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First Level Analysis Report: HVS Testing of the Concrete Inlays on the N3 near Hilton: Test 424A5

Version: Final

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DOCUMENT RETRIEVAL PAGE			**Report No: CR-2004/59**
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Abstract:

This report presents the validated data and analysis of HVS tests done on concrete inlays (slow lane, north bound) on the N3 near Pietermaritzburg in KwaZulu-Natal. The concrete inlays, built in 1998 have been used as a rehabilitation option to cater for slow moving and heavy traffic on the N3. Since 1998, 6.5 million axle loads have been carried by the continuously reinforced PCC pavement (CRCP) and, through visual inspections, it became evident that certain areas of the pavement had reached the end of their structural life. The objective of this series of HVS testing is to determine the remaining life of these in-service concrete inlays on the N3. This was achieved through HVS accelerated trafficking of a representative section of the highway. The pavement consists of 3 lanes, 2 of which are constructed with asphalt, the slow lane (outer lane), which consists of a 180mm thick CRCP concrete inlay, was subjected to accelerated HVS loading.

The following were investigated in this study:

- The deterioration of the concrete slab under the influence of accelerated loading;
- Stress sensitivity of the pavement;
- The deterioration of the concrete slab under the influence of water; and
- The influence of the environment on the behaviour of the CRCP section.

Proposals for implementation: None

Related documents (e.g. software, interim or other reports, working drawings etc):

This report is one of three reports related to the concrete research done in the last year by various researchers and companies. The other reports are:

Construction Report: HVS Testing of the Concrete Test Sections on the N3 near Hilton, Report No CR-2004/33, CSIR-Transportek, July 2004.

First Level Analysis Report: HVS Testing of the Concrete Test Sections on the N3 near Hilton, Tests 421A5 to 423A5, Report No CR-2004/43, CSIR-Transportek, September 2004

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DISCLAIMER

The views and opinions expressed in this document are those of the authors and do not necessarily represent the views or policy of CSIR, Transportek.

REVIEW STATEMENT

This is a draft report for review.

GENERIC CONTENTS OF THE FIRST AND SECOND LEVEL HVS ANALYSIS REPORTS

In accordance with the agreement between Gautrans and CSIR Transportek, the contents of the first, second and third level analysis reports for HVS testing are defined as follows:

The primary purpose of a first level HVS report is to present a complete and validated set of HVS data without detailed analysis or interpretation of the data. The scope of a first level report is confined to the HVS data and associated test results from a single HVS site. The conclusions of the first level report are therefore site-specific with little interpretation and should not be generalised. The primary aim of the second level analysis report is to interpret and explain the observed behaviour contained in the first level report. By combining the results from a number of test sites or combining the HVS and associated test results with data from other case studies on the same material and pavement type, it is possible to determine whether the observed response and behaviour are representative of the response and behaviour of the material and pavement type in general. However, if any pavement behaviour or design models are developed during the second level analysis, their scope is limited to the particular HVS site under investigation. The content of the third level analysis is similar to that of the second level analysis, but the data that were generated from HVS and associated testing on a number of sites are combined to develop general behaviour and design models for the material or pavement type under investigation.

1. First Level Report

The final first level report, on the basis of a single HVS test, or of a number of tests combined into a single phase of a larger project, should build on the data from the monthly technical meeting with the aim of:

- Providing recommendations on the resilient modulus values of the materials in the pavement structure being tested;
- Providing recommendations on the durability of the materials used in the construction of the test section;
- Identifying, if possible, the modes of distress for the particular pavement and materials being tested; and
- Identifying, if possible, the most critical mode of distress, i.e. the mode of failure.

In accordance with the agreement between Gautrans and CSIR Transportek, the content of the monthly technical presentations should be the same as as that of the first level HVS report. These monthly technical presentations should, together with additional interpretations made by the coordinating committee, form the basis for the first level report. The following guidelines were therefore set for the preparation of the monthly technical presentations:

- Visual condition information:
  - At least one composite photograph showing the cracking; and
  - Any additional visual information of interest from the surface of the test section or from the test pit.
- Surface instrumentation data:
  - Joint Deflection Measuring Device (JDMD) data, plotted for the duration of the test, with any change in test conditions being indicated on the graph.
- Depth instrumentation data:
  - Elastic depth deflections
    - plotted against the number of load repetitions for the duration of the test with any change in test conditions being indicated on the graph; and
    - plotted against depth in the pavement structure.
  - Permanent MDD displacements
    - plotted against the number of load repetitions for the duration of the test, with any change in test conditions being indicated on the graph;
    - plotted against depth in the pavement structure, with an indication of the relative contribution of the individual pavement layers to the total permanent deformation.
- Additional material, environmental and operational information:
  - Nuclear gauge density and moisture content profiles as per the test instrumentation plan;
  - Temperature data as per test instrumentation plan (thermocouples at the top, mid-depth and bottom of the concrete slab);
  - Tyre temperature and pressure data; and
  - Visual, density, moisture content and DCP data for the test section and test pit after completion of an HVS test. This will only be presented at the technical meeting after completion of a specific HVS test.

Additional work that may be required to address the aims of the first level report may include:

- Regression modelling of the deflection and relative movement data.

2. Second Level Report

The aims of the second level report are:

- To interpret and explain the response and behaviour contained in the first level report;

- To develop site-specific response and behaviour models for the material or pavement type being investigated, based on the data from one site. These models are calibrated in terms of the deflections and relative movements measured during HVS testing, whereas the first level report only quantifies the deflections and relative movements for the particular wheel loads at which the HVS tests were done;
- To evaluate how representative the results from the specific site are by comparing the HVS and associated test results with results from similar studies done in other locations.

The data from different phases of HVS testing, done at different types of joints/cracks and on different types of concrete on the same site, are combined to achieve the goals of the second level report.

The second level report should attempt to:

- Provide site-specific models for the materials being investigated;
- Provide site-specific pavement response and structural design models (transfer functions) for the modes of distress of the materials being investigated. These models should accommodate the variability of the data that were used to develop the models and should be calibrated for the relevant crack widths and environmental conditions;
- Provide guidelines on sound engineering design and construction principles for the materials being investigated; and
- Make conclusions and give guidelines regarding the durability of the materials being investigated.

The second level report should make use of all available data to be able to reach these objectives, including:

- The HVS and associated (JDMD, MDD, strata gauge, temperature, etc.) field-test data;
- Standard and advanced laboratory test data associated with the HVS site; and
- Results from other full-scale or laboratory case studies.

3. Third Level Report

The objectives and content of the third level report are the same as those of the second level report, but with the additional aim of developing generalised response and design models. The HVS and associated test data from a number of HVS tests done at different locations and using different materials of the same material type are combined for the third level analysis.

REFERENCE TO THE HVS DATABASE

The electronic data collected from HVS tests are stored in the HVS database. The contact person for access to the HVS database is:

Name:	Mr Colin Fisher	
Contact details:	Phone No:	(012) 841-3960
	Fax No:	(012) 841-2690
	E-mail:	cfisher@csir.co.za

The database consists of text files with Multi-Depth Deflectometer (MDD), Joint Deflection Measuring Device (JDMD), Permanent Deformation (PD) Temperature and Tyre Pressure data.

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1 INTRODUCTION

Based on the use of cncPave and the monitoring of the performance of existing concrete roads and trial sections, it became apparent that verification was needed to refine and calibrate the performance models used in the following four major categories, vie slab support, materials, load transfer and variation introduced through construction methods.

In order to address the load transfer issue specifically, two HVS experimental Portland cement concrete (PCC) sections were constructed at the Hilton off ramp, approximately mid-way between Pietermaritzburg and Howick on the N3. The test sections were constructed between the off ramp and the southbound carriageway of the N3. The details of the HVS testing at the Hilton site are contained in two reports (1,2).

Another objective of this series of HVS testing is to determine the remaining life of an in-service concrete inlay on the N3. Concrete inlays were used as a rehabilitation option to cater for slow moving and heavy traffic on the N3 in the vicinity of Pietermaritzburg in the northbound direction (uphill). The pavement consists of 3 lanes, 2 of which are constructed with asphalt, the slow lane (outer lane) consisting of a 180mm thick continuously reinforced concrete inlay.

This report details the first level analysis on the performance of the concrete inlays under the influence of HVS accelerated trafficking.

2 SITE SELECTION AND EXISTING CONDITIONS

After several visual inspections along various inlay sections on the N3, an area at kilometre post 16.3 (northbound) was selected for HVS testing (see Pictures 2.1 and 2.2). The following important factors played a roll in the selection of an appropriate site:

- The pavement structure of the selected testing area should be identical to the as built data of the whole inlay pavement. As certain deviations might occur, it was important that the concrete thickness and subbase conditions of the section selected should be the same as that believed to be typical of all the concrete inlay sections. This was achieved through the drilling of cores in various areas.
- The selected site should be safe. It was essential that the sight distance be adequate to warn oncoming motorists, and that road width be sufficient, to

accommodate the HVS so as to ensure that no accidents occurred during testing.

- The gradient of the road. As these inlays were constructed on the northbound carriageway of the N3, most of them are in areas with steep gradients. In order for the HVS to function properly it was important that it be placed on a section where the gradient and crossfall are as small as possible.
- The condition of the selected testing area should represent the general state of bad sections of all the concrete inlays on the N3. It was important that the cracking patterns, both in terms of degree and extent, in the selected HVS testing area should be similar to the cracking pattern in some of the bad areas of the concrete inlays. In order to determine the remaining life of the concrete inlay sections it is important that the results from the HVS testing should be relevant to the existing condition of the concrete inlays.

Picture 2.1: Site selected for HVS testing at km 16.3 N3 north bound

Picture 2.2: Forward view of HVS test site, N3 concrete inlays

The concrete layer consists of 180 mm of continuously reinforced concrete (CRCP). In the longitudinal direction Y16 steel rods were placed at 180 mm intervals. Approximately 22 rods were used to cover the total lane width of 4200 mm. The characteristics of the substructure are detailed in Table 2.1.

Table 2.1: Engineering characteristics of the substructure

Total depth mm	Layer depth mm	CBR		GM	PI	in-situ moisture content (%)	Description
		95% density	DCP derived				
150	150	-	-				Asphalt
400	250	66	N/A	2.64	SP	5.7	Stabilized dolerite crushed rock
680	430	49	50	2.44	10	10.1	Weathered shale
880	450	21	33	0.57	9	10.7	Weathered shale

The base layer consisted of a cement-stabilised crushed dolerite on top of 2 subbase layers, a reddish-brown dense weathered shale and finally a weaker dark-brown weathered shale. The data displayed in Table 2.1 are based on results of tests carried out on material recovered from the test pit nearest to the HVS test section at km 16.03. The test pit was opened in September 1998 and the in-situ moisture content of the layers at that time was as indicated in Table 2.1. CBR values were determined from laboratory samples compacted to 95% MOD AASHTO density and were also derived from DCP testing

The test section finally selected at km 16.3 had a significant amount of transverse cracking prior to HVS testing. Five transverse cracks were found in the 8m long test area. (See Picture 2.3). As indicated above, this particular test section was selected on basis that the crack patterns should be representative of those in typical concrete inlays.

A schematic representation of the crack patterns is given in Figure 2.1.

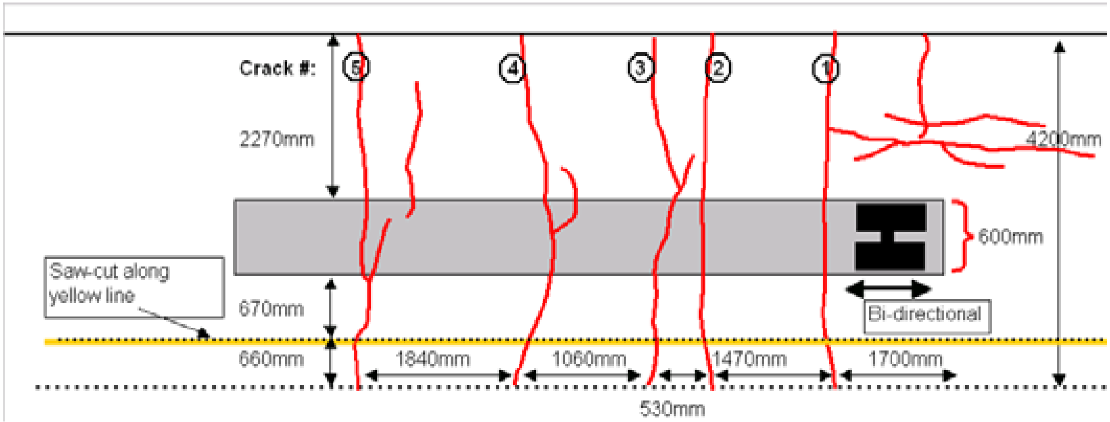


Figure 2.1: Crack patterns of 424A5 prior to HVS testing

Picture 2.3: Selected area for HVS testing on the N3 concrete inlays.

The concrete slab at the particular spot where HVS testing took place is 4,2m wide and the yellow line is painted 660mm from the outer longitudinal edge of the reinforced concrete slab. During visual surveys it was noted that most of the bad sections on the N3 inlays occurred in the inner wheel path in areas where the road curved to the right. It was therefore important to place the HVS in such a way that the same area (inner wheel path) would be tested. However, owing to the size of the HVS and the width of the concrete section, it was not possible to position a test pad close to the adjacent traffic asphalt lane and it was decided to test the area of the outer wheel path. Because the selected section at km 16.3 was constructed with a 4,2m wide concrete slab, which is not typical of the N3 inlays it was decided to saw-cut the longitudinal edge of the test section to a depth of 200mm. Testing in this manner (as seen in Figure 2.1) was, therefore, similar to the real traffic scenario.

During the HVS testing the performance and behaviour of the 5 transverse cracks indicated in Figure 2.1 were investigated.

3 GENERAL ASPECTS AND HVS TEST LOADING PLAN

To get the maximum production from the HVS in a limited amount of time, it was decided that bi-directional loading would be applied to the section. Because the inlays are constructed of continuously reinforced concrete it was felt that deterioration of the joint normally found in plain aggregate interlock and doweled sections under the influence of uni-directional traffic would not play a significant role and that bi-directional trafficking would yield the same result.

The complete test plan is shown in Table 3.1

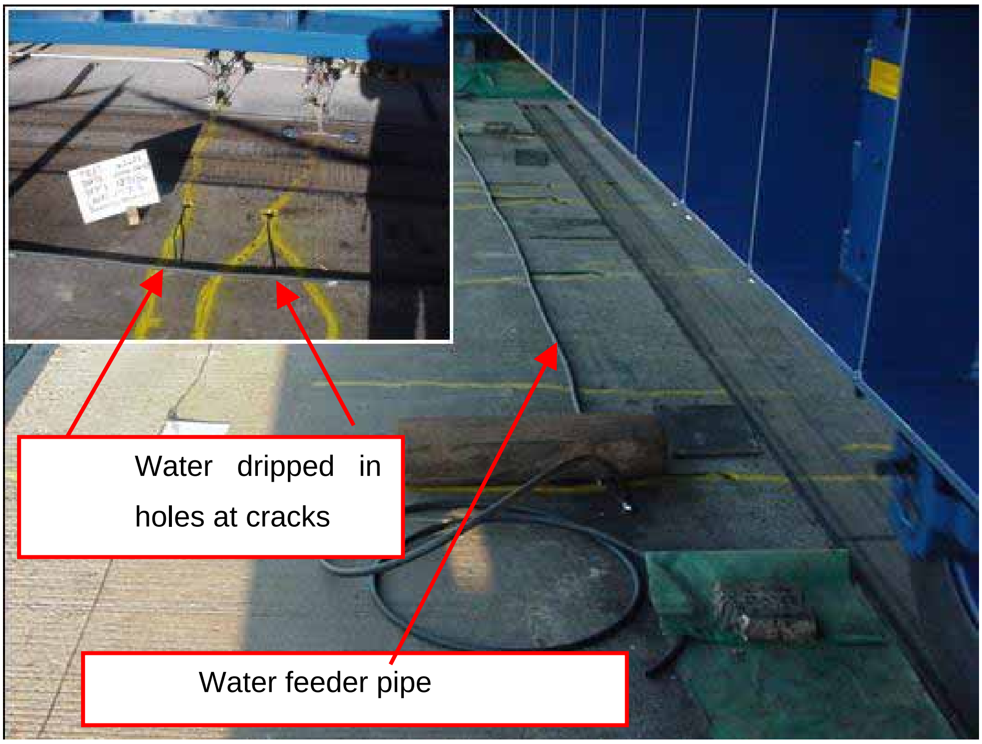
Table 3.1: Test plan of 424A5

Trafficking mode:	Canalized	
Temperature control:	Ambient	
Loading direction:	Bi-directional	
Trafficking load:	60kN	0 - 344 114 reps
	80kN	344 114 - 562 555 reps
Tyre Pressure:	720 kPa	@ 60kN
	800 kPa	@ 80kN
Instrumentation measurement interval:	every 2 hours	
Test Load	60 or 80 kN	
	40kN once a day	
Crack width measurements	twice daily: 8:00 and 14:00	
Strata gauge moisture measurements:	once per week	

Although the trafficking load was set at 60kN, it was decided that surface deflections would also be measured under a standard half axle load of 40kN. This was done once a day early in the morning after the daily service. Collecting data in this fashion makes the analysis less complex as all deflection analysis techniques are based on a 40kN half axle. Deflection data in this report is therefore presented under the influence of both 60 and 40kN loads. This should not be confused with the trafficking load. Although a 40kN was used to measure deflections, the test pad was subjected to 60kN loading 24 hours a day, 7 days a week.

Due to time constraints it was decided that if no dramatic change in pavement performance was observed after a week of testing (approximately 140 000 repetitions) water would be added to the cracks to speed up the deterioration process. Only four weeks of HVS testing was allowed for and it was felt that the addition of water might assist in deriving a useful result. Picture 3.1 shows the watering system. Water was introduced at the edge of the trafficking area through a dripping system at each crack. Water was added from the 4 June onwards after 131 873 load repetitions.

Because of the strong behaviour of the slab it was decided to accelerate the damage caused by the loading by increasing the load from 60 to 80kN after 344 114 repetitions. This was done on 15 June. The wheel load was then kept constant at 80 kN for the remainder of the test, from 344 114 to 562 555 repetitions.



Picture 3.1: Watering system used during testing

3.1 HVS LOADING

Figures 3.1 and 3.2 show the loading distribution for the 60 and 80kN load cases.

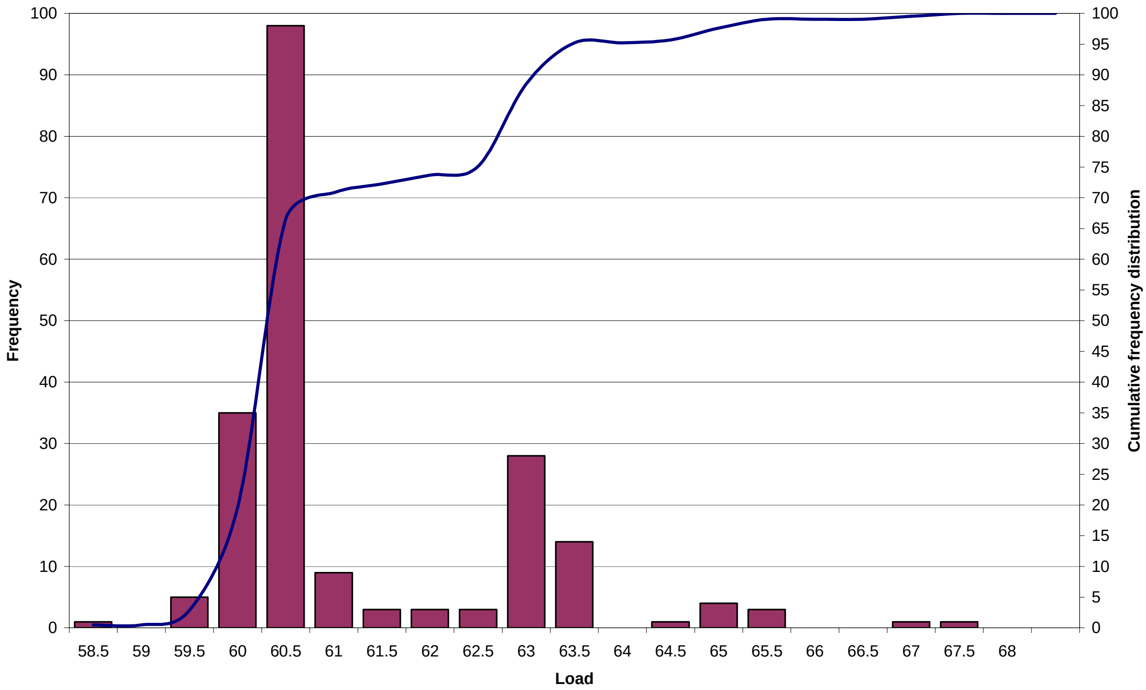


Figure 3.1: 60kN loading histogram

The 50th percentile value is 60.5 kN which is 0.5kN higher than the specified 60kN. The distribution is skewed to the right, indicating that for a greater percentage of time the load exceeded the required 60kN, rather than being lower.

