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Reflective Cracking Study: First-Level Report on HVS Testing on Section 590RF — 90 mm MB4-G Overlay

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Partnered Pavement Research Program (PPRC) Contract Strategic Plan Element 4.10:
Development of Improved Rehabilitation Designs for Reflective Cracking

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Abstract:
This report is the first in a series of first-level analysis reports that describe the results of HVS testing on a full-scale experiment being performed at the Richmond Field Station (RFS) to validate Caltrans overlay strategies for the rehabilitation of cracked asphalt concrete. It describes the results of the first HVS reflective cracking testing section, designated 590RF, carried out on a 90-mm full-thickness MB4-G overlay. The test forms part of Partnered Pavement Research Center Strategic Plan Element 4.10: "Development of Improved Rehabilitation Designs for Reflective Cracking."

HVS trafficking on the section commenced on January 13 and was completed on June 16, 2004. A temperature chamber was used to maintain the pavement temperature at 20°C±4°C. A dual tire (720 kPa pressure) and bidirectional loading with lateral wander configuration was used. A total of 1,981,365 load repetitions, equating to 37 million ESALs and a Traffic Index of 14, was applied during this period. Findings and observations based on the data collected during this HVS study include:

- No reflective cracking from the underlying severely cracked DGAC layer was observed on the MB4-G overlay after almost two million HVS repetitions. The MB4-G overlay thus appeared to successfully prevent any cracking in the underlying layer from reflecting through to the surface, despite final-to-initial deflections indicating that considerable damage had occurred in the asphalt layers under loading.
- The average maximum rut depth across the entire test section at the end of the test was 12.7 mm, equivalent to the Caltrans (and experiment) failure criteria of 12.5 mm. The maximum rut depth measured on the section was 19.0 mm, with a maximum rut depth of 12.5 mm reached after about 1.17 million repetitions, soon after the load increase from 60 to 90 kN.
- Ratios of final-to-initial elastic surface deflections under a 60 kN wheel load increased by between 1.9 and 3.0 times along the length of the section. The ratios for in-depth deflections show that damage increased at all depth in the pavement structure (between 1.9 and 2.2 times) by the end of trafficking. Loss of stiffness was highest in the area of most severe cracking in the underlying DGAC layer.
- Analysis of surface profile and in-depth permanent deformation measurements indicates that most of the permanent deformation (between 67 [at MDD6] and 88 percent [at MDD10]) occurred in the asphalt-bound surfacing layers (overlay and cracked DGAC) with marginal deformation in the base layer and negligible deformation in the subgrade.

No recommendations as to the use of MB4-G mixes are made at this time. These recommendations will be included in the second-level analysis report, which will be prepared and submitted on completion of all HVS and laboratory testing.

Keywords:
Reflective cracking, overlay, modified binder, HVS test, MB Road

Proposals for implementation:

Related documents:
UCPRC-RR-2005-03

Signatures:

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DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PROJECT OBJECTIVES

The objective of this project is to develop improved rehabilitation designs for reflective cracking for California.

This objective will be met after completion of four tasks identified by the Caltrans/Industry Rubber Asphalt Concrete Task Group (RACTG):

1. Develop improved mechanistic models of reflective cracking in California
2. Calibrate and verify these models using laboratory and HVS testing
3. Evaluate the most effective strategies for reflective cracking
4. Provide recommendations for reflective cracking strategies

This document is one of a series addressing Tasks 2 and 3.

ACKNOWLEDGEMENTS

The University of California Pavement Research Center acknowledges the assistance of the Rubber Pavements Association, Valero Energy Corporation, and Paramount Petroleum which contributed funds and asphalt binders for the construction of the Heavy Vehicle Simulator test track discussed in this study.

REFLECTIVE CRACKING STUDY REPORTS

The reports prepared during the reflective cracking study document data from construction, Heavy Vehicle Simulator (HVS) tests, laboratory tests, and subsequent analyses. These include a series of first- and second-level analysis reports and two summary reports. On completion of the study this suite of documents will include:

1. Reflective Cracking Study: Summary of Construction Activities, Phase 1 HVS testing and Overlay Construction (UCPRC-RR-2005-03).
2. Reflective Cracking Study: First-level Report on the HVS Rutting Experiment (UCPRC-RR-2007-06).
3. Reflective Cracking Study: First-level Report on HVS Testing on Section 590RF — 90 mm MB4-G Overlay (UCPRC-RR-2006-04).
4. Reflective Cracking Study: First-level Report on HVS Testing on Section 589RF — 45 mm MB4-G Overlay (UCPRC-RR-2006-05).
5. Reflective Cracking Study: First-level Report on HVS Testing on Section 587RF — 45 mm RAC-G Overlay (UCPRC-RR-2006-06).
6. Reflective Cracking Study: First-level Report on HVS Testing on Section 588RF — 90 mm AR4000-D Overlay (UCPRC-RR-2006-07).
7. Reflective Cracking Study: First-level Report on HVS Testing on Section 586RF — 45 mm MB15-G Overlay (UCPRC-RR-2006-12).
8. Reflective Cracking Study: First-level Report on HVS Testing on Section 591RF — 45 mm MAC15-G Overlay (UCPRC-RR-2007-04).
9. Reflective Cracking Study: HVS Test Section Forensic Report (UCPRC-RR-2007-05).
10. Reflective Cracking Study: First-level Report on Laboratory Fatigue Testing (UCPRC-RR-2006-08).
11. Reflective Cracking Study: First-level Report on Laboratory Shear Testing (UCPRC-RR-2006-11).
12. Reflective Cracking Study: Back Calculation of FWD Data from HVS Test Sections (UCPRC-RR-2007-08).
13. Reflective Cracking Study: Second-level Analysis Report (UCPRC-RR-2007-09).
14. Reflective Cracking Study: Summary Report (UCPRC-SR-2007-01). Detailed summary report.
15. Reflective Cracking Study: Summary Report (UCPRC-SR-2007-03). Four page summary report.

CONVERSION FACTORS

SI* (MODERN METRIC) CONVERSION FACTORS				
APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	**Convert From**	**Multiply By**	**Convert To**	**Symbol**
LENGTH				
in ft	inches feet	25.4 0.305	millimeters meters	mm m
AREA				
in^2 ft^2	square inches square feet	645.2 0.093	square millimeters square meters	mm^2 m2
VOLUME				
ft^3	cubic feet	0.028	cubic meters	m^3
MASS				
lb	pounds	0.454	kilograms	kg
TEMPERATURE (exact degrees)				
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	C
FORCE and PRESSURE or STRESS				
lbf lbf/in^2	poundforce poundforce/square inch	4.45 6.89	newtons kilopascals	N kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	**Convert From**	**Multiply By**	**Convert To**	**Symbol**
LENGTH				
mm m	millimeters meters	0.039 3.28	inches feet	in ft
AREA				
mm^2 m^2	square millimeters square meters	0.0016 10.764	square inches square feet	in^2 ft^2
VOLUME				
m^3	cubic meters	35.314	cubic feet	ft^3
MASS				
kg	kilograms	2.202	pounds	lb
TEMPERATURE (exact degrees)				
C	Celsius	1.8C+32	Fahrenheit	F
FORCE and PRESSURE or STRESS				
N kPa	newtons kilopascals	0.225 0.145	poundforce poundforce/square inch	lbf lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

EXECUTIVE SUMMARY

This report is the first in a series of first-level analysis reports that describe the results of HVS testing on a full-scale experiment being performed at the Richmond Field Station (RFS) to validate Caltrans overlay strategies for the rehabilitation of cracked asphalt concrete. It describes the results of the first HVS reflective cracking testing section, designated 590RF, carried out on a 90-mm full-thickness MB4-G overlay. The testing forms part of Partnered Pavement Research Center Strategic Plan Element 4.10: "Development of Improved Rehabilitation Designs for Reflective Cracking."

The objective of this project is to develop improved rehabilitation designs for reflective cracking for California. This objective will be met after completion of the following four tasks:

1. Develop improved mechanistic models of reflective cracking in California
2. Calibrate and verify these models using laboratory and HVS testing
3. Evaluate the most effective strategies for reflective cracking
4. Provide recommendations for reflective cracking strategies

This report is one of a series addressing Tasks 2 and 3. It consists of three main chapters. Chapter 2 provides information on the experiment layout, pavement design, HVS trafficking of the underlying layer, and the test details, including test duration, pavement instrumentation and monitoring methods, loading program, test section failure criteria, and the environmental conditions recorded over the duration of the test. Chapter 3 summarizes the data collected and includes discussion of air and pavement temperatures during testing (measured with thermocouples), elastic deflection (measured on the surface with the Road Surface Deflectometer and at depth with Multi-depth Deflectometers), permanent deformation (measured on the surface with the Laser Profilometer and at depth with Multi-depth Deflectometers), and visual inspections. Chapter 4 provides a summary and lists key findings.

The underlying pavement was designed following standard Caltrans procedures and it incorporates a 410-mm (16.1 in) Class 2 aggregate base on subgrade with a 90-mm (3.5 in) dense-graded asphalt concrete (DGAC) surface. Design thickness was based on a subgrade R-value of 5 and a Traffic Index of 7 (~121,000 equivalent standard axles, or ESALs). This structure was trafficked with the HVS in 2003 to induce fatigue cracking then overlaid with six different treatments to assess their ability to limit reflective cracking. The treatments included:

- Half-thickness (45 mm) MB4 gap-graded overlay (referred to as "45 mm MB4-G" in this report)
- Full-thickness (90 mm) MB4 gap-graded overlay (referred to as "90 mm MB4-G" in this report)

- Half-thickness MB4 gap-graded overlay with minimum 15 percent recycled tire rubber (referred to as "MB15-G" in this report)
- Half-thickness MAC15TR gap-graded overlay with minimum 15 percent recycled tire rubber (referred to as "MAC15-G" in this report)
- Half-thickness rubberized asphalt concrete gap-graded overlay (RAC-G), included as a control for performance comparison purposes (the section discussed in this report)
- Full-thickness (90 mm) AR4000 dense-graded overlay (AR4000-D), included as a control for performance comparison purposes

The thickness for the AR4000-D overlay was determined according to Caltrans Test Method 356. The other overlay thicknesses were either the same or half of the AR4000-D overlay thickness. Details on construction and the first phase of trafficking are provided in an earlier report.

Laboratory fatigue and shear studies are being conducted in parallel with HVS testing. Results of these studies will be detailed in separate reports. Comparison of the laboratory and test section performance, including the results of a forensic investigation to be conducted when all testing is complete, will be discussed in a second-level report once all the data from all of the studies have been collected and analyzed.

HVS trafficking on the section commenced on January 13, 2004, and was completed on June 16, 2004. During this period a total of 1,981,365 load repetitions at loads varying between 60 kN (13,500 lb) and 100 kN (22,500 lb) were applied, which equates to approximately 37 million ESALs, using the Caltrans conversion of $(\text{axle load}/18{,}000)^{4.2}$, and to a Traffic Index of 14. A mechanical breakdown on the HVS interrupted testing between March 24, 2004, and April 27, 2004, when the cumulative traffic repetitions was approximately 1,213,500. A dual tire (720 kPa [104 psi] pressure) and bidirectional loading with lateral wander was used. A temperature chamber was used to maintain the pavement temperature at 20°C±4°C (68°F±7°F).

Findings and observations based on the data collected during this HVS study include:

- No reflective cracking from the underlying severely cracked DGAC layer was observed on the MB4-G overlay after almost two million HVS repetitions. The MB4-G overlay thus appeared to successfully prevent any cracking in the underlying layer from reflecting through to the surface, despite final-to-initial deflections indicating that considerable damage had occurred in the asphalt layers under loading.

- The average maximum rut depth across the entire test section at the end of the test was 12.7 mm (0.5 in), equivalent to the Caltrans (and experiment) failure criteria of 12.5 mm (0.5 in). The maximum rut depth measured on the section was 19.0 mm (0.7 in), with a maximum rut depth of 12.5 mm reached after about 1.17 million repetitions, soon after the load increase from 60 kN (13,500 lb) to 90 kN (20,250 lb).
- Ratios of final-to-initial elastic surface deflections under a 60 kN (13,500 lb) wheel load increased by between 1.9 and 3.0 times along the length of the section. The ratios for in-depth deflections show that damage increased at all depths in the pavement structure (between 1.9 and 2.2 times) by the end of trafficking. Loss of stiffness was highest in the area of most severe cracking in the underlying DGAC layer.
- Analysis of surface profile and in-depth permanent deformation measurements indicates that most of the permanent deformation (between 67 [at MDD6] and 88 percent [at MDD10]) occurred in the asphalt-bound surfacing layers (overlay and cracked DGAC) with marginal deformation in the base layer and negligible deformation in the subgrade.

No recommendations as to the use of modified binders in overlay mixes are made at this time. These recommendations will be included in the second-level analysis report, which will be prepared and submitted on completion of all HVS and laboratory testing.

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1. INTRODUCTION

1.1. Objectives

The first-level analysis presented in this report is part of Partnered Pavement Research Center Strategic Plan Element 4.10 (PPRC SPE 4.10) being undertaken for the California Department of Transportation (Caltrans) by the University of California Pavement Research Center (UCPRC). The objective of the study is to evaluate the reflective cracking performance of asphalt binder mixes used in overlays for rehabilitating cracked asphalt concrete pavements in California. The study includes mixes modified with rubber and polymers, and it will develop tests, analysis methods, and design procedures for mitigating reflective cracking in overlays. This work is part of a larger study on modified binder (MB) mixes being carried out under the guidance of the Caltrans Pavement Standards Team (PST) (1), which includes laboratory and accelerated pavement testing using the Heavy Vehicle Simulator (carried out by the UCPRC), and the construction and monitoring of field test sections (carried out by Caltrans).

1.2. Overall Project Organization

This UCPRC project is a comprehensive study, carried out in three phases, involving the following primary elements (2):

- Phase 1
  - The construction of a test pavement and subsequent overlays;
  - Six separate Heavy Vehicle Simulator (HVS) tests to crack the pavement structure;
  - Placing of six different overlays on the cracked pavement;
- Phase 2
  - Six HVS tests to assess the susceptibility of the overlays to high-temperature rutting (Phase 2a);
  - Six HVS tests to determine the low-temperature reflective cracking performance of the overlays (Phase 2b);
  - Laboratory shear and fatigue testing of the various hot-mix asphalts (Phase 2c);
  - Falling Weight Deflectometer (FWD) testing of the test pavement before and after construction and before and after each HVS test;
  - Forensic evaluation of each HVS test section;
- Phase 3
  - Performance modeling and simulation of the various mixes using models calibrated with data from the primary elements listed above.

Phase 1

In this phase, a conventional dense-graded asphalt concrete (DGAC) test pavement was constructed at the Richmond Field Station (RFS) in the summer of 2001. The pavement was divided into six cells, and within each cell a section of the pavement was trafficked with the HVS until the pavement failed by either fatigue (2.5 m/m^2 [0.76 ft/ft^2]) or rutting (12.5 mm [0.5 in]). This period of testing began in the summer of 2001 and was concluded in the spring of 2003. In June 2003 each test cell was overlaid with either conventional DGAC or asphalt concrete with modified binders as follows:

- Full-thickness (90 mm) AR4000-D dense graded asphalt concrete overlay, included as a control for performance comparison purposes (AR-4000 is approximately equivalent to a PG64-16 performance grade binder);
- Full-thickness (90 mm) MB4-G gap-graded overlay;
- Half-thickness (45 mm) rubberized asphalt concrete gap-graded overlay (RAC-G), included as a control for performance comparison purposes;
- Half-thickness (45 mm) MB4-G gap-graded overlay;
- Half-thickness (45 mm) MB4-G gap-graded overlay with minimum 15 percent recycled tire rubber (MB15-G), and
- Half-thickness (45 mm) MAC15-G gap-graded overlay with minimum 15 percent recycled tire rubber.

The conventional overlay was designed using the current (2003) Caltrans overlay design process. The various modified overlays were either full (90 mm) or half thickness (45 mm). Mixes were designed by Caltrans. The overlays were constructed in one day.

Phase 2

Phase 2 included high-temperature rutting and low-temperature reflective cracking testing with the HVS as well as laboratory shear and fatigue testing. The rutting tests were started and completed in the fall of 2003. For these tests, the HVS was placed above a section of the underlying pavement that had not been trafficked during Phase 1. A reflective cracking test was next conducted on each overlay from the winter of 2003-2004 to the summer of 2007. For these tests, the HVS was positioned precisely on top of the sections of failed pavement from the Phase 1 HVS tests to investigate the extent and rate of crack propagation through the overlay.

In conjunction with Phase 2 HVS testing, a full suite of laboratory testing, including shear and fatigue testing, was carried out on field-mixed, field-compacted, field-mixed, laboratory-compacted, and laboratory-mixed, laboratory-compacted specimens.

Phase 3

Phase 3 entailed a second-level analysis carried out on completion of HVS and laboratory testing (the focus of this report). This included extensive analysis and characterization of the mix fatigue and mix shear data, backcalculation of the FWD data, performance modeling of each HVS test, and a detailed series of pavement simulations carried out using the combined data.

An overview of the project timeline is shown in Figure 1.1.

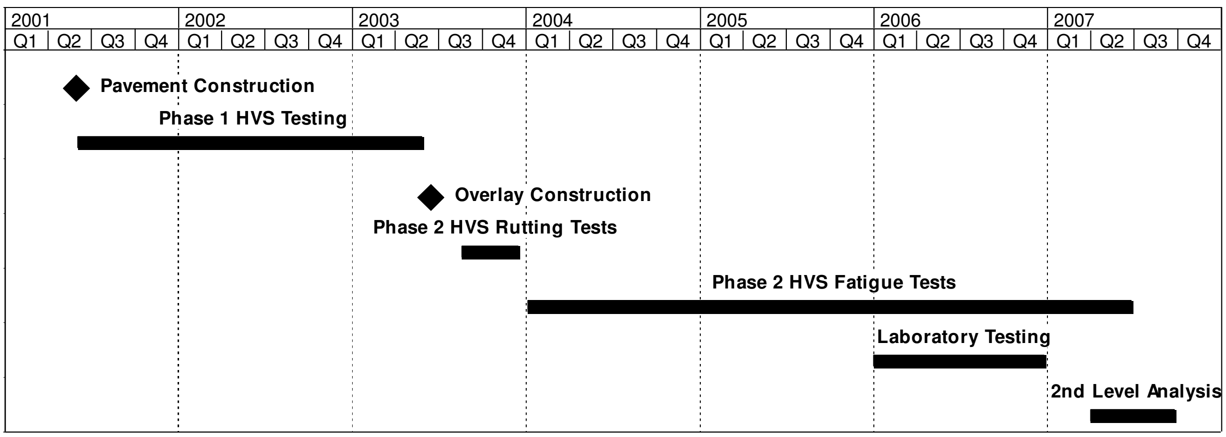

Figure 1.1: Timeline for the Reflective Cracking Study.

Reports

The reports prepared during the reflective cracking study document data from construction, HVS tests, laboratory tests, and subsequent analyses. These include a series of first- and second-level analysis reports and two summary reports. On completion of the study this suite of documents will include:

- One first-level report covering the initial pavement construction, the six initial HVS tests, and the overlay construction (Phase 1);
- One first-level report covering the six Phase 2 rutting tests (but offering no detailed explanations or conclusions on the performance of the pavements);
- Six first-level reports, each of which covers a single Phase 2 reflective cracking test (containing summaries and trends of the measured environmental conditions, pavement responses, and pavement performance but offering no detailed explanations or conclusions on the performance of the pavement);
- One first-level report covering laboratory shear testing;
- One first-level report covering laboratory fatigue testing;
- One report summarizing the HVS test section forensic investigation;
- One report summarizing the backcalculation analysis of deflection tests,

- One second-level analysis report detailing the characterization of shear and fatigue data, pavement modeling analysis, comparisons of the various overlays, and simulations using various scenarios (Phase 3), and
- One four-page summary report capturing the conclusions and one longer, more detailed summary report that covers the findings and conclusions from the research conducted by the UCPRC.

1.3. Structure and Content of This Report

This report presents the results of the HVS test on the full-thickness (90 mm) MB4 gap-graded asphalt concrete overlay (referred to as "90 mm MB4-G" in this report), designated Section 590RF, with preliminary analyses relative to observed performance and is organized as follows:

- Chapter 2 contains a description of the test program including experiment layout, loading sequence, instrumentation, and data collection.
- Chapter 3 presents a summary and discussion of the data collected during the test.
- Chapter 4 contains a summary of the results together with conclusions and observations.

