Evaluation of a nano-silane-modified emulsion stabilised base and subbase under HVS traffic

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Synopsis-This paper describes the findings of a Heavy Vehicle Simulator (HVS) test on road D1884 in the Gauteng province of South Africa. The pavement structure was rehabilitated using an anionic nano-silane modified bitumen emulsion stabilising agent in the base and sub-base. The rehabilitation design traffic loading (20 year design period) is based on 3 million Equivalent Standard 80kN Axle Loads (ESALs). The 50 year old pre-rehabilitated road exhibited severe distress with in-situ materials in the upper pavement layers that have weathered to a tested G8 quality – materials usually considered unsuitable for use in base/sub-base layers of a pavement structure. The HVS test subjected the pavement structure to an equivalent of at least 3.5 million ESALs (using calculated damage factors). Using the standard damage factor of 4,2, 7.5 million ESALs were applied. Although the dual wheel load was increased to 80 kN and water introduced (in depth as well as on the surface), no structural failure could be induced during the test (a final rut of only 8 mm was measured). The in-situ stabilisation of available materials using an anionic nano-silane modified bitumen emulsion compared with the standard approach of importing high quality materials realised a saving of 43%. The HVS test was conducted on a site where materials of "unacceptable" quality stabilised with a laboratory proven technology were used. This provided road authorities with quick results for the consideration of the future use of this technology which potentially embodies the cost-effective service delivery of high-quality roads.

Keywords: Anionic nano-silane modified bitumen emulsion, modified emulsion-treated materials, HVS testing, APT, cost-effective roads

I. INTRODUCTION

The significant cost associated with upgrading, maintaining and rehabilitating of road infrastructure severely impacts the life-cycle cost and capacity of South Africa's transport network. The Council for Scientific and Industrial Research (CSIR) is currently conducting and planning several studies associated with understanding the behavioural characteristics of innovative, modified road construction materials that could potentially be significantly more cost-effective than standard designs and materials. In order to determine the structural capacity of these modified road materials under different traffic demand classes and environmental conditions, a series of Heavy Vehicle Simulator (HVS) tests have been planned on different pavement classes across Gauteng, such as that on Provincial Road D1884 which was rehabilitated using an anionic nano-silane modified bitumen emulsion stabilising agent for the in-situ stabilisation of the base and sub-base of a pavement structure with an ES3 design (1 to 3 Million Equivalent Standard 80kN Axle Loads (ESALs) over its design life). This was a bespoke design specifically for the chemistry of the virgin materials before modification whilst taking traffic and environmental conditions into consideration. The HVS programme and its impact is described elsewhere [1], [2].

This paper describes the findings of a Heavy Vehicle Simulator (HVS) test of road D1884 near Meyerton in the Gauteng province of South Africa. The pavement structure incorporated a nano-silane-modified emulsion in the base and subbase and was designed for 3 million Equivalent Standard 80kN Axle Loads (ESALs). The virgin material used in the base and subbase was classified as a G8, which is usually unsuitable for use in a base or subbase. The objective of this test was to assess whether this material, after being stabilised with silane-modified emulsion, can be used effectively as base and/or sub-base material without needing to haul expensive good quality materials to the site. This paper summarises the findings from the laboratory and HVS testing. Detailed analysis of the data is provided by Rust *et al.* [3].

II. BRIEF LITERATURE REVIEW

The 'nanotechnology' field refers to material particles at a nanoscale. Steyn [4] indicated that, nanotechnology is applicable in a number of science fields including physics and chemistry but has also found a purpose in road building materials. According to Kelsall *et al* [5] nanotechnology can be defined as the knowledge of the design, construction and utilisation of functional structures with at least one characteristic dimension measured in nanometres (nm). The minute size of the particles yields a significant change in the nature, function and properties of nano-materials. Much of this knowledge has however, not been utilized by road design professionals in South Africa [6].

It is well known that clays are problematic in road building materials [5]. It has been shown that emulsion is very suitable for the stabilisation of materials containing smectite and mica as well as some cohesion-less sands [7]. Road building materials modified with nano-technology (such as a nano-silane-modified emulsion) should therefore be characterized through X-ray diffraction (XRD), to allow the designer to alter the modifying agent to be suitable for a specific material [8].

Polymer-modified bitumen emulsions are also widely used in road construction and can increase the viscosity and stiffness of the bitumen. However, it was noted that this modification type may not improve elasticity and elastic strength properties [9, 10].

Jordaan *et al* [8] provided a preliminary material classification for nano-modified emulsions (NMEs) ranging from the highest quality NME1 (comparable to a G1 base), to NME4 (typically containing 0.5% to 0.7% modified emulsion) which is comparable to a G4 material. Material in the NM-EG5 classification requires a base material quality of less than that of a G5 material. The classification by Jordaan [8] calls for, *inter alia*, ITS values ranging from greater than 175 kPa (NME1) to greater than 80 kPa (NME4) in the dry state. Jordaan [8] furthermore indicates that NMEs should be treated differently from normal road building materials due to the fact that they do not conform to normal specifications for road building materials. In addition, nano-silanes and nano-polymers contain chemicals that require alternative protocols for sample preparation and testing.

Nano-silane-modified emulsion has been used on a number of projects in South Africa with satisfactory early field performance. This includes road D1884 (the topic of this paper), the K46 William Nicol drive in Johannesburg, the rehabilitation of Rural Road D3718 in the Limpopo province of South Africa, and road D29 in Swaziland (north of Manzini) [11].

The Central Roads Research Institute in India reported positive results from the use of nan-modification in both asphalt and soils [12]. NCAT reported positively on the use of a diluted cationic emulsion containing a nano-technology additive in tack coats, particularly as to their moisture resistance [13].

III. PAVEMENT DESIGN

The conventional design for ES3 class roads used by the Gauteng Province and the alternative design using nano-silane modified bitumen emulsion are shown in Figure 1 below.

The purpose of this project was to evaluate the bearing capacity and structural strength of the pavement constructed as part of the D1884 rehabilitation project which has been constructed using a nano-silane-modified emulsion material. Furthermore, the performance when used to improve the quality of marginal quality materials (i.e. poor-quality materials which will generally not be used in a road base or subbase) was evaluated. The virgin material was classified as a G8 material. The stabilising agent was a specially designed (material compatible) anionic nano-silane modified bitumen emulsion. It consists of an anionic SS60 bitumen emulsion with a emulsifying agent consisting of a Sodium Hydroxide basic molecule with a alkyl group consisting of a $CH_3(CH_2)_n$, carbon-chain where n > 15. (Vinsol resin or equivalent). Jordaan *et al* [8] reported the details of recommended project specifications used for the rehabilitation of Road D1884. Since no design failure criteria are currently available for nano-silane modified bitumen emulsion stabilising agents the criteria developed from HVS tests using unmodified bitumen emulsion were used [14]. These failure criteria are considered conservative or "safe" for implementation, since the behaviour characteristics of the nano-silane modifications are expected to be considerably better. One of the outputs expected from this and future HVS tests is to either confirm these design curves or recommend adjustment to the analysis of normal bitumen emulsion stabilisation compared to that of anionic nano-silane modified bitumen emulsion. These recommendations should enable engineers to (in future) further optimise designs using this technology.

