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Evaluation of a Hot Mix Asphalt Perpetual Pavement

FINAL REPORT

April 2010

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Research Study # FEP-02-02

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16. Abstract

In 2003, WisDOT constructed two perpetual pavement test sections on the entrance ramp to I-94 from the Kenosha Safety and Weigh Station Facility in Southeastern WI. Test section 1 (TS1) HMA layers were constructed as follows: 2-in surface layer (PG 76-28, 6% air voids); 4.5-in middle layer (PG 70-22, 6% air voids); 4.5-in lower layer (PG 64-22, 4% air voids). Test section 2 HMA layers were constructed as follows: 2-in surface layer (PG 70-28, 6% air voids); 4.5-in middle layer (PG 70-22, 6% air voids); 4.5-in lower layer (PG 64-22, 6% air voids). The test sections were subjected to nearly 100% truck traffic with a projected 75 million ESALs over 20 years.

After seven years in service, premature longitudinal and alligator cracking was present in the wheel paths of both test sections, with TS1 displaying a slightly higher level of distress. No rutting was observed in either test section. Forensic coring showed that the cracking was top-down. The early distresses were likely due to segregation and over-compaction that occurred during construction.

Strain induced by trucks with known loads was measured using strain gages installed during construction. Strain at the bottom of the HMA pavement was typically lower than 70×10^{-6}, the currently-accepted HMA fatigue endurance limit. Strains up to 100×10^{-6} were measured with high axle loads (47 kips), slow travel speeds (32 mph) and high pavement temperatures (90-103°F).

The perpetual pavement performance was acceptable overall. Distresses were limited to the surface HMA layer, which can be milled and replaced without affecting the lower layers. Strains were low at the bottom of the HMA pavement, indicating that the pavement system adequately resisted fatigue damage. Mechanistic analysis showed that low air voids (4 to 5%) in the lower layer provided the longest fatigue life. Low air voids are therefore recommended in the bottom layer to achieve maximum service life.

17. Key Words Perpetual pavement, HMA pavement, strain gage instrumentation, mechanistic analysis, FWD testing		**18. Distribution Statement** Distribution unlimited, approved for public release	
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1. Introduction

Escalating traffic volumes, especially in the commercial truck traffic category, place increased demands on pavement systems everywhere in the U.S. The state of Wisconsin is not exempt from this issue and has expanded the number of high-strength and extended-life pavements in its set of design alternatives. In the hot mix asphalt (HMA) pavement category, these pavement design alternatives include:

- *Full-depth pavement* - a thick HMA layer constructed directly over subgrade
- *Deep-strength pavement* - thicker than typical HMA pavement, paved over granular base course
- *Perpetual pavement* - a type of deep-strength pavement; a three-layer system in which each layer plays a specific role in resisting traffic loads

The perpetual pavement design has become popular in the U.S. and internationally, and many research efforts have focused on defining the best-performing three-layer system. The Wisconsin Department of Transportation (WisDOT) was among several state agencies interested in implementing perpetual pavement system test sections on network highways. For this study, a unique opportunity was available: the perpetual pavement system was constructed on a truck weigh station interstate entrance ramp. This setup offered several benefits, including accelerated loading (all trucks concentrated in a single lane), documented weight information, and a controlled system that could be closed to traffic for testing and performance reviews.

This report documents results of perpetual pavement test sections constructed on the Kenosha Safety and Weigh Station Facility entrance ramp to Interstate 94 westbound in Southeastern Wisconsin. The pavement was constructed in 2003 and was monitored for distresses until 2009. The pavement layers were instrumented with strain gages to assess load-induced strain levels. Material analyses were also conducted to determine properties of the HMA pavement.

2. Background

2.1 Perpetual Pavement Description

A perpetual pavement system consists of three HMA pavement layers, a base course layer, and subgrade. The concept of a perpetual pavement is to design fatigue- and rut-resistant lower HMA layers, thereby limiting any distresses to top-down cracking in the relatively thin, high-quality surface HMA layer. The pavement system can be rejuvenated by periodically removing and replacing this thin surface layer when it reaches a critical distress level. The lower layers are left undisturbed and can remain in place for many years. Pavement reconstruction therefore occurs much less often, which reduces costs associated with construction, materials, and user delay.

A perpetual pavement system typically involves three HMA layers. The lower layer is paved with a fatigue-resistant mixture; its purpose is to resist strains induced by traffic loads. If strain is limited to the HMA mixture's endurance limit (often estimated to be in the range of 70 to 100×10^{-6}), little or no

bottom-up fatigue cracking should occur in this lower layer. [*1, 2*] The actual value of the HMA endurance limit is likely affected by many material and environmental properties, however, and is still being researched. [*3, 4, 5*] The more conservative estimate of $70x10^{-6}$ will be referenced in this study.

The middle layer is paved with a rut-resistant mixture. Its thickness is adjusted to give the pavement system adequate structure for the estimated traffic loads. The upper layer is thin (typically 1.5 to 3 inches) and is paved with a rut- and wear-resistant mixture. As mentioned above, this is a sacrificial layer that is replaced before top-down distresses extend into the middle layer. The top layer can be a Superpave, stone matrix asphalt (SMA), or open-graded friction course (OGFC) mixture.

2.2 Other State Experiences

In 2005, the Kansas Department of Transportation (DOT) constructed four perpetual pavement test sections on U.S. Highway 75. The HMA pavement layers consisted of a 1.5-in upper layer and a 2.5-in middle layer (both with PG 70-28 asphalt binder), and four different lower layers with thicknesses varying from 6 to 12 inches and PG 64-22 or 70-28 binders. Strain sensors were installed at the bottom of the lower HMA layers in all test sections. Strain response was measured in 2005 and 2007 using loaded trucks with known axle weights. Tests were conducted at different truck speeds and pavement temperatures. Warm pavement temperatures and slower truck speeds resulted in higher measured tensile strain values. In nearly all cases, measured tensile strain was below $70x10^{-6}$. [*6, 7*]

In 2005, the Oregon DOT constructed perpetual pavement sections on Interstate 5. Two HMA pavement structures were tested: the first was a 2-in wearing layer over a 10-in lower layer over 8 inches of rubblized concrete over 9 to 12 inches of existing base course. The second was a 2-in wearing layer over a 10-in lower layer over 16 inches of new base course. A mechanistic evaluation was used to calculate tensile strains at the bottom of the HMA layers. Strains up to approximately $91x10^{-6}$ were noted with heavy axle loads modeled in warm temperature conditions. In the months of October through May, however, no strain in excess of $70x10^{-6}$ was noted for steering axles modeled up to 20 kips. Strains induced by tandem axles modeled up to 46 kips only produced strains in excess of $70x10^{-6}$ in July and August. Strain gages were also installed at the bottom of the HMA pavement in both test sections. Preliminary strain data collected with unknown vehicle weights showed lower strains in the section constructed over rubblized concrete. Measured strain in both test sections was less than $70x10^{-6}$. [*8*]

In 2005, the Ohio DOT and Ohio University began an in-situ perpetual pavement study. The test section structure was designed so that the maximum strain at the bottom of the HMA layer would not exceed $70x10^{-6}$. The test section consisted of the following structure from top to bottom: 3.25-in upper layer (PG 76-22 binder); 9-in middle HMA layer (PG 64-22 asphalt binder); 4-in lower HMA layer with 3 percent air voids; and 6-in dense, crushed aggregate base course. Strain sensors were installed at several locations in the cross section. Controlled vehicle load testing was conducted at various test speeds under warm and cold pavement temperature conditions. Test loads included tandem axle loads up to 40.15 kips and single axle loads up to 28.2 kips, which were greater than those of most commercial truck traffic in Ohio. When pavement temperatures were approximately 32°F (0°C), strain did not

exceed $35x10^{-6}$ at any load or speed. In warm weather months with pavement temperatures up to 126°F (52°C), strain values up to $80x10^{-6}$ were measured at the bottom of the HMA layers at high speeds (45 and 55 mph). Strain was higher (maximum of $128x10^{-6}$) during 5 mph tests. [*9, 10*]

Other states with environmental and traffic conditions similar to those in Wisconsin have also constructed perpetual pavements. Many of these pavements are planned to be monitored under research studies but have not yet published conclusive results. [*11, 12*]

2.3 Previous Wisconsin Perpetual Pavement Experience - STH 50

Prior to the construction of the test sections for this study, a perpetual pavement had been constructed on Wisconsin's state trunk network. The project, constructed in 2000, was located on state trunk highway (STH) 50 in Kenosha and Walworth Counties. The 6.9-mile project extended from 381st Avenue in the east to U.S. Highway 12 in the west. The road is a 4-lane divided freeway; test sections were constructed in the westbound lanes.

The construction year (2000) average daily traffic (ADT) was 7,285, and the design year (2020) ADT was 9,185. With 10.6 percent truck traffic, the equivalent single axle load (ESAL) level for this 20-year period was approximately 2 million. While this represents fewer ESALs than a typical perpetual pavement is designed for, valuable information was still obtained from these sections.

Four 1,000-ft test sections were constructed, each with 9 inches of HMA pavement. Two control sections were defined within the planned mainline pavement (7-in HMA). Paving was completed in three layers. All sections were constructed on 4 inches of open graded base course over 8 inches of dense graded crushed aggregate base course. The test sections compared various asphalt cement (AC) binder grades and air void contents. The AC content was constant for all sections. A layout of the test and control sections is shown in Figure 1, and descriptions of each section are provided in Table 1.

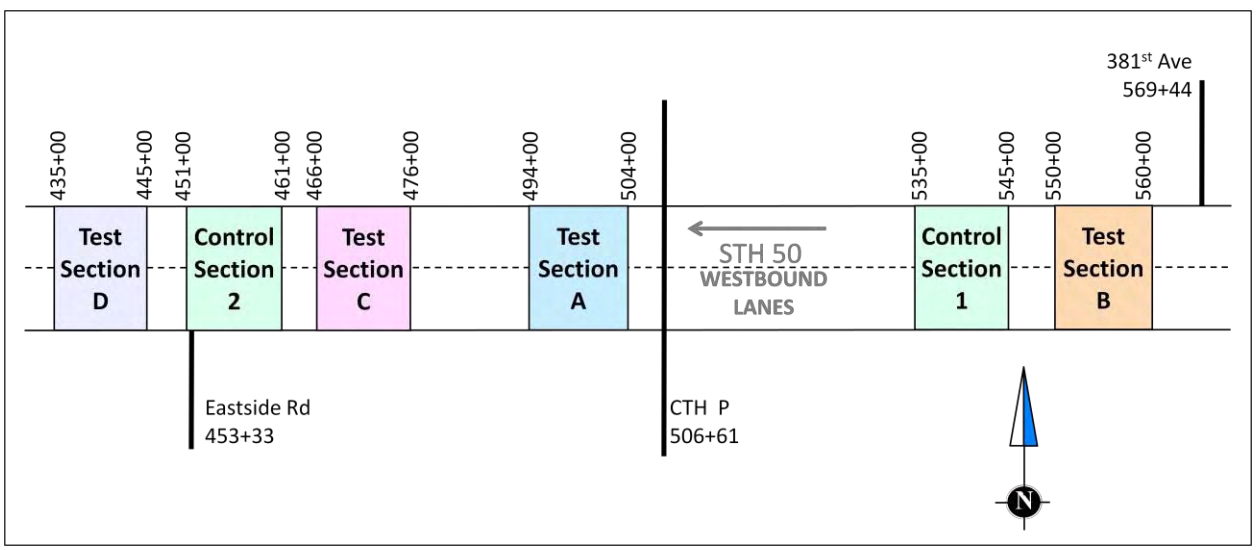


Figure 1. STH 50 perpetual pavement test section layout.

