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MODEL MOBILE LOAD SIMULATOR TESTING IN COMPARISON TO HEAVY VEHICLE SIMULATOR TESTING ON TRUNK ROAD 11 SECTION 1 – MALMESBURRY TO NATIONAL ROUTE 1

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1 Introduction

Following the discussions at the Heavy vehicle simulator (HVS) Technical Meeting held on 1 November 2002 at the N7 site offices, the Stellenbosch University was requested to prepare a proposal for model mobile load simulator Mk3 (MMLS3) testing on the TR11/1 (N7) between Malmesburry and the N1. The proposal was submitted to Mrs. Elzbieta Sadzik of Gautrans. The University was subsequently instructed to proceed with the testing as proposed. The MMLS3 testing programme ran parallel to the HVS testing programme completed by the CSIR and site testing took place from 09 December until 19 December 2002 and 6 January through to 17 January 2003.

This report gives some general information on the background of MMLS3 and how the testing device compares to the HVS machine of Gautrans in the second chapter. Five MMLS3 tests have been completed with different conditioning, of which three tests were on the asphalt surfacing and two directly on the foamed bitumen treated base. The different types of testing as well as the test result and the discussion thereof is dealt with in the third chapter.

The fourth chapter discusses the laboratory tests that were carried out on asphalt cores and foamed bitumen treated base cores extracted from the test sections and reference sections. The laboratory tests have been performed in an effort to determine trends and compare between properties before and after trafficking, as well as between the different types of conditioning.

In the fifth chapter a comparison is made between the results of the HVS testing and those of the MMLS3 testing in an effort to determine correlations between the two APT devices.

Finally, this report finishes with conclusions and recommendations in the sixth and final chapter.

2 Background

The CSIR has been carrying out accelerated pavement testing on the slow lane of the N7 in southbound direction using their heavy vehicle simulator (HVS) device. This device applies wheel loads to the pavement structure by means of a standard dual wheel configuration (half axle). The CSIR testing programme included applying loads of 40 kN, 80 kN and 100 kN both under dry and wet conditions up to speeds of 8 km/h. A tyre pressure of 800 kPa was typically used. The CSIR HVS testing on the slow lane commenced in September 2002 and was completed towards the end of February 2003.

The accelerated pavement testing device currently used by the University of Stellenbosch is the Model Mobil Load Simulator Mk3 (MMLS3). This device is a 1/3 scale model of a full size super single wheel and uses wheels with a diameter of 300mm and a width of 80mm. The tyres can be inflated to pressures of 800 kPa and loaded up to 2.7 kN. To simulate one wheel of a standard dual wheel configuration (20kN) a load of 2.2 kN (1/9 of full size wheel load) needs to be applied. The MMLS3 device is capable of applying 7200 load repetitions per hour (wheels travelling at a speed of 9 km/h). Because the MMLS3 is a third scale in terms of size, the loading time applicable when the MMLS3 wheels are travelling at 9 km/h equal the loading time for a truck wheel travelling at 27 km/h.

With the MMLS3 both the temperature at which the loads are applied, as well as the frequency (loading time) of the loads can be varied. This makes it possible to test asphalt pavements at a number of combinations of pavement temperature and loading time and determine the temperature and frequency dependant behaviour of asphalt pavements.

Compared with the full scale APT devices, the MMLS3 has some distinct advantages:

- Because it is a scaled model, it is easy to transport and can quickly be deployed, both on site and in the laboratory;
- A high number of load repetitions can be applied over a short period of time, which reduces the total testing duration;
- The MMLS3 has to possibility to apply loading under controlled pavement temperatures;
- The cost of testing is low compared to full-scale testing.

Besides stand-alone MMLS3 APT tests, the above makes the MMLS3 an ideal APT device to supplement a full-scale (HVS) APT investigation. The MMLS3 could i.e. be used in a phased approach, in which a number of time- and cost effective MMLS3 tests can be carried out to test different pavement structures under different testing conditions (temperature and moisture). The results and information gathered by MMLS3 testing on the larger number of variables, could then be used to define a more effective HVS testing programme.

The objectives of the MMLS3 testing on the N7, as stated in the proposal that was submitted, were as follows:

- To analyse and compare the scaled APT device (MMLS3) with the full-scale HVS and possible augmentation of the full-scale results,
- To establish possible correlations/trends between the HVS and the MMLS3 testing devices,
- To analyse the performance of the asphalt surfacing layer and the foamed bitumen treated base layer upon exposure to different levels of moisture and temperature conditioning.

It was furthermore stated that the testing would consist of:

- Parallel MMLS3 testing to the HVS,
- Supplementary MMLS3 testing to the HVS,
- Laboratory MMLS3 testing,
- Laboratory testing on cores extracted from the N7.

3 MMLS3 testing

3.1 Test conditions

Five MMLS3 tests have been carried on the N7-SB in the slow lane at the same location as were the HVS testing by the CSIR was carried out. Details of the tests are set out below in Table 1:

Table 1: Schedule of MMLS testing

No.	Date	Layer	Conditioning
1	10 – 14 Dec ‘02	Surface	Ambient
2	14 – 19 Dec ‘02	Surface	Hot (55ºC)
3	09 – 10 Jan ‘03	Base	Ambient
4	13 – 14 Jan ‘03	Surface	Wet (50ºC)
5	16 – 17 Jan ‘03	Base	Wet (30ºC)

During all tests 200,000 load repetitions were applied. The first two tests were carried out applying 2100 load repetitions per hour with the wheels travelling at a speed of 2.7 km/h. This resulted in a loading time of 0.08 seconds at the surface, which is the same as for the HVS when its wheel is running at 8 km/h. The first two tests, the surface ambient and surface hot test were carried out using a loading rate equal to that of the HVS since the stiffness of the asphalt surface layer is depending on the rate of loading and reduced resistance to rutting is to be expected at lower loading rates.

Intermezzo:
It is noted that faster loading rates applied by real traffic are substantially lower (0.02sec. and 0.01sec for a truck travelling at 40 km/h and 80 km/h respectively) than the rate of loading applied by the accelerated pavement tools discussed in this report and that this has a considerable effect on the stiffness and resistance to rutting of asphalt layers. To illustrate this it can be determined with the nomographs of van der Poel [1] that the bitumen stiffness of a bitumen at 25ºC with a penetration of 60 and a P.I. of 0 is 14,300 MPa at a loading time of 0.01sec. (real traffic) and just 3,620 MPa at a loading time of 0.08sec. (HVS). Rut resistance of asphalt under accelerated pavement tools operating at low wheel speeds is thus invariably lower than under real traffic (with the exception of the behaviour of asphalt under real loading at intersections, stop areas and gradients).

The other tests, base dry and wet test as well as the surface wet test, were carried out by applying 7,200 load repetitions per hour, which means that the wheels are travelling at a speed of 9.0 km/h and that the loading time is 0.02s.

The wheel load of the MMLS3 was set at 2,2kN to simulate half of a dual wheel configuration of a 80kN axle and the tyres were inflated to a pressure of 800 kPa.

With the wheel load and tyre pressure set as discussed above, the contact stress under the MMLS3 loading is similar to that under HVS loading. To illustrate this principle the vertical compressive stresses in the pavement structure are analysed using the linear elastic multi-layer computer programme BISAR [2] and are based on the following pavement structure (Table 2):

Table 2: Pavement structure N7-SB slow lane

Layer	Thickness [mm]	Stiffness [kPa]
Asphalt surfacing	53[1]	2500
FBT base	250	600
Granular sub-base	125	150
Subgrade	Infinite	100

1) This includes the Novachip™ layer.

The Poisson's ratio for all materials was assumed to be 0.35. The results of this analysis are shown in Figure 1 below:

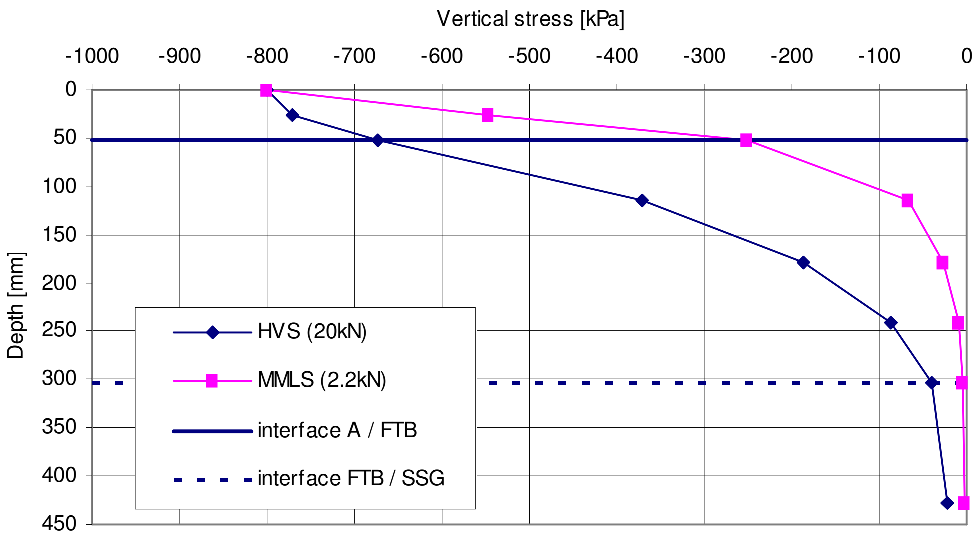


Figure 1: Comparison of vertical stress under HVS and MMLS

From Figure 1 above it can be seen that the vertical stress at the surface is the same for both the HVS and the MMLS, but that deeper into the pavement structure the vertical stress under MMLS load decreases more rapidly than under HVS load. This in itself is not a problem as long as it is taken into account during the evaluation and comparison of the rut depths that occurred under the two APT devices.

To get closer to the HVS stress situation in the base, the two MMLS3 tests on the base have been carried out with the asphalt layer removed. Originally it was proposed to mill off the asphalt surfacing to such a depth that approximately 15 mm of asphalt surfacing remains. In this way the stresses at the top of the foamed bitumen treated layer would be comparable under both HVS and MMLS loading (see Figure 2 below). In practice it proved quite difficult to remove such part of the asphalt surfacing to keep an even and uniform asphalt layer with a thickness of 15mm. Therefore the entire asphalt surfacing layer was removed and a fine grade slurry using a Colemat-L™ bitumen emulsion was placed on the base to a thickness of approximately 15mm (see Figure 3 below).

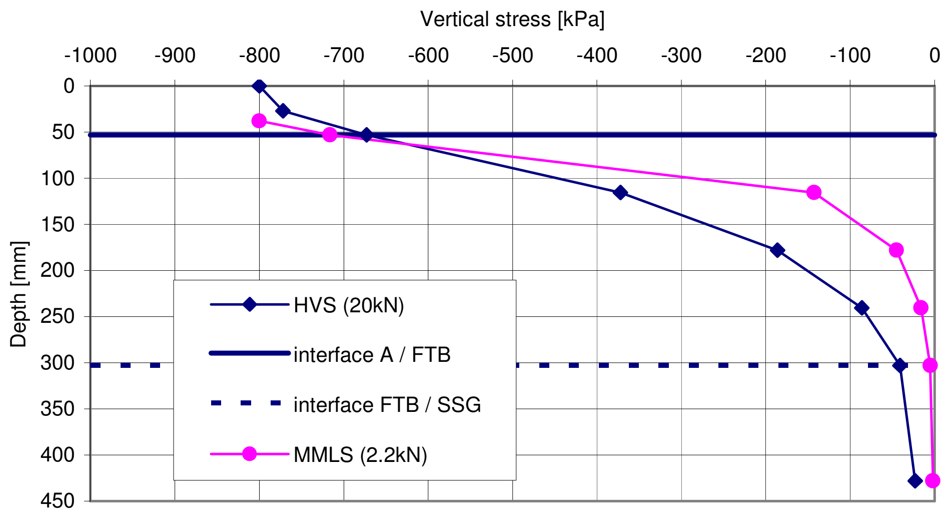
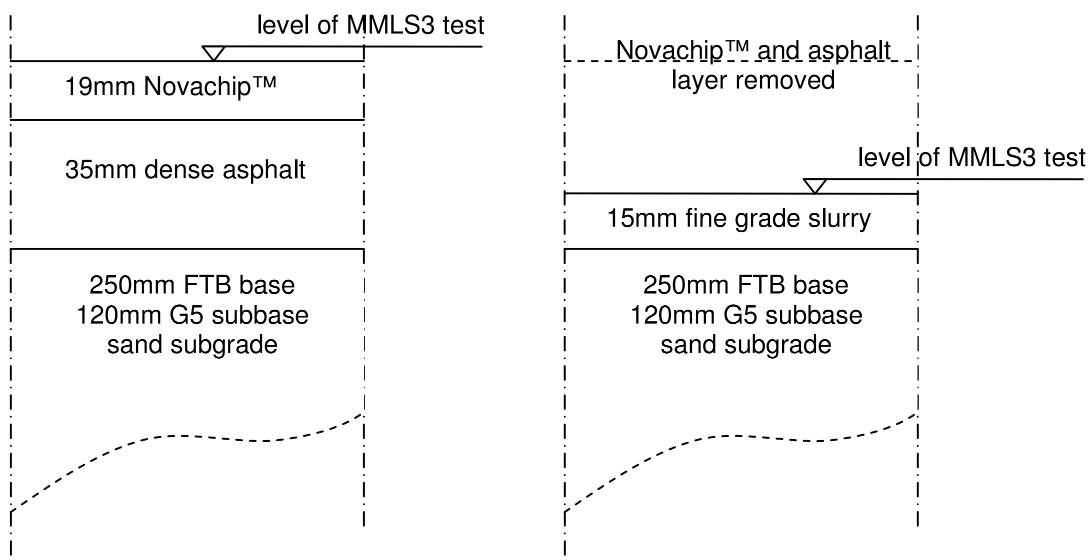


Figure 2: Stresses under HVS and MMLS with part of asphalt surfacing removed



Pavement structure for MMLS tests on surface

Pavement structure for MMLS tests on base

Figure 3: Upper part of pavement structure for surface tests and base tests

3.2 Test results

3.2.1 Measurements taken

During all tests the surface deformation was measured, while the pavement temperature at various depths was monitored.

The deformation at the surface has been measured using a profilometer. This profilometer is a computer operated measuring device that measures by rolling a sensor over the surface and electronically measuring the displacement at variable intervals (5mm in for this project). In this manner a profile of the surface can be compiled and surface deformation monitored. The computer-operated profilometer was used during all tests, except the dry test on the base. Due to unavailability of the automated profilometer, a manually operated profilometer was used. This manually operated profilometer operates on the same principle, however the sensor is moved over the surface manually and the volt readings, which are to be converted to displacements, are also logged manually.

The wheel path of the MMLS3 wheel is approximately 1,0m long and profiles have been measured at various cross sections. Index bars were placed on both sides of the MMLS, see Figure 4. The profilometer is placed on the index bar and can be positioned at 50mm intervals. The index bars provide a fixed reference, which position remains unchanged throughout the test. This method of measuring against a fixed reference is different from straight edge measurements, where the straight edge is placed on the surface, which deforms during the tests.