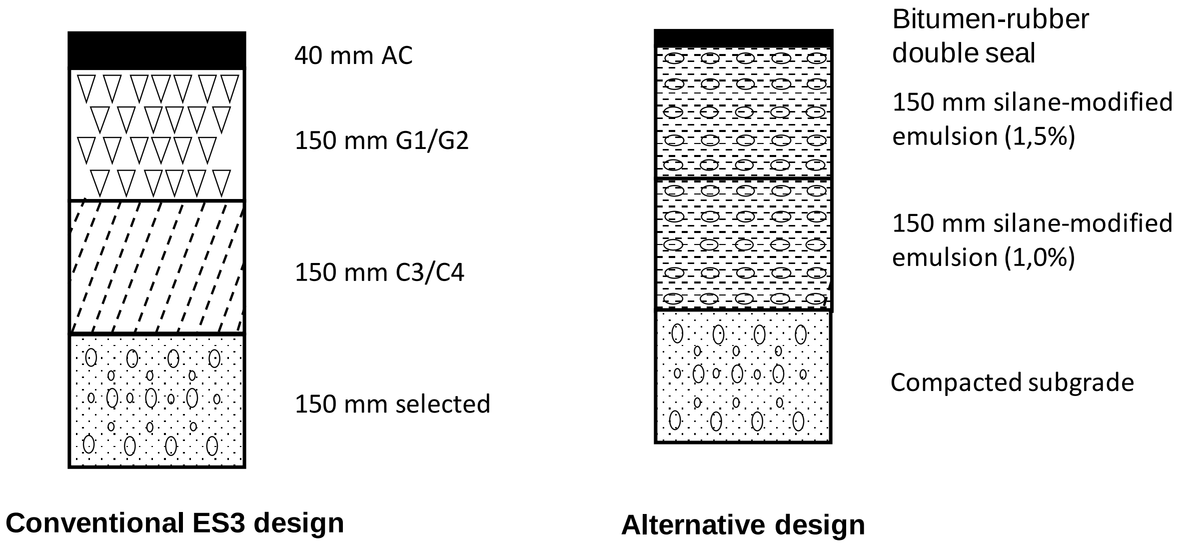

Fig. 1. Conventional and alternative designs for the D1884

IV. Methodology and testing regime

Laboratory work was conducted to characterize the materials in the pavement structure before and after modification. During the HVS test standard (80kN axle load) and a high load (120 – 160kN axle loads) tests were used to study the bearing capacity, structural strength and load sensitivity of this structure in dry and wet conditions. The following load applications were applied to the D1884 HVS road section:

- 321,350 repetitions of a 40 kN dual wheel load (80 kN axle load)
- 372,600 repetitions of a 60kN dual wheel load (120kN axle load)
- 96,882 repetitions of an 80 kN dual wheel load (160 kN axle load) in the dry state, and
- 155,649 repetitions of an 80 kN dual wheel load (160 kN axle load) in the wet state.

The total number of repetitions applied was 946,481. Water was applied both in depth through holes drilled at depths of up to 400 mm as well as on the surface while applying an 80 kN wheel load. The test section layout is shown in Figure 2.

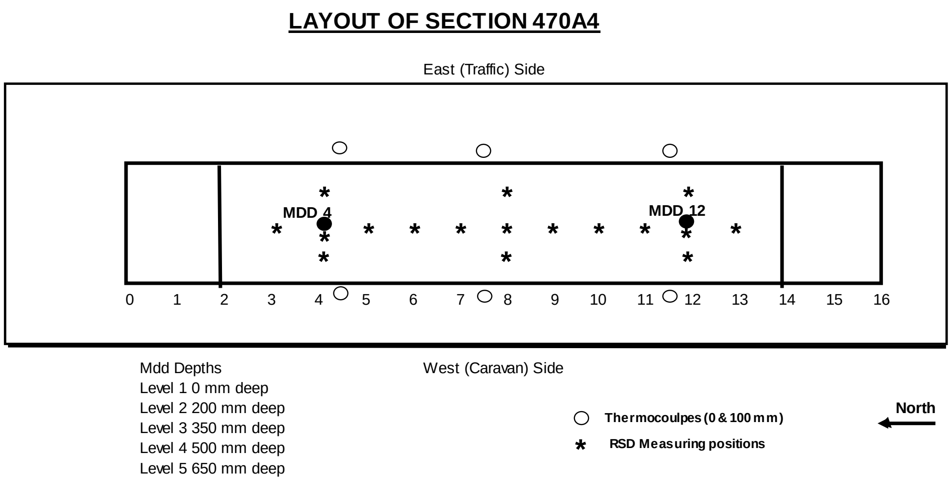

Fig. 2. Layout of the HVS test section

From Figure 2 it can be noted that the following measurements were taken during HVS testing:

- Profilometer measurements to determine the surface deformation across the test section;
- Road surface deflectometer (RSD) readings to determine the surface deflection characteristics;
- Multi-depth deflectometer (MDD) readings at two points, and
- Temperature readings using thermocouples on the test section as well as a control section.

Light weight deflectometer readings were taken before the test commenced on both the test section and a control section. DCP analyses were conducted after the test, both outside the test section ("before" state) and inside the test section ("after" state). Post-test laboratory testing was conducted on cores taken both outside and inside the test section.

In addition to the water added manually to the test section, significant rainfall was recorded during the HVS test (see Figure 3).

