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BEHAVIOUR OF VARIOUS REHABILITATION OPTIONS OF A "CRACKED-AND-SEATED" SEMI-RIGID PAVEMENT DURING ACCELERATED TESTING IN TRANSVAAL

By

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SUMMARY

This paper summarises the full scale accelerated research done during the past six years with the Heavy Vehicle Simulator (HVS) belonging to the Roads Branch of the Transvaal Provincial Administration (TPA). The research concentrated on the structural behaviour of pavements incorporating lightly cementitious base and subbase layers. Specific failure mechanisms were identified which include aspects such as crushing failure of deep pavements and effective fatigue of shallow pavements. Improved in-situ pavement evaluation methods such as with the Dynamic Cone Penetrometer (DCP) are also described. Detail information regarding various rehabilitation options for these pavements are given together with their HVS performance.

OPSOMMING

Hierdie referaat som verskeie navorsingsbevindinge op van die volskaalse versneldetoetswerk wat gedurende die afgelope ses jaar met die Swaarvoertuig Nabootser (SVN) van die Paaie Tak van die Transvaalse Provinsiale Administrasie (TPA) gedoen is. Die navorsing het gekonsentreer op die strukturele gedrag van plaveisels met lig-gesementeerde kroon- en stutlae. Spesifieke swigtingsmeganismes is ge-identifiseer en sluit die volgende in, nl. vergruisingswigting van diep plaveisels en effektiewe vermoeiing van vlak plaveisels. Verbeterde in situ plaveiselevalueringsmetodes soos met die Dinamiese Kegelpenetrometer (DKP) word ook beskryf. Detail inligting rakende verskeie rehabilitasie opsies vir hierdie plaveisels, tesame met die SVN gedrag word bespreek.

INTRODUCTION

The Heavy Vehicle Simulator No. 4 (HVS04), of the Transvaal Provincial Administration (TPA), was commissioned in 1977 and is operated by the Division of Roads and Transport Research (DRTT) of the CSIR, on behalf of the Roads Branch of the TPA. Since 1977, a total of fifteen different pavement structures were evaluated with this HVS with the emphasis on gaining a better understanding of basic pavement behaviour.

This paper updates information presented at the Annual Transportation Conventions (ATC) of 1985 and 1987 (Kleyn et al, 1985 and De Beer, 1987), and describes in detail the most recent series of HVS tests on a **rehabilitated pre-cracked-and-seated semi-rigid pavement structure**. Although some of the HVS testing regarding the rehabilitation options are not yet complete because of the long term nature of the tests, the findings so far on the completed tests are given. Reference is also made to the initial basic research done on lightly cementitious (semi-rigid) pavements, including relatively deep and shallow pavement structures. The information given concentrates on the structural behaviour aspects of these pavements and is in line with the longer term goals of the TPA, namely, to gain understanding of these pavements and to implement the knowledge towards more cost effective new, as well as rehabilitation, designs. The estimated funding needs of the provincial road network in the Transvaal is currently in the order of 1000 million rand per annum, which is about double the allocated budget for 1991/92. Even during 1989 it was estimated that the implementation HVS results was saving the TPA about 13 million rand annually, emphasising the role of implementation orientated research in terms of strict budgets. Since 1985 the emphasis of HVS research with the TPA machine was on the structural behaviour of lightly cementitious (semi-rigid) pavement compositions. From 1989 onward, a detailed testing programme on various rehabilitation options for such pavements was initiated, with the overall objective of being able to optimise the life cycle strategies for various pavement types. It is foreseen that similar advances and savings to the above, will accrue through implementation of these results.

OBJECTIVES

The objectives of HVS testing in Transvaal can be summarised as follows:

- Evaluation of **light granular** pavement structures (1977-1980)
- Evaluation of **medium to heavy granular** pavement structures (1980-1982)
- Evaluation of **strongly cemented** base pavements (1982-1984)
- Evaluation of **lightly cementitious** base pavements (1984-1988)
- Evaluation of various **rehabilitation options** for lightly cementitious pavements (1988-1993)
- Evaluation of light pavement structures with **marginal and alternative materials** (1994 onwards)

Both the longer and short term test programmes have been regularly reviewed and if necessary revised by the HVS committee. In the following paragraphs a short summary of the main research findings with the HVS on lightly cementitious pavements (1984-1988) is given, together with a detailed summary of the progress with the current HVS research regarding various rehabilitation options for these pavements.

SUMMARY OF RESEARCH FINDINGS: 1984-1988

Background

The research on pavements incorporating lightly cementitious bases and subbases started during 1984/85 on a relatively "deep" pavement structure, ie. road 1932, near Rooiwal. The term "deep" pavement refers to the vertical strength distribution in the pavement, as is defined by in situ strength measurements (De Beer et al., 1988a). The detailed HVS and related research findings on lightly cementitious pavements are given elsewhere (De Beer, 1990). The following paragraphs summarise the most important findings.

In situ Pavement Strength Measurements

Prior to this research, detailed investigation and basic definitions related to the in situ testing of a pavement (DCP based technology) were formulated by Kleyn et al (see De Beer et al, 1988a). However, with the HVS research on road 1932, as well as follow-up research on a relatively "shallow" pavement structure, ie road 2212 near Bultfontein, north of Pretoria, the DCP based technology was further advanced and in consultation with the TPA some of the definitions revised and others added. From this research a **universal DCP classification system** was developed, based on the concept of strength-balance of pavements (De Beer et al, 1988a). With this system the concept of **strength-balance paths** was also introduced. The strength-balance path describes or quantifies the **structural changes** or **traffic moulding** of the pavement under trafficking in terms of two fundamental DCP parameters (The results indicated that shallow pavements may convert to deep pavements, and if overstressed, the deep pavements convert to inverted pavements, etc.). This classification system not only enhances pavement evaluation, but also improves rehabilitation design of pavements based on in-situ measurements such as the DCP based technology (De Beer, 1991a, Van Der Merwe et al,

1991). In addition to the DCP classification system, concurrent DCP testing with several HVS tests resulted in the development of an **empirical structural bearing capacity prediction model** based on the development of permanent deformation in pavements with thin surfacings and lightly cementitious bases. The model is based on various in situ strength parameters, and is correlated with the observed approximate linear rate of permanent deformation (R_f) found on these pavements(see also Kleyn et al., 1985). Lastly, also resulting from the concurrent DCP and HVS testing, an **empirical relationship between DCP penetration rate (DN) and back-calculated effective elastic moduli** from measured multi-depth deflections (MDD) was developed. This development is regarded as very important because it provides an **objective link** between DCP based technology to current mechanistic design methods in South Africa. Details of this relationship and its application is given elsewhere (De Beer, 1991a).

Permanent deformation behaviour

It can be summarised that the HVS results regarding permanent deformation on pavements with thin surfacings and lightly cementitious bases (C3/C4; TRH14, 1985), indicate that the rate of deformation is approximately linear. For relatively deep pavements the origin of the rutting is initially within the thin surface seal, and, as a result of compression failure (crushing) of the top 50 mm of the base, it changes to the upper portion of the base. The rate of rutting, however, is strongly influenced by moisture content, and, in the crushed state these pavements can become very sensitive for surface water ingress. Shallow pavements initially fail in fatigue, after which the pavement composition becomes deeper. The subsequent failure mechanism is then similar to those found for deep pavements, ie. crushing of the base. Most of the results on both the deep and shallow pavements on relatively good subgrades (in situ CBR > 30 %) indicated that more than 80 % of the permanent deformation originated from within the base layer only.