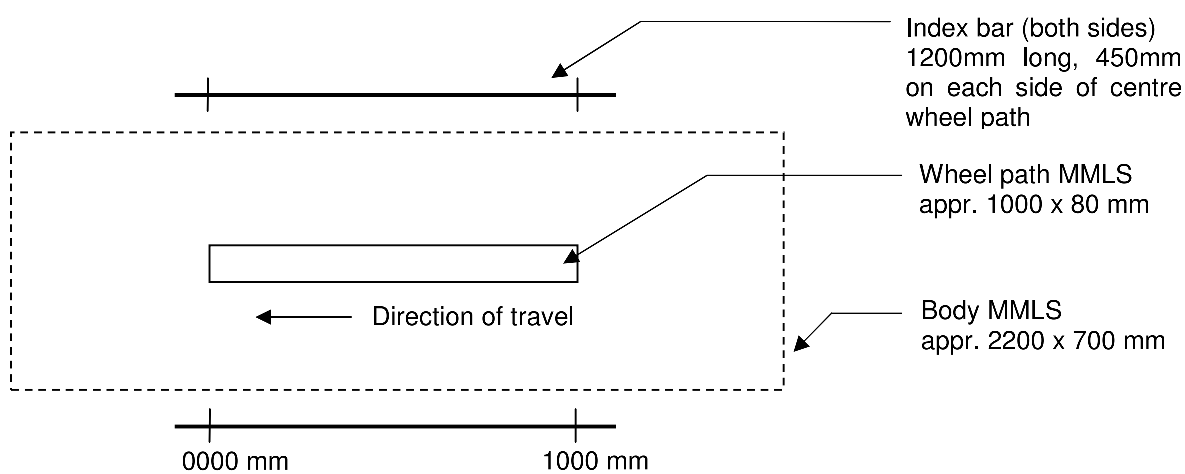


Figure 4: Lay-out of test bed

The numbering of the cross-sectional profiles in this report refers to the longitudinal distance of the profilometer relative to the end of the wheel path. At each cross section 10 measurements were taken at intervals of load repetitions as follows:

Table 3: Profile measurement intervals

Measurement No.	Cumulative No. of load repetitions
1	0
2	1,000
3	2,000
4	5,000
5	10,000
6	20,000
7	50,000
8	100,000
9	150,000
10	200,000

The profiles were measured up to 250mm either side of the centreline of the wheel path. By subtracting the initial reading (measurement No. 1 after 0 load repetitions) from the subsequent measurements one can determine a rut profile. A typical example of such a rut profile is given in Figure 5 below:

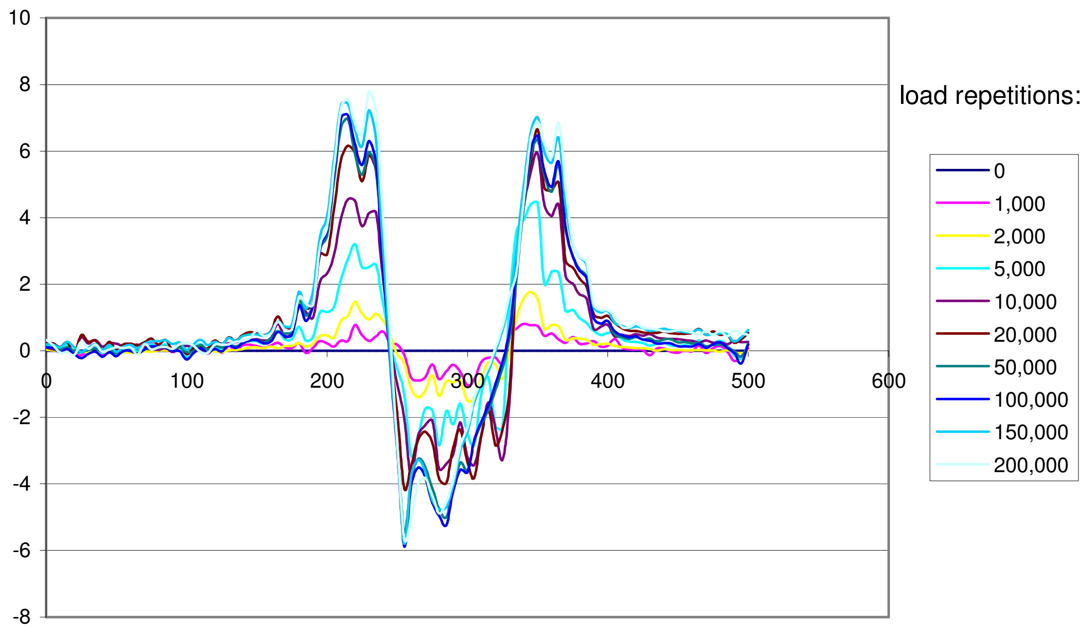


Figure 5: Typical example of rut profile (surface hot test, cross section at 900mm)

The rut depth used for further evaluation is the average rut depth over the middle 60 – 65mm of the wheel path calculated from the zero-line and does therefore not include the heaving next to the wheel path as a result of shoving.

The results per test are summarised in the following sections. The raw data is included in ***Appendix A*** to this report.

During all tests the temperature has also been monitored by means of thermocouples. Three thermocouples were installed on each side of the wheel path as closely as possible to the wheel path without disturbing possible heaving (approximately 100mm away from the outside of the wheel path) at different depths (surface, 20mm into pavement and 40mm into pavement), see Figure 6. Graphs summarising the temperature measurements are reflected in the sections below and the full data records are included in ***Appendix B*** to this report.

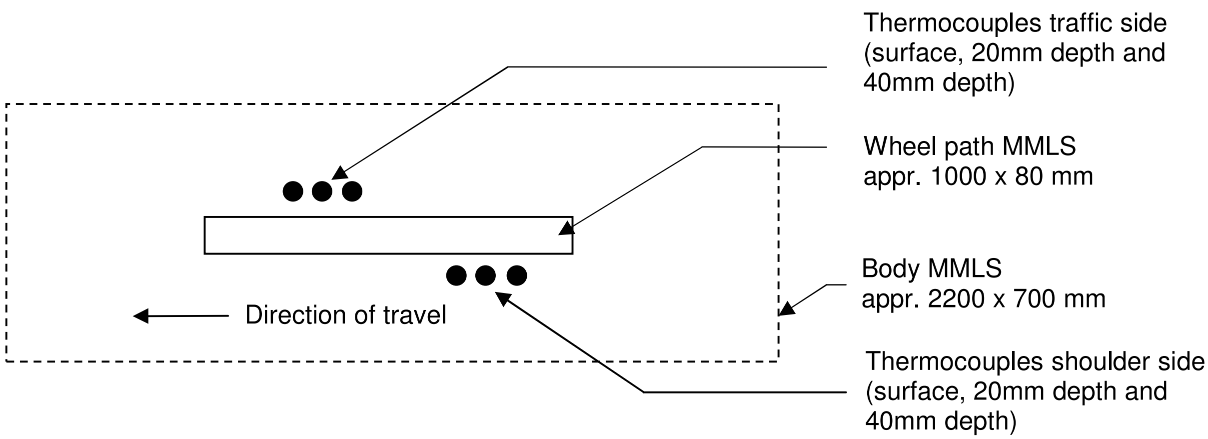


Figure 6: Lay-out of thermocouples installed

3.2.2 Surface ambient test

On 11 December 2002 the surface ambient test started, which was continued through to 14 December 2002, during which period 200,000 load repetitions were applied at an application rate of 2,100 load repetitions per hour. The surface deformation at four cross sections along the wheel path has been measured and the deformation at the surface is summarised in Table 4 overleaf:

Table 4: Rutting MMLS3 surface ambient test

No. of load repetitions	Profile 1 (200mm)	Profile 2 (400mm)	Profile 3 (650mm)	Profile 4 (900mm)	Average [mm]	Standard deviation	C.O.V
0	0.00	0.00	0.00	0.00	-	-	-
1,000	0.00	0.14	0.11	0.08	0.11	0.03	0.25
2,000	0.01	0.20	0.10	0.15	0.14	0.04	0.31
5,000	0.17	0.31	0.28	0.24	0.28	0.03	0.10
10,000	0.55	0.58	0.72	0.65	0.65	0.06	0.09
20,000	1.31	1.18	1.59	1.53	1.43	0.18	0.13
50,000	1.79	1.27	1.64	1.96	1.69	0.31	0.18
100,000	1.45	1.32	1.85	1.84	1.64	0.25	0.15
150,000	1.49	1.36	1.78	1.85	1.65	0.22	0.13
200,000	1.59	1.38	1.76	1.85	1.67	0.20	0.12

This data is graphically reflected in Figure 7 below:

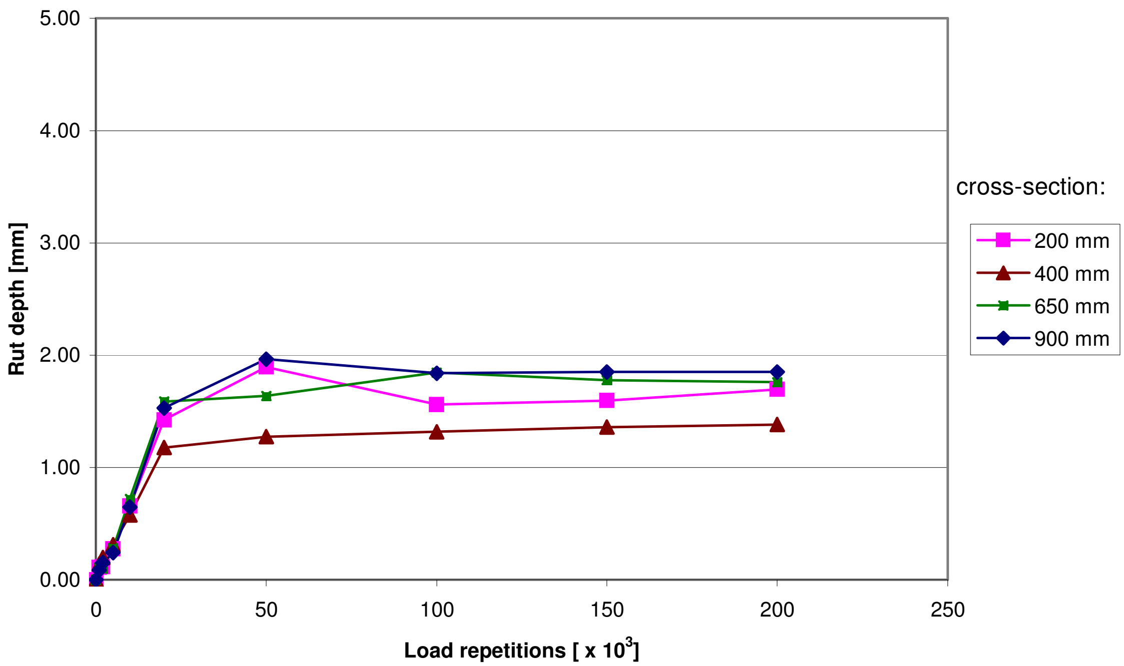


Figure 7: Rutting MMLS3 test surface ambient

The temperature of the pavement has been monitored and is reflected in Figure 8 overleaf.

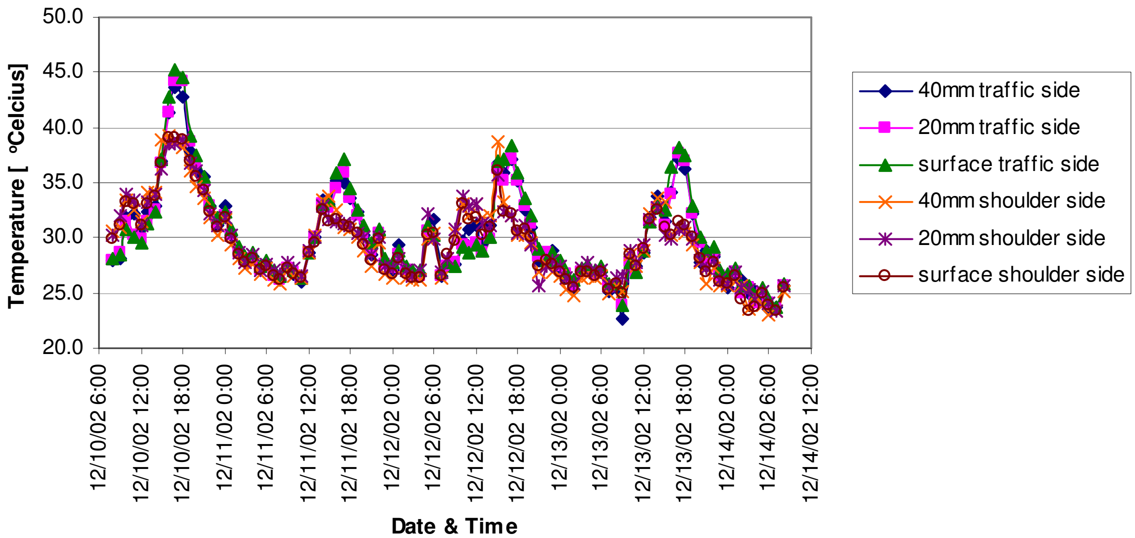


Figure 8: Pavement temperatures during surface ambient test

Approximately 60% of the traffic load was applied while the pavement temperature was 30ºC or less and only approximately 10% of the traffic load was applied while the pavement temperature exceeded 35ºC.

3.2.3 Surface hot test

Directly after the surface ambient test the surface hot test commenced on 14 December 2002 and continued through to 20 December 2002. 200,000 load repetitions were applied at a rate of 2,100 per hour. Four profiles have been measured and the deformation at the surface is summarised in Table 4 below:

Table 5: Rutting MMLS3 surface hot test

No. of load repetitions	Profile 1 (200mm)	Profile 2 (400mm)	Profile 3 (650mm)	Profile 4 (900mm)	Average [mm]	Standard deviation	C.O.V
0	0.00	0.00	0.00	0.00	-	-	-
1,000	0.51	1.53	1.14	0.68	0.97	0.46	0.48
2,000	1.27	2.13	1.76	1.07	1.56	0.48	0.31
5,000	1.98	2.99	2.87	2.21	2.51	0.49	0.20
10,000	2.51	3.75	3.4	2.87	3.13	0.55	0.18
20,000	2.79	3.85	3.57	3.25	3.36	0.46	0.14
50,000	3.65	4.49	4.15	3.97	4.07	0.35	0.09
100,000	3.91	4.71	4.11	4.19	4.23	0.34	0.08
150,000	3.59	4.66	3.91	3.77	3.98	0.47	0.12
200,000	3.78	4.77	4.07	3.88	4.13	0.44	0.11

This data is graphically reflected in Figure 9 below:

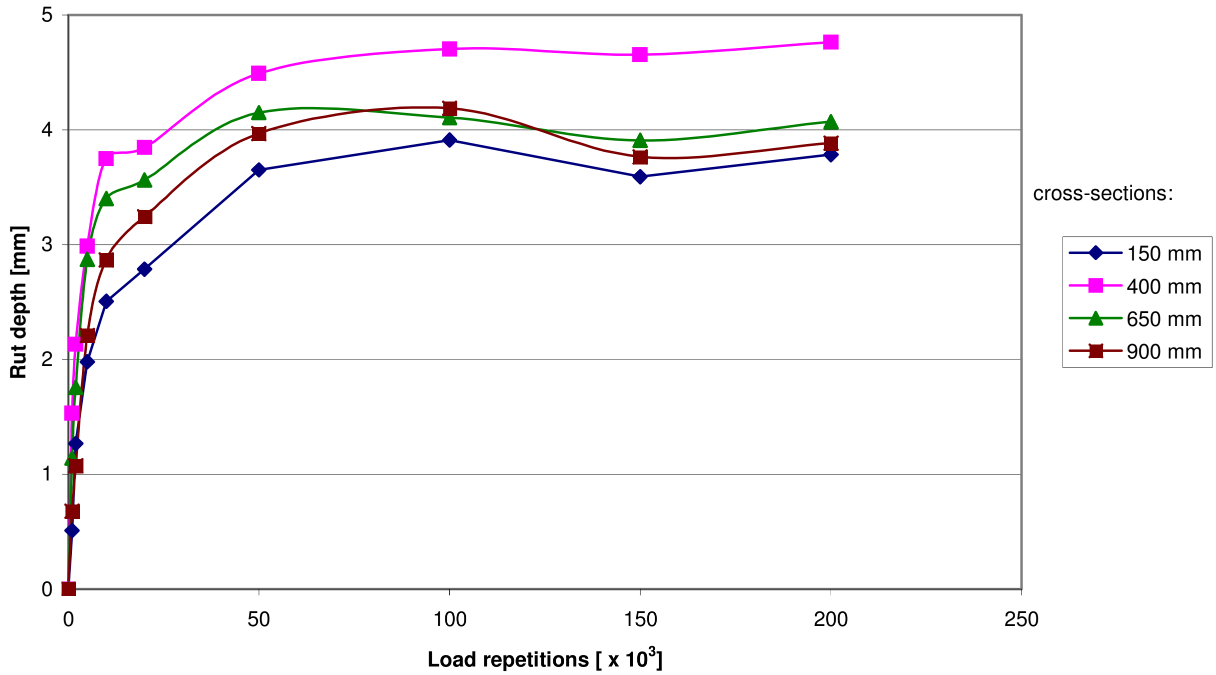


Figure 9: Rutting MMLS3 test surface hot

Temperature readings have been taken at hourly intervals during the test at three depths (at the surface, 20mm into the base and 40mm into the base) on either side of the wheel path. The aim was to keep the pavement at a constant temperature of 50ºC. The temperature fluctuation is reflected in Figure 10 and Figure 11 overleaf.

