CHAPTER 4

PERMANENT DEFORMATION BEHAVIOUR OF PAVEMENTS WITH LIGHTLY CEMENTITIOUS LAYERS

CONTENTS

4.1 INTRODUCTION

More than 80 percent of the pavement structures in the Province of the Transvaal incorporate lightly cementitious gravel base and/or subbase layers. This type of material is also often used by the other Provinces, as well as some of the independent and self governing states in southern Africa, because this material provides a relatively economical way of improving the structural strength (bearing capacity) of marginal granular materials. This type of material is also used on a large scale as subbase layers for bituminous base and granular base pavements in South Africa today (De Beer, 1985; Freeme et al, 1982; Freeme et al, 1984; Van Zyl et al, 1983; Otte, 1972, 1978).

Following a major study, undertaken by the Division for Roads and Transport Technology (DRTT) and the Directorate Roads Branch of the Transvaal Provincial Administration (TPA), on the behavioural characteristics of pavements with granular layers, the TPA requested the DRTT to investigate the structural characteristics of pavements with lightly cementitious gravel layers. The previous study on the granular materials was completed during the period 1978 to 1981, and is reported elsewhere (Maree, 1982; Kleyn et al, 1985).

This chapter discusses the permanent deformation characteristics of the series of Heavy Vehicle Simulator (HVS) tests on relatively deep - and relatively shallow pavements. The terms "deep" and "shallow" refers to the DCP-classification system described in Chapter 3. The Provincial numbers of these pavements are 1932 and 2212, respectively, and are situated north of Pretoria near the Rooiwal Power Station. Hereafter, they shall be referred to as Road 1932 (Rooiwal) for the relatively deep pavement and Road 2212 (Bultfontein) for the relatively shallow pavement.

In the discussion of the behaviour of these pavements, the following characteristics are addressed:

- Total permanent deformation on the surface (rutting) of the HVS test sections;

- permanent deformation at different depths in the pavement;
- rate of deformation in both relatively dry and relatively wet conditions and
- failure mechanisms occurring under accelerated testing.

4.2 BEHAVIOUR OA A DEEP PAVEMENT: ROAD 1932 (ROOIWAL)

In this section the emphasis will be on the permanent deformation characteristics of the deep pavement, as quantified under accelerated testing by the TPA - HVS.

4.2.1 Pavement structure and location of HVS - test sections

The layout of the various HVS test sections on this road is illustrated in Figure 4.1(a). The pavement sections were selected with the aid of the DCP, and according to these results a relatively deep pavement section was identified. (See Chapters 3 and 6 for detail DCP-classification and DCP-analysis of this pavement). The test wheel load as well as the tyre pressure used during HVS testing are also indicated on the figure and will be discussed in more detail in Paragraph 4.2.3.

In Figure 4.1(b), the general in situ pavement structure is illustrated. The figure indicates that the base and subbase were of C3-quality (TRH4, DRTT, 1985), stabilised with 3 percent Portland blastfurnace cement (PBFC). The parent material for both base and subbase layers, including the selected (G4) and in situ (G5) layers, is of a relatively good quality ferricrete. Stabilisation of the base and subbase however was necessary to reduce the plasticity and to improve the bearing capacity of these layers. According to the as-built records of TPA (Pienaar, 1979), the in situ material was non - plastic, with a grading modulus of approximately 2, and a CBR of approximately 45 percent at 100 percent modified AASHTO density.

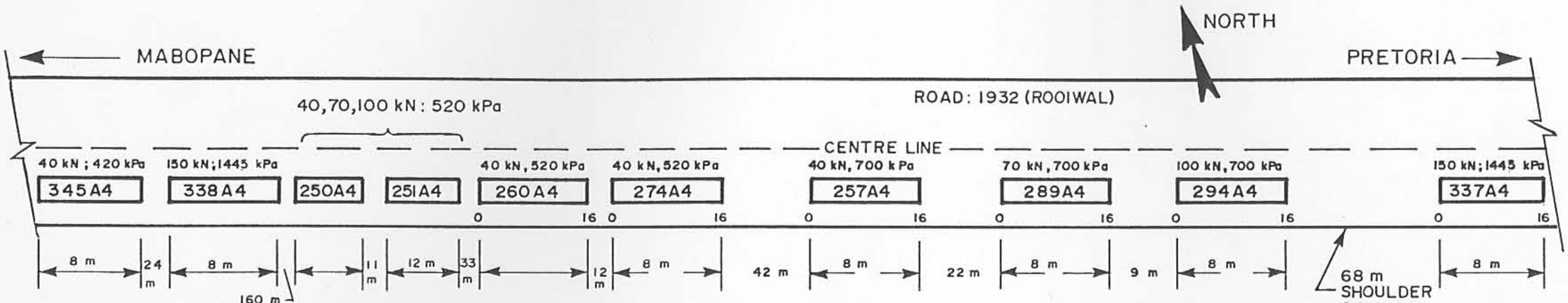


FIGURE 4.1 (a) *LAYOUT OF THE VARIOUS HVS TEST SECTIONS ON ROAD 1932 (ROOIWAL)*

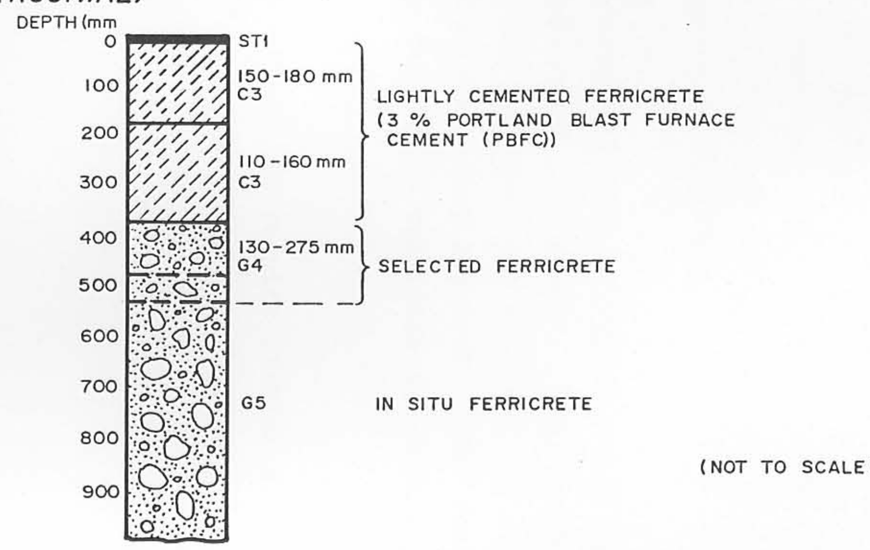


FIGURE 4.1 (b) *DETAIL OF THE RELATIVELY DEEP PAVEMENT STRUCTURE AT THE HVS SITE (km 2,9) ON ROAD 1932, NEAR ROOIWAL*

4.2.2 Original design considerations

The design was a typical TPA Class III pavement, designed to carry 0,4 to 1 million E80s (Transvaal Roads Department, 1978, Kleyn, 1988). Owing to the relatively good in situ material as well as the use of an above - average quality of material for the selected layers, a relatively deep pavement resulted (Kleyn et al., 1985).