Crushing failure

As mentioned in the previous paragraph, crushing failure occurred in the upper portion of the cementitious base layers of both types of pavements. The results from these tests indicated that the crushing failure is directly related to the in situ strength of the base layers (in terms of DCP measured unconfined compressive strength (UCS)) and the tyre contact stress (σ_t). A linear relationship was established between the ratio of tyre contact stress and UCS (σ_t/UCS), with the log of the number of stress repetitions to initiate this crushing failure (Nc). For more detail see De Beer (1990).

Effective fatigue

Tests on the shallow pavement indicated that the initial failure mechanism is principally related to fatigue failure of the cementitious base layer. Detailed test results indicated that a linear relationship (already existing for laboratory beam testing) exists between the ratio of induced modified tensile strain and the strain at break ($\varepsilon_s/\varepsilon_b$), and the log of the number of strain repetitions to effective fatigue (N_{ef}). In this case effective fatigue is defined as a state of 2 mm permanent deformation and a resilient standard 40 kN road surface deflection (RSD) ranging between 0.5 mm and 0.75 mm (De Beer, 1990).

Structural Capacity and Preventative maintenance

The detailed study with the HVS on these pavements indicated that the effective structural bearing capacity of these pavements are in **excess of 5 Million Standard Axles (MISA)** - if tyre pressures and overloading are adequately controlled. The effect of tyre pressures can be quantified using the developed crushing failure criterion, and the "life" to initiation of crushing failure (Nc) is a reasonable estimate of the fatigue cracking life of the surface treatment. Therefore, life cycle based maintenance strategies, based on this criterion, can be better planned for these types of pavements in order to seal the surface as a preventative measure, rather than carrying out major structural rehabilitation owing to the advanced crushing of the base.

ASPECTS OF REHABILITATION OF PAVEMENTS WITH LIGHTLY CEMENTITIOUS LAYERS

Background

As a result of the extensive research and associated findings with the study during 1984 to 1988, it was decided to focus future research with the HVS04, on various rehabilitation options for pavements with cementitious layers. At this stage the basic structural behaviour of these pavements was well understood, and the need to evaluate and understand the structural behaviour of various rehabilitation designs developed. Therefore, during early 1989, it was decided to embark on a long term study to evaluate three different rehabilitation designs on road 2212, near Bultfontein. The rehabilitation options were selected by the TPA, and consisted of a 150 mm crushed stone (G1) base, 35 mm continuously graded asphalt surfacing and a double surface treatment (ST2), (13.2mm/6.7mm). The detail of the various pavement structures is illustrated in Figure 1.

"Crack-and-Seat" (C & S) Operation

Vibratory Rolling

The problem, however, was to convert relatively large areas of existing pavement (40 m and 80 m, full lane width of 3.5 m) into the so-called end condition similar to the end state produced by the HVS on the existing pavement. This was needed because the normal HVS section of 1 m by 10 m was regarded as to small for a proper structural evaluation of the different rehabilitation options. It was therefore decided to use a "crack-and-seat" (C & S) method on the shallow pavement, using a heavy vibratory roller. Two areas (Sections P1 and P4) on this pavement near the existing HVS site, were selected for the C & S operation. The permanent deformation, DCP characteristics, road surface deflection and in situ densities were used to quantify the "end state" after rolling. In Figure 2 the development of permanent deformation as a result of vibratory rolling is illustrated. The figure indicates that permanent deformation between 5 mm and 10 mm developed during relatively dry conditions. Most of the measured permanent deformation on the HVS sections was also less than 10 mm. The standard 40 kN road surface deflection varied between 0.5 mm and 0.75 mm after rolling, which corresponds to deflections varying between 0.6 mm and 0.9 mm on the HVS sections. The reason for the higher deflection on some of the HVS sections is that it originated from within the crushed layer, whilst on the C & S sections, the crushed material (fine powder) was removed before RSD measurements were taken. According to DCP measurements similar results between the HVS sections and the roller sections (P1 and P4) were obtained. The DCP parameters used in this case are summarised in Table 1.

Surface Deflection

Standard 40 kN surface deflection (RSD) measurements were also taken as a control on these sections. In Figures 3a and 3b the deflection results before and after rolling on the two sections (P1 and P4) are illustrated. As a result of the rolling, an increase in deflection occurred, but relatively high coefficients of variance (cov) in the deflection were obtained. For the two sections the cov varied between 12 % and 20 % before rolling to 22 % to 27 % after rolling. During the rolling process the top 30 mm of the cementitious base and the surfacing crushed completely and was removed before deflection measurement. Inspection of the base layer after rolling indicated block cracking with block sizes of approximately 150 mm by 150 mm. Also noticeable was the appearance of flaky fractures within the base layer, which may possibly explain the higher cov in deflection after rolling.

