated whereby c and ϕ are indirectly calculated from the K_f-line parameters, a and α, derived from a p_f-q_f diagram. The relationships for c and ϕ are as follows:

$$q_f = a + p_f \tan \alpha \quad \text{.......... 5.1}$$

$$\sin \phi = \tan \alpha \quad \text{.......... 5.2}$$

$$\text{and } c = a/\cos \phi \quad \text{.......... 5.3}$$

where q_f = shear stress
p_f = normal stress
c = cohesion

The p_f-q_f diagram is obtained from the results of normal triaxial tests and a straight line regression may be performed on the results from the maximum τ-values at each stress state, to obtain the relevant parameters a and α from which c and ϕ are computed (Lambe et al., 1969), using equations 5.1, 5.2 and 5.3.

In one of his studies, Thompson (1966) used a statistical method proposed by Herrin (1954) to determine c and ϕ of cementitious materials. This method is illustrated in Figure 5.2 and includes the following procedure:

1. A plot of σ_1, maximum normal stress applied, vs σ_3, confining pressure, is prepared (Figure 5.2).
2. The best least-squares regression equation for the data is determined. If the equation is a straight line $\sigma_1 = a + b\sigma_3$, the c and ϕ are calculated from the following relationships:

$$c = a/(2\sqrt{b}) \quad \text{.......... 5.4}$$

$$\text{and } \sin \phi = (b-1)/(b+1) \quad \text{.......... 5.5}$$

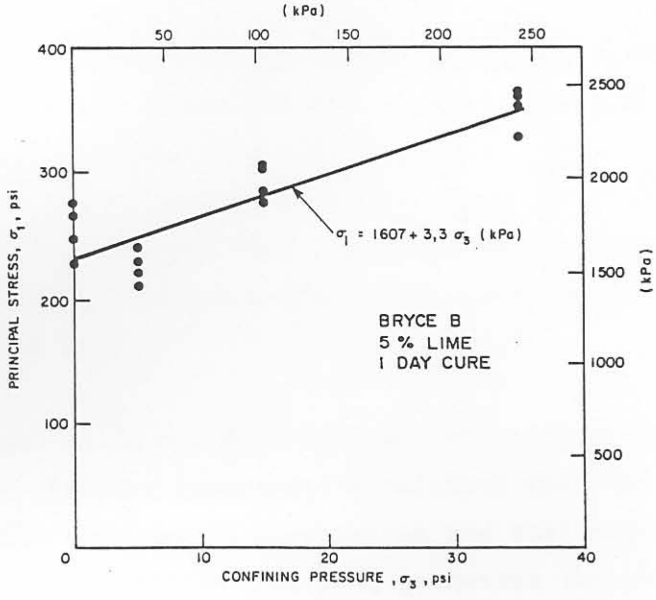


FIGURE 5.2

INFLUENCE OF CONFINING PRESSURE ON MAXIMUM PRINCIPAL STRESS (Thompson, 1966)

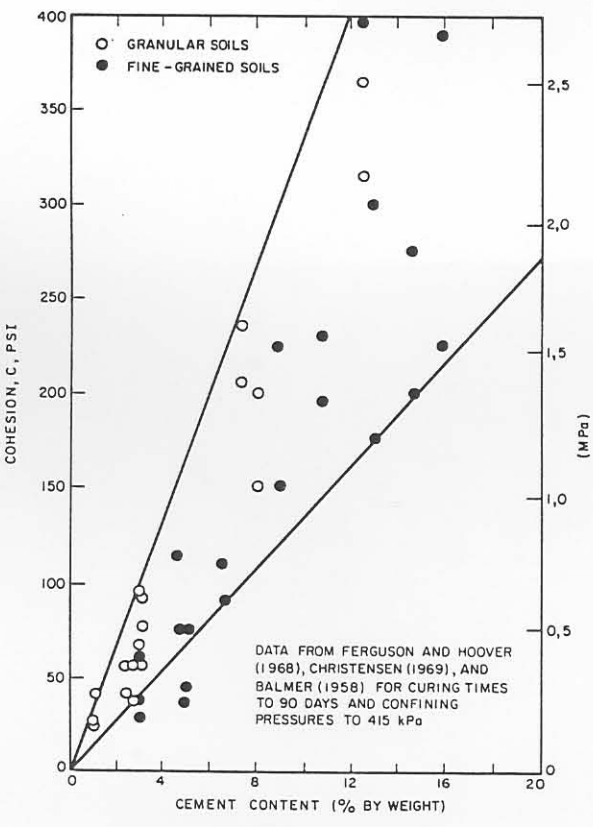


FIGURE 5.3

THE EFFECT OF CEMENT CONTENT ON THE COHESION INTERCEPT FOR SEVERAL SOIL AND CEMENT MIXTURES (MITCHELL, 1976)

5.2.2 Major findings of previous research

Some of the most important findings on the effects of c and ϕ of cementitious materials reported by Wissa et al. (1965) are listed below:

(1) The effective stress principle applies to the strength behaviour of saturated stabilised soils, ie, the Mohr-Coulomb envelope in terms of effective stresses is essentially independent of drainage conditions whereas the total stress envelope is dependent on the drainage conditions during shear.

(2) The addition of a cementing material such as hydrated lime or portland cement can substantially increase the Mohr-Coulomb effective cohesion c, of both coarse-grained and fine-grained soils. The c increases with increasing cement content and increasing curing time (strength of the cement). The development of shrinkage cracks owing to volume changes which occur during curing or weathering (cycles of wet-dry) will cause a decrease in c, and may induce premature fracture at low consolidation pressures (while the cracks remain open) which exhibits itself as an apparent further decrease in c and an apparent increase in ϕ.

(3) Cementation has essentially no effect on the angle of internal friction ϕ of a coarse-grained soil provided the density of the soil excluding the cement is kept constant. Cementation can increase ϕ of a fine-grained soil by as much as 10 degrees. The more plastic the soil and more cement added the larger is the increase. Curing time (strength of the cementation) and weathering have no effect on ϕ.

(4) For stabilised soils the Mohr-Coulomb criterion of failure in terms of effective stresses represents conditions when the sum of the cohesive resistance due to cementation and the frictional resistance due to particle contact, geometric interference (interlocking of particles), and dilatancy in the case of drained shear, is a maximum.

(5) At any given axial strain the total shearing resistance of a soil cemented with hydrated lime or portland cement can be considered to have two components: a cohesive resistance, independent of effective stress and a frictional resistance, dependent on effective stress and therefore a function of the pore pressures developed during shear.

(6) The maximum cohesive resistance of a soil cemented with a nonductile material, such as lime or cement, occurs at smaller strains than the effective Mohr-Coulomb envelope. The c is lower than the maximum cohesive resistance because partial breakdown of the cementation has occurred by the time the envelope is reached and it can be as little as 30 per cent of the maximum value. By the time ultimate conditions are reached the cohesive resistance in the zone of shearing is completely destroyed and the shearing resistance is purely frictional (crushed state).

(7) The maximum frictional resistance occurs at larger strains than the Mohr-Coulomb envelope and it is fully mobilised at ultimate conditions when the effective stress and shear stress remain constant with further straining. It can be 5 to 10 degrees higher than the Mohr-Coulomb ϕ.

(8) The larger increase in the frictional resistance of a fine-grained soil when a stabiliser such as lime or cement is added is believed to be due to the formation of strongly cemented large aggregates of fine-grained particles which are held together by a weaker continuous cementation. The aggregation forms during mixing and compaction and remains essentially constant during curing which accounts for ϕ being independent of length of cure. In the case of sands, aggregation does not occur during mixing because of the relatively large size of the grains and therefore ϕ is independent of the cementation per se.

(9) The addition of lime or cement to a fine-grained soil causes its undrained shear strength to increase because of increase in c and in some cases ϕ. The excess pore pressures which develop during undrained shear are not significantly influenced by the cementation.

(10) The shear strength of a stabilised soil increases with increasing consolidation pressure. The rate of increase in strength with increase in consolidation pressure is greater in drained shear than in undrained shear.

(11) The addition of lime or cement to a soil causes an increase in its initial tangent modulus of elasticity and reduces the strain required to reach the maximum principle stress difference.

5.2.3 Parameters influencing c

In Figures 5.3, 5.4 (a), (b), (c) and (d) the effects of stabiliser content, age of curing and compaction density on cohesion, c, as was found by several investigators, are illustrated. Figure 5.3 illustrates the linear relationships between cohesion c and cement content of both granular and fine-grained soils as was summarised by Mitchell (1976), and indicates a general increase in c and cement content.