According to the DCP structural capacity model, originally developed by Kleyn (1984) for relatively light gravel base and subbase pavements (Model 3 in Chapter 6), the structural capacity of this road is in excess of 10 million E80s. However, it must be noted that this model is not calibrated for pavements incorporating cementitious layers, and therefore the prediction of the structural capacity of such pavements is overestimated, and must be viewed critically especially if DCP-defined poorly-balanced or inverted pavements (Chapter 3, De Beer et al., 1988) are analysed. The reason for this is that the current DCP model for structural capacity is strictly a function of the DSN_{800}, which is the total number of blows taken to penetrate 800 mm of the pavement structure. It is my opinion that the model must be further calibrated and improved to compensate for structural imbalance as well as for inverted pavement structures, as a high DSN_{800} is not necessary an indication of a high structural capacity. A correction factor, however, is already introduced to compensate for in situ moisture content. Detailed analysis, however, of this model as well as the development of alternative models are discussed in Chapter 6.

In Appendix A, Table A.1, a summary of the design data for the base and subbase material of this pavement is given. The table indicates relatively good parent material for both the base and the subbase, but in order to achieve a relatively deep pavement with less erodible (pumpable) materials, it was decided to stabilise the materials for both layers.

According to the initial laboratory UCS-results (Tables A.1 and A.2, in Appendix A), a minimum of 3 per cent PBFC was necessary to provide a C3-quality material (TRH13, DRTT, 1986).

The pavement was constructed during 1978 and was approximately 7 years old at the start of HVS testing in 1985.

4.2.3 HVS test programmes on Road 1932 (Rooiwal)

A summary of the HVS test programmes followed on Road 1932 (Rooiwal) is given in Appendix A, Table A.3. On this pavement a total of 12 different HVS tests were done over a test period of approximately three years. During this period a total of 150 million E80s (r=4 in $(P/40)^r$) were applied to this road. The dual wheel load (P) varied between 40 kN and 100 kN, and tyre pressures between 420 kPa and 700 kPa. Two tests, however, were done at excessively high single wheel loadings, using an aircraft wheel (Boeing 747). The single wheel load for the two tests was 150 kN, with tyre pressures varying between 960 kPa and 1445 kPa (see Table A.3, HVS-Sections 337A4 and 338A4, in Appendix A). During most of the test periods, in situ moisture (relatively dry and natural rain) temperature conditions prevailed. Moisture was also introduced artificially on the surface as well as in depth of the pavement at selected intervals.

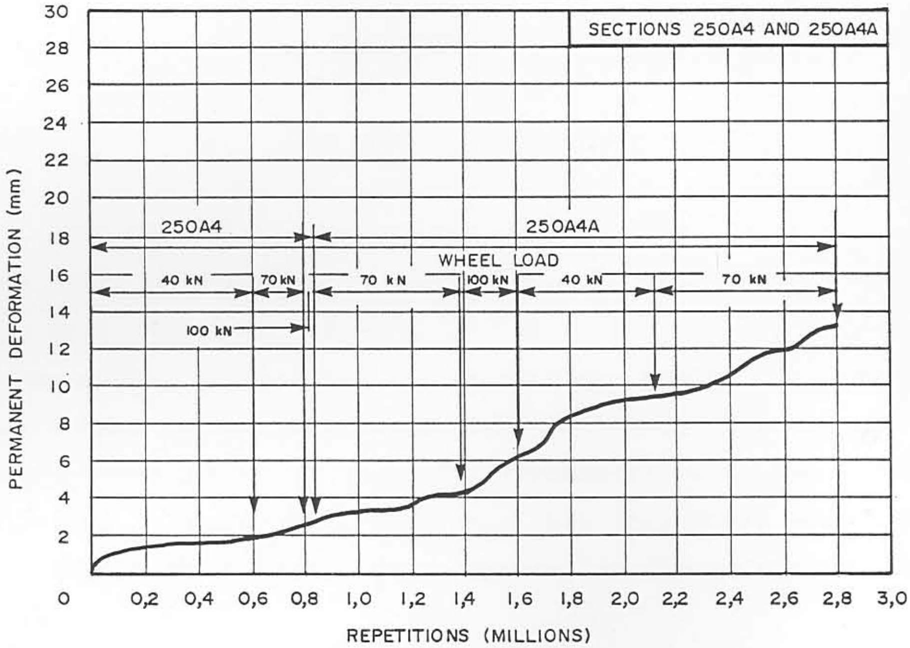
4.2.4 Permanent deformation behaviour and failure mechanism

The permanent deformation (plastic deformation, rutting), as a direct result of traffic loading, measured on the surface of the pavement is normally used to describe the state and behaviour of the pavement. This is also one of the primary indicators of the behaviour of HVS test sections under accelerated testing.

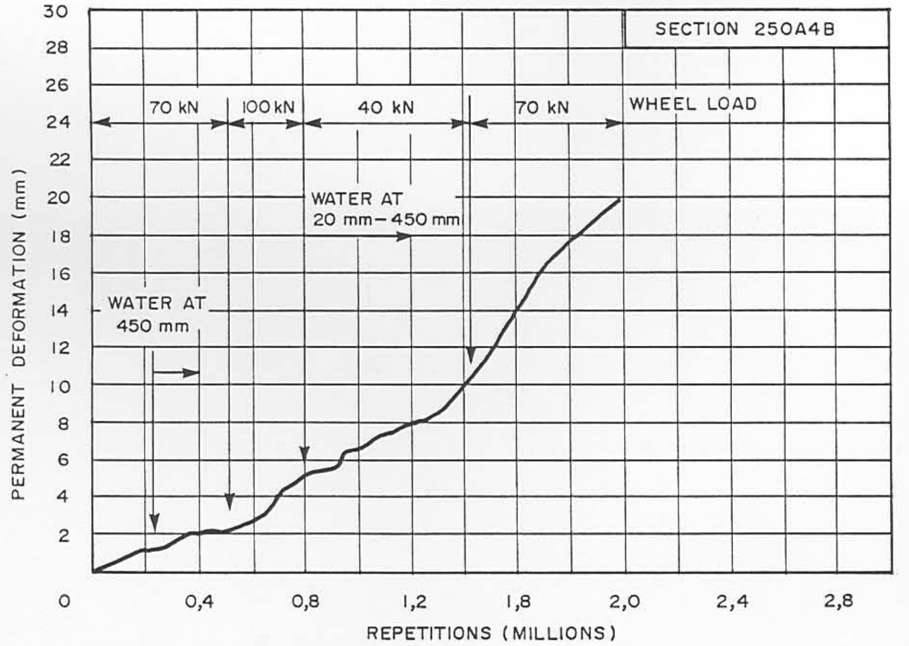
Detailed analysis into the prime reasons for the development and origin of the permanent deformation, facilitates the accurate identification of the failure mechanism of the pavement, and is therefore of utmost importance to be done effectively.

