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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 the Dynamic Cone Penetrometer (DCP) are also described. Detail information regarding various rehabilitation options for these pavements are given together with their HVS performance.

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 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

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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-rich) 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 medium to heavy granular pavement structures (1980-1982)
- Evaluation of strongly cemented base pavements (1982-1984)
- Evaluation of lightly cementitious base pavements (1984-1986)
- 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 findings with the HVS on lightly cementitious pavements is given (1984-1988) 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, i.e. road 1932, near Roodeval. 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 Kleytn (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, i.e. road 2212 near Buffelsfontein, 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 reduces 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 (% found on these pavements) (see also Kleytn et al., 1988). 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 (MOD) 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, TR44, 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, i.e. crushing of the base. Most of the results on both the deep and shallow pavements on relatively good subgrades (in situ CSR > 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 (N)). A linear relationship was established between the ratio of tyre contact stress and UCS (s/UCS), with the log of the number of stress repetitions to initiate this crushing failure (Ne). 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 (s/Ne) and the log of the number of strain repetitions to effective fatigue (Ne). 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 "flip" to initiation of crushing failure (Nc) is a reasonable estimate of the fatigue cracking life of the surface treatment. Therefore, lifec cycle based maintenance strategies, based on this criterion, can be better planned for these types of pavements in order to seal
ASPECTS OF REHABILITATION OF PAVEMENTS WITH LIGHTLY CEMENTIZED LAYERS

Background

As a result of extensive research and associated field trials during the 1980s and 1990s, it was observed that the use of lightly cementated layers in pavement repair was a viable and cost-effective solution. These layers can be designed to have different properties, depending on the specific needs of the project. This type of rehabilitation is particularly useful in areas where heavy traffic or severe climate conditions can lead to rapid deterioration of the pavement.

"Crack and Seal" (C & S) Operation

This technique involves the insertion of a narrow crack seal into the existing pavement to prevent water from entering the cracks. The crack seal is typically made of a rubber-based material and is applied using a hot-melt gun. Once the seal is cured, a calibrated load is applied to the pavement to achieve the desired deformation.

Vibratory Rolling

Following the crack and seal application, vibratory rolling is used to compact the seal and ensure that it is fully anchored within the pavement. This process helps to improve the bond between the seal and the existing pavement, ensuring that it remains effective in preventing water ingress.

Surface Deflection

Surface deflection measurements are taken using a digital deflection plate. The deflection plate is placed on the surface of the pavement, and a load is applied to induce deformation. The deflection is then measured using a digital gauge. This method allows for the accurate assessment of the pavement's response to traffic loads, which is crucial for determining the effectiveness of the rehabilitation measures.

The results of these measurements are then compared to the established criteria for acceptable performance. If the deflection values exceed the specified limits, additional measures may be required to ensure the longevity and safety of the pavement.
FIGURE 2
PERMANENT DEFORMATION AS A RESULT OF VARIOUS ROLLER APPLICATIONS ON ROAD 2212 (SECTIONS P1 AND P4)

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 67.7 % was achieved during construction. After 4 days the slushing process started. The slushing was done with the same roller (without vibration). A clear mastic 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 13.2 mm/6.7 mm 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 indicated 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) before the application of the granular base layer.

Table 1 DCP parameters on roller sections P1 and P4 (De Beer et al, 1988)

<table>
<thead>
<tr>
<th>ROLLER SECTION P1</th>
<th>ROLLER APP.</th>
<th>DS/N00 (Blows)</th>
<th>A-PARAMETER</th>
<th>B-PARAMETER</th>
<th>AVERAGE DCP PENETRATION RATES, DN (mm/blow)</th>
<th>DEPTH (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0-96</td>
<td>99-298</td>
</tr>
<tr>
<td>16</td>
<td>234</td>
<td>1479</td>
<td>35</td>
<td>3.9</td>
<td>3.8</td>
<td>5.2</td>
</tr>
<tr>
<td>1690</td>
<td>250</td>
<td>600</td>
<td>19</td>
<td>1.5</td>
<td>2.9</td>
<td>4.9</td>
</tr>
<tr>
<td>396A**</td>
<td>243</td>
<td>1629</td>
<td>28</td>
<td>2.0</td>
<td>3.5</td>
<td>5.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>ROLLER SECTION P4</th>
<th>ROLLER APP.</th>
<th>DS/N00 (Blows)</th>
<th>A-PARAMETER</th>
<th>B-PARAMETER</th>
<th>AVERAGE DCP PENETRATION RATES, DN (mm/blow)</th>
<th>DEPTH (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0-96</td>
<td>99-298</td>
</tr>
<tr>
<td>10</td>
<td>236</td>
<td>2298</td>
<td>29</td>
<td>1.3</td>
<td>4.2</td>
<td>7.1</td>
</tr>
<tr>
<td>2400</td>
<td>227</td>
<td>1/91</td>
<td>22</td>
<td>1.9</td>
<td>2.8</td>
<td>5.3</td>
</tr>
<tr>
<td>396A**</td>
<td>267</td>
<td>2304</td>
<td>27</td>
<td>1.3</td>
<td>5.2</td>
<td>5.3</td>
</tr>
</tbody>
</table>