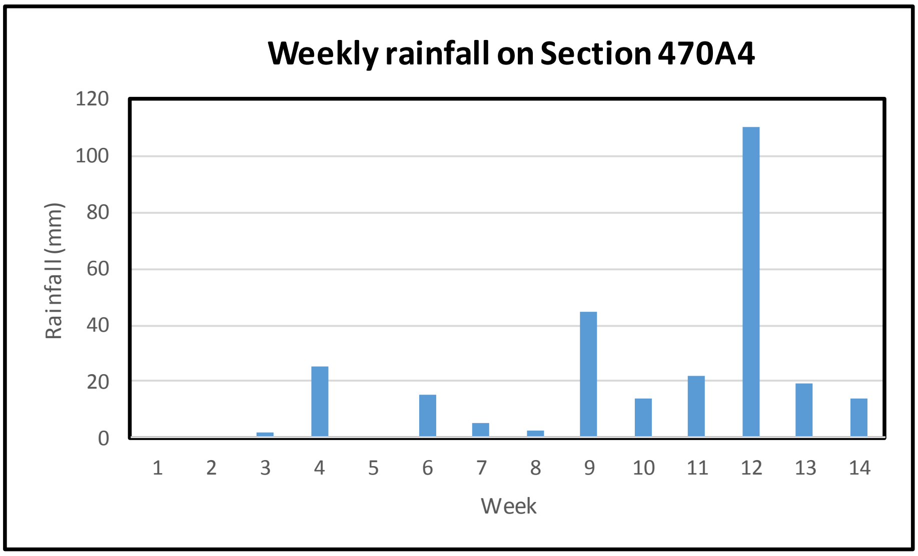

Fig. 3. Weekly rainfall during the HVS test

V. Materials Testing

A summary of the materials testing prior to modification is given in Table 1 below. It can be noted that the materials were of very low quality for use in a base and sub-base supported by sub-grade with a high clay content in each layer. Special attention needs to be drawn to the percentage passing through the 0.075 mm sieve size as these results have a significant influence on the modified stabilising agent implemented. These results are indicative of considerable weathering due to chemical decomposition over the past 50 years since the road was originally constructed. In fact, material from this site presented design engineers with considerable challenges and this work could well form a basis for the laboratory analysis of any potential material stabilisation/modification product earmarked for future laboratory testing and evaluation.

TABLE I. MATERIALS TESTING RESULTS PRIOR TO STABILISATION

		Base layer	Subbase-layer	Upper selected
Sieve analysis (mm)	63.0	100	100	100
	53.0	100	95	96
	37.5	100	82	89
	28.0	100	76	88
	20.0	93	72	88
	14.0	77	65	88
	5.0	53	56	86
	2.00	46	50	83
	0.475	32	43	75
	0.075	13	22	32
	SANS 3001 - PRS			
Fine Sand		4	10	17
Silt & Clay		33	44	39
Atterberg Limits	LL (%)	24	23	21
	PI	7	4	3
	LS (%)	3.5	2.1	1.4
GM	GM	2.09	1.85	1.1
Mod-AASHTO	OMC%	7.2	11.4	9.1
	MDD	2170	2030	1967
CBR	Comp MC	7	11.2	8.8
	% Swell	0.16	0.29	0.52
CBR @ density	100%	27	32	20
	98%	22	28	15
	97%	19	26	13
	95%	15	23	9
	93%	12	20	7
	90%	9	16	4
TRH14 material classification		G8	G7	G10

The laboratory material design results for road D1884 in terms of Unconfined Compressive Strength (UCS) (dry and wet) and Indirect Tensile Strength (ITS) (dry and wet) were conducted using various percentage of anionic nano-silane modifications in combination with various percentages of anionic SS60 bitumen emulsions. A summary of the optimum design using an anionic nano-silane modified bitumen emulsion with the associated engineering properties in terms of compressive and tensile strengths are given in Table II. The test procedures followed are described in detail elsewhere [16].

TABLE II. DESIGN LABORATORY RESULTS

Material	ITS_{dry} (kPa)	ITS_{wet} (kPa)	(ITS_{wet}/ITS_{dry})	UCS_{dry} (kPa)	UCS_{wet} (kPa)	(UCS_{dry}/ UCS_{wet})
Base layer: 1.5% modified emulsion (1,5* liter / m^3)	232	184	79%	2620	1865	71%
Sub-base layer: 1.2% modified emulsion (1,5* liter / m^3)	268	206	77%	4947	1670	34%
Sub-base layer: 1.0% modified emulsion (1,5* liter / m^{3*})	420	321	76%	4807	831	17%

* Anionic nano-silane modifier

Table II contains several results that emphasise the importance of detailed material design. The high percentages of material passing the 0.075 mm sieve of which more than 30% consist of clay with a crystal size of less than 1 nm have a marked influence on the results. A bitumen molecule is in the order of 2 – 6 µm in size – considerably bigger than clay crystals. The consequence is that higher percentages of bitumen results in lower tensile values with the clay crystals “swimming” in the binder resulting in lower tensile strengths. Hence the considerably higher tensile strengths at 1% versus the 1.5% anionic nano-silane modified bitumen emulsion [8].

In terms of pavement behaviour, the maximum tensile strain usually occurs at the bottom of the lower stabilised layer (in this case the sub-base) where the design should be such that these strains can be accommodated without failure. In addition, the maximum vertical compressive stress will be at the top of the base layer. In this case the design should ensure that the results obtained during the stabilisation process ensure that the base layer have the best resistance against compressive stresses and hence a high UCS (dry and wet) to also prevent quick deterioration against the ingress of water [8].

It should be noted that the “exceptional high” ITS values tested using a bitumen emulsion based stabilising agent are not unusual for anionic nano-silane modified bitumen emulsion tested on materials throughout southern Africa [15]. The performance characteristics and expected behaviour trends of the pavement structure may also differ substantially from the norm contained in the TG2 [16] – several explanations to these differences exists [8] that may require a variation on the approach recommended for bituminous treated materials as contained in the TG2.

VI. HVS Test Results

A. *Rut measurements conducted with the profilometer*

The rut depth as measured with the CSIR's laser profilometer is depicted in Figure 4. Figure 5 shows the progression of rutting across the test section. From the rut measurements indicated in these figures, the following can be seen:

- The average rut depth is 8 mm which is excellent performance after about 7 million E80s (using a damage coefficient of 4.2) or 3,5 million E80s using the damage coefficients determined through this single HVS test (see below);
- The maximum rut depth is 10.6 mm.

During the test, the surfacing failed due to a number of factors including the very hot weather experienced during the HVS test combined with the very high wheel load (80 kN) being applied. There was also some oil spillage over the HVS section (that may have softened the binder) and contributed to the failure. However, the main objective of this test was to test the bearing capacity and structural integrity of the base and subbase and, as the subsequent data will indicate, the structural performance has been excellent.