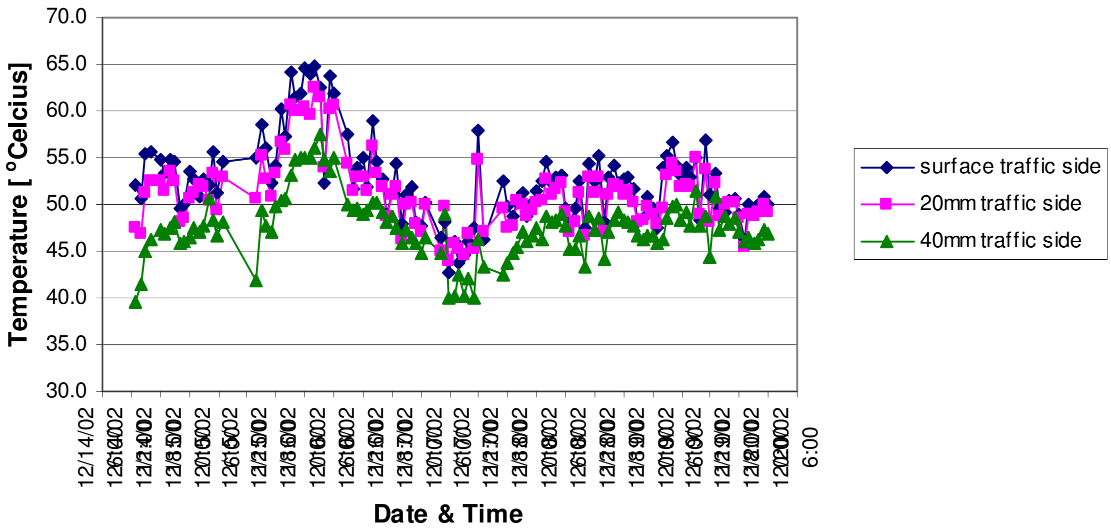


Figure 10: Pavement temperatures during the surface hot test (traffic side)

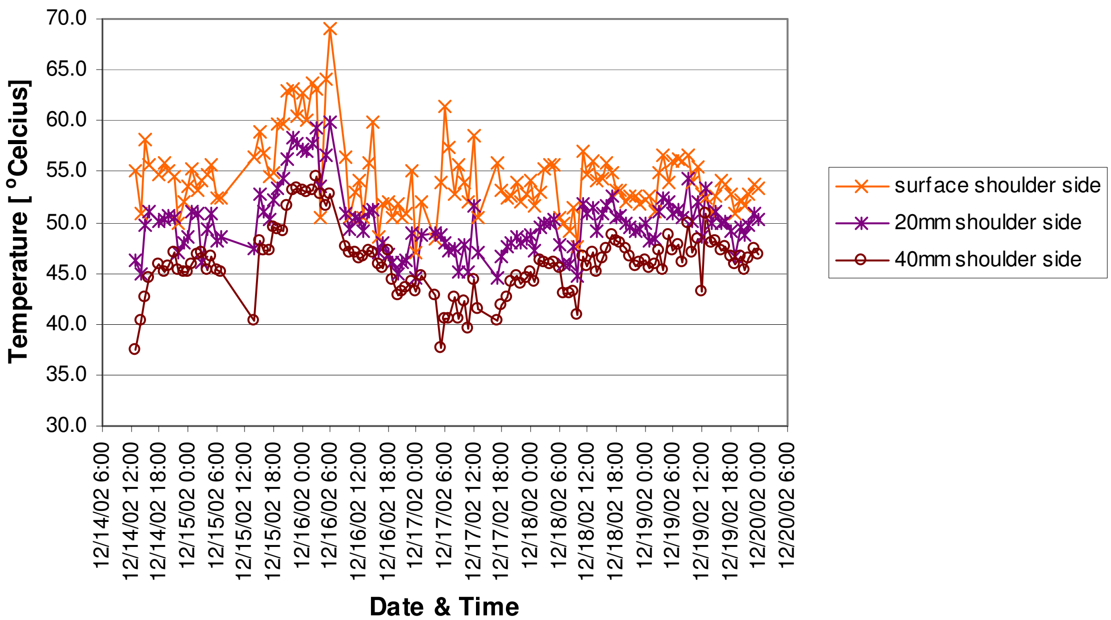


Figure 11: Pavement temperatures during the surface hot test (shoulder side)

3.2.4 Surface wet test

The surface wet test started on 13 January 2003 and continued for two days. A total of 200,000 load repetitions have been applied at a rate of 7,200 repetitions per hour. The rate of loading was increased compared with the first two surface tests because of time constraints.

Three profiles have been measured and the deformation at the surface is summarised in Table 6 below:

Table 6: Rutting MMLS3 surface wet test

No. of load repetitions	Profile 1 (250mm)	Profile 2 (500mm)	Profile 3 (750mm)	Average [mm]	Standard deviation	C.O.V
0	0.00	0.00	0.00	-	-	-
1,000	0.74	0.95	0.54	0.74	0.20	0.27
2,000	1.03	1.41	0.70	1.04	0.36	0.34
5,000	1.41	1.65	-	1.53	0.17	0.11
10,000	1.62	1.98	1.16	1.59	0.41	0.26
20,000	2.10	2.54	1.63	2.09	0.46	0.22
50,000	3.30	3.13	2.34	2.92	0.51	0.17
100,000	3.75	3.50	2.66	3.30	0.57	0.17
150,000	3.81	3.63	2.76	3.40	0.56	0.16
200,000	3.89	3.71	2.85	3.48	0.55	0.16

This data is graphically reflected in Figure 12 below:

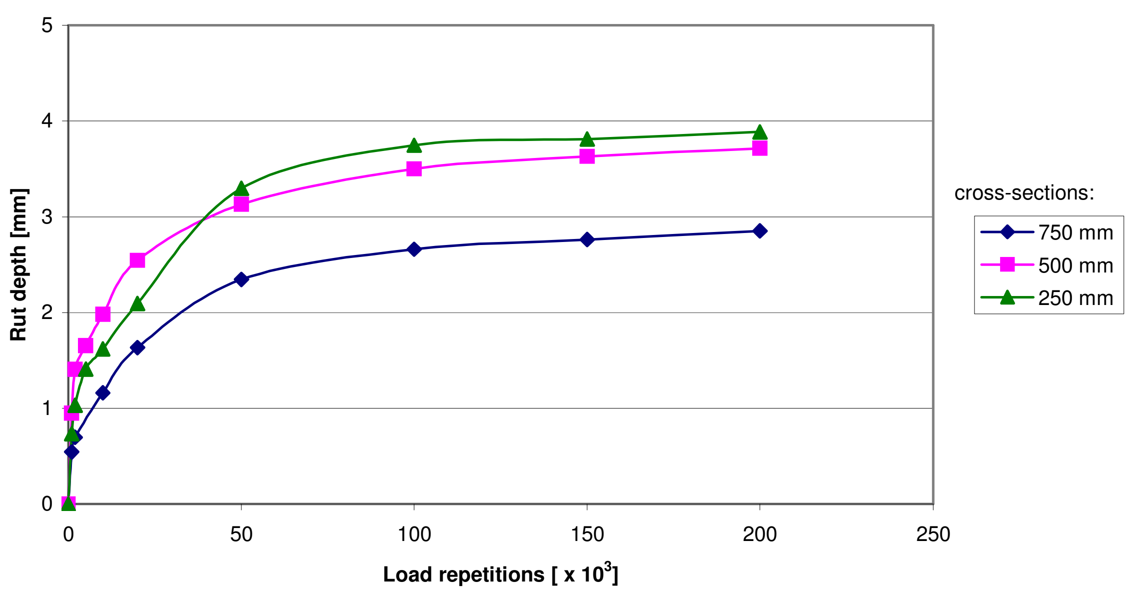


Figure 12: Rutting MMLS3 test surface wet

Temperature readings have been taken at hourly intervals during the test at three depths (at the surface, 20mm into the base and 40mm into the base, see Figure 6) on either side of the wheel path. The aim was to keep the pavement at a constant temperature of 50ºC. The temperature however was difficult to control, since lots of water was lost through the porous Novachip™ layer and could not be circulated and kept at a constant temperature. At the traffic side of the test pad, the side on which the hot water was applied the pavement temperature averaged around 45ºC, however on the shoulder side the average pavement temperature averaged around 30ºC. The temperature fluctuation is reflected in Figure 13 below:

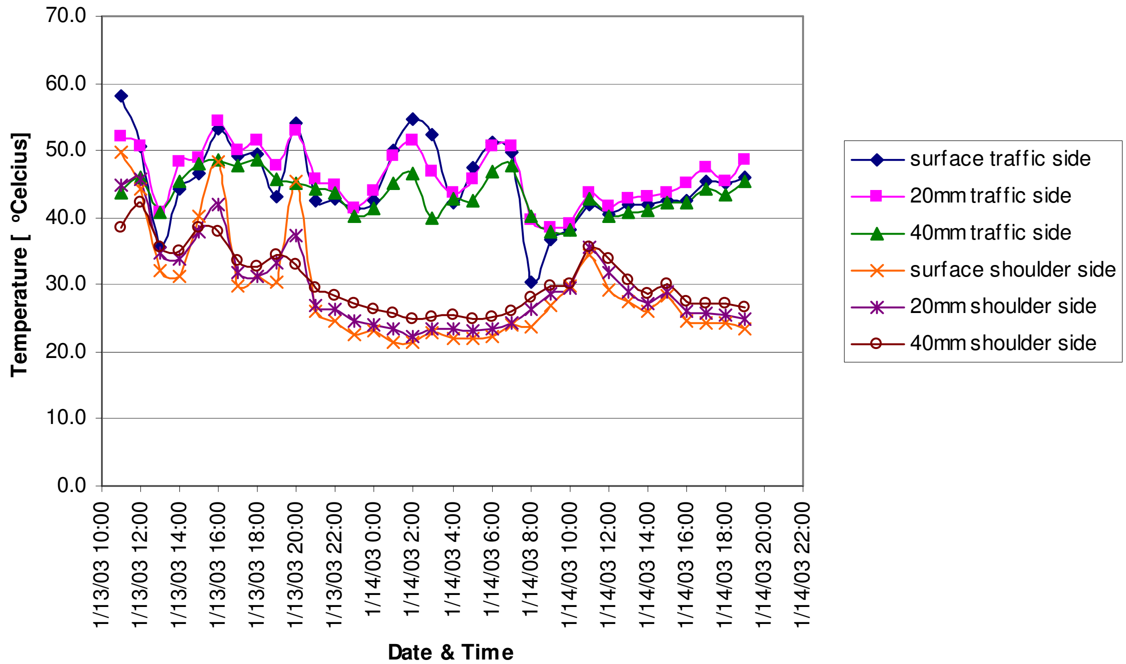


Figure 13: Pavement temperatures during the surface wet test

The problem to control the water circulation on a porous surface and associated therewith the ability to control the pavement temperature has led to the development of a new water application system with spray and suction bars where the water is not only applied on the 'high' side of the wheel path, but from both sides of the wheel path.

3.2.5 Base dry test

The base dry test was the first test after the Christmas shut down period and was performed on 9 and 10 January 2003. A total of 200,000 load repetitions have been applied at the planned rate of 7,200 repetitions per hour.

Three profiles have been measured using the manual profilometer and the deformation at the surface of the 15mm slurry on top of the base is summarised in Table 7 below:

Table 7: Rutting MMLS3 base dry test

No. of load repetitions	Profile 1 (250mm)	Profile 2 (500mm)	Profile 3 (750mm)	Average [mm]	Standard deviation	C.O.V
0	0.00	0.00	0.00	-	-	-
1,000	0.64	0.70	0.59	0.64	0.05	0.08
2,000	0.51	0.62	1.01	0.71	0.26	0.36
5,000	0.83		1.26	1.04	0.31	0.29
10,000	0.55	0.91	1.02	0.83	0.25	0.30
20,000	0.42	0.47	0.85	0.58	0.24	0.41
50,000	0.20	0.68	0.97	0.62	0.39	0.63
100,000	0.36	0.52	1.17	0.68	0.43	0.63
150,000	0.39	0.69	1.31	0.80	0.47	0.59
200,000	0.31	0.74	1.33	0.79	0.51	0.64

This data is graphically reflected in Figure 14 below:

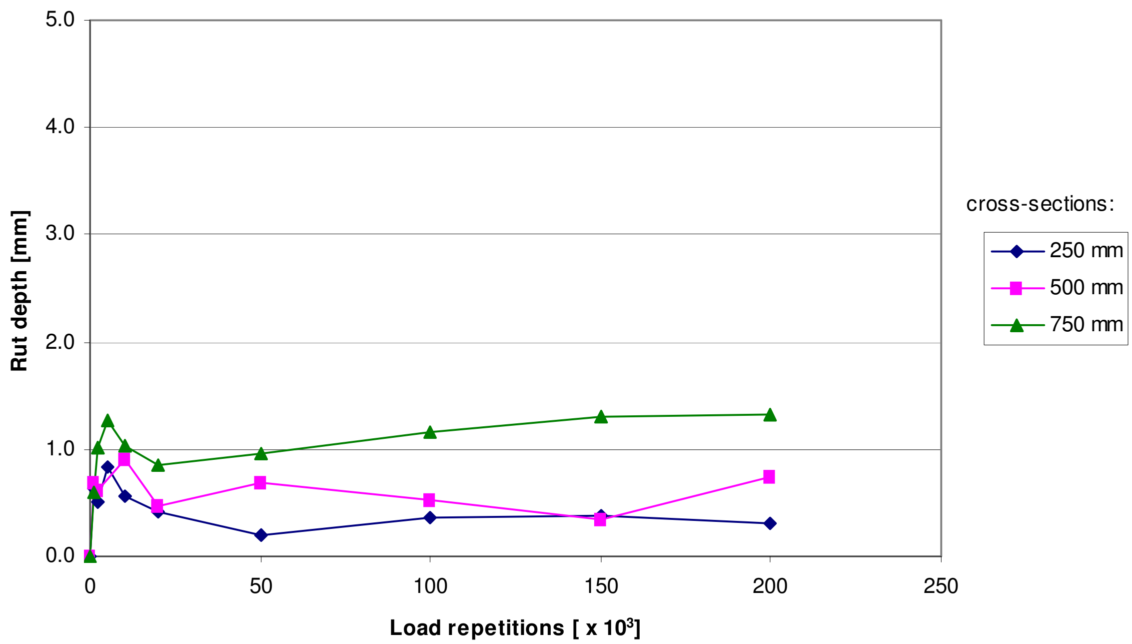


Figure 14: Rutting MMLS3 test base dry

Temperature readings have been taken during at hourly intervals during the test at three depths (at the surface, 20mm into the base and 40mm into the base) on either side of the wheel path and the temperature fluctuation is reflected in Figure 17 below:

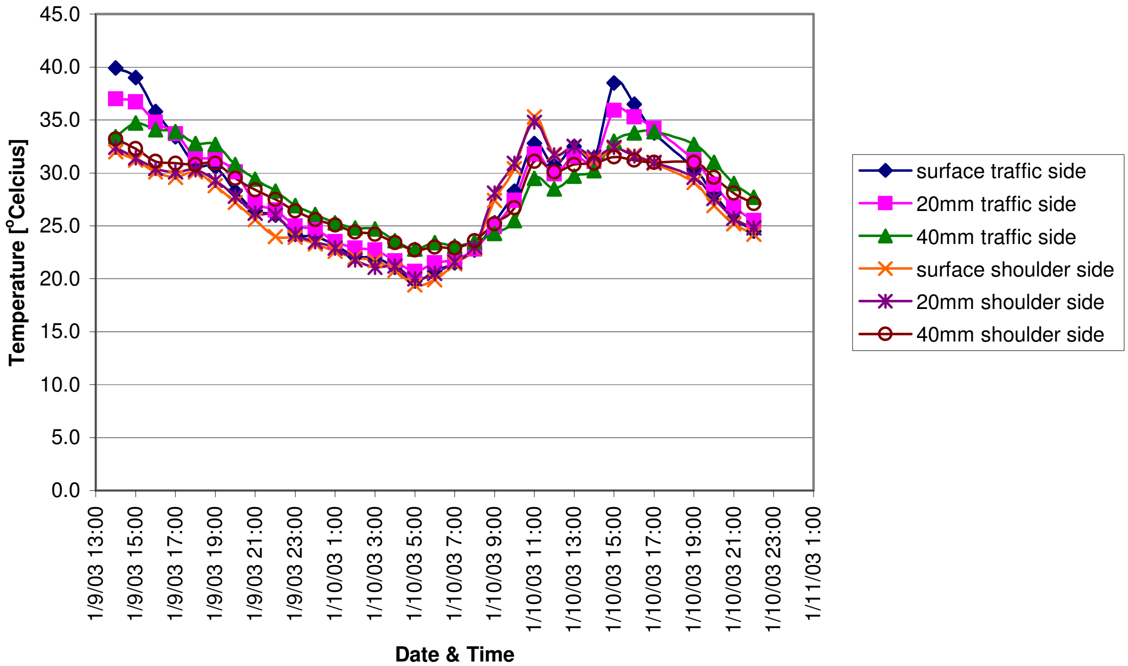


Figure 15: Pavement temperatures during base dry test

3.2.6 Base wet test

The last test performed was the base wet test, which started on 16 January 2003 and also lasted for two days. A total of 200,000 load repetitions have been applied at a rate of 7,200 per hour.