The loads applied during the 80kN phase were closer to the target load (see Figure 3.2). The total load variation was between 79 and 81.5 kN, the 50th percentile value being 80kN.

In general the loading was very accurately applied and was within 5% of the target load.

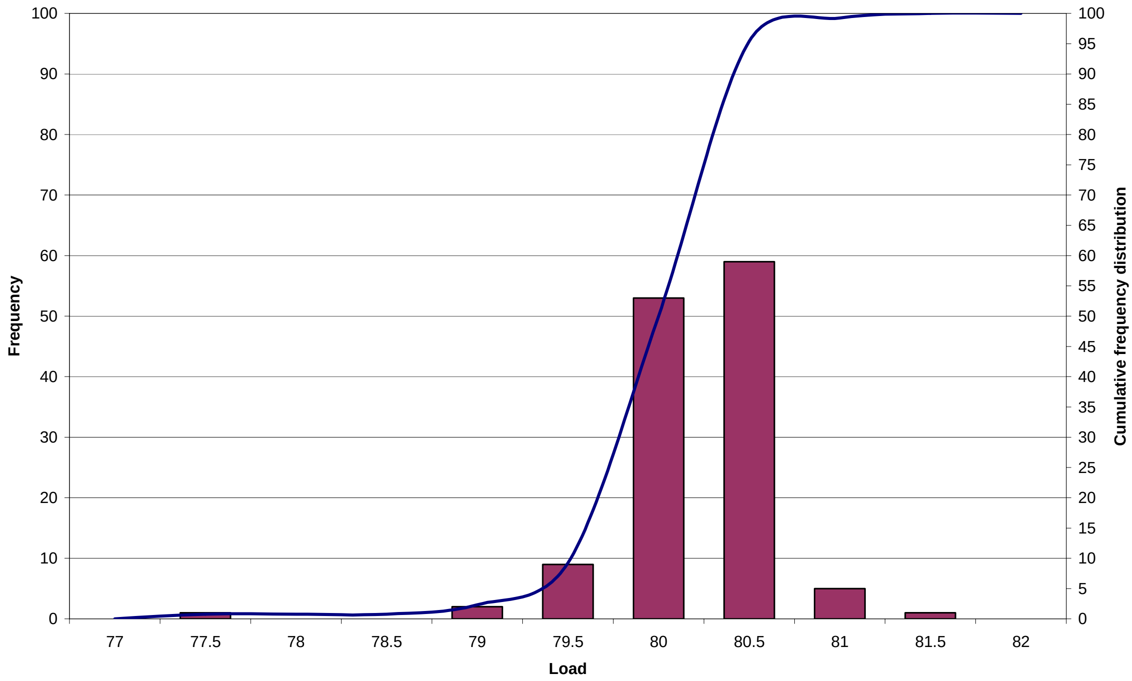


Figure 3.2: 80kN loading histogram

4 INSTRUMENTATION

4.1 INSTRUMENTATION USED FOR THE HVS TESTS

In order to monitor the functional and structural behaviour of the pavement under accelerated traffic, various instruments and sensors were used. The description and function of these instruments are briefly described below.

<u>Multi-Depth Deflectometer (MDD)</u>

MDD modules are linear variable displacement transducers (LVDTs) placed inside the pavement at various depths. In the case of testing of the PCC inlays, two 2 modules were placed on top of each other, one inside the subbase and one on the surface of the concrete layer. MDD modules record the elastic movement (deflection) under the influence of the rolling HVS wheel at the depth where the module is installed, as well as the plastic deformation that takes place with time. These plastic movements may be load related (plastic deformation or densification), or due to environmental effects.

MDDs were used to record the following:

- Surface deflections
- Relative movement across cracks

  (This is done through comparing the deflection bowls of 2 surface mounted modules, placed on either side of a crack.)
- Slab curl due to daily temperature variations
- The formation of voids under the slab at cracks.

  (This is done through comparing the deflection of a surface module with that of a module placed in the subbase immediately below the bottom of the concrete slab.)

The MDDs were placed right under the trafficking wheel at the following positions:

- MDD 1 on the left side of Crack # 1
- MDDs 2 & 3: both sides of Crack # 3
- MDD 4: on the approach side of Crack # 5

MDD modules were placed inside the concrete at a depth of 20mm and in the subbase at a depth of 210mm. An illustration of the placement of the various MDDs can be seen in Figure 4.1.

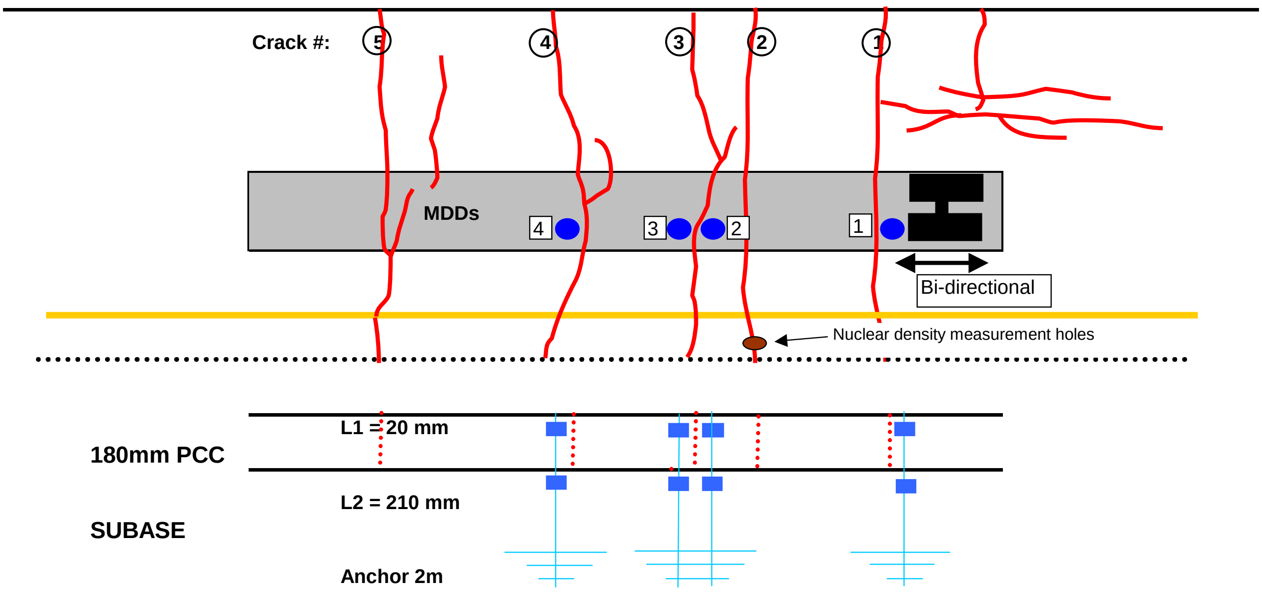


Figure 4.1: Placement of MDDs at test 424A5

Joint Deflection Measuring Devices (JDMS)

JDMDs are linear variable displacement transducers (LVDTs) mounted vertically on the concrete slab. Like MDDs they record surface movements under the influence of Traffic (deflections) and the environment (temperature-related curling effects). The main function of the JDMDs is to measure the deterioration of load transfer across cracks with accelerated trafficking by studying the relative movement of two adjacent JDMDs placed on either side of a crack. The JDMDs can also capture vertical up / down movements due to day / night temperature differential variations between the top (surface) and the bottom of the concrete layer.

In order to measure the degree of slab contraction and expansion, mainly due to daily temperature fluctuations, JDMDs were also mounted horizontally across certain cracks.

JDMDs were installed right next to the HVS testing pad at the following positions:

- JDMD 1 & 2: On both sides of Crack # 1
- JDMD 3 & 4 and Horizontal 1: on either side of, and across, Crack # 2
- JDMD 5 & 6 and Horizontal 2: on either side of, and across, Crack # 3
- JDMD 7 & 8 and Horizontal 3: on either side of, and across, Crack # 4
- JDMD 9 & 10: on both sides of Crack # 5.

The positions of all the JDMDs are shown in Figure 4.2

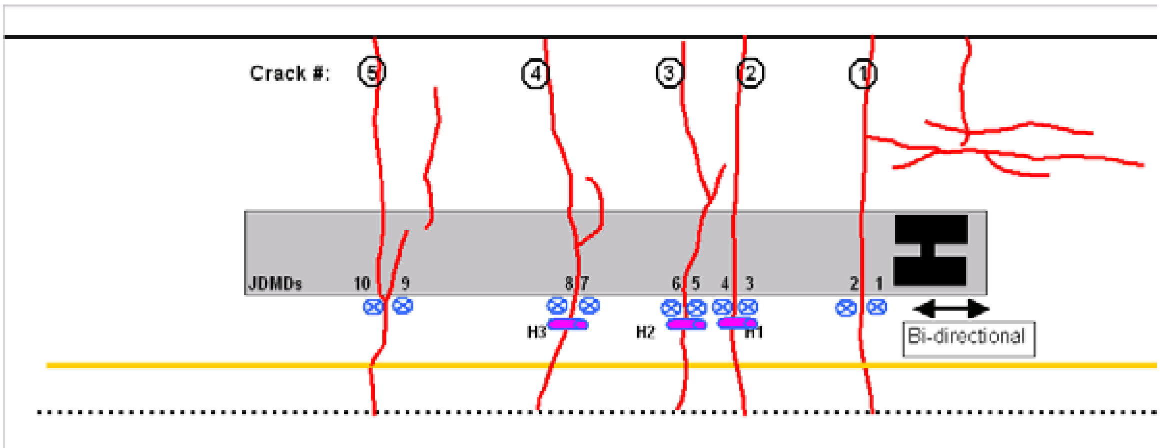


Figure 4.2: Placement of JDMDs at test 424A5

Figures 4.1 and 4.2 show the placement of JDMDs, one MDD as well as the horizontal JDMD that records the horizontal crack movements.

<u>Thermocouples</u>

Thermocouples were installed to measure the temperature of the air, as well as of the surface and at various depths inside the concrete. Three thermocouples were installed for this test, two in close proximity to JDMDs 3,4 and 5,6 (T3 and T2 respectively) and one in the sun outside the influence area of the HVS and of its shade (T1).

Thermocouples were placed at the surface (0mm) at 90mm and at the bottom (180mm) of the PCC concrete layer. The placement of the various thermocouples is illustrated in Figure 4.3

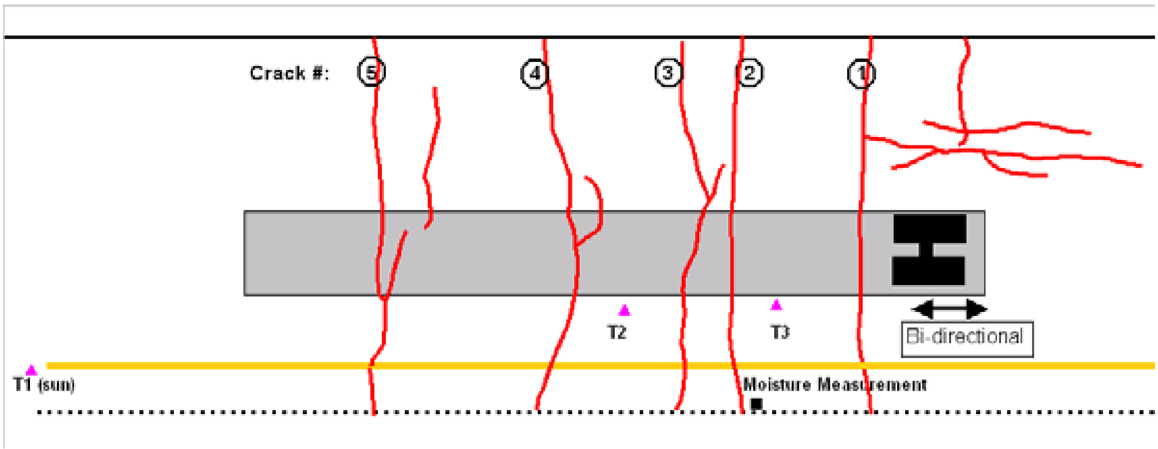
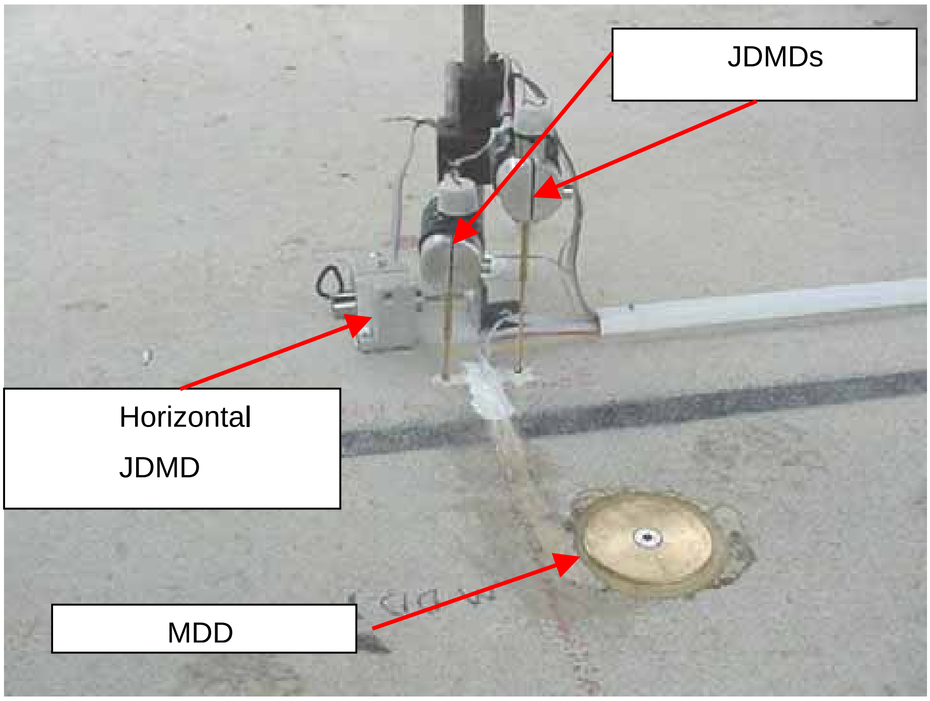


Figure 4.3: Placement of thermocouples at test 424A5

Picture 4.1 shows the placement of JDMDs, one MDD as well as the horizontally placed JDMD that recorded the horizontal crack movements



Picture 4.1: JDMDs, MDD and horizontal JDMD used during testing

Weather data

There was also a weather station was at the HVS site. The following data were collected at 2-hour intervals:

- Air temperature
- Rainfall (collected daily)
- Wind speed and wind direction
- Barometric pressure
- Relative Humidity.

Moisture measurements

A nuclear density probe was used to measure the moisture content of the layers below the concrete to a depth of 600mm. One hole was drilled to a depth of 750mm on the shoulder side of the HVS test pad. The placement of the holes with respect to the various HVS testing areas can be seen in Figure 4.1

5 HVS TEST RESULTS: 424A5 N3 PCC INLAYS TESTING RESULTS

This chapter is divided into 3 main sections. The first section deals with all the environmental data collected at the site during HVS testing. These include data such as rainfall, temperature, humidity etc. The second section details visual observations recorded during testing. These include any visual change that were detected during testing, including crack growth, degree and extent of pumping, joint faulting etc. The last section describes the data that were collected using electronic sensors, such as MDDs and JDMDs.

5.1 ENVIRONMENTAL DATA

5.1.1 Rainfall

Testing started on 28 May 2004 and finished on 28 June 2004. Rain only fell on 2 occasions during this testing period, 17.5mm of rain was recorded on Tuesday 8 June after 217 900 repetitions and 3mm on Monday 28 June after 562 555 repetitions. The 2-hourly rainfall distribution within a 24-hour cycle can be seen in Table 5.1. A total of 20.5mm of rain was recorded during the whole testing period.

Table 5.1: Rainfall data during HVS testing

Date	Hour of day	2-hourly rain (mm)	Daily cum total (mm)	Repetitions	Relative humidity (%)
08-Jun-04	0	0	0	200 150	46
	2	0	0	201 875	50
	4	0	0	203 608	64
	6	0	0	205 219	71
	8	1.5	1.5	206 683	81
	10	0	1.5	207 258	82
	12	1	2.5	209 063	74
	14	0	2.5	210 736	47
	16	0	2.5	212 512	77
	18	5	7.5	214 324	90
	20	10	17.5	216 116	92
	22	0	17.5	217 927	95
09-Jun-04	24	0	17.5	219 716	96
28-Jun-04	0	0	0	556 257	56
	2	0	0	557 741	57
	4	0	0	559 376	58
	6	0	0	560 799	90
	8	1	1	562 555	94
	10	2	3	testing stopped	
	12	0	3		
Total rain during testing period (28/05 - 28/06)				20.5 mm	

5.1.2 Humidity

The relative humidity data as recorded by the weather station at the HVS testing site can be seen in Figure 5.1. Data were collected 2-hour intervals. The graph shows the maximum relative humidity during any given 24-hour cycle within the testing period

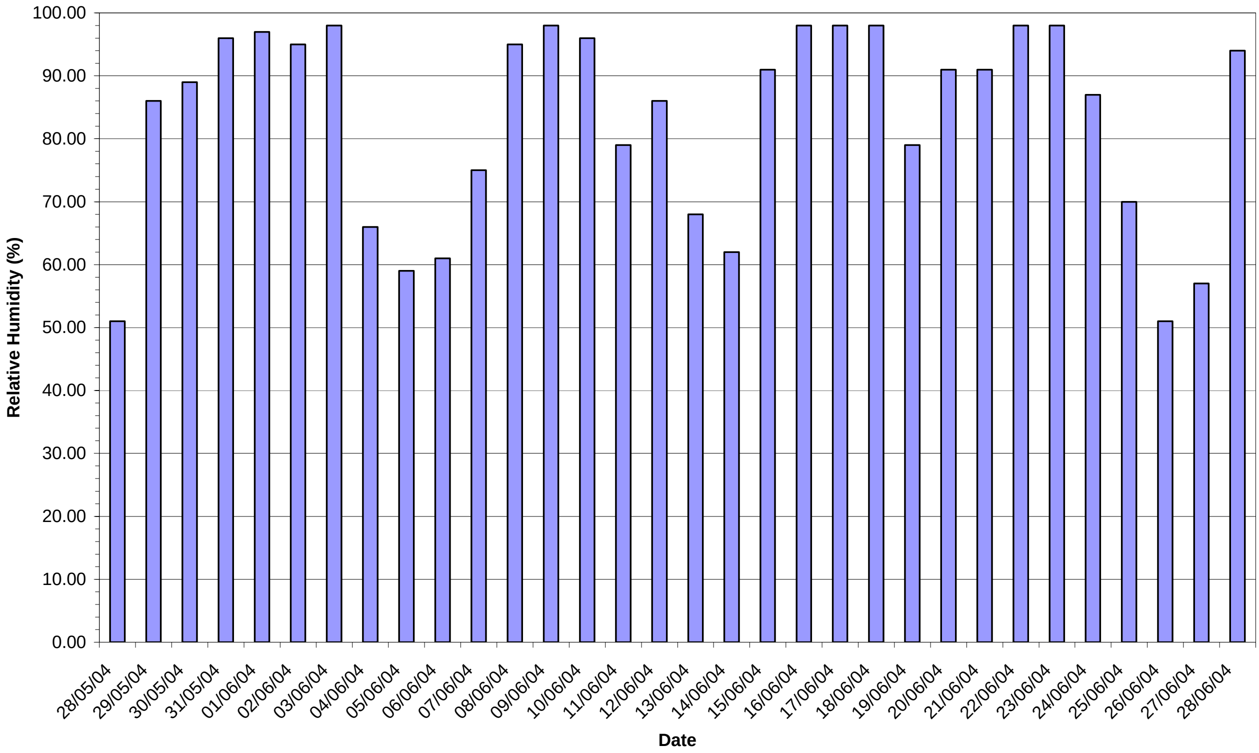


Figure 5.1: Relative humidity during HVS testing

The 2-hourly relative humidity for the days on which rain fell can be seen in Table 5.1.