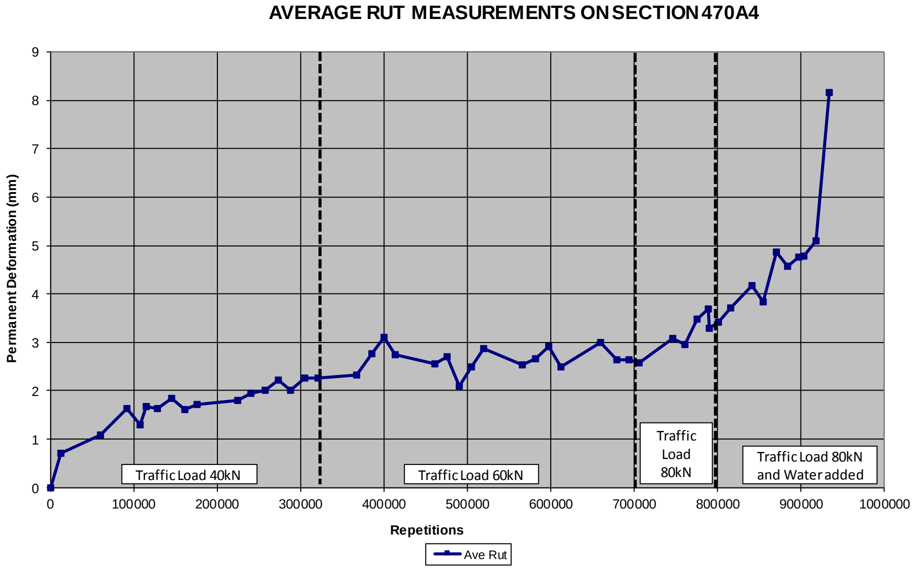

Fig. 4. Average rut depth measurements on the total section

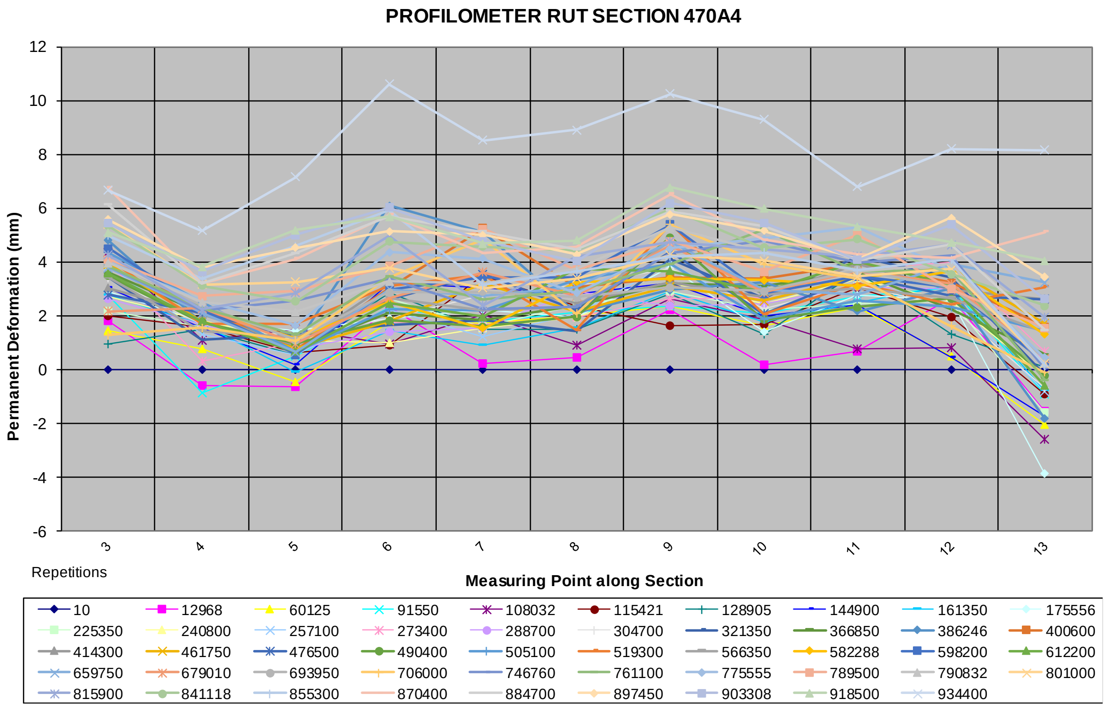

Fig. 5. Progression of cross section rutting during the HVS test

B. *Road surface deflections*

The surface deflections as measured with the RSD under 40 kN, 60 kN and 80 kN test wheel load through the test are given in Figure 6.

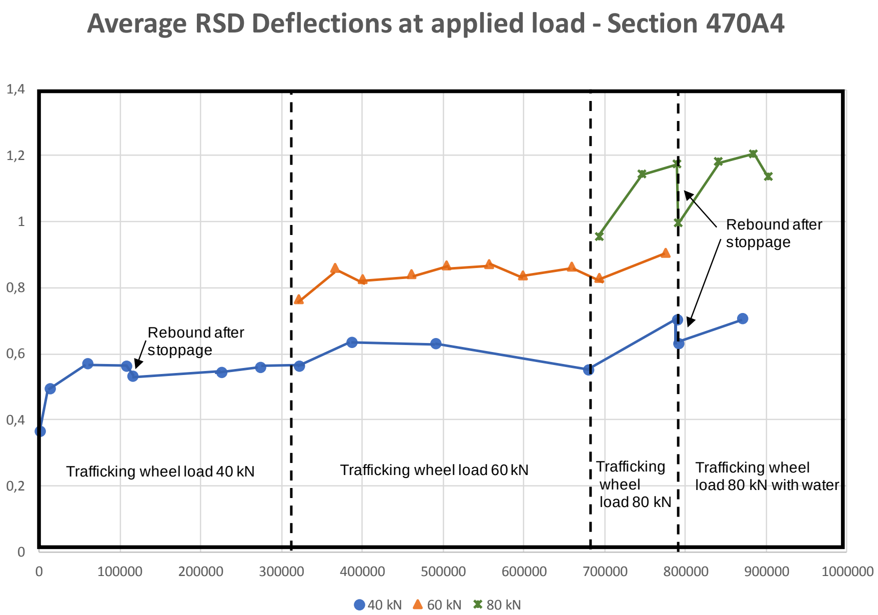

Fig. 6. Average surface RSD deflections (40 kN wheel load, tyre pressure 800 kPa)

Figure 7 shows the progression of deflection measured longitudinally over the test section under a 40 kN wheel load (800 kPa tyre pressure) through the test - both longitudinal centre line deflections and off-centre deflections.

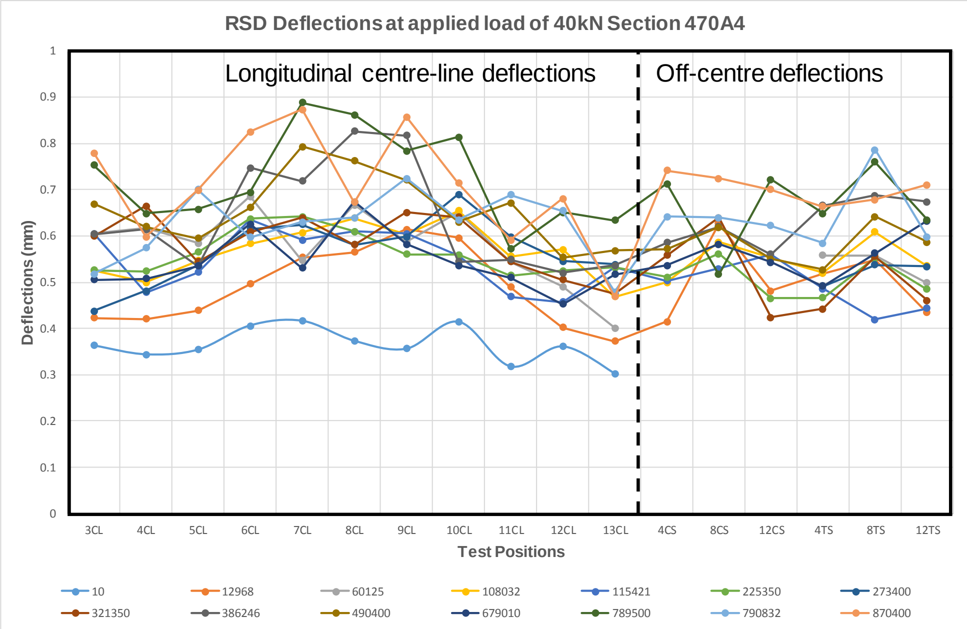

Fig. 7. Longitudinal and off-centre RSD deflections (40 kN wheel load, 800 kPA tyre pressure) at various stages (load repetitions)