Figure 5.4(a) illustrates nonlinear relationships between cohesion and cement content, as found by Dupas et al. (1979), and the figure indicates that c increases with the increase in both compaction density and curing age. Figure 5.4(b) indicates a very marked increase in c with increase in curing age and cement content of a uniformly - graded beach sand, stabilised with cement (Akinmusuru, 1987). Thompson (1966) also found that c increases with curing age, but the rate of increase may differ and may even decrease with increase in curing age, depending on the soil and stabiliser type, see Figure 5.4(c). In Figure 5.4(d), Mitchell (1976) shows that the rate of increase in c related to curing age of a cement - stabilised silty soil, is markedly higher than that of a cement - stabilised A-1 sand.

Generally, c increases with an increase in cement content, curing age and compaction density, mainly because of the formation of stronger cementitious bonds between the aggregates. With an increase in compressive strain, these bonds are destroyed and the cohesion strength c decreases to zero as a result of increasing strain, while the frictional strength ϕ increases to a maximum value, when fully mobilised. This effect is illustrated in Figure 5.5. The initial increase in c is

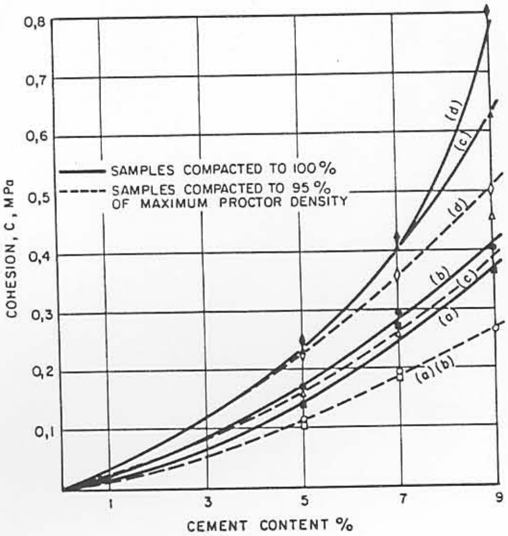


FIGURE 5.4(a)
COHENSION OF UPPER SAND-CEMENT SAMPLES:
(a) 14 DAYS; (b) 28 DAYS - 90 DAYS; (d) 180 DAYS
(DUPAS et al., 1979)

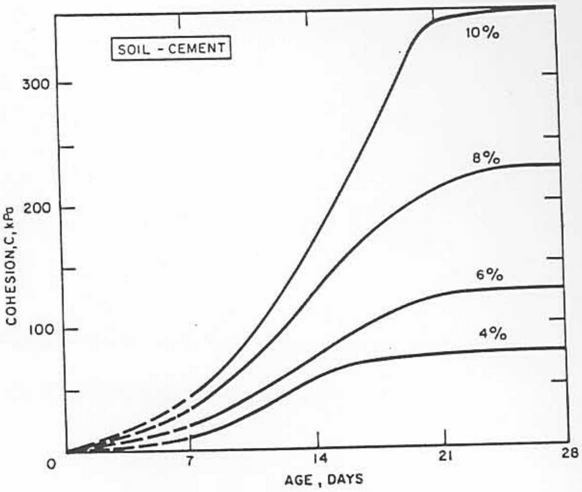


FIGURE 5.4(b)
EFFECT OF CURING AGE ON THE COHESION FOR DIFFERENT CEMENT CONTENTS (AKINMUSURU, 1987)

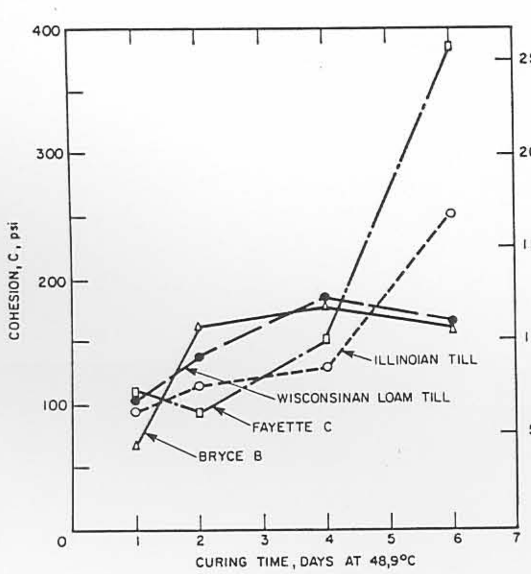


FIGURE 5.4(c)
INFLUENCE OF CURING TIME ON COHESION OF LIME-SOIL MIXTURES (THOMPSON, 1966)

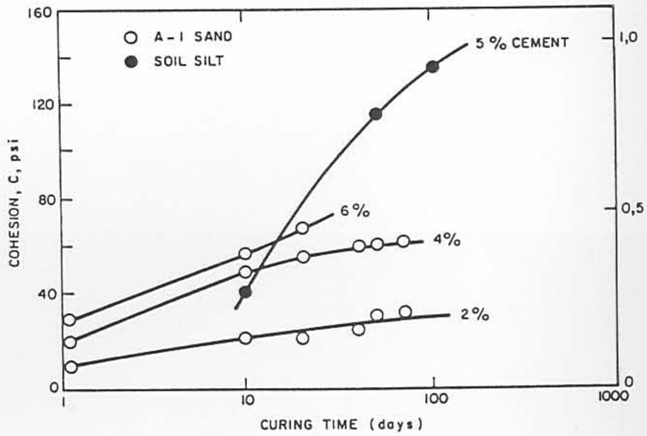


FIGURE 5.4(d)
THE EFFECT OF CURING TIME ON THE COHESION INTERCEPT FOR DIFFERERNT CEMENT TREATMENT LEVELS (Mitchell, 197

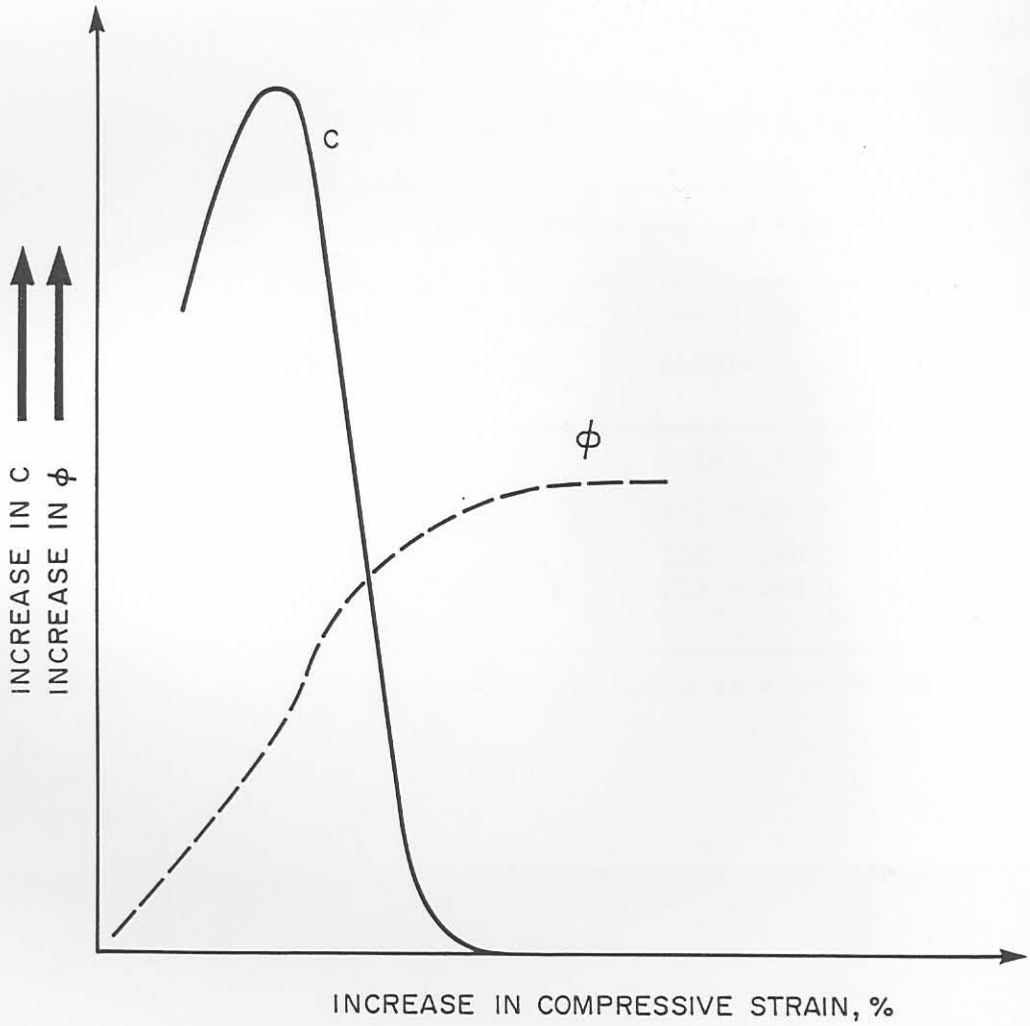


FIGURE 5.5

EFFECTIVE COHESION AND EFFECTIVE ANGLE OF INTERNAL FRICTION VERSUS COMPRESSIVE STRAIN FOR UNIFORMLY CEMENTED SAND, AS A PERCENTAGE (SAXENA et al, 1978) most probably related to micro fractures within the cementitious material. These fractures close as a result of increasing compressive strain, before the breakdown of the cementitious bonds (decrease in c) is initiated.