4.2.4.1 Permanent deformation on the surface (rutting)

Typical permanent deformation behaviour, as measured under a 3 m straight edge is illustrated in Figure 4.2(a) and (b). The figures indicate the average maximum permanent deformation development on the



(a) HVS SECTIONS 250A4 AND 250A4A



(b) HVS SECTION 250A4B

FIGURE 4.2

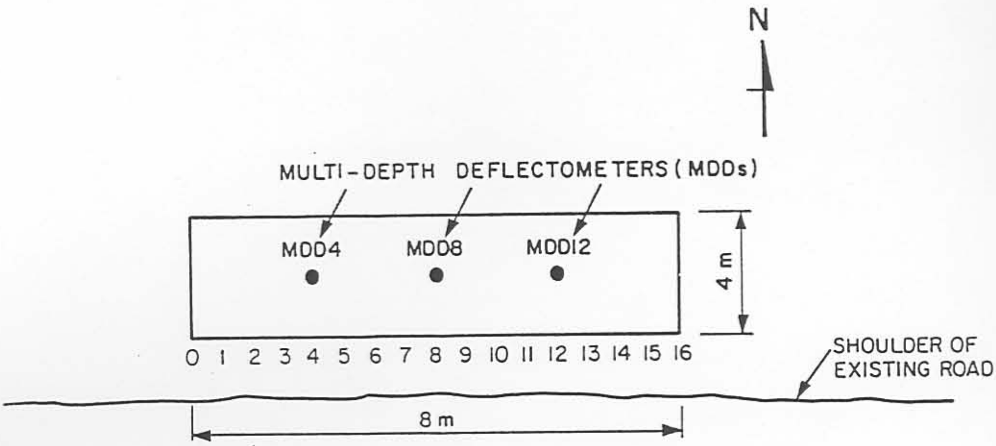
AVERAGE PERMANENT DEFORMATION AS MEASURED IN HVS TESTS 250A4, 250A4A AND 250A4B UNDER INDICATED DUAL WHEEL LOADS ON ROAD 1932 (ROOIWAL) surface of the test sections after various stages of trafficking, loading and under certain moisture conditions. In general, the figures indicate that the rate of deformation (per wheel load) is linear for all practical purposes, and increase with the introduction of surface water especially (mostly rain) on the sections. The permanent deformation results of the rest of the HVS test sections are illustrated in Appendix A, Figures A.1, A.2 and A.3.

Excessive potholing, however, resulted on Sections 251A4, 275A4 and 294A4 (See Figures A.1(a)), A.2(b)) and A.3(b), in Appendix A). On the two latter sections, it was found that the top 50 mm to 75 mm of the cemented gravel base was pumped out, resulting in potholes, while the rest of the base and the subbase, although fatigued, was structurally sound. According to the DCP-results, before and after HVS testing, the top 50 mm to 75 mm of the base material had decreased in strength, indicating an effective weakening in the cemented material. This crushing was primarily responsible for the loss of support to the asphalt surfacing layer (13 mm/6 mm double seal in this case). These HVS tests, again, demonstrate the destructive effect of water under pressure (excessive porewater pressure state, EPWP) in this weak upper layer, which caused the ultimate failure (potholing) of this pavement.

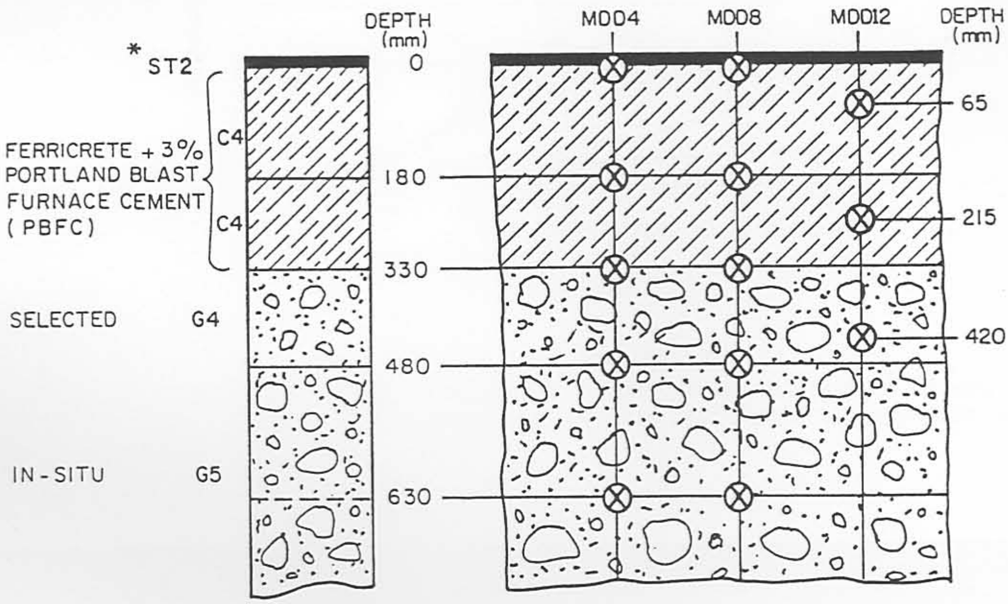
4.2.4.2 Permanent deformation at different depths

On four of the HVS test sections, viz Sections 260A4, 274A4, 275A4 and 294A4, permanent deformation at different depths in the pavement were measured using the Multi-depth deflectometer (MDD). The MDD is normally used to measure the depth deflections in accelerated testing of pavements in South Africa (De Beer et al., 1988), but are also used to measure permanent deformation at different depths by studying the permanent change in the vertical position of these instruments (LVDTs). In Figure 4.3 the layout of the MDD instrumentation in the test sections is illustrated. The figure indicates that the MDDs were located at different depths in the pavement structure, mostly at the various layer interfaces.

In Figures 4.4 (a), (b) and (c), typical permanent deformations at different depths and at various stages of trafficking are illustrated.