1.4. Measurement Units

Metric units have always been used in the design and layout of HVS test tracks, and for all the measurements, data storage, analysis, and reporting at the eight HVS facilities worldwide (as well as all other international accelerated pavement testing facilities). Continued use of the metric system facilitates consistency in analysis, reporting, and data sharing.

In this report, metric and English units are provided in the Executive Summary, Chapters 1 and 2, and the Conclusion. In keeping with convention, only metric units are used in Chapter 3. A conversion table is provided on Page iv at the beginning of this report.

2. TEST DETAILS

2.1. Experiment Layout

Six overlays, each with a rutting test section and a reflective cracking test section, were constructed as part of the second phase of the study as follows:

1. Sections 580RF and 586RF: Half-thickness (45 mm) MB4 gap-graded overlay with minimum 15 percent recycled tire rubber (referred to as “MB15-G” in this report);
2. Sections 581RF and 587RF: Half-thickness (45 mm) rubberized asphalt concrete gap-graded (RAC-G) overlay;
3. Sections 582RF and 588RF: Full-thickness (90 mm) AR4000 dense-graded asphalt concrete overlay (designed using CTM356 and referred to as “AR4000-D” in this report);
4. Sections 583RF and 589RF: Half-thickness (45 mm) MB4 gap-graded overlay (referred to as “45 mm MB4-G” in this report);
5. Sections 584RF and 590RF: Full-thickness (90 mm) MB4 gap-graded overlay (referred to as “90 mm MB4-G” in this report), and
6. Sections 585RF and 591RF: Half-thickness (45 mm) MAC15TR gap-graded overlay with minimum 15 percent recycled tire rubber (referred to as “MAC15-G” in this report).

These sections and the corresponding Phase 1 fatigue test sections are shown in Figure 2.1. Prior to the Phase 2 reflective cracking testing, a rutting study was carried out whereby HVS loading at high temperature was applied adjacent to the reflective cracking experiments to evaluate the rutting behavior of the overlay mixes. The rutting study will be discussed in a separate report.

2.2. Test Section Layout

The test section layout for Section 590RF is shown in Figure 2.2. Station numbers refer to fixed points on the test section and are used for measurements and as a reference for discussing performance.

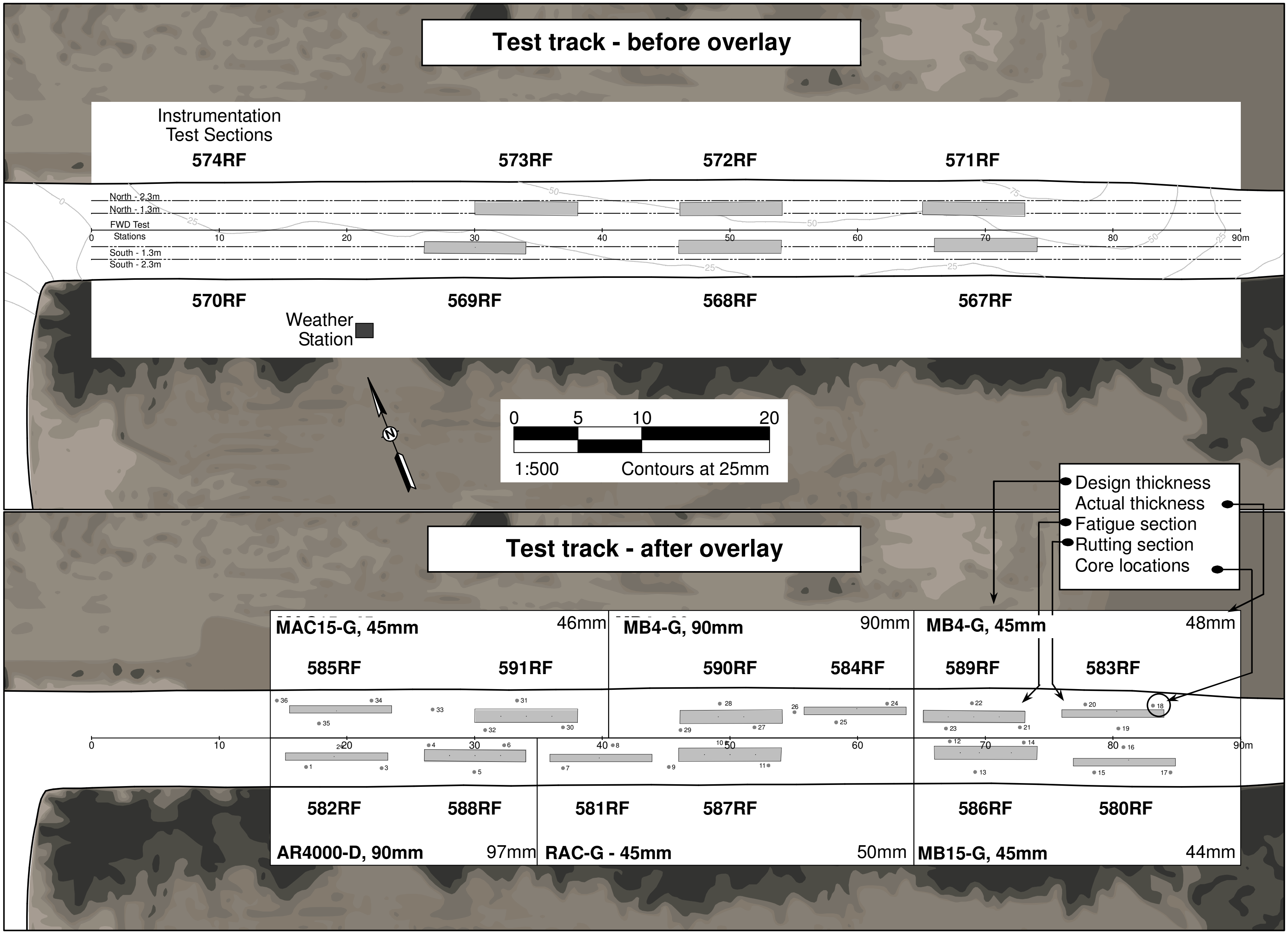

Figure 2.1: Layout of Reflective Cracking study project.

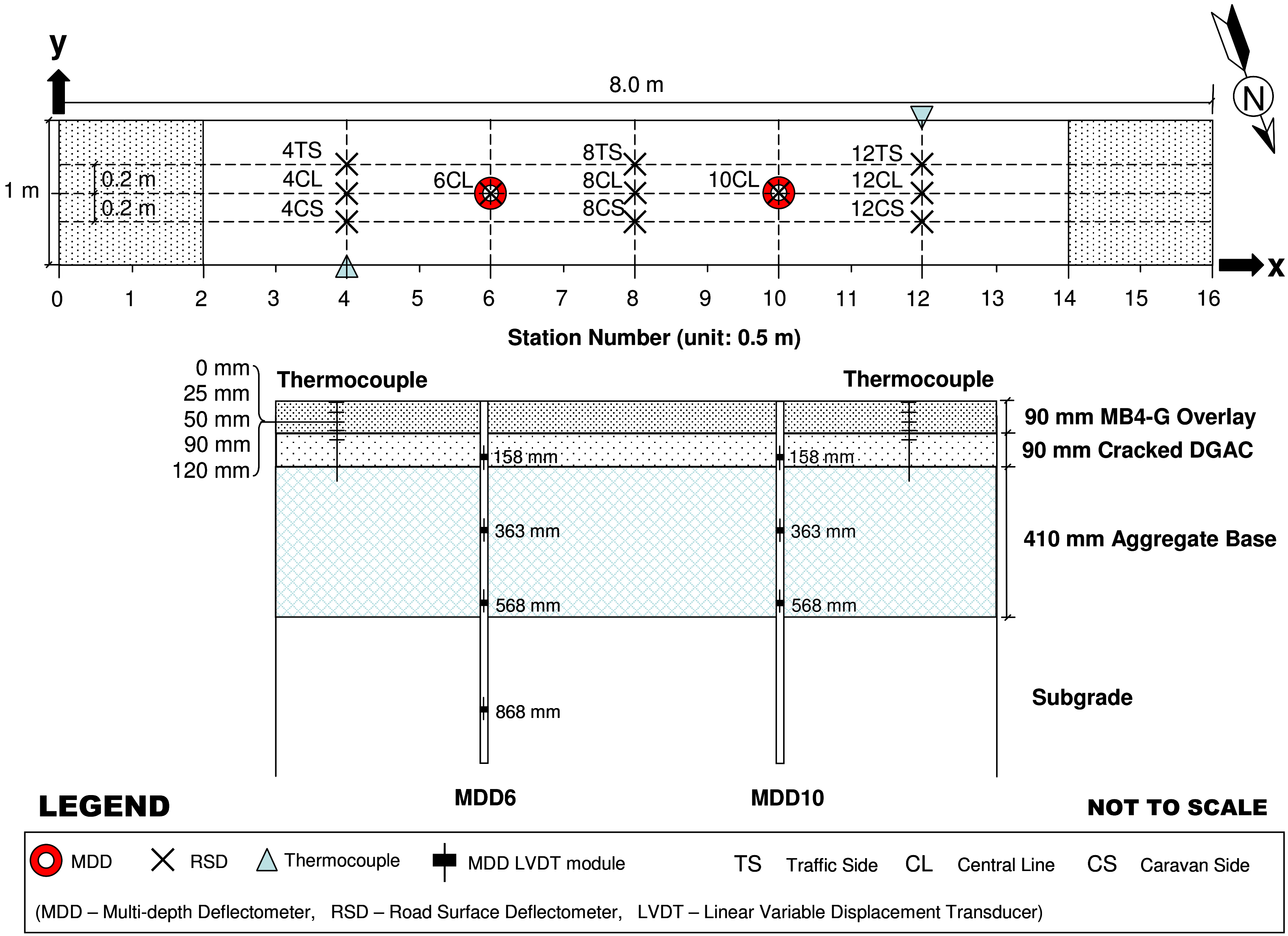

Figure 2.2: Section 590RF layout and location of instruments.

2.2.1 Pavement Instrumentation and Monitoring Methods

Measurements were taken with the following instruments:

- Road Surface Deflectometer (RSD), measuring surface deflection;
- Multi-depth Deflectometer (MDD), measuring elastic deflection and permanent deformation at different depths in the pavement;
- Laser Profilometer, measuring surface profile (at each station);
- Falling Weight Deflectometer (FWD), measuring elastic deflection before and after testing, and
- Thermocouples, measuring pavement temperature and ambient temperature.

Instrument positions are shown in Figure 2.2. Detailed descriptions of the instrumentation and measuring equipment are included in Reference 4. Intervals between measurements, in terms of load repetitions, were selected to enable adequate characterization of the pavement as damage developed.

2.3. Underlying Pavement Design

The pavement for the first phase of HVS trafficking was designed according to the Caltrans Highway Design Manual Chapter 600 using the computer program *NEWCON90.* Design thickness was based on a tested subgrade R-value of 5 and a Traffic Index of 7 (~121,000 ESALs) (2). The pavement design for the test road and the preliminary as-built pavement structure for Section 590RF (determined from cores removed from the edge of the section) are illustrated in Figure 2.3.

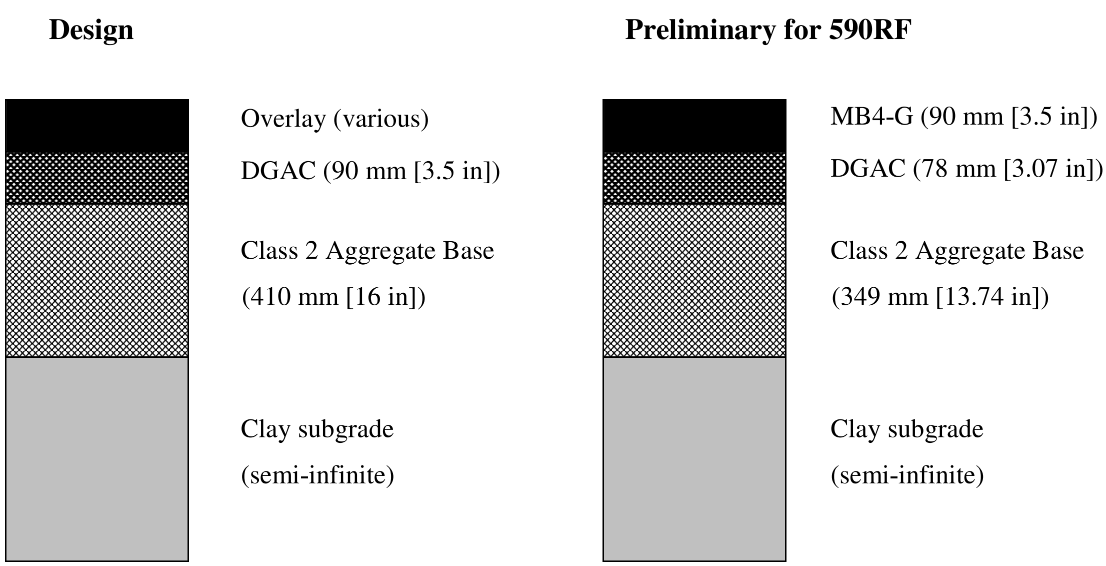

Figure 2.3: Pavement design for the Reflective Cracking Study test track.
(Design and preliminary actual for 590RF)

The existing subgrade was ripped and reworked to a depth of 200 mm (8 in) so that the optimum moisture content and the maximum wet density met the specification per Caltrans Test Method CTM 216. The average maximum wet density of the subgrade was 2,180 kg/m^3 (136 pcf). The average relative compaction of the subgrade was 97 percent (2).

The aggregate base was constructed to meet the Caltrans compaction requirements for aggregate base Class 2 using CTM 231 nuclear density testing. The maximum wet density of the base determined according to CTM 216 was 2,200 kg/m^3 (137 pcf). The average relative compaction was 98 percent.

The DGAC layer consisted of a dense-graded asphalt concrete (DGAC) with AR-4000 binder and aggregate gradation limits following Caltrans 19-mm (0.75 in) maximum size coarse gradation (2). The target asphalt content was 5.0 percent by mass of aggregate, while actual contents varied between 4.34 and 5.69 percent. Nuclear density measurements and extracted cores were used to determine a preliminary as-built mean air-void content of 9.1 percent with a standard deviation of 1.8 percent. The air-void content after traffic compaction and additional air-void contents from cores taken outside the trafficked area will be determined on completion of trafficking of all sections and will be reported in the second-level analysis report.

2.4. Summary of Testing on the Underlying Layer

Phase 1 trafficking of the underlying Section 572RF took place between January 23, 2003, and March 12, 2003, during which 537,074 repetitions were applied. Figure 2.4 presents the final cracking pattern after testing and shows significant cracking.

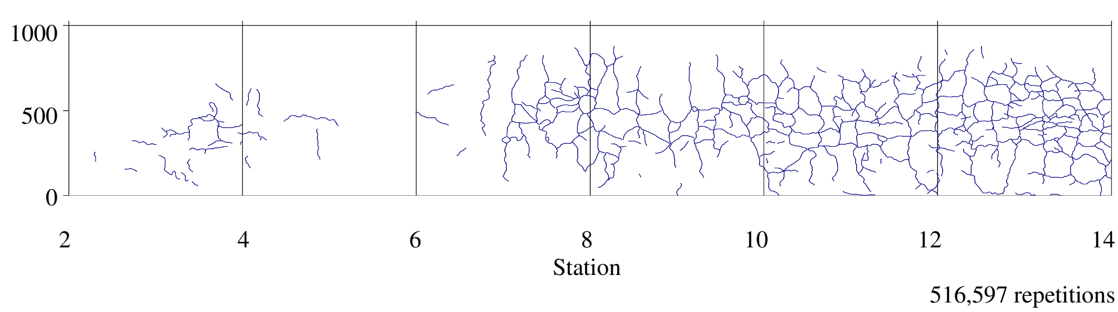

Figure 2.4: Cracking pattern on Section 572RF after Phase 1 HVS testing.

2.5. Reflective Cracking Section Design

Section 590RF was located on the 90-mm MB4 gap-graded overlay constructed precisely on top of Section 572RF. Section 572RF had significant alligator cracking on one side but was relatively intact on the remaining half (Figure 2.4). The overlay thickness for the experiment was determined according to Caltrans Test Method CTM 356 using Falling Weight Deflectometer data from the Phase 1 experiment. The actual layer thickness of Section 590RF was measured from cores extracted from the edge of the test section and from Dynamic Cone Penetrometer (DCP) tests taken outside the trafficked area. The measured average thicknesses for the section were (see Figure 2.3):

- MB4-G Overlay: 90 mm (min 82 mm; max 101 mm; standard deviation, 7.8 mm)
  [3.5 in (min 3.2 in; max 4.0 in; standard deviation, 0.3 in)]
- Cracked DGAC layer: 84 mm (min 73 mm; max 93 mm; standard deviation, 7.5 mm)
  [3.3 in (min 2.9 in; max 3.7 in; standard deviation, 0.3 in)]
- Aggregate base: 398 mm (15.7 in)

Exact layer thicknesses will be determined from measurements in test pits after HVS testing has been completed on all sections.

Laboratory testing was carried out by Caltrans and UCPRC on samples collected during construction to determine actual binder properties, binder content, aggregate gradation, and air-void content. The MB4 binder met the Caltrans MB4 modified binder specification, based on testing performed by Caltrans. The ignition-extracted binder content, corrected for aggregate ignition, showed an average value of 7.77 percent, somewhat higher than the design binder content of 7.2 percent. It is not clear whether this is a function of the test or contractor error. The aggregate gradation met Caltrans specifications for a 19.0 mm (3/4 in) maximum size gap gradation, with material passing the 6.35 mm (1/4 in) and 9.5 mm (3/8 in) sieves on the limit for course gradation. Gradation is illustrated in Figure 2.5. The preliminary as-built air-void content was 6.5 percent with a standard deviation of 0.6 percent, based on cores taken outside of the HVS sections. Final air-void contents will be determined from trenching and coring to be performed after trafficking of all sections.

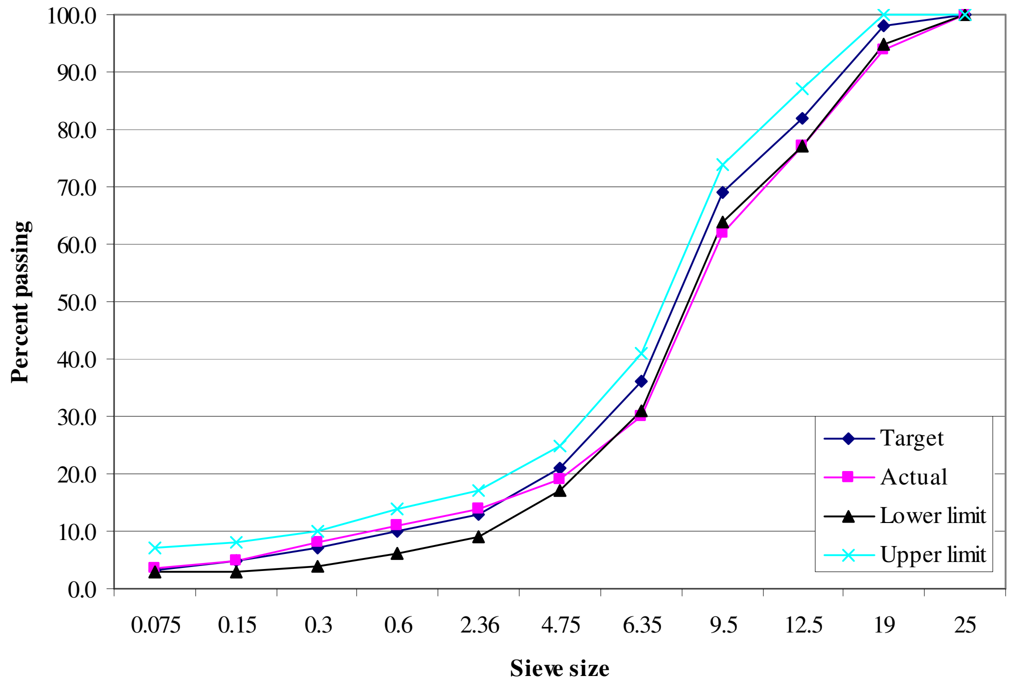

Figure 2.5: Actual vs. target gradation for MB4-G overlay.