5.1.3 Wind Speed

The wind speed as recorded by the weather station can be seen in Figure 5.2 for the duration of HVS testing. Like Figure 5.1, the data shown in Figure 5.2 represent the maximum wind speeds recorded within a 24-hour daily cycle.

If Figures 5.1 and 5.2 are compared it can be seen that periods with lower relative humidity correspond to periods of high wind speeds.

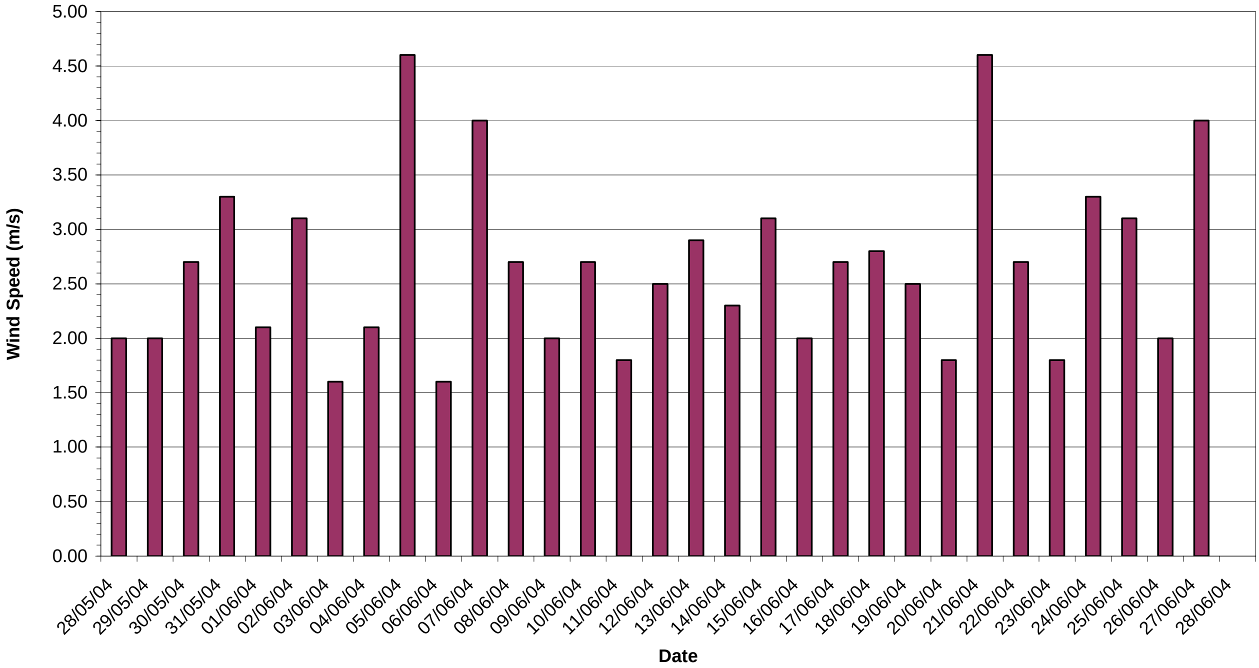


Figure 5.2: Wind speeds recorded at the HVS site

5.1.4 Temperature data

Thermocouples were installed as detailed in Section 4.1 (see Figure 4.3). An additional thermocouple was connected to the weather station, which recorded the ambient outside air temperature. The data are shown in Figures 5.3 to 5.5 for thermocouples T1 to T3,. The air temperatures collected from the weather station are also shown on each of the graphs. Each thermocouple recorded 3 temperatures: at the surface (0mm), at a depth of 90mm and at the bottom of the PCC layer (at a depth of 180mm).

It should be remembered that thermocouple T1 was placed in the sun, outside the influence area of the HVS and its shade, whereas thermocouples T2 and T3 were placed under the HVS in close proximity to the JMDS (see Figure 4.3).

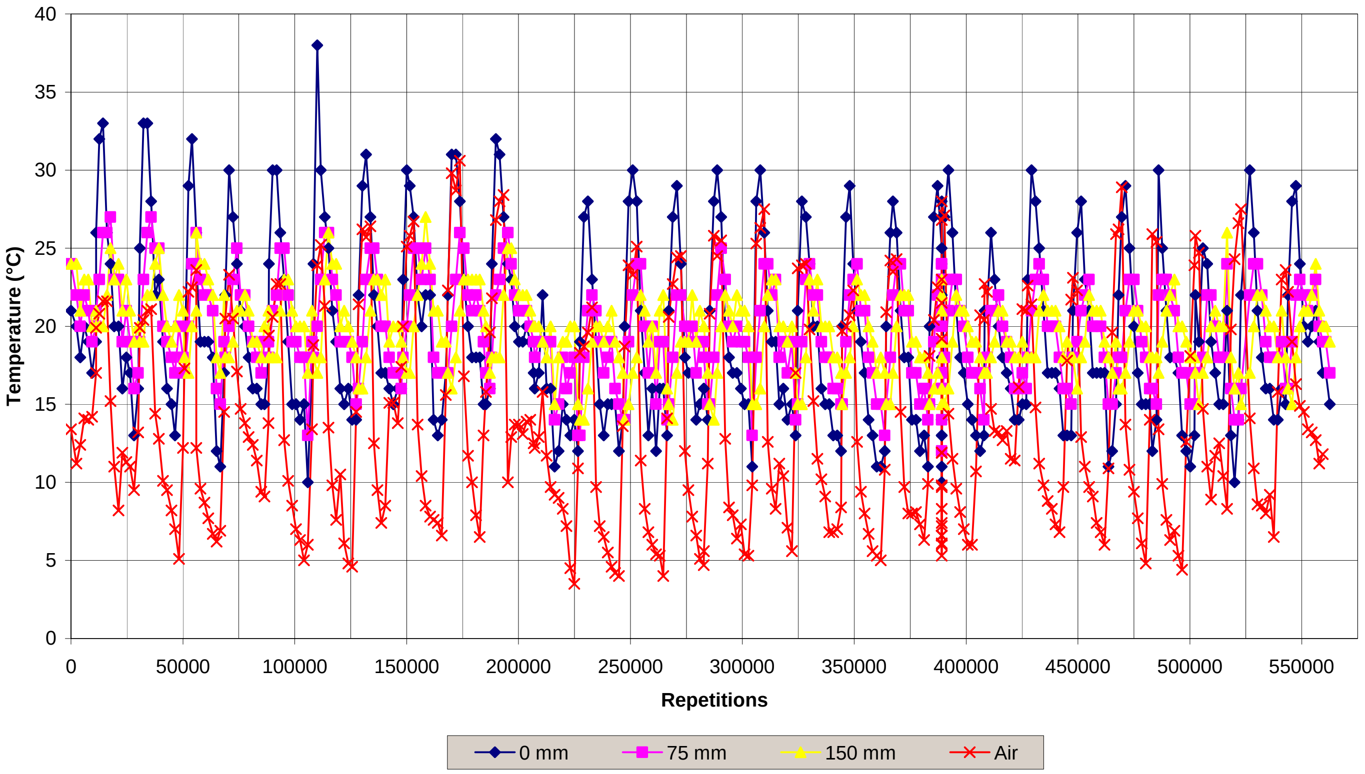


Figure 5.3: Temperatures recorded with thermocouple T1

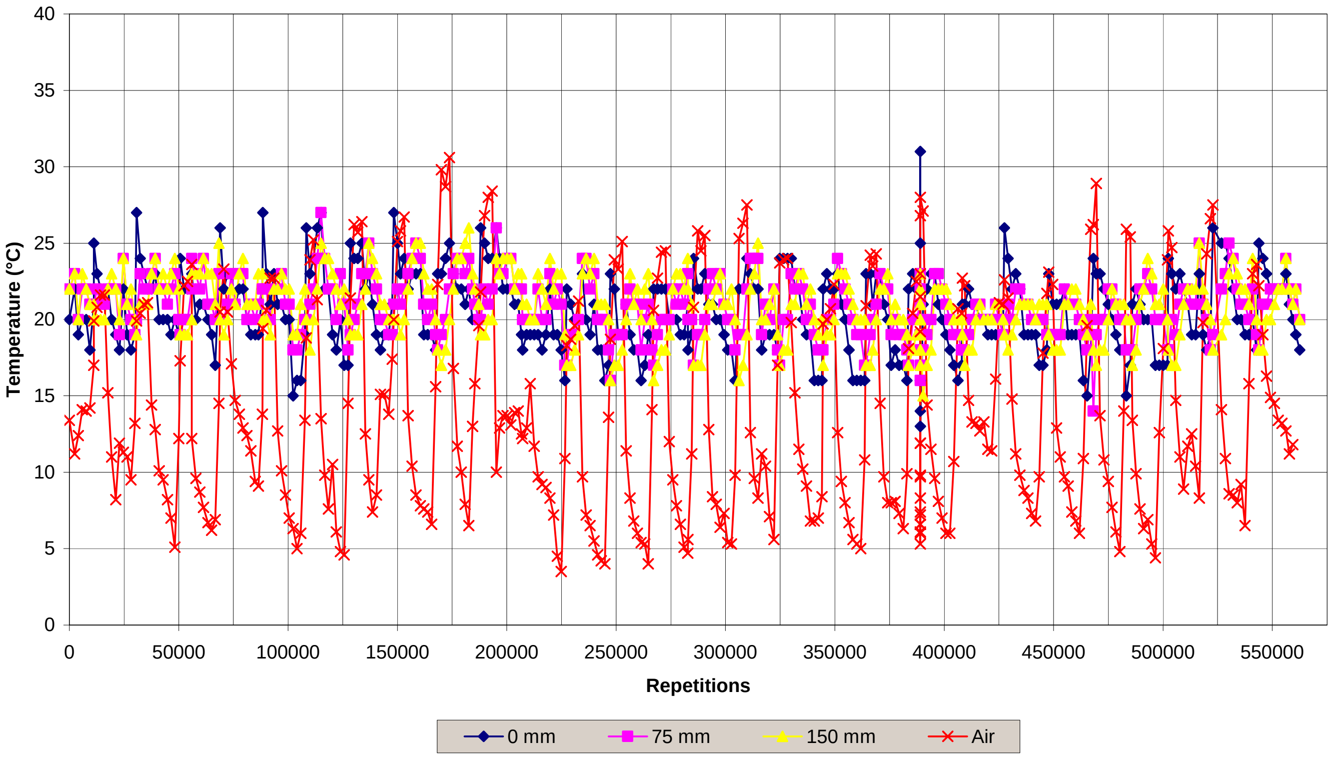


Figure 5.4: Temperatures recorded with thermocouple T2

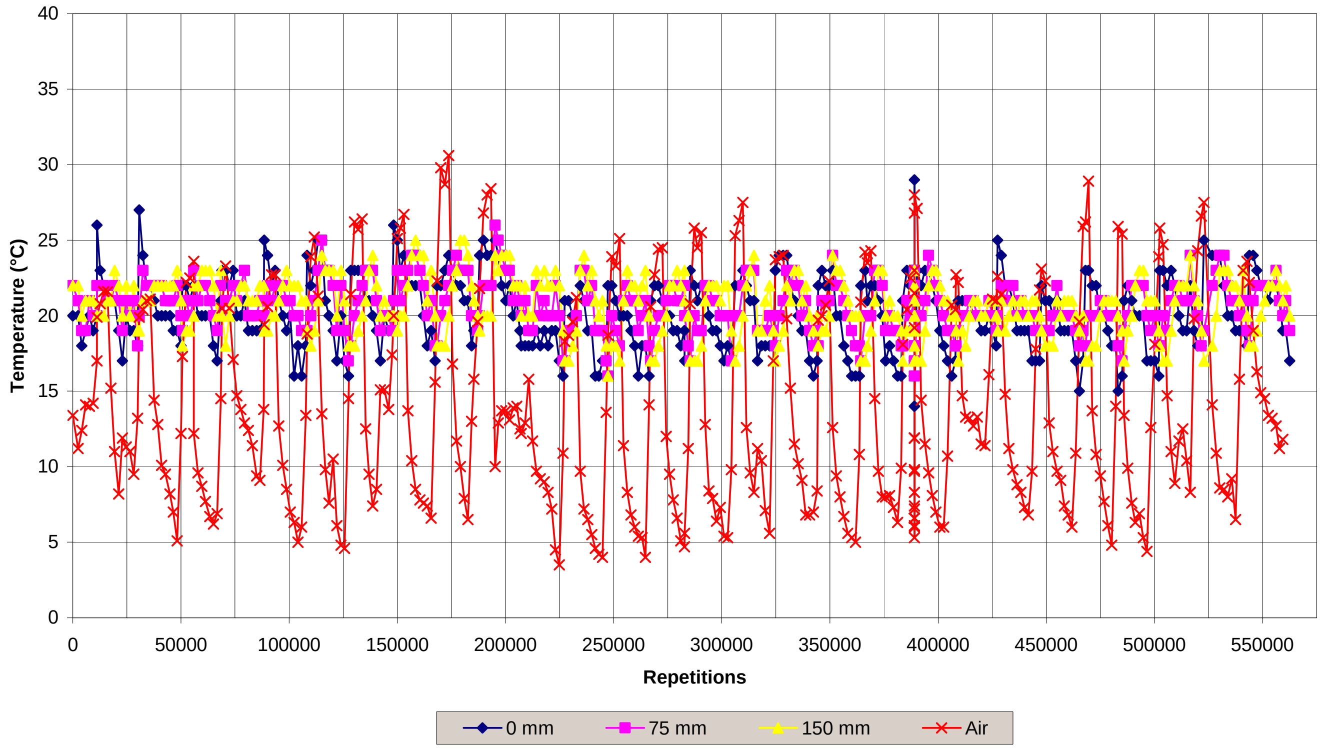


Figure 5.5: Temperatures recorded with thermocouple T3

To investigate the effects which temperature differentials (temperature at the surface – temperature at the bottom) have on slab curling effects, it is required to determine the temperature differentials and then to analyse it together with the plastic response of the slabs. In order to do this the three periods when the greatest temperature differentials were observed were identified. Figure 5.6 shows the data for the periods when thermocouple T1 recorded the highest temperature differentials within a 24-hour cycle for the complete testing period 28 May – 28 June 2004.

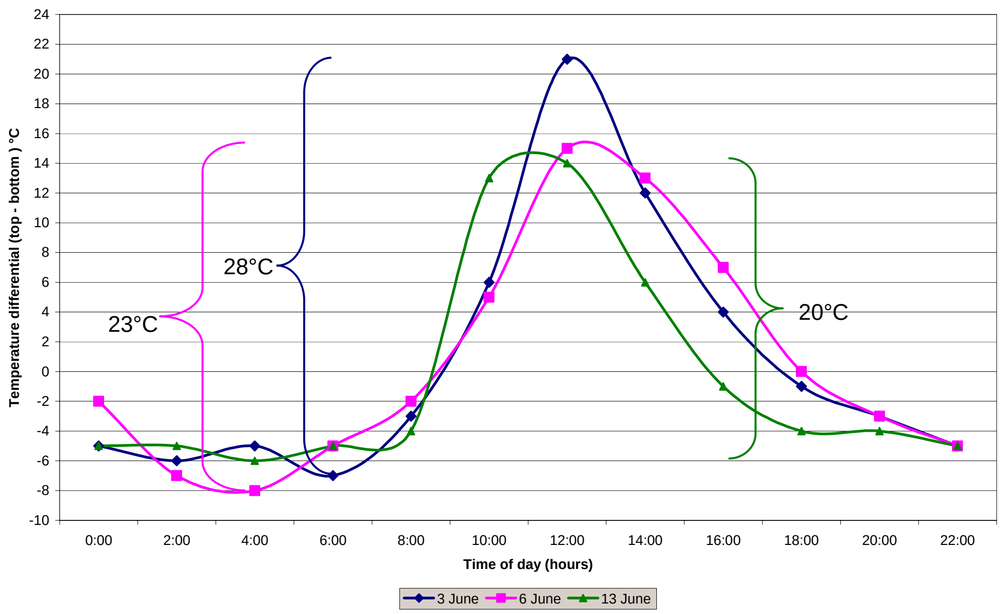


Figure 5.6: Most significant temperature differentials in a 24-hour cycle

The highest temperature differential between the top and the bottom of the PCC layer was observed on 3 June at 12:00. The difference was 21°C. The corresponding negative temperature differential was –7°C, which occurred at 6:00 in the morning. This means that the total temperature difference span during those 24 hours was 28°C as indicated in Figure 5.6.

5.1.4 Moisture measurements

Moisture measurements of the substructure were measured using a nuclear density probe. Measurements were done from a depth of just below the PCC slab (50mm) to a depth of 600mm. Holes were drilled to a depth of 750mm on shoulder side of the HVS test pad. The placement of the holes with respect to the various HVS testing areas can be seen in Figure 4.1. Data was collected on a weekly basis.

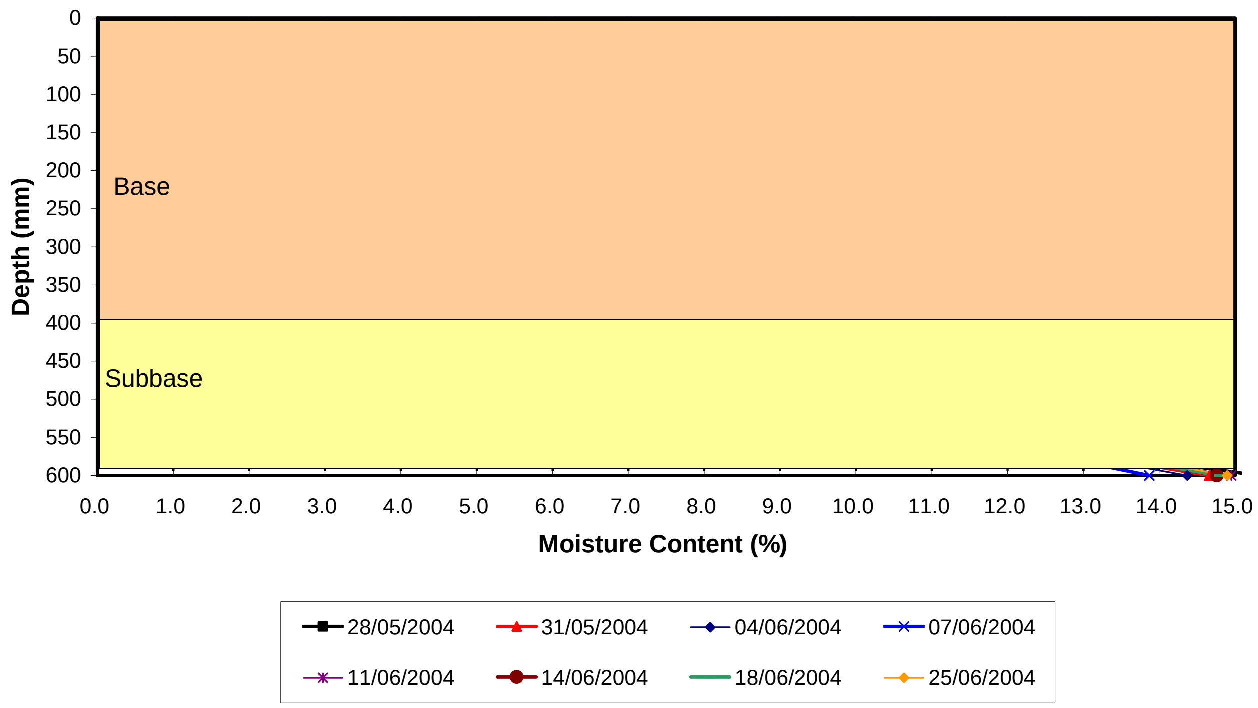


Figure 5.7: Nuclear gauge density moisture content data at the HVS testing site

In Figure 5.7 it can be seen that there was no significant variation in the moisture contents of the substructure during the testing period (28 May – 28 June). The moisture contents of the base layer varied between 3 and 6 per cent for the first 150mm, and increased to between 7 and 9.5 per cent, thereafter, peaking at a depth of 250mm. The top 100mm of the subbase had a moisture content of between 8 and 9 per cent, which increased with depth to 15 per cent. The "S" shape of the moisture profile is consistent with other HVS tests done in the vicinity of Hilton. (1,2) . As no material sampling was done at the site to substantiate these moisture contents, the moisture contents measured by the nuclear gauge should be regarded as only an indication of the moisture profile.