5.2.4 Relationships between c and Unconfined Compressive Strength (UCS)

In Figures 5.6 and 5.7, linear relationships between c and UCS are illustrated. The relationships are :

$$c = 64 + 0{,}292UCS \qquad \text{.....5.6}$$

with c and UCS in kPa,

as was found by Thompson (1966) for lime stabilised materials, and

$$c = 48 + 0{,}225UCS \qquad \text{.....5.7}$$

with c and UCS in kPa,

as was found by Mitchell (1976) for fine-grained cement stabilised soils. Applying Equation 5.6 to the current UCS limits for the cementitious categories indicated in TRH14 (NITRR, 1985), the estimated cohesion values for the different cementitious materials are indicated in Table 5.1.

TABLE 5.1 ESTIMATED COHESION (C) VALUES FOR CEMENTITIOUS MATERIALS

Cemented material	UCS-range (MPa)		Cohesion-range, c (kPa)	
	Laboratory design, 7-days at:			
	100 % Mod. AASHTO	97 % Mod. AASHTO	100 % Mod. AASHTO	97 % Mod. AASHTO
C1	6 - 12	4 - 8	1816 - 3568	1232 - 2400
C2	3 - 6	2 - 4	940 - 1816	648 - 1232
C3*	1,5 - 3,0	1 - 2	502 - 940	356 - 648
C4*	0,75 - 1,5	0,5 - 1	283 - 502	210 - 356

* These maximum strength requirements are given as a guide only.

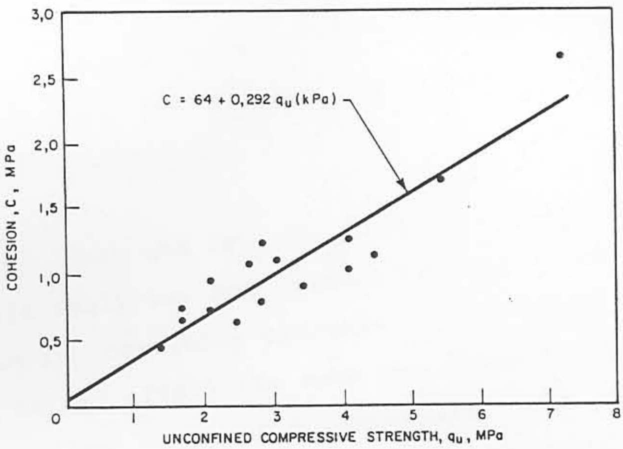


FIGURE 5.6

COHESION vs UNCONFINED COMPRESSIVE STRENGTH OF LIME-SOIL MIXTURES. (Thompson, 1966)

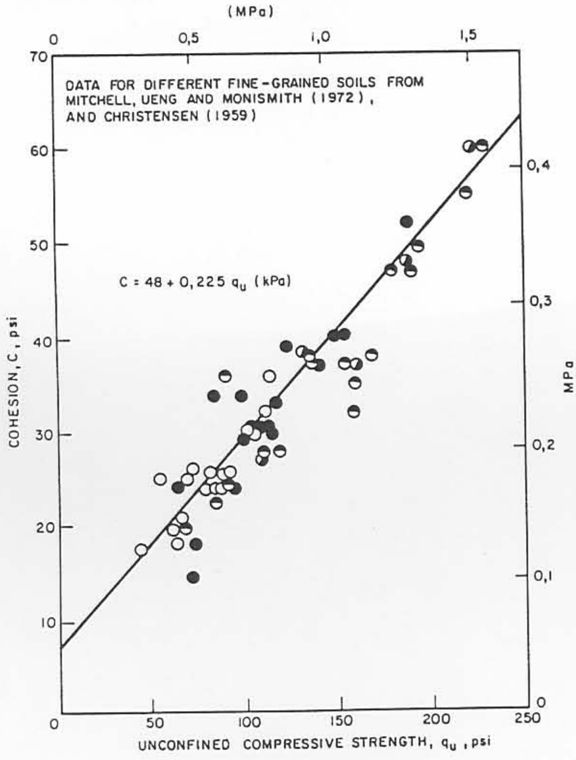


FIGURE 5.7

THE RELATION BETWEEN THE UNCONFINED COMPRESSIVE STRENGTH AND THE COHESION INTERCEPT OF FINE GRAINED SOIL AND CEMENT MIXTURES (Mitchell, 1976)

The cohesion values indicated in the table are 10 to 40 times higher than those obtained for relatively high quality crushed - stone materials (G1/G2/G3) and even more for the lower quality granular materials indicated by Maree, (1982). This is a clear indication of the beneficial effect of cementation in order to withstand shear failure in cementitious layers.

The friction angle ϕ appears to be constant and not affected by cementation, therefore the friction angle values proposed by Maree (1982) for the different crushed - stone materials should be used to calculate the safety factor against shear failure, if cementation of the material is considered.

5.3 ELASTIC PROPERTIES OF CEMENTITIOUS MATERIALS

According to Thompson (1966) the elastic properties in compression of cementitious materials may be determined using a composite or average stress - strain relationship at a given confining pressure, σ_3. See Figure 5.8.

In Figure 5.9 typical stress - strain curves for natural and lime-treated soil are illustrated. The effect of cementation is well illustrated in the figure. The figure also indicates that the percentage compressive strain at failure of the cementitious material is markedly lower than that of natural (untreated) soil, and is approximately at 1 per cent strain.

Alfi (1978) also found that the failure strain of relatively strongly cemented sand was relatively low and varied between 1 and 1,5 per cent strain with increase in confining pressure. See Figure 5.10. In a study by Robbertson et al. (1987) the same was found for cement treated materials, but for lime - treated materials, the failure strain increased with increase in lime content, approaching that of raw (natural) soil. This is an illustration of the greater flexibility of a soil - lime mixture above that of a soil - cement mixture, produced from the same raw material. See Figure 5.11.

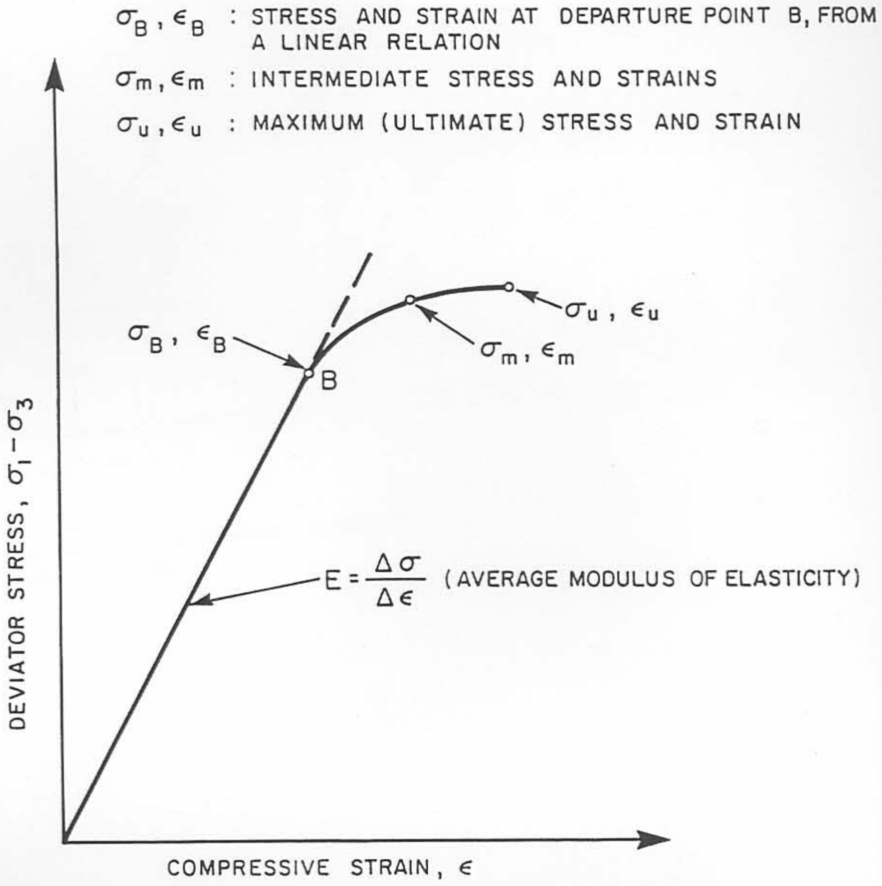


FIGURE 5.8

COMPOSITE STRESS-STRAIN CURVE OF CEMENTITIOUS MATERIALS (Thompson, 1966)