2.6. Summary of Testing on Reflective Cracking Section

2.6.1 Test Section Failure Criteria

Failure criteria for analyses were set at:

- Cracking density of 2.5 m/m^2 (0.76 ft/ft^2) or more, and/or
- Average maximum surface rut depth of 12.5 mm (0.5 in) or more.

2.6.2 Environmental Conditions

For the first one million repetitions, the pavement surface temperature was maintained at 20°C±4°C (68°F±7°F) to minimize rutting in the asphalt concrete and to promote fatigue damage. Thereafter, the pavement surface temperature was reduced to 15°C±4°C (59°F±7°F) to further accelerate fatigue damage. A temperature control chamber (5) was used to maintain the test temperatures.

The pavement surface received no direct rainfall as it was protected by the temperature control chamber. The section was tested predominantly during the wet season (January to June) and hence water could have infiltrated the pavement from the side drains and through the raised groundwater table.

2.6.3 Test Duration

HVS trafficking on Section 590RF was initiated on January 13, 2004, and completed on June 16, 2004, after the application of just under two million (1,981,365) load repetitions. Testing was interrupted during a breakdown between March 24 and April 27, 2004, when the cumulative traffic repetitions were approximately 1,213,500.

2.6.4 Loading Program

The HVS loading program is summarized in Table 2.1.

Table 2.1: Summary of HVS Loading Program

Start Date	Start Repetition	Wheel Load (kN) - [lb]		Wheel	Tire Pressure (kPa) - [psi]	Direction
		Planned	Actual			
1/13/04	0	40 - [9,000]	60	Dual	720 - [104]	Bi
3/15/04*	1,071,004	60 - [13,500]	90	Dual	720 - [104]	Bi
5/10/04	1,439,898	80 - [18,000]	80	Dual	720 - [104]	Bi
5/21/04	1,629,058	100 - [22,500]	100	Dual	720 - [104]	Bi
* Testing was interrupted during a breakdown between 03/24/04 and 04/27/04.						

The loading program followed differs from the original test plan due to an incorrect hydraulic control system setup on loads less than 65 kN (14,600 lb) in the Phase 1 experiment. The loading pattern from the Phase 1 experiment was thus retained to facilitate comparisons of performance between all tests in the Reflective Cracking Study. Testing was undertaken with a dual-wheel configuration, using radial truck tires (Goodyear G159 - 11R22.5 - steel belt radial) inflated to a pressure of 720 kPa (104 psi), in a bidirectional loading mode. Lateral wander over the one-meter (39.4 in) width of the test section was programmed to simulate traffic wander on a typical highway lane.

Cumulative traffic applications and the loading history are shown in Figure 2.6. As this was the first HVS test on the overlays, the 60 kN (13,500 lb) loading pattern was retained for an extended period to prevent excessive initial deformation (rutting) of the newly constructed overlay. A total of 1,981,365 load repetitions were applied during this period consisting of:

- 1,071,004 repetitions of a 60 kN (13,500 lb) load,
- 368,894 repetitions of a 90 kN (20,250 lb) load,
- 189,160 repetitions of an 80 kN (18,000 lb) load, and
- 352,307 repetitions of a 100 kN (22,500 lb) load.

This loading equates to approximately 37 million equivalent standard axles (ESALs), using the Caltrans conversion of $(\text{axle load}/18000)^{4.2}$, which in turn equates to a Traffic Index of 14.

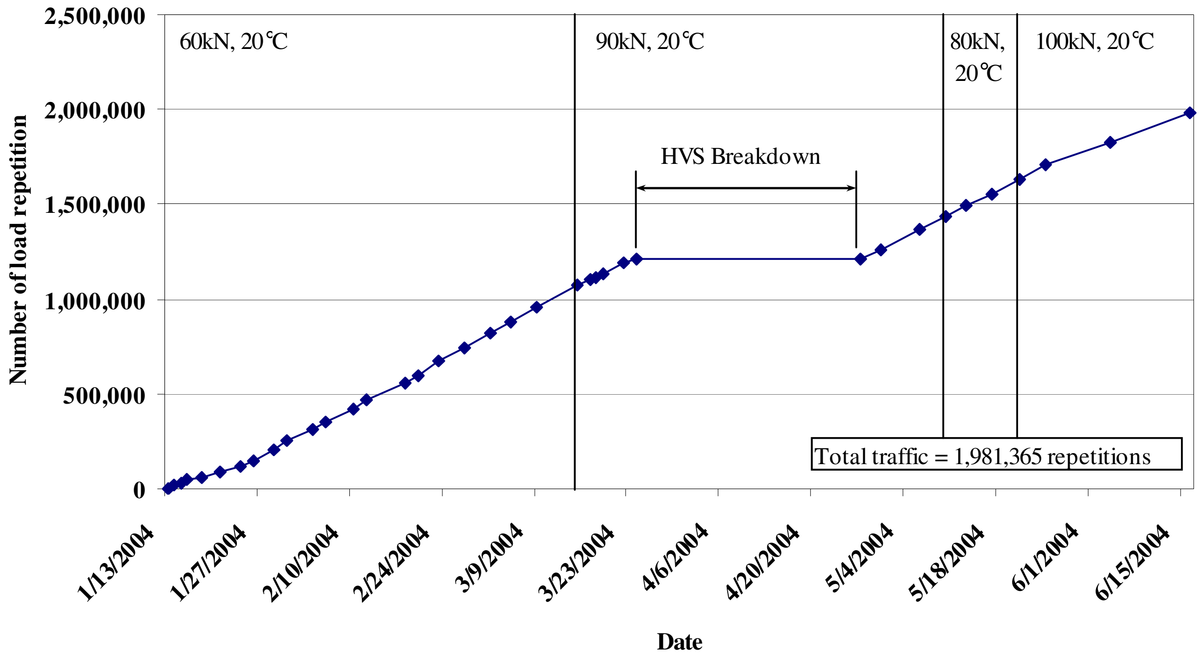

Figure 2.6: Cumulative traffic applications and loading history.

2.6.5 Measurement Summary

Table 2.2 lists the reading schedule of MDD and RSD measurements at various wheel loads. Missing data is discussed in Chapter 3. Surface deflection measurements with the RSD were obtained at the reference points along the centerline (CL) of the section and at locations 200 mm (8.0 in) on either side of the centerline (traffic and caravan side), as shown in Figure 2.2. MDD and RSD measurements were taken with a 60 kN (13,500 lb) load throughout the test as well as with the load being applied at the time of measurement (i.e., 80 kN [18,000 lb], 90 kN [22,500 lb], or 100 kN [22,500 lb]). The figures in Chapter 3 only show the measurements taken with the 60 kN (13,500 lb) load.

Measurements of surface rut depth taken by transverse scans with the Laser Profilometer were obtained at each station (Figure 2.2) on the same schedule as that of the MDD and RSD. The following rut parameters, which are discussed in more detail in Chapter 3, were determined from these measurements:

- Location and magnitude of the maximum rut depth,
- Average rut depth for the entire test section, and
- Rate of rut development.

Table 2.2: Summary of MDD and RSD Measurements

Date	Repetitions	MDD6				MDD10				RSD Center line[1]				RSD Sides[2]			
		60*	90*	80*	100*	60*	90*	80*	100*	60*	90*	80*	100*	60*	90*	80*	100*
1/13/04	0	✓				✓				✓				✓			
1/14/04	15,000	✓				✓				✓							
1/15/04	30,000	✓				✓				✓							
1/16/04	45,000	✓				✓				✓							
1/18/04	60,250	✓				✓				✓							
1/21/04	90,000	✓				×				✓							
1/24/04	120,000	✓				×				✓				✓			
1/26/04	150,000	✓				×				✓							
1/29/04	208,829	✓				×				✓				✓			
1/31/04	250,000	✓				×				✓							
2/04/04	310,000	✓				×				✓							
2/06/04	349,022	✓				×				✓							
2/10/04	420,000	✓				×				✓							
2/12/04	472,804	✓				✓				✓				✓			
2/18/04	560,000	✓				✓				✓							
2/20/04	600,000	✓				×				✓							
2/23/04	670,000	✓				✓				✓				✓			
2/27/04	740,000	✓				✓				✓							
3/02/04	822,810	✓				✓				✓				✓			
3/05/04	882,437	✓				×				✓							
3/09/04	955,918	✓				✓				✓							
3/15/04	1,071,004	✓	✓			✓	✓			×	✓			✓	✓	✓	✓
3/17/04	1,101,000	✓	✓			✓	✓			✓	✓						
3/18/04	1,116,000	✓	✓			✓	✓			✓	✓						
3/19/04	1,131,000	×	×			×	✓			✓	✓						
3/22/04	1,189,500	×	×			✓	✓			✓	✓						
4/30/04	1,262,979	✓	✓			✓	✓			✓	✓						
5/06/04	1,370,344	×	×			×	✓			✓	✓						
5/10/04	1,439,898	×	×	×		✓	×	✓		✓	×	×		✓	✓	✓	✓
5/13/04	1,490,006	✓		✓		✓		✓		×		✓					
5/17/04	1,556,385	✓		✓		✓		✓		✓		✓					
5/21/04	1,629,058	✓	✓	✓	✓	×		✓	✓	✓		✓	✓	✓	✓	✓	✓
5/25/04	1,706,884	✓			×	✓			✓	✓			✓				
6/04/04	1,827,158	×			✓	✓			✓	✓			✓				
6/16/04	1,981,365	×		×	✓	✓		×	✓	×			✓	✓	✓	✓	✓

* Wheel load in kN

[1] Measurements at points 4, 6, 8, 10, and 12

[2] Measurements at points 4, 8, and 12

✓ Data collected; × Suspect data, not used; shaded: No data collection scheduled

Falling Weight Deflectometer (FWD) measurements were taken before, during, and after testing at the center of and on the outside of the trafficked area. Measurements are not usually taken during HVS testing as the FWD does not fit under the HVS. However, in March the HVS had to be removed from the section for repairs, which enabled deflections to be determined. A summary of the measurement schedule is provided in Table 2.3.

Table 2.3: Summary of FWD Measurements

Date	Time	Location	Interval (m) - [ft]
1/05/04	08:54	Center	0.3 - [1.0]
3/22/04	11:17	Center	0.3 - [1.0]
3/22/04	13:14	Side	0.3 - [1.0]
6/16/04	16:27	Center	0.9 - [3.0]
6/16/04	17:00	Side	0.9 - [3.0]
6/21/04	09:36	Center	0.9 - [3.0]
6/21/04	10:42	Side	0.9 - [3.0]

Pavement temperature measurements were derived from thermocouples (depths and surface locations shown in Figure 2.2) at one-hour intervals during HVS operation. Air temperatures were measured in a weather station next to the test section and recorded at the same intervals as the thermocouples.

Crack development was monitored using visual inspection of the road surface and photographs.

3. DATA SUMMARY

This chapter provides a summary of the data collected from Section 590RF, and a brief discussion of the first-level analysis. Interpretation of the data in terms of pavement performance will be discussed in a separate second-level analysis report.

3.1. Temperatures

Pavement temperatures were controlled using the temperature control chamber. Both air (inside and outside the temperature box) and pavement temperatures were monitored and recorded hourly during the entire loading period. Figure 3.1 illustrates the frequencies of recorded temperatures at each hour during the testing period. Hourly temperatures were collected for approximately 70 percent of the test period. No temperatures were recorded during the period of breakdown. As seen in the figure, the hour counts from 10:00 to 15:00 hours (on a 24-hour clock) are relatively low, this being the period when measurements were taken. As a consequence, temperature interpolation/extrapolation will be necessary when interpreting the backcalculation results from the MDD and RSD measurements (second-level analysis). In assessing fatigue performance, the temperature at the bottom of the asphalt concrete and the temperature gradient are the two important controlling temperature parameters used to evaluate the stiffness of the asphalt concrete and to compute the maximum tensile strain as accurately as possible.

3.1.1 Air Temperature in the Temperature Control Unit

Air temperatures inside the temperature control chamber ranged from 15°C to 25°C during the testing period. Temperatures were adjusted to maintain a pavement temperature at 50 mm depth of 20°C±4°C; that temperature range expected to promote fatigue damage leading to reflective cracking while minimizing rutting of the asphalt concrete layer. The air temperature distribution in the control chamber had a mean of 20.7°C with a standard deviation of 2.8°C. The daily average air temperatures recorded in the temperature control unit, calculated from the hourly temperatures recorded during HVS operation, are shown in Figure 3.2. Vertical error bars on each point on the graph show daily temperature range.

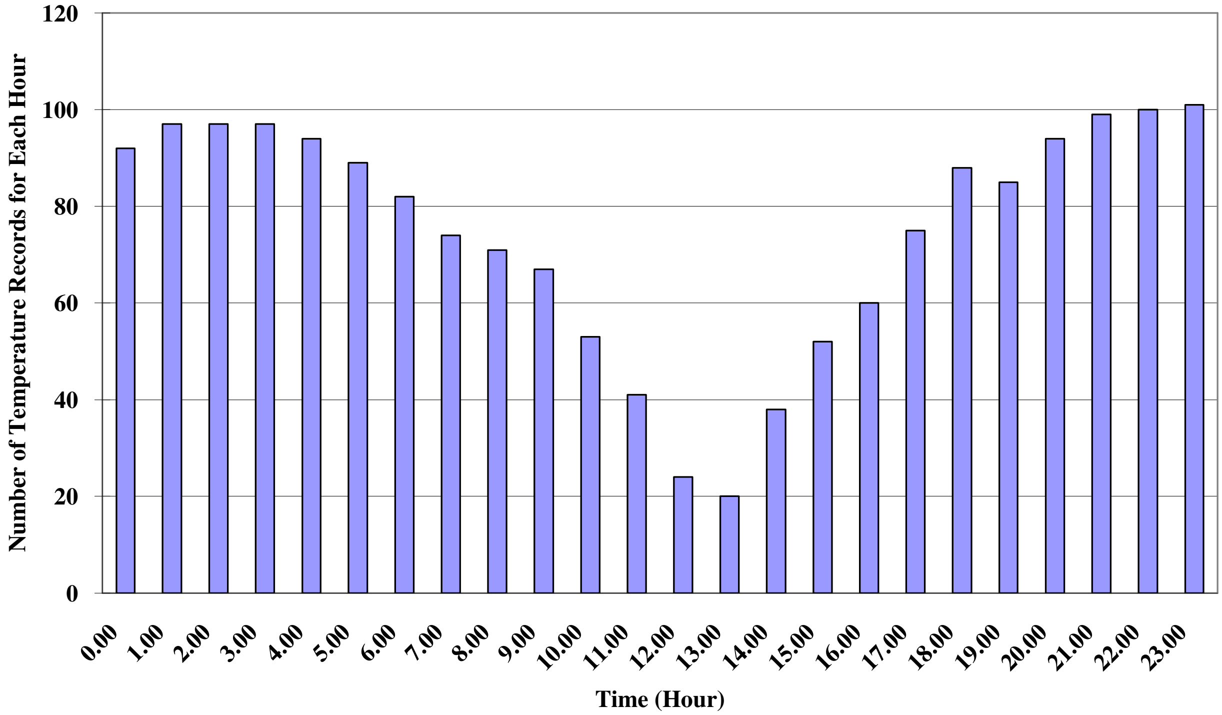

Figure 3.1: Frequencies of recorded temperatures.

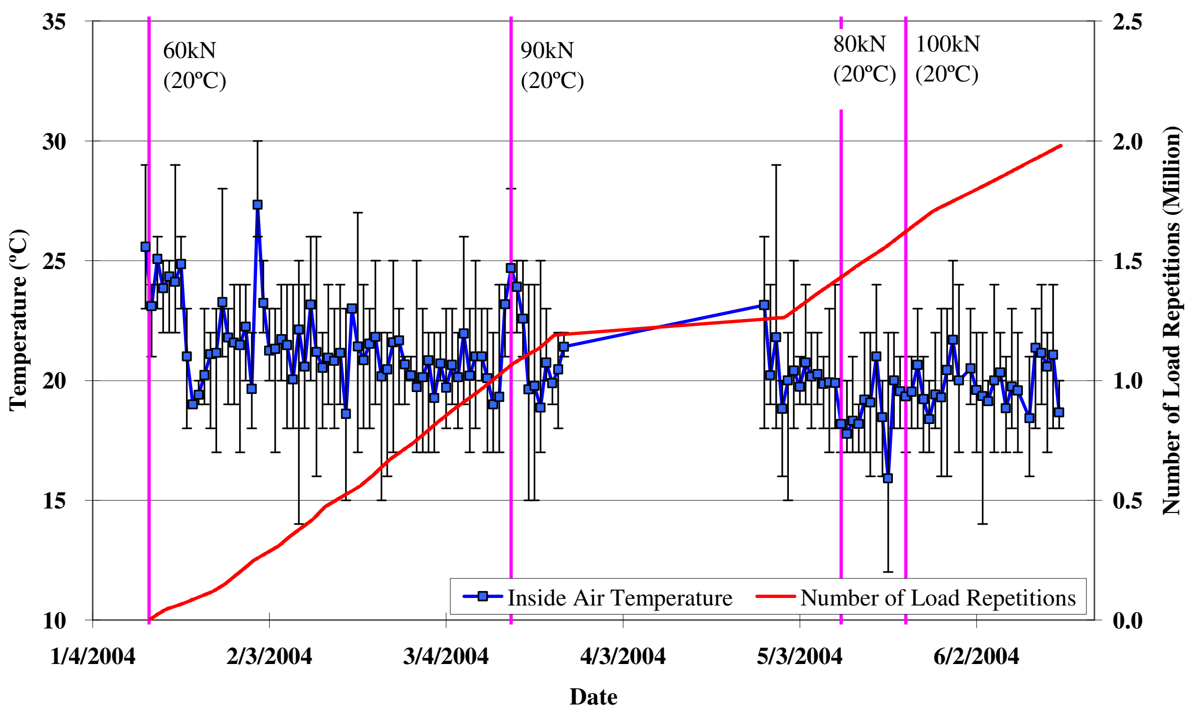

Figure 3.2: Daily average air temperatures inside the temperature control chamber.

3.1.2 Outside Air Temperatures

Outside air temperatures ranged from 3.0°C to 31.0°C with an average of 12.9 C and are summarized in Figure 3.3. Vertical error bars on each point of the graph show daily temperature range.

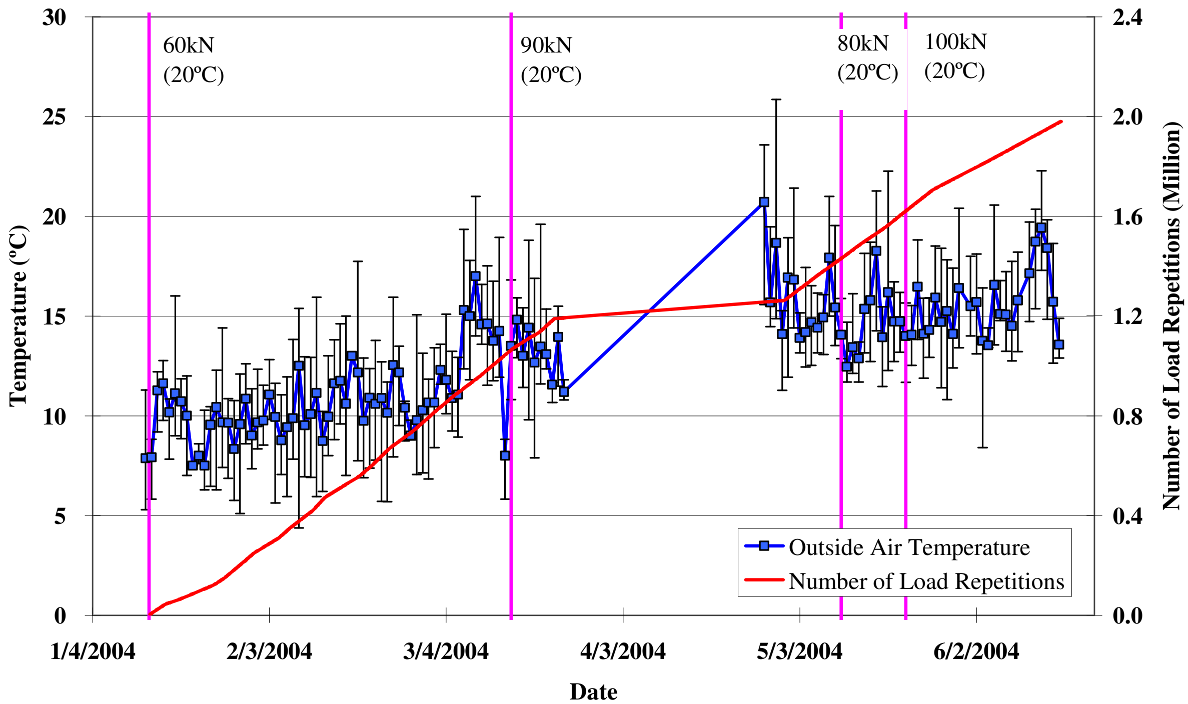

Figure 3.3: Daily average air temperatures outside the temperature control chamber.