Table 1. STH 50 Perpetual Pavement Test Section Design

Section	HMA Layer	Layer Thickness (inches)	Total Thickness (inches)	Binder	Design Air Voids (%)	Notes
Test Section A	Surface Middle Bottom	2 3.5 3.5	9	PG 58-28 PG 58-28 PG 58-28	8 8 8	Standard WisDOT 9-in HMA pavement.
Test Section B	Surface Middle Bottom	2 3.5 3.5	9	PG 58-28 PG 64-22 PG 64-22	6 6 4	Designed by Dr. Marshall Thompson, University of Illinois.
Test Section C	Surface Middle Bottom	2 3.5 3.5	9	PG 64-28 PG 58-28 PG 58-28	6 6 4	Designed by WisDOT. Standard lower layers, stiff upper layer.
Test Section D	Surface Middle Bottom	2 3.5 3.5	9	PG 58-28 PG 70-22 PG 70-22	6 6 4	Designed by WAPA.* Stiff lower layers, standard upper layer.
Control 1	Surface Middle Bottom	1.5 2.75 2.75	7	PG 58-28 PG 58-28 PG 58-28	8 8 8	Standard WisDOT 7-in HMA pavement.
Control 2	Surface Middle Bottom	1.5 2.75 2.75	7	PG 58-28 PG 58-28 PG 58-28	8 8 8	Standard WisDOT 7-in HMA pavement.

*WAPA: Wisconsin Asphalt Pavement Association

Falling weight deflectometer (FWD) testing was performed on the test sections in 2000, 2001, and 2005. FWD testing was performed in the right wheel path of the driving lane. Three test loads were used: 5, 9, and 12 kips. Strain values computed from the 9-kip drop load test data are presented in Table 2. Strain was calculated at the bottom of the HMA pavement layer. As expected, the strains in the 9-in test sections were consistently lower than in the 7-in control sections. High strains were noted in some test sections in 2000; this was likely due to the fact that FWD testing was performed for these sections directly after paving of the third layer, when the pavement was still very warm and therefore less resistant to deflection during loading. Variation in temperature was also the likely cause of differences in strain levels among test years: higher temperatures result in higher strain at the bottom of the pavement layer (temperature information was not available for the test dates). Test sections B and D, with stiff lower layers, typically had lower strains than the other test sections. Strains computed in the test and control sections in 2005 were well below the 70×10^{-6} endurance limit proposed for perpetual pavements.

Table 2. STH 50 Strain at Bottom of HMA Pavement Layer, 10^{-6}

Section	2000	2001	2005
Test Section A	140	88	41
Test Section B	64	78	36
Test Section C	314	104	46
Test Section D	52	84	39
Control Section 1	N/A	122	69
Control Section 2	N/A	130	60

Visual pavement surveys were conducted in 2005, 2007, and 2008. Distresses recorded during these surveys are shown in Tables 3 through 5. A blank space in these tables indicates that no distresses were recorded. The most prevalent distress recorded in all sections was distress of the longitudinal paved joint. This distress is a function of the paving operation, however, and is not directly related to the perpetual pavement performance. Transverse cracking was noted in all sections starting in 2007 but was worst in the two control sections. Test section C, with PG 64-28 binder and 6 percent air voids in the surface layer, had the fewest transverse cracks in 2008. The cracks were low-severity according to WisDOT's pavement distress index (PDI) categories. It is not known whether the cracks extended from the surface downward, or from the bottom up. No pavement rutting was noted, and the ride quality in all sections was very good.

Table 3. STH 50 Visual Pavement Survey Results, 2005

Section	Longitudinal cracking (total feet)	Distressed longitudinal paved joint (total feet)	Transverse cracking (number)
Test Section A			
Test Section B			
Test Section C	8		
Test Section D			
Control Section 1			
Control Section 2		50	

Table 4. STH 50 Visual Pavement Survey Results, 2007

Section	Longitudinal cracking (total feet)	Distressed longitudinal paved joint (total feet)	Transverse cracking (number)
Test Section A		66	8
Test Section B	2		3
Test Section C	3	100	2
Test Section D		120	7
Control Section 1		445	12
Control Section 2		224	11

Table 5. STH 50 Visual Pavement Survey Results, 2008

Section	Longitudinal cracking (total feet)	Distressed longitudinal paved joint (total feet)	Transverse cracking (number)
Test Section A		370	8
Test Section B	2		6
Test Section C	9	930	3
Test Section D		120	7
Control Section 1		683	12
Control Section 2		537	12

All test sections on the STH 50 project were performing well as of 2008, with the perpetual pavement sections showing less load-induced strain and fewer transverse cracks than the standard pavement sections. The test sections with stiff lower layers (B and D) showed the lowest load-induced strain levels, while the test section with PG 64-28 binder in the upper layer (C) had the fewest transverse cracks.

3. Study Description

3.1 Motivation

Based on the trial performance of the perpetual pavement on STH 50 in Wisconsin and on the reported performance of perpetual pavements worldwide, WisDOT developed a research work plan for the construction, testing, monitoring, and evaluation of a second perpetual pavement. This project was to be located on the entrance ramp to I-94 from the Kenosha Safety and Weigh Station Facility in Southeastern Wisconsin. This research endeavor was approved by the Federal Highway Administration (FHWA) as a Federal Experimental Project (FEP). It was conducted as a partnering effort between WisDOT, FHWA, and the Wisconsin Asphalt Pavement Association (WAPA).

When this research effort began, there was very little perpetual pavement performance data from Wisconsin roadways. Evaluation of a perpetual pavement at the weigh station facility would help fill this information gap and improve WisDOT's ability to provide more cost-effective pavement design alternatives.

3.2 Objectives

The primary objective of this study was to develop a design philosophy for longer-lasting pavements, thus reducing user delays that are inevitable during rehabilitation and reconstruction operations. Other objectives included:

1. Evaluate the theory of perpetual pavements,
2. Analyze different pavement layer materials,
3. Provide data that can be used to develop and/or validate perpetual pavement guidelines, and
4. Determine the preferred design methodology for perpetual pavements in Wisconsin.

Results from this study were combined with other findings to develop guidelines for the proper selection and design of HMA perpetual pavements.

3.3 Project Location

The site chosen for the perpetual pavement evaluation was the entrance ramp to I-94 westbound from the Kenosha Safety and Weigh Station Facility in Kenosha County (Figure 2a). I-94 physically runs north and south in this area; the entrance ramp traffic direction is northbound. The weigh station is located in the far southeastern corner of the state at county trunk highway (CTH) ML, just north of the Illinois state line (Figure 2b). This section of I-94 consists of three travel lanes in each direction and is the main thoroughfare between Milwaukee and Chicago. Truck traffic is heavy along this corridor, and trucks must pass through the weigh station when it is open. This site was therefore ideal because heavy truck traffic would accelerate the perpetual pavement loading, and because the pavement could be monitored and tested on days when the weigh station was closed.

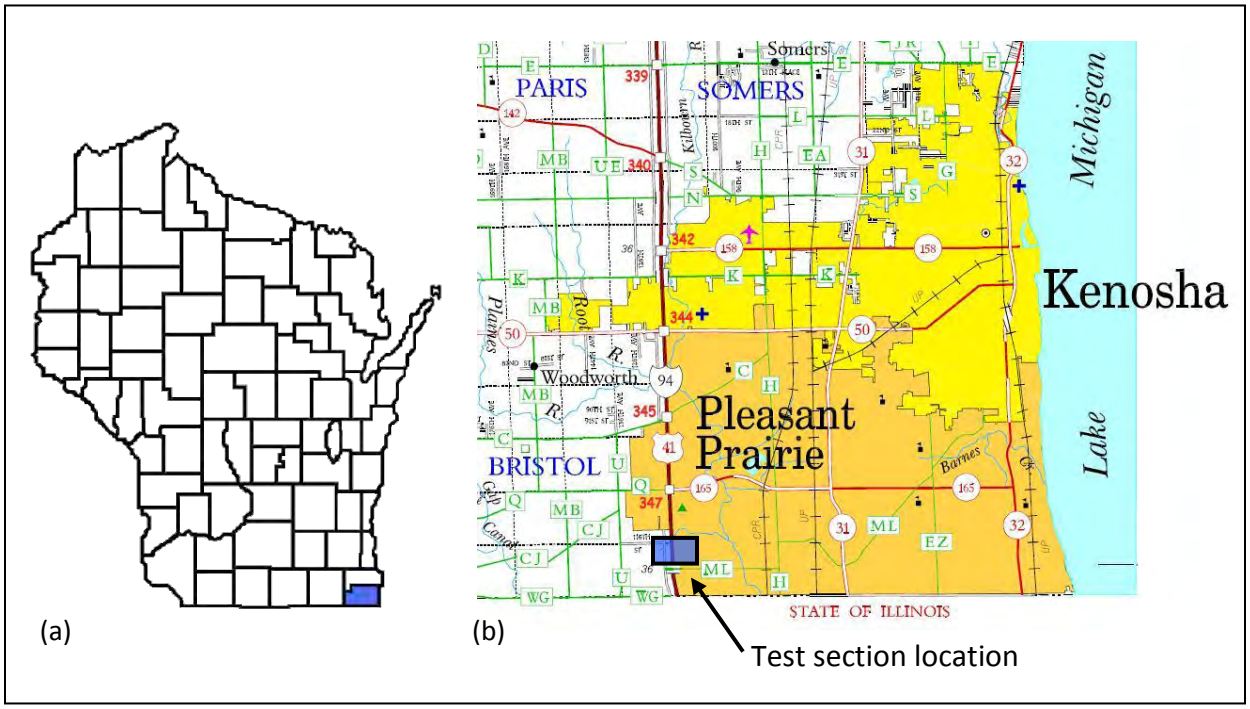


Figure 2. Test section location: (a) Kenosha County; (b) location of test section on I-94 westbound.

3.4 Site Conditions

3.4.1 Traffic

The 2003 construction year ADT for the weigh station entrance ramp was 9,476, and the 2023 design year ADT was 14,900. Based on a truck percentage of 100 percent, the total ESALs for the 20-year design period were over 75 million.

3.4.2 Soil

Soil information used in the perpetual pavement design was obtained from the USDA Soils Conservation Service's "Soil Survey of Kenosha and Racine Counties Wisconsin." The predominate soils found along the project were of the Morley-Beecher-Ashkum association and were characterized as well-drained to poorly drained soils that had a silty clay or silty clay loam subsoil. These soils typically had an AASHTO classification of A-4, A-6, or A-7.

For flexible pavement design, WisDOT uses the Design Group Index (DGI), a subjective value related to the standard group index, soil silt content, frost action potential, and pavement performance observations. DGI values range from 0 to 20, with low values representing better subgrade soils. The

DGI for the soils along this project site was 10, indicating a medium-strength soil. The soil frost index was F-3, and the soil support value (SSV) was approximately 4.5.

3.5 Perpetual Pavement Design

Design of the perpetual pavement in this study was a joint effort among WisDOT, the HMA industry, and academia. Two unique test sections were designed based on material cost data and the STH 50 project described in Section 2.3. The final test section designs are shown in Table 6 and Figure 3, and the design methodology is described below.

The HMA mixture for all layers was specified as E-30x, which defines mixture property requirements for pavements designed for greater than 30 million ESALs. [*13*] The mixture gradation for the top layers was 12.5 mm (0.5 in) nominal size, and for the lower layers was 25.0 mm (1.0 in) nominal size, as specified in the WisDOT Standard Specifications. [*13*]

As noted in the STH 50 study, the perpetual pavement sections that exhibited the lowest load-induced strains had relatively stiff lower layers. Therefore, PG 64-22 binder was chosen for the bottom asphalt layers in both test sections for the Kenosha weigh station study. To evaluate the relationship between asphalt binder content and HMA tensile strain, two air void percentages were chosen for the test sections: 4 and 6 percent. The 4 percent air voids in test section 1 was to be achieved by increasing the binder content in that layer. The design thickness for the bottom layer was 4.5 inches.