(a) Typical layout of an HVS test section



*MATERIAL CODES IN ACCORDANCE WITH TRH14 (NITRR, 1985)

⊗ MDD MODULES

(b) Pavement structure and MDD layout

FIGURE 4.3

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

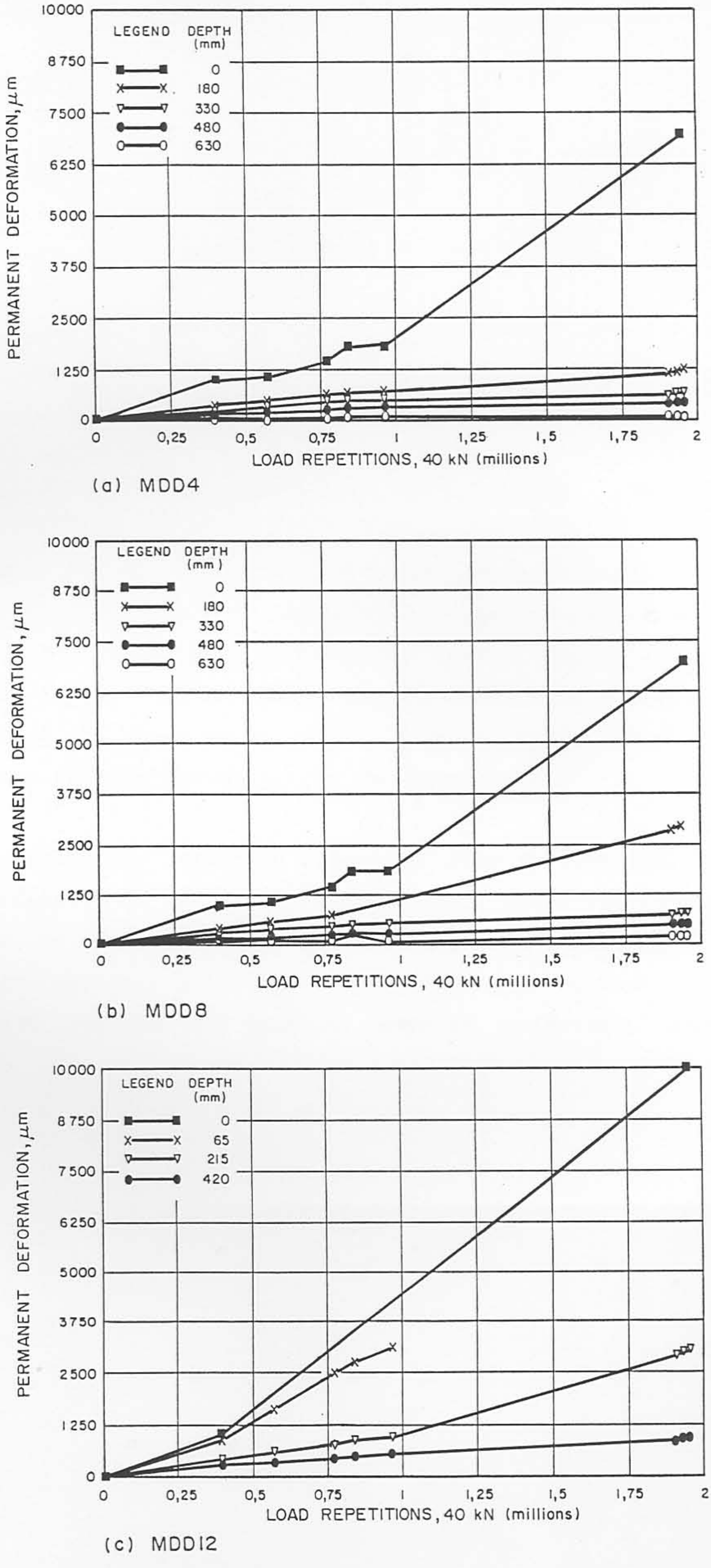


FIGURE 4.4

PERMANENT DEFORMATION AT DIFFERENT DEPTHS AT VARIOUS STAGES OF TRAFFICKING ON HVS TEST SECTION 260A4 (ROAD 1932, ROOIWAL)

For the rest of the test section these permanent deformations are illustrated in Appendix A, Figures A.4, A.5, A.6. The figures indicate that most of the deformation occurred between 0 mm and 180 mm, that is, in the cemented gravel base layer, and increase towards the end of the tests.

In Table 4.1, a summary of the average percentage permanent deformations in the different layers, at the end of the various tests, are given. The table indicates that approximately 67 per cent of the permanent deformation in the relatively dry state and approximately 75 per cent of the permanent deformation in the relatively wet state, occurred in the base layer only. This is indicative of the crushing failure mechanism in the base layer, discussed in the following section.

4.2.4.3 **Crushing failure mechanism**

Test pits made after HVS testing on all these test sections confirmed that a crushed zone of approximately 50 mm to 75 mm developed directly under the seal in the top of the base layer, as a result of trafficking.

Back - analysis of the effective elastic moduli of the base layer, using linear elastic analysis (based on measured depth deflections, (MDD)), indicates a marked reduction in effective elastic moduli of the base layer, thereby indicating a weakening, ie crushing in this case, of the layer (see Chapter 6, as well as De Beer, 1986a, 1986b, 1986c, 1986d).

It must, however, be emphasised that this failure mechanism is strictly associated with relatively deep pavements because the deeper strength of the pavement structure provides relatively strong support to the upper layers in the system, thereby increasing the repetitive stresses in the upper section of the base layer. These higher stresses then initiate the compression failure (crushing) in the base layer.

This failure mechanism, which is different from the classical fatigue failure associated with cemented road building materials (Otte, 1972, 1978), led to the development of the "**crushing life**" concept for lightly cemented gravel pavement materials. According to published research on the compression failure of lightly cemented materials, compression failure (crushing) of the materials occurs at low compressive strains, ie at approximately one (1) per cent axial strain (see Chapter 5 for detailed analysis of these strains). Using this definition of crushing failure, together with permanent deformation measurements at the different depths in the above mentioned test sections, the **crushing life span, Nc**, for the different test sections could be determined. It was shown that Nc was more a function of the tyre contact pressure on the roads surface, than the load. Detailed description and analysis of this crushing failure is given in Chapter 5.

TABLE 4.1 AVERAGE PERCENTAGE PERMANENT DEFORMATION MEASURED AT THE END OF THE HVS TESTS ON THE VARIOUS TEST SECTIONS (%)

DEPTHS (mm)	RELATIVELY DRY STATE	RELATIVELY WET STATE
0-180	67 (17)	75 (7)
180-330	9 (4)	11 (7)
330-480	10 (7)	7 (3)
480- ∞	14 (9)	7 (2)

( ) Standard deviation

4.2.4.4 Rate of permanent deformation, R_L, and relative damage

The structural life (capacity) of a pavement may be defined in many different ways. One method is to determine the number of equivalent standard 80 kN axles (E80s) which will cause a specific level of permanent deformation on the pavement. Normally, a terminal level of 20 mm deformation over a certain length of the road (depending on the road category (TRH4, DRTT, 1985a), at the end of the structural design period, is selected. With this definition of structural life of the pavement, no identification is given to the actual failure mechanism nor the effective rate of failure (deformation).