3.1.3 Temperature in the Asphalt Concrete Layer

Daily averages of the surface and in-depth pavement temperatures are listed in Table 3.1 and shown in Figure 3.4. The daily average temperatures at various depths ranged from 19.5°C to 22.0°C with a mean pavement temperature of 20.7°C. The pavement temperatures show a gradually decreasing pattern from the surface down to 100 mm in January (cold season), a steady pattern in March, and a gradually increasing pattern from the surface down to 100 mm in June. The standard deviations in Table 3.1 show that pavement temperatures fluctuated less than the air temperatures.

Table 3.1: Temperature Summary for Air and Pavement

Location	Mean Temperature (°C)	Standard Deviation (°C)
Air (inside the chamber)	20.75	2.82
Surface	20.67	1.25
- 25 mm below surface	20.73	1.07
- 50 mm below surface	20.73	1.10
- 75 mm below surface	20.68	1.09
- 100 mm below surface	20.65	1.12

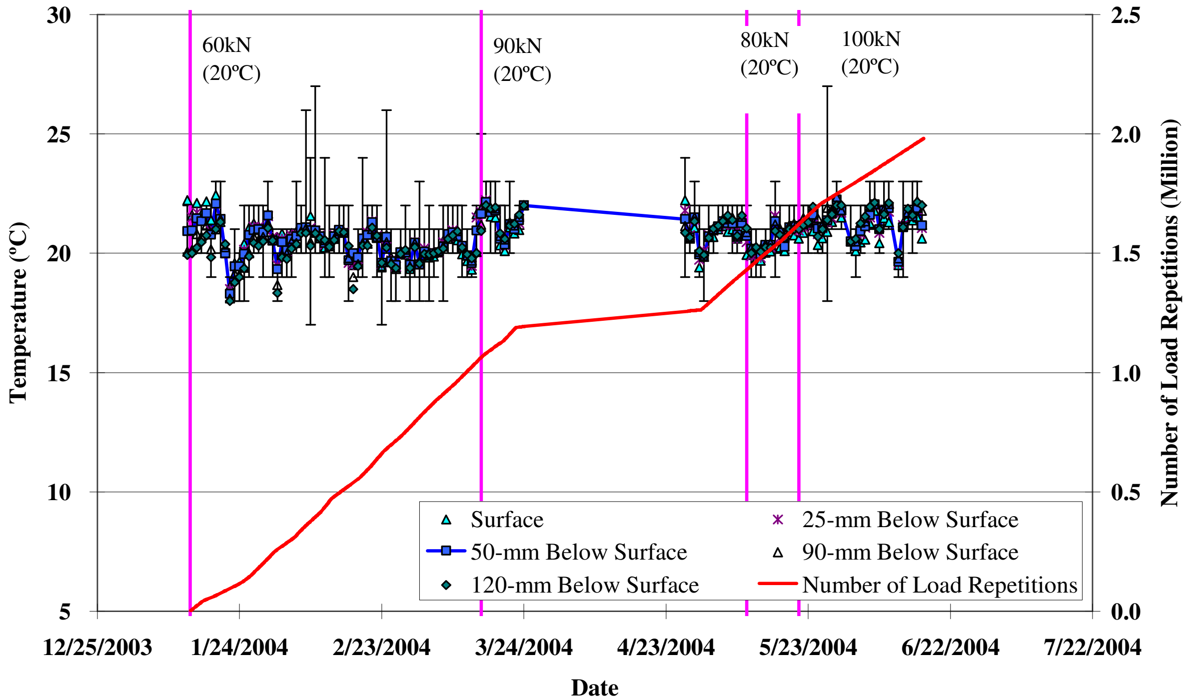

Figure 3.4: Daily average temperatures at pavement surface and various depths.

3.2. Rainfall

Figure 3.5 shows the monthly rainfall data from May 2003 to September 2004 as adapted from the National Climatic Data Center (NCDC), National Oceanic and Atmospheric Administration (NOAA) Satellite and Information Service, and the monthly rainfall data recorded at the weather station at the Richmond Field Station HVS site.

3.3. Elastic Deflection

Elastic (recoverable) deflections provide an indication of the overall stiffness of the pavement structure and, therefore, a measure of the load-carrying capacity. As the stiffness of a pavement structure deteriorates, its ability to resist the deformation/deflection caused by a given load and tire pressure decreases. During HVS testing elastic deflections are measured with two instruments: the RSD to measure surface deflections and the MDD to measure in-depth deflections. MDD modules could not be installed at the surface (0 mm) due to the limited thickness of the overlay and thus it is not possible to directly compare surface deflections between the two instruments. In addition to RSD and MDD measurements, FWD measurements were taken before, during and after HVS trafficking to evaluate the initial and final conditions of the pavement.

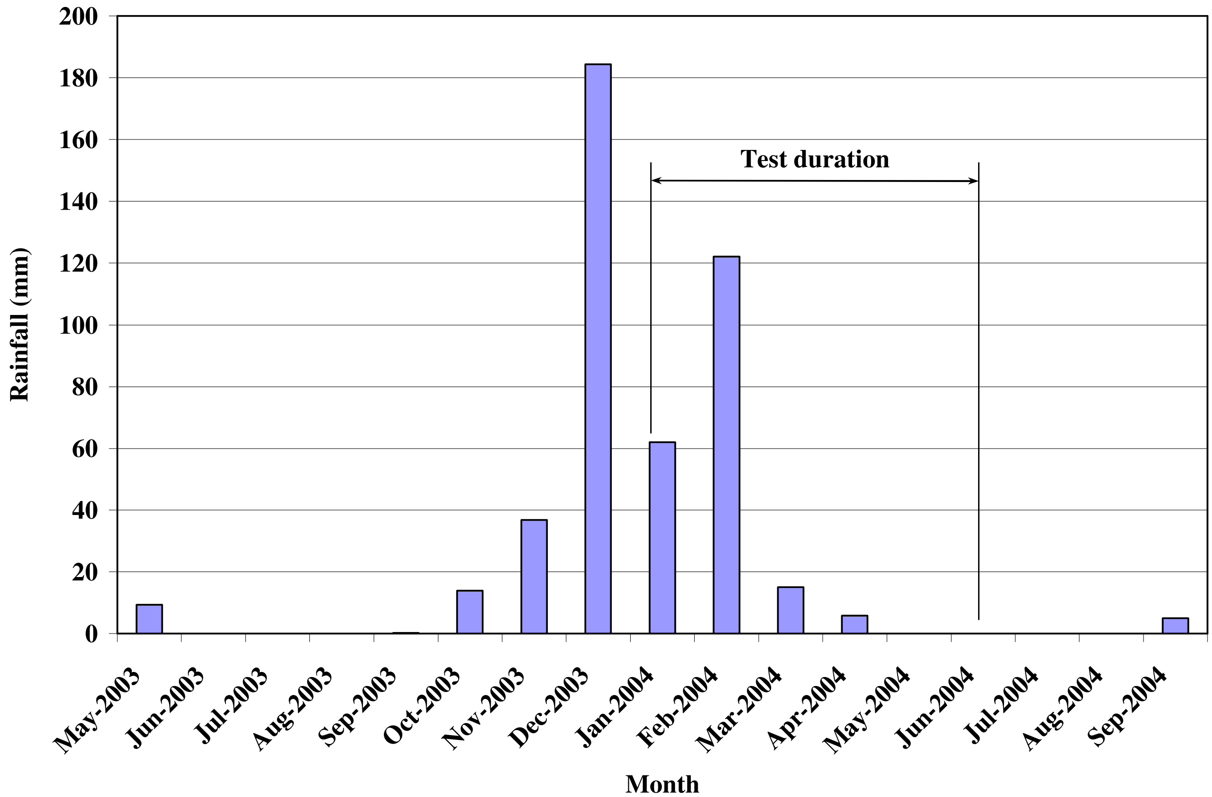

Figure 3.5: Monthly rainfall for Richmond Field Station HVS site.

3.3.1 Surface Elastic Deflection from RSD

In this section of the report, surface deflections as measured by the RSD under a load of 60 kN are summarized (*Note:* although the load is increased during the test program, deflection measurements are always taken with a 60 kN load.).

Table 3.2 compares the average 60 kN RSD deflections for centerline locations 4CL, 6CL, 8CL, 10CL, and 12CL before and on completion of testing. The relatively high standard deviation for the average deflection after trafficking is attributed to variability in the cracking of the underlying DGAC layer, which is discussed below.

Table 3.2: Average 60 kN RSD Centerline Deflections Before and After Testing

Position	Parameter	Deflection (microns)		
		Before Trafficking	After Trafficking	Ratio of Final/Initial
All	Average	493	1,228	2.49
	Std. Deviation	60	278	-
4CL	Average	395	966	2.45
6CL	Average	457	898	1.97
8CL	Average	538	1,304	2.42
10CL	Average	537	1,603	2.99
12CL	Average	537	1,368	2.55

At the start of the test, initial deflections were all within 0.15 mm of each other, with higher deflections (i.e., weaker pavement) recorded at the end overlying the significantly cracked DGAC (Figure 3.6). During the course of the test, substantial damage occurred on the overlay over the entire section under HVS trafficking. This is confirmed by the ratio of final-to-initial deflections for all RSD locations, which show that surface deflections increase by between two and three times along the length of the test section, indicating significant damage in the pavement structure in terms of loss of stiffness. Although the ratio of final-to-initial deflections was fairly consistent across the section, when the results are considered in conjunction with Figure 2.4, lower deflections (Stations 4CL and 6CL) were recorded at the end of the section that had the least amount of cracking in the underlying pavement (stiffer pavement), while those with the highest deflections (Stations 8CL, 10CL, and 12CL) were over the severely cracked area (weaker pavement).

Deflections and damage rates both increased with increase in load. Figures 3.6 to 3.10 compare the deflection bowls at the same locations at test start, at load change intervals, and at test completion. The same scale is used on all figures, and the increasing deflection over time and with load is clear. The higher deflections at Stations 8CL, 10CL, and 12CL over the cracked underlying DGAC, are clearly visible.

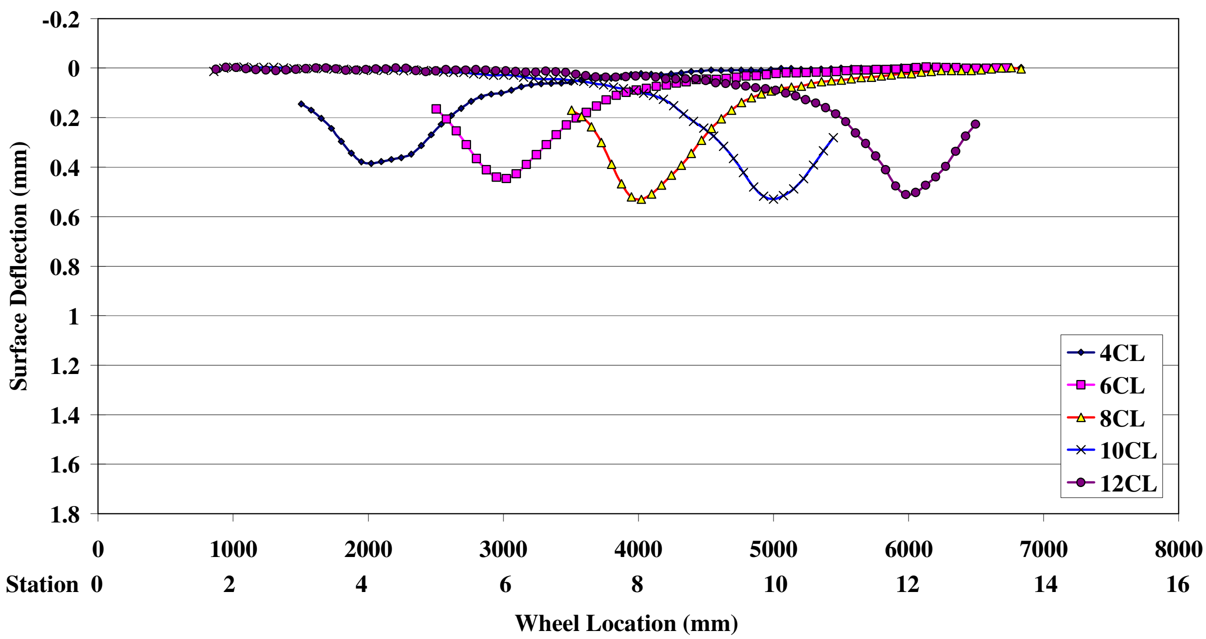

Figure 3.6: RSD deflections at CL locations with 60 kN test load at test start.

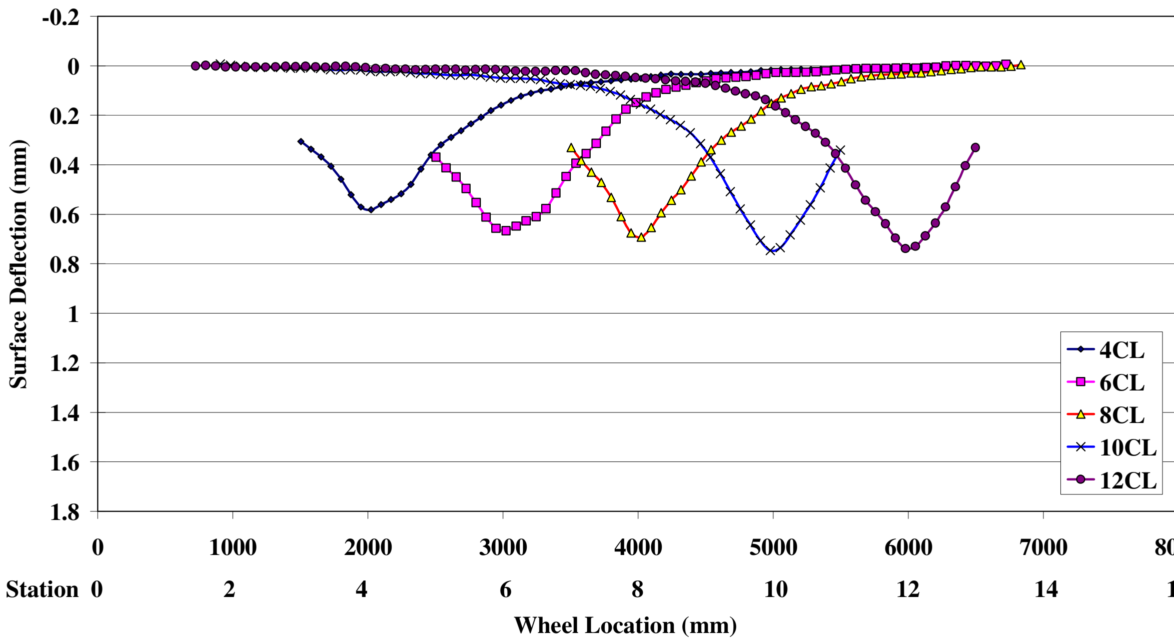

Figure 3.7: RSD deflections at CL locations with 60 kN test load after 1,071,004 repetitions. (60 kN load change)

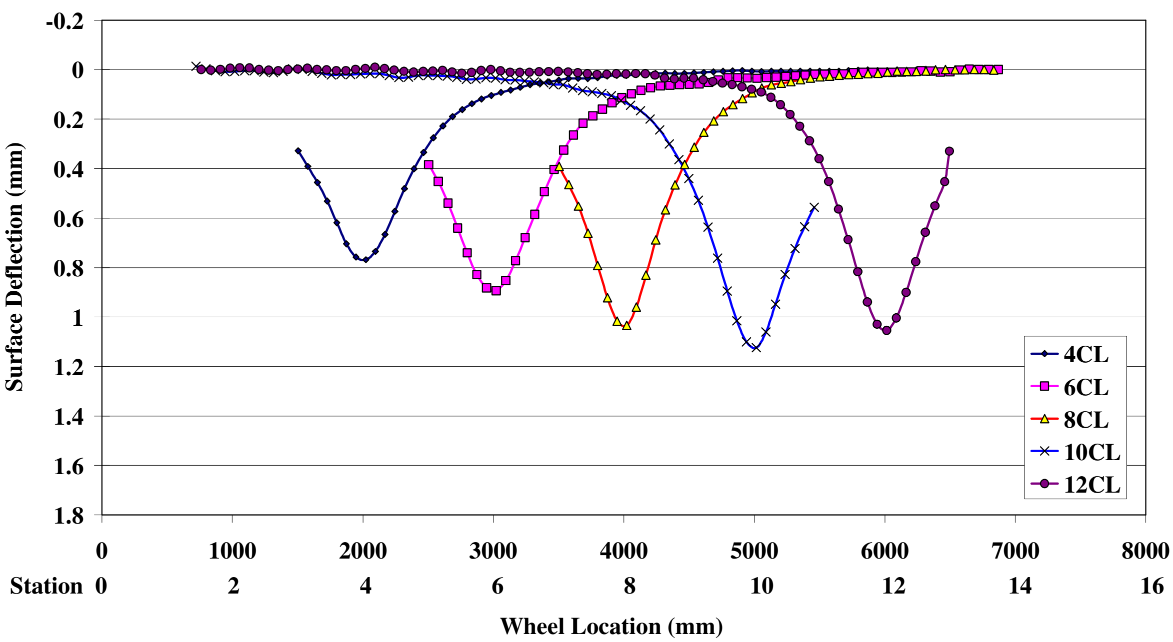

Figure 3.8: RSD deflections at CL locations with 60 kN test load after 1,439,898 repetitions. (80 kN load change)

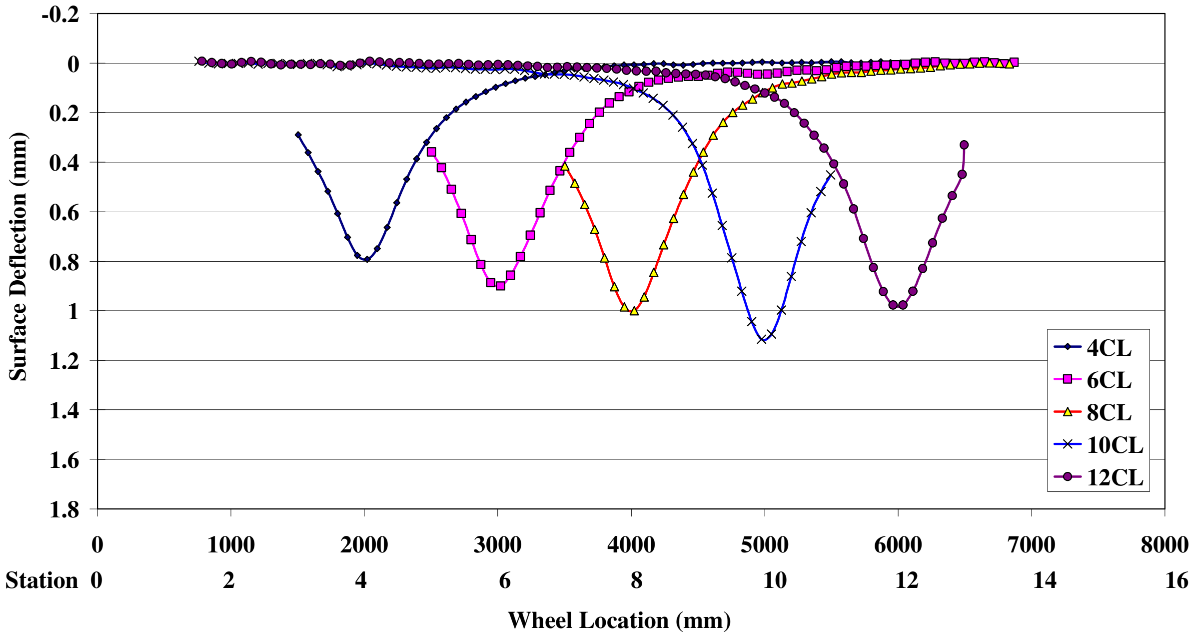

Figure 3.9: RSD deflections at CL locations with 60 kN test load after 1,629,058 repetitions. (100 kN load change)

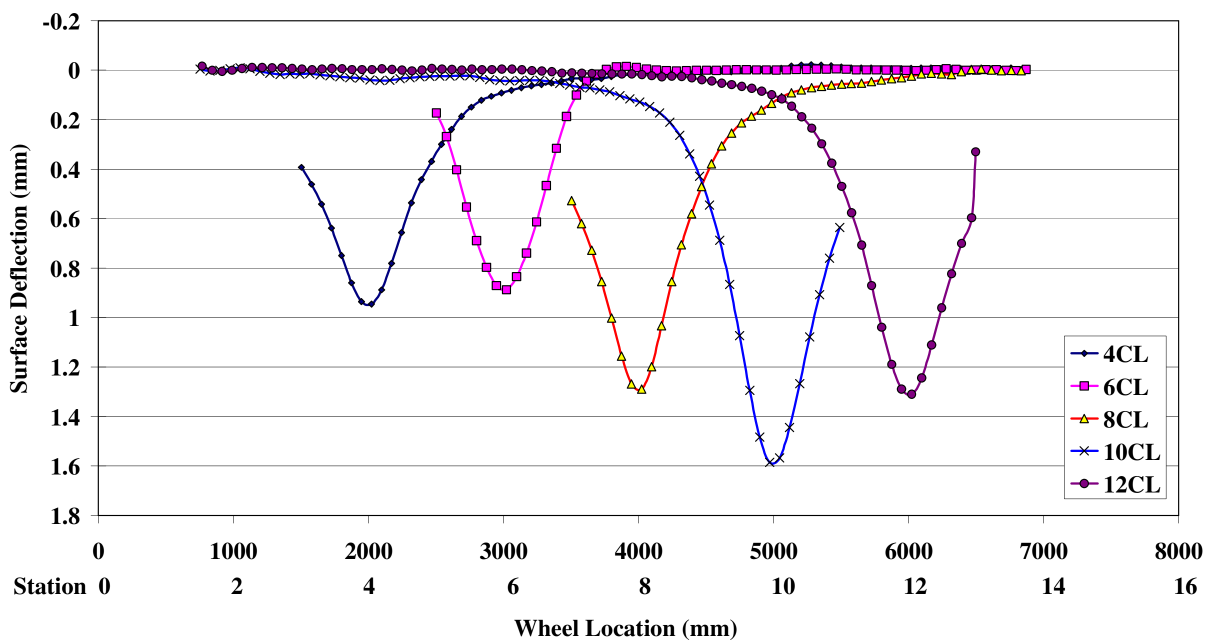

Figure 3.10: RSD deflections at CL locations with 60 kN test load at test completion.