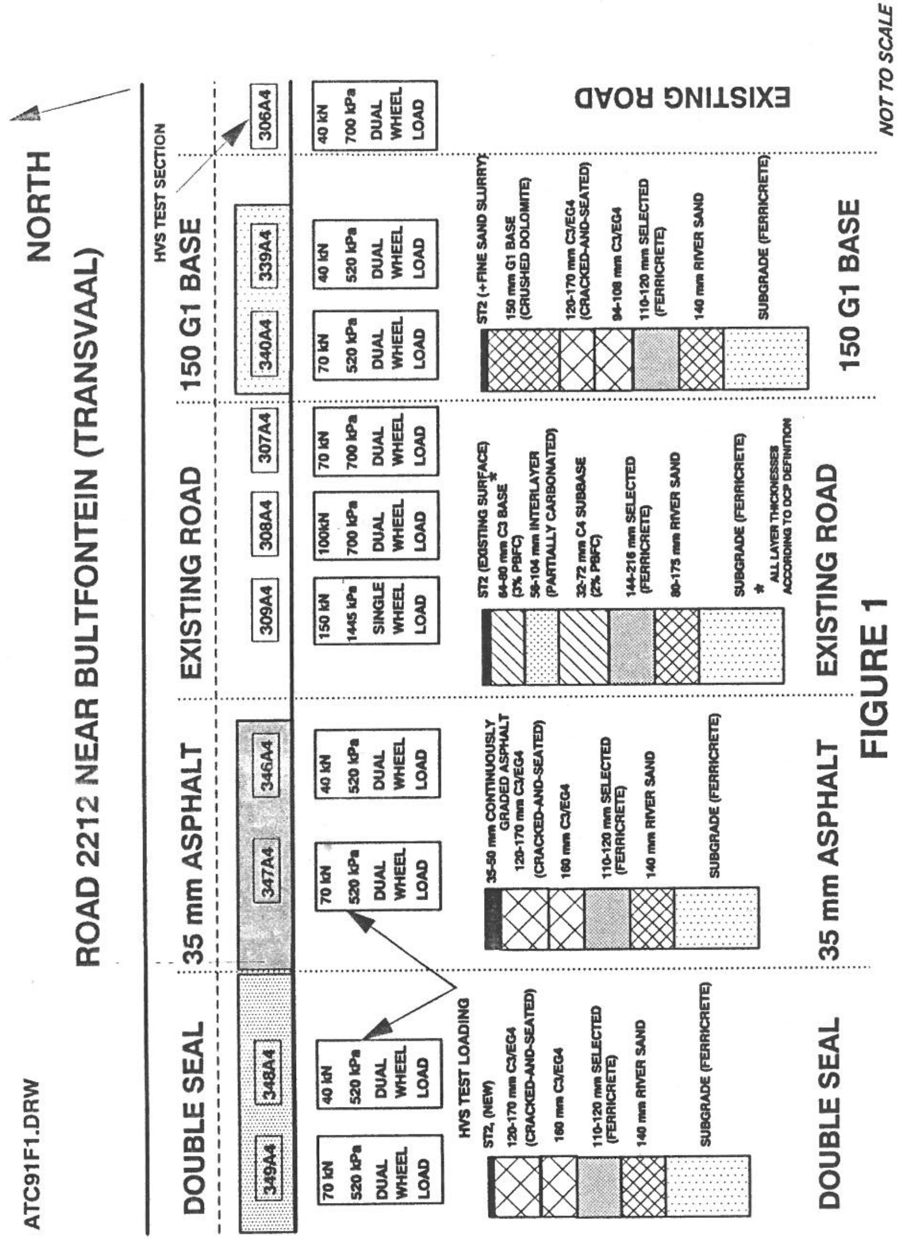

FIGURE 1

LAYOUT OF HVS TEST SECTIONS AND REHABILITATION DETAIL

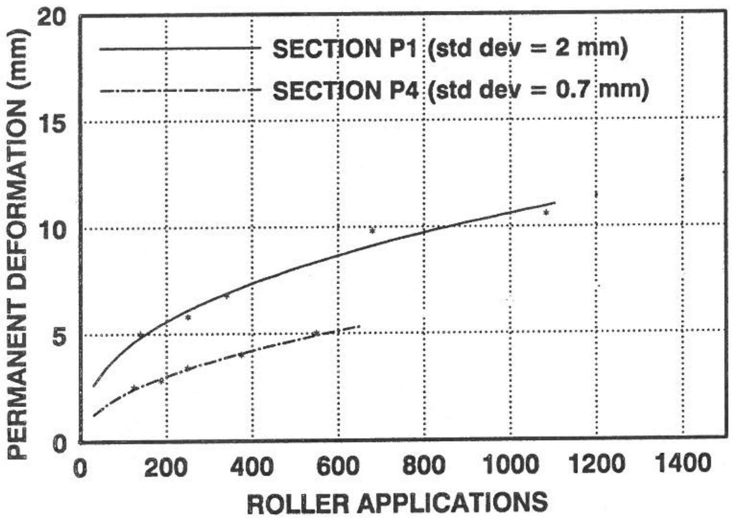

FIGURE 2

PERMANENT DEFORMATION AS A RESULT OF VARIOUS ROLLER APPLICATIONS ON ROAD 2212 (SECTIONS P1 AND P4)

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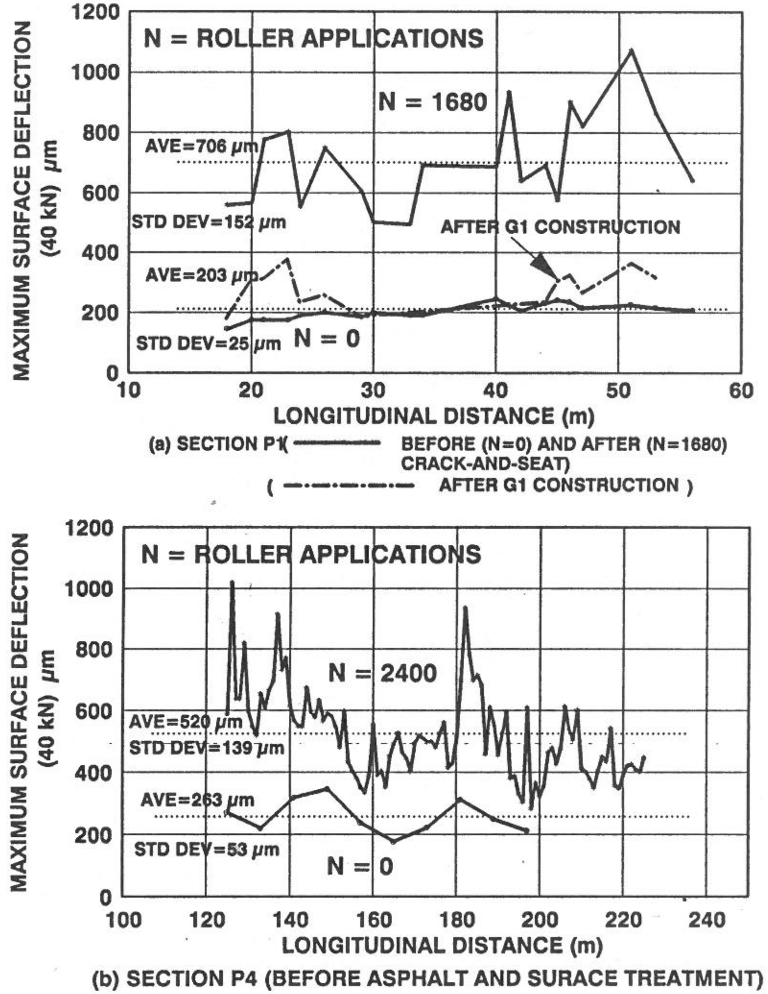

FIGURE 3

INCREASE IN MAXIMUM ROAD SURFACE DEFLECTION AS A RESULT OF ROLLER APPLICATIONS ON BOTH SECTIONS

In situ Densities

In situ densities of the base layer were measured with the nuclear apparatus, and a reduction of approximately 5.3 % in the top 50 mm, and 4.6 % in the top 100 mm in density of the base were measured. This reduction occurred as a result of crushing and fatigue failure of the base. An increase in density of 1.7 % at a depth of 200 mm was also observed. Similar results were obtained under various HVS sections, and therefore it was concluded that the "end states" of the roller sections and HVS sections were sufficiently similar.