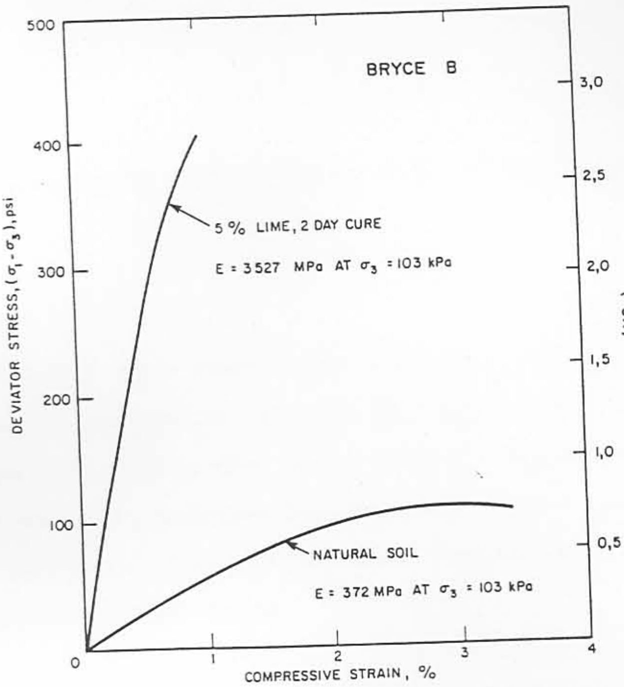


FIGURE 5.9
TYPICAL STRESS-STRAIN CURVES FOR NATURAL AND LIME TREATED SOIL (Thompson, 1966)

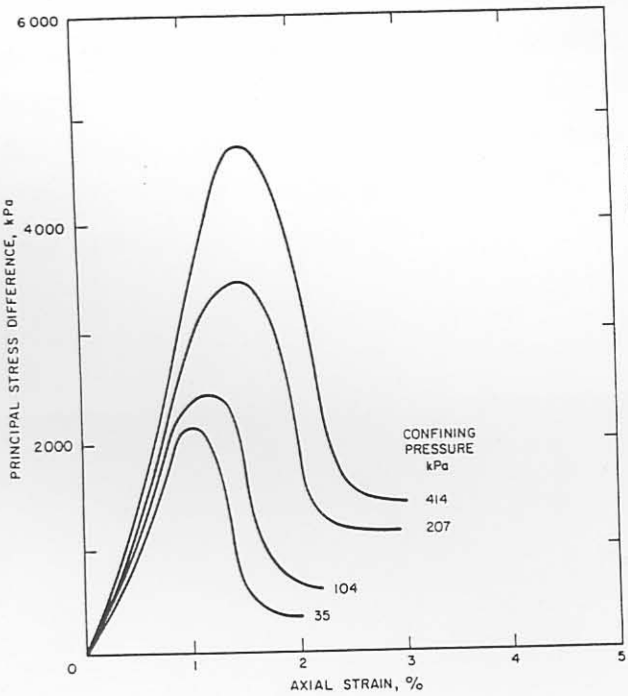


FIGURE 5.10
TYPICAL STRESS-STRAIN CURVES, OF A STRONGLY CEMENTED SAND (Alfi, 1978)

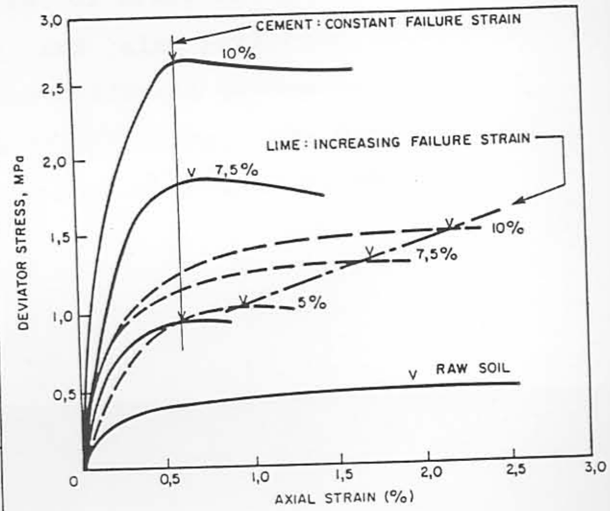


FIGURE 5.11
STRESS-STRAIN CHARACTERISTICS OF SOIL-CEMENT AND SOIL-LIME PRODUCED WITH THE SAME SOIL, ILLUSTRATING THE GREATER FLEXIBILITY OF SOIL-LIME WITH INCREASING STABILIZER CONTENT. (Robbertson, et al 1987)

A summary of the percentage failure strains obtained by several investigators for naturally uncemented, naturally cemented and artificially cemented materials is given in Appendix B, Table B.1. The table indicates that the highest percentage strains occurred for the naturally uncemented materials (1 to 12 per cent), and for naturally cemented materials 1 to 9 per cent. For artificially cementitous materials, the failure strain is approximately 1 to 2 per cent, depending on the age and the curing conditions. From the literature it is evident that the compressive failure strain of cementitious materials decreases in relation to that of untreated (unstabilised) materials. According to Thompson (1966) the average ultimate (failure) strain at confining pressure of 103 kPa for cementitious materials is approximately 1 per cent.

It is therefore my opinion that if the vertical permanent deformation (as a result of load repetitions) within a cementitious pavement layer exceeds approximately 1 per cent of the layer thickness, it may be regarded as the **initiation of compression failure** within the layer itself.

5.4 PERMANENT DEFORMATION MEASUREMENTS WITH DEPTH ON A DEEP PAVEMENT

5.4.1 General

In order to establish the number of in situ stress repetitions (Nc) needed to initiate compression failure in these pavements, the vertical permanent deformation within the layer must be measured. Nc is obtained at the point where the deformation exceeds the 1 per cent criterion stated in the previous paragraph, and indicates the **initiation of compression failure** in that layer.

With the accelerated testing (HVS) programme in South Africa, the Multi - Depth Deflectometer (MDD) was developed to measure the resilient deflection profiles at different stages of trafficking in pavements primarily. However, these instruments are also perfectly suited to measure the permanent deformation in the different pavements layers at different stages of stress repetitions (trafficking) (De Beer et al., 1988). In the following section the results of several of these tests and results are discussed.