* 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 surfaced was constructed using a normal asphalt paver. The 13.2 mm/6.7 mm surface treatment (S12), 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 surfaceing, 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

Initially 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

<table>
<thead>
<tr>
<th>HVS TEST NO.</th>
<th>TYPE OF REHAB.</th>
<th>DUAL TEST LOAD (kN)</th>
<th>TYPRE PRESSURE (kPa)</th>
<th>REPEETIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>339A4</td>
<td>150 mm G1-BASE</td>
<td>40</td>
<td>520</td>
<td>4,854,451</td>
</tr>
<tr>
<td>240A4</td>
<td>150 mm G1-BASE</td>
<td>70</td>
<td>520</td>
<td>1,918,558</td>
</tr>
<tr>
<td>346A4</td>
<td>35 mm ASPHALT</td>
<td>60</td>
<td>520</td>
<td>2,800,000</td>
</tr>
<tr>
<td>347A4</td>
<td>35 mm ASPHALT</td>
<td>70</td>
<td>520</td>
<td>800,000</td>
</tr>
<tr>
<td>348A4</td>
<td>SURFACE TREAT.</td>
<td>60</td>
<td>520</td>
<td>50,000*</td>
</tr>
<tr>
<td>349A4</td>
<td>SURFACE TREAT.</td>
<td>70</td>
<td>520</td>
<td>50,000*</td>
</tr>
</tbody>
</table>

* 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 MSA 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 (Rd) and relative damage, based on the well known power law viz.:

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HVS SECTION 339A4, BULTFONTEIN, ROAD 2212
150 mm CRUSHED DOLOMITE (G1) BASE

40 kN DUAL WHEEL LOAD

70 kN

DEFORMATION (mm)

LOAD REPETITIONS (MILLION)

MAX AVERAGE RUT  MAX AVERAGE RUT EXCLUDING MDDs  AVE RUT

SURFACE WATER: N = 3 724 300 - 4 286 700

SURFACE WATER: N = 5 488 900 - 5 653 000

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

\[ D_2 = (P/40)^d \]

where \( D_2 \) = 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 (i.e. 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>
<thead>
<tr>
<th>TEST LOAD (kN)</th>
<th>RATE, ( R_1 ) **</th>
<th>RATE, ( R_2 ) **</th>
<th>RATE, ( R_3 ) ***</th>
<th>RATE, ( R_4 ) ***</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(mm/10⁶ rep.</td>
<td>(mm/10⁶ rep.</td>
<td>(mm/10⁶ rep.</td>
<td>(mm/10⁶ rep.</td>
</tr>
<tr>
<td>40</td>
<td>0.54</td>
<td>16.00</td>
<td>0.54</td>
<td>11.54</td>
</tr>
<tr>
<td>70</td>
<td>2.34</td>
<td>48.00</td>
<td>2.34</td>
<td>48.00</td>
</tr>
</tbody>
</table>

RELATIVE DAMAGE BASED ON \( D_2 = (P/40)^d \)

\[ D_2 = 4.33 \quad \text{and} \quad 2.62 \]

\[ d = 1.99 \quad \text{and} \quad 2.58 \]

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 \( \mu \)m, and increased to a steady level (upper limit) around 500 \( \mu \)m. According to TPA practice based on experience, the current threshold and warning levels of maximum standard deflection are 300 \( \mu \)m and 400 \( \mu \)m, respectively. An increase in deflection to approximately 640 \( \mu \)m 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 $\alpha$ test, where $\alpha$ is automatically applied pro rata to the increase in $\alpha$, the vertical stress. Detail of this apparatus can be found elsewhere (Sommerlin et al., 1991a, 1991b). For the G1 material the following parameters were determined: $K_1$ and $K_2$ in $K16^\circ$; cohesion $C$ and angle of internal friction $\phi$ and range of Poisson's Ratio, $\mu$. 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 (339A4) 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, after which 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) 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)