The introduction of water to the cracks (4 June 2004) had a slight influence on the recorded moisture contents (compare the data set recorded on 31 May with the data sets after 4 June). It seems that the moisture contents at a depth of between 200 and 400mm increased by approximately 1 per cent after water had been introduced to the cracks. No significant increase could be seen in the moisture contents in the deeper part of the structure as a result of the influx of water from the top.

5.2 VISUAL OBSERVATIONS

According to design drawings of the inlays at the HVS testing site, the pavement structure should have been constructed with 180mm PCC on top of 150mm of asphalt placed on top of the granular layers (see Table 2.1). The left hand shoulder of the road consisted of a concrete shoulder with a catch water drain. This structure is schematically represented in Figure 5.8

Figure 5.8: Expected pavement structure at km 16.3

While the holes were being cored at the HVS site it was discovered that there was no asphalt under the areas where the MDD holes were drilled. One possible explanation for not finding any asphalt under the concrete inlay might be that, because of width restrictions, the contractor could not fit 3 lanes together with the concrete shoulder and catch water drain in the available space and the PCC inlay was constructed partially inside the shoulder as indicated in Figure 5.9

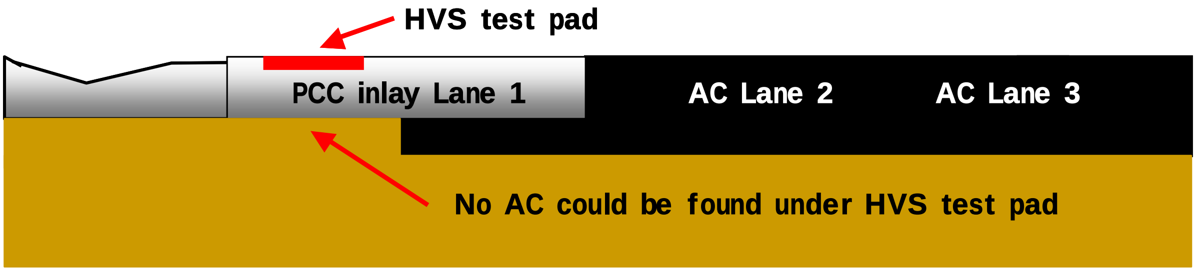


Figure 5.9: Possible pavement structure at km 16.3

Because of the traffic demand, the area in which the HVS testing was conducted was reopened to traffic just after the completion of the tests and no test pits were opened up to reveal the actual pavement structure underneath the HVS testing site.

The crack pattern prior to the start of HVS testing can be seen in Figure 2.1 and Picture 2.3. The 180mm inlay consisted of continuously steel-reinforced concrete (CRCP). Thus, even though a significant number of cracks were visible on the pavement surface, the steel rods kept the concrete together, keeping the pavement structurally sound.

5.2.1 Factors monitored during HVS testing

The following concrete pavement behaviour characteristics were monitored with the use of several instruments:

a) Slab curling and contraction / expansion

Slab corner curling is caused by temperature differentials between the top and the bottom of the slab. Concrete contracts as the temperature decreases and expands as the temperature rises. If the top and bottom of the concrete slab are not at the same temperature, the resulting temperature differential causes the slab to curl, with the corners moving either upwards or downwards.

During daytime, the upper surface area (exposed to direct sunlight) heats up and causes the upper surface to expand. The bottom surface (180 mm below the top surface) does not reach the same temperature, as it is not exposed to sunlight. The net effect of this is that the slab corners tend to curl downwards, because the bottom of the slab expands less than the top.

During the night the effect is reversed. The surface, being exposed to the air, cools more rapidly than the underside. This causes the top part of the slab to contract to a greater extent than the bottom part (which is not exposed to the ambient air temperature). This causes the slab corners to curl upwards.

This effect is illustrated in Figure 5.10 for edge and corner curling.

The JDMDs placed vertically all along the edge of the tested slabs (see Sec 4.1) will assist in the characterisation of this particular movement.

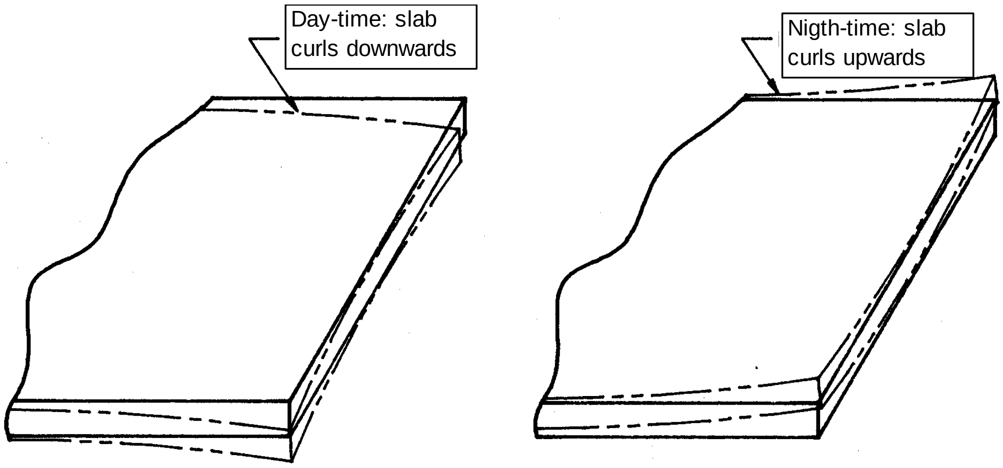
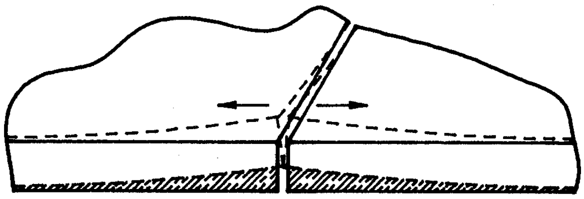


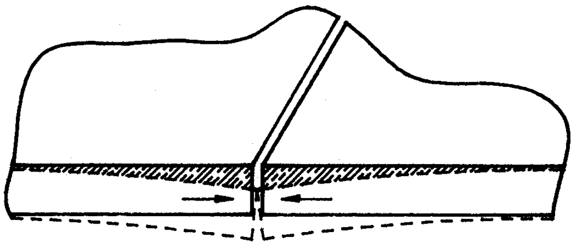
Figure 5.10: Illustration of slab curl due to temperature differentials

Because of slab expansion / contraction caused by the rise and fall of the ambient temperature, crack widths fluctuate. During the night, the slabs cool down and the slabs contract, causing the cracks to widen. During the day, the opposite occurs: the slab heats up and expands and the crack widths decrease. The effect of this was measured by JDMDs placed horizontally (H1, H2 and H3) as detailed in Sec 4.1. This effect is illustrated in Figure 5.11.

The maximum curling along the edge and at the cracks and slab contraction / expansion are obviously influenced by the total area exposed to direct sunlight. It is obvious that the shade (due to the HVS itself) and the effect of the reinforced steel would limit the movements recorded in the slabs but, by using the instrumentation, attempts were made to record these effects.



Night-time: slabs cool off and crack widths increase



Day-time: slabs heat up and crack widths decrease

Figure 5.11: Crack width changes with daily temperature variations

b) Elastic Slab movements

This effect is due to the applied load. Various instruments (JDMDs and MDDs) were used to measure the elastic deflection bowls caused by the applied wheel load.

c) Crack deterioration

Crack widths were recorded twice daily, early in the morning and at 14:00. Crack lengths and growth were measured on a daily basis.

d) General slab deterioration

All aspects of visual deterioration were monitored continuously. Observation and recording of spalling, pumping, edge breaks, additional cracking, etc formed part of the daily visual survey during testing.

5.2.2 Visual observations during testing.

Testing started on 28 June according to the test plan as detailed in Section 3.0, with a 60kN load and a tyre pressure of 720 kPa. After 1 week of testing and 131 873 repetitions no obvious deterioration of any kind could be detected and it was decided to add water to the cracks. The amounts of water added to all the cracks were measured at 2-hourly intervals and are presented in Figure 5.12. An average of approximately 47 litre of water was added every 2 hours or 564 litres per day. It seemed as if the water drained very rapidly under the pavement as only a small degree of pumping was observed throughout the testing period. The 2-hourly water consumption and the total amount of water added the sections can be seen in Figure 5.12.

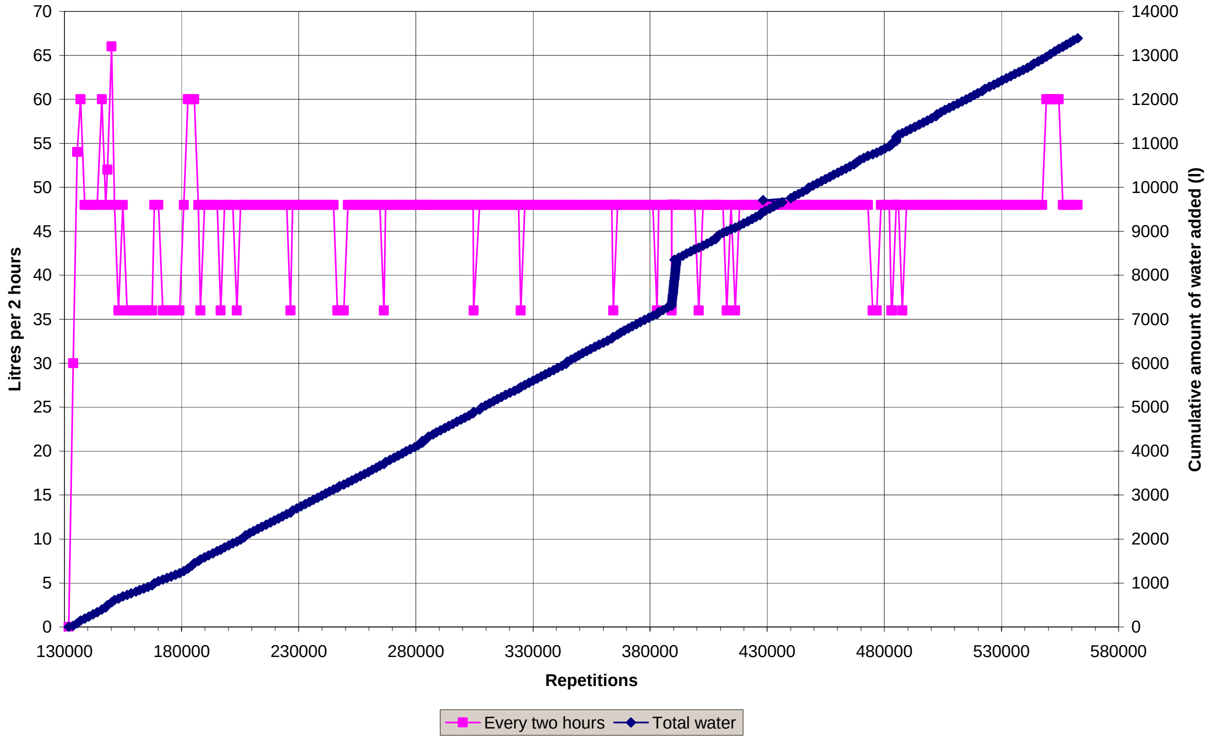


Figure 5.12: Total amount of water added to the cracks

A total of 1340 litres were added to the section during the 25 day watering period (4 – 28 June).

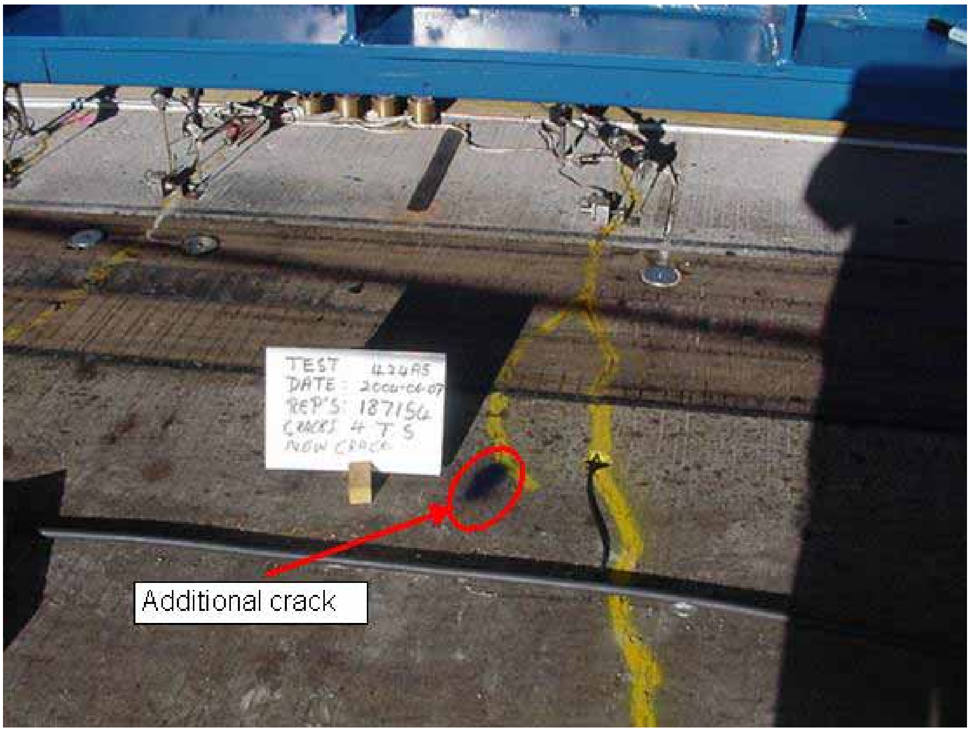
A minor degree of pumping of fines was observed at all cracks after the addition of 444 litres of water and 15 700 repetitions. At that time a total of 147 591 repetitions had been applied to the section. (See Picture 5.1).

Picture 5.1: Minor degree of pumping after water has been added

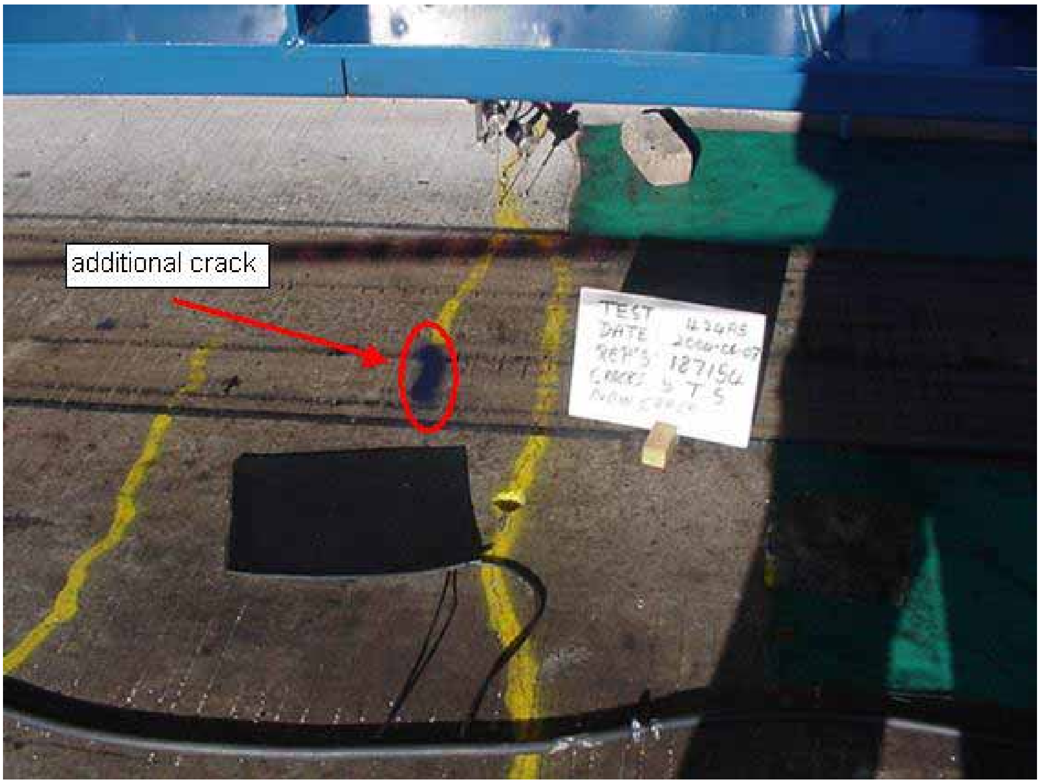
Clean water (no fines) also came out of the MDD holes (Picture 5.2)

Picture 5.2: Water seeping out of the MDD holes

Two small additional cracks were observed at cracks 4 and 5 on 7 June after 187 154 repetitions. These new cracks were however very small and their widths could not be measured (see Pictures 5.3 and 5.4).



Picture 5.3: Additional hairline crack at crack # 4

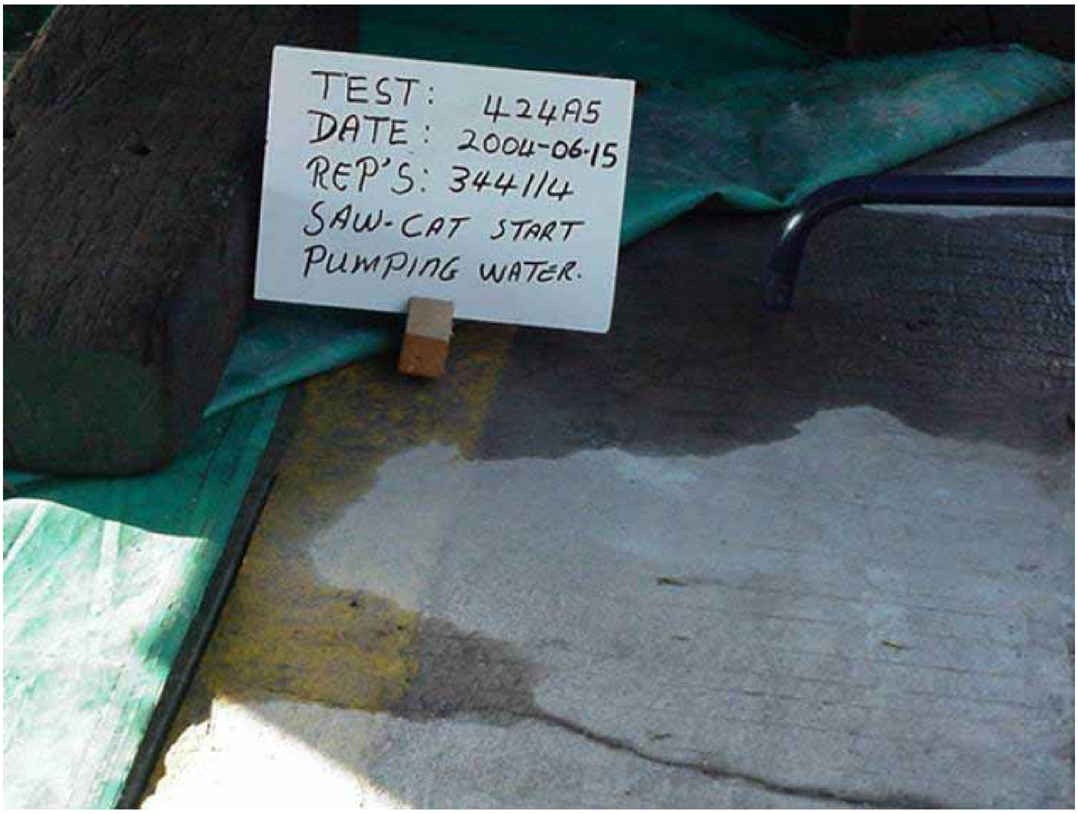


Picture 5.4: Additional hairline crack at Crack 5

On 8 June (after 206 683 repetitions) 1,5mm of rain fell, which caused some additional pumping, but no significant changes were observed. See Picture 5.5.