Four profiles have been measured and the deformation at the surface of the 15mm slurry on top of the base is summarised in Table 8 below:

Table 8: Rutting MMLS3 base wet test

No. of load repetitions	Profile 1 (200mm)	Profile 2 (400mm)	Profile 3 (600mm)	Profile 4 (800mm)	Average [mm]	Standard deviation	C.O.V
0	0.00	0.00	0.00	0.00	-	-	-
1,000		0.65	0.21	0.47	0.44	0.22	0.51
2,000	0.66	0.79	0.69	0.48	0.66	0.13	0.20
5,000	0.78	0.95	0.80	0.55	0.77	0.16	0.21
10,000	0.97	1.01	0.88	0.59	0.86	0.19	0.22
20,000	1.21	1.13	1.04	0.74	1.03	0.21	0.20
50,000	1.51	1.36	1.14	1.43	1.36	0.16	0.12
100,000	1.59	1.51	1.31	1.61	1.51	0.14	0.09
150,000	1.52	1.45	1.36	1.58	1.48	0.10	0.06
200,000	1.54	1.51	1.36	1.68	1.52	0.13	0.08

This data is graphically reflected in Figure 16 below:

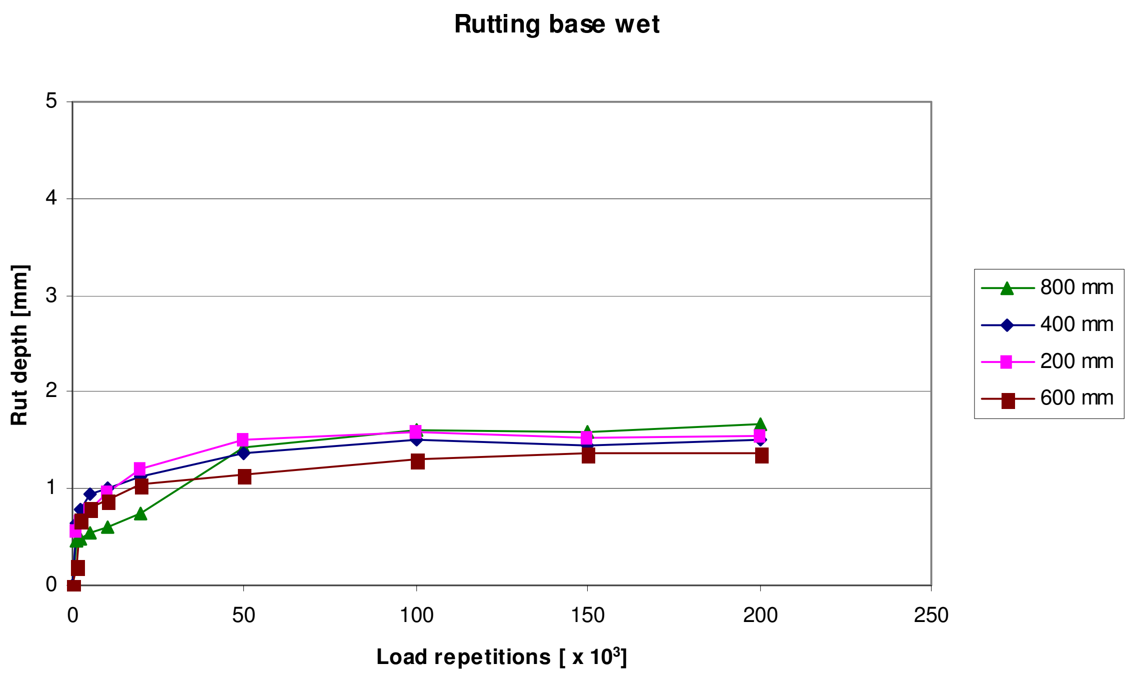


Figure 16: Rutting MMLS3 test base wet

Temperature readings were taken during at hourly intervals during the test at three depths (at the surface, 20mm into the base and 40mm into the base) on either side of the wheel path and the temperature fluctuation is reflected in Figure 17 below. A temperature of 30ºC was aimed for.

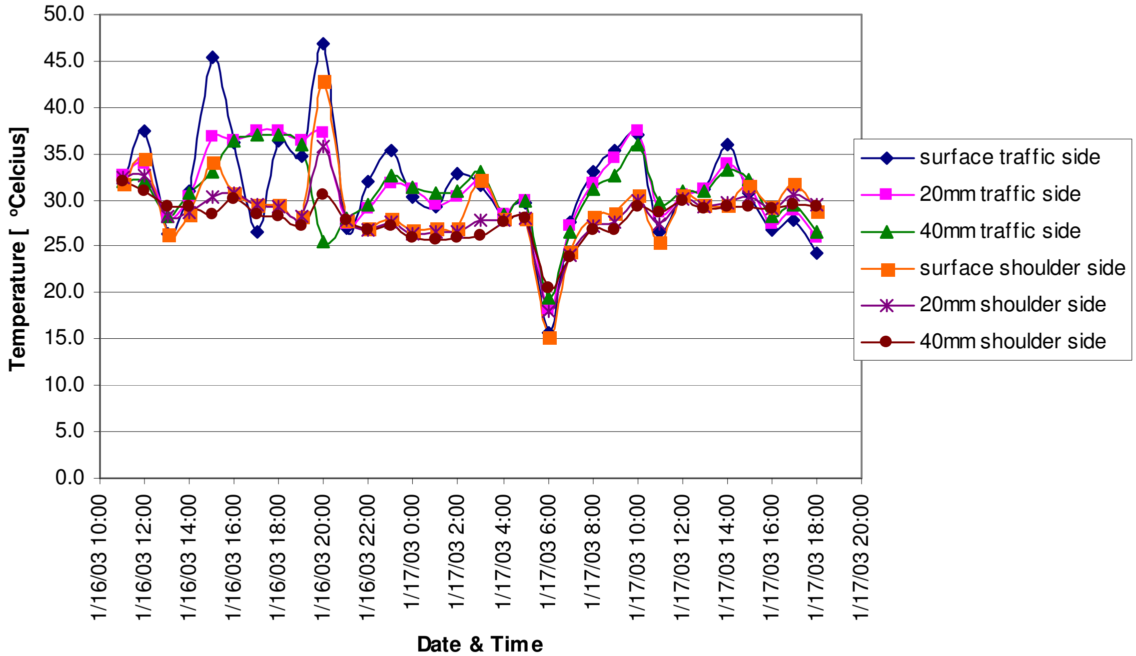


Figure 17: Pavement temperatures during base wet test

3.3 Discussion of results

3.3.1 Asphalt layer

The three MMLS test carried out directly on the asphalt surfacing layer show rutting in the following order (from less rutting to most rutting):

Ambient < Wet < Hot

The average rutting after 200,000 load repetitions is 1.7mm for the ambient test, 3.5mm for the wet test and 4.1mm for the hot test. The average rut development of the several tests is reflected in Figure 18.

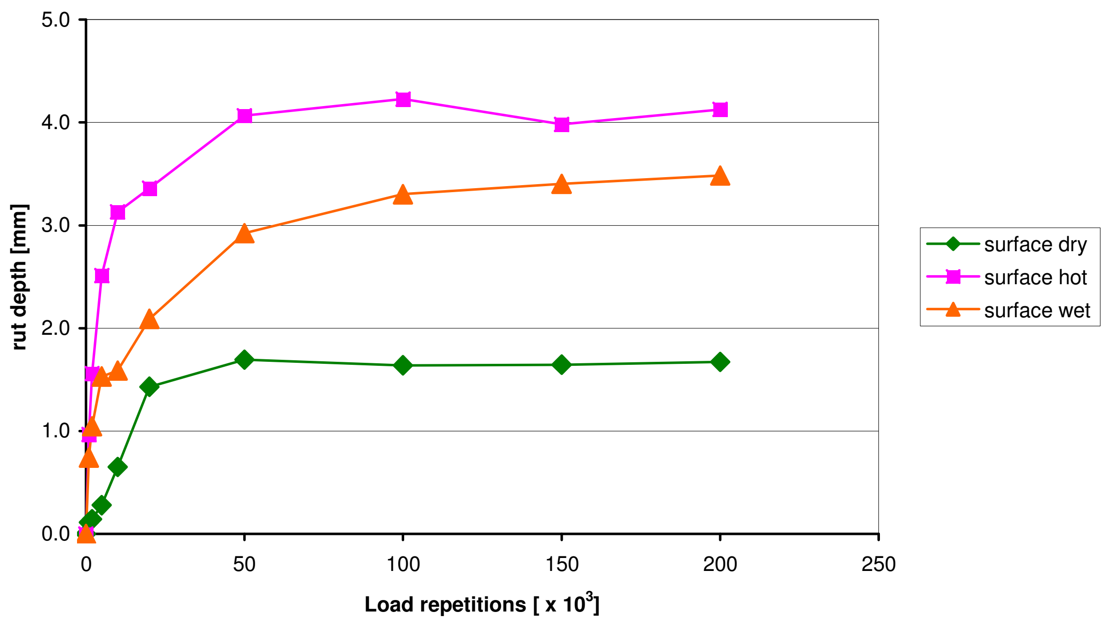


Figure 18: Average rut development of the MMLS3 tests on the surface

It was to be expected that in case of increased pavement temperatures the rutting resistance of the asphalt surfacing would decrease. Softening of the bitumen and related to that reduced shear resistance within the bitumen itself as well as within the asphalt mix as a whole are the main causes for this phenomenon.

It was aimed for to run the surface hot test and surface wet test at similar temperatures (50ºC). It can be seen that from Figure 10 that there is a reasonable spread in pavement temperatures at different depths and positions along the wheel path, however on average the temperature always was between 45ºC and 50ºC. The lower than aimed for temperatures during the surface wet test will be taken into account while making the comparison in section 5.3.

The difference in pavement temperatures during the surface hot and surface wet test possibly explains the difference in rutting depth measured during the two tests.

Another possible explanation for the slightly lower rutting during the wet test is the different loading rates used in the two tests. The loading time in the surface hot test was 0.08s, while in the surface wet test it was 0.02s. This results in a difference in the response stiffness of the bitumen and the asphalt mix, with the bitumen and mix stiffness under shorter loading times being higher than under longer loading times.

It can however be concluded that the rut development is not considerably accelerated under the influence of water addition.

The rut development as monitored up to 200,000 load repetitions can be extrapolated and used to predict rutting after an extended number of load repetitions. Such prediction is based on the assumption that on a log-log scale the rut develops according to a linear relation. Such linear relation is shown in Figure 19 below:

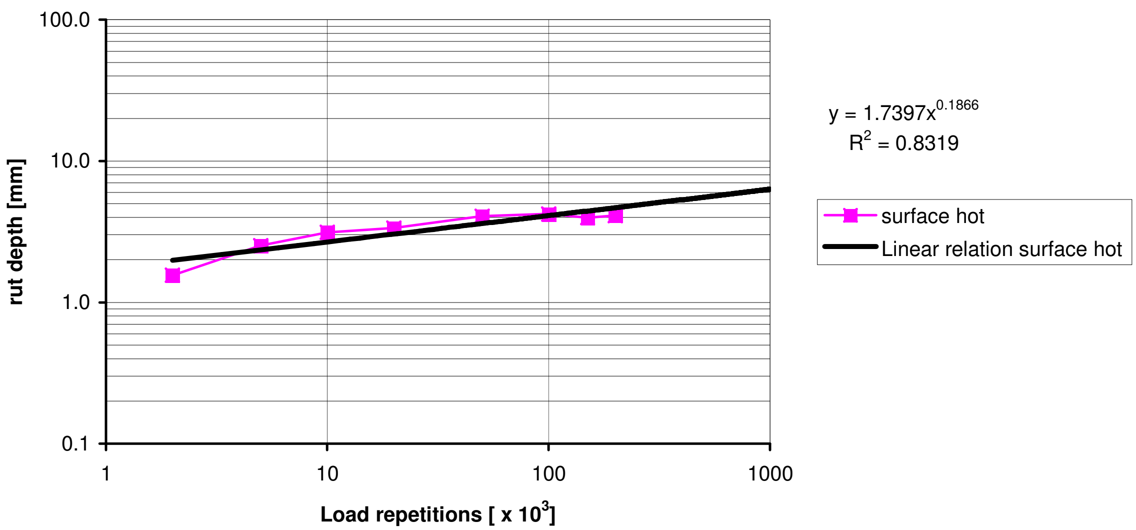


Figure 19: Rut development on log-log scale for the surface hot test

The power functions and the R-squared value for the linear regression lines are summarised in Table 9 overleaf.

Table 9: Rut development regressions lines

Test on surface	Rut development function	Correlation (R-squared)
Ambient	$0.1489\ x^{0.5284}$	0.83
Hot	$1.7397\ x^{0.1866}$	0.83
Wet	$0.9231\ x^{0.2668}$	0.97

1) Since this prediction is based on an irregular data pattern, use of this predicted value is not recommended

From Table 9 above can be seen that the correlation for all three cases (ambient, hot and wet) appears to be good. One has to however exercise care adopting this approach, since the plain application of this method would lead to a conclusion that the dry trafficking would lead to equal or even higher rutting which is contradicting the prior interpretation of the actual test results.

Although the R-squared values for both the ambient and the hot test are equal (0.83), the rut development function determined for the hot test shows visibly a better 'fit' than the ambient test (compare Figure 19 and Figure 20).

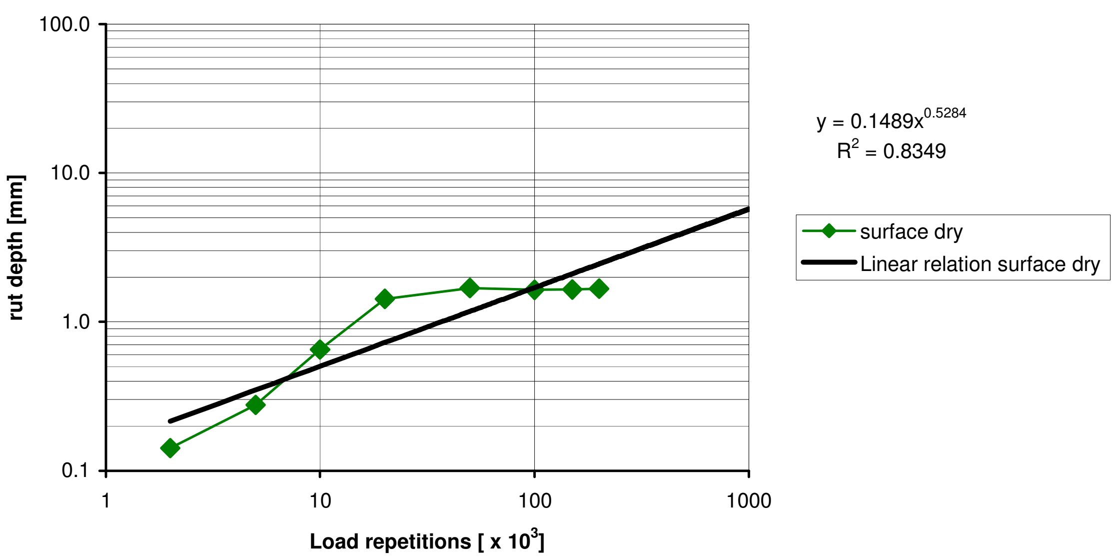


Figure 20: Rut development on log-log scale for the ambient test

In Figure 7 and Figure 20 it can be seen that in the ambient test there was a steep increase up to 20,000 load repetitions and subsequently a plateau condition up to 200,000 load repetitions. In Figure 8 can be seen that the pavement temperatures during the ambient test increased up to 45ºC under influence of sunlight during the first part of the test. This might well have caused the steep increase in rutting during the first 20,000 load repetitions. This first section of the rut development line has a greater influence in algebraic rut development function as indicated in Table 9 than the latter part, because it is plotted on a log scale.

If one considers the rut development from 20,000 load repetitions onwards (Figure 7 and Figure 20) and extrapolates to 1 million MMLS3 load repetitions, one would predict not even 2 mm rutting after 1 million MMLS3 load repetitions, which greatly differs from the value one would obtain by extrapolation based on data including the first 20,000 load repetitions (as in Figure 20).

This difference gives an indication that one should perhaps not apply one linear regression of the entire range of load repetitions, but to divide the rut development in different stages. From Figure 20 it can be seen that there is a distinctive change in the slope of the rut development curve, which might be explained as a change in rut development stage. The rut development up to this number of load repetitions may be an initial bedding or consolidation in stage (stage 1) with the rutting thereafter continuing to develop, but at a lower rate (stage 2; plateau stage). It is to be expected that the rut development goes into a third stage of "tertiary flow", material failure and excessive rutting. This usually occurs when very low void contents are reached. The duration of each stage is believed to be dependant on loading and material / pavement characteristics.

Figure 21 is a copy of Figure 20 but with a two-phase linear approximation of the rut development on a log-log scale. Compared with the approximation as indicated in Figure 20, the two-phase approximation shows a much better fit. It is clear that during the ambient test on the surface, the rut development did not go into a third phase.