It is, however, my opinion that the permanent behaviour of pavements, and hence structural capacity, may be better described by using the **rate of deformation**, rather than the total number of E80s carried during the structural life of the pavement. Kleyn et al (1985) indicates that this rate of failure may be seen as representing the "absolute damage rate" of a particular load on a particular pavement. The rate of permanent deformation, R_L, of a pavement in terms of mm per million load repetitions at a particular load, L, affords a very useful means of describing the structural life of pavements, and may assist in better defining the failure mechanism associated with that pavement.

Furthermore, the deformation rate for a standard (P=40 kN) wheel load may then be seen as indicating the basic bearing capacity for that pavement. Comparing the deformation rate of a wheel load with the standard wheel load would give the relevant equivalency factor, D_L. By applying the relative damage formula $D_{80} = (P/40)^r$, the relative damage coefficient, r, relating the rate of damage, may then be calculated (Kleyn et al., 1985).

The different R_L and D_L-factors for the various test sections on Road 1932 (Rooiwal) is summarised in Table 4.2. The table indicates that the rate of deformation increases with increased wheel load and moisture conditions. In this case then "dry" refers to the in situ moisture condition of the base layer, which was approximately optimum (before stabilisation) plus 2,5 per cent. "Wet" refers to the moisture condition of the base layer after rain or the artificial introduction of water on the test section, and was approximately optimum plus approximately 3,5 per cent. On average the rate of deformation, R_L, during the "wet" conditions is approximately 2,7 to 3,8 times higher then during the "dry" conditions on this pavement.

In Figure 4.5 the rate of permanent deformation for various wheel loads and moisture contents on this pavement is illustrated. The effect of wheel load and moisture content of the base layer is well illustrated on the figure.

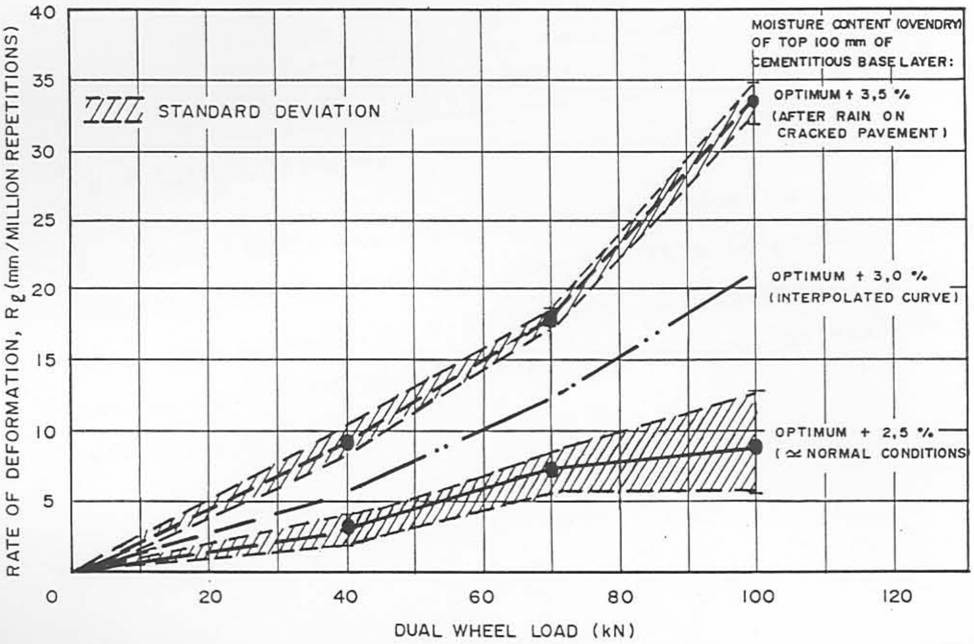


FIGURE 4.5

RATE OF PERMANENT DEFORMATION (R_ℓ) FOR VARIOUS DUAL WHEEL LOADS AND MOISTURE CONDITIONS ON A RELATIVELY DEEP PAVEMENT WITH LIGHTLY CEMENTITIOUS BASE LAYERS (SITE: ROAD 1932, ROOIWAL)

TABLE 4.2 PAVEMENT PARAMETERS OF ROAD 1932 (ROOIWAL)

HVS-SECTION	R_L-FACTOR, mm/million reps						D_L *			
	P=40 kN		P=70 kN		P= 100 kN		P=70 kN		P=100 kN	
	DRY	WET	DRY	WET	DRY	WET	DRY	WET	DRY	WET
250A4	3,33	-	6,36	-	8,75	-				
250A4B	-	8,33	5,00	18,18	12,50	-				
251A4	-	-	7,50	-	-	35,00				
260A4	2,50	9,48	-	-	-	-				
274A4	3,33	8,33	-	-	-	-				
275A4	2,50	11,33	-	-	-	-				
289A4	-	-	8,75	17,50	-	-				
294A4	-	-	-	-	5,56	32,35				
AVERAGE	2,92	9,37	6,90	17,84	8,94	33,68	2,36	1,90	3,06	3,59
STD. DEV.	0,48	1,41	1,60	0,48	3,47	1,87				

* $D_L = R_L(P)/R_L(40) = (P/40)^r$, with P=trafficking wheel load and r= relative damage coefficient

In Table 4.3 the average relative damage coefficients, r, for this road are summarised. It must, however, be emphasised that although r is the coefficient equating the **rate in deformation** in this case, it is the same as the relative damage coefficient normally used for the **absolute deformation** after a specified number of repetitions because the rate of deformation is taken as approximately linear, in the cases discussed in this dissertation.

TABLE 4.3 AVERAGE RELATIVE DAMAGE COEFFICIENT, r^* (ROAD, 1932)

70 kN		100 kN	
DRY	WET	DRY	WET
1,53	1,15	1,22	1,39

* $r = \ln(D_L)/\ln(P/40)$

The table indicates that r is less than the value of 4, which is normally used for relative damage. This is probably related to the fact that this pavement is a relatively deep pavement, which is normally not as load sensitive in terms of fatigue, as shallow pavements (Kleyn et al., 1985).

4.2.4.5 In situ densities and moisture contents

In situ nuclear dry density (drilled through the cemented layers) and oven dried moisture contents were measured on all the test sections concerned to investigate the effect of traffic loading on these parameters. These measurements were taken on the section as well as outside the tested area (positions A,B,C and D), using test pits (see Figure 4.6), in all the pavement layers.