From Figure 6, it can be noted that the average deflection increased from an initial value of 0.36 mm to 0.71 mm under a 40 kN wheel load (tyre pressure 800 kPa). The deflections under 40 kN did not vary significantly across the test section (see Figure 7). The maximum deflection measured during the test under 40 kN was 0.89 mm. Detailed deflection measurements at 60 kN and 80 kN were reported by Rust *et al.* [3]

C. *Multi-depth deflectometer (MDD) measurements at Point 4 on the HVS test section*

The multi-depth deflectometer (MDD) measurements at Point 4 are depicted in Figure 8 (permanent deformation measurements in depth) and Figure 9 (elastic deflections in depth).

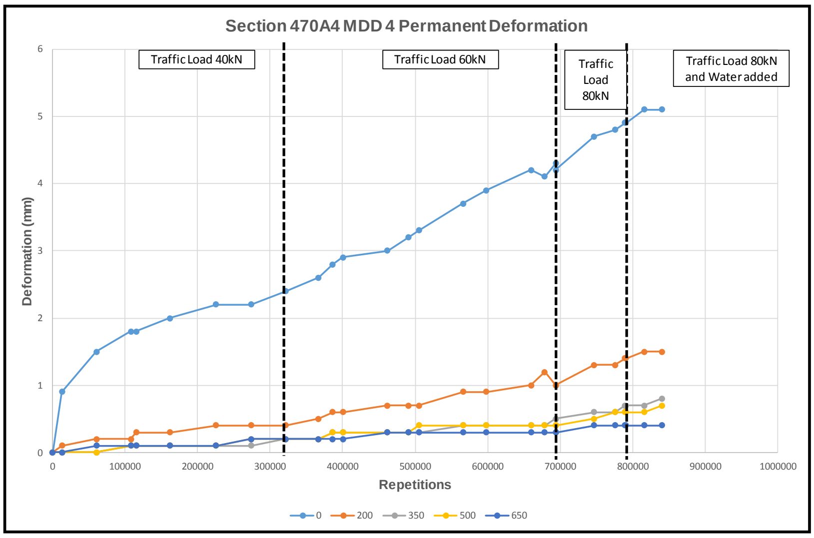

Fig. 8. Permanent deformation (PD) measurements at MDD 4

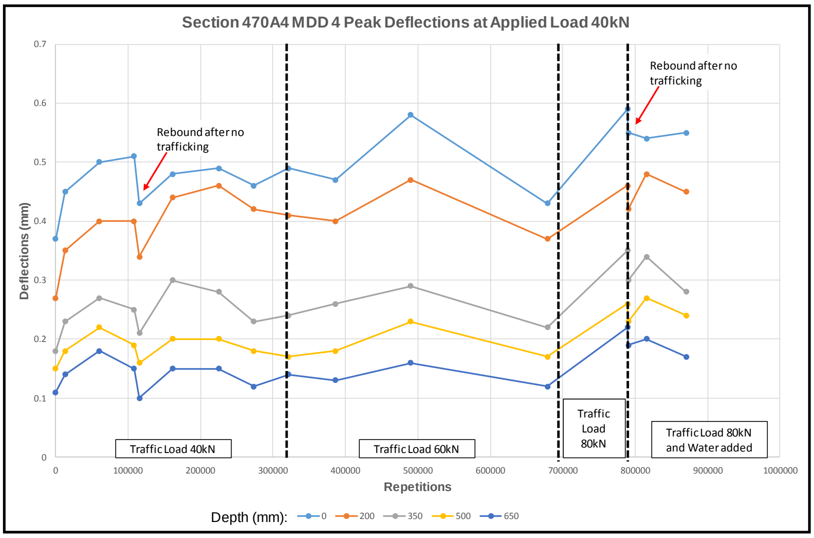

Fig. 9. Multiti-depth deflections at MDD 4 (40 kN wheel load, 800 kPa tyre pressure)

From the graphs in Figures 8 and 9 it can be noted that, although most of the permanent deformation came from the base layer, this deformation is still very low in terms of pavement design standards and is measured at about 3.6 mm within the base as it consolidated under traffic. The total deflection under the 40 kN wheel load is 0.59 mm, of which, at 490,000 repetitions, 0.11 mm came from the silane-modified base-layer and 0.18 mm came from the silane-modified subbase layer.

D. *Multi-depth deflectometer (MDD) measurements at Point 12 on the HVS test section*

The multi-depth deflectometer measurements at point 12 are depicted in Figures 10 and 11.