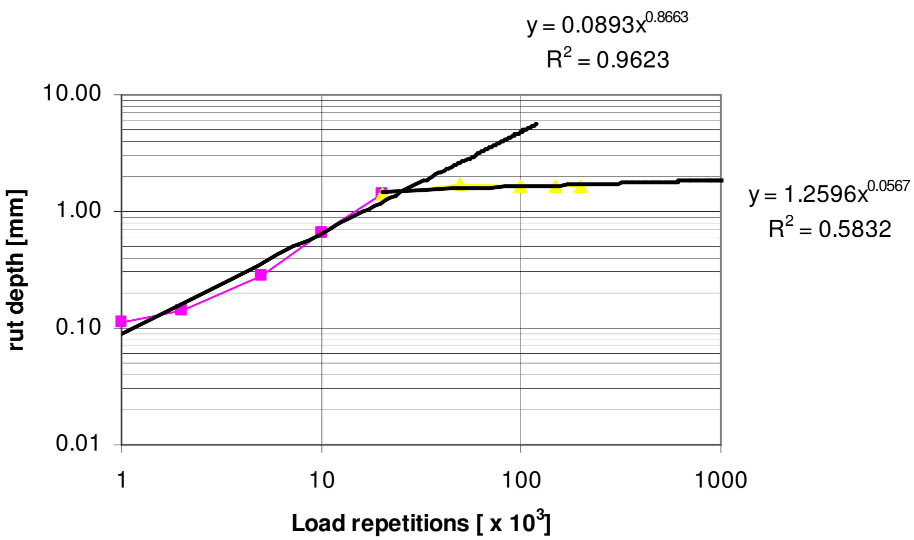


Figure 21: Two phase rut development on log-log scale for the ambient test

The slopes of the two linear approximations of each phase approach each another when the testing is performed at higher pavement temperatures. At low pavement temperatures (< ± 40ºC) rutting mainly consist of consolidation with little secondary rutting.

Whether a tertiary stage will be reached with MMLS3 loading and after how many load repetitions that will be can not be predicted based on the tests carried under this project and this would require further research and loading up to a high number of load repetitions.

It is therefore recommended that, because of the irregular rut data of the surface ambient test and the further research required to determine a predictable pattern of rut development, no prediction of the rutting for an extended number of load repetitions based on the test data is made.

3.3.2 Foamed bitumen treated layer

As can be seen in Figure 22 below the two test on the base show very little rutting. Relatively to each other the average rutting after 200,000 load repetitions during the base dry (average of 0.79mm with a standard deviation of 0.51mm) and base wet test (average of 1.52mm with a standard deviation of 0.13mm) differs considerably. However, the absolute difference between the rutting of both tests (0.73mm) is as small as the standard deviations of both tests combined (0.64mm). This means that there is a considerable overlap of the two normal distribution curves (see Figure 23) and that, although there is a trend that the wet trafficking results in higher rutting, is not possible to conclude based on the two test that were conducted that wet trafficking will always result in higher rutting than dry trafficking.

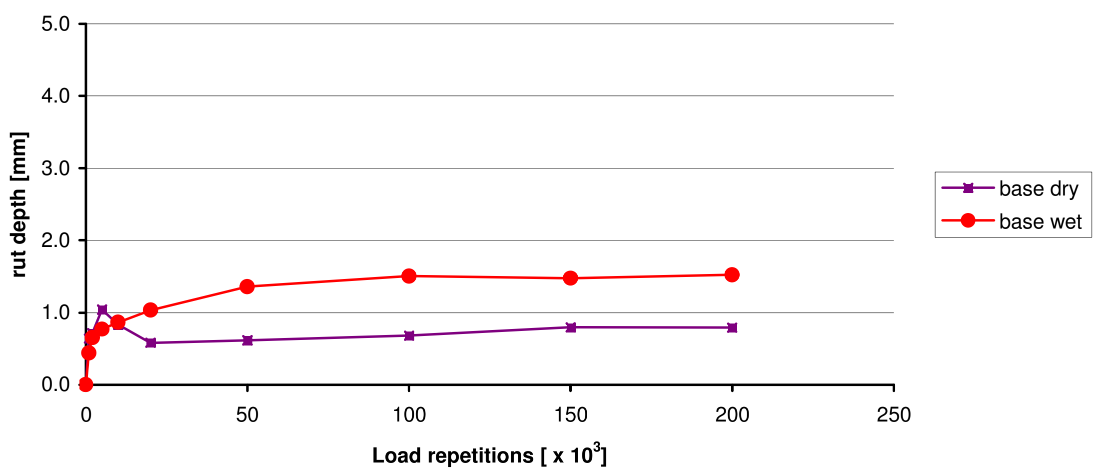


Figure 22: Average rut development of MMLS3 tests on the base

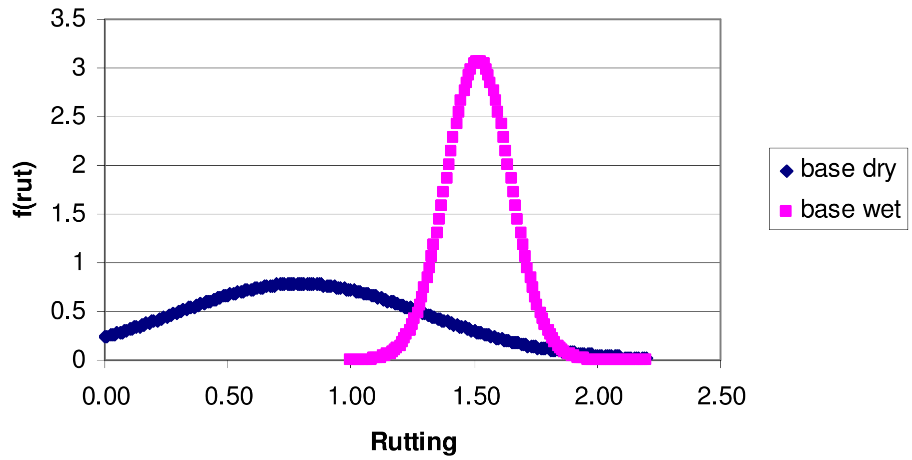


Figure 23: Normal distribution curves of base dry and wet test

The irregular pattern of rut development during the base dry test may be attributed to manually operated profilometer used for the measurements for the base dry test (see section 3.2.1), which has a lesser accuracy and a larger measurement error.

As indicated in section 3.1 the tests on the base were performed on an approximately 15mm Colemat-L^{TM} fine graded slurry placed on the foamed bitumen treated base after the asphalt surfacing layer had been removed. It is very likely that the main contributing factor to the approximately 1mm rutting during the two base tests, if not the only contributing factor, is deformation of the fresh slurry layer.

If one considers the latter part of the rut development curve (after 50,000 MMLS3 load repetitions), it can be seen that the curve is more or less flat and that there is no marked increase in rutting. This supports the statement that the little rutting measured can be ascribed to deformation of the fresh slurry layer.

Based on the above it is concluded that under the dry and wet MMLS3 test the permanent deformation that took place in the foamed bitumen treated base layer was very small and negligible when compared to the rutting that took place in asphalt surfacing layers.

4 Laboratory Testing

A total of 24 100mm Ø asphalt cores with a diameter of 100mm have been extracted from the 3 MMLS3 sections (ambient, hot and wet) as well as from a reference section (untrafficked).

A total of 16 cores 100mm Ø were taken from the foamed bitumen treated (FTB) base; six cores each from the tested sections (dry and wet) and four cores from a reference section.

Some difficulties were experienced with coring the FBT base. When water was added during the coring, the fines would wash out of the base material and the core would loose integrity. However, if no water was added the core barrel would quickly overheat. Furthermore it proved to be quite difficult to core to the full depth of the FBT base and maximum depth of the coring extended up to 150 – 200 mm. A purpose-made device was used extract the cores from the FTB layer (see Figure 24 below).
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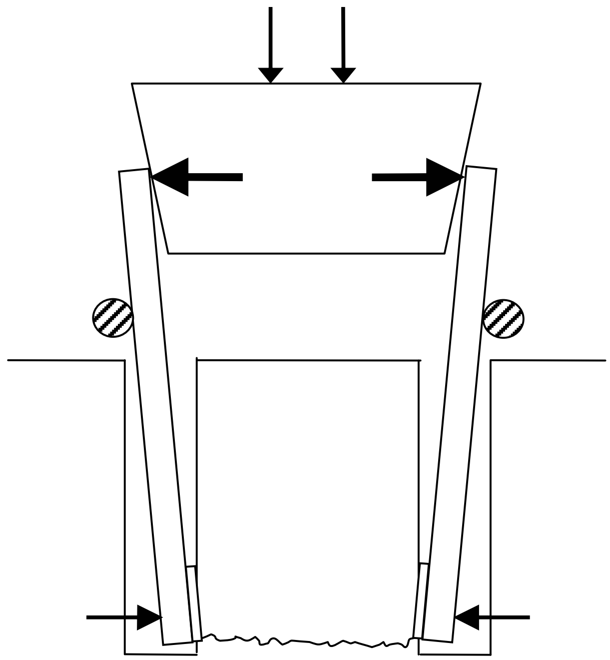


Figure 24: FTB core extraction device

Three curved steel plates that form a 100mm Ø cylinder with small teeth at the bottom end were slid into the 4mm wide saw cut around the core and a steel ring was placed around these sections to keep them together. By forcing a circular wooden wedge into the top of the steel cylinder, the teeth would clamp onto the core at the bottom and by rotating the entire device while clamping the core a shear fractured face would be created at the bottom. The entire device with the core could then be lifted out of the FTB layer.

However, because of the narrow width of the saw cut around the core, it proved quite difficult to complete this procedure successfully. Combined with the loss of integrity as a result of the drilling, not all cores extracted were in perfect condition. The 'best' cores were selected for testing.

The following tests have been carried out on the cores:

Table 10: Test schedule

Test	Asphalt	FTB
B.R.D.	24	-
ITS	14	
ITT[1]	24	16

1) In the ITT test each specimen is subjected to two tests, with the second test on the specimen rotated 90º

From each asphalt core the Novachip™ layer has been removed from the continuously graded surfacing mix by means of cutting with a diamond saw. This was done because the Novachip™ layer has an open grading and irregular void pattern, which would result in scatter in density measurements and resilient modulus testing.

The FTB cores were all trimmed to a final height of approximately 40mm, originating from as closely as possible to the top of the base layer.

4.1 Bulk relative density

The results of the Bulk Relative Density measurements are indicated in Table 11 below.

Table 11: Bulk relative densities of the continuously graded asphalt surfacing (without Novachip™ layer)

Section	Number of specimens	Average BRD	Standard deviation	C.O.V.	5% probability level	95% probability level
Reference	6	2.264	0.009	0.40%	2.249	2.279
Ambient	6	2.279	0.009	0.39%	2.264	2.293
Hot	6	2.319	0.002	0.09%	2.315	2.323
Wet	6	2.285	0.009	0.39%	2.269	2.300

The results of the BRD testing are graphically presented in Figure 25 below:

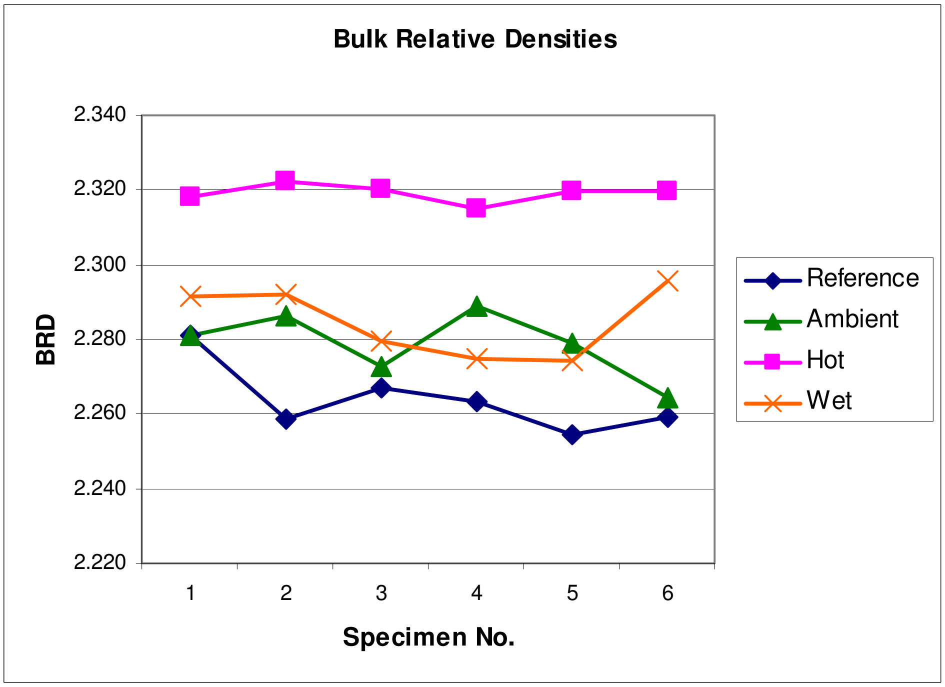


Figure 25: BRD test results on asphalt cores

From Table 11 above it can be seen that the standard deviation of the test results for each section is low and of the same order. The variance within the test results is very low (maximum COV is 0.40%). From the data it can be seen that the asphalt wearing course has densified during all three MMLS tests (ambient, hot and wet) in comparison to the reference section (untrafficked). It can also be seen that densification is most pronounced under the hot test. Except for the densities of the cores subjected to the hot MMLS test, the coefficient of variance of the BRD testing is such that statistically no densification can be distinguished for the ambient and wet MMLS test in comparison with the untrafficked section. One has to however take into account that the cores are from the bottom part of the asphalt surfacing layer since the Novachip™ layer has been removed. It is therefore possible that the main part of the densification has taken place in the top part of the asphalt surfacing.

4.2 Indirect tensile strength (ITS) testing

The ITS testing on the asphalt specimens has been carried out at 25ºC and at a constant displacement rate of 0.847mm per second (Marshall speed). The results are shown in Table 12 overleaf.

Table 12: ITS test result on asphalt cores

Section	Number of specimens	Average ITS [kPa]	Standard deviation	C.O.V.	5% probability level	95% probability level
Reference	3	1139	48	4.2%	1059	1218
Ambient	3	1205	34	2.8%	1148	1261
Hot	4	1139	53	4.7%	1052	1226
Wet	4	1188	102	8.5%	1021	1356

The results of the ITS testing is graphically presented in Figure 26 below.

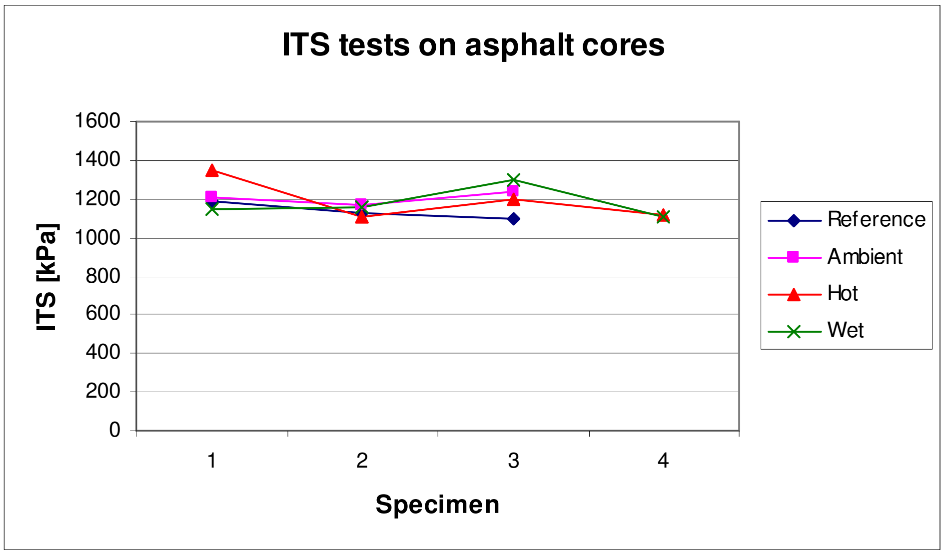


Figure 26: Results of ITS testing on asphalt cores

The test result show acceptable variance. From the test results it can be concluded that no difference in ITS can be determined for the different types of MMLS3 testing. The test has a reasonable repeatability (low standard deviation), but little discriminating ability. Except for the fact that the ITS has a reasonable value (it exceeds the COLTO minimum requirement of 800 kPa for surfacing mixes by far), no valuable information can be derived from this test in regard to the effect of the different types conditioning during the MMLS3 testing.

A Student t-test has been carried out on the data sets to determine the probability the samples have similar underlying populations with the same mean. These probabilities are summarised in the matrix overleaf (Table 13).

Table 13: Results of Student t-test

	Reference	Ambient	Hot	Wet
Reference	X	0.13	0.45	0.48
Ambient	0.13	X	0.82	0.60
Hot	0.45	0.82	X	0.86
Wet	0.48	0.60	0.86	X

As can be seen from Table 13 the t-test indicates that only for the reference and ambient samples there is a small probability that these samples form part of the same population. All other combinations show a fair to high probability that the samples form part of the same population. It can therefore be concluded that the ITS test as carried out does not indicate any significant difference in material properties for the different sections.