The average 60 kN RSD deflections at centerline and side locations (200 mm from centerline within the trafficked area) are illustrated in Figure 3.11. These deflections are all within 0.2 mm of each other, although as expected the side deflections are less than those of the centerline. Deflections increased sharply after the 90 kN load change, remained relatively constant during the 80 kN loading phase, and then continued to increase again after the 100 kN load change. These results indicate that damage was somewhat greater in the vicinity of the centerline compared to the area away from the centerline where fewer repetitions were applied by the programmed wander of the HVS trafficking pattern.

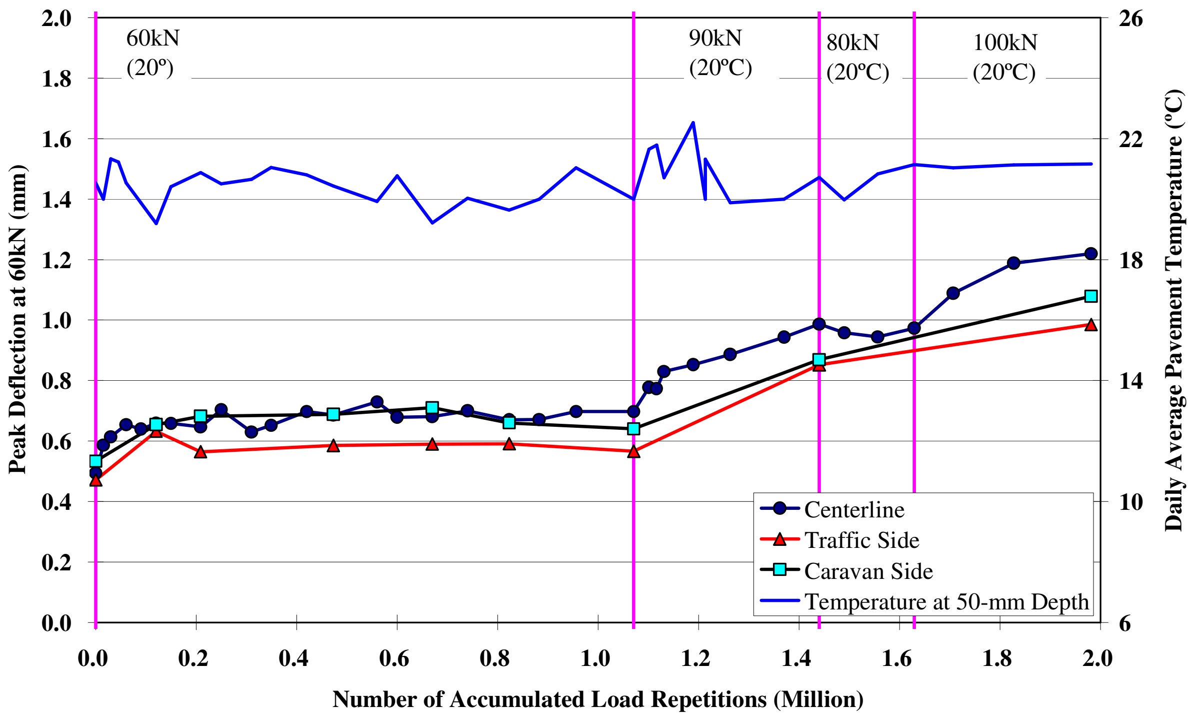

Figure 3.11: Average RSD surface deflections with 60 kN test load (centerline and sides).

Figure 3.12 shows the average 60 kN deflection at centerline as well as the averages for measurements taken at the end of the section with significantly cracked DGAC underneath (Stations 10CL and 12CL) and the end with less cracking (Stations 4CL, 6CL, and 8CL). The difference in deflections is evident between the two ends. In Figures 3.11 and 3.12, some sensitivity of RSD deflection to temperature is apparent, for example at approximately 180,000, 250,000, and 320,000 repetitions, and at the 90 kN load change. The influence of temperature on deflection will be discussed in the second-level analysis report. The sensitivity of the RSD to a load reduction is evident during the 80 kN loading period.

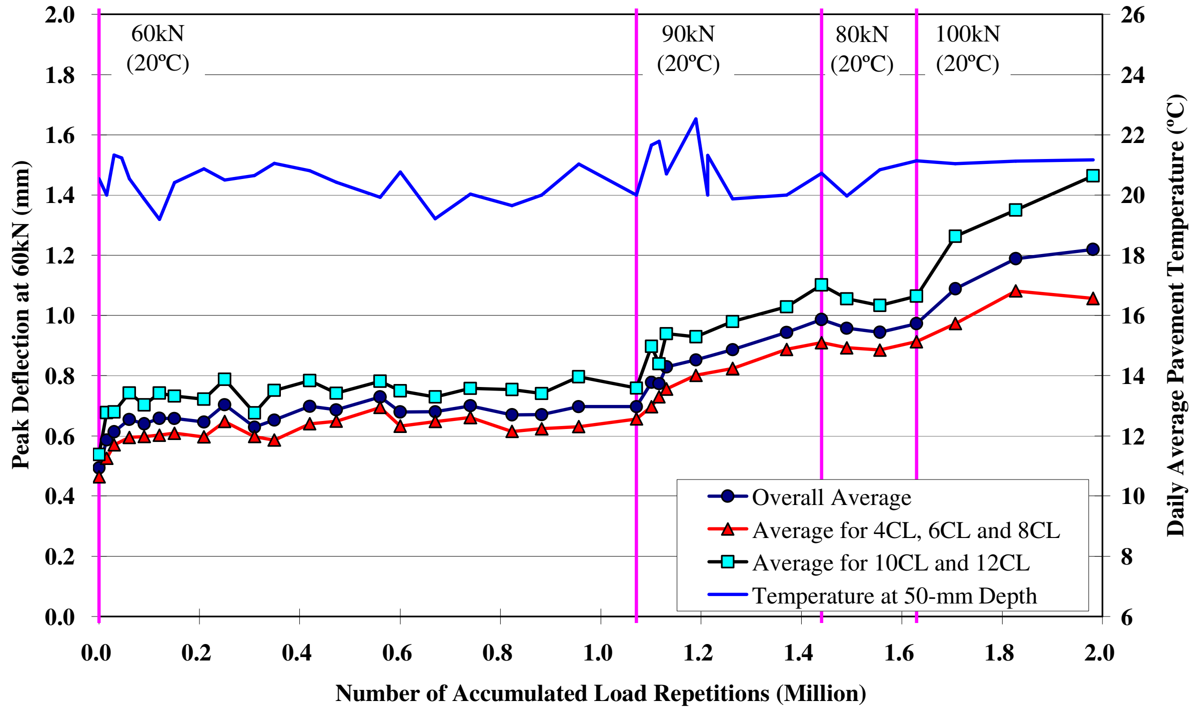

Figure 3.12: Average RSD surface deflections with 60 kN test load (centerline and subsection).

3.3.2 Surface Elastic Deflection from FWD

FWD testing was conducted on the section before, during, and after HVS trafficking to monitor the changes in layer moduli. Table 3.3 summarizes the date, location, temperatures and average deflections for the section. Temperatures listed are average temperatures. Recordings from two sensors (1 and 6) and two locations (section centerline and side of section) are shown. Sensor 1 (the sensor directly under the falling weight) provides an indication of the deflection of the composite pavement. Sensor 6 provides an indication of the deflection in the subgrade. Centerline readings show deflection on the trafficked area, while readings from the side of the section are used to compare trafficked and untrafficked areas. The surface temperatures recorded for the FWD tests during and after HVS testing were considered questionable and will not be used for analysis purposes. Corrected temperatures (28.3°C and 27.8°C respectively) derived from the second degree polynomial relationship of asphalt concrete temperature and surface temperature will be used for backcalculation of moduli. Backcalculation will be discussed in the second-level analysis report.

Table 3.3: Summary of FWD Measurements

Date	Location	Temperatures (°C)		FWD Deflection at 40 kN (microns)[1]			
				Sensor 1		Sensor 6	
		Air	Surface	Average	Std. Dev.	Average	Std. Dev.
After completion of Section 572RF							
05/16/04	Centerline	N/A	29.4	516	170	54	8
Before start of Section 590RF							
01/05/04	Centerline	7.0	6.3	123	7	41	2
During testing on Section 590RF							
03/22/04	Centerline	14.6	23.0	353	45	54	4
03/22/04	Side[2]	14.9	22.4	231	16	74	3
After completion of Section 590RF							
06/16/04	Centerline	16.1	26.1	412	53	57	4
06/16/04	Side	16.0	26.9	283	15	83	3

[1] Deflections based on measurements between Stations 3 and 13 inclusive
[2] Side location is 1.0 m from the test section, representing untrafficked area

Figures 3.13 through 3.17 show the FWD deflection measurements recorded on the section. *Note that scales differ between plots.* Figure 3.13 shows the effect of damage on the pavement over the course of the experiment. Deflection measured on Section 572RF prior to placing the overlay was relatively high, especially in the area of significant cracking. Placement of the overlay considerably reduced the deflection. However, the measurements taken in the middle of the test (during HVS breakdown) show that damage had occurred as a result of loading, with more damage recorded in the area of significant cracking. Damage increased during the course of the test. The overlay provided some structural improvement over the cracked area, adding some structural integrity to the cracked section, but not to the overall pavement structure. The figure also shows that the deflection was influenced by temperature, with higher deflections measured in the morning (lower temperature) compared to those measured in the afternoon (higher temperature) at the end of the test. Throughout the test, higher measurements were recorded in the vicinity of the significantly cracked underlying area compared to the marginally cracked area.

Figure 3.14 shows deflections in the subgrade before, during and after the test. These measurements indicate that there was no significant change during the course of the experiment. The overlay did not provide any significant structural improvement to the overall pavement structure in terms of protection of the subgrade. It appears that the subgrade was less stiff at the end of the section that had more cracking in the underlying DGAC. The slight increase in deflection over the course of the test could be attributed to moisture in the subgrade and/or increased deflections in the upper layers.

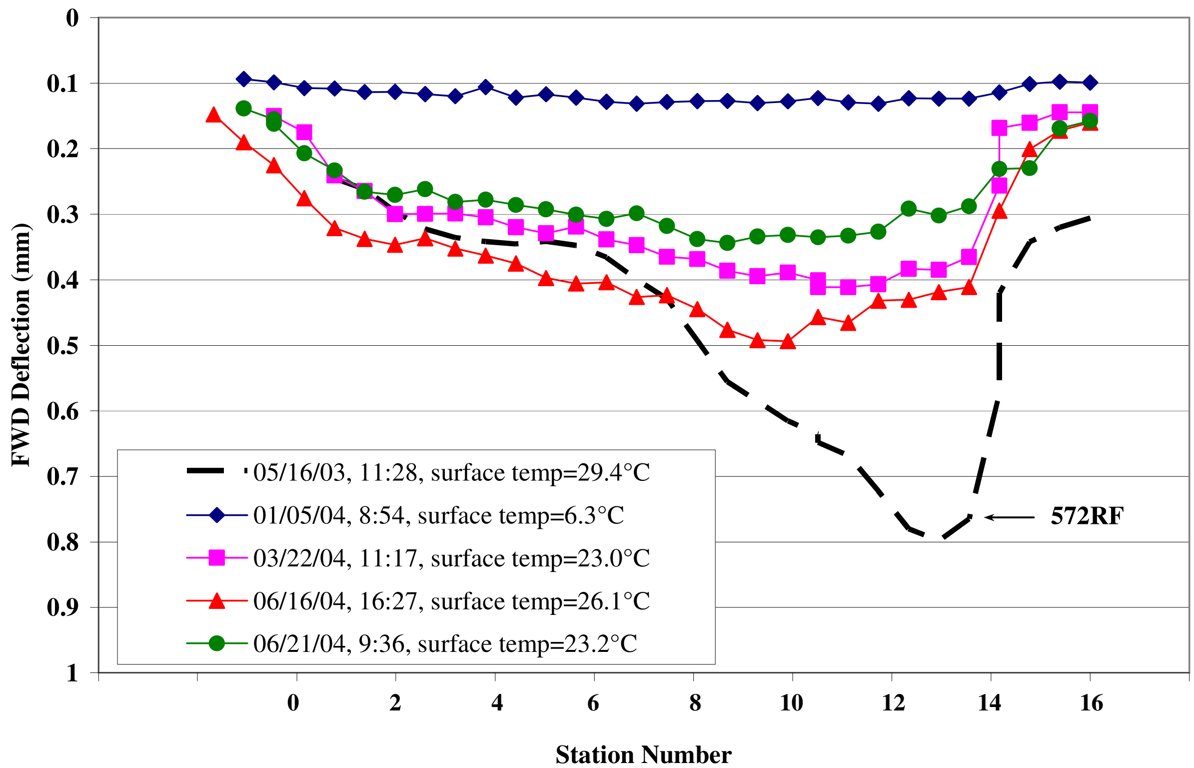

Figure 3.13: Composite pavement stiffness (FWD Sensor 1) on section centerline.

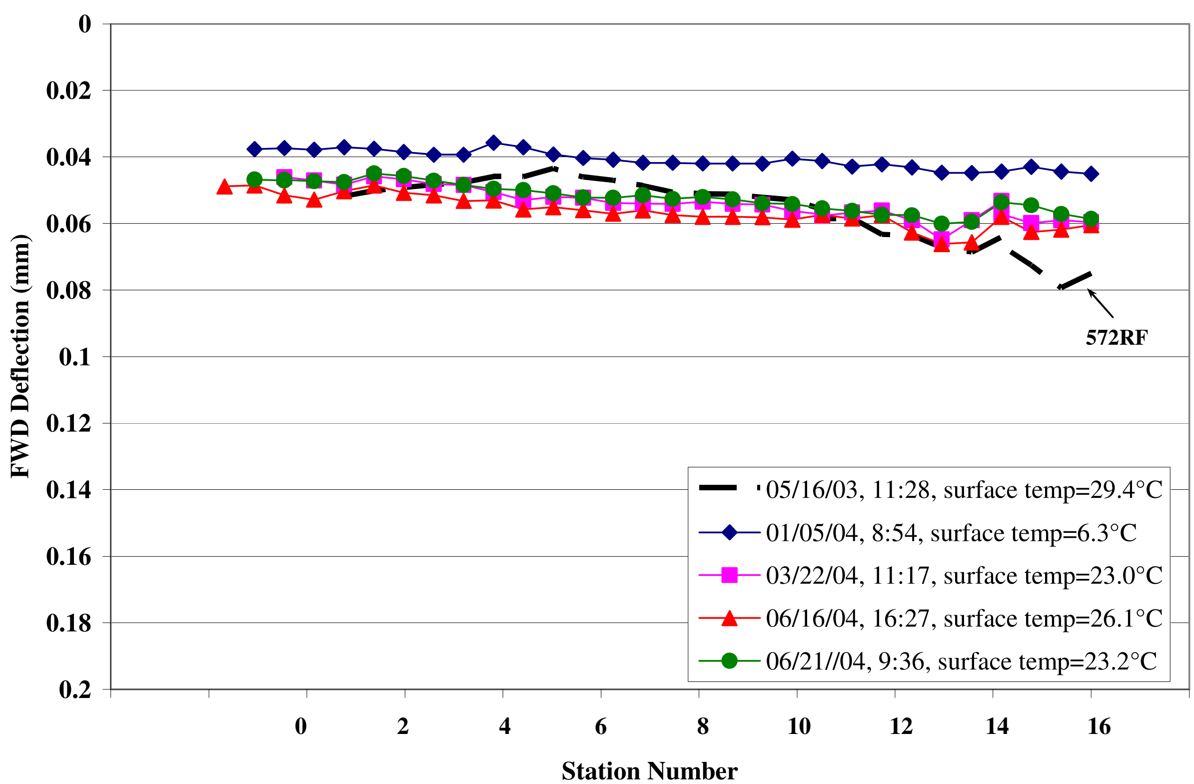

Figure 3.14: Subgrade pavement stiffness (FWD Sensor 6) on section centerline.

Figures 3.15 and 3.16 show FWD deflections taken along the side of the HVS test section but outside the trafficked area (i.e., the area tested did not have traffic damage). The figures can be used to understand the influence of environmental conditions on the performance of the section. The figures show very little change over the course of the experiment, but do show some influence of temperature. No temperature measurement was recorded on June 21, 2004, but time of measurement indicates that the temperature would have been lower than that recorded on June 16, 2004.