Two surface binder grades (PG 76-28 and PG 70-28) were used to study rutting properties and determine if the extra cost of the PG 76-28 binder was warranted. The different surface layer binders were also used to evaluate whether a stiffer surface layer distributed traffic loads through the pavement structure differently (i.e. resulted in different strain levels). Six percent air voids was specified for both surface layers. The surface layers were designed to be 2 inches thick.

The pavement design for the middle layer was identical for both test sections. The binder used was PG 70-22, and 6 percent air voids was specified. The middle layer was also designed to be 4.5 inches thick. The perpetual pavement sections were constructed on 4 inches of open graded base course over 17 inches of dense graded crushed aggregate base course.

Table 6. Kenosha Weigh Station Test Section Pavement Design

Test Section	Layer	Thickness (in)	Gradation	HMA Mix Type	Binder	In-place Air Voids
1	Surface	2	12.5 mm (0.5 in)	E-30x	PG 76-28	6%
	Middle	4.5	25 mm (1.0 in)		PG 70-22	6%
	Bottom	4.5	25 mm (1.0 in)		PG 64-22	4%
2	Surface	2	12.5 mm (0.5 in)	E-30x	PG 70-28	6%
	Middle	4.5	25 mm (1.0 in)		PG 70-22	6%
	Bottom	4.5	25 mm (1.0 in)		PG 64-22	6%

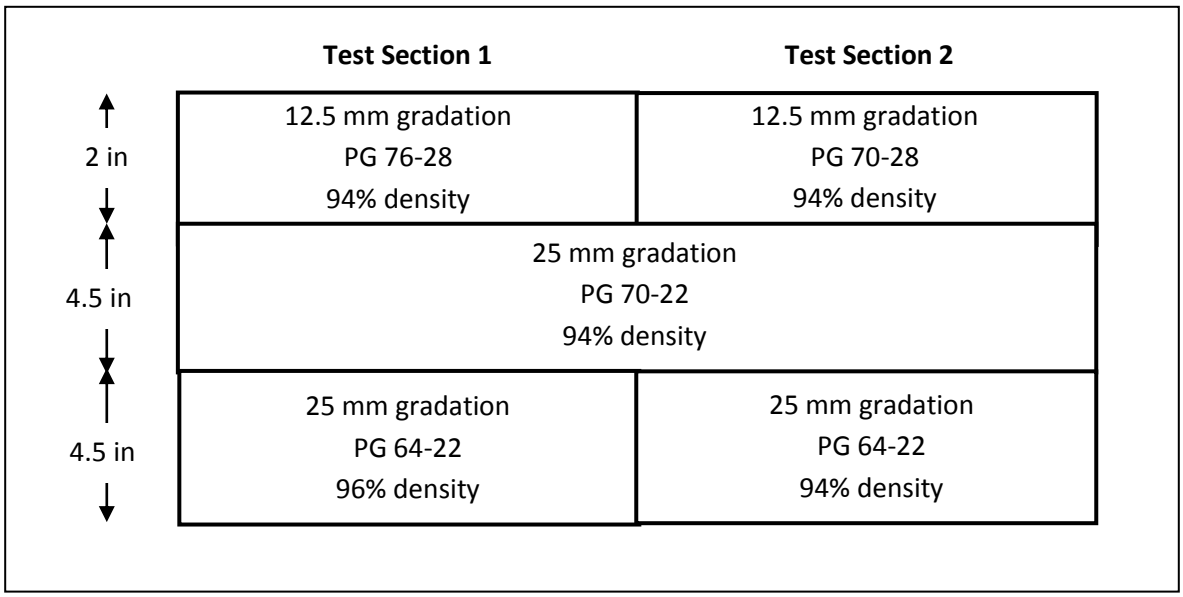


Figure 3. Test section cross sections. (Not to scale)

3.6 HMA Mixture Design

Two mixture designs created by the paving contractor were used for the test section pavement: 9 inches of a 25.0 mm gradation (two 4.5-in layers) for the bottom and middle layers, and 2 inches of a 12.5 mm gradation for the surface layer. The mixture designs were dense-graded aggregate structures both using local limestone and manufactured sands with a known performance history. Material source information is provided in Table 7, and aggregate gradation plots are shown in Figures 4 and 5. Both design gradations fall within WisDOT specification limits. [*13*] The 12.5 mm gradation line passes through the "caution zone," but this is not uncommon for Wisconsin mix designs.

Table 7. HMA Mixture Material Source Information

Material	**Source**	**Source Location**
#1 Stone	Warrenville Quarry	S4 T2N R19E Racine Co.
7/8" Chip 5/8" Chip 3/8" Chip	Franklin Quarry	S10 T5N R21E Milwaukee Co.
Manufactured Sand	Honey Creek Pit	S6 T3N R19E Racine Co.
PG 64-22 Binder PG 70-28 Binder	Construction Resources Management	301 E. Washington St, Milwaukee, WI
PG 70-22 Binder PG 76-28 Binder	Seneca Petroleum Company (SBS modified)	13301 S. Cicero Ave, Crestwood, IL

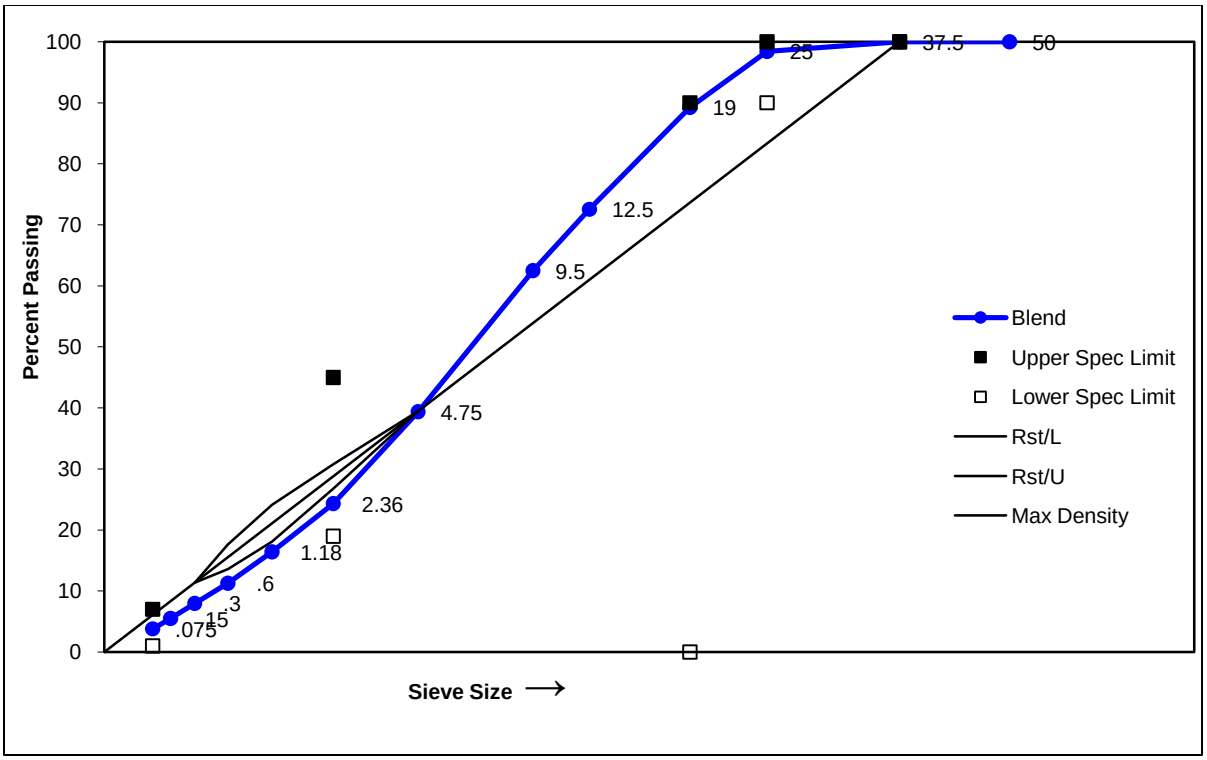


Figure 4. Aggregate gradation chart, 25.0 mm mix.

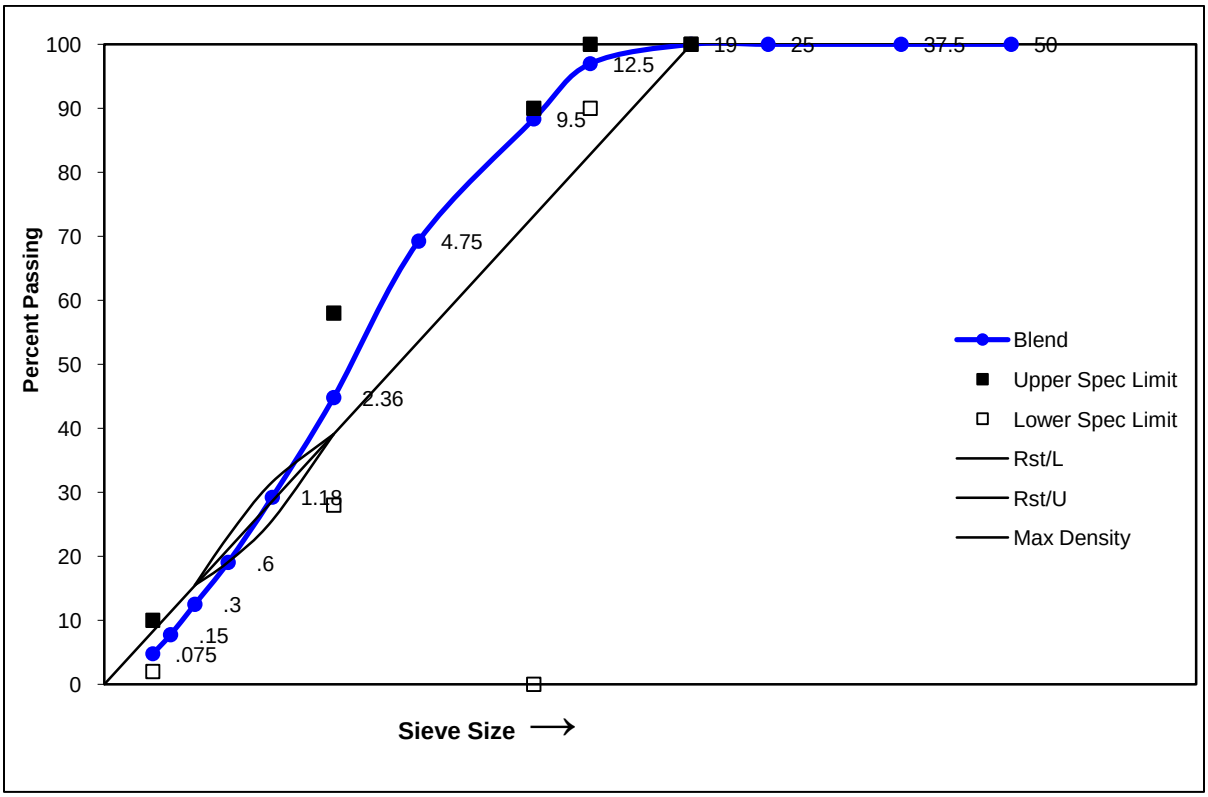


Figure 5. Aggregate gradation chart, 12.5 mm mix.

To create the "rich" bottom layer in test section 1 and achieve the desired 4 percent in-place air voids (96 percent density), it was agreed that the asphalt binder content would be increased from the original design target for the 25.0 mm mixture, from 4.5 to 4.7 percent. Similarly, the original design target asphalt binder content for the 12.5 mm mixtures was increased from 5.4 to 5.6 percent. It was also decided that the gyration level for all mixtures targeting 6 percent air voids would be decreased for field quality control purposes. To establish the new job mix formula targets for production control, the volumetric properties were back calculated using the original mixture design trial data, which were generated with a design gyration level (N_{des}) of 125, to correlate with a N_{des} of 100.