4.3 Indirect Tensile testing (ITT)

A total of three asphalt cores of each section have been subjected to Indirect Tensile Testing to determine the resilient modulus. Each specimen has been subjected to ITT testing twice, the second one being rotated 90º compared to the first one. As much as possible cores with identical BRD values have been selected. The ITT tests were carried out at a temperature of 25ºC. A haversine loading of 750N with static pre-load 100N was applied at a frequency of 10Hz.

The results of the resilient modulus testing are summarised in Table 14 below:

Table 14: Indirect Tensile Test results asphalt cores

Section	Number of specimens	Average Mr [MPa]	Standard deviation	C.O.V.	5% probability level	95% probability level
Reference	3	8173	1228	15.0%	6153	10192
Ambient	3	8001	913	11.4%	6499	9502
Hot	3	9767	294	3.0%	9284	10250
Wet	3	7699	59	0.8%	7602	7795

From the data given in Table 14 normal distribution curves can be derived for the resilient moduli of the cores of the different sections as shown in Figure 27 below:

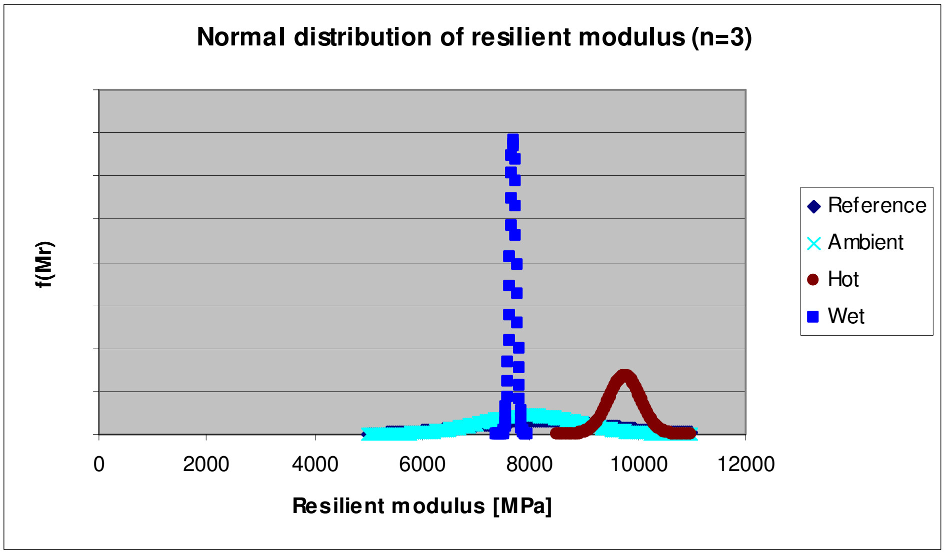


Figure 27: Normal distribution curves of resilient moduli asphalt cores

From these graphs it can be seen that little distinction can be made between the average resilient moduli measured for the cores from the reference ambient and wet sections. Furthermore it can be concluded that the distribution for the resilient modulus for cores form the reference in ambient section show a very wide distribution (high coefficient of variance). As for the bulk density however, a marked increase in the resilient modulus of cores from the hot section can be established. There is possibly a direct relation between the density of the cores and the resilient modulus and asphalt cores with a higher density showing increased resilient moduli.

The average resilient modulus of the wet trafficked asphalt cores is lower than those for the cores of the reference section and the ambient and hot trafficked cores. This could be the result of water damage. It can however not be concluded that moisture damage took place because of the high standard deviations of the resilient modulus of the reference section and ambient trafficked cores, while at the same time the occurrence of moisture damage cannot be excluded as a possibility.

The ITT tests on the cores from the foamed bitumen treated base were carried out at a temperature of 25ºC. A haversine loading of 500N with static pre-load 100N was applied at a frequency of 10Hz.

The results of the resilient modulus testing are summarised in Table 15 below:

Table 15: Indirect Tensile Test results foamed bitumen treated base cores

Section	Number of specimens	Average Mr [MPa]	Standard deviation	C.O.V.	5% probability level	95% probability level
Reference	2	686	5	0.7%	677	694
Ambient	3	521	269	51.6%	78	964
Wet	3	629	241	38.3%	233	1026

The results of the resilient modulus testing of the cores extracted from the foamed bitumen treated base course show a wide distribution and combined with the fact that only few samples have been tested this makes it difficult to confidently determine a resilient modulus for the material.

From the results in Table 14 and Table 15 no clear trend of the influence of the different types of conditioning on the resilient modulus can be established. As for the ITS test it may be concluded that also the Indirect Tensile Test is in this case not a good test to determine the difference in material properties of the different sections. One would have to explore other means of testing (*e.g.* SCB testing or fatigue testing) in order to determine the influence of the different types of conditioning on the material properties.

5 Comparison between MMLS3 and HVS

5.1 Summary of HVS data

Two sections on the slow lane of the southbound carriageway have been trafficked by the HVS and were monitored by the CSIR. These two sections have the identifiers 415A5 and 416A5 respectively.

Section 415A5 has first been trafficked using a wheel load of 40 kN (80kN axle load) and a tyre pressure of 620 kPa. After 48,623 HVS load repetitions the wheel load was increased to 80kN and the tyre pressure to 800 kPa. At completion of the test 498,863 HVS load repetitions had been applied.

Section 416A5 was also first trafficked applying a wheel load of 40 kN and a tyre pressure of 620 kPa. After 1,168,850 HVS load repetitions the wheel load was increased to 80 kN with a tyre pressure of 800 kPa. At completion of the test 1,742,850 HVS load repetitions had been applied.

Both test sections have been tested under ambient temperature conditions. After 1,618,850 HVS load repetitions on section 416A5 water was continuously applied to the surface of the test section for the remainder of the test. Therefore the last 124,000 HVS load repetitions on 416A5 were under wet conditions.

Table 16: Summary of HVS testing details section 415A5 and 416A5

Section	HVS Load repetitions		Wheel load	Tyre pressure	Conditioning
	Start	End			
415A5	0	48,623	40 kN	620 kPa	Dry
	48,623	498,863	80 kN	800 kPa	Dry
416A5	0	1,168,850	40 kN	620 kPa	Dry
	1,168,850	1,618,850	80 kN	800 kPa	Dry
	1,618,850	1,742,850	80 kN	800 kPa	Wet

In this chapter the data resulting from the abovementioned test section as provided by the CSIR has been used and will be discussed below.

5.1.1 Rutting

The monitoring of the permanent deformation at progressive loading under HVS loading consisted of two types of measurement. Firstly the surface deformation was monitored by means of manually measuring the rut depth using a straight edge and secondly, multi-depth deflection (MDD) meters were installed in the test section, which provided data of permanent deformation at different depths into the pavement.

For a detailed description of these measurement methods reference is made to the relevant CSIR reports.

The straight edge rut measurements were taken at a number of cross-sections with 500mm intervals (from points 2 up to 12 as labelled by the CSIR). At each cross section 5 points were measured along the cross-section (200, 400, 500, 600 and 800mm).

The top cap of the MDD's was placed 20mm below the surface level. This is approximately in the middle of the asphalt layer. The other sensors were placed at a depth of 303mm, just below the bottom of the foamed bitumen stabilised base, at a depth of 473mm, well below the crushed stone subbase in the in-situ subgrade and at 650mm and 850mm depth in the subgrade. The MDD's were anchored at a depth of 3m and all deformation is relative to this anchor point. At each test section 3 MMD's were installed at point labelled by the CSIR as 4, 8 and 12 respectively.

The rut development is summarised in Table 17 below. These values are read off from graphs of the straight edge measurements provided by the CSIR. The graphs are included in ***Appendix C*** to this report. The MDD raw data was also provided by the CSIR.

Table 17: Rut development based on information provided by CSIR

Section	HVS Load repetitions	Straight edge rut [mm]	MDD deformation at 20mm[1] [mm]	Remarks
415A5	48,623	1.3	0.2	End of 40 kN load interval
	200,000	4.7	2.9	Comparable no. of load repetitions
	498,863	7.4	5.8	End of test. Load increased to 80 kN after 48,623 repetitions
416A5	200,000	2.5	0.6	Comparable no. of load repetitions
	1,168,850	3.1	1.7	End of 40 kN load interval
	1,526,450	5.0	4.0	Last MMD measurement available
	1,618,850	5.9	no data	Load increased to 80 kN during this interval
	1,742,850	14.6	no data	End of test. Water added during this interval

1) For section 415A5 the average of the 3 MDD's installed is used. For section 416A5 however only MDD8 is used, because it appears that MDD's 4 and 12 were not anchored correctly (they show negative permanent deformation below the 303mm level)

The difference between the straight edge rut measurement and the deformation measured by the MDD's at 20mm could be the result of the following:

- Deformation of the top 20mm of the asphalt surfacing (almost entirely consisting of the Novachip™ layer) is not included in the total MDD deformation;
- Accuracy of the manual straight edge measurements;
- Placement of the straight edge on a changing reference (heaving next to wheel path);
- Testing error in the MDD's measurements.

On average the straight edge measurements read 1.5mm more than the MDD deformation. It is assumed that this is the permanent deformation in the Novachip™ layer (top 20mm).

5.1.2 Temperature

The pavement temperature of the test sections has been monitored continuously by the CSIR using temperature buttons, which recorded the temperature at 2-hourly intervals. These temperature buttons were installed at each side of the test sections (caravan side and traffic side) at points labelled by the CSIR as 4 and 12. At each point a button was installed both at the surface and at 35mm depth in the asphalt layer. It was indicated by the CSIR that the temperature button at point 12 on the traffic side at 35mm depth had not recorded any data.

The raw data supplied by the CSIR was analysed by the University of Stellenbosch to be used in the comparison later on in this chapter. The temperature fluctuations are shown in the graphs in ***Appendix D***. Table 18 below shows what percentage of the total time the pavement temperature was within a certain temperature range.

Table 18: Section 415A5 Pavement temperature ranges, percentage of total time

Point	side	Depth	Temperature range					
		[mm]	<15ºC	15ºC ≤ T < 20ºC	20ºC ≤ T < 25ºC	25ºC ≤ T < 30ºC	30ºC ≤ T < 35ºC	≥ 35ºC
4	TS	0	5.7%	51.2%	32.9%	7.6%	2.4%	0.2%
		35	10.5%	53.3%	24.5%	6.2%	4.9%	0.6%
	CS	0	5.5%	48.5%	35.8%	8.6%	1.5%	0.2%
		35	4.5%	40.8%	44.1%	10.2%	0.2%	0.2%
12	TS	0	8.3%	53.2%	29.8%	4.2%	3.6%	1.0%
		35						
	CS	0	6.2	48.6%	31.8%	10.7%	2.3%	0.5%
		35	6.5%	50.1%	32.9%	9.4%	0.8%	0.3%

Table 19: Section 416A5 Pavement temperature ranges, percentage of total time

Point	side	Depth	Temperature range						
		[mm]	<15ºC	15ºC ≤ T < 20ºC	20ºC ≤ T < 25ºC	25ºC ≤ T < 30ºC	30ºC ≤ T < 35ºC	35ºC ≤ T < 40ºC	≥ 40ºC
4	TS	0	0.9%	9.5%	43.7%	31.7%	8.4%	4.9%	0.8%
		35	0.4%	9.8%	47.4%	26.6%	10.6%	4.0%	1.1%
	CS	0	0.9%	8.5%	45.2%	29.6%	12.3%	2.9%	0.7%
		35	0.5%	6.6%	39.3%	37.4%	14.8%	1.1%	0.2%
12	TS	0	1.4%	13.2%	45.8%	27.3%	7.6%	3.6%	1.2%
		35							
	CS	0	0.8%	10.5%	46.2%	26.9%	11.1%	3.8%	0.7%
		35	0.9%	10.3%	47.7%	27.5%	11.9%	1.3%	0.5%

The data of Table 18 and Table 19 is graphically presented in pie-charts in ***Appendix E***. The maximum surface temperature measured was 39.0 ºC and 46,5ºC on section 415A5 and 416A5 respectively. The 95-percentile temperature on section 415A5 is approximately 30ºC and the 50-percentile approximately 20ºC. For section 416A5 these values are approximately 35ºC and 25ºC respectively. The difference in temperature between the two sections results from the fact section 415A5 was tested during spring and section 416A5 during summer.

It is noted that during testing the pavement temperature has not been very high. In fact the pavement temperatures during which the greater part of the load applications were made can be considered much lower than critical pavement temperatures in terms of rut development. This might be the result of the fact that the test sections of the HVS are always protected from direct sunlight by the roof construction of the HVS and only during the early mornings and late afternoons the test section is exposed to direct sunlight, but at those times of the day the intensity of the sunlight is much less than during mid-day. Therefore asphalt pavements tested by HVS under ambient conditions will never reach similar critical pavement temperatures that do exist during real traffic loading.

The pavement temperature prevailing during loading is very important in analysing rut development in asphalt pavements. Full-scale real truck loading accelerated pavement testing at the NCAT pavement test track in Alabama has shown that rut development was restricted to those time periods at which surface temperatures exceeded 40ºC and for the asphalt pavements tested it was found that below this temperature the pavement may be trafficked without any significant increase in rutting [3]. In relation to the aforesaid it is noted that during the HVS testing on sections 415A5 and 416A5 the pavement temperatures only exceeded 40ºC for less than 1% of the total testing time.

Another important parameter is the temperature gradient in the asphalt pavement. As noted above, the temperature of the HVS sections has been monitored both at the surface and 35mm into the asphalt pavement. The two points at the caravan side show very small temperature gradients (both sections) with 95-percentile values of less than 2.5ºC gradient (50-percentile is 0ºC gradient). This is the eastern side with only exposure during direct sunlight for short time window during the morning.

The traffic side (western side) however, shows a much larger temperature gradient. Unfortunately the pavement temperature at 35mm into the pavement has only been recorded at one point on this side. The 95-percentile value of the temperature gradient on this side is approximately 12,5ºC and the 50-percentile approximately 3.5ºC.

5.1.3 Lateral wander

The HVS loading has been applied using lateral wander. The distribution curve has been provided by the CSIR and is reflected in Figure 28 . As can be seen from Figure 28 the distribution has a triangular shape. The total width of the wheel path with lateral displacement of the HVS is 1000mm. The total width of the wheel path in case of unilateral trafficking would be 650mm. Hence when trafficking with lateral wander there is always an overlap in the middle and the number of wheel passes in the middle is independent on the mode of trafficking (with or without lateral wander). The MDD's have been installed in the middle of the wheel path and it is assumed that the maximum deformation measured by means of the straight edge also occurred in the middle. Therefore in the conversion of the number of load repetitions with lateral wander to a number of load repetitions without lateral wander a 1 : 1 ratio can be used, because the number of load repetitions in the middle is the same in both cases.

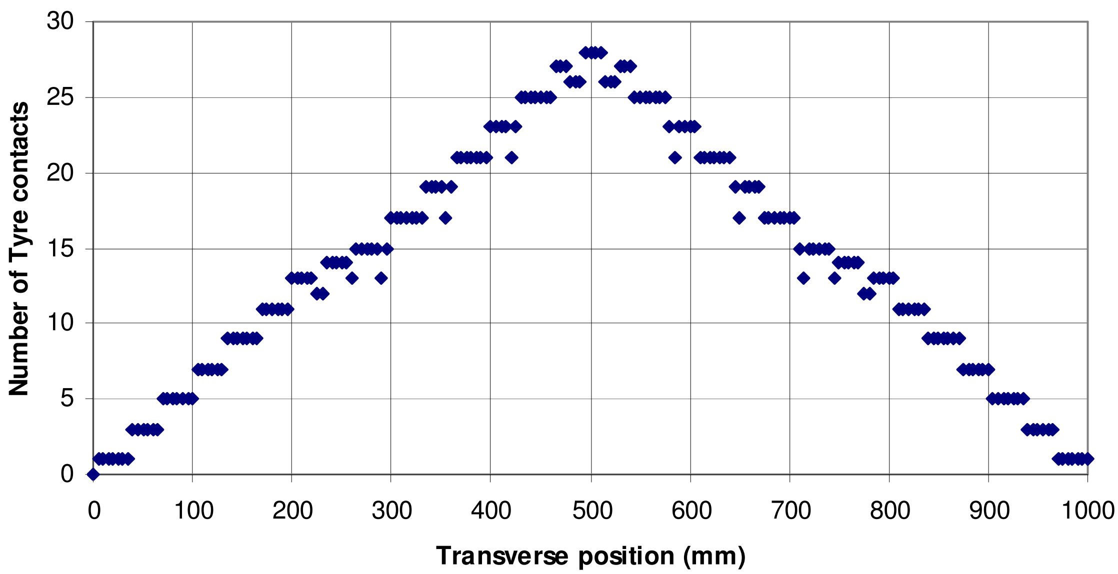


Figure 28: HVS distribution curve

5.1.4 Direction of trafficking

Whereas for the MMLS3 the wheels are continuously running in the same direction, the wheel of the HVS is moving back and forth. The influence of applying loads in either one or two directions is studied by various researchers. These studies have come up with contradicting results. A comparative study in Finland showed that there was no difference between unidirectional and bi-directional trafficking. White *et al.* and found Brown *et al.* concluded that bi-directional trafficking increases the rutting measured.