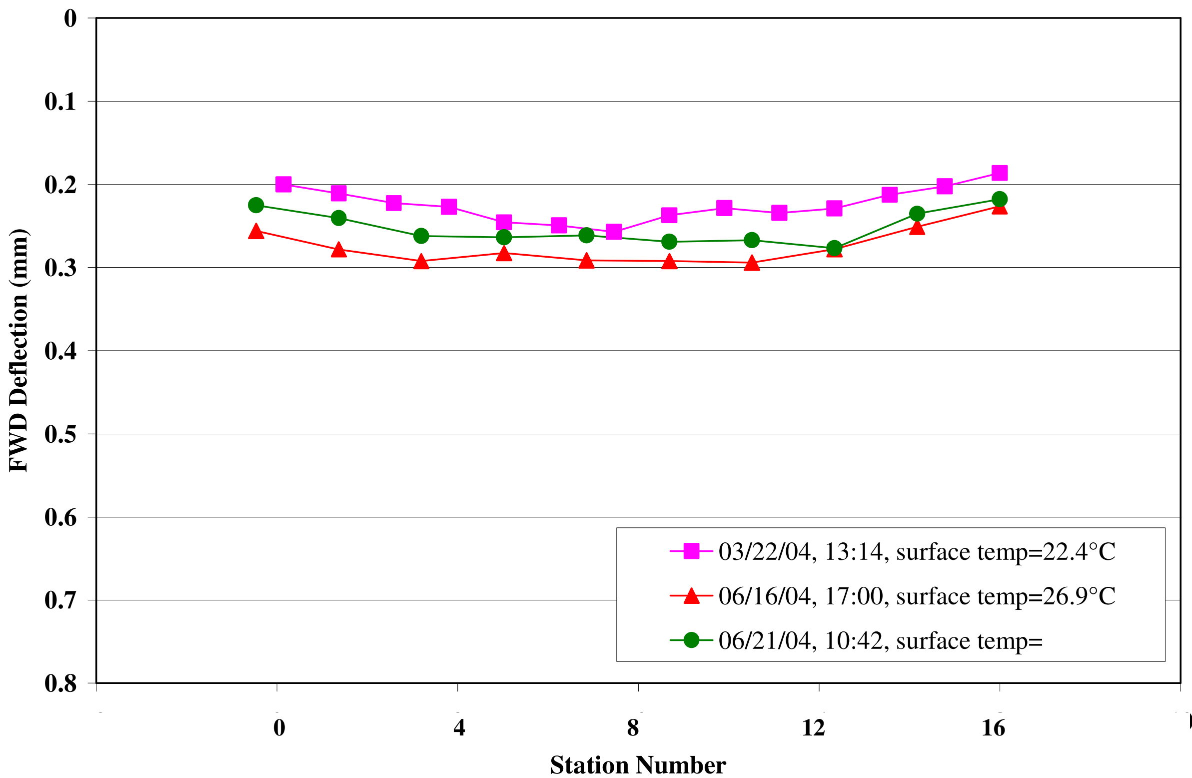

Figure 3.15: Composite pavement stiffness (FWD Sensor 1) outside trafficked area.

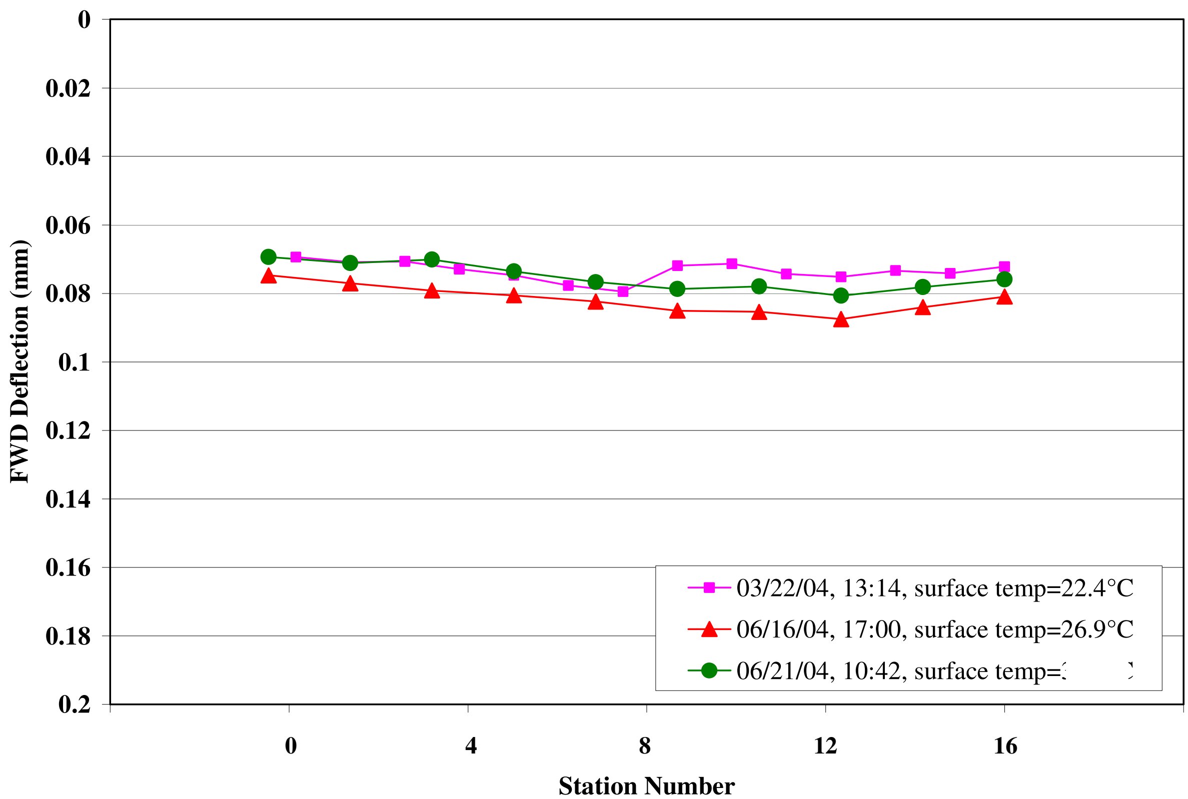

Figure 3.16: Subgrade pavement stiffness (FWD Sensor 6) outside trafficked area.

3.3.3 In-Depth Elastic Deflection from MDD

The schedule of MDD measurements with various test loads is listed in Table 2.2. Only limited data was collected at 90 kN, 80 kN, and 100 kN test loads and hence the following discussion will focus on results obtained with the 60 kN load. Table 3.4 and Figures 3.17 and 3.18 summarize the in-depth elastic deflections measured at various depths with MDD6 and MDD10, respectively. The figures include RSD measurements taken on the surface at the same locations as the MDDs.

Table 3.4: Summary of 60 kN In-Depth Elastic Deflections

Depth (mm)	Layer	Elastic Deflection (microns)		
		Before Trafficking	After Trafficking	Ratio of Final/Initial
			MDD6	
158	Bottom of cracked DGAC	545	1,045	1.92
363	Middle of aggregate base	430	880	2.05
568	Bottom of aggregate base	360	710	1.97
868	280 mm below top of subgrade	-	-	
			MDD10	
158	Bottom of cracked DGAC	580	1,270	2.19
363	Middle of aggregate base	480	1,060	2.21
568	Bottom of aggregate base	410	-	-
868	280 mm below top of subgrade	-	-	-

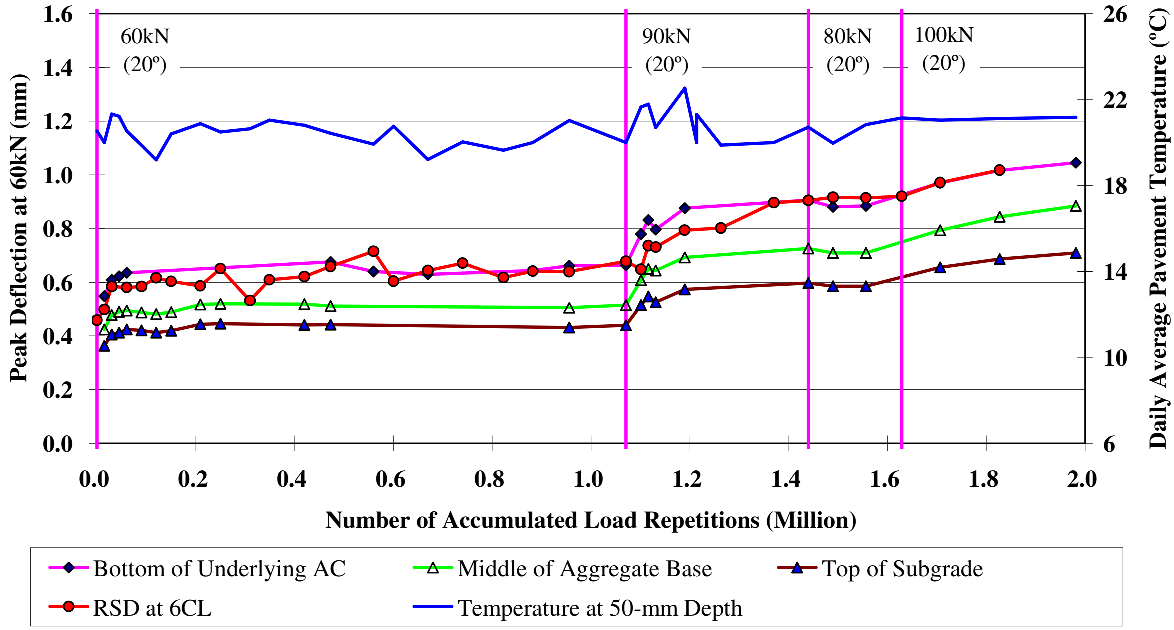

Figure 3.17: Elastic deflections at MDD6 with 60 kN test load.

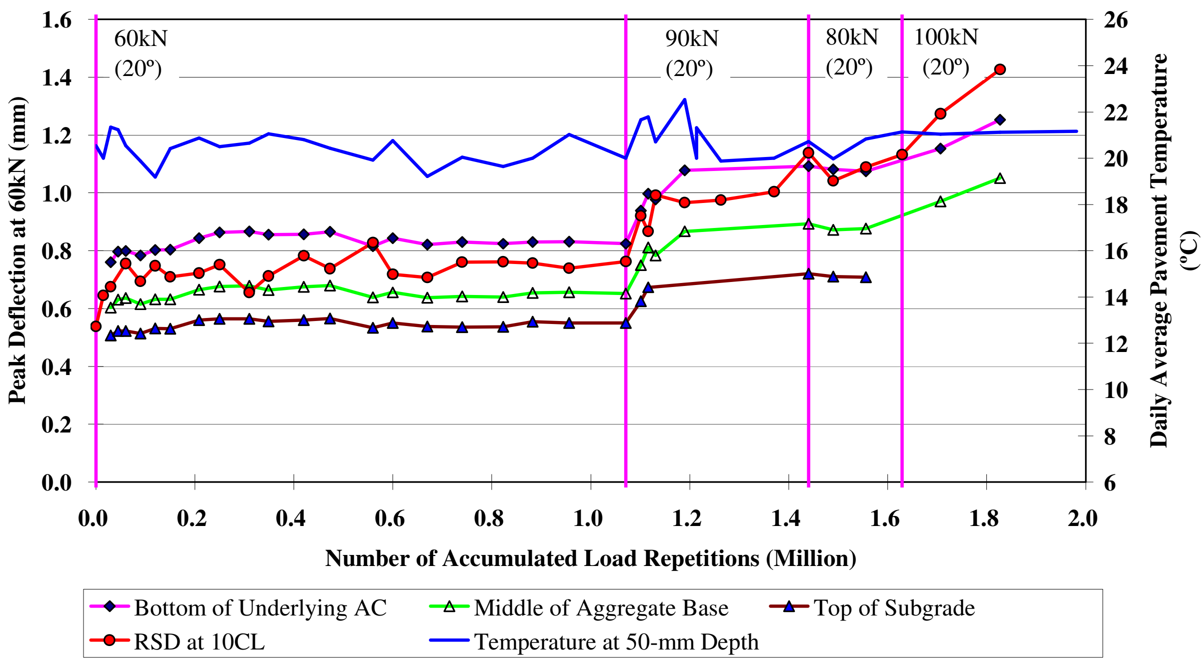

Figure 3.18: Elastic deflections at MDD10 with 60 kN test load.

The following observations were made from the data collected:

- At MDD6 (overlay on marginal cracking) the damage rate climbs sharply during the early phase of 60 kN trafficking. Part of this damage could be reversal of temporary "healing" in the underlying DGAC that occurred during the absence of trafficking between the Phase 1 and Phase 2 HVS tests. Pavement temperature increased by about 1.5°C at the same time. Surface deflections (measured with the RSD) also show some sensitivity (increasing) to temperature. Thereafter, the damage rate decreases asymptotically at all depths. The same phenomenon is observed at other trafficking loads except during the 80 kN trafficking period. Some of the increase in deflection at the 90 kN load is attributed to an increase in pavement temperature of about 1.5°C. It is not clear why the temperature increased at load change. There is no distinct temperature influence at the 80 kN and 100 kN load changes, although the damage rate increases noticeably after the 100 kN change. The sensitivity of the MDD to load reduction is evident during the 80 kN loading period.

- At MDD10 (overlay on significant cracking), similar trends to MDD6 are observed. However, the deflections throughout are higher than those recorded at MDD6. The rate of damage also shows a greater increase at load change compared to MDD6, especially at the 100 kN load level, indicating that the structure, after overlay, did not have as much structural capacity in the end of the test section near MDD10 than it did near MDD 6.
- The in-depth elastic deflections increase at all depths whenever trafficking loads increase, as expected. Deflections initially decreased during the 80 kN loading phase and then increased as damage increased.
- The effect of trafficking load on elastic deformation decreases with increasing depth, as expected.
- Ratios of final-to-initial MDD deflections show that deflections approximately doubled at all depths in the pavement structure by the end of trafficking. The final-to-initial deflection ratio is somewhat greater for MDD10 than for MDD6, indicating that the structure nearer MDD10 endured more damage, in terms of loss of pavement stiffness, than that around MDD6. This loss of stiffness indicates damage in the asphalt concrete layers, which increases shear stresses in the underlying layers. Later in this report these results will be compared with permanent deformation and rutting patterns.

3.4. Permanent Deformation

Permanent deformation at the pavement surface (rutting) was monitored with the Laser Profilometer and at various depths within the pavement with two Multi-depth Deflectometers (MDDs). These measurements are discussed below.

3.4.1 Permanent Surface Deformation (Rutting)

Deformation and rutting on HVS tests are usually analyzed using two definitions, namely maximum rut depth and average deformation (4), as illustrated in Figure 3.19. The Laser Profilometer is used to measure these distresses and provides sufficient information to evaluate the evolution of permanent surface deformation of the entire test section at various loading stages.

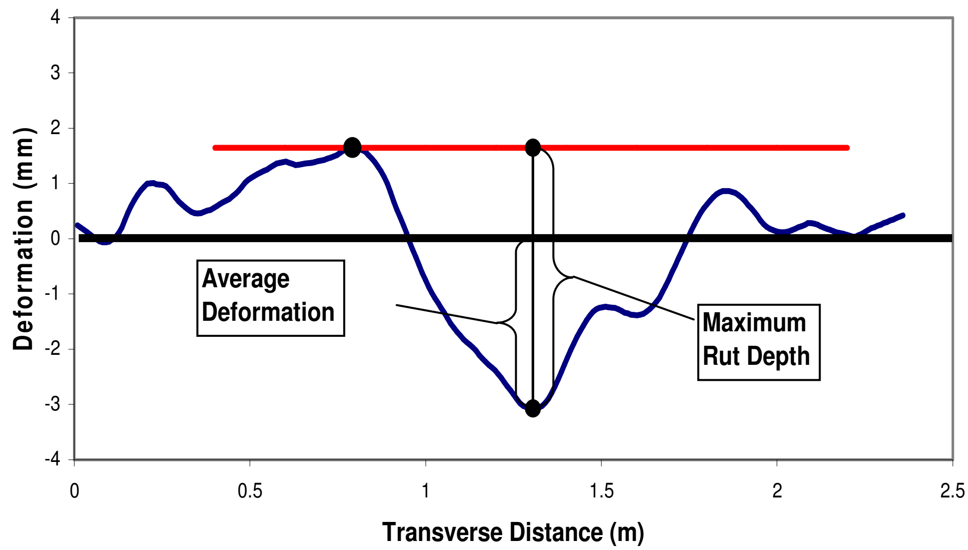

Figure 3.19: Illustration of maximum rut depth and average deformation of a leveled profile.

Figure 3.20 shows the average transverse cross section measured with the Profilometer at various stages of the test. This plot clearly shows the increase in rutting and deformation over the duration of the test.

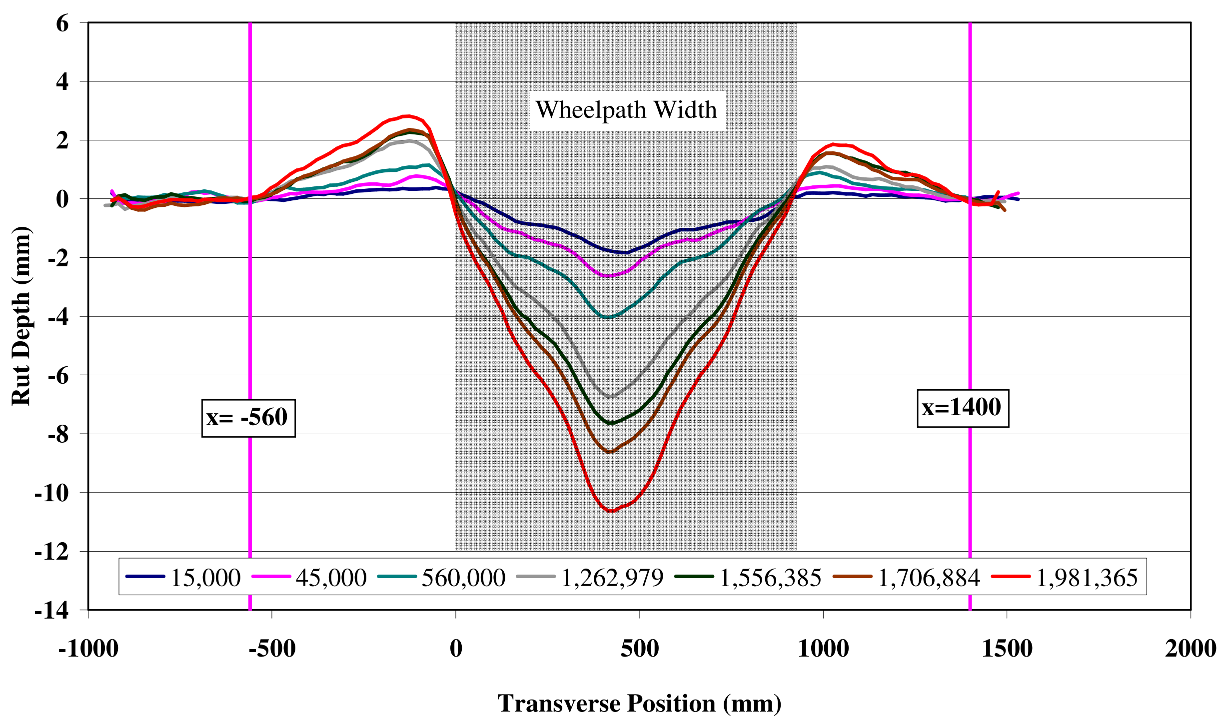

Figure 3.20: Laser Profilometer cross section at various stages of trafficking.

During HVS testing, rutting usually occurs at a high rate initially, then typically diminishes as trafficking progresses until reaching a steady state. If the load level is subsequently increased, the pavement will undergo another phase of rapid rutting development until a steady phase for that new load level is reached.

This initial phase is referred to as the “embedment” phase. Figures 3.21 and 3.22 show the development of permanent deformation (average deformation and maximum rut, respectively) with load repetitions as determined by the Laser Profilometer for the test section. *Note that the scales in the figures are different*. Embedment phases are apparent at the beginning of the experiment and after each load change. Error bars on the average reading indicate variation along the length of the section. The figures also show average maximum deformation and average maximum rut for Stations 3 to 8 and 9 to 13. Stations 3 to 8 are over the end of the section where the underlying DGAC was marginally cracked, while Stations 9 to 13 are over the end with significant cracking. The figures show that deformation and rut are considerably higher over the significantly cracked section. The embedment phases at the 60 kN, 90 kN and 100 kN load changes are also distinct.