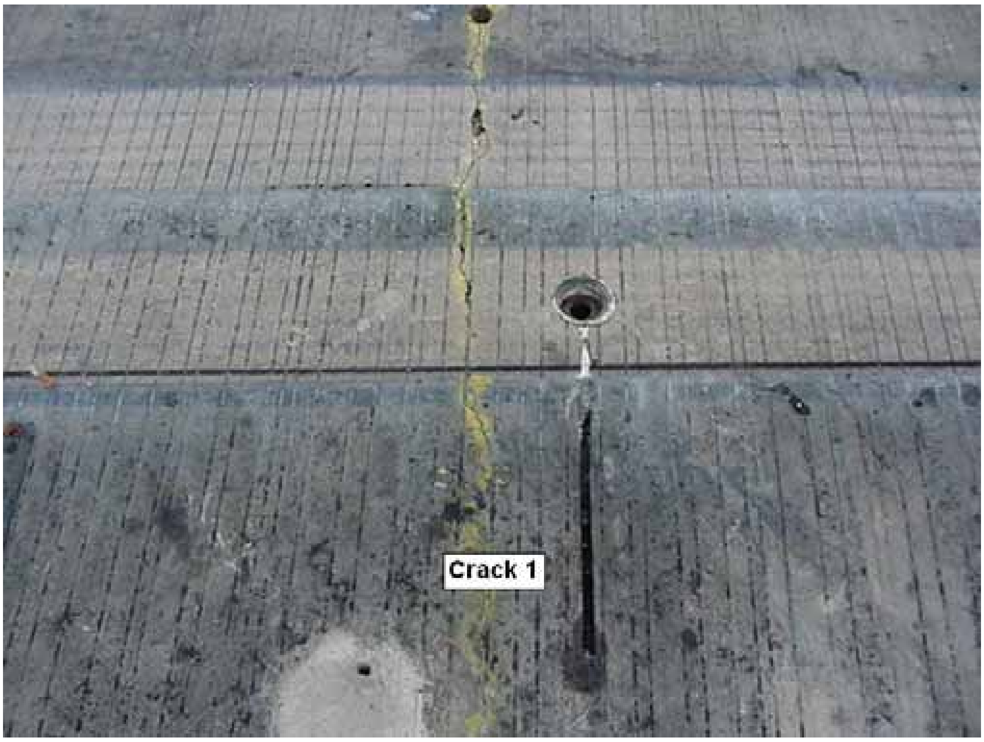
Picture 5.5: Pumping of fines after 206 683 repetitions

The addition of water, including the rain did not have any visible effect on the rate of deterioration and after 344 114 repetitions it was decided to increase the trafficking load from 60 to 80 kN. At this time water also started to come out of the longitudinal saw-cut joints along the yellow line (Picture 5.6).

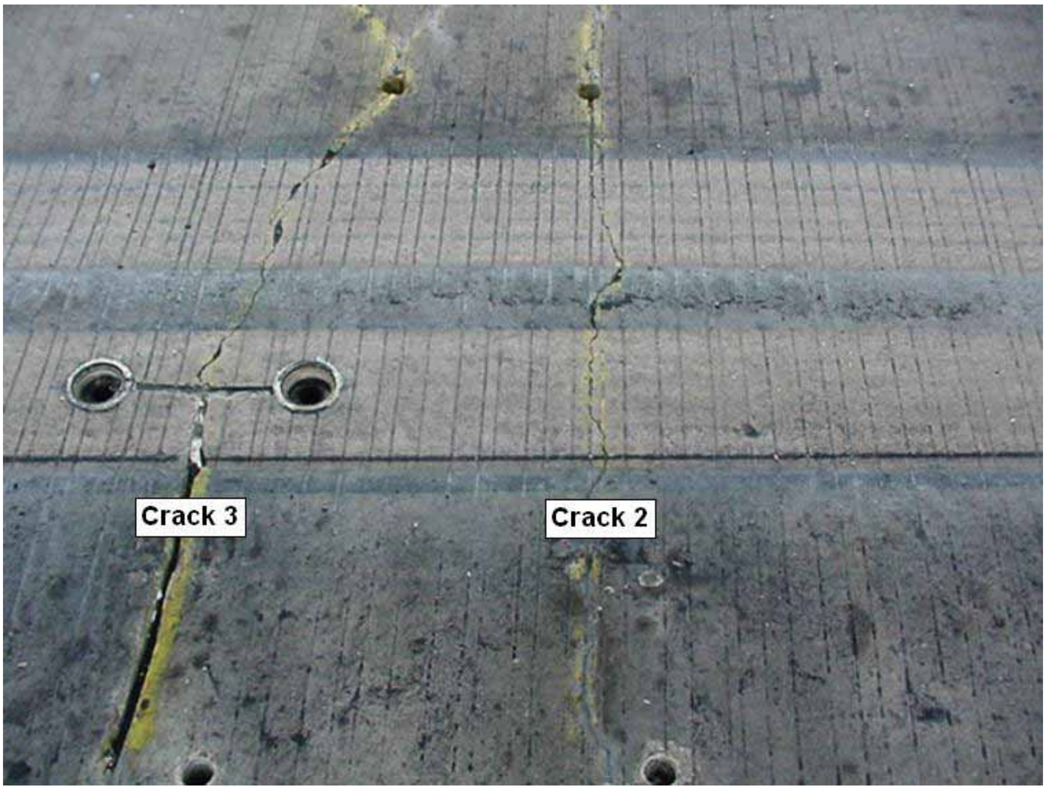


Picture 5.6: Clean water pumping out of longitudinal saw-cut edge after 344 114 repetitions

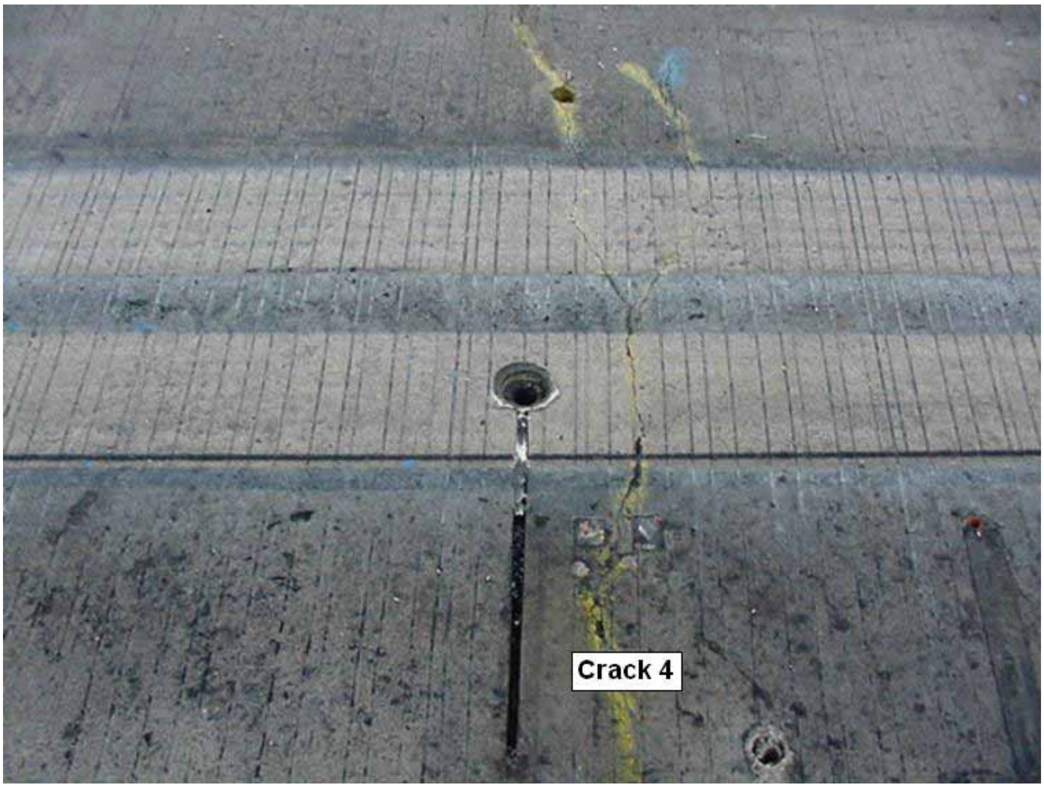
The load was increased to 80 kN on 15 June and was kept at that level for a further 218 441 repetitions, until testing was finally stopped on 28 June 2004. A total of 562 555 load repetitions had then been applied to the HVS section. The final state of the testing surface area can be seen in Pictures 5.7 to 5.10.



Picture 5.7: Final condition of Crack 1



Picture 5.8: Final condition of Cracks 2 and 3



Picture 5.9: Final condition of Crack 4

Picture 5.10: Final condition of Crack 5

The surface of the tested area did not show any significant degree of deterioration after the HVS testing. Even after water had been applied to all cracks at a rate of approximately 23.5 litres per hour and under the aggressive loading of 80kN, no visible degree of surface deterioration could be detected. It is obvious that the reinforcing played a major role in keeping the pavement in a structurally sound state.

5.3 CRACK-WIDTH MEASUREMENTS

Crack widths were measured twice daily at 8:00 and 14:00 for the duration of the test. Each crack was measured at exactly the same spot inside the trafficked area. No changes were, however, detected. It is possible that the measuring instrument (feeler gauge) did not have the required resolution to pick up the differences in crack widths resulting from daily temperature fluctuations, and the reinforced steel bars restricted the ability of the concrete to move. The measured widths of each crack remained constant throughout the test and no variation due to temperature or wheel repetitions could be detected.

The crack widths measured were:

Crack 1:	0.5mm
Crack 2:	0.4mm
Crack 3:	0.4mm
Crack 4:	0.4mm
Crack 5:	0.5mm

From the above it is clear that the loading had no effect on the cracks and no deterioration could be detected. Even after 344 114 60kN and 218 441 80kN load applications the crack widths at the end of the test were exactly the same as at the beginning.

5.4 SLAB CURL DUE TO ENVIRONMENTAL INFLUENCES

The slab movements due to daily temperature variations and the temperature differentials between the top and the bottom of the concrete slab were monitored using MDDs and JDMDs. This was done to characterise the slab behaviour due to environmental influences as detailed in Section 5.2.1 (a). In order to achieve this, the data were isolated for the periods during which the greatest temperature differences between the top and the bottom of the PCC slab were observed during the testing period. As indicated in Figure 5.7, three periods were investigated:

1) On 3 June when the total temperature differential span for a 24 hour period was 28°C (from -7 to 21°C) and total surface temperature change of 28°C;
2) On the 6 June when the total temperature differential span for a 24 hour period was 23°C (from -8 to 15°C) and a total surface temperature change of 18°C; and
3) On the 13 June when the total temperature differential span for a 24 hour period was 20°C (from -6 to 14°C) and a total surface temperature change of 19°C.

A fourth period was also investigated, a 40-hour cycle at the end of the testing period 26 – 27 June 2004. This was done in order to investigate the influence which the trafficking wheel might have had on the ability of the slab to curl up- or downwards.

The data for all the JDMDs placed along the edge of the section (JDMD1 to 10) can be seen in Figure 5.13. The data are plotted in 2-hour intervals, starting at 18:00 and ending 40 hours later, at 10:00 on the next day but one.

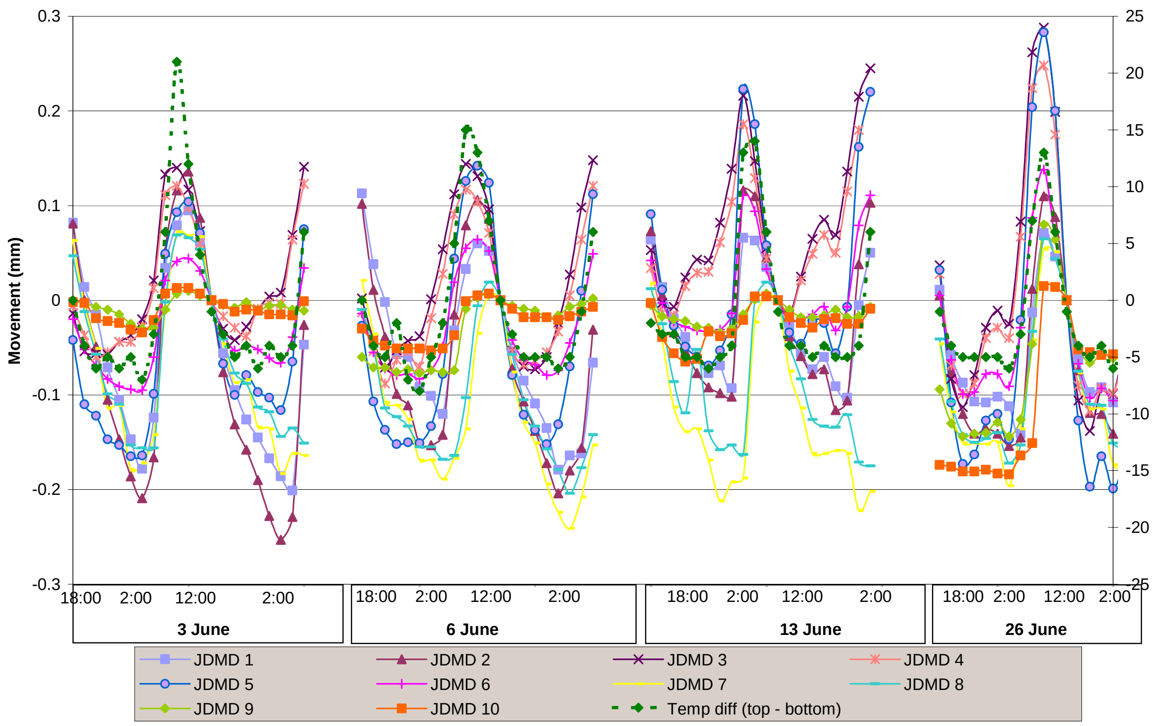


Figure 5.13 Measurements of slab curl during HVS testing

The minimum and maximum movements during the 40-hour time window are summarised in Table 5.2.

Table 5.2 Minimum and maximum daily movements during four selected periods

	03-Jun			06-Jun			13-Jun		
Instrument	**Max Up Movement** (mm)	**Max Down Movement** (mm)	**Max Total Movement** (mm)	**Max Up Movement** (mm)	**Max Down Movement** (mm)	**Max Total Movement** (mm)	**Max Up Movement** (mm)	**Max Down Movement** (mm)	**Max Total Movement** (mm)
JDMD 1	0.10	-0.20	0.30	0.11	-0.18	0.29	0.07	-0.10	0.17
JDMD 2	0.14	-0.25	0.39	0.11	-0.20	0.31	0.12	-0.12	0.23
JDMD 3	0.14	-0.06	0.20	0.15	-0.07	0.22	0.25	-0.02	0.26
JDMD 4	0.12	-0.06	0.18	0.12	-0.09	0.21	0.19	-0.02	0.20
JDMD 5	0.10	-0.17	0.27	0.14	-0.15	0.29	0.22	-0.07	0.29
JDMD 6	0.04	-0.10	0.14	0.06	-0.08	0.15	0.11	-0.03	0.14
JDMD 7	0.07	-0.18	0.26	0.02	-0.24	0.26	0.02	-0.22	0.24
JDMD 8	0.07	-0.16	0.23	0.02	-0.20	0.22	0.02	-0.18	0.19
JDMD 9	0.01	-0.03	0.04	0.02	-0.08	0.10	0.01	-0.03	0.04
JDMD 10	0.01	-0.03	0.05	0.01	-0.05	0.06	0.00	-0.06	0.07
All JDMDs: Ave Max total movement			0.20			0.21			0.18
Maximum surface temperature			38		31			30	
Minimum surface temperature			10		13			11	
Maximum temperature differential			21		15			14	
Minimum temperature differential			-7		-8			-6	

At end of test 25 June 2004			
Instrument	**Max Up Movement** (mm)	**Max Down Movement** (mm)	**Max Total Movement** (mm)
JDMD 1	0.07	-0.16	0.23
JDMD 2	0.11	-0.18	0.29
JDMD 3	0.29	-0.14	0.43
JDMD 4	0.25	-0.12	0.37
JDMD 5	0.28	-0.20	0.48
JDMD 6	0.14	-0.11	0.25
JDMD 7	0.05	-0.20	0.25
JDMD 8	0.07	-0.18	0.24
JDMD 9	0.08	-0.14	0.22
JDMD 10	0.02	-0.18	0.20
All JDMDs: Ave Max total movement			0.30
Maximum surface temperature			25
Minimum surface temperature			10
Maximum temperature differential			13
Minimum temperature differential			-6

As can be seen in the table and graph, during the hottest day (3 June) the surface temperature was 38°C which caused a total temperature differential span between the top and the bottom of the slab to be 28°C (21°C to -7°C). This caused the edge of the slab to go through an upwards and downwards curing cycle. The area of the slab which recorded the highest movements lies between JDMDs 1 and 8 (cracks 1 to 4).

It is interesting to note that the loading did cause some degree of deterioration. At the end of the test, on 25 June, a total temperature differential of 19° was recorded but the total curling of the slab was greater than that recorded on the hottest day (3 June). One possible explanation of this may be that, because of the aggressive trafficking load, the contact between the steel bars and the concrete deteriorated, allowing the concrete slab to curl more freely.

5.4.1 Assumptions regarding curling caused by differential shrinkage and daily temperature differentials

As all movements are relative to a pre-determined reference point, the following methodology was followed for this study:

It was assumed that the concrete slab is in a planar (flat) position when there is a zero temperature differential between the top and the bottom surfaces of the slab. This assumption ignores the effects of differential shrinkage, which causes warping. During a 24-hour cycle, a concrete slab typically experiences two periods when the temperature differential is zero: early in the morning and again in the late afternoon. In this study, sensor displacement at the time during which the temperature differential (top – bottom) was zero, was used as the reference point for all the measurements.

It is important to note that the shape of the concrete slab after setting is not planar. It is quite possible that, because of differences in shrinkage between the top and bottom of the slab, the shape of the slab at zero temperature differential will not be planar and that the slab may actually be permanently warped with zero planar temperature differentials. It has been found in research done under the HVS in California near Palmdale (3,4) that the PCC sections did not have planar horizontal slabs and that separation between the slab and the base was observed all along the slab edge. These observations indicate that the corners and edges of the concrete slabs at Palmdale had warped upwards.

The effect of this is that a negative temperature difference (during the night) will cause the slab edges and corners to curl upwards even more and that a positive temperature difference (during the day) will cause the slab corners and edges to curl downwards, but they may never go down past a planar shape.

Another way of presenting the data is to ignore the whole discussion of the slabs' true position when they have been in a state of zero temperature differences, and simply to report the total elastic movement during a 24 hour cycle in relation to the total maximum temperature differences between the top and the bottom of the slabs during the same time period.

The results discussed in Section 5.4 reflect both these approaches.