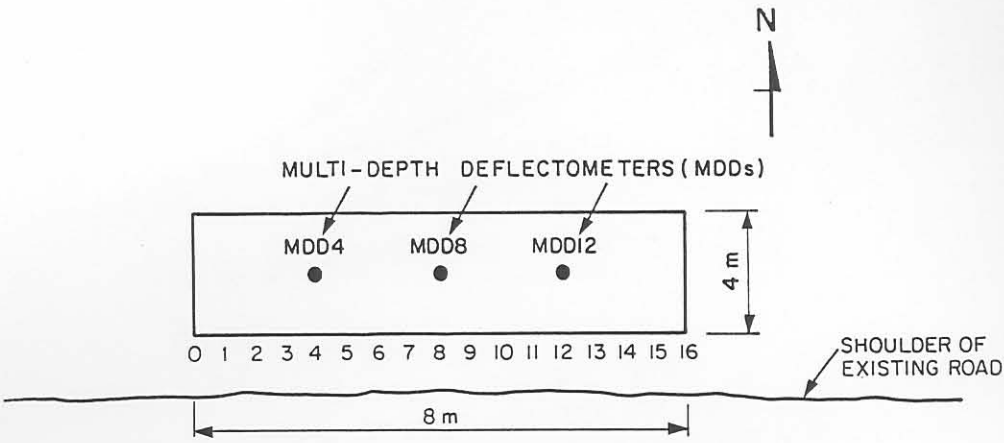
5.4.2 MDD results

The results of twenty two (22) different multi-depth deflection (MDD) measuring positions on eight (8) test sections (Sections 260A4, 274A4, 275A4, 289A4, 294A4, 309A4, 337A4 and 338A4) were used to investigate the in situ permanent deformation characteristics of a relatively deep (DCP- defined) pavement structure, viz Road 1932, near Pretoria. The structure of the pavement, HVS test layout and the different depths of the MDD modules are illustrated in Figure 5.12. The MDD modules of MDD4 and MDD8 (all sections) were placed at depths of 0, 180, 330, 480 mm , and for Sections 260A4 and 274A4 also at a depth of 630 mm in the pavement. For MDD 12, the depths were 65, 215, and 420 mm respectively.

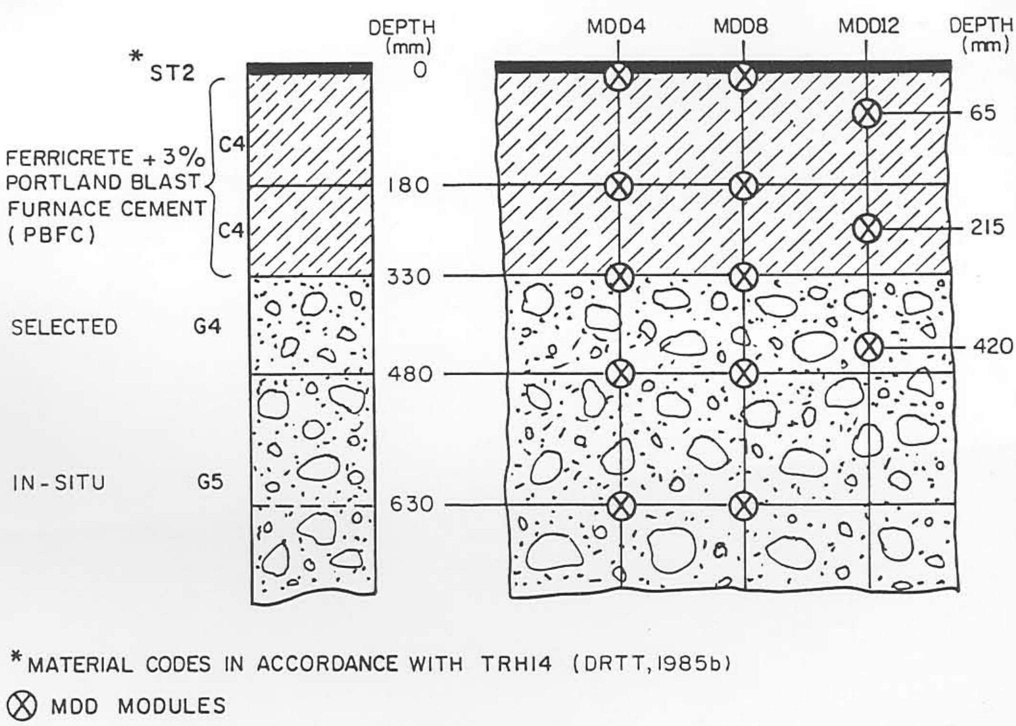
The permanent deformation on the surface of the pavement was measured with the electronic profilometer and also with a straight edge. The thicknesses of the cementitious base and subbase layer were approximately 180 mm and 150 mm respectively.

Based on the **compressive failure criterion of 1 per cent compressive failure strain**, the number of load repetitions (Nc) needed to initiate compressive failure within the cementitious layer is obtained by studying the MDD measured permanent deformation results. The principle of this method is illustrated in Figure 5.13. For the base layer, initiation of compression failure was assumed to occur when the permanent deformation exceeded 1,8 mm (1 per cent of 180 mm), and for the subbase 1,5 mm (1 per cent of 150 mm). It is also important to note that fatigue failure of the 13 mm double seal (surface treatment) on top of the cementitious base layer was observed soon after Nc was reached during HVS testing. This was found on all the sections tested and is possibly related to the loss of support because of compression failure in the upper section of the the cementitious base. After extensive fatigue failure, the seal came loose from the base and was also crushed and granulated, even during the dry state of HVS testing.

Figure 5.14 shows the typical manifestation of compression failure in the base of this road. Typical permanent deformation results at different depths measured with the MDDs on HVS test Sections 274A4, are illustrated in Figure 5.15 (a) and (b). Also indicated on the figures are the different Nc - values. Deformation results at different depths



(a) Typical layout of an HVS test section



(b) Pavement structure and MDD layout

FIGURE 5.12

LAYOUT OF HVS TEST SECTIONS FOR PERMANENT DEFORMATION MEASUREMENTS ON ROAD 1932 (ROOIWAL)