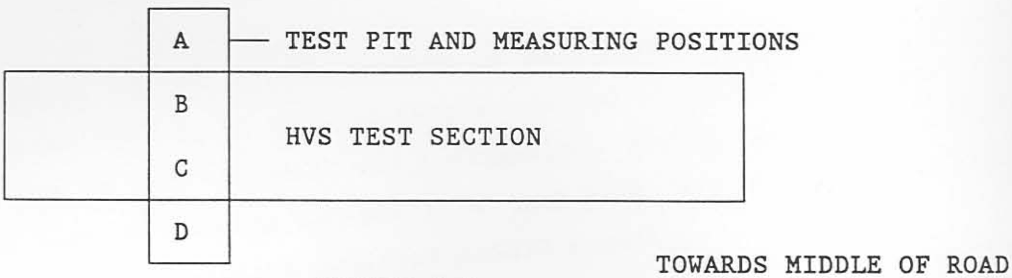


FIGURE 4.6

TYPICAL LAYOUT OF A TEST PIT ON THE HVS TEST SECTION AFTER HVS TESTING

The actual measurements are summarised in Table A.5, in Appendix A. The results indicate that some densification on Sections 250A4A and 250A4B, occurred inside the trafficked area (positions B and C). On the rest of the sections some de-densification was measured, with the highest on Section 294A4. The de-densification on this section (top 50 mm of the base layer) varied between 0,74 per cent and 4,84 per cent, if the inside density (B and C) is compared to that outside (A and D) the test area. This is an indication of the crushing effect in the base, which was mentioned in Paragraphs 4.2.4.2 and 4.2.4.3. It must be noted, however, that it appears that the nuclear apparatus did not always indicate the crushing as a de-densified zone, as most of the density measurements were taken in a drilled hole through the upper 100 mm of the cemented base, while close observation in the test pits confirmed that the thickness of the crushed zone was approximately 50 mm to 75 mm. In most cases large areas of this crushed zone were pumped out as a result of an excessive porewater pressure state, and it was not possible to measure the densities very effectively. In general, the in situ moisture contents of the upper layers (> 400 mm) appears to be higher than those of the deeper layers, and is believed to be related to rain and surface water ingression.

4.3 BEHAVIOUR OF A SHALLOW PAVEMENT: ROAD 2212 (BULTFONTEIN)

As for the deep pavement, the emphasis will be on the permanent deformation behaviour of this pavement, as was quantified under accelerated testing with the TPA - HVS.

4.3.1 Pavement structure and location of HVS - test sections

In Figure 4.7 (a), the layout of the various HVS test sections on this road at km 12,6 is illustrated. These pavement sections were selected by using the DCP, and according to these results a relatively shallow pavement section was identified (see also Chapter 6, for detail DCP-classification and analysis of this pavement). The test wheel load as well as the tyre pressure used during testing are also indicated in the figure, and will be discussed later in more detail in Paragraph 4.3.3.

In Figure 4.7 (b), the general in situ pavement structure is illustrated. The figure indicates that the base and subbase were of C3 and C4-quality (TRH4, DRTT, 1985a), stabilised with 3 and 2 per cent Portland blastfurnace cement (PBFC), respectively. The parent material for both base and subbase layers, including the selected (G4) and in situ (G5) layers, is of a relatively good quality mix granophyre and ferricrete (see Marais, 1981). Stabilisation of the base and subbase however was necessary in order to reduce the plasticity and to improve the bearing capacity of these layers.

According to the as - built records of TPA (Marais, 1981), the selected layers were 110 mm to 120 mm ferricrete and a 140 mm layer of river sand. The in situ material was a slightly - plastic ferricrete, with a grading modulus varying between 1,74 and 2,34, and a CBR approximately 30 percent at 97 percent modified AASHTO density.

At the time of selecting the test sections, DCP measurments as well as observations in an open test pit, indicated that there was a relatively

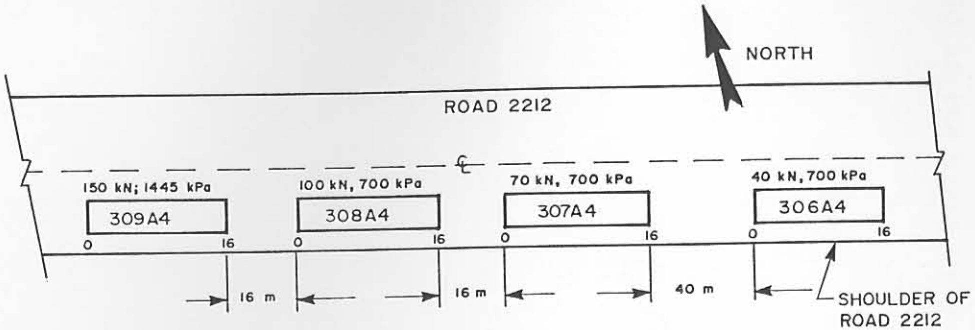


FIGURE 4.7 (a) *LAYOUT OF THE VARIOUS HVS TEST SECTIONS ON ROAD 2212, BULTFONTEIN*

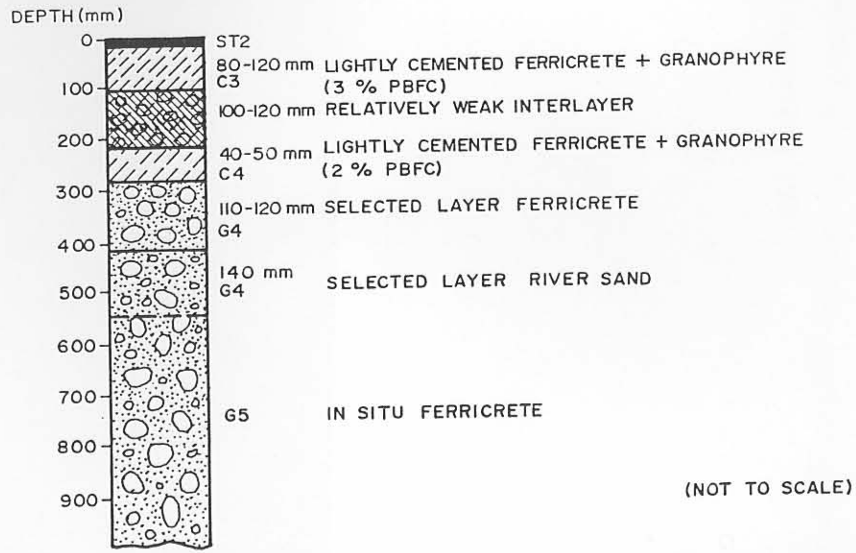


FIGURE 4.7 (b) *DETAIL OF THE RELATIVELY SHALLOW PAVEMENT STRUCTURE AT THE HVS SITE AT km 12,6, ON ROAD 2212 NEAR BULTFONTEIN*

weak zone of approximately 100 mm to 120 mm between the base and subbase layer (see Figure 4.7 (b)). According to the theory of the strength balance (DCP-defined) of pavements (see Chapters 3 and 6), it was suspected that this weak zone would have a marked effect on the permanent deformation behaviour of this pavement. The test pit detail and the DCP, showed the base layer (80 mm to 120 mm) to be relatively well-cemented with in situ UCS of approximately 3 MPa. The relatively weak support underneath the base made the pavement load sensitive and therefore fatigue failure of the base should dominate the behaviour. This, however, will be discussed later in more detail in Chapter 7.