5.4.2 Horizontal movements due to daily temperature variations

As discussed in Section 5.2, concrete contracts and expands under the influence of daily temperature changes. With the aid of the JDMDs placed horizontally across certain cracks it was possible to measure the horizontal movement of the slab between the cracks. To investigate this effect, the same four periods mentioned above were investigated (3, 6, 13 and 26 June 2004). The data are illustrated in Figure 5.14 and are summarized in Table 5.3. Because the expansion and contraction of the slab are mainly a function of the surface temperature (and, to a lesser extent, the temperature difference between the top and the bottom of the slab), the surface temperatures for each 40-hour cycle are also shown on the graph.

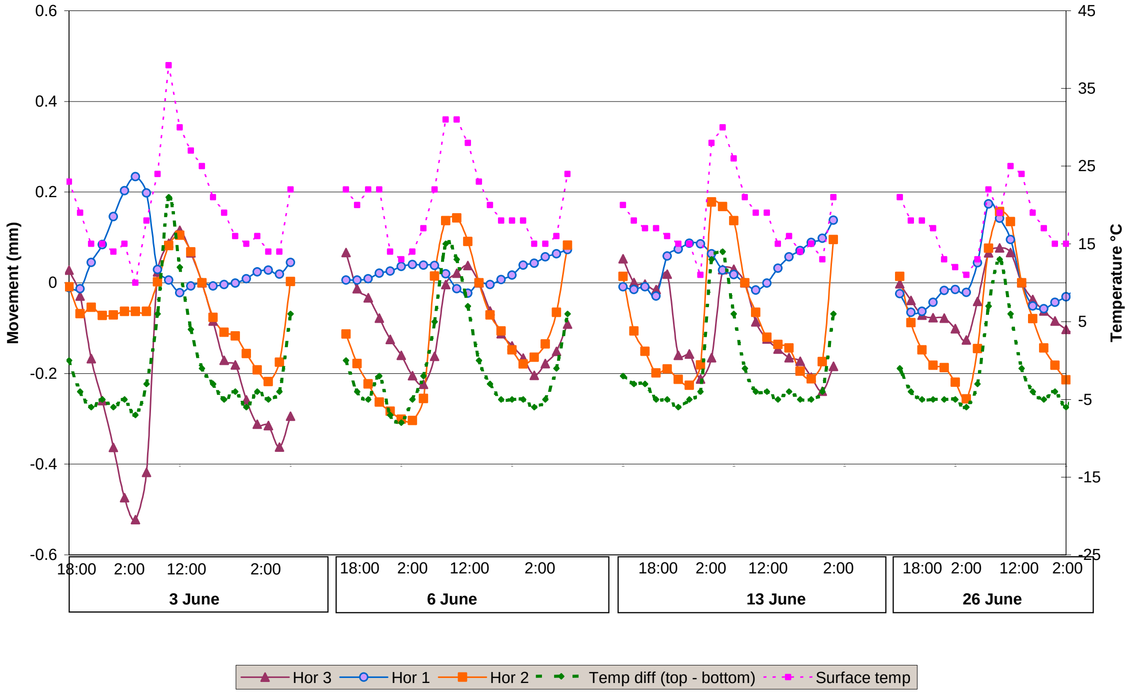


Figure 5.14: Horizontal slab expansion / contraction due to daily temperature changes

Table 5.3: Summary of horizontal slab movements due to daily temperature changes

Instrument	Placement across crack	03-Jun Max Expansion (mm)	03-Jun Max Contraction (mm)	03-Jun Max Total Movement (mm)	06-Jun Max Expansion (mm)	06-Jun Max Contraction (mm)	06-Jun Max Total Movement (mm)	13-Jun Max Expansion (mm)	13-Jun Max Contraction (mm)	13-Jun Max Total Movement (mm)
JDMD Hor 1	2	0.23	-0.02	0.26	0.07	-0.02	0.10	0.14	-0.03	0.17
JDMD Hor 2	3	0.11	-0.22	0.32	0.14	-0.30	0.45	0.18	-0.23	0.40
JDMD Hor 3	4	0.12	-0.52	0.64	0.07	-0.22	0.29	0.05	-0.24	0.29
All Hor JDMDs: Ave Max total movement				0.41			0.28			0.29
Maximum surface temperature				38		31			30	
Minimum surface temperature				10		13			11	
Maximum temperature differential				21		15			14	
Minimum temperature differential				-7		-8			-6	

At end of test 25 June 2004				
Instrument	**Placement across crack**	**Max Expansion** (mm)	**Max Contraction** (mm)	**Max Total Movement** (mm)
JDMD Hor 1	2	0.19	-0.06	0.26
JDMD Hor 2	3	0.16	-0.28	0.43
JDMD Hor 3	4	0.08	-0.13	0.21
All Hor JDMDs: Ave Max total movement				0.30
Maximum surface temperature				25
Minimum surface temperature				10
Maximum temperature differential				13
Minimum temperature differential				-6

Note: Data of JDMD Hor 1 for 3rd of June is unreliable (see Fig 5.14)

The methodology used for the determination of slab expansion and contraction is the same as what was used for the determination of upward or downward curl (see Section 5.3.1) and should be interpreted with caution. For the sake of clarity it might be more useful to look only at the total crack movement measured across cracks as the calculation of slab contraction / expansion depends on the arbitrarily selected reference point used for the calculation of slab expansion / contraction.

It is interesting to note that, unlike in the case of slab curl, the horizontal movements were not influenced by the trafficking load to the same extent as the vertical movement. The total horizontal movements remained relatively constant during the testing period In general a temperature change of approximately 18°C caused an average change in the crack width of 0.28 mm. At the end of the test a temperature change of 15° caused an average change in crack width of 0.30mm

5.5 ELASTIC SLAB DEFLECTIONS

The elastic deflections of the concrete slab under the influence of the HVS trafficking wheel were measured with the MDDs and JDMDs (see Section 4.1). Although the load on the trafficking wheel was originally set at 60kN (for the first 344 114 repetitions) and then increased to 80kN (up to 652 555 repetitions), deflection measurements were also taken at a standard half axle wheel load of 40kN. Deflections were recorded every two hours under the trafficking wheel load and deflections were measured under the standard half axle 40kN load daily at approximately 10:00.

5.5.1 JDMD Deflections

Figure 5.15 shows the deflections recorded by all the JDMDs placed along the edge of the slab recorded. The figure shows the deflections measured "on the fly" under the trafficking load at the time of data collection every 2 hours. The daily temperature difference (top – bottom) is also shown on the graph.

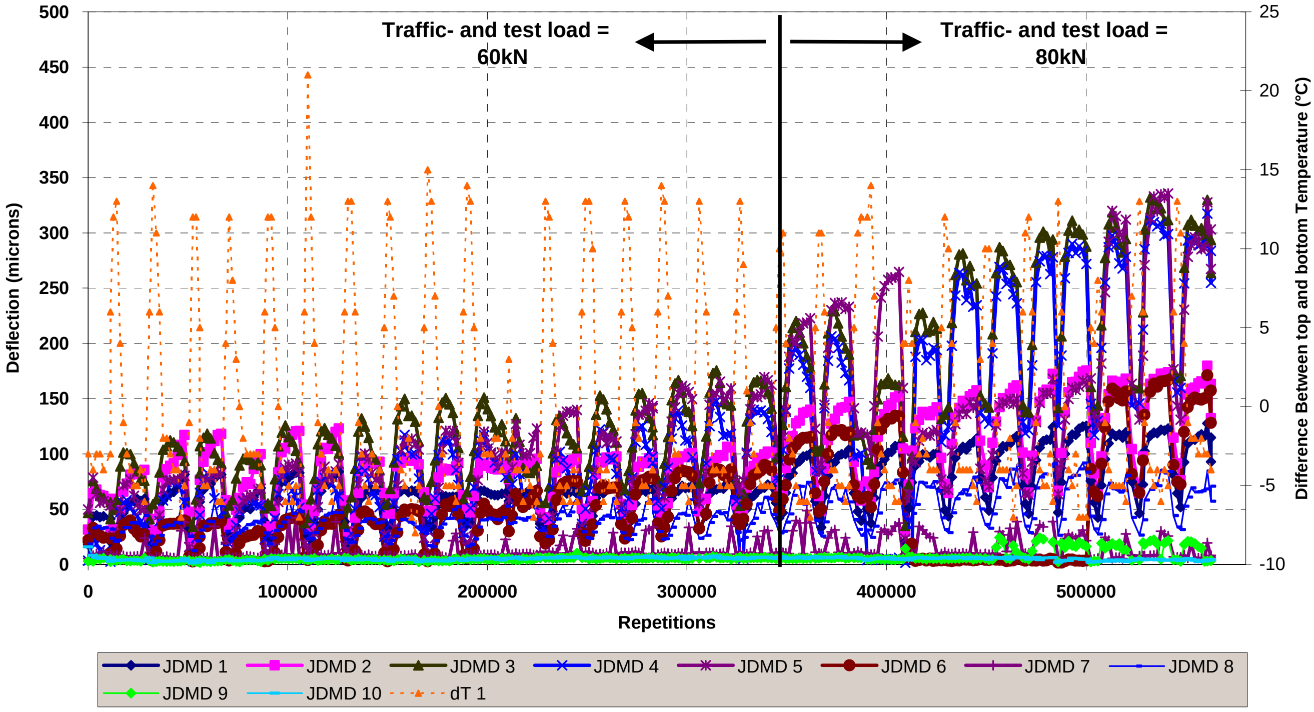


Figure 5.15: JDMD deflections

The cyclic variations recorded by the JDMDs are due to the daily temperature fluctuations. As the temperature differential increases (during the hotter part of the day) The slab curls downwards. The downward movement causes the slab to have better contact with the underlying support structure, resulting in a decrease in deflections all along the edge. At night a negative temperature differential was recorded (the slab being colder at the top than at the bottom) which resulted in the slab curling upwards. This upward movement caused the edge of the slab to become detached from the underplaying support, resulting in higher recorded deflections.

After 344 114 repetitions the trafficking and testing load were increased to 80 kN, which explains the sudden increase in deflections from that point onwards.

To investigate Figure 5.15 in greater detail, the deflections during the hottest part of the day were isolated from the deflections during the coldest part of the night. Figure 5.16 shows the maximum deflection that occurred at night together with the minimum surface temperature and corresponding temperature differential (top – bottom) during nine selected periods during testing. Figure 5.17 shows the minimum deflection that occurred during the hottest part of the day. The figure also shows the maximum surface temperature together with the corresponding temperature differential (top – bottom) on the 2nd Y-axis.

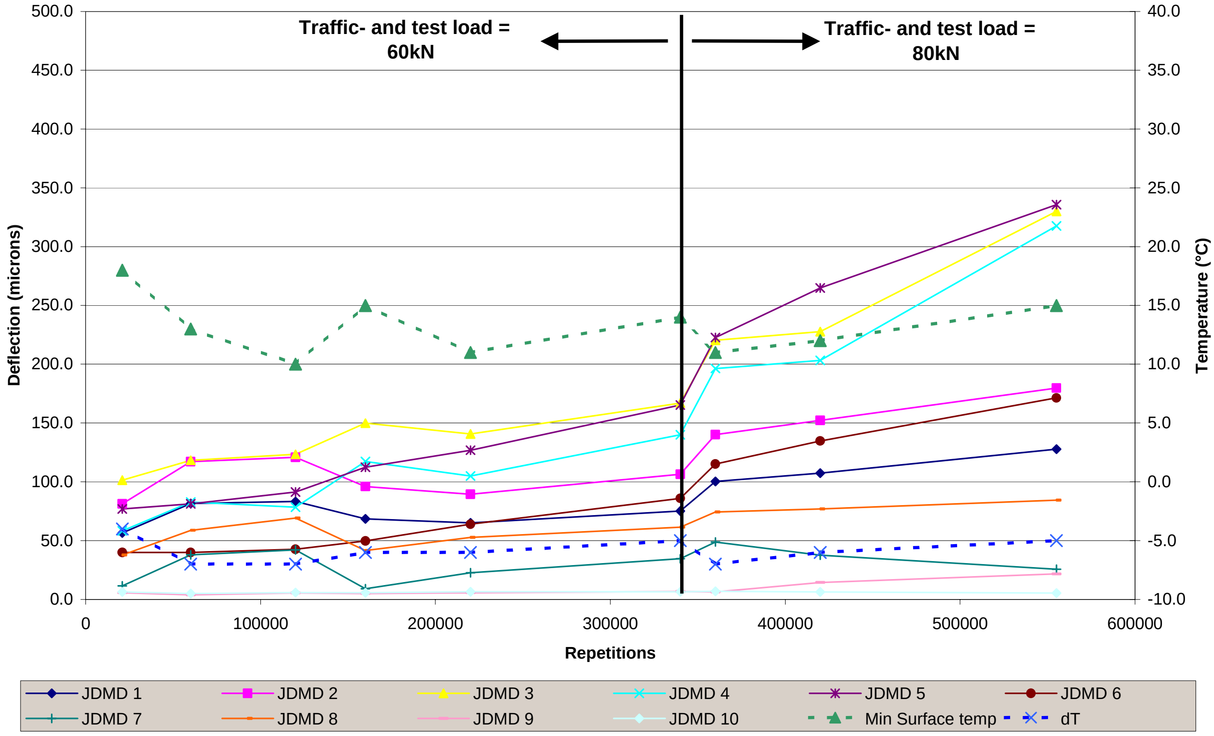


Figure 5.16: JDMD maximum deflections at night

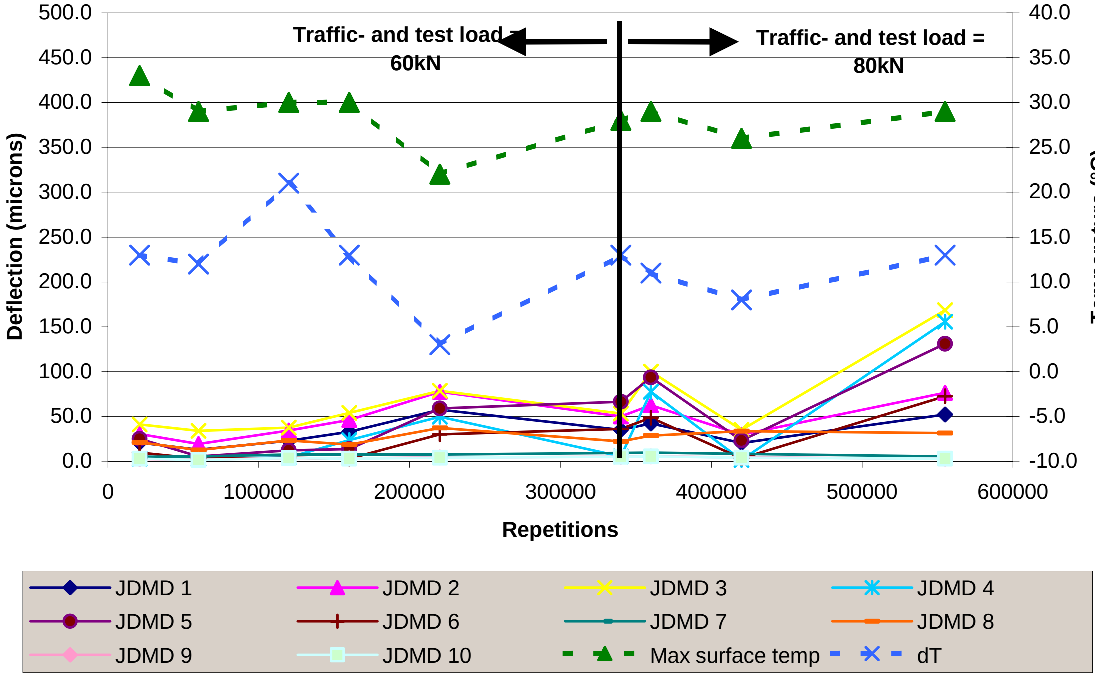


Figure 5.17: JDMD deflections during the day

The first important observation is the relationship between surface temperature, temperature differentials and deflection. During the night as the slab cools down, the temperature differentials were in the order of –5°C (the surface temp was approximately between 10 and 15 °C at that time) and deflections were approximately 3 times higher for the 60kN case, than those recorded during the hottest part of the day when the temperature differentials were in the order of 10 to 15 °C (with corresponding surface temp of approximately between 25 and 35 °C). The same conclusion is made as with Figure 5.15: During the day, the slab expands and curls downwards, which results in better contact between the slab and the underlying support layers. This higher degree of contact between the PCC slab and the base results in an decrease in deflection. At night the opposite occurs: The edge of the slab curls away from the base and this reduction in support causes an increase in deflection.

In Figure 5.15 can also be seen that a degree of damage did occur with time. There is a noticeable increase in the edge deflections with an increase in the number of load repetitions for both the 60kN and 80kN phases.

Figure 5.18 shows the 40kN test load data which was collected at approximately 8:00 every day.

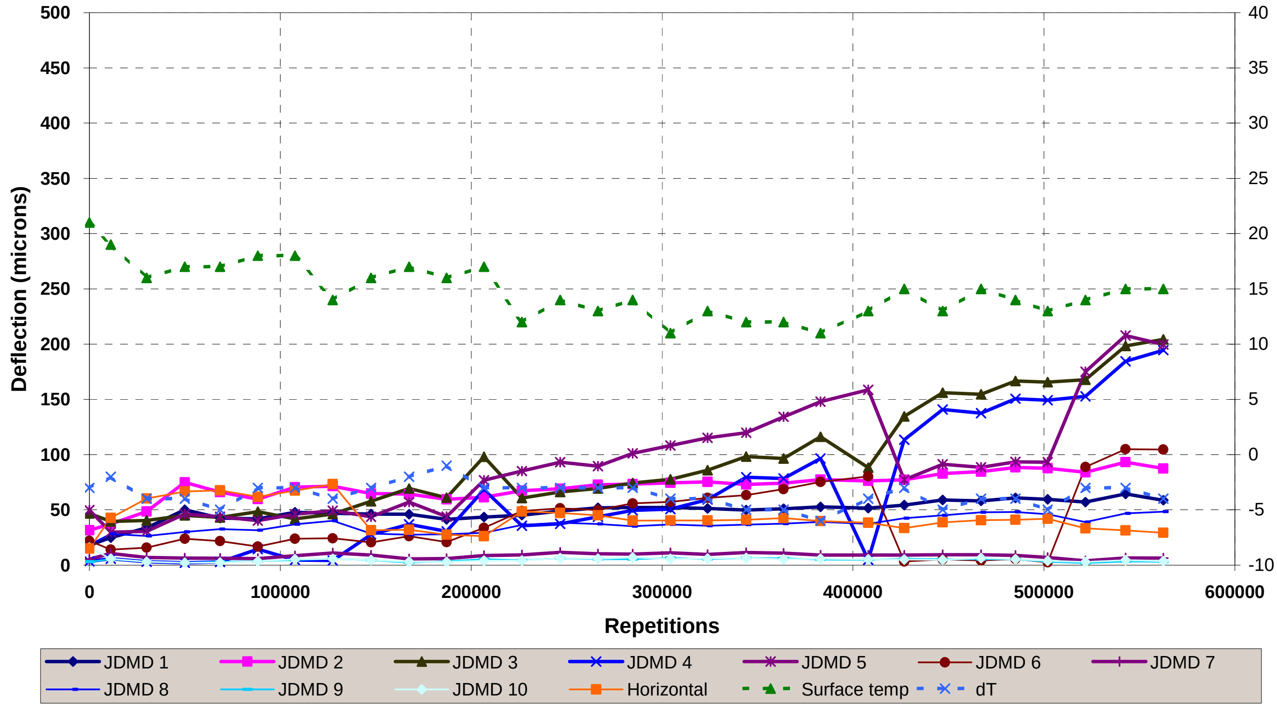


Figure 5.18: JDMD deflections under a 40kN test load

As can be seen from Figure 5.18 the temperature was fairly constant at about 8:00 in the morning when the data were collected and the increase in deflections is due to the damage caused by the applied loading. The irregularities in the data from JDMD 4 and 5 between about 400 000 and 500 000 repetitions are probably due to the water which was added to the cracks.

5.5.2 Horizontal elastic crack movements

Figure 5.19 shows the elastic horizontal movement under the influence of a 40kN test load. The data in the graph show the elastic horizontal crack movement of Crack 1 (H1), Crack 3 (H2) and Crack 4 (H3), as explained in Section 4.1.