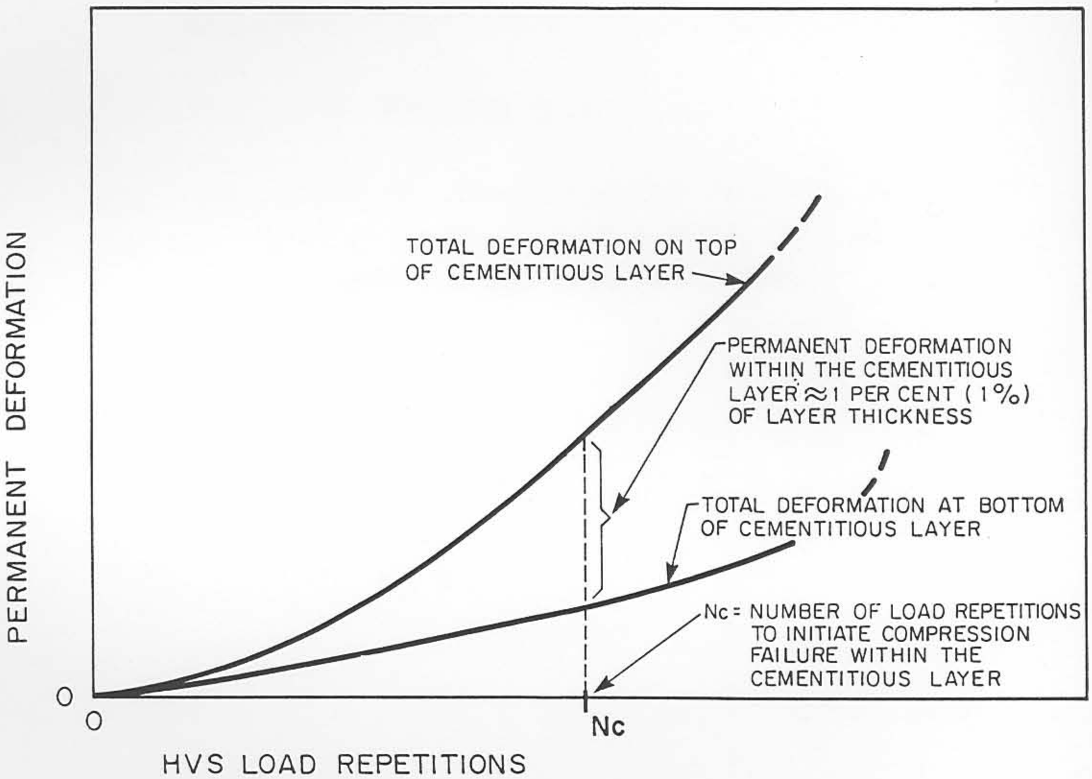


FIGURE 5.13

DETERMINATION OF THE NUMBER OF HVS LOAD REPETITIONS TO INITIATE COMPRESSION FAILURE WITHIN LIGHTLY CEMENTITIOUS LAYERS

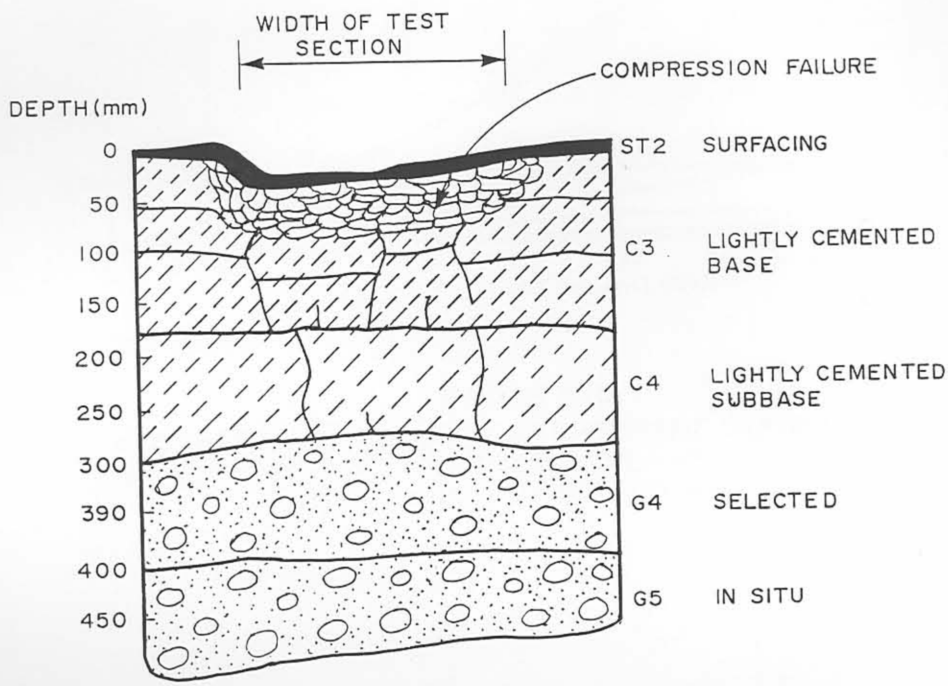
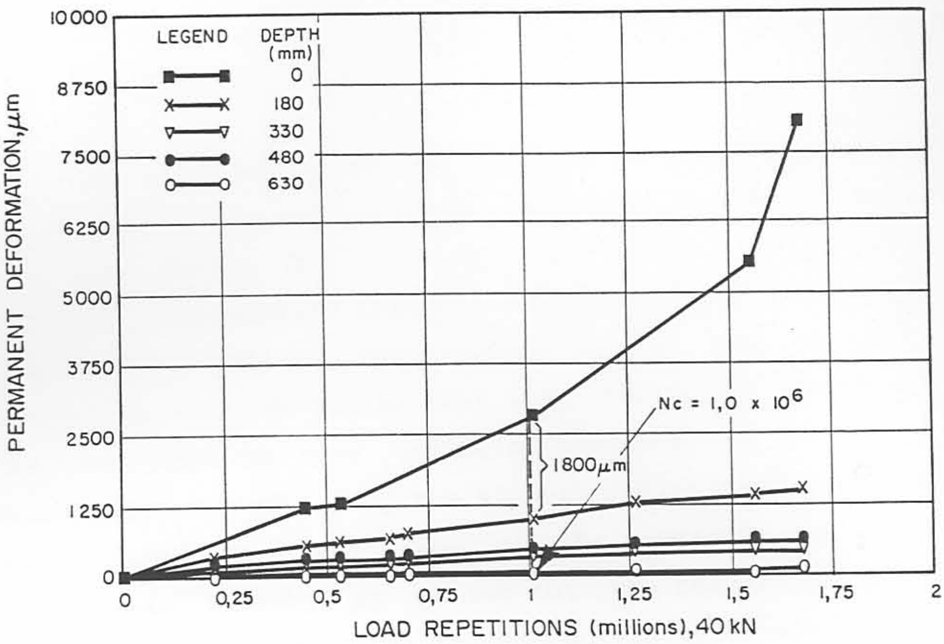
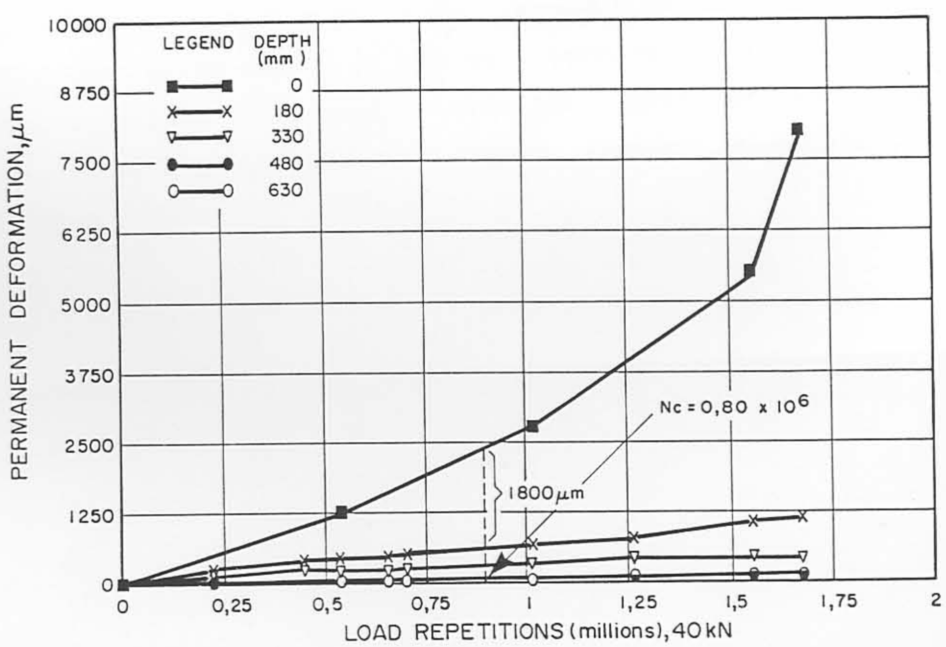


FIGURE 5.14

MANIFESTATION OF COMPRESSION FAILURE IN THE LIGHTLY CEMENTITIOUS BASE LAYER OF A DEEP PAVEMENT (ROAD 1932, ROOIWAL)



(a) MDD4



(b) MDD8

FIGURE 5.15

PERMANENT DEFORMATION AT DIFFERENT DEPTHS AT VARIOUS STAGES OF TRAFFICKING ON HVS TEST SECTION 274A4 (ROAD 1932, ROOIWAL) for other test sections are given in Appendix B, Figures B.1, B.2 and B.3. These figures clearly indicate that most of the permanent deformation occurred within the cementitious base and subbase layers. Close inspection of the top 65 mm of the cementitious base layer and in some cases, the cementitious subbase, revealed that visible compression failure (crushing) did indeed occur, and contributed largely (more than 80 per cent) towards the total permanent deformation (rutting) on the pavement towards the end of the HVS tests. The crushing process, however, is an ongoing failure of the cementitious layers, ie: when Nc for 180 mm is reached, crushing occurs thus reduces the effective layer thickness. A new Nc for the layer with reduced thickness causes a further crushing so that eventually the top of the layer is cracked (crushed) far more than the bottom of the layer as shown in Figure 5.14. As a result of complete crushing and subsequent pumping (moisture accelerated distress, MAD) in the top of the base, the crushed material was completely removed from the test section resulting in extensive potholing. This is an indication of the importance of establishing Nc, for both initial design and maintenance planning on these type of pavements.