4.3.2 Original design considerations

This typical TPA Class II pavement was designed to carry 0,2 to 0,4 million E80s (Transvaal Roads Department (TPA), 1978; Marais, 1981). Owing to the relatively weak in situ material as well as the use of average quality material for the selected layers, a shallower pavement than the previously discussed deep pavement resulted. Detailed analysis of the DCP-results, however, is discussed in Chapter 6.

In Appendix A, Tables A.6 and A.7, a summary of some of the original design data for this pavement is given, and indicate characteristics of the parent material for both the base and the subbase. As this was a TPA-Class II pavement, and the fact that the material was plastic, stabilisation was necessary to achieve less erodible (pumpable) materials for both the base and subbase of this pavement. According to the initial laboratory UCS-results (Appendix A, Table A.7, a minimum of 3 per cent PBFC was necessary in order to provide a C3 - quality base material and 2 per cent PBFC to provide a C4 - quality subbase material (TRH13, DRTT, 1986).

This pavement was constructed during 1981 and was approximately 6 years old at the start of HVS testing during 1987.

4.3.3 HVS test programmes on Road 2212 (Bultfontein)

A summary of the HVS test programme followed on Road 2212 (Bultfontein, Transvaal) is given in Appendix A, Table A.8.

On this road a total of 4 different HVS tests was done over a test period of 19 months. During this period a total of almost 129 million E80s (r=4 in $(P/40)^r$) was applied to this road. The dual wheel loading (P) varied between 40 kN and 100 kN, with a tyre pressure of 700 kPa. One test on this pavement, however, was done at an excessively high single wheel loading, using an aircraft wheel (Boeing 747), similar to two additional tests on the deep pavement (Road 1932 at Rooiwal), (see later in Paragraph 4.4). The single wheel load for these latter tests was 150 kN, with a tyre pressure varying between 960 kPa and 1445 kPa (see Appendix A, Table A.8, HVS-Section 309A4). Detailed discussions of these tests are given in Paragraph 4.4.

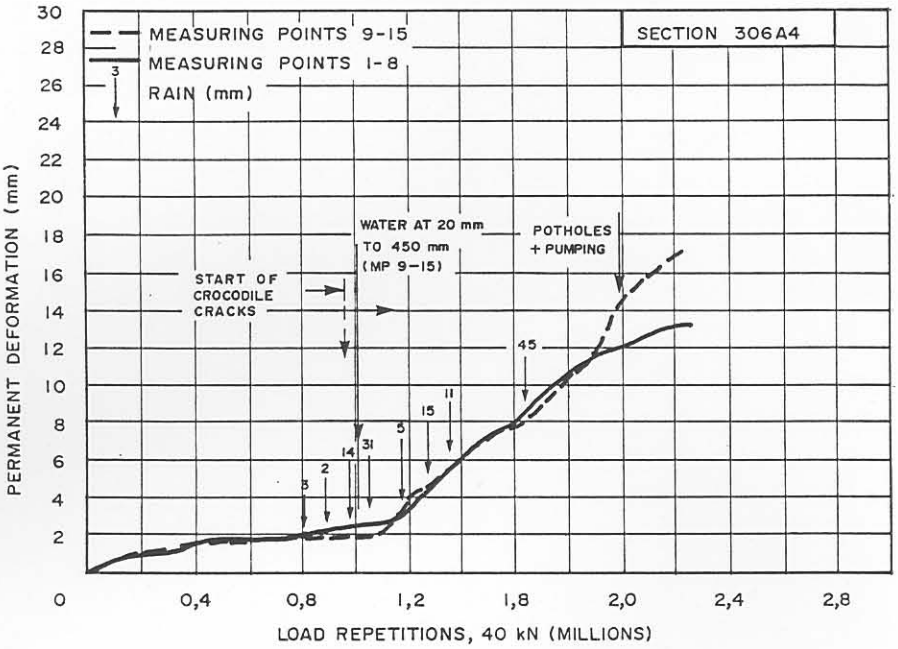
HVS testing was done during in situ moisture and natural temperature conditions up to approximately one million load repetitions. Water was introduced artificially on the surface as well as into the pavement after approximately one million repetitions on Sections 306A4, 307A4 and 308A4.

4.3.4 Permanent deformation behaviour and failure mechanism

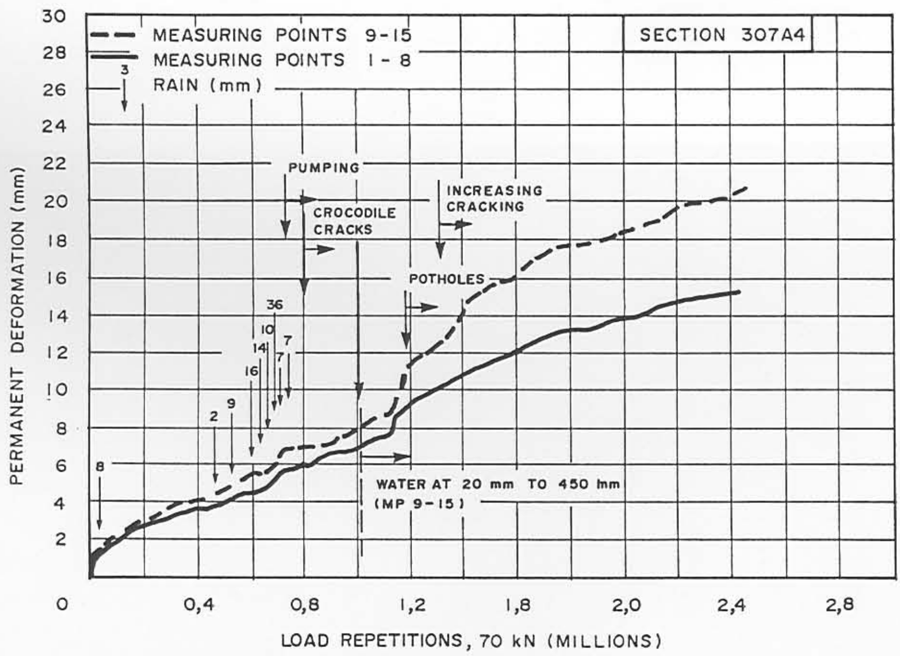
Permanent deformation as a direct result of trafficking as measured on the the surface of the pavement (as mentioned for the deep pavements), is used to describe the state and behaviour of the pavement. It is also one of the primary indicators of the behaviour of the HVS test sections on this relatively shallow pavement under accelerated testing.