Hence is it difficult to make any assumptions on the influence of direction of trafficking. Hugo *et al.* analytically concluded that there is a difference, but that this is only valid for deep asphalt pavements. The type of pavement structure thereof also plays a role in the influence of direction of trafficking.

In this study no effort has been made to determine the influence of unidirectional vs. bi-directional trafficking as it is believed that this falls outside the scope of this project. The tests undertaken were also not aimed at possibly determining differences as a result of direction of trafficking. In the comparison between the rutting under HVS loading and under MMLS3 loading the influence of direction of trafficking has been omitted.

5.2 Theories for comparison

A procedure to compare MMLS3 rutting to full-scale traffic rutting has first been developed by Walubita *et al.* [4] in the evaluation of MMLS3 in comparison with the TxMLS for test results in Jacksboro (US). This procedure has further been developed by Epps et al. [5] for the MMLS3 testing carried out at Westrack (US) and Smit et al. [3] for the MMLS3 testing at the NCAT test track (US). The procedure is also adopted here and described below.

The first element in the comparison is the determination of a *Field Rutting Ratio (FRR)*. This is the ratio between the measured *Rut Depth (RD)* values under MMLS3 loading and full-scale loading after equal number of load repetitions. In determining the equal number of load repetitions one has to take into account lateral wander, if applicable.

$$FRR = \frac{RD_{MMLS3}}{RD_{HVS}} \tag{1}$$

A *Theoretical Rutting Ratio (TRR)* can also be defined as the ratio of rutting between the MMLS3 and HVS using stress analyses under both APT devices.

$$TRR = \frac{Theoretical\ RD_{MMLS3}}{Theoretical\ RD_{HVS}} \tag{2}$$

It is difficult to determine theoretical rut depths for both MMLS3 loading and HVS loading and therefore the assumption is made that the ratio between these two theoretical rut depths is equal to the ratio between the potential for rutting to develop under both types of loading. This potential is assumed to be dependent on the vertical compressive stress. The *Rutting Potential Ratio (RPR)* is defined as follows:

$$RPR = \frac{TFC \times SP_{MMLS3}}{SP_{HVS}} \tag{3}$$

Where

SP = *Stress Potential*
TFC = *Temperature-Frequency Correction*

The *Stress Potential* is defined as the area under the vertical compressive stress vs. depth curve up to the depth of influence.

$$SP = \int_{z1}^{z2} \sigma_v dz \tag{4}$$

The *Temperature-Frequency Correction (TFC)* accounts for differences in temperature and loading time between the MMLS test and HVS test.

In an ideal situation and with all influencing factors properly accounted for the *Rutting Potential Ratio* is equal to the *Field Rutting Ratio* and hence the ratio between the two would be unity. This ratio is defined the *Prediction Ratio* ($PR_{rutting}$).

$$PR = \frac{TRR}{FRR} \tag{5}$$

If this *Prediction Ratio* is known and validated it can be used to predict rutting of full-scale trafficking based on rutting under the MMLS3.

5.3 Comparison of rutting

5.3.1 Rutting potential ratio

As discussed in the previous section, the determination of the rutting potential ratio is used to approximate the theoretical rutting ratio and the principle of stress potential is applied. The linear elastic multi-layer program BISAR [2] has been used to carry out

the stress analyses for the determination of the stress potential. The pavement structure has been modelled as detailed in
Table 20. For the stiffness of the FBT base layer a value of 650 MPa as determined in the ITT tests (see section 4.3) has been used. The stiffness of the FBT base layer has is assumed to be non-viscous linear elastic.

The Novachip™ and asphalt wearing course layer however are assumed to be visco-linear elastic and their stiffness is dependent on the temperature and loading time. The stiffness values determined in the ITT test give an indication of the stiffness that can be expected under field conditions and loading, however, with no means available to accurately determine the time and temperature dependant behaviour of the Novachip™ and the dense asphalt wearing course, quite rough estimates of the stiffness of these two layers under HVS and MMLS loading had to be made. The loading conditions and assumed stiffness are summarised in Table 21. The influence of the stiffness of the asphaltic layers on the stress potential has been evaluated by performing the calculations with different assumed stiffnesses. The influence thereof on the stress potential is, because of the relative thick and stiff FTB base layer, rather limited.

Based on this limited dependency of the stress potential on the stiffness of the asphaltic layer, the estimates of the stiffness summarised in Table 21 are deemed justifiable and will result in a good first approximation of the rutting potentials.

Table 20: Pavement structure used for stress analyses

Layer	Thickness [mm]	Stiffness [MPa]	Poisson's ratio [-]
Novachip™ layer	19	variable	0.44
Dense Asphalt wearing course	35	variable	0.44
FBT base	250	650	0.35
Granular sub-base	125	150	0.35
Subgrade	Infinite	100	0.35

Table 21: Loading times, pavement temperatures and asphalt stiffness

Type of trafficking	Loading time [sec]	Pavement temperature [º Celsius]	Stiffness[1] [MPa]
HVS 415A5	0.08	20	4000
HVS 416A5	0.08	25	3300
MMLS3 ambient	0.08	30	2700
MMLS3 hot	0.08	50	1200
MMLS3 wet	0.02	40	1600

1) The stiffness values are 'best estimates'. A sensitivity analysis has been carried out which showed that variation in the assumed stiffness value have a minor effect on the stress potential.

The vertical compressive stress in the upper 200mm of the pavement structure for all relevant loading conditions is reflected in Figure 29 below.

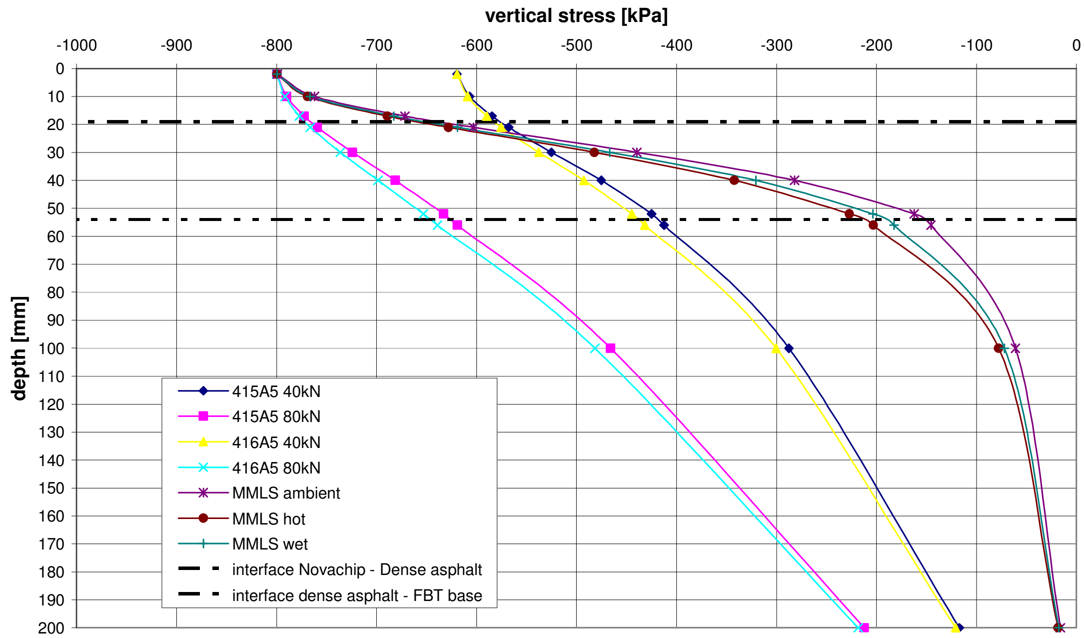


Figure 29: Vertical compressive stress under HVS and MMLS loading conditions

The stress potentials (refer equation 4) have been determined as the area under the vertical stress curves. Based on the assumption made in section 3.3.2 that the MMLS did not induce any permanent deformation in the FBT-base layer, the depth of influence used for comparing rutting has taken as 54mm (the asphalt – FBT base interface). This depth is chosen for the purpose of comparison and it does not mean that either of the APT devices have no influence below this depth. The stress potentials are summarised in Table 22 below.

Table 22: Stress potentials

Loading	Stress potential [kPa.mm]
415A5 40kN	28766
416A5 40kN	29327
415A5 80kN	39315
416A5 80kN	39873
MMLS ambient	26646
MMLS hot	28544
MMLS wet	27862

Because section 415A5 was first trafficked with a 40 kN load for the first 50,000 repetitions and subsequently the load was increased to 80 kN, the rut developed after 200,000 load repetitions is the result of loads with different stress potentials (a stress potential of 28766 for the 40 kN load and a stress potential of 39315 for the 80 kN load). A pro-ratio stress potential according to the rutting developed during the two phases was determined at 27711 to be used in the comparison of rutting after 200,000 load repetitions.

5.3.2 Field rutting

Since the depth of influence for comparison has been taken as 54mm (interface asphalt – FBT base layer), also only rutting occurring in this part of the pavement need to be taken as field rutting. For the MMLS tests all permanent deformation is assumed to have taken place in the asphaltic layers. This is however not the case for the HVS test.

The MDD data obtained is helpful in the determination what part of the total permanent deformation occurred in the several layers, however not conclusive. Unfortunate in this regard is the following:

- The MDD's did not record permanent deformation in the Novachip™ layer (top 20mm, refer section 5.1.1), as the first MDD sensor was installed on the interface between the Novachip™ layer and dense asphalt layer;
- The second MDD sensor was installed at the bottom of the FBT base layer, which makes it only possible to determine the permanent deformation in the dense asphalt layer and FBT base layer combined, but not in each layer separately.

The first problem can be overcome by assuming that the average difference of 1.5mm between the straight edge measurements and the MDD measurements can be regarded as the permanent deformation in the Novachip™ layer. It is furthermore assumed that this 1.5mm of permanent deformation develops during the first phases of loading, prior to permanent deformation developing in the layers deeper into the pavement structure and that the deformation in the Novachip™ layer subsequently remains constant while permanent deformation develops in the layers below.

The second problem makes that it is only possible to determine what the combined permanent deformation in the dense asphalt layer and FBT base layer is and not what the individual contributions of each of these two layers is to the total permanent deformation of the two combined. The combined deformation of the dense asphalt layer and the FBT base layer is summarised in Table 23 below.

The column with the MDD total deformation indicates the average deformation that is measured by the MDD's over the full depth and includes deformation in all layers below interface between the dense asphalt and the Novachip™ layers. It does therefore not include the deformation in the Novachip™ layer itself. The next column (combined permanent deformation AC + FBT base layer) indicates the deformation that took place in dense asphalt layer and the FBT base layer together. This is the deformation measured between the top of the dense asphalt layer and the bottom of the FBT base layer.

Table 23: Permanent deformation in dense asphalt layer and FBT base layer

Section	Load repetitions	MDD total deformation [mm]	Combined permanent deformation AC+ FBT base layer [mm]	Percentage of total MDD deformation
415A5	200,000	2.9	2.0	70%
416A5	200,000	0.6	0.5	88%

Taking into account the behaviour of both the dense asphalt layer and the FBT base layer it is expected that the first would show much more permanent deformation than the latter, however, this has not been confirmed by actual measurements. Therefore the following scenarios have been analysed:

1. An extreme case with all permanent deformation in the AC + FBT base layer attributed to the dense asphalt layer and none to the FBT base layer;
2. The other extreme case where all permanent deformation in the AC + FBT base layer is attributed to the FBT base layer and none to the dense asphalt layer; and
3. A ‘most likely’ scenario, where 75% of the deformation measured by the MDD for the dense asphalt layer and FBT base layer combined is attributed to the dense asphalt layer and the other 25% to the FBT base layer.

Taking into account the above the rut depths to be used in the comparison have been determined and summarised in Table 24 below.

Table 24: Rut depths of the HVS and MMLS test after comparable number of load repetitions

Loading	Rutting after 200,000 load repetitions [mm]		
	Scenario		
	1	2	3
415A5	3.5	1.5	3.0
416A5 40kN	2.0	1.5	1.9
416A5 80kN	Only commenced after more than 200,000 40kN load repetitions		
MMLS ambient	1.7		
MMLS hot	4.1		
MMLS wet	3.5		

Note: The assumed deformation in the dense asphalt layer based on the percentages given in Table 23 is firstly determined, after which the assumed deformation in the Novachip™ layer has been added

Because the 80kN loads were only applied to test section 416A5 after 1,168,850 load repetitions of 40kN, it is not possible to compare the effect of the 80kN load on section 416A5 with the MMLS loading and will therefore be left out of this comparison. In the comparison of the rut depths measured under MMLS3 loading and HVS loading as indicated in Table 24, preference should go to the comparison with the 416A5 test result, because this was trafficked by one load type only (40 kN). The fact that during first 200,000 load repetition of the 415A5 test two types of loading (40kN and 80kN) were applied might hamper the comparison.

5.3.3 Rutting ratio's

As approximation for the TFC factor, the inverse of the ratio of the stiffnesses of the HMA layer under the two types of APT loading as indicated in Table 21 has been used. These factors are summarised in Table 25 below.

Table 25: TFC factor used (inverse of the stiffness ratio)

	HVS 415A5	HVS 416A5	MMLS ambient	MMLS hot	MMLS wet
HVS 415A5[1]	-	0.83	0.68	0.30	0.40
HVS 416A5	1.21	-	0.82	0.36	0.48
MMLS ambient	1.48	1.22	-	0.44	0.59
MMLS hot	3.33	2.75	2.25	-	1.33
MMLS wet	2.50	2.06	1.69	0.75	-

The rutting potential ratio’s (refer equation 3) are summarised in Table 26 below:

Table 26: Rutting potential ratio's (RPR)

		HVS 415A5	HVS 416A5	MMLS ambient	MMLS hot	MMLS wet
HVS 415A5[1]	Sc. 1: Sc. 2: Sc. 3:	-	0.966 0.780 0.943	0.870 0.702 0.849	0.361 0.291 0.352	0.493 0.398 0.481
HVS 416A5	Sc. 1: Sc. 2: Sc. 3:	1.035 1.283 1.061	-	0.901	0.374	0.510
MMLS ambient	Sc. 1: Sc. 2: Sc. 3:	1.149 1.425 1.178	1.110	-	0.415	0.567
MMLS hot	Sc. 1: Sc. 2: Sc. 3:	2.771 3.434 2.839	2.677	2.410	-	1.366
MMLS wet	Sc. 1: Sc. 2: Sc. 3:	2.028 2.514 2.078	1.959	1.765	0.732	-

1) The RPR’s for the three scenarios are given. These differ because the stress potential of the combined loading (40kN + 80kN) is assumed to be dependant on the rutting

2) Sc. 1 = scenario 1; Sc. 2 = scenario 2, etc.

The field rutting ratio’s (refer equation 1) are summarised in Table 27 below.

Table 27: Field rutting ratio's (FRR)

		HVS 415A5	HVS 416A5	MMLS ambient	MMLS hot	MMLS wet
HVS 415A5	Sc. 1: Sc. 2: Sc. 3:	-	1.750 1.000 1.579	2.059 0.882 1.765	0.854 0.366 0.732	1.000 0.429 0.857
HVS 416A5	Sc. 1: Sc. 2: Sc. 3:	0.571 1.000 0.633	-	1.176 0.822 1.118	0.488 0.366 0.463	0.571 0.429 0.543
MMLS ambient	Sc. 1: Sc. 2: Sc. 3:	0.486 1.133 0.567	0.850 1.133 0.895	-	0.415	0.486
MMLS hot	Sc. 1: Sc. 2: Sc. 3:	1.171 2.733 1.367	2.050 2.733 2.158	2.412	-	1.171
MMLS wet	Sc. 1: Sc. 2: Sc. 3:	1.000 2.333 1.167	1.750 2.333 1.842	2.059	0.854	-

Based on the data provided in Table 26 and Table 27 the prediction ratio (refer equation 5) can be determined. The prediction ratio's are summarised in Table 28 below.

Table 28: Prediction ratio's (PR)

		HVS 415A5	HVS 416A5	MMLS ambient	MMLS hot	MMLS wet
HVS 415A5	Sc. 1: Sc. 2: Sc. 3:	-	0.552 0.780 0.597	0.423 0.796 0.481	0.423 0.796 0.481	0.493 **0.928** 0.561
HVS 416A5	Sc. 1: Sc. 2: Sc. 3:	1.811 1.283 1.675	-	0.765 **1.021** **0.806**	0.766 **1.021** **0.806**	**0.893** **1.191** **0.940**
MMLS ambient	Sc. 1: Sc. 2: Sc. 3:	2.367 1.257 2.079	1.306 **0.980** **1.241**	-	**1.001**	**1.167**
MMLS hot	Sc. 1: Sc. 2: Sc. 3:	2.365 1.256 2.077	1.306 **0.979** **1.240**	**0.999**	-	**1.166**
MMLS wet	Sc. 1: Sc. 2: Sc. 3:	2.028 **1.077** 1.782	**1.120** **0.840** **1.064**	**0.857**	**0.858**	-

Note: The tests that have prediction ratio's approaching unity are indicated in bold

A prediction ratio with the value equal or close to one indicates that there is a good correlation between the ratio of the actual field rutting resulting from trafficking with the HVS and MMLS3 and the ratio of the theoretical rutting potential of both APT devices. Hence, in cases where the prediction ratio is close to one, the field rutting measured after MMLS3 trafficking can be used to predict rutting under the HVS and vice versa.