5.4.3 Nc - values

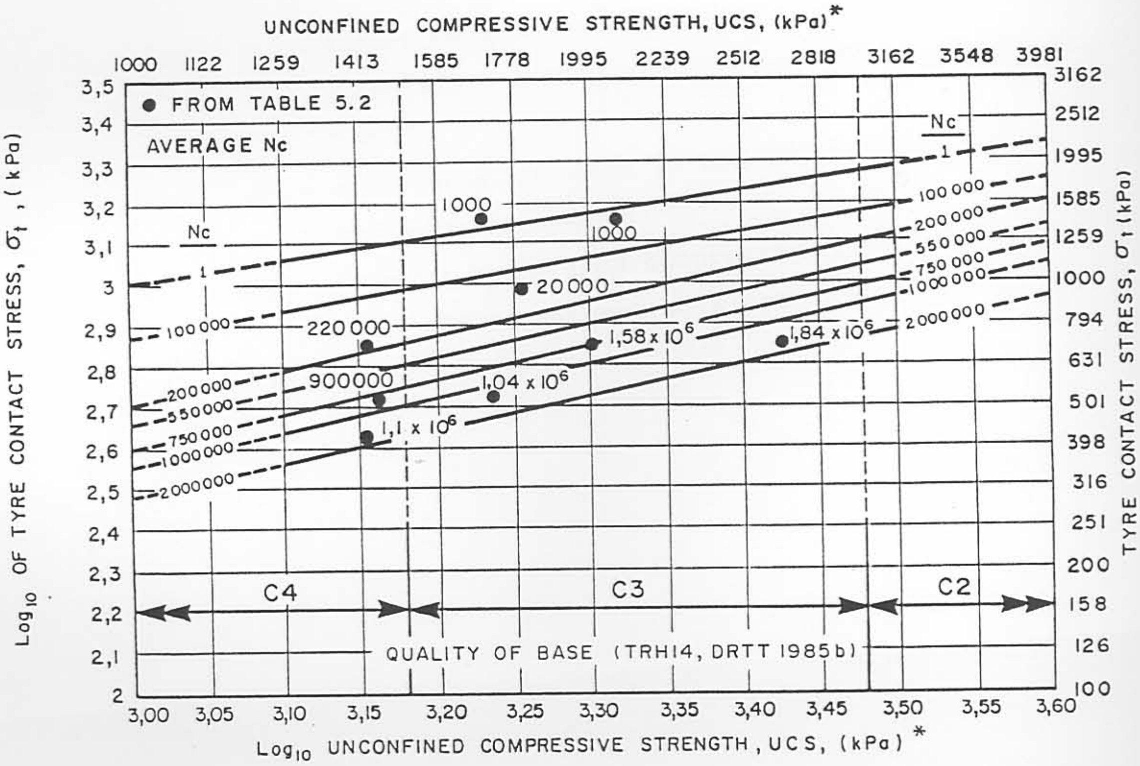
Table 5.2 gives a summary of the Nc - values and associated data for the different test sections and measuring positions together with the tyre contact stress during HVS testing. The table indicates that Nc (initiation of compression failure) varies from 500 repetitions to approximately 2,14 million repetitions, whilst the vertical cyclic stress varies between 420 kPa to 1445 kPa. (Normally, the tyre contact stress is lower than the tyre inflation pressure, but in this case the conservative approach of using the higher inflation pressure equal to the contact stress on the surface of the pavement was used in the analysis). It is, however, my opinion that the results in the table may assist in establishing practical design curves to obtain the number of load (stress) repetitions needed to initiate compression failure (Nc) in lightly cementitious layers. To accomplish this, the following assumption has to be made: if the tyre contact stress σ_t approaches the in situ Unconfined Compressive Strength (UCS) of the cementitious material, compression failure starts almost immediately in the cementitious layer, i.e. at Nc = 1.

TABLE 5.2 NUMBER OF HVS REPETITIONS NEEDED TO INITIATE COMPRESSION FAILURE (Nc) ON THE DIFFERENT SECTIONS TESTED

Test-Section	MDD-position	Contact stress,* σ_t (kPa)	UCS** (kPa)	Repetitions, Nc, (million) At MDDs	AVERAGE	Stress Ratio σ_t/UCS
260A4	4	700	1720	1,123		0,407
	8	700	1720	1,270		0,407
	12	700	1720	0,725	1,04	0,407
274A4	4	520	1454	1,000		0,358
	8	520	1454	0,800	0,90	0,358
275A4	4	700	2007	1,580		0,349
	8	700	2007	1,650		0,349
	12	700	2007	1,500	1,58	0,349
289A4	4	700	1433	0,280		0,488
	8	700	1433	0,080		0,488
	12	700	1433	0,300	0,22	0,488
294A4	4	700	2671	1,990		0,262
	8	700	2671	1,380		0,262
	12	700	2671	2,140	1,84	0,262
309A4	6	1445	2115	0,001		0,683
	10	1445	2115	0,001	0,001	0,683
337A4	4	1445	1706	0,0005		0,847
	8	1445	1706	0,0015		0,847
	12	1445	1706	0,001	0,001	0,847
338A4	4	960	1803	0,022		0,532
	8	960	1803	0,021		0,532
	12	960	1803	0,017	0,02	0,532
345A4	4,8,12	420	1445	1,100	1,10	0,291

* Tyre Contact stress assumed = tyre inflation pressure
** UCS: DCP-derived (Kleyn, 1984), in situ Unconfined Compressive Strength of the upper 50 mm of the cementitious gravel base

From this assumption, "additional" data is obtained at Nc = 1, which can be used with the data in Table 5.2, to aid in the development of the design curves. The in situ UCS of the upper 50 mm of the cementitious base was determined with the DCP (Kleyn, 1984) at start of testing on each section, and is also indicated in Table 5.2. The results from Table 5.2 are illustrated in Figure 5.16.



* DCP – DERIVED IN SITU UCS (KLEYN, 1984)

FIGURE 5.16

SUGGESTED RELATIONSHIPS BETWEEN TYRE CONTACT STRESS (σ_t), IN SITU DCP-DERIVED UNCONFINED COMPRESSIVE STRENGTH (UCS) AND NUMBER OF STRESS REPETITIONS (Nc) TO INITIATE COMPRESSION FAILURE (CRUSHING) IN THE TOP 50 mm OF LIGHTLY CEMENTITIOUS BASES. THE HVS RESULTS ARE ALSO INDICATED

Too little data, however, exist to perform any regression analysis between these two variables in Figure 5.16, therefore the curves in are hand-drawn to indicate only possible relationships.

The figure also demarcates the strength boundaries for the different quality of cementitious gravel materials (DRTT,1985a, 1985b) as summarised in Table 5.1. The curves are only valid for C3-materials and to a lesser extent for C2- and C4- materials. The suggested curves, without the HVS data, are illustrated in Figure 5.17 and are proposed for use in determining the Nc of pavements with lightly cementitious gravel bases, if the in situ UCS (upper 50 mm of the cementitious gravel base) and the tyre contact stress, σ_t, are known.

Generally, the higher the tyre contact stress on a specific pavement, the lower Nc, and vice versa. For example, if the UCS is 1995 kPa and σ_t is 630 kPa, Nc is approximately one (1) million (See Figure 5.17).

The information in Figure 5.17 may be presented using a single compression failure curve by normalising, as depicted in Figure 5.18. This curve is similar to the better known fatigue curves of cementitious materials (De Beer, 1989) and concrete. In Figure 5.18, the ratio between tyre contact stress, σ_t, and in situ UCS (σ_t/UCS), is plotted against the measured number of stress repetitions (N_c).

The compression failure curve is also described below:

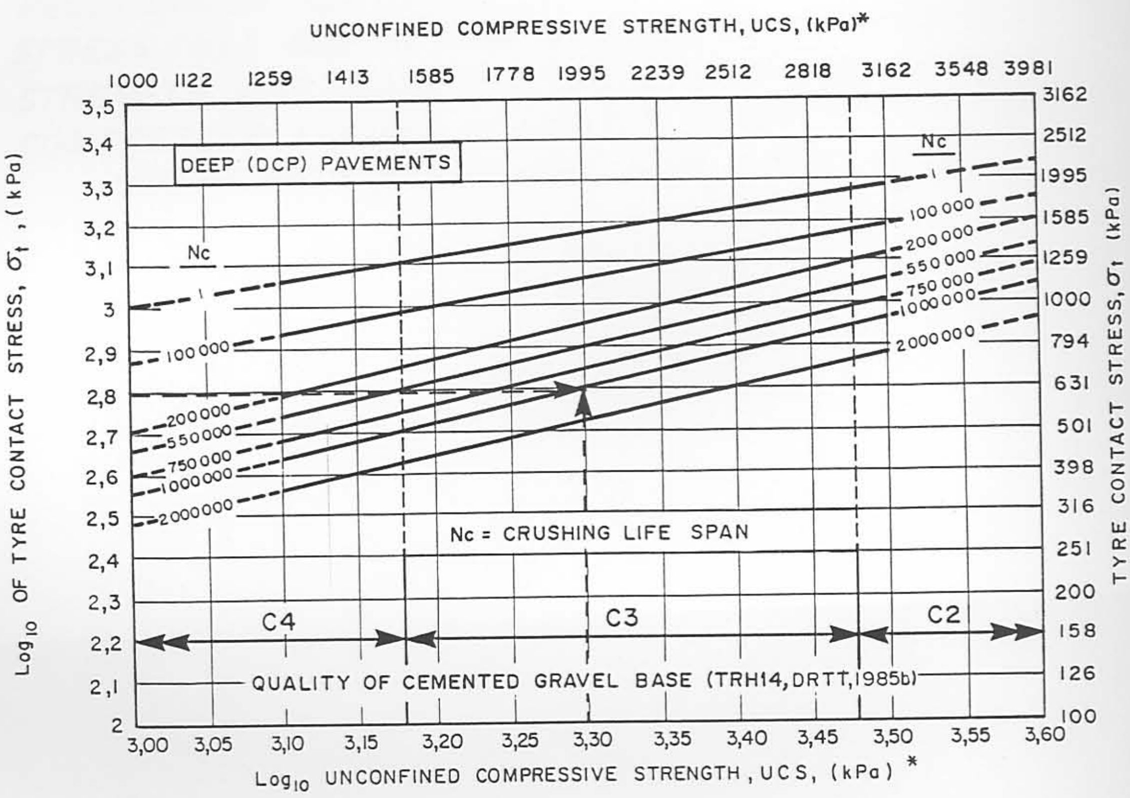
$$N_c = 10^{8,21(1 - \sigma t/(1,2UCS))} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.8$$

with N_c = stress repetitions to initiate compression failure in weakly cemented materials (UCS: 0,75 MPa to 3 MPa)

σ_t = tyre contact stress in kPa (σ_t assumed = tyre inflation pressure)

UCS = in situ unconfined compressive strength derived from DCP measurements (Kleyn, 1984).