4.3.4.1 Permanent deformation on the surface (rutting)

The permanent deformation behaviour, as measured under a 3 m straight edge, of the various test sections is illustrated in Figure 4.8 (a), and (b) and Figure 4.9. The figures indicate the average maximum permanent



(a) HVS SECTION 306A4



(b) HVS SECTION 307A4

FIGURE 4.8

AVERAGE PERMANENT DEFORMATION AT VARIOUS STAGES OF TRAFFICKING ON HVS TESTS 306A4 AND 307A4 UNDER THE INDICATED DUAL WHEEL LOADS ON ROAD 2212 (BULTFONTEIN)

940-4-5900/16 BD

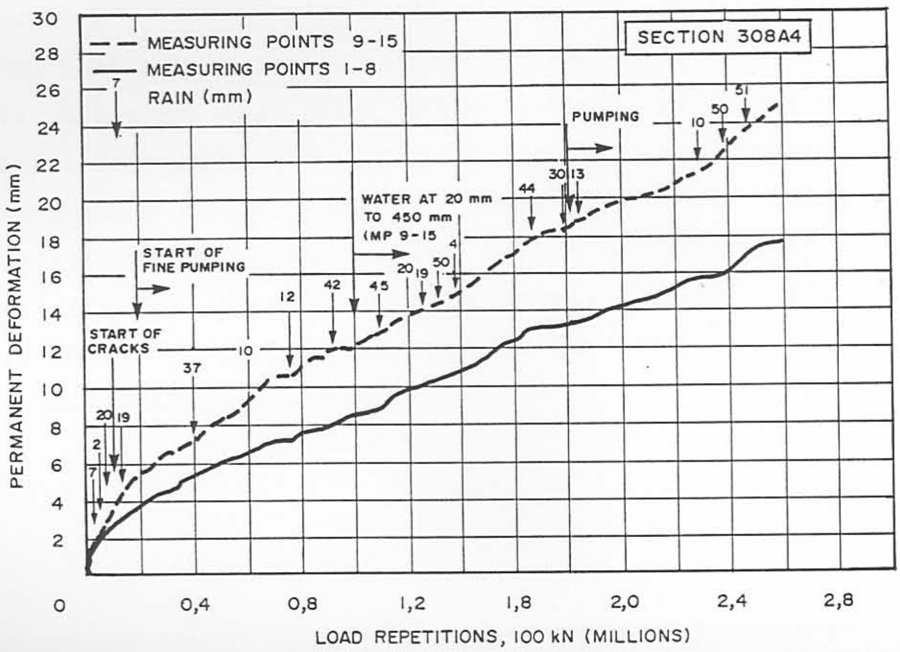


FIGURE 4.9

AVERAGE PERMANENT DEFORMATION AT VARIOUS STAGES OF TRAFFICKING ON HVS TEST 308A4 UNDER A 100 kN DUAL WHEEL LOAD ON ROAD 2212 (BULTFONTEIN) deformation development on the surface of the test sections after various stages of trafficking, loading and under certain moisture conditions. In general, the figures indicate that the rate of deformation per wheel load is approximately linear (similar to that of the previous deeper pavement) and increases with the introduction of surface water (mostly rain) and the artificial introduction of depth water into the pavement structure.

Figure 4.8 (a) illustrates the permanent deformation development on Section 306A4, and indicates that after approximately 1,1 million repetitions, the rate of deformation, R_L, increased from 2,27 mm per million repetitions to 10 mm per million repetitions. At this turning point in the deformation development, crocodile cracks were observed on the surface of the test section and approximately 50 mm of rain had fallen on the test section during this period.

At this stage water was also introduced into one part of the section (measuring points 9 to 15), using perforated metal pipes, which were placed at a depth of 20 mm to 450 mm. These pipes were installed about 600 mm to the side of the test section.

With the introduction of water into the depth of the pavement the increase in the rate of deformation was minimal, as the measured increase was not exclusive to this wetter part of the test section, (ie measuring positions 9 to 15, see Figure 4.8 (a)). Almost the same increase in rate of deformation occurred on the drier part of the section where no water was introduced (ie measuring positions 1 to 8).

Although it rained on the entire section, no HVS trafficking was allowed during that period. HVS trafficking commenced after the rain. However, it is believed that rain water entered the fatigue cracks on the section, increasing the moisture content on the sides of the cracks, weakening the load transfer (cohesion) at the cracks, as well as increasing the moisture content of the weak zone at the bottom of the base layer at a depth of approximately 110 mm. This weaker zone is believed to be the primary reason for the increase in the rate of deformation on this section.

The number of repetitions (E80s in this case) at this turning point (increase in rate of deformation) may be described as the **"effective fatigue life"** of the cemented gravel base layer, and is approximately one million E80s. A more detailed discussion of this aspect based on linear elastic analysis may be found in Chapter 7.

Towards the end of the HVS test, excessive potholing and pumping resulted on this section at measuring points 9 to 15. This was a result of the introduction of surface water and rain. Test pits after the HVS test on this section confirmed that compression failure (crushing) had also occurred in the upper 50 mm to 75 mm of the cemented gravel base layer of this pavement, similar to those found on the deep pavement discussed in Paragraph 4.2.4.3.

4.3.4.2 Failure mechanisms: "deep" versus "shallow" pavements

Although similar crushing behaviour was noted on both the deeper and shallower pavements towards the end of the HVS tests, there is an important difference: for the deeper pavement, crushing failure dominates from the beginning of the test (traffic loading), while for the shallower pavement, classical fatigue failure initially dominates (effective fatigue life), after which crushing failure occurs in the upper section of the base, similar to that of a deeper pavement. This is probably so because of the <u>initial</u> imbalance in the base and subbase of the shallow pavement. It is important to note that this failure mechanism is what was observed during the tests and must not be construed as applicable to all similar pavements without further evidence.

Owing to this imbalance, "traffic moulding" occurs during traffic loading of the pavement, "punching" the relatively well-cemented cracked base material into the softer underlying weak layer. This process continues until the weaker layer is more densely compacted than in its initial state, thereby increasing its load-bearing capacity, and hence its support to the already-cracked base. Eventually, the repetitive stresses in the base increase and caused the crushing failure in the upper section of the cemented base.