Construction of Rehabilitation Sections

G1-base layer on Section P1

After the C & S operation, the pavement was broomed and cleaned before a standard prime coat was applied. For the G1 base (Section P1) a 150 mm concrete berm was constructed on the centreline of the existing pavement, with a 150 mm gravel (G4 material) shoulder. The G1-layer was then "boxed-in" between these two side constraints. The material was spread with a normal grader and compacted in three 50 mm layers. An apparent density of 87.7 % was achieved during construction. After 4 days the slushing process started. The slushing was done with the same roller (without vibration). A clear mosaic appearance of the surface appeared after brooming. After 5 days of drying out, the prime coat was applied, and 10 days after the prime a 13.2mm/6.7mm surface seal was hand spread on the section. Owing to a relatively rough surface it was decided to apply an additional fine slurry seal to provide a smooth surface, to facilitate accurate surface measurements during HVS testing. The slurry was applied approximately 2 months after the surface treatment. Deflection measurements after the G1 construction indicted a reduction in average deflection from approximately 706 μm to approximately 330 μm (See Figure 3a). The G1 layer therefore resulted in a structural improvement on the C & S pavement. It is, however, important to note that deflections higher than 800 μm on pavements in need of structural rehabilitation, may result in difficulty with the compaction and final density of the G1 material. Such conditions should therefore be improved (compaction or stabilisation) <u>before</u> the application of the granular base layer.

TABLE 1 DCP PARAMETERS* ON ROLLER SECTIONS P1 AND P4 (De Beer et al, 1988b)

ROLLER APPLIC.	DSN800 (Blows)	A-PARA-METER	B-PARA-METER	AVERAGE DCP PENETRATION RATES, DN (mm/blow) DEPTH (mm) 0-68	69-208	209-440	441-800
ROLLER SECTION P1							
10	334	1473	35	0.9	2.8	5.2	8.4
1680	250	803	29	1.8	2.3	4.9	8.7
306A4**	243	1629	28	2.0	2.5	5.0	10.0
ROLLER SECTION P4							
10	238	2208	29	1.3	4.2	7.1	9.9
2400	237	1701	22	1.9	2.8	5.3	6.3
308A4**	267	2304	27	1.3	3.2	5.3	7.6

* For definition of these DCP parameters, refer to De Beer et al (1988a).
** HVS Sections

Asphalt and surface treatment on Section P4

The 35 mm continuously graded asphalt surfacing was constructed using a normal asphalt paver. The 13.2mm/6.7mm surface treatment (ST2), however, was also hand spread on a relatively irregular surface of the pre-cracked base, and resulted in a relatively rough final surface. The surface treatment option was selected as a relatively inexpensive holding action treatment on such pavements. The HVS testing was planned to determine the real "life" of the different rehabilitation and holding actions, and to perform some detail testing on the actions to facilitate relative comparisons. Further, with the emphasis on verification of considerations regarding traffic moulding, pavement strength balance and thus advance the understanding of pavement behaviour towards more effective life cycle strategy design. Deflection measurements after the construction of the relatively thin asphalt and surface treatment indicated similar deflection levels as before the surfacing, indicating that these two measures only act as surface or tyre contact improvements, rather than structural improvement like the G1-layer.

HVS testing of some of the rehabilitation sections

Originally it was planned to do two HVS tests per rehabilitation option. A summary of the test programme is given in Table 2. The table indicates that testing will be continued until 1993/94 before the project is complete. Test periods may reduce as a result of premature failures, in which case the total test programme will be shortened accordingly.

Table 2 Summary of planned HVS testing on the rehabilitation sections

HVS TEST NO.	TYPE OF REHAB.	DUAL TEST LOAD (kN)	TYRE PRESSURE (kPa)	REPETITIONS
339A4	150 mm G1 - BASE	40 70	520 520	4 634 442 1 018 558
340A4	150 mm G1 - BASE	70	520	(1993/94)
346A4	35 mm ASPHALT	40 70	520 520	2 800 000 800 000
347A4	35 mm ASPHALT	70	520	(1991)
348A4	SURFACE TREATMENT	40 70	520 520	50 000* (1992)
349A4	SURFACE TREAT.	70	520	(1992/93)

* Current Testing

Permanent Deformation on the G1-Section

During March 1990 the 40 kN test on the G1-base section (339A4) was completed. On this test a total of 4.634 MISA and 1.02 million 70 kN load repetitions were applied, without excessive average deformation (absolute deformation), excluding the initial 2 mm occurring within the slurry during relatively dry conditions (The initial deformation of approximately 2 mm (see Figure 4) resulted from within the sand slurry layer, and is not regarded as normal for G1 type base behaviour, see Maree et al., 1980). During wet conditions (surface water) the average deformation increased to approximately 7 mm (See Figure 4). During heavier loading at 70 kN the deformation increased further, also with varying rates of deformation during the dry and wet conditions.

The different rates of permanent deformation (R_f) and relative damage, based on the well known power law viz.:

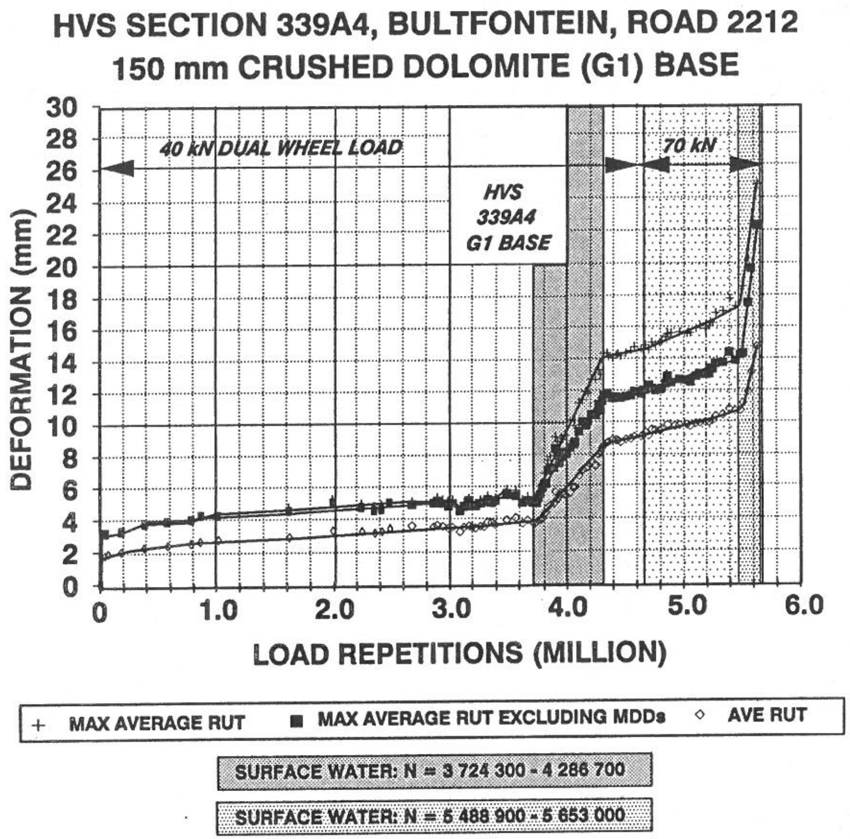

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FIGURE 4
PERMANENT DEFORMATION ON THE G1 BASE SECTION AT VARIOUS STAGES OF TRAFFICKING

$D_l = (P/40)^d$ 1

where D_l = relative damage factor
d = relative damage exponent
P = test wheel load in kN,

is given in Table 3. Inspection of the HVS sections indicated that local failures (ie. cracking and deformation) occurred around the MDD positions as well as at two DCP measurement positions on the section, especially during the wet test. In Figure 5 the permanent deformations measured at different depths in this pavement at the three MDD positions are illustrated. The figure indicates that very little deformation occurred on the test section except at the positions of the MDDs during wet conditions. On drying out of the pavement the rate of deformation also reduced. The local failures around the MDD positions are not regarded as normal for the pavement and therefore should be excluded from the data normally used to calculate deformation rates and relative damage. Based on this, the average relative damage exponent (d) for this section is 2.31 for dry, and 2.44 for wet conditions. No cracks (other than those around the MDDs and DCP test positions) appeared on this section, and the majority of the deformation resulted from stone loss from the surfacing. Nuclear density measured before and after the HVS test indicated a slight increase of 2.9 % in dry densities (compaction) occurred (87.7 % to 89.9%) as a result of HVS trafficking. During this period the average moisture content also increased from approximately 1.8 % to between 3 % and 4% as a result of surface water ingress.