R^2 = 0,89 and n= 22



* DCP – DERIVED IN SITU UCS OF TOP 50 mm OF CEMENTITIOUS GRAVEL BASE

FIGURE 5.17

SUGGESTED RELATIONSHIPS BETWEEN TYRE CONTACT STRESS(σ_t) AND UNCONFINED COMPRESSIVE STRENGTH (UCS) TO DETERMINE THE STRESS REPETITIONS (Nc) TO INITIATE COMPRESSION FAILURE IN LIGHTLY CEMENTITIOUS BASE

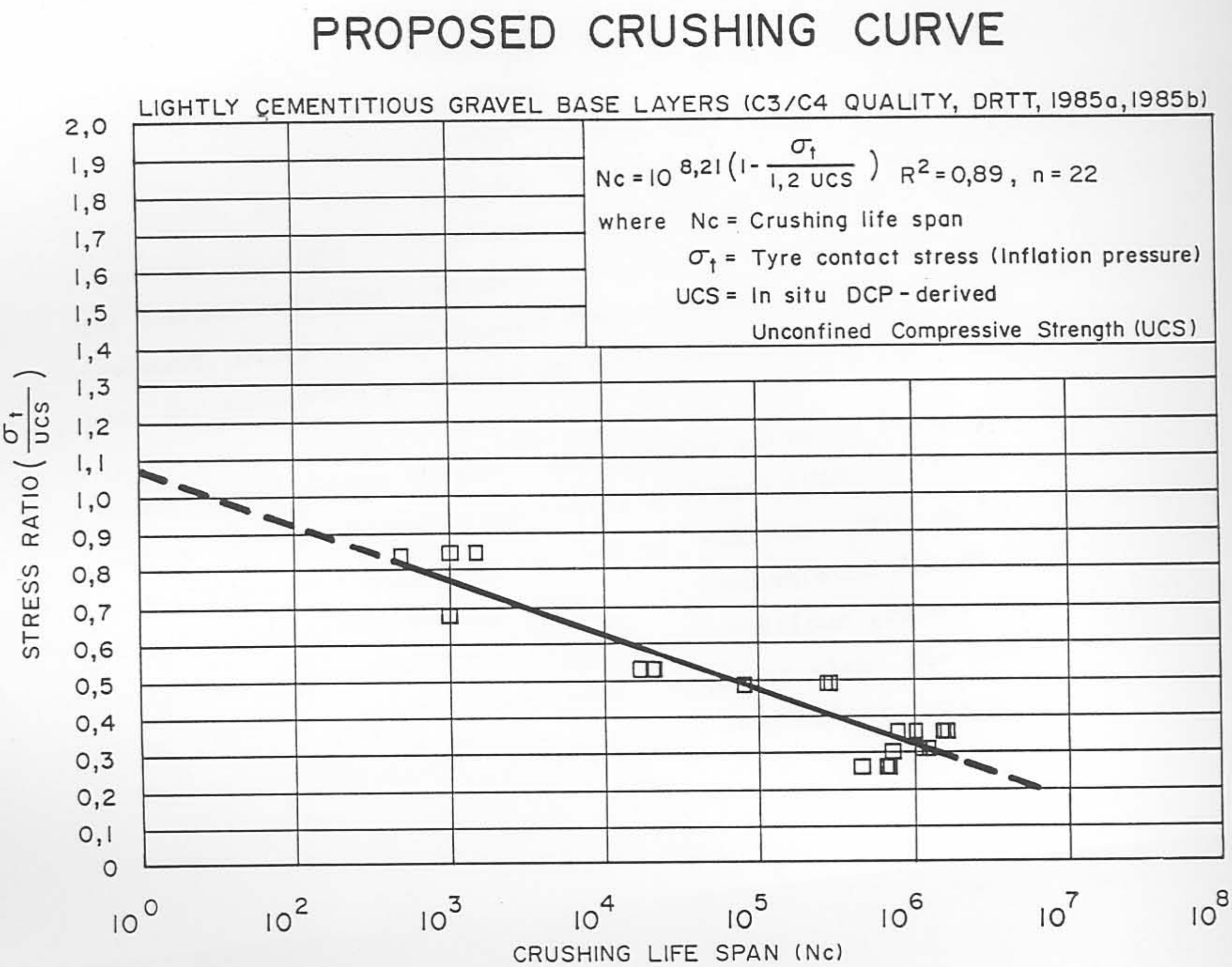


FIGURE 5.18

RELATIONSHIP BETWEEN THE RATIO OF TYRE CONTACT STRESS (σ_t) AND IN SITU UNCONFINED COMPRESSIVE STRENGTH AND NUMBER OF REPETITIONS TO INITIATE COMPRESSION FAILURE IN LIGHTLY CEMENTITIOUS BASE LAYERS WITH RELATIVELY THIN SURFACINGS

This compression failure relationship is proposed as a <u>tentative design curve</u> to obtain estimates (50 % probability) be used in new designs, rehabilitation and/or during maintenance planning on pavements with lightly cementitious gravel base and/or subbase layers. Future research, however, should be done to establish an improved curve with more data on a wider range of cementitious materials.

5.5 SUMMARY AND CONCLUSIONS

In this chapter, a summary of the strength parameters of lightly cementitious materials, namely, cohesion c, and angle of internal friction ϕ is given. A description is also given of compression failure of lightly cementitious layers under repetitive loading (stress) conditions. The strength of cementitious materials is described mainly by the cohesion, which increases as a result of the type of raw material, stabiliser content, curing time and compaction density. The c and ϕ can be determined using the Mohr-Coulomb theory, or using p_f-q_f diagrams.

Compression failure (crushing) of lightly cementitious layers is a reality and must be taken into account during design and maintenance planning of these types of pavements. During compression failure, the cohesion (cemented strength) of the material decreases as a result of compressive axial strains in excess of 1 per cent. These compression failures are measured as permanent deformation in the cementitious layers with the MDD equipment (De Beer et al., 1988b). If the vertical permanent deformation exceeds 1 per cent of the layer thickness, compression failure is considered to have started (initiation of crushing) within the cementitious layer. Soon after compression failure, fatigue failure of the asphalt surface treatment occurs. If moisture enters the cracked pavement, moisture-accelerated distress (MAD) occurs, resulting in loss of seal and excessive potholing on this type of pavement.

An empirical design curve, relating the ratio of tyre contact stress σ_t and in situ unconfined compressive strength (UCS) to the number of stress repetitions to initiate compression failure (Nc), is proposed (Figure 5.18). This curve can be used in the mechanistic design, and maintenance planning of pavements with lightly cementitious gravel layers, to estimate the life of surface treatments, together with the number of expected stress repetitions needed to initiate compression failure.

Preventive maintenance by timeous sealing, as well as by controlling the tyre contact pressure of heavy vehicles or provision of a stress distribution layer, for example, G1/G2, may largely increase the life of pavements with lightly cementitious gravel layers.

It is also important to note that this information indicates that lightly cementitious gravel base materials must be of **adequate strength (UCS)** to increase the crushing life span (Nc), especially if relatively high tyre contact stresses are expected.

Although the compression failure results discussed here were strictly derived from the cementitious base layers, it is my opinion that cementitious sub-layers in the pavement system may also be evaluated for this failure, using Equation 5.8, substituting the tyre contact stress (σt) by the calculated vertical stress on top of the cementitious layer under consideration. Further research, however, is necessary to verify this assumption.

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