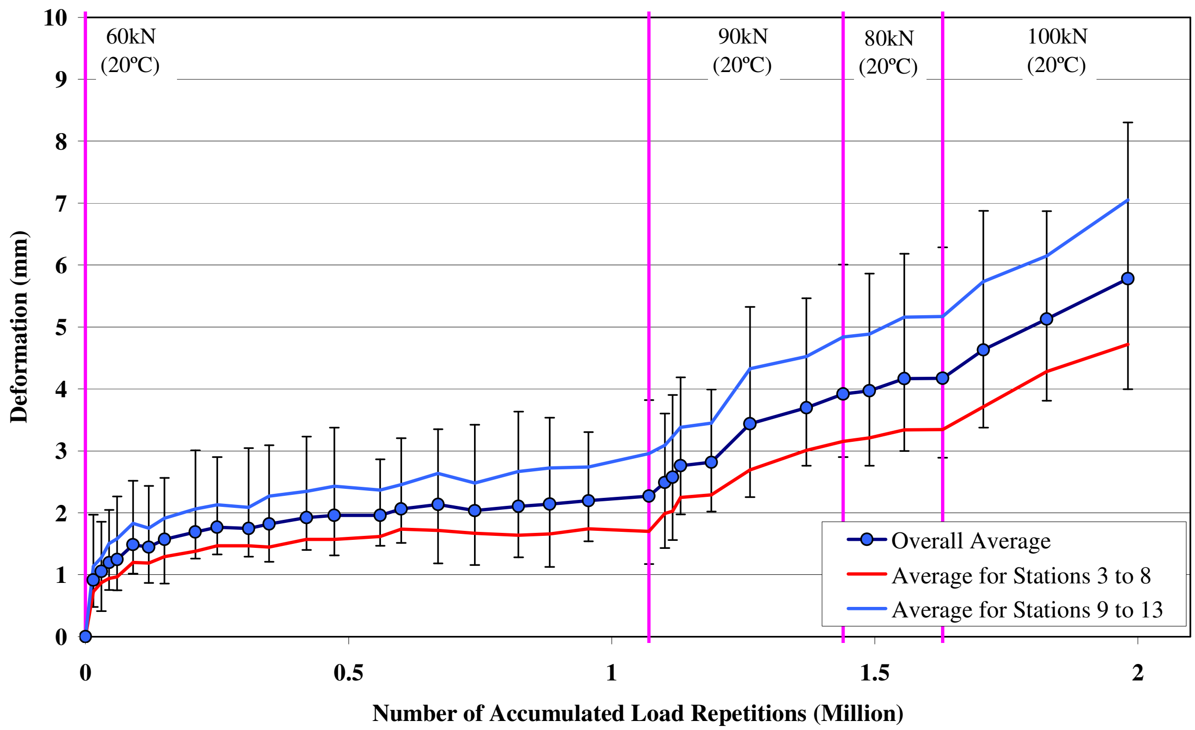

Figure 3.21: Average deformation determined from Laser Profilometer data.

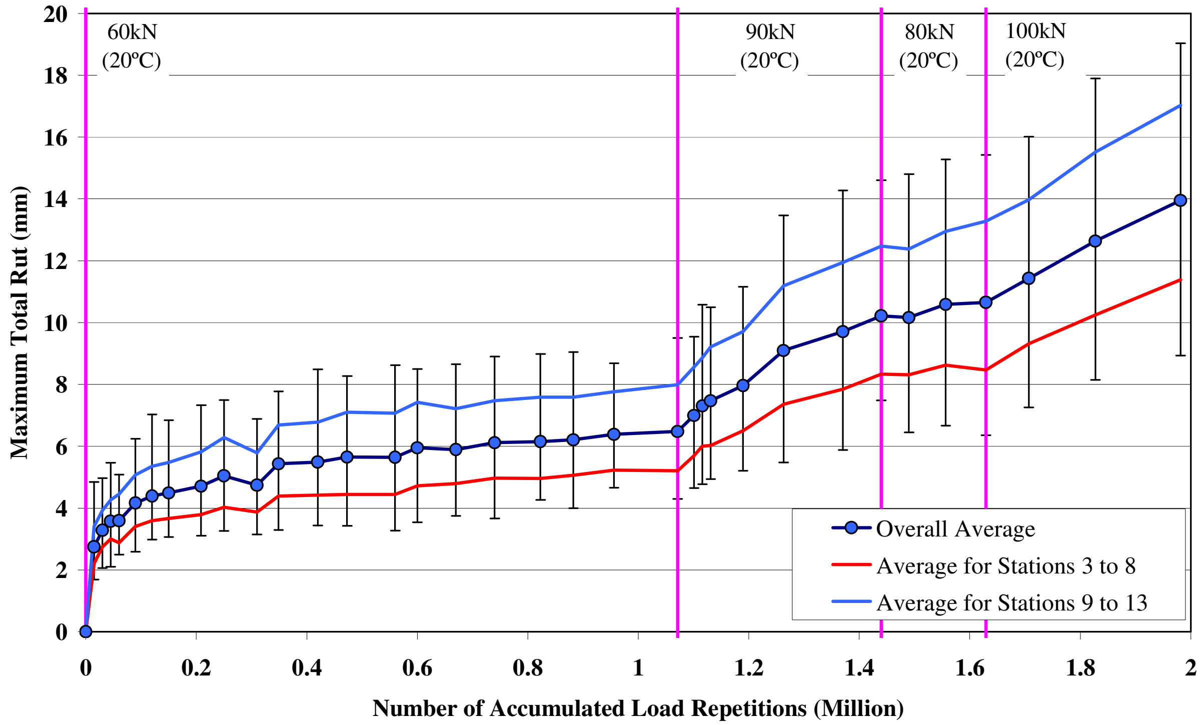

Figure 3.22: Average maximum rut determined from Laser Profilometer data.

Figure 3.23 shows a contour plot of the pavement surface after 15,000 repetitions, the approximate point at which rutting began.

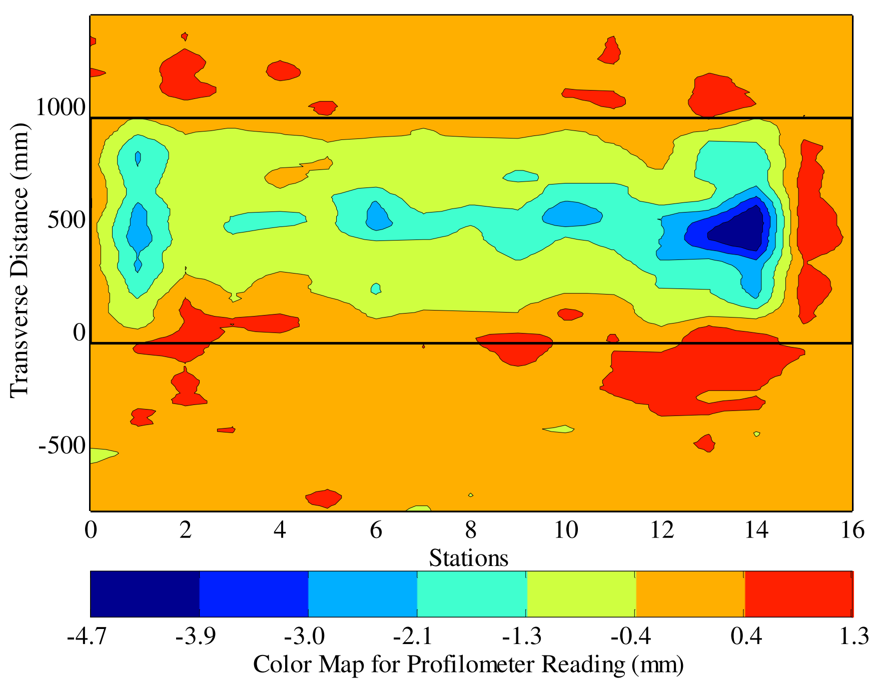

Figure 3.23: Contour plot of permanent deformation after 15,000 repetitions.

Figures 3.24 through 3.28 show contour plots of rutting progression on the surface at 560,000, 1.26 million, 1.55 million, 1.7 million repetitions and at test completion (1,981,365 repetitions).

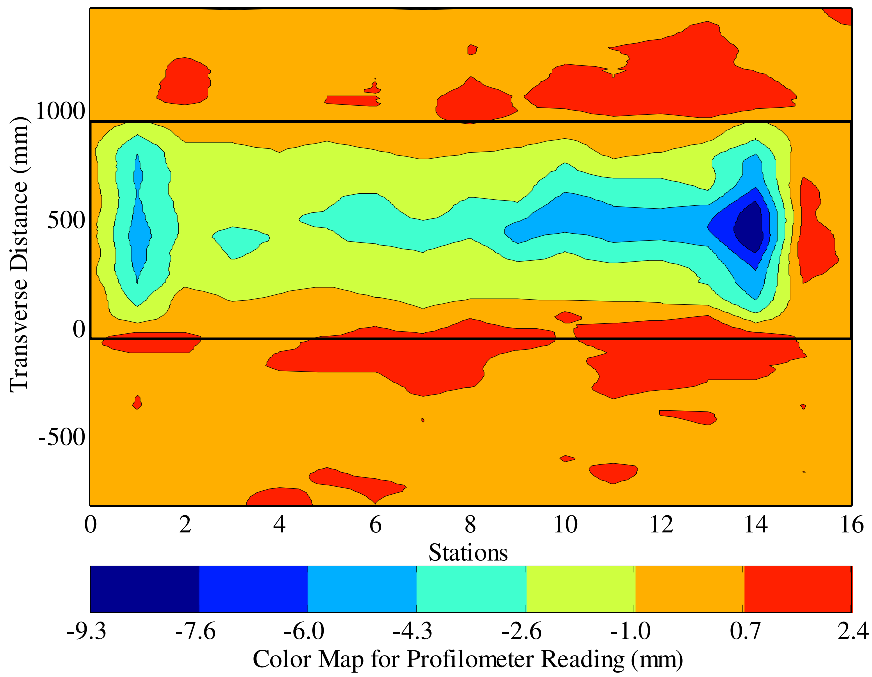

Figure 3.24: Contour plot of permanent deformation after 560,000 repetitions.

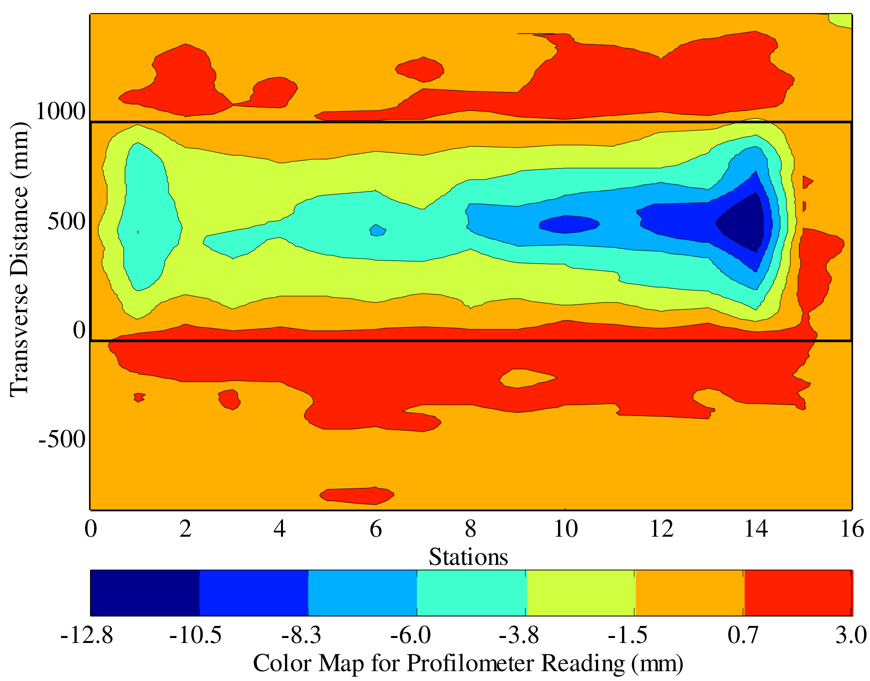

Figure 3.25: Contour plot of permanent deformation after 1,262,979 repetitions.

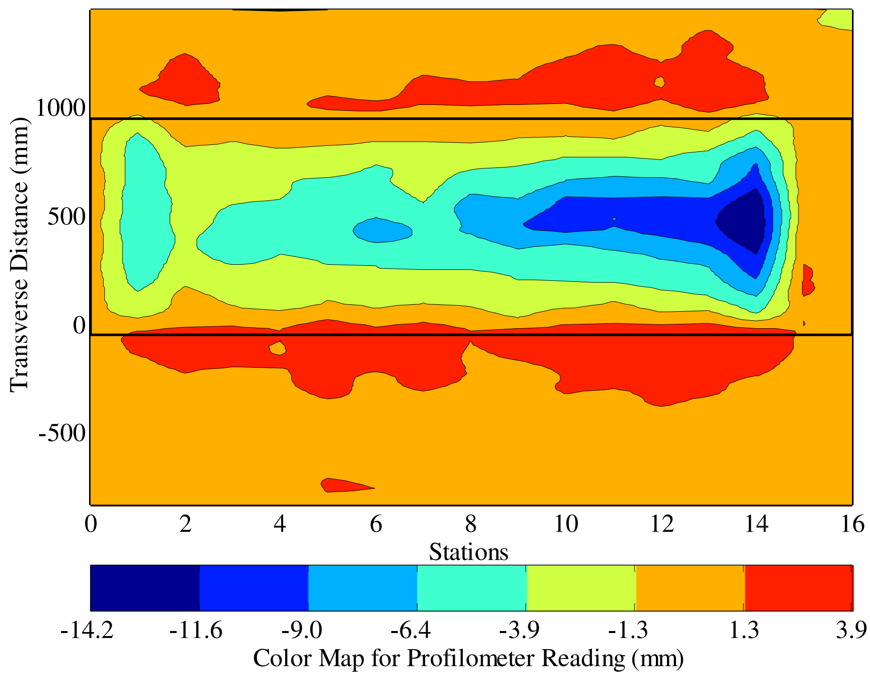

Figure 3.26: Contour plot of permanent deformation after 1,556,385 repetitions.

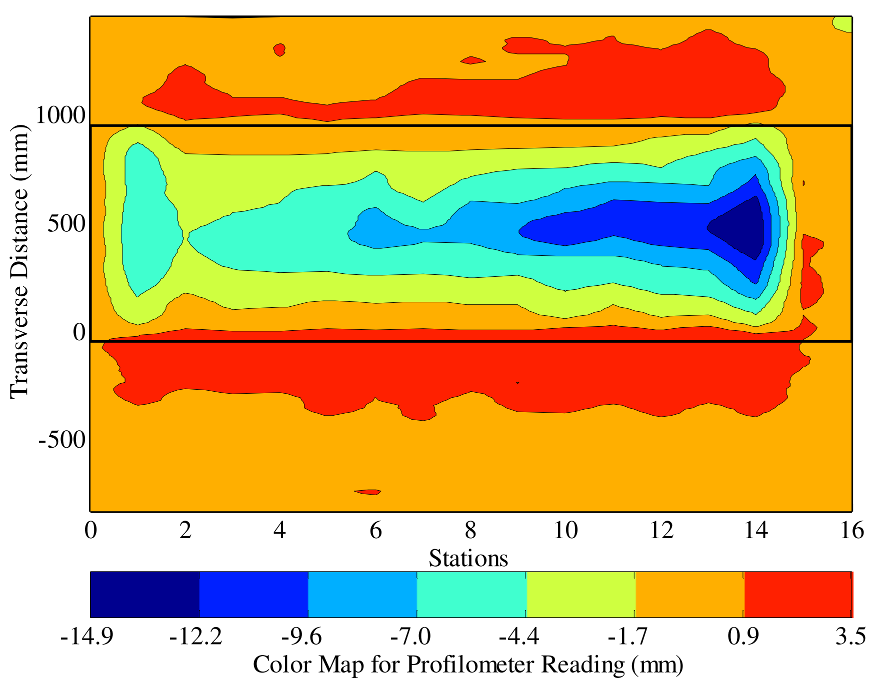

Figure 3.27: Contour plot of permanent deformation after 1,706,884 repetitions.

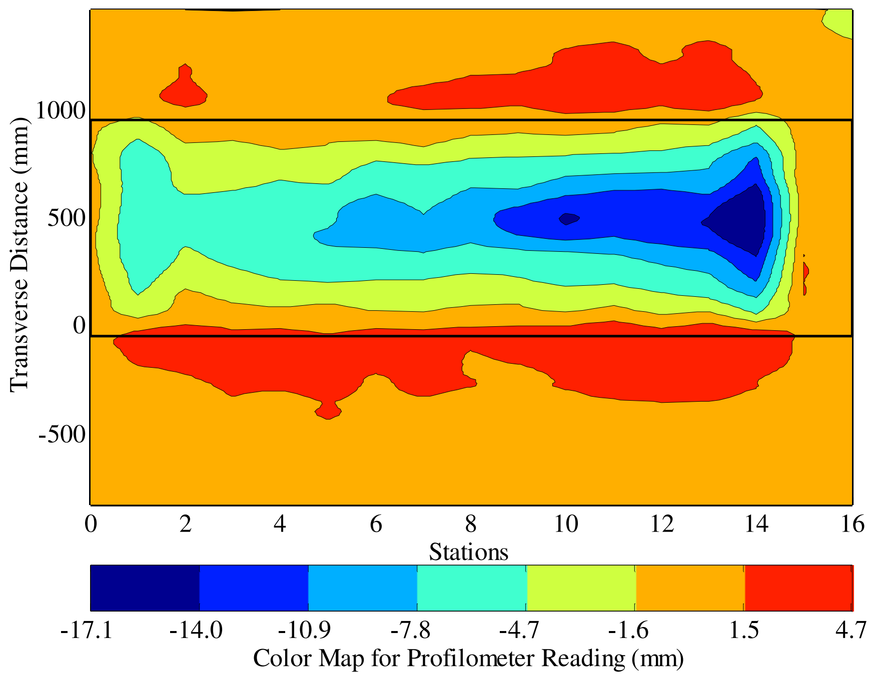

Figure 3.28: Contour plot of permanent deformation after 1,981,365 repetitions.

After completion of trafficking, the average maximum deformation and the average maximum rut depth were 8.5 mm and 12.7 mm, respectively. The maximum rut depth measured on the section was 19.0 mm, at Station 13. Testing was halted when the average maximum rut depth exceeded 12.5 mm, in line with the failure criterion determined for the experiment. The final surface rutting pattern of the overlay corresponds with the fatigue cracking pattern of the cracked DGAC layer as shown in Figure 3.29, with the deepest rut occurring in the area over the most severe cracking in the underlying layer.

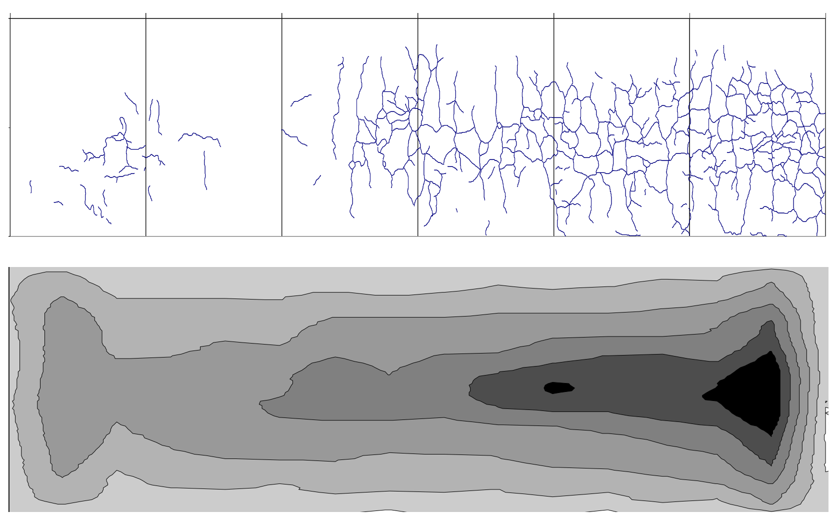

Figure 3.29: Comparison of cracking pattern from Phase 1 and rutting in Phase 2.

3.4.2 Permanent In-Depth Deformation

The accumulation of vertical deformation at various depths in the pavement was measured with the MDD Linear Variable Displacement Transducer (LVDT) modules during the course of the HVS test. Permanent deformation measured by each LVDT is the total permanent deformation of the pavement between the anchoring depth (3.0 m) and the depth of the module. Accordingly, LVDT modules in the upper part of the pavement typically measure larger permanent deformation than those in the lower part. The difference in measured permanent deformation between two LVDT modules represents the permanent deformation accumulated in the layers between those two modules. This is known as differential permanent deformation. Module locations are shown in Figure 2.2. A module was not installed on the surface of the MB4-G overlay due to thickness constraints.

Table 3.5 and Figures 3.30 to 3.33 provide an indication of the permanent deformation recorded at MDD6 and MDD10 respectively. Figures 3.30 and 3.32 show permanent deformation at the MDD modules, while Figures 3.31 and 3.33 show the differential deformation calculated for the various layers.