It is clear that the trafficking load had very little influence on the horizontal movement of Cracks 2 and 4. The measured crack movements were less than 75 microns throughout the test. However, the movement recorded at Crack 3 increased with loading. At the start of the test movements in the order of 50 microns were recorded, which gradually increased to about 150 microns towards the end of the test. The fact that very little or no change in horizontal movements could be detected with loading is probably due to the steel reinforcement, which kept the concrete slab intact despite the severe loading regime and existing crack pattern.

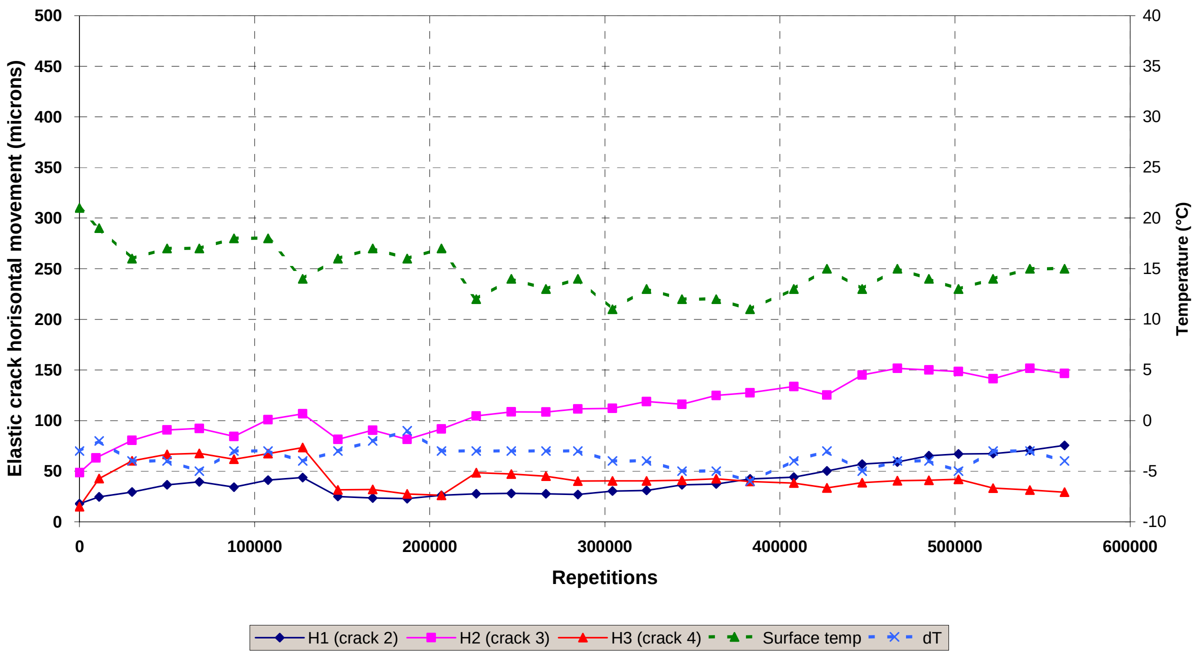


Figure 5.19: Horizontal crack movements under 40kN loading

The data for the 60 and 80kN loading cases can be seen in Figure 5.20. The same cyclic effect as that seen with the JDMDs (see Figure 5.15) is observed here. Elastic crack movements are reduced at high surface temperatures and vice versa. Crack 3 had the greatest degree of movement followed by Cracks 2 and 4. It is obvious that the high trafficking load (80kN) had an impact on the crack movement and a noticeable increase in crack movement is observed after the wheel load was increased to 80kN. This effect was more evident in the case of Crack 3 than in the others.

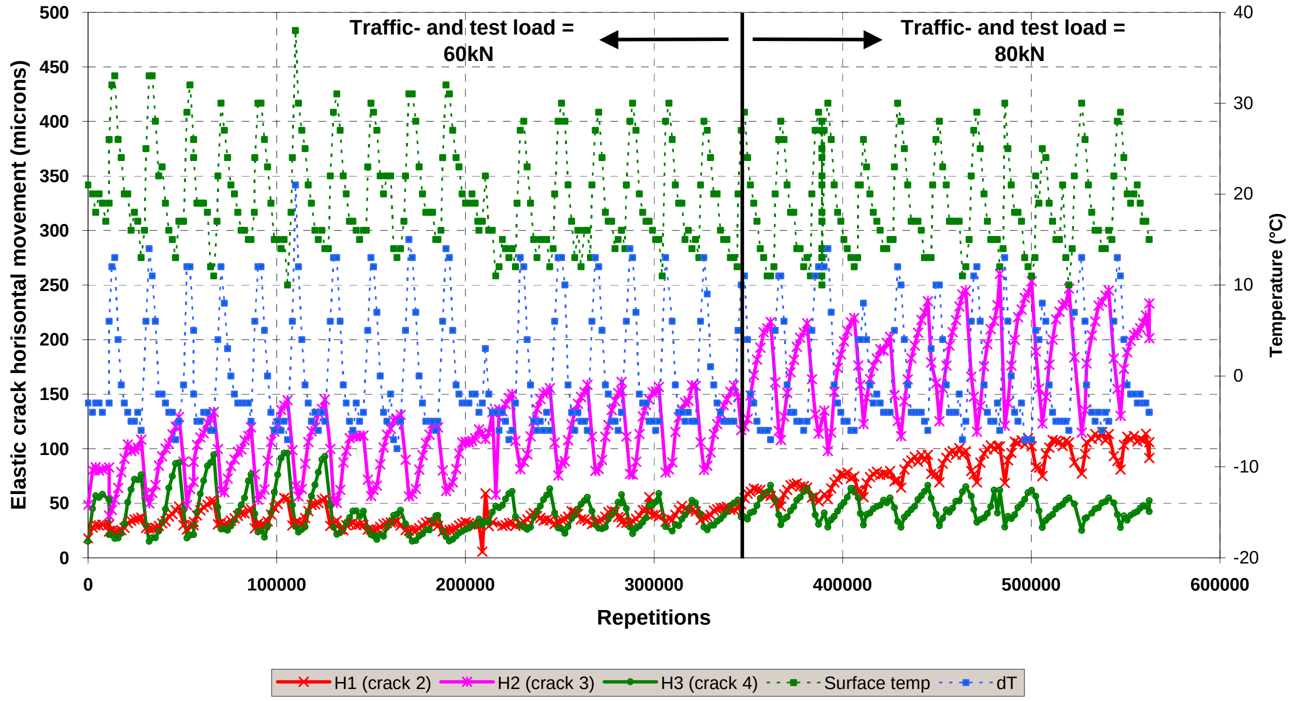


Figure 5.20: Horizontal crack movements under 60 and 80kN loading

5.5.3 MDD deflections

MDDs were placed according to Figure 4.1. Their layout is not repeated here. Figures 5.22 and 5.23 show the MDD deflection data for all the MDD modules under both 60 and 80kN loading.

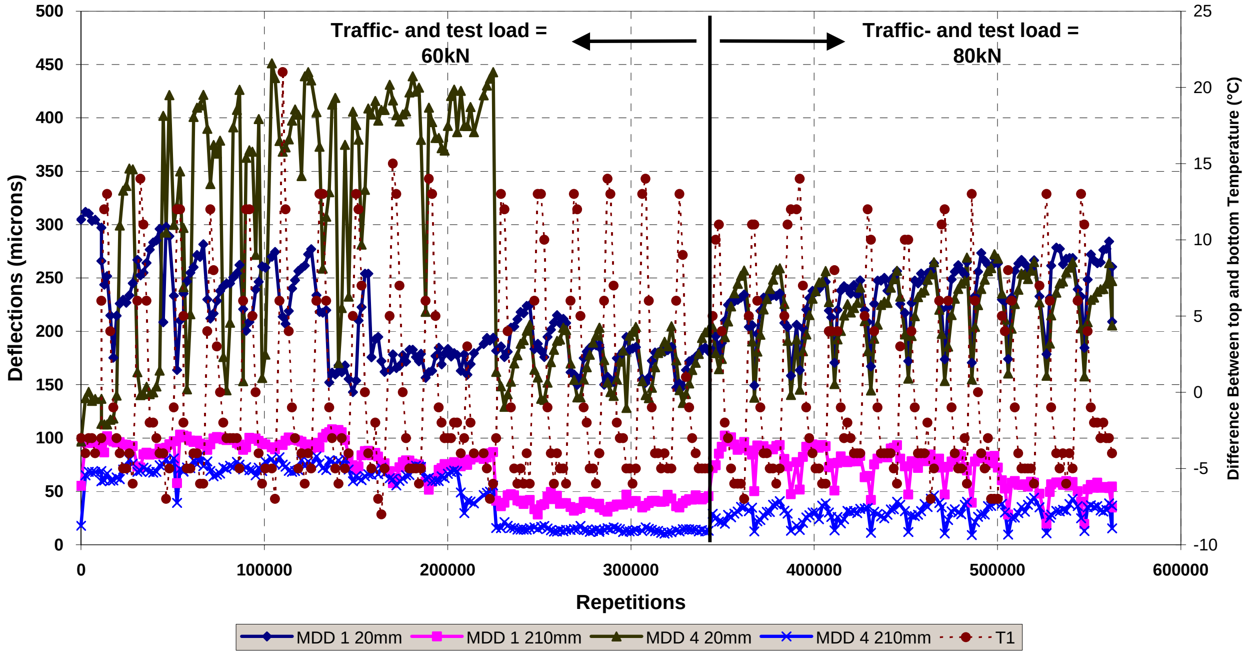


Figure 5.21: Deflection data from MDDs 1 and 4

It is clear that MDD 2 at 20mm recorded erroneous results during the first 220 000 repetitions. The reason for this is not known and this data set should be excluded from any analysis.

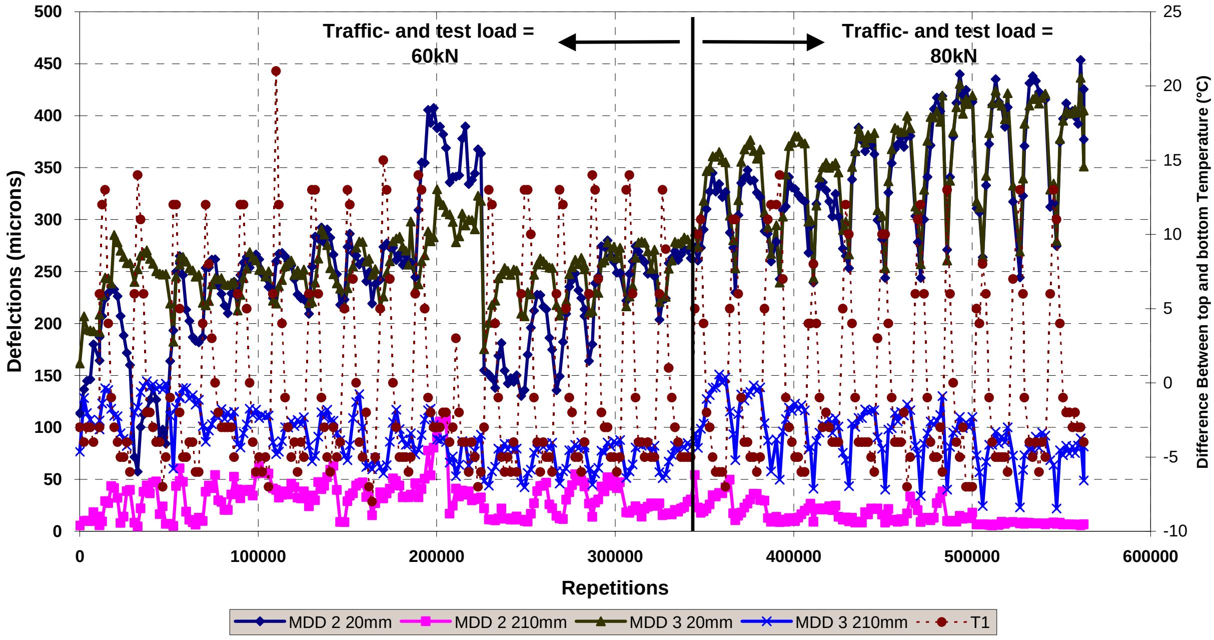


Figure 5.22 Deflection data MDDs 2 and 3

The deflections measured by the MDDs are higher than those measured by the JDMDs. The reason for this is that the JDMDs were placed along the edge of the pavement some distance away from the travelling wheel whereas the MDDs were placed directly under the wheel. Because of the physical obstruction they cause, JDMDs cannot be positioned under the wheel. (see Photo 4.1).

The sudden changes in the deflections measured by the top modules (20mm) during the first 220 000 repetitions were unexpected and are difficult to explain. They may be due to the water which was added to the cracks, which affected the accuracy of the modules (see Picture 5.2). The deflections recorded in the concrete (20mm) with MDDs 2 and 3 (placed on either side of Crack 3) were higher than those recorded with MDDs 1 and 4 (placed close to Cracks 1 and 4). This observation is in agreement with what was seen in with the JDMDs. The deflections around Cracks 2 and 3 were generally significantly higher than those recorded at Cracks 1 and 4.

The deflections originating in the base (210mm) remained relatively constant throughout the testing period, except for the expected daily variations due to temperature changes.

The deflections measured in the concrete are significantly higher than those measured in the base just below the concrete. This is an indication of the degree of slab lift-off or delamination between the base and the bottom of the PCC layer. If the PCC layer was in full contact with the base course, almost all the deflection originating at the surface would have been transferred to the base but as seen in Figures 5.22 and 5.23, only 20 to 35 % of the deflections were transferred to the MDD modules placed in the upper part of the base course. This observation is also true for the 80kN loading case. As can be seen from the graphs, the deflections measured in the concrete showed a significant increase after the wheel load was increased to 80kN, but the deflections measured in the base remained almost constant irrespective of the magnitude of the traffic load.

Figures 5.24 and 5.25 show the MDD deflection data under a test load of 40kN. These readings were taken daily at approximately 08:00.

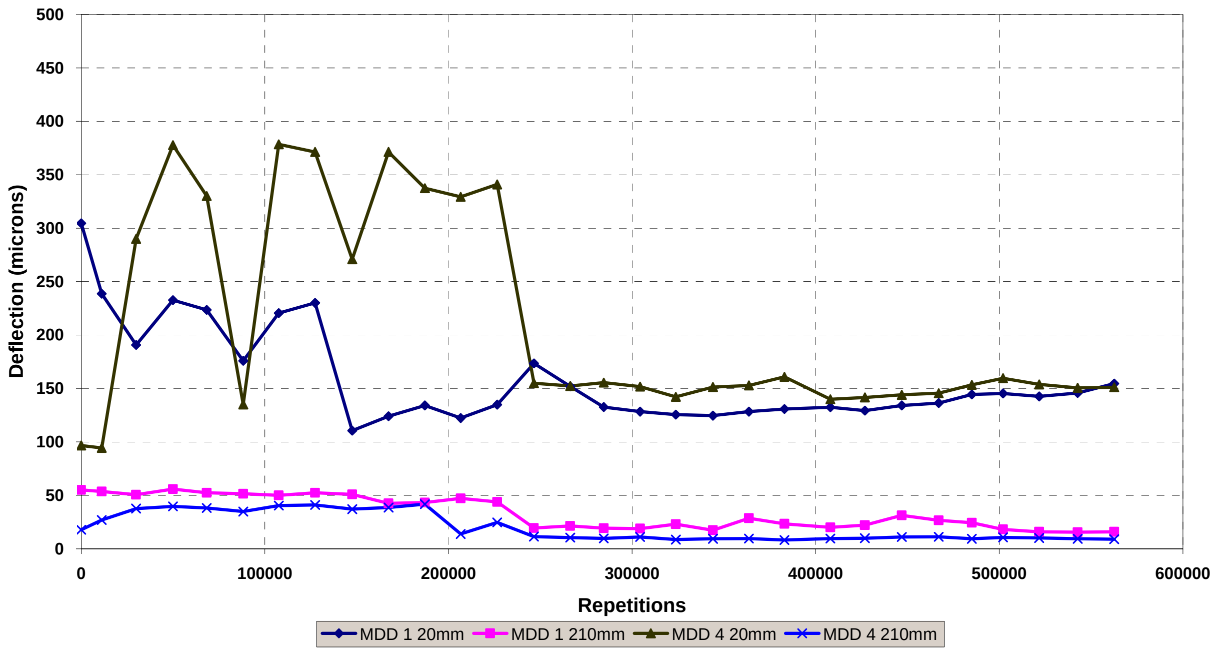


Figure 5.23: MDD 1 and 4 deflections under a 40kN test load

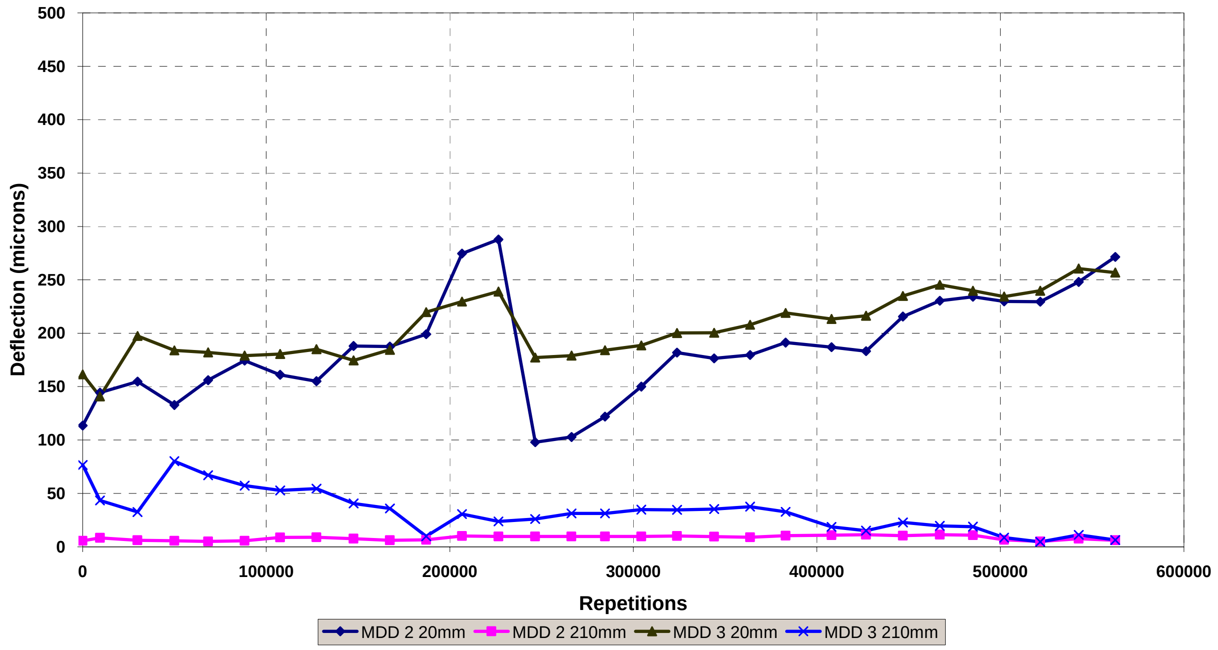


Figure 5.24: MDD 2 and 3 deflections under a 40kN test load

Trends similar to those observed under the 60 and 80kN test loads were observed under 40kN loading: The deflections measured at Cracks 1 and 4 remained relatively constant during the whole testing period whereas the deflections measured at Crack 3 (MDDs 2 and 3) showed an increase with the number of repetitions. The deflection in the concrete at Crack 3 started at approximately 150 microns and increased to approximately 250 microns towards the end of the test.

5.5.4 Relative movements

With the aid of the MDDs (placed on both sides of Crack 3) and the JDMDs it is possible to calculate the relative movement or Load Transfer efficiency across cracks. Because the concrete layer consisted of reinforced steel (see Section 2.0) the cracks are not expected to show a high degree of relative movement and very low values were recorded.