3.7 Test Section Layout

The two perpetual pavement test sections were constructed end-to-end on the weigh station entrance ramp. There was no control section. Test section 1 was constructed 1,840 ft (560 m) long, and test section 2 was 1,110 ft (340 m) long. The remainder of the entrance ramp and other weigh station service roads were built with 11 inches of Portland cement concrete (PCC) pavement. Figure 6 depicts the layout of the weigh station facility and test sections.

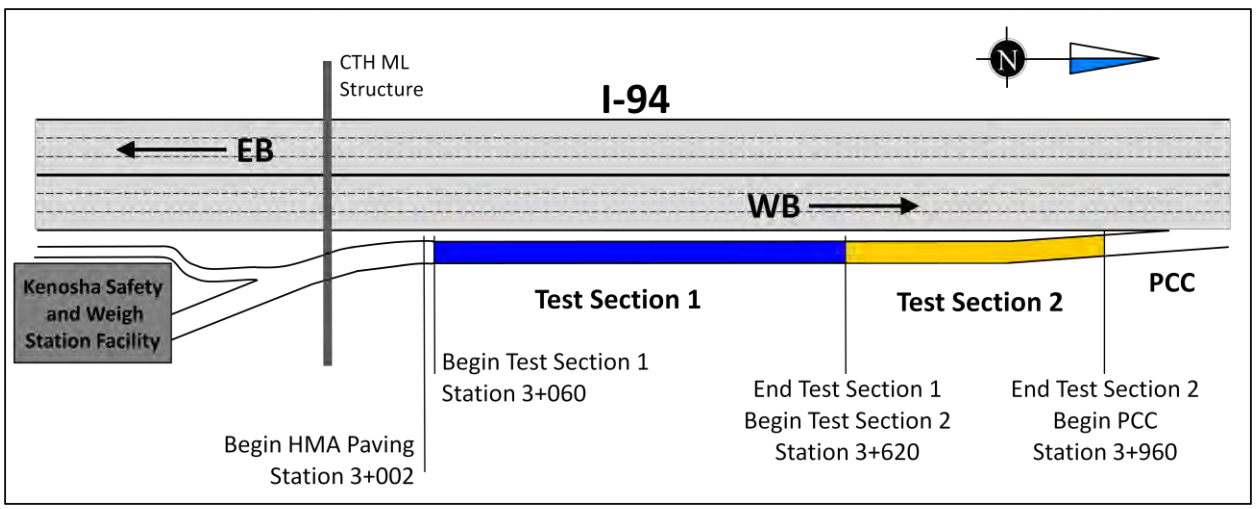


Figure 6. Test section layout. Note: metric stationing

4. Construction

4.1 Overview

Ramp construction at the Kenosha Safety and Weigh Station Facility was completed in 2003 under state project I.D. 1032-05-01. Paving of the test sections was completed by Payne and Dolan, Inc., of Waukesha, WI. HMA material was also provided by Payne and Dolan, Inc. The asphalt plant, located in Racine, WI, was approximately 15 miles northeast of the project site. The HMA plant was a stationary Gencor counter-flow drum plant with a 500 ton per hour capacity. With eight cold feed bins and two recycled asphalt pavement (RAP) bins, the plant proportioned the HMA material using one weigh bridge for the virgin aggregate and a separate weigh bridge for the RAP. The plant was equipped with two horizontal and one vertical binder storage tanks. HMA was stored in one of four 200-ton capacity insulated silos, where it was loaded into quad axle and live bottom dump trucks and transported to the project (approximate 20-minute haul time).

Payne and Dolan paved the test sections with a Blawknox PF3200 paver using full automatic controls with sonic tracking skis. The following equipment was used for compaction:

1. Ingersoll-Rand DD-130 vibratory compactor,
2. Hypac 778 vibratory compactor,
3. Bomag BW151 vibratory roller, and
4. Caterpillar PS130B pneumatic rubber tire roller.

4.2 HMA Mixture Production and Placement

Construction of the perpetual pavement test sections occurred on August 5, 7, and 8, 2003, with one layer of HMA placed per day. Temperatures were average for the month of August in Wisconsin, with cool mornings ranging from 57 to 59°F followed by warm afternoons ranging from 73 to 79°F. This caused some uniformity challenges for achieving the desired in-place density, which will be discussed later in the report.

Mixture production quality control (QC) followed standard WisDOT Quality Management Program requirements. Production testing laboratories were required to be WisDOT qualified and to be on location at the plant site. WisDOT certified personnel (department and contractor) performed the necessary production sampling and testing. These test methods are listed in Table 8.

Table 8. Test Methods for HMA Production Sampling and Testing

Test Procedure	ASTM	AASHTO	WisDOT
Field Solvent Extraction Method for Determining the Aggregate Gradation of HMA Samples			1560
Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures (G_{mm})		T 209	
Bulk Specific Gravity of Compacted Asphalt Mixtures Using Saturated Surface-Dry Specimens (G_{mb})		T 166	
Preparing and Determining the Density of HMA Specimens by Means of the Superpave Gyratory Compactor		T 312	
Percent Air Voids in Compacted Dense and Open Asphalt Mixtures (V_a)		T 269	
Materials Finer Than 75-µm (no. 200) Sieve in Mineral Aggregates by Washing		T 11	
Sieve Analysis of Fine and Coarse Aggregates		T 27	
Density of Bituminous Concrete in Place by Nuclear Methods	D 2950		

4.2.1 Sample Collection

HMA production samples were collected at the Racine plant from the back of loaded trucks. Samples were split, and additional material was retained for further research and related testing. Sample tonnages collected are shown in Table 9.

Table 9. HMA Sample Tonnages

Date	Sample Tonnage	Time of Day
25.0 mm Mixture		
8/5/2003	325	9:15
	927	13:00
8/7/2003	256	8:30
	876	11:30
	1688	14:30
8/8/2003	538	N/A
12.5 mm Mixture		
8/8/2003	254	12:10
	733	14:00
8/9/2003	190	7:00
8/15/2003	87	12:00

4.3 In-Place Density Measurements

During paving, achieving the targeted density levels was problematic. To reach the design density in the bottom layer of test section 1 (96 percent density/4 percent air voids), the initial design called for additional binder to enhance the compactive effort. However, QC air voids measured low and would not allow for that option. The final solution was to pull dust at the plant. As dust was pulled, air voids increased, and additional binder could be added to achieve higher density levels. For the 25.0 mm mix, the adjusted asphalt content was bumped from 4.7 to 4.8 percent. In the 12.5 mm mix, the binder content was bumped from 5.6 to 5.8, and then to 6.0 percent. Production volumetrics are provided in Appendix A.

Density measurement locations were randomly selected using ASTM D 2950. [*14*] Calibration of the nuclear density testing equipment was verified daily. QC values were measured by the contractor, and quality verification (QV) values were obtained by WisDOT. Plots of in-place density measurements are shown in Figures 7 through 9 for each HMA layer and test section. Specific density values are provided in Appendix B. Density in the middle layer was in the target range: the average readings were 94.6 and 94.2 percent for test sections 1 and 2, respectively. The density was low, however, for the lower and upper layers. The average measurements for the lower layer were 94.0 and 93.1 percent for test sections 1 and 2, respectively. The average measurements for the upper layer were 92.3 and 91.3 percent for test sections 1 and 2, respectively.

Low in-place density might have been due to low temperatures in the mat during compaction (temperature was not monitored), or due to stiffness of the base course layer, which affects the compactive effort of layers paved above. The low in-place density would allow for additional compactive action by truck traffic, but was not exactly as designed for the perpetual pavement test sections.

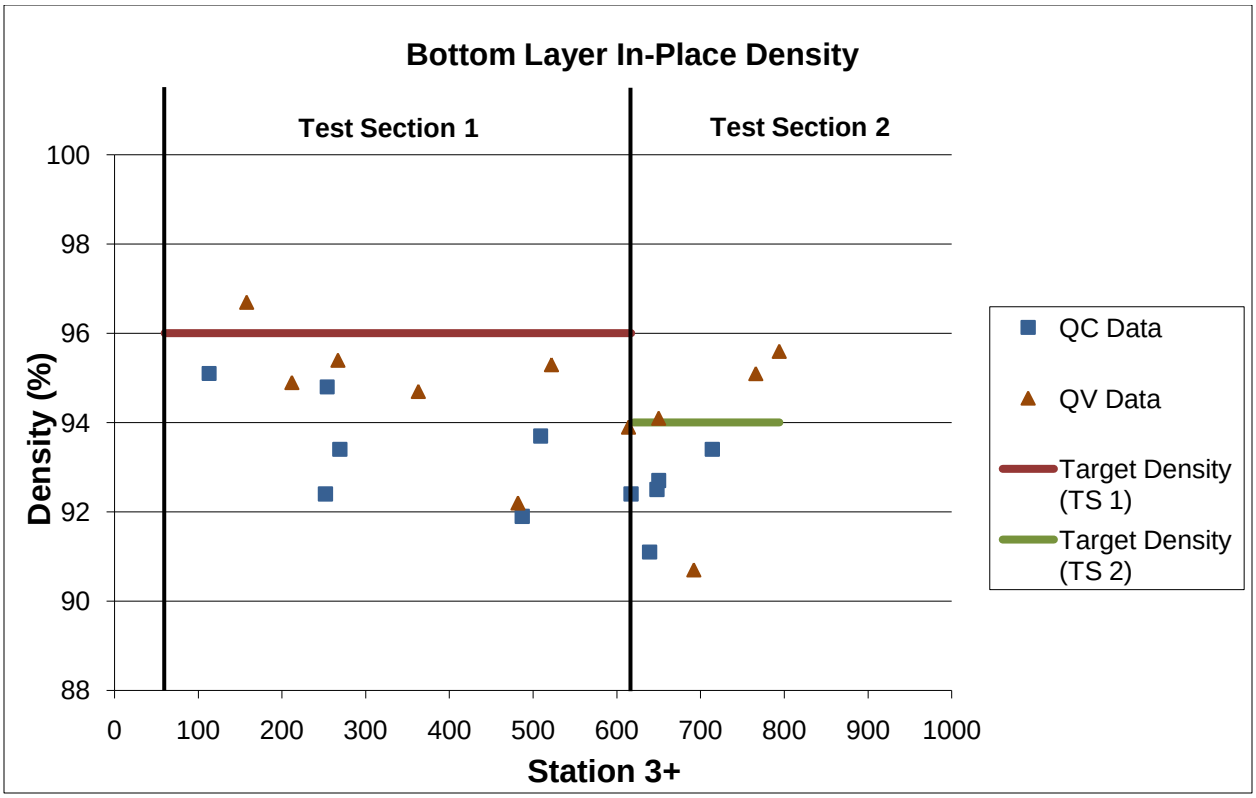


Figure 7. In-place density measurements, bottom HMA layer. Note: metric stationing.