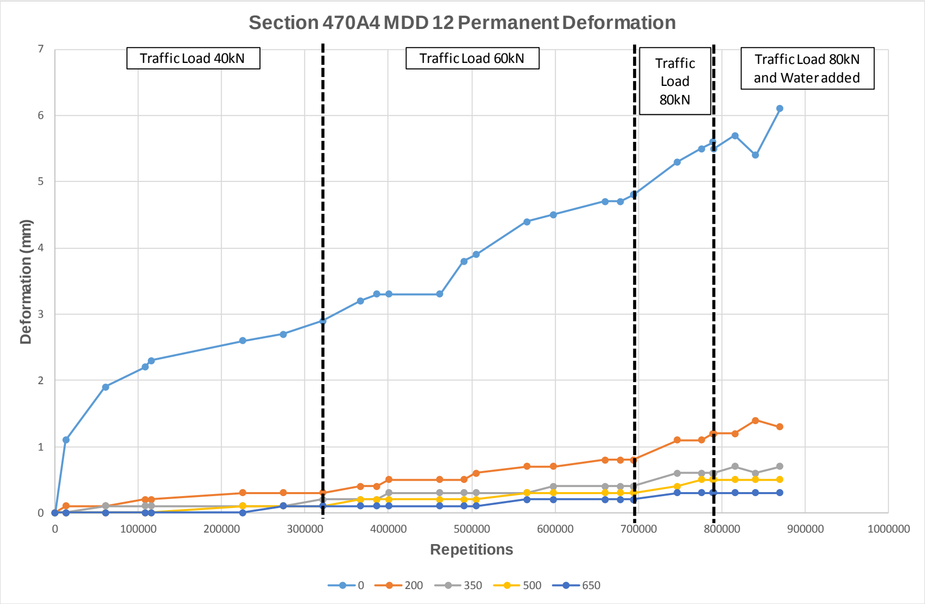

Fig. 10. Permanent deformation (PD) measurements at MDD 12

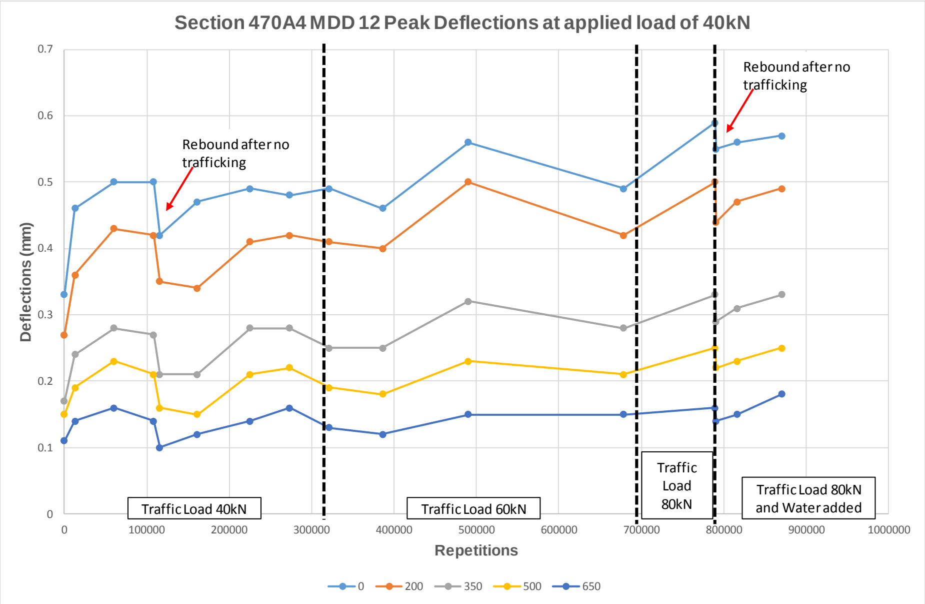

Fig. 11. Multi-depth deflections at MDD 12 (40 kN wheel load, 800 kPa tyre pressure)

It can be noted from Figures 10 and 11 above, that once again, the permanent deformation was low - measured as 6.1 mm of which 4.8 mm came from the base layer. The total deflection under the 40 kN wheel load is 0.59 mm (after 790,832 repetitions at the end of the dry phase) of which only 0.08mm comes from the silane-modified base layer and 0.16 mm from the silane-modified subbase layer.

VII. Damage Coefficients and Equivalent Traffic

It is very difficult to determine damage coefficients from only one HVS test especially if the test section did not fail. Kekwick [17] suggested more than one approach to determine a damage coefficient from an HVS test. However, in this case an attempt was made by using the rut rate in mm per repetition applied which is the most common method. Due to the fact that this study is based on a single HVS test, the damage factors are at best estimations. The rut rate in mm rut per repetition that was used for the damage coefficient calculation is shown in Figure 12 below.

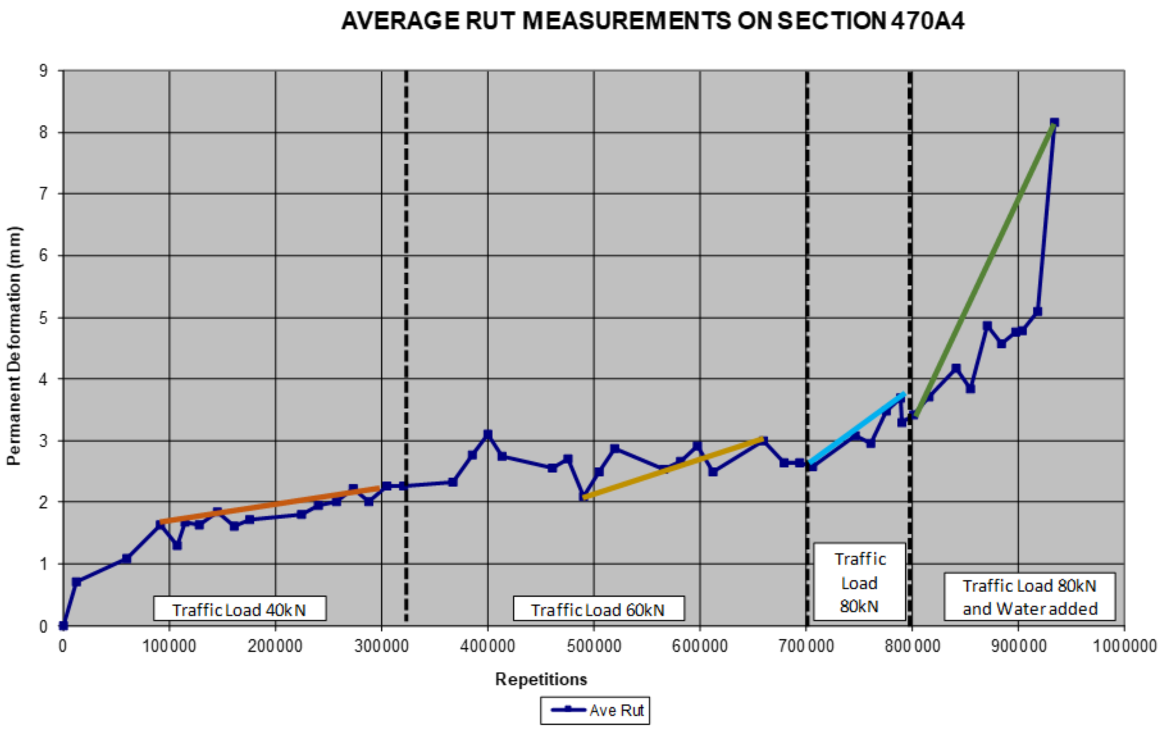

Fig. 12. Rut (mm) rate per repetition used for damage factor calculation

The first part of the rutting curve is due to settling in of the base and subbase and therefore, as usually interpreted during HVS testing, this part of the curve is not used for calculating the damage coefficient. The rut graph in Figure 12 indicates recovery of the test section in a number of cases. This was due to some stoppages due to maintenance and break downs as well as the fact that the rut levels were low. Particularly for the 60 kN part of the test therefore, the full test period was not used but rather only the latter part as indicated in the figure.

The formula used for calculating the damage coefficient was (Kekwick [17]):

$$d = \log (M1/M2) / \log (P1/P2) \qquad (1)$$

Where:

- M1 and M2 are the slope gradients for loads P1 and P2, in rut (mm) per repetition,

- P1 is the applied load, and
- P2 is the standard 40 kN wheel load.