The following description of the calculation of relative movement is extracted from the 1st level report on the HVS testing at the Hilton concrete testing site (2) and is repeated below for the sake of clarity:

With two JDMDs or MDDs directly adjacent to each other, but on opposite sides of a crack, it is possible to calculate the variation in load transfer efficiency of the crack over time. Figure 5.25 is a plot of typical deflection curves, measured with two JDMD sensors on the same time scale.

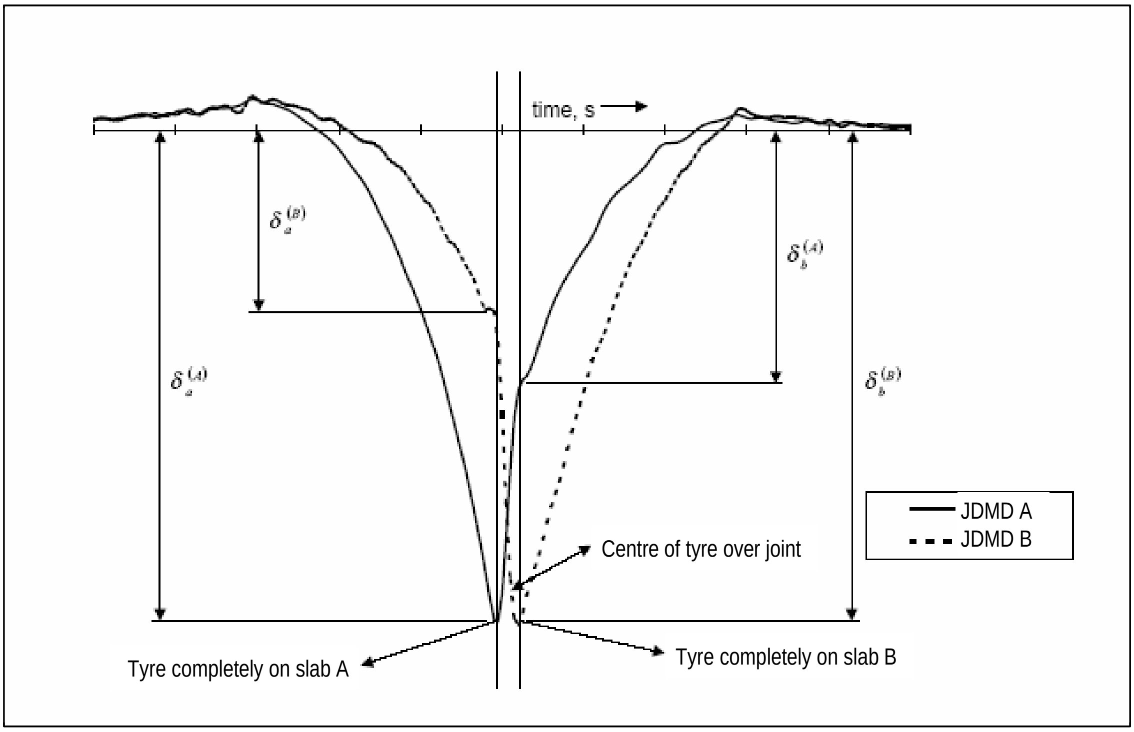


Figure 5.25: Calculation of load transfer efficiency from two JDMD sensors

The efficiency of the load transfer at the joint is calculated as:

$$e_\delta^{(fwd)} = \frac{\delta_a^{(B)}}{\delta_a^{(A)}} \quad \text{(Eq. 5.1)}$$

in the forward direction, or as:

$$e_\delta^{(back)} = \frac{\delta_b^{(A)}}{\delta_b^{(B)}} \quad \text{(Eq. 5.2)}$$

in the backward direction, where $\delta_a^{(B)}$ is the displacement response at JDMD B when JDMD A experiences a peak, $\delta_a^{(A)}$ is the peak displacement response at JDMD A. $\delta_b^{(A)}$ is the response at JDMD A when JDMD B experiences a peak, and $\delta_b^{(B)}$ is the peak response at JDMD B. The difference in the actual values of e_δ computed from Equations 5.1 and 5.2 give an indication of the degree of asymmetry of the joint. The average load transfer efficiency is given by:

$$e_\delta^{(avg)} = \frac{1}{2}\left[e_\delta^{(fwd)} + e_\delta^{(back)}\right] \quad \text{(Eq. 5.3)}$$

The relative movement at the crack can be determined in a similar fashion, either by calculating the difference between the peak displacements measured on either side of the crack, or by calculating:

$$RM_{\delta}^{(fwd)} = \delta_a^{(A)} - \delta_a^{(B)} \qquad \text{(Eq. 5.4)}$$

in the forward direction, and:

$$RM_{\delta}^{(back)} = \delta_b^{(B)} - \delta_b^{(A)} \qquad \text{(Eq. 5.5)}$$

in the backward direction. The average relative movement can then also be calculated as for the average load transfer efficiency (Equation 5.3).

To summarise: When the relative movement across a crack is low this means that good load transfer is taking place and that little deterioration of the load-bearing capacity at the crack has taken place. On the other hand, when the relative movement across a crack is high this suggests that a high degree of deterioration has taken place and that joint faulting will or is developing.

For the sake of clarity the following definitions were used in this report:
The relative movement reported by sensor A was calculated using Equation 5.4 and the relative movement at sensor B (placed right next to a on the other side of the crack) was calculated using the backward calculation (Equation 5.5)

Figure 5.26 shows the relative movement at Crack 3 as measured by MDDs 2 and 3 under the influence of a 40kN test load.

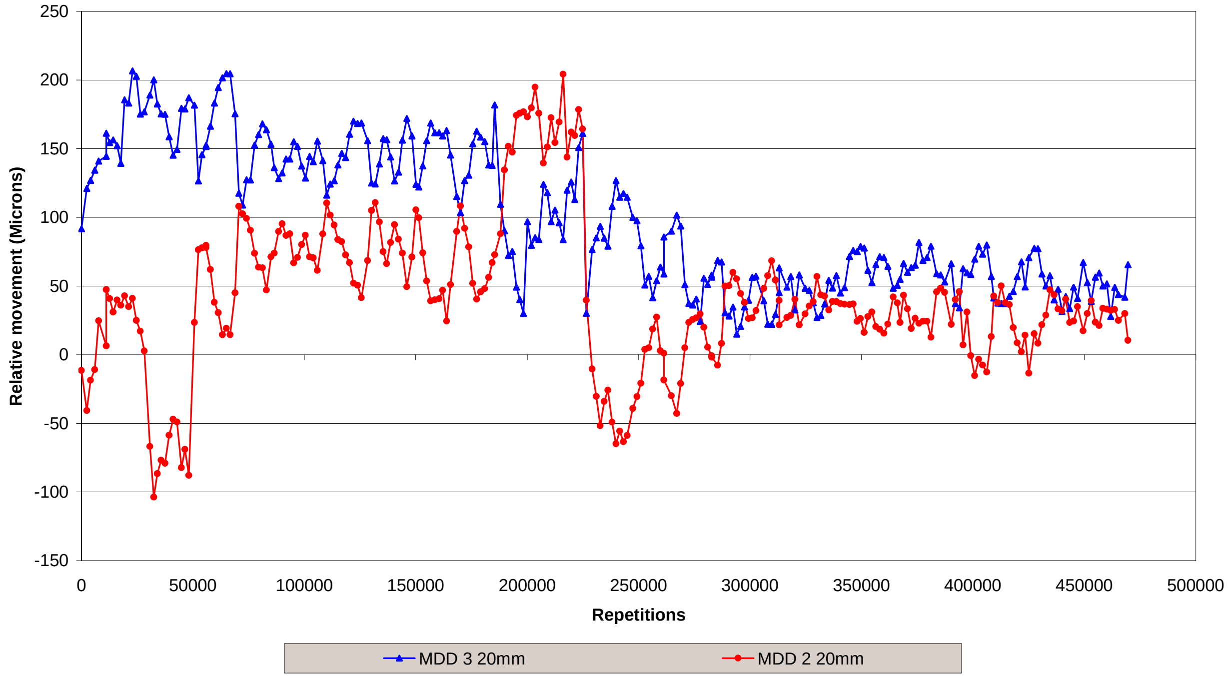


Figure 5.26: Relative movement at Crack 3 measured by MDDs

As explained earlier, the MDD results of the first 220 000 repetitions are suspicious and should be ignored. As seen from Figure 5.26 the relative movements in the forward and backward calculations are low (in the order of 0.05 mm) under the influence of a 40kN test load. This indicates that despite the crack and the reinforced steel together with aggregate interlock, good load transfer is taking place and even after the 80kN loading phase, no deterioration of the crack could be detected.

Figure 5.27 displays the relative movements as calculated by the various JDMDs.

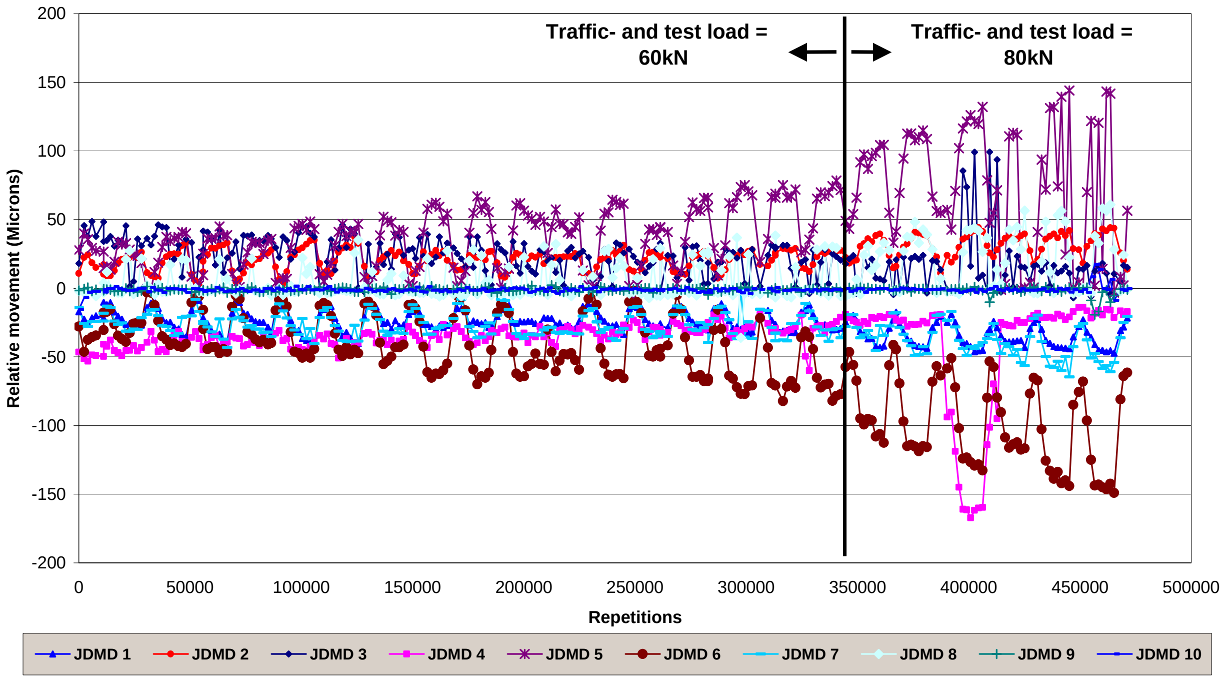


Figure 5.27: Relative movement at all cracks measured by the JDMDs

As seen from Figure 5.27, almost all the cracks display a very low degree of relative movement except at Crack 3. The relative movement at Crack 3 is measured by JDMD 5 (in the forward calculation) and JDMD 5 (in the backward calculation), which show a noticeable degree of deterioration after the load was increased to 80kN. Relative movements in the order of zero to 50 microns were recorded, which increased to approximately 150 microns towards the end of the test. A relative movement of 150 microns (0.015 mm) is still considered to be low, especially in light of the fact that these movements were caused by a 80kN half axle load.

The cyclic variation is due to the daily temperature variations as explained earlier. Obviously during the cold part of the night the slabs cool down and contract. This contraction causes a drop in the degree of aggregate interlock and an increase in relative movements is observed. During the day the slabs expands which causes an increase in the degree of aggregate interlock. This obviously leads to a decrease in the relative movements as displayed in Figure 5.27.

6 CONCLUSIONS

This report summarises the main results from HVS testing conducted on a CRCP concrete inlay on the N3 near Pietermaritzburg at km 16.3 on the northbound side. The pavement, which was built in 1998, has carried approximately 6.5 million standard 80kN axle loads and has severe cracking patterns in certain sections. The aim of this series of HVS testing was to determine the remaining life of the inlays through accelerated HVS testing. In order to obtain a representative result, an HVS testing area was selected that closely resembles the distress areas of the existing pavement and was selected at an area where 5 significant transverse cracks were found within a 6m long test strip.

The pavement inlay consisted of a 180mm continuously re-inforced concrete slab placed on top of an asphalt base, a stabilised dolerite subbase and 2 selected weathered shale layers. HVS testing was, however, conducted in an area where there was no asphalt under the concrete.

HVS testing started on 28 May 2004 and finished on 28 June 2004 after the application of 562 555 load repetitions. Because of financial constraints testing was only allowed for a limited time period and various techniques were implemented to ensure that a meaningful result was obtained within the allowed time frame. These interventions include:

a) Greater than normal trafficking load:
Loading was done in 2 phases: a 60kN phase which ran from the beginning until 344 114 repetitions and a second phase of 80kN loading which ran from 344 114 repetitions until the end of the test. This loading regime is substantially heavier than the design half-axle load of 40kN. Using the 4.2 power damage law a total of 1.89 million standard loads were simulated by the 60kN loading case and an additional 4.01 million standard loads simulated by the 80kN loading phase. The testing area with its 5 cracks were, therefore, subjected to a total of 5.9 million standard axle loads before testing was stopped.

b) Water was added to the cracks.
It was decided that if no visible damage could be detected after one week's testing, water would be added to the cracks to promote pumping and to accelerate deterioration.

The most significant observations are briefly discussed:

Environmental influences on the behaviour of the concrete slab

Temperature plays a major role in the behaviour of the concrete slab in two ways. Firstly, it is demonstrated in the study that daily variations in the surface temperature of the slab cause the slab to go through cycles of expansion and contraction. Horizontal slab movements in the order of 0.3mm were recorded under the influence of a 13 to 15°C change in surface temperature.

Secondly, owing to temperature differentials (the temperature at the surface of the slab – the temperature at the bottom) the concrete slab goes though cycles of being curled upwards (at night when the top is colder than the bottom) and being curled downwards (during the day when the top is hotter than the bottom). Total vertical curling movements of up to 0.48mm were recorded under the influence of a total temperature difference of 19°C (+13°C during the day to –6°C at night) between the top and bottom of the concrete slab.

These two effects played a major role in the behaviour of the concrete under accelerated loading. Throughout this study the influence of these slab movements is very visible on the parameters used to determine the extent and degree of damage. Elastic deflections, which serve as an indication of the structural strength of the pavement, showed variations of up to 700% due to temperature variations. The deflections measured at night were at least double what were recorded during the day at the same position. It is obvious that deflection measurements were highly dependent on the time of day. It is therefore very important that slab curl resulting from temperature variations should be built into any deflection analysis on concrete pavements.

Permanent warping due to differential shrinkage

Although concrete shrinkage was limited by the application of reinforced steel bars during construction and curing, it is shown in this study that slab warping due to differential shrinkage between the upper and the lower part of the concrete layer played a significant role in the measured deflections. Deflection sensors placed just below the concrete in the base layer registered small deflections even under a 80kN test load. Where MDDs placed inside the concrete slab registered deflections in the region of 450 microns under the influence of a 80kN test load, the MDDs placed in the base just below the bottom of the PCC layer (210mm) registered less than 130 microns. This means that only 30 per cent of the deflection that originated in the concrete was recorded in the base by the sensors placed just below the surface. One explanation for this observation is that, due to differential shrinkage, the slab is slightly curled upwards all along its longitudinal edge, creating a cavity between the bottom of the PCC layer and the base course.

<u>Traffic-induced changes in the behaviour of the concrete slabs</u>
Despite the cyclic variation in deflections due to daily temperature variations, a noticeable degree of deterioration could be detected, especially at Cracks 2 and 3, as a result of the accelerated loading. Crack 5 showed the least amount of damage during the testing period and very little deterioration could be detected. Maximum deflections with a 60kN test load at Cracks 2 and 3 were in the order of 100 microns, which increased to approximately 160 microns after the first phase of testing. During the second phase (the 80kN phase) 80kN test load deflections at the same cracks (2 & 3) started at approximately 220 microns, increasing to approximately 330 microns at the end of the test.

Standard 40kN test loads were measured once a day throughout the study in order to monitor deflections with time under a constant test load. Cracks 1,4 and 5 showed very little deterioration during the whole testing period. The 40kN test load caused deflections in the order of 50 microns at Cracks 2 and 3, which increased to approximately 200 microns at the end of the test.

Because of the slow progression in the deterioration of the tested area, water was added to all cracks after the first week of testing. An average of 47 litres of water every 2 hours, or 564 litres of water per day was poured into the cracks. During the testing period of 25 days a total of 1340 litres of water was poured into the cracks. This intervention had almost no effect on the pavement response parameters measured during this study. It seemed as if the water just drained into the subgrade without having any effect on the pavement structure. Very little pumping could be detected during the testing period. Clean water was pumped out at certain times and water was observed coming through the MDD holes.

Crack width and crack growth were measured twice daily and very little deterioration was detected. Only 2 additional hairline cracks could be detected at Cracks 4 and 5 after the completion of the test. No obvious visual deterioration could be detected and at the end of the test, the surface of the tested area was visually in the same condition as that observed at the start, prior to accelerated loading.

The final important engineering parameter that was measured during this study was the relative movement across cracks. Through measurement of this parameter it is possible to determine the load transfer efficiency (LTE) of the concrete slabs across the cracks. A high LTE (low relative movement) is indicative of a concrete pavement with good aggregate interlock and bonding between the slabs and of the ability of the slab to distribute the load across cracks.

Throughout the testing period, the relative movements across the cracks were very small, in the order of less than 0.05 mm, even under the 80kN wheel load. Crack 3 was the only crack in which a noticeable increase in relative movements could be detected with increased number of repetitions. Values in the order of 0.15 mm were recorded towards the end of the test. The contraction and expansion of the concrete slab due to daily temperature variations also had a significant effect on the LTE. During daytime, slab expansion caused a higher degree of aggregate interlock, with consequent lower relative movements, than at night when slab contraction took place. On account of the reinforced steel and good aggregate interlock, no noticeable degree of joint faulting was observed at any of the cracks.

The testing area was subjected to a total of 6.5 million standard axle loads prior to accelerated trafficking and to an additional 5.9 million for a total of 12.4 million standard load applications. As the pavement was not considered to have failed at the end of the testing period, it is not possible to determine with an acceptable degree of certainty the remaining life of the inlays.

7 RECOMMENDATIONS

Firstly, it is recommended that a test pit should be dug at the HVS testing site on the PCC inlays. This will reveal the actual underlying structure of the pavement and will provide valuable information regarding the way in which water dissipated through the pavement without causing any visible damage.

For the determination of the remaining life of the PCC inlays it will be important to do additional falling weight deflectometer (FWD) testing at various sections of the inlays around Pietermaritzburg. In line with the findings in this research study, it is recommended that FWD testing should preferably be done during the coldest part of the day (typically just before sunrise) in order to characterise the maximum deflection behaviour.

Air, surface as well as in-depth temperatures of the concrete structure should be collected at regular intervals during FWD testing. Owing to the non-linear influences of various environmental factors such as humidity, rainfall, temperature and wind speed, deflection normalization based on temperature is not recommended in this study.

Care should be exercised in the interpretation of the calculated additional traffic in terms of number of standard load applications that the tested section will be able to carry before failure. Although the well-recognised 4.2 damage factor was used, it is possible that the pavement tested is much less load-sensitive and that pavement failure may occur before the projected time of failure.

In order to characterise the behaviour of CRCP inlay sections, additional HVS testing is recommended. Because of the important influence which the environment has on the behaviour of concrete pavements, more controlled tests are recommended in order to isolate the effects of loading on the residual life of this type of pavement.

8 REFERENCES

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