Table 3.5: Vertical Permanent Deformation in Pavement Layers

Layer	Thickness (mm)	Vertical Permanent Deformation (mm)		Percentage Total Deformation (%)	
		MDD6	MDD10	MDD6	MDD10
Surface (Profilometer)	-	8.5*	14.2*	43	71
Bottom of cracked DGAC	90	4.6	3.3	24	17
Upper aggregate base	180	3.9	1.7	20	8
Lower aggregate base	230	2.6	0.8	13	4
Subgrade	Semi-infinite	Not measured	Not measured	Not determined	Not determined
Total (AC+base)		11.2	5.9	100	100
* Laser profilometer measurement on MDD topcap - top MDD module measurement					

Permanent deformation at the upper MDD module (158 mm) accumulated at similar rates for both MDDs up until the 90 kN load change. At this change, permanent deformation at this module in MDD10 was approximately 1.1 mm higher than that recorded at MDD6. The rate of increase continued though the 90 kN loading phase. The permanent deformation in the upper portion of the pavement at MDD10 on completion of testing was about 1.5 mm higher than that at MDD6. This was attributed to the cracked DGAC layer. The permanent deformation recorded at the 363 mm and 588 mm modules for both MDDs are very similar. At the 363 mm modules, permanent deformation did not increase significantly during the 60 kN loading phase, but increased sharply at the 90 kN load change. Thereafter a gradual increase was recorded through the remainder of the 90 kN and through the 80 kN phases. The rate of permanent deformation increased again through the 100 kN loading phase. Very little permanent deformation was recorded at the 588 mm modules.

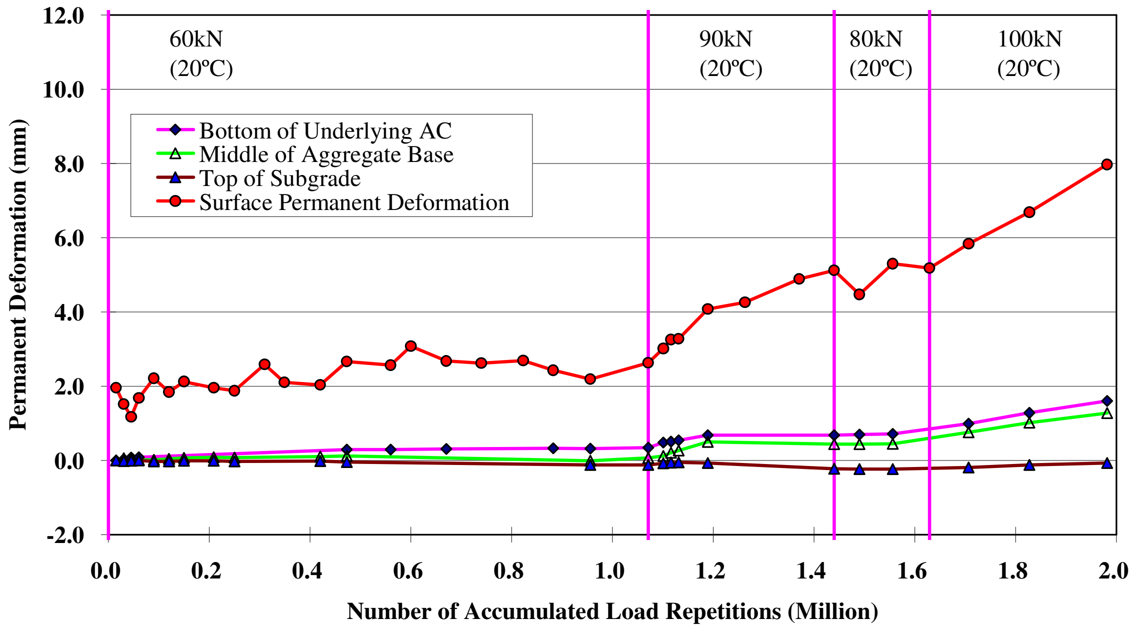

Figure 3.30: In-depth permanent deformation at MDD6.

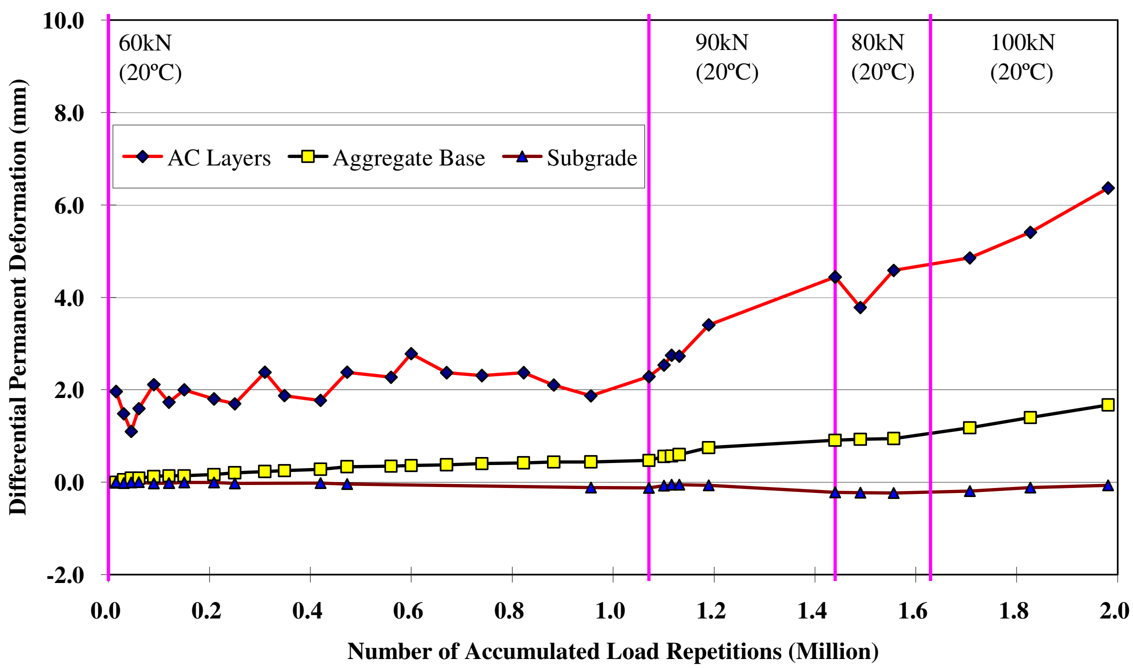

Figure 3.31: In-depth differential permanent deformation of various layers at MDD6.

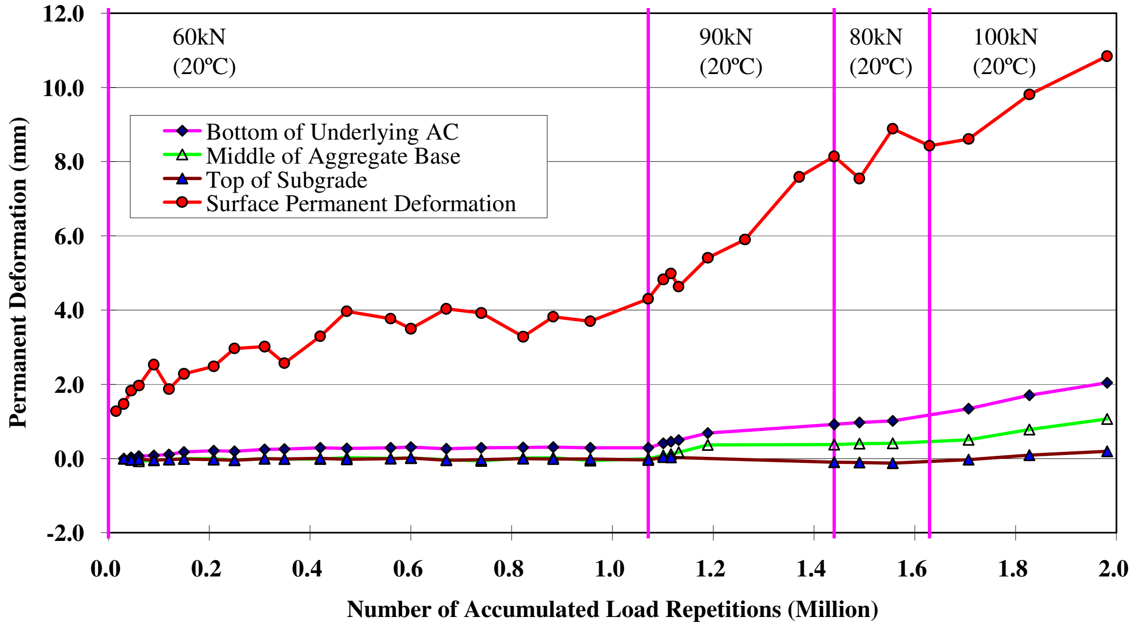

Figure 3.32: In-depth permanent deformation at MDD10.

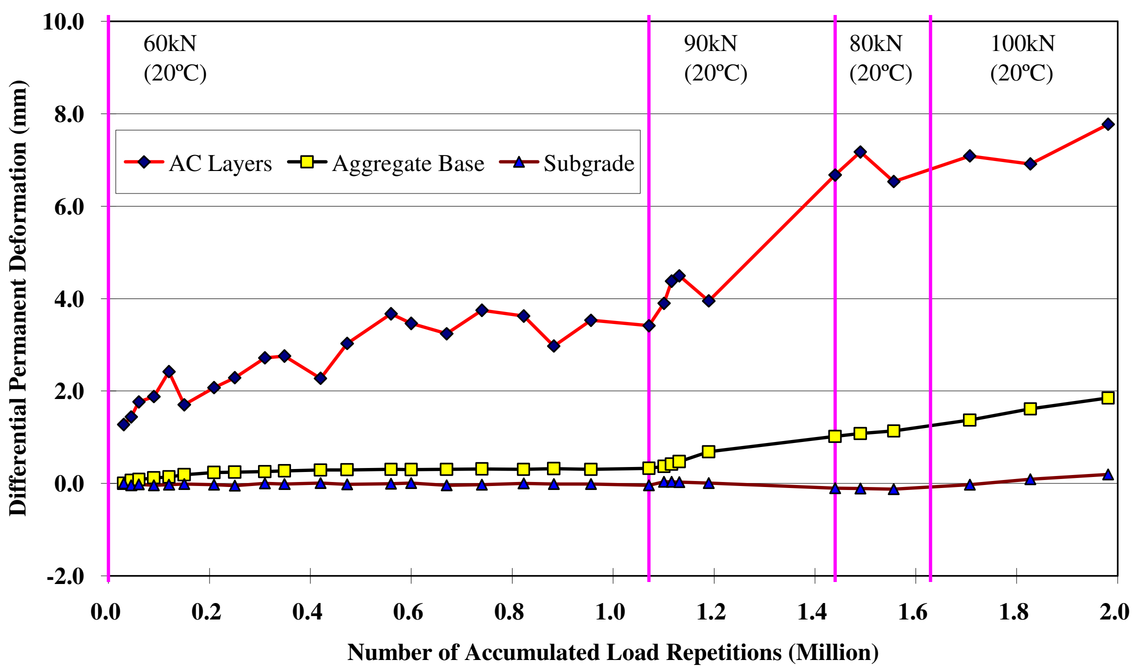

Figure 3.33: In-depth differential permanent deformation of various layers at MDD10.

Figures 3.31 and 3.33 show the permanent deformation calculated for the various layers. These figures clearly show that most of the deformation is in the asphalt-bound surface layers (surface deformation measured with the profilometer minus deformation measured at the 158 mm MDD module), with significantly more deformation at that end of the section overlying the area of significant cracking in the

DGAC. These figures also confirm that permanent deformation in the aggregate base (deformation at the 568 mm module minus deformation at the 158 mm module) at both modules was similar (although marginally higher at MDD6), and that very little permanent deformation occurred in the subgrade. These trends confirm that the effect of increasing wheel load on permanent deformation diminishes with depth in the pavement structure. The planned forensic investigation will confirm these findings and will be reported on in the second-level analysis report.

3.5. Visual Inspection

Fatigue distress in an asphalt concrete pavement manifests itself in the form of surface cracks. Since this study centered on fatigue cracking and the ability of the overlay to limit reflective cracking from the underlying layer, crack monitoring was an essential component of the data collection program. This entails:

- Visual inspections of the test section and marking of visible cracks;
- Photographic documentation of the marked cracks;
- Correction of the photographs for camera angle;
- Digitization of the photographs;
- Calculation of the crack length using *Optimas*™ software, and
- Presentation of the cracking in terms of crack length per area of pavement.

After 1,981,365 repetitions with increasing wheel loads (from 60 kN to 100 kN) under controlled pavement temperatures (20°C), no surface cracking was observed. The MB4-G overlay thus appeared to successfully prevent any reflective cracking from the underlying layer from appearing on the surface, despite calculated ratios of final-to-initial deflections indicating that considerable damage had occurred in the asphalt layers during the course of trafficking. A photograph of the section after trafficking is shown in Figure 3.34.

3.6. Forensic Evaluation

A forensic evaluation (coring and test pit) can only be undertaken when HVS testing on all of the six sections has been completed. Results of the forensic evaluation will be discussed in a separate report on completion of the tests.

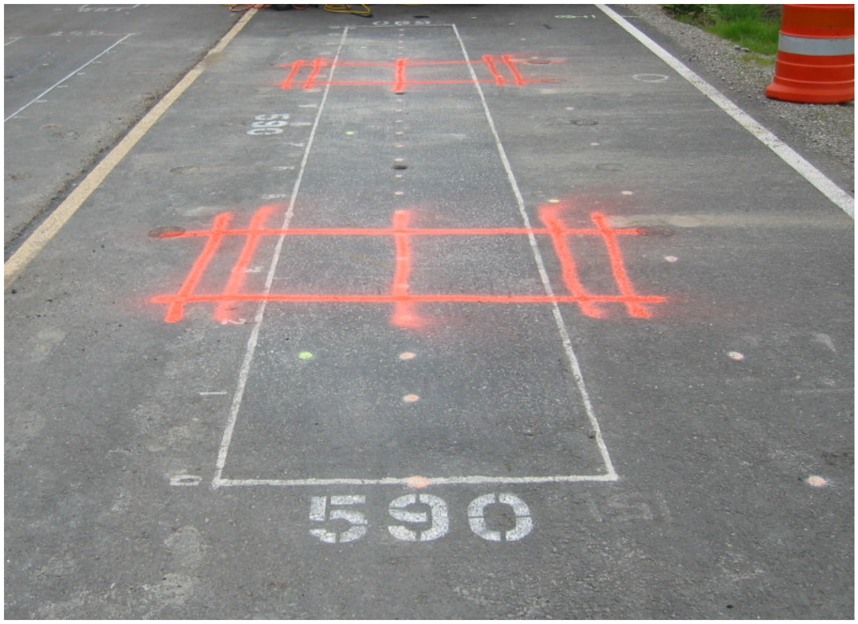

Figure 3.34: Section surface after completion of trafficking.

3.7. Second-Level Analysis

A second-level analysis report will be prepared on completion of all HVS testing and a forensic evaluation. This report will include:

- Actual layer thicknesses,
- Backcalculation of moduli from RSD, MDD, and FWD measurements,
- Verification of data collected from in-depth measurements with visual observations from test pits,
- Comparison of performance between test sections,
- Comparisons of HVS test results with laboratory test results, and
- Recommendations.

4. CONCLUSIONS

This first-level report is the first in a series of studies detailing the results of HVS testing being performed to validate Caltrans overlay strategies for the rehabilitation of cracked asphalt concrete. It describes the results of the first HVS reflective cracking testing section (590RF) carried out on a 90-mm (3.5 in) full-thickness MB4-G overlay. Other overlays that will be tested during the course of the experiment include:

- Half-thickness (45 mm) MB4 gap-graded overlay (45 mm MB4-G);
- Half-thickness MB4 gap-graded overlay with minimum 15 percent recycled tire rubber (MB15-G);
- Half-thickness MAC15TR gap-graded overlay with minimum 15 percent recycled tire rubber (MAC15-G);
- Half-thickness rubberized asphalt concrete gap graded overlay (RAC-G) overlay, included as a control for performance comparison purposes, and
- Full-thickness (90 mm) AR4000-D overlay, included as a control for performance comparison purposes.

The pavement was designed according to the Caltrans Highway Design Manual Chapter 600 using the computer program *NEWCON90*. Design thickness was based on a subgrade R-value of 5 and a Traffic Index of 7 (~121,000 ESALs). The overlay thickness was determined according to Caltrans Test Method (CTM) 356 using Falling Weight Deflectometer (FWD) deflections.

HVS trafficking on the section commenced on January 13, 2004, and was completed on June 16, 2004. A temperature chamber was used to maintain the pavement temperature at 20°C±4°C (68°F±7°F). During this period a total of 1,981,365 load repetitions (tire pressure of 720 kPa [104 psi], and bi-directional trafficking pattern with wander) were applied consisting of:

- 1,071,004 repetitions of a 60 kN (13,500 lb) load,
- 368,894 repetitions of a 90 kN (20,250 lb) load,
- 189,160 repetitions of an 80 kN (18,000 lb) load, and
- 352,307 repetitions of a 100 kN (22,500 lb) load.

This loading equates to approximately 37 million equivalent standard axles, using the Caltrans conversion of $(\text{axle load}/18000)^{4.2}$, which in turn equates to a Traffic Index of 14.

Testing was interrupted between mechanical breakdowns between March 24 and April 27, 2004, when the cumulative traffic repetitions was approximately 1,213,500.

Laboratory fatigue and shear studies have been conducted in parallel with HVS testing. Results of these studies will be detailed in separate reports. Comparison of the laboratory and test section performance, including the results of a forensic investigation to be conducted when testing is complete, will be discussed in a second-level report once the data from all of the studies have been collected.

Findings and observations based on the data collected during this HVS study include:

- No reflective cracking from the underlying severely cracked dense graded asphalt concrete layer was observed on the MB4-G overlay after almost two million HVS repetitions. The MB4-G overlay thus appeared to successfully prevent any cracking in the underlying layer from reflecting through to the surface, despite final-to-initial deflections indicating that considerable damage had occurred in the asphalt layers under loading.
- The average maximum rut depth across the entire test section at the end of the test was 12.7 mm (0.5 in), equivalent to the Caltrans (and experiment) failure criterion of 12.5 mm (0.5 in). The maximum rut depth measured on the section was 19.0 mm (0.7 in), with a depth of 12.5 mm (0.5 in) reached after about 1.17 million repetitions, soon after the load increase from 60 kN (13,500 lb) to 90 kN (20,250 lb). Despite conducting HVS testing at a relatively low pavement temperature of 20°C±4°C (68°F±7°F), the MB4-G overlay appeared susceptible to rutting from early in the experiment.
- Ratios of final-to-initial elastic surface deflections under a 60 kN (13,500 lb) wheel load increased by between 1.9 and 3.0 times along the length of the section. The ratios for in-depth deflections show that damage increased at all depths in the pavement structure (between 1.9 and 2.2 times) by the end of trafficking. Loss of stiffness was highest in the area of most severe cracking in the underlying DGAC layer.
- Analysis of surface profile and in-depth permanent deformation measurements indicates that most of the permanent deformation (between 67 [at MDD6] and 88 percent [at MDD10]) occurred in the asphalt-bound surfacing layers (overlay and cracked DGAC) with marginal deformation in the base layer and negligible deformation in the subgrade.

No recommendations as to the use of MB4-G mixes are made at this time. These recommendations will be included in the second-level analysis report, which will be prepared and submitted on completion of all HVS and laboratory testing.

5. REFERENCES

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2. **Reflective Cracking Study: Workplan for the Comparison of MB, RAC-G, and DGAC Mixes Under HVS and Laboratory Testing.** 2003. Davis and Berkeley, CA: University of California Pavement Research Center. (UCPRC-WP-2003-01).
3. BEJARANO, M., Jones, D., Morton, B., and Scheffy, C. 2005. **Reflective Cracking Study: Summary of Construction Activities, Phase 1 HVS Testing, and Overlay Construction**. Davis and Berkeley, CA: University of California Pavement Research Center. (UCPRC-RR-2005-03).
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5. HARVEY, J., Du Plessis, L., Long, F., Deacon, J., Guada, I., Hung, D. and Scheffy, C. 1997. **CAL/APT Program: Test Results from Accelerated Pavement Test on Pavement Structure Containing Untreated Base – Section 501RF.** Davis and Berkeley, CA: University of California Pavement Research Center. (Report Numbers UCPRC-RR-1997-03 and RTA-65W4845-3).