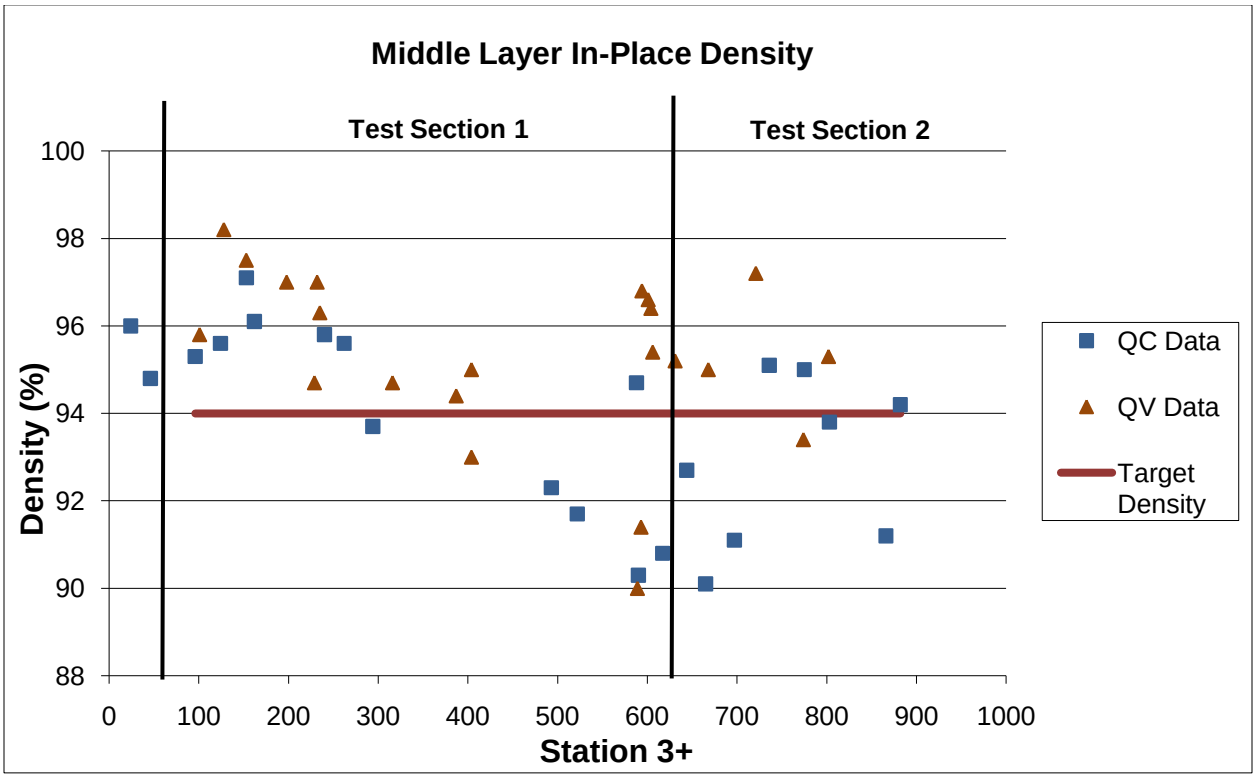


Figure 8. In-place density measurements, middle HMA layer. Note: metric stationing.

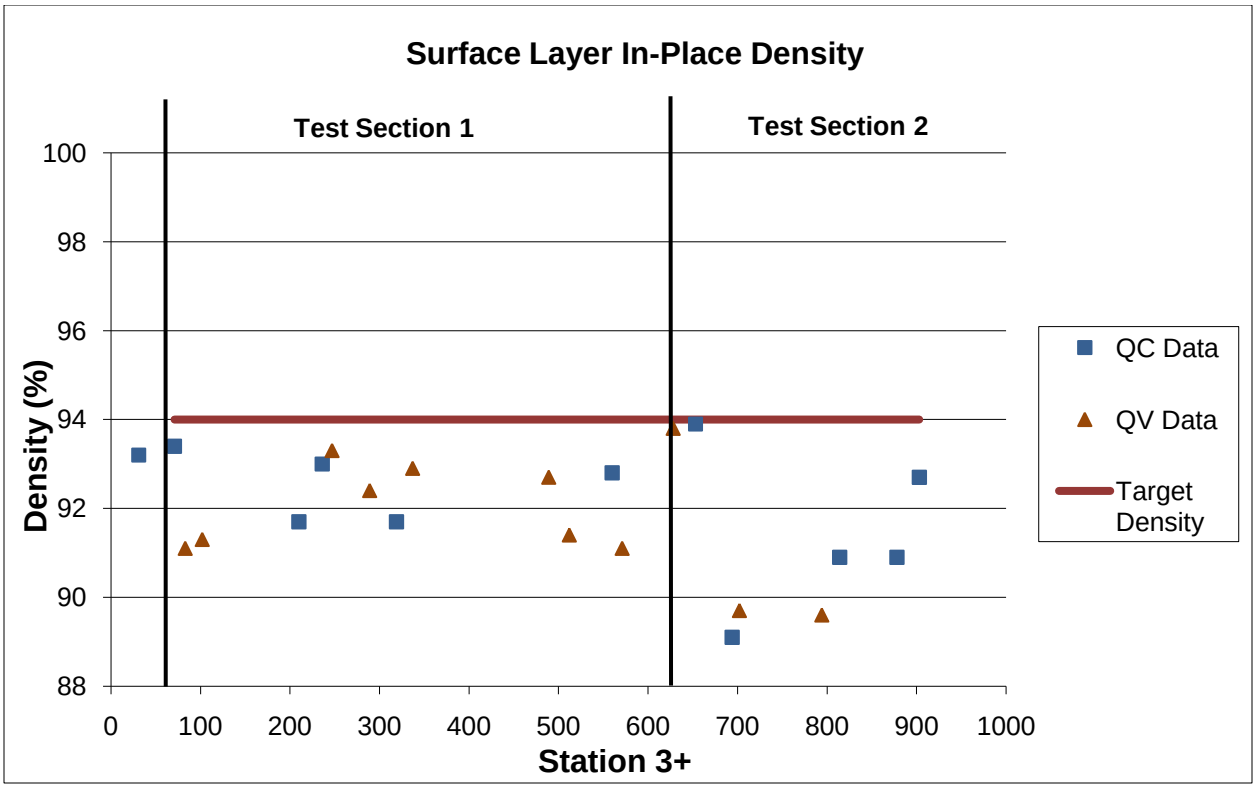


Figure 9. In-place density measurements, surface HMA layer. Note: metric stationing.

4.4 Other Construction Observations

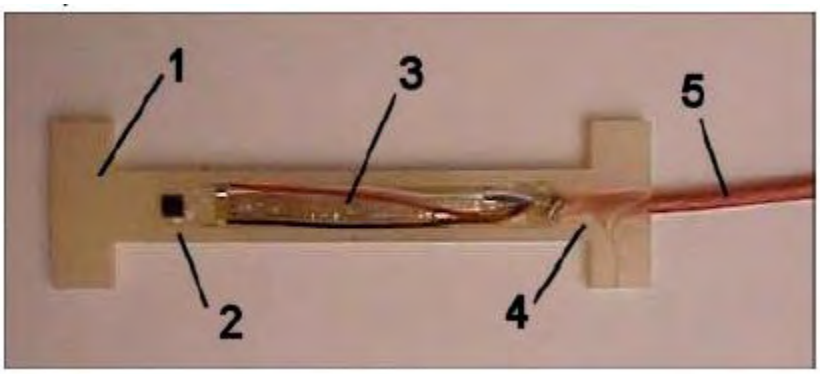
At paving start-up, a problem was encountered in meeting the lower layer design thickness. There was uncertainty in the placement depth of the bottom layer to achieve the 4.5-in compacted depth. Therefore, the beginning of test section 1 was moved ahead, to station 3+060, where the design thickness was achieved.

During paving, there was observed segregation at the outer edge of the pavement and transversely across the pavement (equally-spaced chevrons). This issue was brought to the attention of the contractor, and efforts were made to correct the problem. However, without the use of a shuttle buggy, segregation occurred along the entire length of the project.

5. Strain and Temperature Sensor Installation

To better understand how the perpetual pavement performed under traffic loading, sensors were installed to measure strain levels in the pavement layers. WisDOT contracted with the Marquette University Department of Civil and Environmental Engineering to instrument the pavement with 16 strain and temperature sensors, and to collect data from the sensors at future dates.

Romus, Inc. designed and provided the sensors for this study. Each sensor consisted of a strain gage and a temperature sensor mounted on an adhesive-coated nylon substrate. This allowed the sensor to become adhered to the bottom of an individual layer of HMA. The temperature range of the gages was -100 to 200°F for continuous measurement. A labeled picture of the strain gage fixture is shown in Figure 10, and a schematic and wiring details are provided in Figure 11. Strain gage specifications are provided in Appendix C.



1. High-temperature nylon sensor body
2. Silicon temperature sensor
3. Strain gage
4. High temperature epoxy
5. High temperature cable

Figure 10. Strain gage fixture.

A total of 16 strain sensors were installed in the perpetual pavement. Eight sensors were installed at the interface between the base course and the bottom layer of HMA, and the remaining eight sensors were placed at the interface between the bottom and middle layers of HMA. Sensors were positioned in groups of four, with two sensors oriented longitudinally (in the direction of traffic flow), and two sensors oriented transversely (perpendicular to traffic flow). This created a redundant set of sensors at each installation location. All sensors were located in the plane of the right wheel path. Sensor orientations are depicted in Figure 12.

Initial plans called for both test sections to be instrumented with a set of strain sensors. However, because the test section limits were altered slightly during construction due to the construction issues described in Section 4, this was not possible. One set of sensors was installed prior to the start of test section 1, and the second set was installed prior to the start of test section 2 (Figure 12).

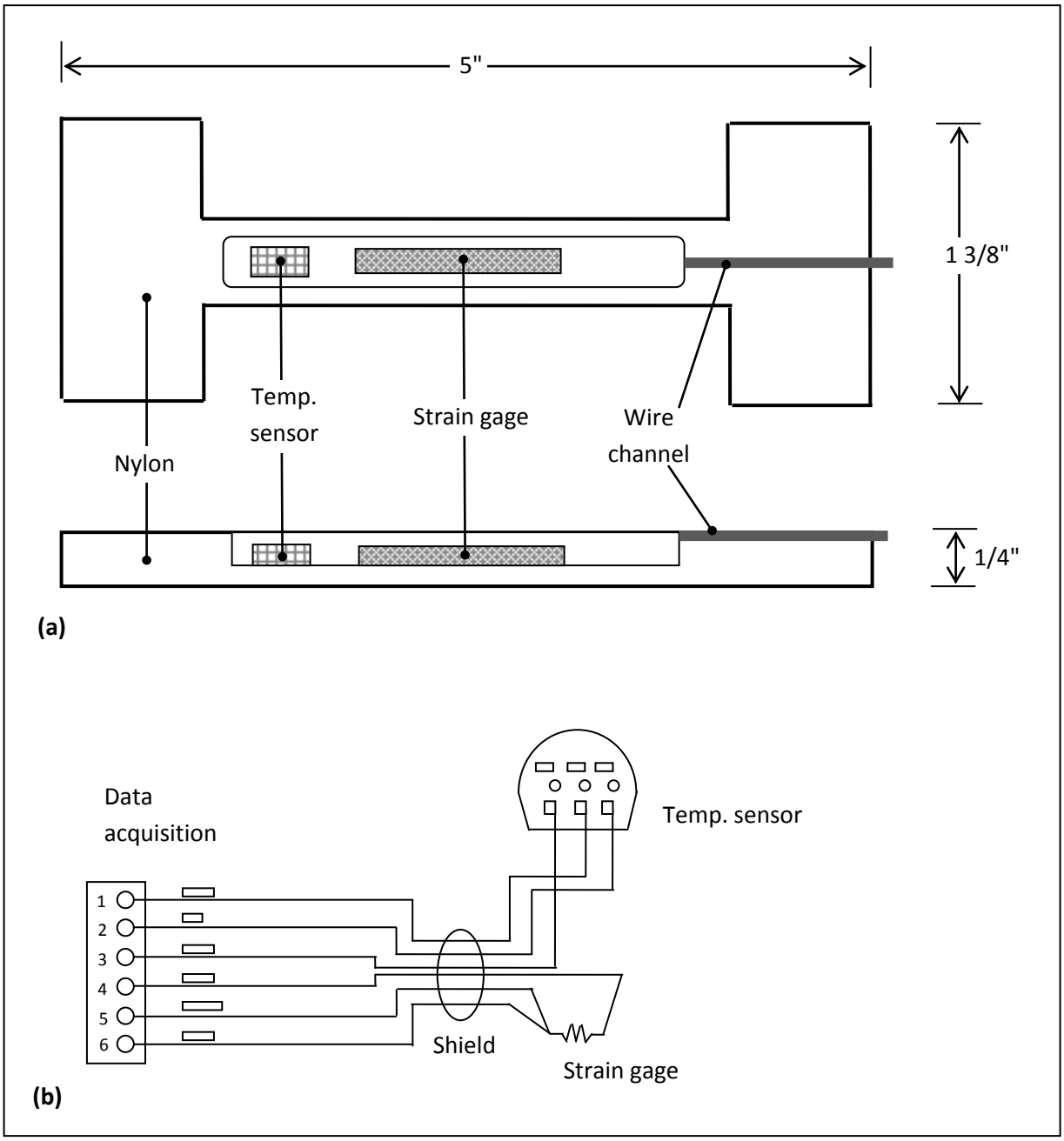


Figure 11. Strain gage (a) schematic diagram, and (b) wiring details.