Table 3 Rates of permanent deformation and relative damage on Section 339A4 (Otte, 1990).

TEST LOAD (kN)	RATE, R_l * (mm/10^6 reps) DRY	WET	RATE, R_l ** (mm/10^6 reps) DRY	WET	RATE, R_l *** (mm/10^6 reps) DRY	WET	RATE, R_l **** (mm/10^6 reps) DRY	WET
40	0.54	16.00	0.54	11.54	0.54	8.54	0.54	6.76
70	2.34	48.80	2.34	48.80	1.64	24.40	1.64	24.40
RELATIVE DAMAGE BASED ON $D_l = (P/40)^d$								
D_l	4.33	3.05	4.33	4.23	3.04	2.86	3.04	3.61
d	2.62	1.99	2.62	2.58	1.99	1.88	1.99	2.29

* Maximum average deformation (MDD positions included)
** Maximum average (MDD positions excluded)
*** Average deformation (MDD positions included)
**** Average deformation (MDD positions excluded)

Resilient Deflection Response of the G1 Section

In Figure 6 the standard 40 kN resilient deflection at various stages of trafficking on the G1-Section is illustrated. The figure indicates that the initial deflection was approximately 330 µm, and increased to a steady level (upper limit) around 500 µm. According to TPA practice based on experience, the current threshold and warning levels of maximum standard deflection are 300 µm and 400 µm, respectively. An increase in deflection to approximately 640 µm occurred during the wet test. In Figure 7 the standard 40 kN depth deflections from the MDDs are illustrated. The figure also indicates that very little changes in depth deflections occurred during the test. Basically the resilient response of this pavement was constant up to 4.4 MISA, which is a good indication of its structural capacity. Back-calculated effective elastic moduli for the various layers are given in Table 4. The table indicates that relatively low E-moduli were calculated for the wet state, but it is believed that under estimation of the layer moduli occurs because of the local failures around the MDD holes during testing in the wet condition. The real moduli, however, is believed to be closer to that of the dry state. Laboratory work on the characteristics of the G1 material used for this study indicate a very high stress dependency for both the elastic moduli and the Poisson's ratio. The tests have been done with the K-mould apparatus, which is a variable σ_3-test, where σ_3 is automatically applied pro rata to the increase in σ_1, the vertical stress. Detail of this apparatus can be found elsewhere (Semmelink, 1991a, 1991b). For the G1 material the following parameters were determined: K1 and K2 in $K1\Theta^{K2}$; cohesion C and angle of internal friction ϕ and range of Poisson's Ratio, μ. Details of these values are given in Appendix A. The use of these values are critically important for correct modelling of the G1 material.

Permanent Deformation on the Asphalt Section

The permanent deformation at various stages of HVS trafficking on the asphalt section (346A4) is indicated in Figure 8. Up to 2.8 MISA, the average deformation was approximately 5.5 mm. From 2.8 MISA, the test wheel load was increased to 70 kN, afterwhich the rate of deformation increased. In Figure 9 the permanent deformation at various depths in the pavement is illustrated, and the figure shows that approximately 50 % of the total surface deflection originated from the top 225 mm of this pavement. On MDD4, almost zero deformation resulted from the asphalt surfacing, whilst at MDD12, approximately one (1) mm deformation originated from the asphalt surfacing. The rate of deformation (R_r) on the surface of the pavement changed from an initial 3.8 mm/MISA to 1.1 mm/MISA up to 2.8 MISA, then increased to an initial 7.1 mm/million 70 kN repetitions, and then reduced to approximately 3.0 mm/million 70 kN repetitions. Testing has only been done in the dry state.

Table 4 Back-calculated effective elastic moduli (Section 339A4)(Otte, 1990)

	Elastic Moduli - Ranges (MPa)	
Layer	Dry State	Wet State
G1-Base	300-600	300-600
C3/EG4 Upper Subbase	800-2000	200
C3/EG4 Lower Subbase	100-200	50
Selected and Subgrade Layers	100-200	50

Resilient Deflection Response of the Asphalt section

In Figure 10 the standard 40 kN surface deflection at various stages of HVS traffic on this section is illustrated. The figure indicates that the initial deflection was approximately 400 µm, and rapidly increased to a steady level around 700 µm. At approximately 1.5 MISA, the deflection increased slowly to approximately 750 µm. This increase, however, may be attributed to an increase in average surface temperature from approximately 20 °C approximately 40 °C. Limited deflections measurements on this pavement indicated that it increased to approximately 2 µm/°C, and that it may be necessary to do temperature corrections (ie standardise deflections at a specific temperature, see also De Beer, 1991b), before comparisons can be made. The increase in surface deflection originated from the total pavement structure as is reflected in the depth deflection (MDD) measurements illustrated in Figure 11. The figure indicates that most of the increase in deflection originated from within the selected and subgrade layers. Some of the deflections measured on the surface of the section with the RSD instrument, indicate lower deflections than at the bottom of the asphalt, which is believed to be a result of dilation (swelling) of the asphalt between the dual wheels of the HVS loading configuration. With the trafficking load at 70 kN, the standard 40 kN deflection increased to approximately 830 µm.