Table 3 below indicates the damage coefficients calculated on this basis. The table also indicates the equivalent 80 kN axles applied using the calculated damage coefficients as well as the standard coefficient of 4.2.

TABLE III. CALCULATED DAMAGE COEFFICIENTS

Wheel load (kN)	Total traffic repetitions	Repetitions for the slope calculation	Rut (mm)	d	E80s (HVS)	E80s (d = 4.2)
40	321,350	229,800	0.63	1	321,350	321,350
60	372,600	169,350	0.9	1.63249	722,287	2,045,626
80	96,882	83,500	1.1	2.26461	465,542	1,780,611
80 (wet)	155,649	133,400	4.738	3.69547	2,016,485	2,860,700
TOT E80s					3,525,664	7,008,287

One of the most important findings of this HVS test, although a single test only, is that the materials seems to be insensitive to the magnitude of the wheel load to some extent – hence the low damage coefficients. This implies that this structure will not be overly sensitive to overloading. It should be noted that at harvesting season there are a significant number of heavy trucks and harvesting machinery on the D1884, which should experience less damage from this type of traffic than a shallower pavement structure.

VIII. VISUAL OBSERVATIONS AND TEST PIT RESULTS

As mentioned above the surfacing failed probably due to a combination of high temperatures, the high wheel load applied, and some oil spillage from the HVS hydraulic system. However, in spite of the problems with the surfacing and the surface and in-depth water added, the base and subbase performed exceptionally well (which was the main objective of the test). Figure 13 shows the failure of the surfacing. Figure 14 shows the blocks of the modified G8 material that were taken from the test pit inside the test section after the HVS test. The strength and cohesion in the base material is evident and is in line with the laboratory test results discussed above. It is noteworthy that these blocks contained only 1,5% of the modified emulsion and ***no cement.*** It must also be noted that no cracking was observed at the surface. The detrimental effect of cracks in cemented layers that are active and cause crack reflection through surface treatments have been well-documented [18], [19].

Fig. 13. Surfacing failure

Fig. 14. Blocks of material taken form the modified base

Data obtained from the test pit indicated that the moisture content as measured with the nuclear gauge varied from 9.1% to 11.7 %. In addition to the nuclear gauge data, samples were taken to determine the moisture content through oven drying. It can be noted that the moisture content in the subgrade was higher than that in the base and subbase layers and peaked at 13.1 % at 450 mm deep. This indicates that some of the added water did penetrate the subgrade layer. The base and subbase remained relatively dry due to the water resistance of the silane-modified emulsion treated material.

IX. Modulus Calculations

The MDD measurements taken during the HVS test were used to back calculate the moduli of the pavement layers using the CSIR back-PADS programme. In addition, after the HVS test, Dynamic Cone Penetrometer testing was conducted both inside and outside the HVS test section. The results are given in Table 4.

TABLE IV. MODULI

	Moduli (Mpa)					
DEPTH	DCP outside section	DCP in section	MDD 4 Average	MDD4 range	MDD 12 Average	MDD 12 Range
Base	252	309	173	69 - 797	127	71 - 214
Subbase	245	246	93	40 - 142	98	71 - 134
350 - 500 mm deep	145	193	154	75 - 316	174	82 - 606
500 - 650 mm deep	108	121	205	136 - 268	198	160 - 268
550 - 800 mm deep	99	220				

The progression of change in the MDD back-calculated moduli is shown in Figures 15 and 16.

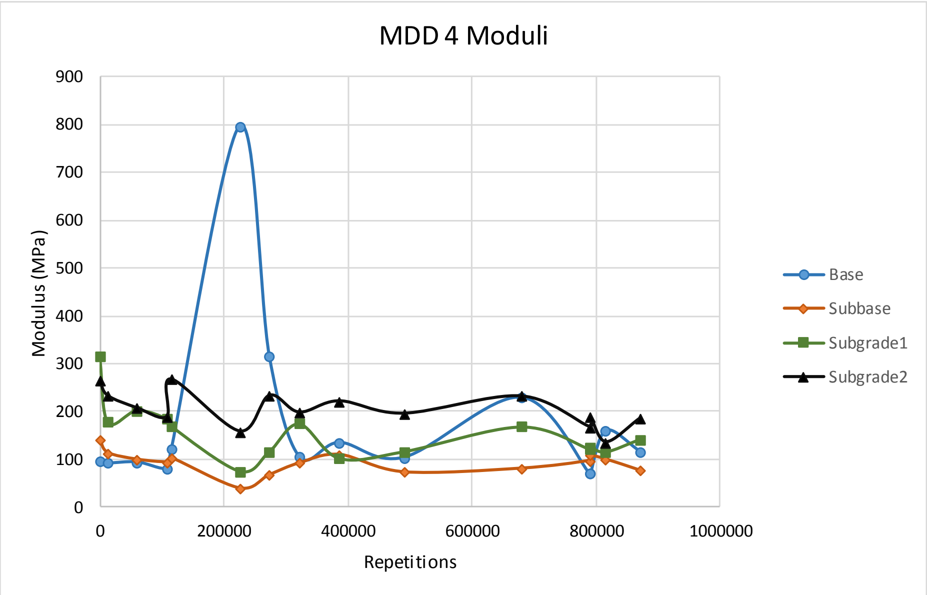

Fig. 15. MDD4: Back-calculated moduli

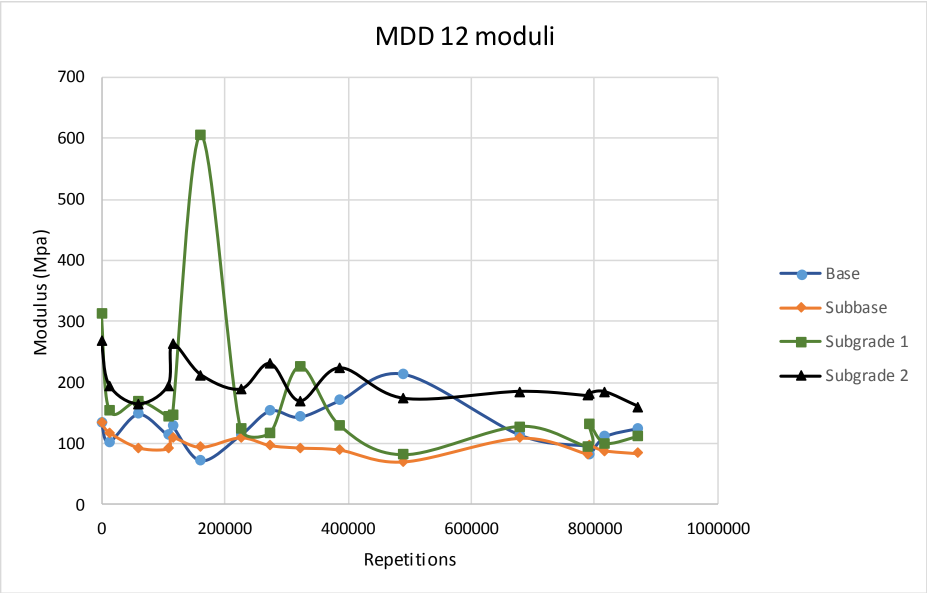

Fig. 16. MDD12: Back-calculated moduli

From Figures 15 and 16 the following can be noted:

- Apart from one or two outliers, the moduli of the layers did not change significantly over the test period;
- However, the base layer seemed to stiffen initially as the base compacted under traffic and then weaken slightly as the base started to fatigue under the heavy 80 kN wheel load;
- As observed from the test pit, there was a stiff, dry clay layer present in the subgrade, which may have influenced the back-calculation of the base and subbase moduli, seeing that it is well-known that linear-elastic theory has shortcomings when a stiff layer is present lower in the pavement structure. These moduli should therefore be viewed with discretion.