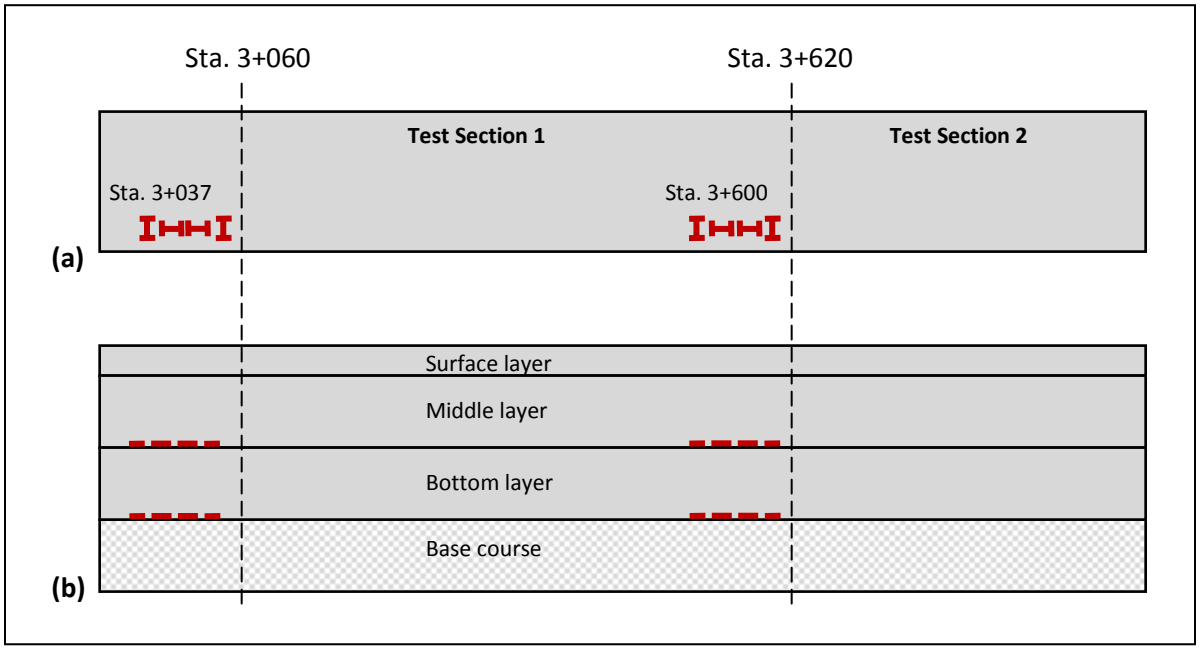


Figure 12. Strain sensor location: (a) plan view, and (b) pavement profile view.

Installation of the strain sensors took place during paving operations on August 5th and 7th, 2003. Marquette University staff coordinated the installation. Data cables were run from the sensors to the edge of the pavement, where the cables ran under a concrete barrier to a vault secured to the outside wall of the barrier. To prevent dislocation during paving, the sensors and data cables were secured with staples to the top of the aggregate base course and the top of the bottom layer of HMA. The cables were encased in protective plastic pipes from the edge of pavement to the vaults. The sensors were stabilized with one shovel of HMA just prior to paving over them. Figure 13 shows the strain sensors as they were installed during paving.

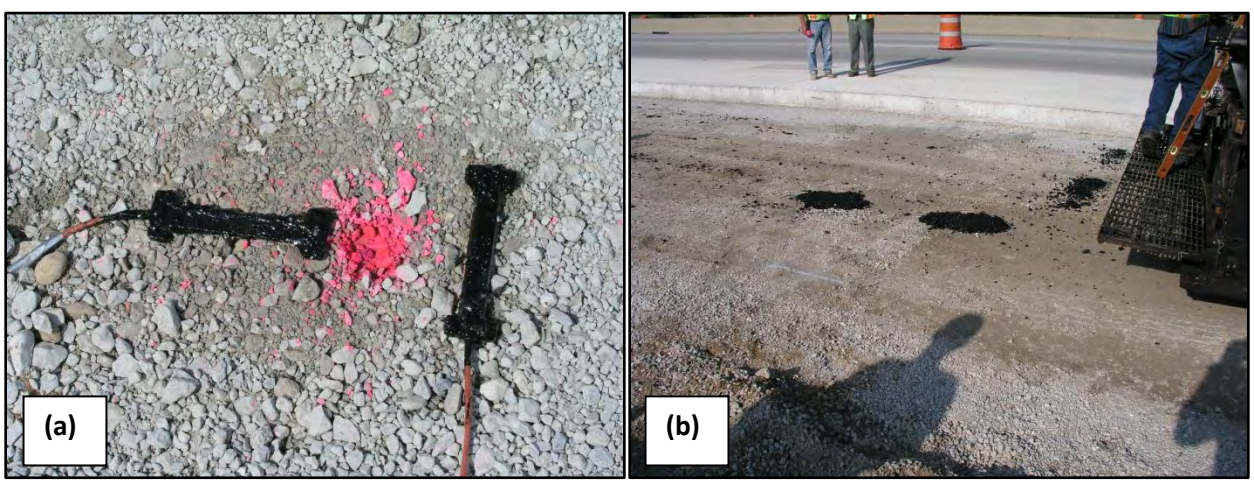


Figure 13. Strain gage installation (a) on top of base course, and (b) just prior to paving of the lower HMA layer.

On August 18th, Marquette University staff returned to the project site to test the strain and temperature sensors and collect preliminary strain data. Data sets were collected by connecting a laptop to the data cables located in the vaults on the outside of the concrete barrier.

Only 3 of the 16 installed sensors were functioning for both strain and temperature data acquisition. The surviving sensors were oriented longitudinally and were located at station 3+037 between the lower and middle HMA layers, at station 3+600 below the bottom HMA layer, and at station 3+600 between the lower and middle HMA layers. The cause of sensor failure was likely due to one or more of the following: inadequate strain relief in the sensor wires, insufficient shielding of the sensor wires, or excessive strains caused by construction loads. [*15*]

6. Pavement Material Data Analysis

6.1 Asphalt Binder Sample Test Results

Prior to paving, samples were collected from shipments of each asphalt binder grade used to construct this study's test sections. The samples were tested in the WisDOT materials testing laboratory according to the following test procedures:

- AASHTO T 240, Standard Method of Test for Effect of Heat and Air on a Moving Film of Asphalt Binder (Rolling Thin-Film Oven Test)
- AASHTO T 313, Standard Method of Test for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer (BBR)
- AASHTO T 315, Standard Method of Test for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)
- AASHTO T 316, Standard Method of Test for Viscosity Determination of Asphalt Binder Using Rotational Viscometer

These tests were used to determine whether the binder properties were within tolerance and fell into the temperature grade in which they were categorized by the supply company. [16] Results from the tests listed above are provided in Appendix D. All but one binder shipment fell into the temperature grade range specified, within allowable tolerances. This shipment of PG 70-22 was used in the paving operation. For a larger-scale HMA paving project, however, an out-of-tolerance binder test would be followed by increased testing frequency and a possible penalty.

6.2 HMA Simple Performance Tests

As part of NCHRP Project 9-19, it was proposed that a suite of three tests be used for the characterization of permanent deformation in HMA mixtures. These tests, part of the HMA Simple Performance Test (SPT) procedures, are as follows: flow time, flow number, and dynamic modulus. The

latter two tests were performed on samples of the HMA mixtures used in this study. Mixture samples were collected in the field during construction, and specimens were produced in the laboratory.

6.2.1 Flow Number Results

In the test for flow number (F_N), a cyclic axial load is applied to an HMA specimen. The load cycle consists of application of the load for 0.1 s, followed by a rest period of 0.9 s. The permanent strain of the specimen is measured at the end of each rest period. [*17*] A viscoelastic solid, such as HMA, follows a particular strain pattern under repeated loading. The strain pattern can be divided into three phases, as shown in Figure 14. In the primary flow phase, the rate of strain ($d\varepsilon/dt$) decreases. During secondary flow, the rate of strain is relatively constant. Tertiary flow commences when the strain rate begins to increase again, and generally continues to increase until specimen failure. The specimen's F_N is defined as the number of load cycles at which tertiary flow begins (Figure 14). Higher F_N indicates greater in-service rutting resistance.

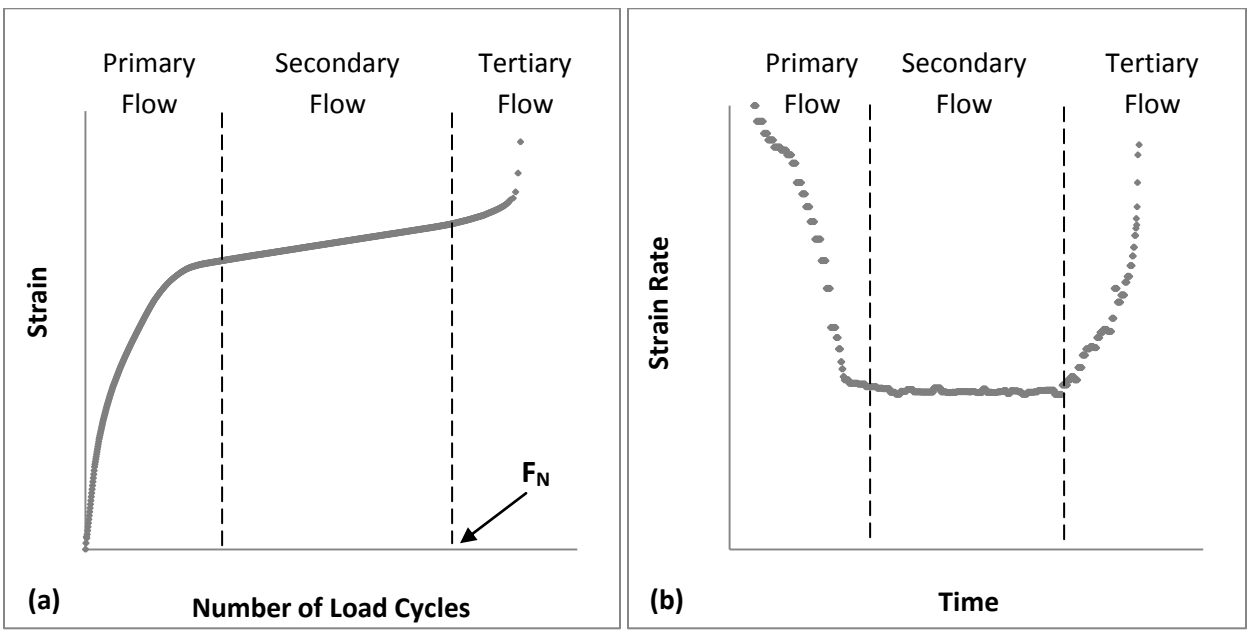


Figure 14. Demonstration of primary, secondary, and tertiary flow for (a) strain measurements and (b) strain rate measurements.