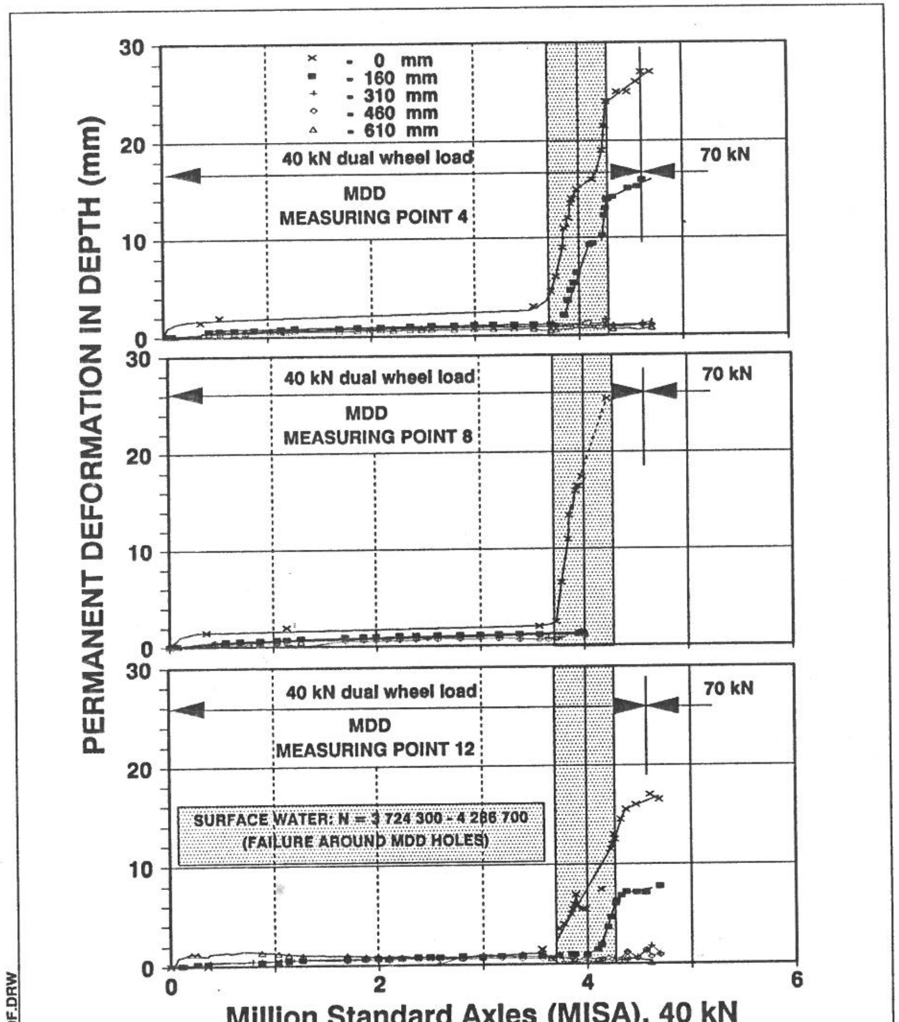

FIGURE 5

PERMANENT DEFORMATION AT VARIOUS DETPHS AS MEASURED WITH THE MDDs ON HVS SECTION 339A4

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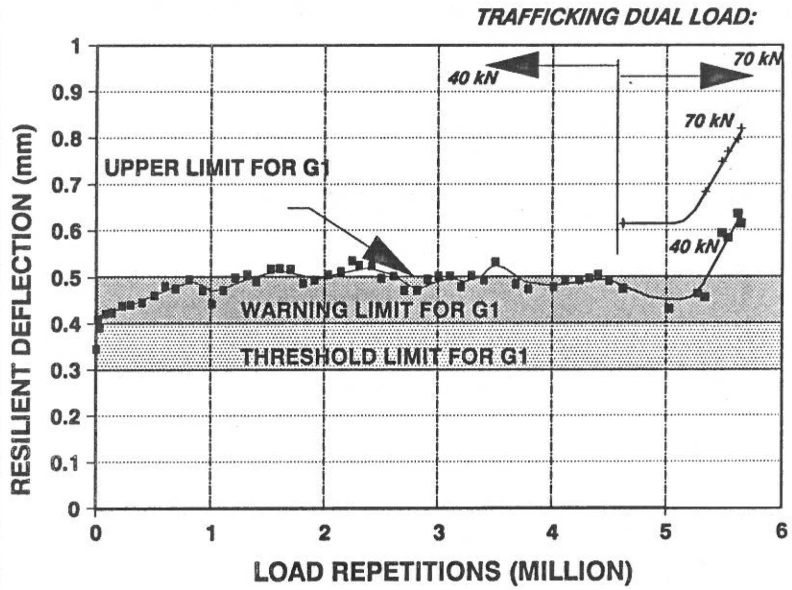

■ AVERAGE OF POINTS 3-13 @ 40 kN
\+ AVERAGE OF POINTS 3-13 @ 70 kN

FIGURE 6

AVERAGE RESILIENT (RSD) DEFLECTION ON THE G1 BASE SECTION AT VARIOUS STAGES OF HVS TRAFFICKING

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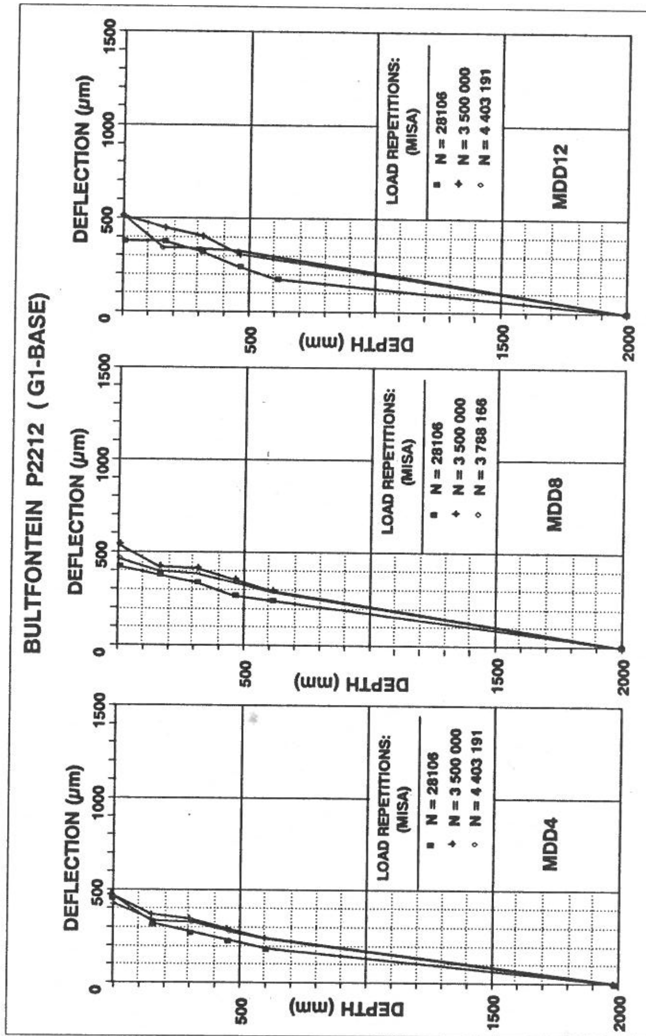

FIGURE 7

MULTI-DEPTH DEFLECTIONS AT MDD MEASURING POINTS 4, 8 AND 12 ON THE G1-BASE SECTION AFTER VARIOUS STAGES OF HVS TRAFFICKING (40 kN)