<table>
<thead>
<tr>
<th>Layer</th>
<th>Dry State</th>
<th>Wet State</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1-Base</td>
<td>300-600</td>
<td>300-600</td>
</tr>
<tr>
<td>C3/EO4 Upper Subbase</td>
<td>800-2000</td>
<td>200</td>
</tr>
<tr>
<td>C3/EO4 Lower Subbase</td>
<td>100-300</td>
<td>50</td>
</tr>
<tr>
<td>Selected and Subgrade</td>
<td>100-200</td>
<td>50</td>
</tr>
</tbody>
</table>

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 $\mu$m, and rapidly increased to a steady level around 700 $\mu$m. At approximately 1.5 MISA, the deflection increased slowly to approximately 750 $\mu$m. This increase, however, may be attributed to an increase in average surface temperature from approximately 20°C approximately 40°C. Limited deflection measurements on this pavement indicated that it increased to approximately 2 $\mu$m/$^\circ$C, and that it may be necessary to do temperature corrections (e standard material 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 it 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 dilatation (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 $\mu$m.

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FIGURE 5

PERMANENT DEFORMATION AT VARIOUS DEPTHS AS MEASURED WITH THE MDDs ON HVS SECTION 339A4
FIGURE 6
AVERAGE RESILIENT (RSD) DEFLECTION ON THE G1 BASE SECTION AT VARIOUS STAGES OF HVS TRAFFICKING
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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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 increased in trafficking load (Wolf, 1991a). A detailed investigation by Wolf (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 vice 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 Mechanico Lattice model 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 the HVS section adequately (including normal mechanic models based 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 Bulfontein) 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.

M DE BEER

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 road, 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.

ACKNOWLEDGEMENTS

Acknowledgement is made to the Deputy Director General (DDG) of the Roads Branch of the TPA, for his continued support and appreciation of this work. The Director of the Division of Roads and Transport Technology (DRTT), and DDG, TPA Roads Branch, is thanked for their permission to publish this paper.

REFERENCES


APPENDIX A: DETAIL LABORATORY DETERMINED CHARACTERISTICS OF THE G1 MATERIAL

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>RANGE</th>
<th>UNITS</th>
<th>MOISTURE CONTENT (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1</td>
<td>2.146 to 4.210</td>
<td>MPa</td>
<td>1.7 to 3.5</td>
</tr>
<tr>
<td>K2</td>
<td>2.46 to 4.585</td>
<td></td>
<td>1.7 to 3.5</td>
</tr>
<tr>
<td>COHESION, C</td>
<td>0 to 903</td>
<td>kPa</td>
<td>1.7 to 3.5</td>
</tr>
<tr>
<td>FRICTION ANGLE</td>
<td>45 to 63.0</td>
<td>Degrees</td>
<td>1.7 to 3.5</td>
</tr>
<tr>
<td>POISSON RATIO</td>
<td>0.1 to 0.36</td>
<td></td>
<td>1.7 to 3.5</td>
</tr>
<tr>
<td>LIQUID LIMIT, LL (%)</td>
<td>0.25 (4%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LINEAR SHRINKAGE, LS (%)</td>
<td>0.00 (4%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PLASTICITY INDEX, PI (%)</td>
<td>0.00 (4%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CRITICAL MOISTURE CONTENT, CMC (%)</td>
<td>2.09 (4%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OPTIMUM MOISTURE CONTENT, CMC (%)</td>
<td>3.06 (4%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MAXIMUM DRY DENSITY (kg/m³)</td>
<td>2932 2941 (Mod. AASHTO)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

GRADING:

<table>
<thead>
<tr>
<th>SIEVE SIZE (mm)</th>
<th>% PASSING</th>
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<tbody>
<tr>
<td>75</td>
<td>100.00</td>
</tr>
<tr>
<td>63</td>
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<tr>
<td>53</td>
<td>100.00</td>
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<tr>
<td>37.5</td>
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<tr>
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<tr>
<td>4.75</td>
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<tr>
<td>2.00</td>
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<tr>
<td>0.425</td>
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<tr>
<td>0.275</td>
<td>6.50</td>
</tr>
<tr>
<td>BULK RELATIVE DENSITY, RH (kg/m³)</td>
<td>2875</td>
</tr>
</tbody>
</table>

* Resilient Modulus: Mₐ = K₁[1(θ)(θ)], with θ = θ₀ + θ₁ + θ₂