Samples from the perpetual pavement layers were tested for F_N at the FHWA Mobile Asphalt Pavement Mixture Laboratory (Mobile Asphalt Lab). Between four and seven samples were tested for each mixture design, and an average F_N value was calculated from a minimum of four samples. A test temperature was calculated for each pavement layer based on NCHRP Project 9-19 recommendations for permanent deformation testing. Test temperatures were calculated according to the following equation: [*17*]

$$T_{eff} = 30.8 - 0.12Z_{cr} + 0.92(MAAT + K_{\alpha}\sigma_{MAAT}) \qquad \text{Eq. 1}$$

where

T_{eff} = effective temperature for permanent deformation (°C)

Z_{cr} = critical depth for the mixture layer in question (mm)

$MAAT$ = mean annual air temperature (°C)

K_{α} = value computed from normal probability table based on designer's selected level of reliability

σ_{MAAT} = standard deviation of the mean annual air temperature.

The test temperatures calculated for the surface, middle, and lower layers were 113, 94.1, and 69.3°F (45.0, 34.5, and 20.7°C), respectively.

Results from F_N testing are shown in Figure 15. The middle and lower layer mixtures had the highest F_N (8500 to 8800), while the surface layer mixtures displayed lower F_N values (approximately 6650 and 2800 for test sections 1 and 2, respectively). The higher F_N values for the middle and lower layer mixtures indicate a greater resistance to rutting than the surface layer mixtures. This result is characteristic of a perpetual pavement system, where the surface layers are intended to reach their service life after 12-15 years and be replaced, while the lower layers must remain in service much longer. The mixture F_N for the surface layer was higher for test section 1 than test section 2, indicating that test section 2 would not have as great rutting resistance as test section 1. This is likely due to the fact that a stiffer asphalt binder was used in test section 1: PG 76-28 versus PG 70-28 in test section 2.

Permanent strain measurements were also recorded during F_N testing; these results are shown in Figure 16. Higher strain levels when tertiary flow is reached (F_N point) and at the end of the test generally indicate that a mixture has less resistance to rutting. However, it has also been proposed that accumulated permanent strain is a parameter that is more useful for comparing specimen quality within one mixture subset than for comparison among mixture types. [*18*] Therefore, more emphasis was placed on the F_N results (Figure 15).

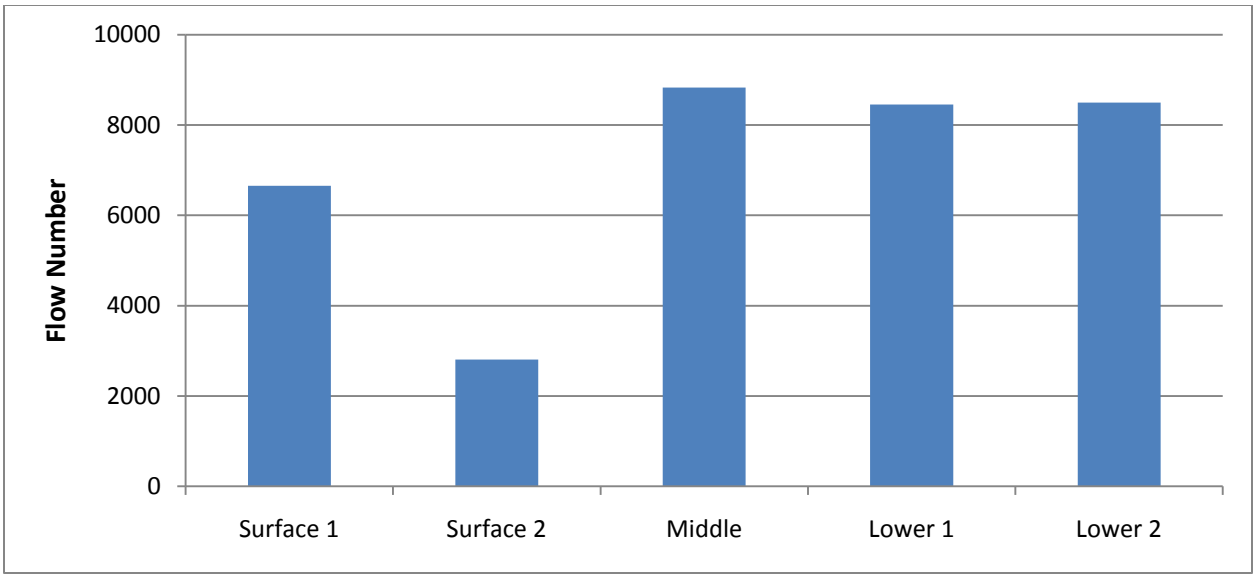


Figure 15. Flow number test results for the five perpetual pavement mixture types.

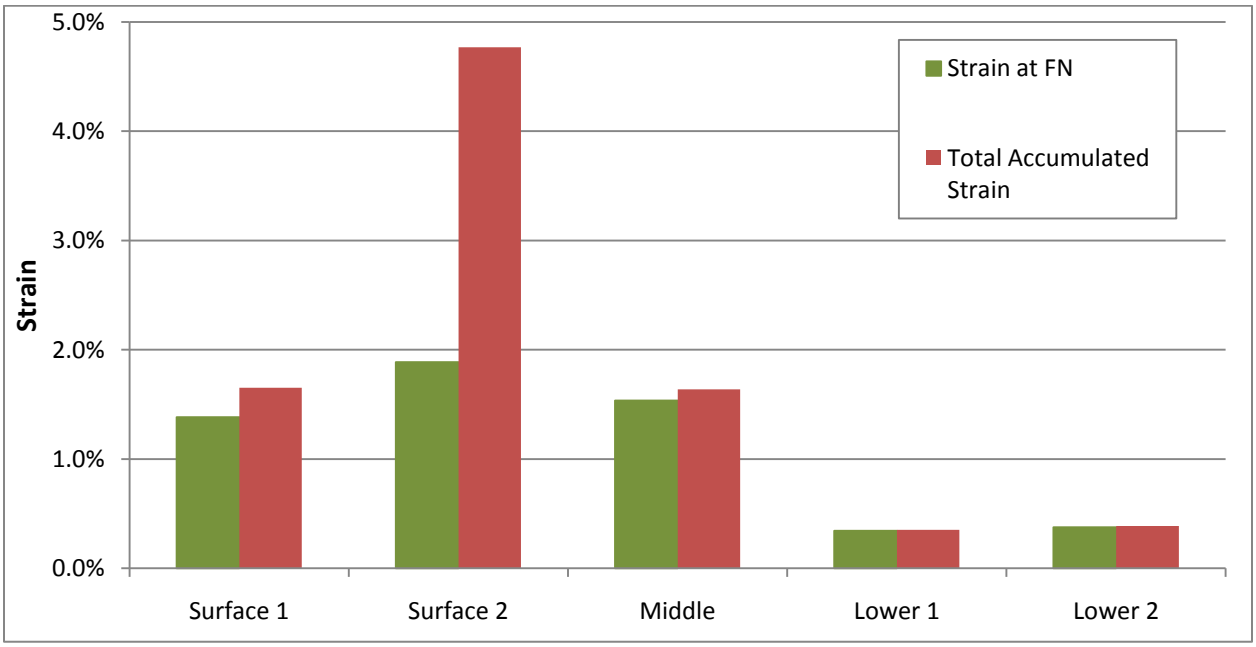


Figure 16. Accumulated strain test results for the five perpetual pavement mixture types.

6.2.2 Dynamic Modulus Results

In the test for dynamic modulus, a cyclic axial compressive load is applied to an HMA specimen. Stress and strain are measured, and the dynamic modulus is calculated as follows: [*17*]

$$|E^*| = \frac{\sigma_0}{\varepsilon_0} \qquad \text{Eq.2}$$

where

$|E^*|$ = dynamic modulus (Pa or psi)

σ_0 = amplitude of applied axial load (Pa or psi)

ε_0 = amplitude of resulting strain.

The cyclic loading is conducted at various frequencies and temperatures to monitor the specimen's behavior over a range of conditions. One specimen can be tested under a series of consecutive frequencies (frequency sweep testing). Specimens for this study were tested at frequencies of 0.1, 0.5, 1.0, 5.0, 10, and 25 Hz, and at temperatures of 60.6, 70.9, and 105°F (15.9, 21.6, and 40.6°C). A minimum of four specimens were tested at each temperature. Dynamic modulus testing was conducted at the FHWA Mobile Asphalt Lab. Data was analyzed by the North Central (NC) Superpave Center in West Lafayette, Indiana. The analysis was summarized in an unpublished report that is provided in Appendix E.

Average dynamic modulus results for each pavement layer are shown in Figure 17. These results are for frequency testing at 25 Hz. The two base layer materials were not tested at 105°F (40.6°C). Dynamic modulus test results can be correlated to the material's potential for permanent deformation (rutting), with rutting resistance increasing with |E*|. [*19*]

The |E*| values for the lower and middle HMA layers were higher than for the two surface layers at all test temperatures. This indicates that the lower and middle layers had more potential to resist permanent deformation (rutting) than the surface layers. The higher |E*| results might be due to the larger aggregate size used in the lower and middle layers. Long-lasting lower layers is important in a perpetual pavement system, as these layers are not meant to be replaced during rehabilitation efforts. The base layer in test section 1 slightly out-performed that for test section 2, but no statistical difference was found between test results for these layers.

With respect to the surface layers, |E*| values were higher for test section 1 than for test section 2, indicating better rutting resistance in test section 1. This is likely due to the stiffer asphalt binder grade used in test section 1.

Dynamic modulus testing could also be used to evaluate the fatigue cracking potential of HMA mixtures. However, a good correlation between these two parameters has not yet been identified. [*19*]

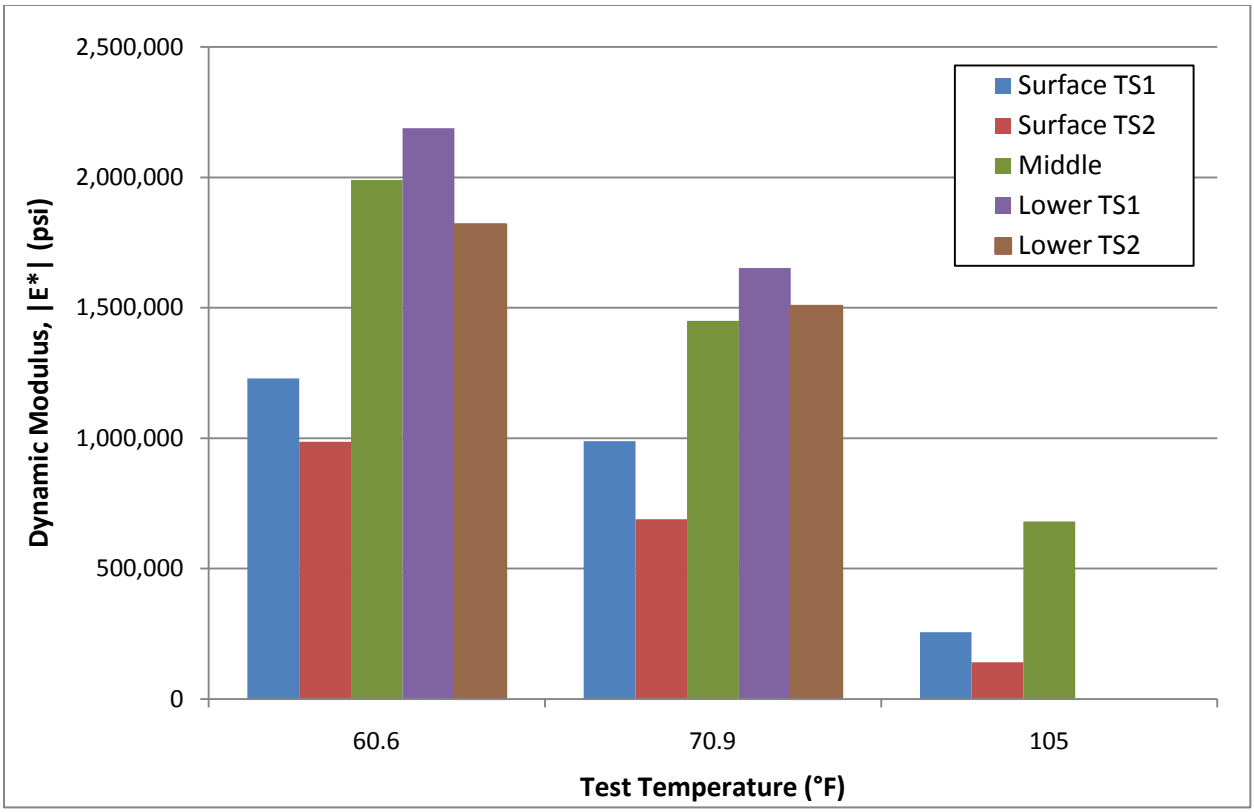


Figure 17. Average dynamic modulus test results, 25 Hz.