HVS SECTION 346A4, BULTFONTEIN, ROAD 2212
35 mm CONTINUOUSLY GRADED ASPHALT SURFACING

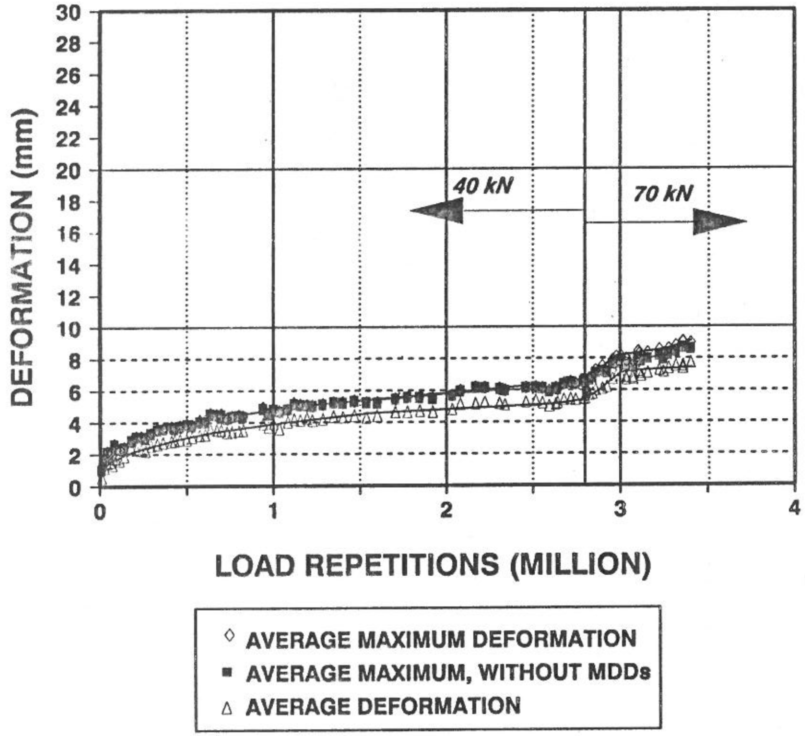

FIGURE 8

PERMANENT DEFORMATION ON THE 35 mm ASPHALT SURFACING SECTION AT VARIOUS STAGES OF HVS TRAFFICKING

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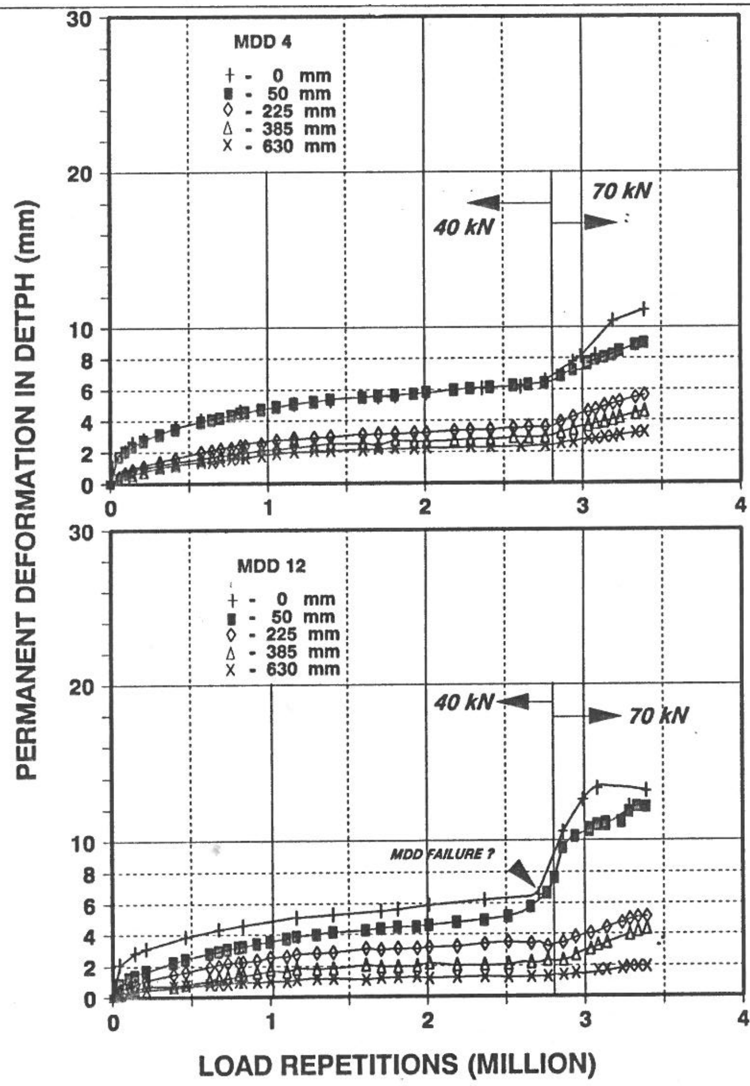

FIGURE 9

PERMANENT DEFORMATION AT VARIOUS DEPTHS AS MEASURED WITH THE MDDs ON SECTION 346A4

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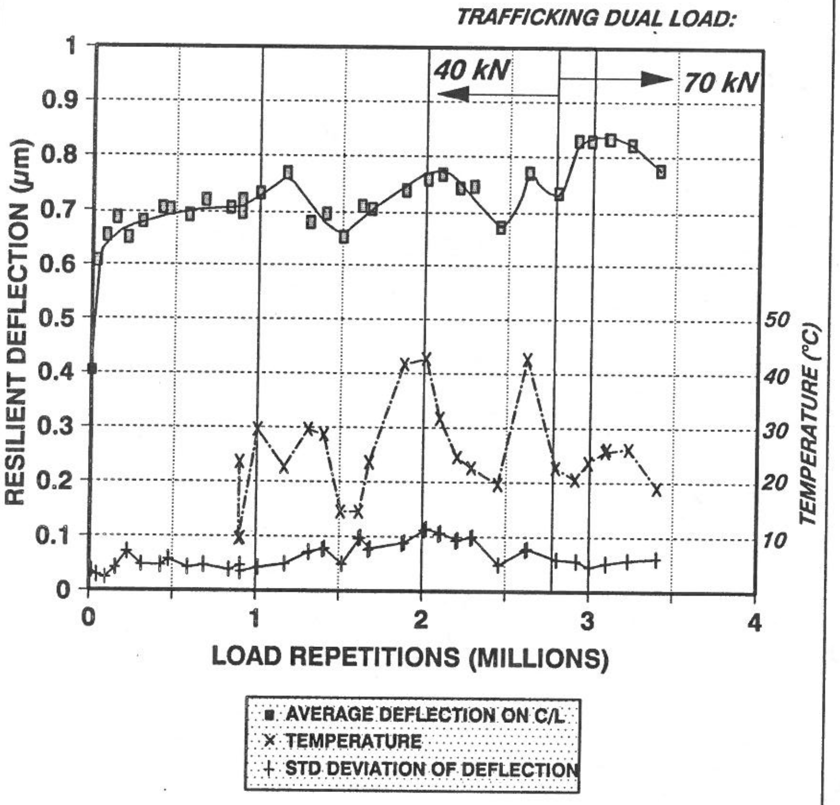

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FIGURE 10

AVERAGE STANDARD (40 kN) RESILIENT DEFLECTION ON THE 35 mm ASPHALT SURFACING SECTION AT VARIOUS STAGES OF HVS TRAFFICKING (TEMPERATURE INCLUDED)