From Table 28 it can be seen that a number of good correlations exists. Good correlations exist between the HVS 416A5 test and all three MMLS3 tests, mostly in case of scenario 2 and 3. Scenario 2 is however the 'most unlikely' scenario. Excellent correlations exist between the three different type of MMLS3 tests (ambient, hot and wet). However, poor correlations were found for most combinations that involved HVS test 415A5.

The poor correlations that exist for most of the combinations involving the HVS 415A5 test may be attributed to the fact that both 40 and 80 kN loading were applied during the first 200,000 load repetitions. Although it was tried to account for this by determining the combined stress potential, it may have lead to distorted correlations.

The fact that excellent correlations exist between the three MMLS3 tests is promising. The reliability of the method of the comparison is however compromised by the fact that the 'most unlikely' scenario (No. 2) seems to result in the best correlations involving both or one of the HVS tests. A critical analysis of the comparison is therefore required.

The following factors that influence the accuracy of the prediction ratio need to be considered:

- In the analyses of the HVS data a number of assumptions had to be made in order to be able to determine the total permanent deformation, as well as the permanent deformation in the HMA layer only. Problematic in this regard was:
  1. The fact that the deformation in the Novachip™ layer was not captured in the MDD measurements, and
  2. The permanent deformation in the HMA layer and FBT base layer combined could be determined using the MDD results, but not the individual deformation in each of these two layers.

  The degree of correctness of rutting determined for the HVS tests, based on the assumptions as detailed in this report has a great influence on the accuracy of the calculated prediction ratio's;

- HVS section 415A5 has been trafficked using different wheel loads during the first 200,000 load repetitions (both 40kN and 80kN) and this makes it difficult to distinguish between the individual influences of each of these wheel loads. Assumptions had to be made to determine a stress potential for the combined influence of the 40kN and 80kN wheel loads;

- The theory for comparison has been developed and validated for MMLS3 tests on deep asphalt pavements. The pavement tested on the N7 differs structurally from these type of full depth asphalt pavements and the N7 pavement structure can be characterised as a composite pavement structure with a relative thin flexible surfacing (the Novachip™ layer and HMA layer) on top of a stiff granular base layer;

- The vertical compressive stresses in the pavement have been calculated using a linear elastic multi layer computer program, whereas both HMA as well and the FBT base material are not linear elastic materials;

- The stiffness of the asphalt surfacing at different temperatures has been estimated based on limited laboratory tests. In this regard however, it has been determined that the stiffness only has a limited influence on the calculated stress potentials;

- Although for full depth asphalt pavements rutting prediction ratio's based on only the vertical compressive stress resulted in good correlation in terms of prediction ratio's, it might be that for composite pavements with a relative thin asphalt surfacing layer, such as the pavement structure tested on the N7, the determination of a stress potential based on only the vertical compressive stress does not suffice. Other factors, such as the horizontal stress and deviator stress may also need to be considered;

- It has been stated previously that the effect of one-directional trafficking versus two-directional trafficking would not be taken into account here;

- The contact stresses under the tyres are assumed to be uniform, which is not a correct assumption. Locally contact stresses can peak up to twice the tyre pressure (refer de Beer). This has not been taken into account in this analysis.

6 Conclusions and recommendations

6.1 MMLS3 field tests

Five MMLS3 tests up to 200,000 load repetitions each have been successfully completed on the N7 at the HVS test site during December 2002 and January 2003. The test have been carried out at ambient conditions, at elevated pavement temperature conditions and with moisture applied at elevated temperatures. Three MMLS3 test were carried out on the surface and two on the base.

The MMLS3 tests were carried out with a wheel load of 2.2 kN (1/3 of 40 kN dual wheel HVS load) and a tyre pressure of 800 kPa. The loading time during the surface ambient and surface hot test was adjusted such that it is similar to the HVS loading time (0.08s). The remainder of the MMLS3 test were carried out with a loading time of 0.02s.

The MMLS3 tests on the surface with elevated pavement temperature (50ºC) showed the most rutting (4.1mm), followed by the wet test with elevated pavement temperature (40ºC; 3.5mm) and a final rut of 1.7mm was measured during the ambient test on the surface.

The MMLS3 tests on the base showed very little rutting (0.8mm for the base ambient test and 1.5mm for the base wet test (30ºC)). It is believed that the main part of this rutting took place in 10 – 15mm slurry layer that was placed on top of the base after the asphalt surfacing was removed and that very little rutting took place in the foamed bitumen treated base layer.

6.2 Laboratory testing

The laboratory tests on cores extracted from the tested sections show that the dense asphalt layer densified during the surface hot test. No increase in density in the dense asphalt layer could be established during the surface ambient and surface wet test.

Densification in the Novachip™ layer has not been monitored.

The ITS tests carried out on the asphalt cores do not show difference in tensile strength of the dense asphalt based on different type of conditioning (ambient, hot, wet).

Resilient modulus testing (ITT) on the asphalt cores show an increase in resilient modulus for the hot trafficked samples, which can be related to the increased density. No trends could be determined for the other types of conditioning. It could not be concluded that moisture damage took place because of the high standard deviations of the resilient modulus of the reference section and ambient trafficked cores. At the same time the occurrence of moisture damage could not be excluded as a possibility.

It can be concluded that, for the asphalt samples tested, the ITS test and the ITT are not suitable to indicate changing material properties resulting from different types of conditioning. It is recommended that in order to be able to determine the influence thereof one should consider more sophisticated testing such as fatigue testing in the SCB test set-up.

It has shown to be problematic to extract intact cores from the foamed bitumen treated layer. Indirect Tensile Tests carried out on cores from the FTB layer show similar average resilient moduli, but an unacceptable high coefficient of variance. The fact that the tested samples (100mm Ø cores) contained large aggregate particles (37.5mm) as well as the difficulties experienced in extracting the cores is probably contributing largely to this variance.

6.3 Comparison of rutting HVS – MMLS3

The field ruts under HVS and MMLS3 loading have been compared using a field rut ratio and a rutting potential ratio.

The stress potentials of the different types of loading have been calculated using only the vertical compressive stresses.

The pavement temperatures during the HVS tests were generally low. Even with high ambient air temperatures and high levels of solar radiation, pavement temperatures under the HVS remain low because of the pavement is covered by the HVS roof construction. This is a factor with considerable effect on the rate rut development in asphalt pavement.

A number of assumptions had to be made to determine the permanent deformation in the asphalt surface layers (Novachip™ and dense asphalt layer) only under HVS loading. The correctness of these assumptions is crucial for the accuracy of the prediction ratio's.

Following the approach of calculating Prediction Ratio's, it was found that very good correlations exist between the three MMLS3 tests (ambient, hot and wet). Good correlations were also found between the three MMLS3 tests and the HVS 416A5 test. However, of these, the best correlations were calculated for the 'most unlikely' scenario in terms distribution of permanent deformation as a result of HVS loading between the HMA layers and the FBT base. Poor correlations were found for the majority of combinations that involve HVS test 415A5. It is believed that the fact that both 40kN and 80kN wheel loads were applied during the evaluated loading duration of this tests may have distorted the correlations.

The number of good correlations in the testing matrix supports the hypothesis that the adopted method can be used to compare rutting resulting from different loading types and that, if rutting resulting from a certain loading type is known, one can predict rutting under other loading types. There are however anomalies in the matrix of Prediction Ratio's, which compromise the reliability of the method of comparison. A critical review of the appropriateness thereof for the tests carried out in this research programme is therefore required.

One of the main factors that influence the reliability of the Prediction Ratio method is the fact that the pavement tested on the N7 is a composite pavement with a flexible hot-mix asphalt surfacing on a rigid granular base. This type of pavement differs from the full depth HMA type of pavement structures that have been used to develop and validate the adopted theory of comparison between the two types of loading.

The influence of unidirectional versus bi-directional trafficking is an important issue, the effect of which has not been established as yet. The effect has not been analysed here and in the comparison the effect, if any, has not been taken into account. The effect of lateral wander has also not been analysed in detail and the MMLS3 test have been carried out without lateral wander.

6.4 Recommendations

It is recommended that should the opportunity arise to do comparative testing on full depth asphalt pavements, an exercise comparable to this one should be undertaken.

It is believed that in research projects of this nature manual measurements (i.e. straight edge measurements) should be limited to a minimum, especially taken into account that the deformations measured here were generally small. It is therefore recommended that surface deformations should be measured with a high resolution and degree of accuracy, while limiting the human testing or measurement error.

More accurate information about the deformation in the different layers of the pavement is required both under MMLS3 loading and HVS loading. Under MMLS3 loading the surface deformation can accurately be measured, but not the deformation within the surface layer or in the base layer. The MDD measurements are useful to determine subgrade deformation or deformation of relative thick layers (say up to 200 – 300 mm) under HVS loading, but not for small deformation in relatively thin surfacing layers (as for the N7 pavement structure). The MDD measurements also do not provide surface deformation. Measuring devices to determine small deformation in surface and upper parts of the base layers should be developed for both HVS and MMLS3 loading. The development of layer deformation pins that are suited for this purpose is in an experimental stage at the University of Stellenbosch and some good results have been obtained already. Further development of these pins should be encouraged.

The influence of pavement temperature is not to be underestimated. With the MMLS3 testing the pavement temperature could be controlled and kept constant, however for the HVS such possibility does not exist. It is recommended that HVS testing (especially for the evaluation of asphalt in pavements) at elevated temperatures be considered.

It is recommended that besides the vertical stress, the influence of other factors *(i.e.* horizontal stresses, deviator stresses, etc.) be included in the determination of the rutting potential ratio.

7 References

1. Shell International Oil Products BV, Bands 2.0 (computer programme), 1998

2. Shell International Oil Products BV, Bisar 3.0 (computer programme), 1998

3. Smit, André. de Fortier., Hugo, Fred. Rand, Dale Powell, Buzz, *Model Mobile Load Simulator Testing at National Centre for Asphalt Technology Test Track*, Transportation Research Record 1832, Journal of the Transportation Research Board, Washington, D. C. USA, October, 2003

4. Walubita, L.F., Hugo, F., Epps Martin, A. L., *Indirect tensile fatigue performance of asphalt after MMLS trafficking under different environmental conditions*, Journal of the SA Institution of Civil Engineering, Johannesburg, South Africa, Vol. 44, Number 3, 2002

5. Martin Epps, A., Ahmed, T., Little, D.C., Hugo, F., Poolman, P. and Mikhail, M, *Performance prediction with the MMLS3 at WesTrack.* CD-Rom of Proceedings of the Ninth International Conference on Asphalt Pavements Copenhagen, 17 – 22 August, 2002

APPENDIX A:

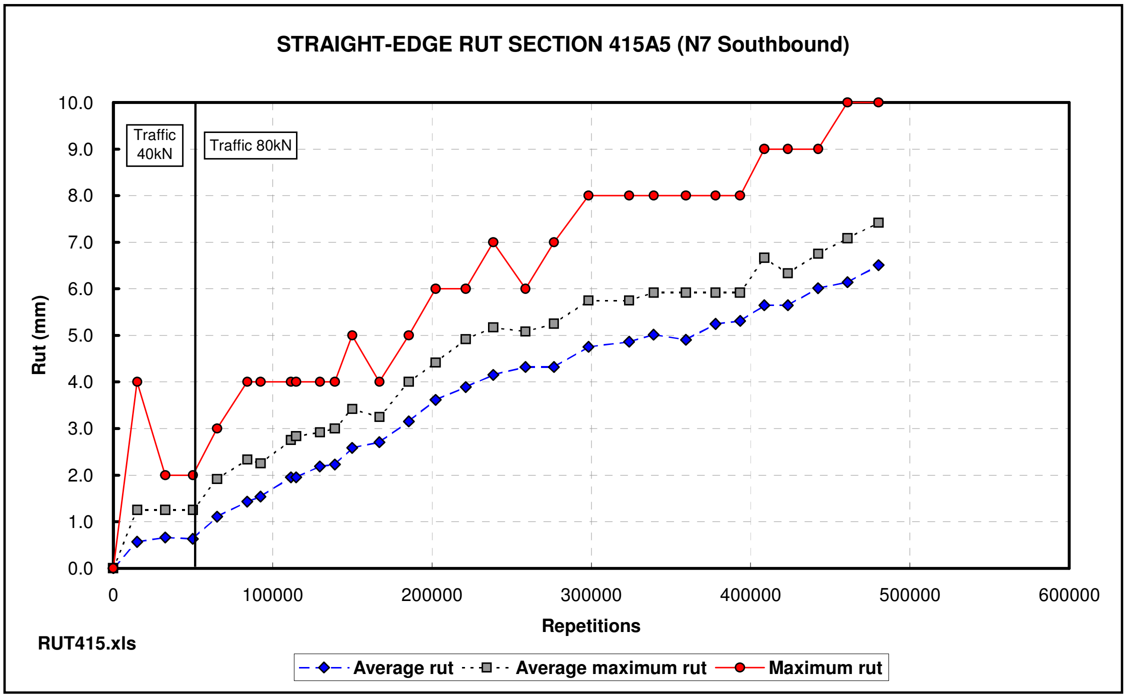
RAW PROFILE DATA MMLS3 TESTS

APPENDIX B:

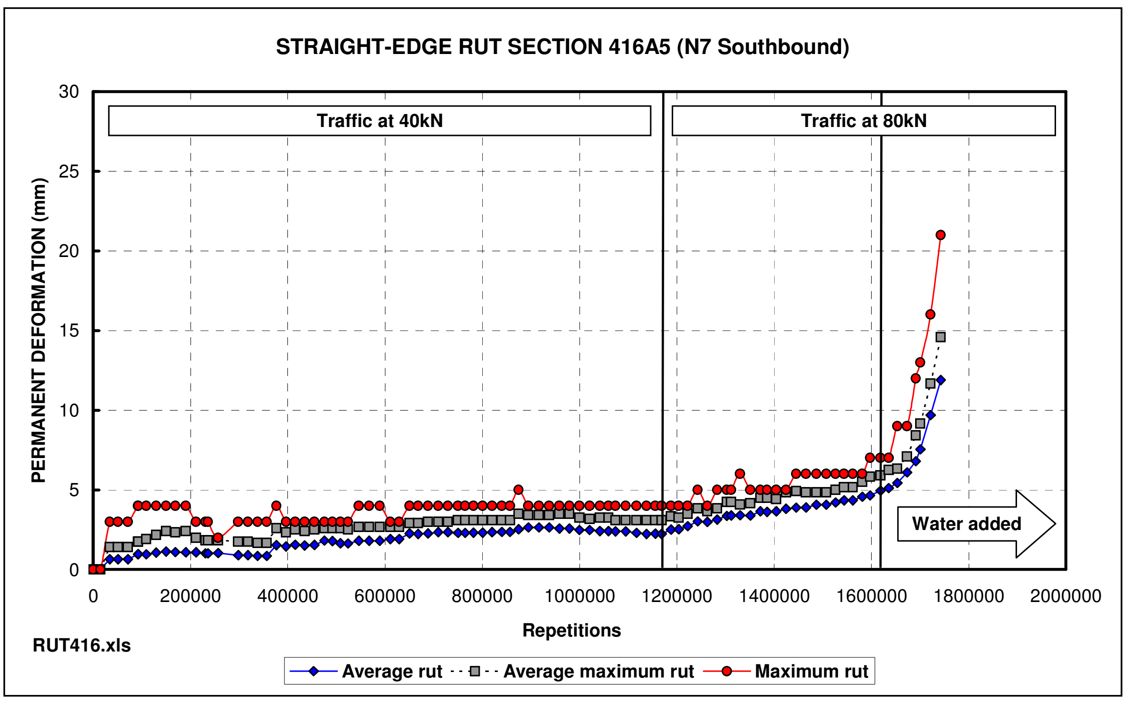
TEMPERATURE RECORDS MMLS3 TESTS

APPENDIX C:

STRAIGHT EDGE MEASUREMENTS HVS SECTION 415A5 AND 416A5



Summary results straight edge measurements section 4154A5 [CSIR]



Summary straight edge results section 416A5 [CSIR]

APPENDIX D:

PAVEMENT TEMPERATURES GRAPHS HVS SECTION 415A5 AND 416A5

APPENDIX E:

PIE-CHARTS PAVEMENT TEMPERATURE HVS SECTIONS 415A5 AND 416A5