The initial stiffening of the base and its subsequent fatigue can be seen if the moduli calculations are separated into three distinct zones:

- Initial stiffening phase due to compaction under the HVS traffic;
- Middle stable phase, and
- End phase in fatigue.

This is shown in Table V Below.

TABLE V. CALCULATED MODULI OF THE BASE OVER THREE PHASES

	Average Calculated Moduli			
LAYER	MDD 4 Base layer	MDD 4 Subbase layer	MDD 12 Base layer	MDD 12 Subbase layer
Phase 1 up to 108,000 repetitions	91	112	125	109
Phase 2 from 108,000 to 679,000 repetitions	258	81	139	97
Phase 3 from 679,000 to 903,000 repetitions	104	96	104	88

From the data in Table V it can be seen that, at MDD4 there was a significant stiffening of the base form the initial Phase to the Middle Phase. The effect is less pronounced at MDD 12. For both MDD's there was a reduction in stiffness due to fatigue of the base layer in the end phase. The moduli for the subbase layer did not vary significantly.

X. COST COMPARISONS

According to the consultant responsible for the design of the D1884 rehabilitation project, the construction cost for the entire length of road was in the order of R 4 million per km for a 3 million E80 design using an anionic silane-modified emulsion stabilized layers. The equivalent conventional design alternative that would normally have been adopted for such a road project would consist of a 40 mm asphalt layer, a 150 mm G1 base, a 150 mm C3/C4 stabilised subbase and a 150 mm selected layer. Based on industry estimations, this design normally costs the GPDRT in the order of R7 million per km, depending on the haulage distance for the G1 crusher run or other imported material. Therefore, the alternative design is seen to be potentially very cost effective in terms of construction costs. However, a full life-cycle cost analysis could assist in confirming this and verify the economic benefits of the alternative design used on the D1884.

XI. DISCUSSION OF RESULTS AND FURTHER WORK

The work described here is based on a single HVS test and therefore the results should be verified through further testing. However, based on this initial study through a single HVS test, the following conclusions can be made:

- The surfacing failed during the test – this could be the result of a combination of a number of factors: an oil spillage under the HVS; high temperatures and the high wheel load used. However, this did not affect the performance of the base and subbase, both of which contained the anionic silane-modified emulsion that repels water.
- The resistance of the modified material to water was also borne out in the laboratory testing where the wet ITS results were excellent with high retained cohesion values (ratio

of wet vs dry ITS). The virgin material was classified as a G8 which under normal circumstances cannot be used in a base or subbase layer. Based on the results of this single HVS test it therefore seems that the anionic nano-silane-modified emulsion treatment can be used effectively to upgrade substandard materials. However, it is recommended that further testing be done to verify this result.

- During the HVS test 3,5 million equivalent axle loads were applied which led to only an 8 mm rut depth. This indicates that the base and subbase will easily carry the design traffic of 3 million equivalent axles and possibly much more.
- The average deflection at the end of the 40 kN test was 0.56 mm under a 40 kN wheel load (tyre pressure 800 kPa). Under the 60 kN wheel load the average 40 kN deflection increased from 0.56 mm to 0.63 mm and then decreased to 0,55 mm due to recovery after stoppages. During the 80 kN test the deflections under a 40 kN wheel load increased from 0.55 to 0.71mm. The surface deflection under a standard 40 kN load therefore did not increase significantly. Consequently, the tensile stresses in the base and the subbase did not cause excessive fatigue damage. This is one of the reasons why the section did not fail during the test.
- The base and subbase performed well under simulated wet conditions where in-depth as well as surface water was added continuously to the section whilst trafficking with an 80 kN dual wheel load.
- The structure (apart from the seal) seemed to be relatively insensitive to high wheel loads, especially in the dry state as indicated by the low damage factors calculated through this single HVS test.
- The results from the test pit indicated that the moisture content after testing was not high with the results varying from 7% to 12.5% as tested by the nuclear gauge. Oven-dry testing indicated moisture content of 13.1 % at 450 mm deep. It can be derived that the clayey material under the compacted subgrade was therefore relatively dry and therefore has a significant bearing capacity.
- The moduli back calculated from the MDD data did not deteriorate significantly during the test, but the base layer did stiffen slightly in the early ages due to post-construction compaction and then decreased slightly as the base layer fatigued under heavy wheel loads.
- The cost comparison indicated that the alternative design for these specific materials could be a very cost-effective solution in Gauteng using “sub-standard” in-situ materials.

The results presented here are based on one single HVS test and therefore must be viewed with caution. However, the initial results were very promising. Further work should include the following:

- Laboratory fatigue testing using the four-point beam apparatus to develop laboratory fatigue curves for typical road building materials modified with the nano-silane emulsion products.
- Laboratory characterisation of nano-silane-modified emulsion treated materials with different virgin materials, taking into account the composition of the virgin material including the mica content.
- Further HVS testing of the technology on weaker subgrades and using different types of virgin material.
- The development of a design guideline based on the results from the testing.

XII. Concluding Remarks

Although the initial work discussed in this report is based on some laboratory testing and a single HVS test, the results were very promising. The single HVS test on D1884 indicated that the section carried at least 3,5 million equivalent standard axles in a dry and wet state after which a rut of only 8 mm occurred which is below a warning level of 10 mm (as described in TRG12). Water was added to the structure in-depth as well as on the surface towards the end of the test period but did not cause the section to fail. The structure at the location of the HVS test also seemed to be insensitive to high loads of traffic as depicted by the low damage factors calculated. The construction cost of the Road on D1884 was R4 million per kilometre. According to experienced consultants, the standard design for this road would have cost R7 million per km depending on the haulage distance for the G1 crusher run. This thus indicates a 43% saving by using the modified emulsion base and subbase - a saving of more than R18m on the 6,2 km project on road D1884. The work indicated that HVS and in-situ testing of these new materials has the potential to save road authorities significant amounts. The effectiveness of this technology to upgrade sub-standard materials to base standard should be investigated further through laboratory work and HVS testing. A design guideline should be developed for this technology

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