M DE BEER

MDD339F.DRW

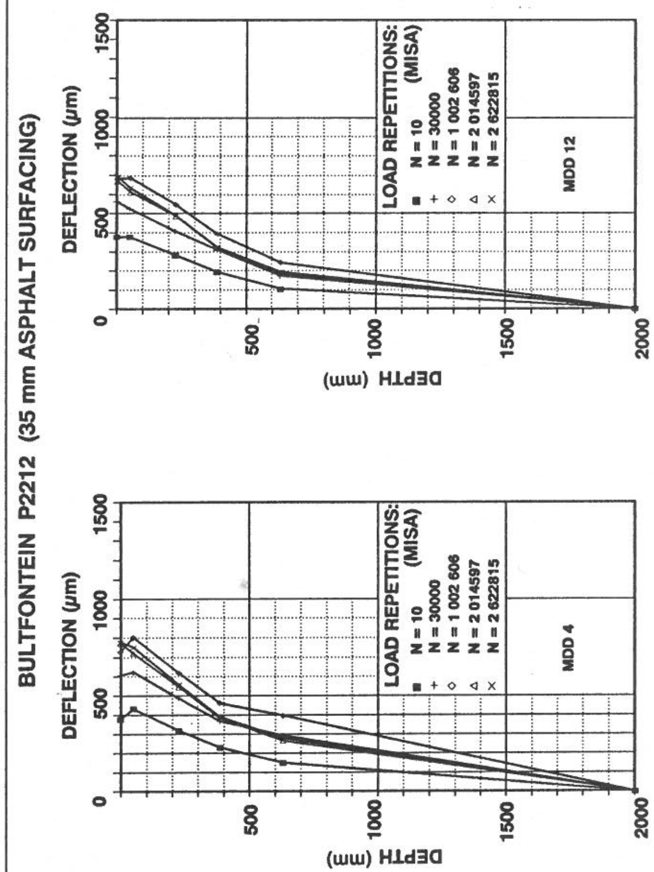

FIGURE 11

MULTI-DEPTH DEFLECTIONS AT MDD MEASURING POINTS 4 AND 12 ON THE 35 mm ASPHALT SECTION AFTER VARIOUS STAGES OF HVS TRAFFICKING (40 kN)

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Crack Development on the Asphalt Section

Several cracks developed on this section after approximately 1.4 MISA. The cracks appear to be fatigue related, and occurred both in the longitudinal and transverse directions to the direction of trafficking, and crack length versus number of repetitions increased with an increase in trafficking load (Wolff, 1991a). A detailed investigation by Wolff (1991b) indicated that the cracks originated from the top as well as from the bottom of the asphalt layer. Studies on asphalt cores from cracked positions (cracks initially treated with fluorescent dye and then drilled), indicated that the majority of cracks visible on the surface were not continuous to the bottom and therefore independent from those at the bottom, and visa versa. Further, the cracks from the surface appear to extend deeper than those from the bottom of the layer. Modelling of this pavement structure using the Mechanno Lattice method proposed by Yandell (1990) predicted crack growth from the top of the asphalt layer. However, this modelling was based on rut development from the HVS test data, and not from tri-axial data as proposed by Yandell (1990). Further, the origin of cracks is highly dependent on the relative plasticities of the asphalt and its supporting layer, which should be very accurately measured before modelling. At this stage it can be concluded from the crack study that current existing models do not predict the time to cracking on this HVS section adequately (including normal mechanistic models bases on linear elastic theory, and models developed by Strauss et al (see BKS, 1982, 1987)). Further research is needed to adequately predict origin and rate of crack growth in asphalt surfacings. Work is, however, continuing at DRTT on this aspect.

SUMMARY AND CONCLUSIONS

In this paper the full scale pavement research programme with the TPA Heavy Vehicle Simulator (HVS04) is updated (ATC'85, Kleyn et al, 1985). Since 1985 the research concentrated on the structural behaviour of pavements incorporating cementitious base and subbase layers, and currently a detailed study on various rehabilitation options on road 2212 (near Bultfontein) is performed. The research findings on the existing pavements included aspects such as:

- Enhanced DCP based technology
- Crushing failure of the cementitious base of a deep pavement structure
- Initial fatigue failure of the cementitious base of a shallow pavement structure, then crushing failure similar to deep pavements
- Structural capacity up to 5 Million Standard Axles (MISA), if tyre pressure and overloading is controlled on these pavements.

Follow-up research on various rehabilitation options on these pavements included:

- Successful cracking of an existing shallow pavement using a "crack-and-seat" method with a vibratory roller.
- Control measurements: DCP, deflection, density and deformation
- Full scale accelerated structural evaluation (HVS) of two rehabilitation options, viz. 150 mm G1-base and a 35 mm asphalt surfacing
- Structural capacity of more than 10 MISA for the G1-base section predicted
- Asphalt cracking after 1.4 MISA, with rutting < 10 mm.
- Cracks in asphalt may originate also from the top of the asphalt surfacing

From this research, although some of the recent tests are not completed yet, it can be concluded that the test programme of this HVS provided South African pavement engineers with valuable new information regarding pavements incorporating cementitious layers, as well as enhanced in-situ pavement evaluation methods. On the other hand, the investment in the HVS research programme can only be as cost effective as the pavement technology is updated and applied.

RECOMMENDATIONS

The following is strongly recommended:

- The extension of the "information matrix" for accelerated testing, regarding the behaviour of our local pavements be continued with HVS04
- Continuation on research regarding the correct modelling of pavement materials, including granular materials and pavement dynamics (Lourens, 1991).
- Further studies of cracking and its origin in thin asphalt surfacings. These studies should also concentrate on the use of bitumen - rubber (BR) binders on certain new, as well as selected pavements with active cracks. The current use of BR binders by the TPA is already 30 % compared to conventional binders, and is still increasing.

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APPENDIX A: DETAIL LABORATORY DETERMINED CHARACTERISTICS OF THE G1 MATERIAL

PARAMETER	RANGE	UNITS	MOISTURE CONTENT (%)
K1 *	2.148 to 4.210	MPa	1.7 to 3.5
K2 *	0.350 to 0.845	-	1.7 to 3.5
COHESION, C	0 to -903	kPa	1.7 to 3.5
FRICTION ANGLE ϕ	46.7-63.0	Degrees	1.7 to 3.5
POISSON'S RATIO,	0.1 to 0.36	-	1.7 to 3.5
LIQUID LIMIT, LL (%)	0.25	(%)	
LINEAR SHRINKAGE, LS (%)	0.00	(%)	
PLASTICITY INDEX, PI (%)	0.00	(%)	
CRITICAL MOISTURE CONTENT, CMC (%)	2.09	(%)	
OPTIMUM MOISTURE CONTENT, OMC (%)	3.96	(%)	
MAXIMUM DRY DENSITY (kg/m^3)	2632 2481 (Mod. AASHTO)	(kg/m^3)	
GRADING:	SIEVE SIZE (mm)	% PASSING	
	75	100.00	
	63	100.00	
	53	100.00	
	37.5	98.00	
	26.5	69.00	
	19.0	52.00	
	13.2	44.19	
	4.75	38.00	
	2.00	28.00	
	0.425	13.00	
	0.075	6.50	
BULK RELATIVE DENSITY, BRD (kg/m^3)	2875	(kg/m^3)	

* Resilient Modulus: $M_R = K1(\Theta)^{K2}$, with $\Theta = \sigma_1 + \sigma_2 + \sigma_3$