When results are available for |E*| testing at a minimum of three temperatures, results can be reduced to form a master curve showing |E*| versus test frequency. This curve is useful because a material's performance at various test frequencies is related to its behavior under different traffic speeds, with higher frequencies representing faster traffic speeds. Data was available to create master curves for the surface and middle HMA layers. These curves are shown in Figure 18. The middle layer was stiffest at all test frequencies. Comparing surface layers, the material from test section 1 was stiffer than that for test section 2.

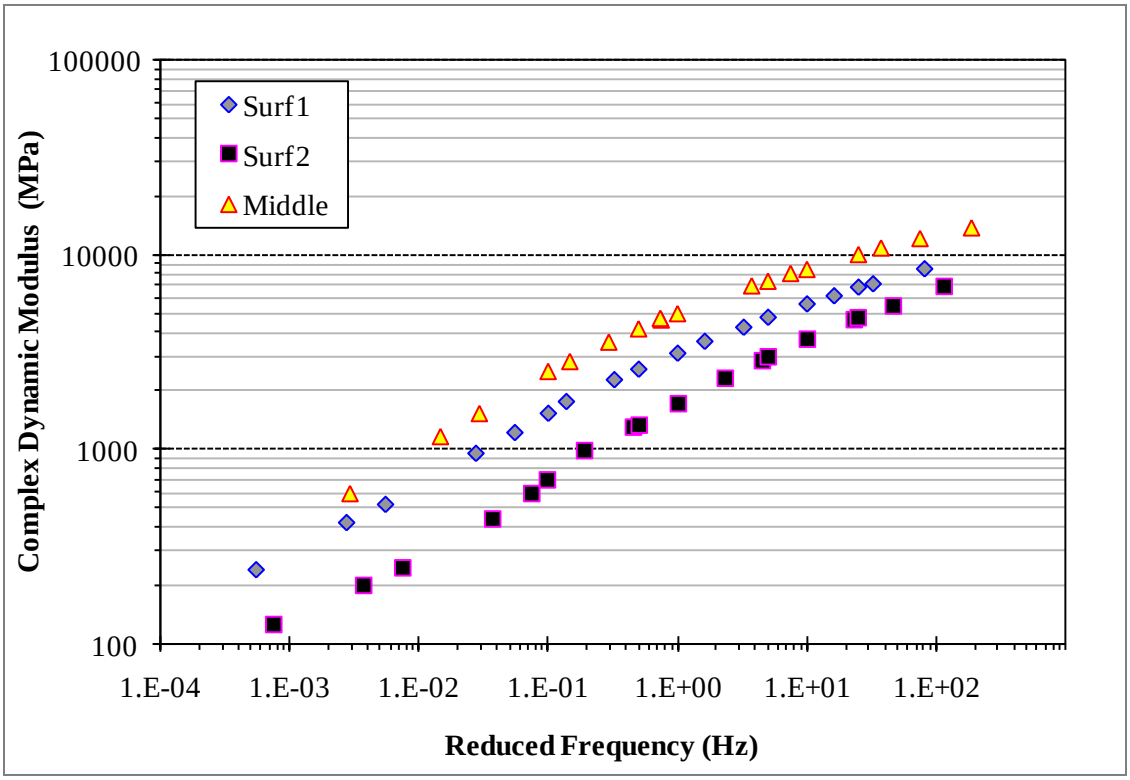


Figure 18. Dynamic modulus master curves for surface and middle layers. From NC Superpave Report (see Appendix E). Note: 1 MPa = 145 psi

6.3 HMA Superpave Shear Tester

The Superpave Shear Tester (SST) is used to determine stiffness and permanent shear strain properties of HMA mixtures. A suite of three tests is included in SST protocol: simple shear test at constant height (SSCH), repeated shear test at constant height (RSCH), and shear frequency sweep test at constant height (FSCH). The latter two tests were performed for the perpetual pavement materials in this study. The "constant height" designation in each test name indicates that the specimen height is held constant while the shear loads are applied. RSCH and FSCH testing was conducted at the Turner-Fairbanks Highway Research Center in McLean, Virginia. Data analysis was performed by the NC Superpave Center.

6.3.1 Repeated Shear Test at Constant Height

The RSCH test is described in AASHTO T320. An HMA specimen is subjected to a repeated shear stress of 10 psi (69 kPa) with a load period of 0.1 s and a rest period of 0.6 s. The test is run for 5000 cycles, or until 5 percent permanent shear strain is reached. Permanent strain is measured during each load cycle, and the total shear strain after 5000 cycles is recorded.

For this study, a minimum of four specimens were tested for each HMA layer. Prior to specimen creation, mixtures were compacted to the void content designed for the perpetual pavement system (6 percent for all layers except the 4 percent lower layer in test section 1). Each specimen was a cylinder 2 in (50 mm) long with a 3-in (75-mm) diameter. Tests were conducted at 136°F (58°C).

The average permanent shear strain profiles recorded during the tests are shown in Figure 19 for all mixtures tested. The cumulative permanent shear strain noted after 5000 cycles can be used to evaluate a mixture's rutting potential, with lower cumulative strain indicating better rutting performance. The test section 1 surface layer performed best, and the test section 2 surface layer performed worst, with the middle and base layers falling in between (the middle and lower 1 strain profiles fall on nearly identical curves).

All mixtures had between 2 and 3 percent permanent shear strain at 5000 cycles, which is a range considered to indicate adequate rutting resistance. According to Asphalt Institute (AI) guidelines, these mixtures would show fair rutting performance. The AI guidelines, however, were for mixtures with 3 to 4 percent air voids, which is a lower range than the mixtures tested in this study. A study at the NCAT Test Track did not find a good correlation between RSCH results and field rutting measurements. [*20*] However, the RSCH values provide a measure of comparison among the mixtures tested in this study's pavement system.

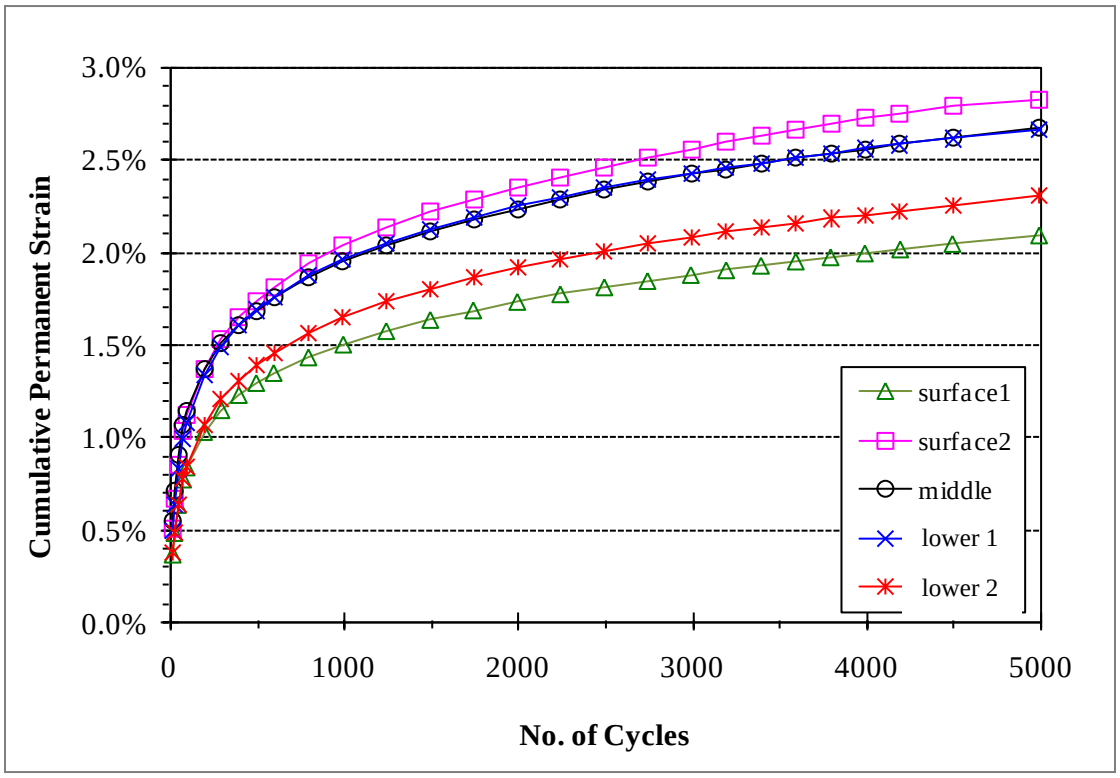


Figure 19. Cumulative permanent shear strain profiles. From NC Superpave Report (see Appendix E).

6.3.2 Shear Frequency Sweep Test at Constant Height

The FSCH test is described in AASHTO T320. An HMA specimen is subjected to a cyclic sinusoidal shear strain with maximum amplitudes of 0.0001 mm/mm. The cyclic loading is performed at frequencies of 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz. The various frequencies represent slow (low frequency) and high (high frequency) traffic speeds. Axial and shear deformations and loads are recorded, and the complex shear modulus (|G*|) is calculated for each test frequency.

For this study, five specimens were tested for each HMA layer. Prior to specimen creation, mixtures were compacted to the void content designed for the perpetual pavement system (6 percent for all layers except the 4 percent lower layer in test section 1). Each specimen was a cylinder 2 in (50 mm) long with a 3-in (75-mm) diameter. Test procedures were repeated at three temperatures: 68, 104 and 122°F (20, 40 and 50°C).

The average |G*| values measured at 10 Hz (high traffic speed) are shown in Figure 20 for all mixtures tested. Outlier data points were excluded from the averages. The middle and lower layer mixtures had higher |G*| values than the surface layer mixtures at all test temperatures. The |G*| was higher for the surface layer of test section 1 than test section 2. Statistical testing showed a statistical difference in |G*| for the surface layer mixtures but not for the two lower layer mixtures.

The complex shear modulus is an indicator of rutting resistance. Based on the FSCH test results, the middle and lower HMA layers would be expected to have greater rutting resistance than the two surface layers, and surface layer 1 would show better rutting performance than surface layer 2. Shear loading performance categories developed by the AI indicate that surface layer 2 would show poor performance, surface layer 1 would show fair performance, and the middle and lower layers would show excellent performance. These guidelines are based on samples with an air void content of 7 percent.

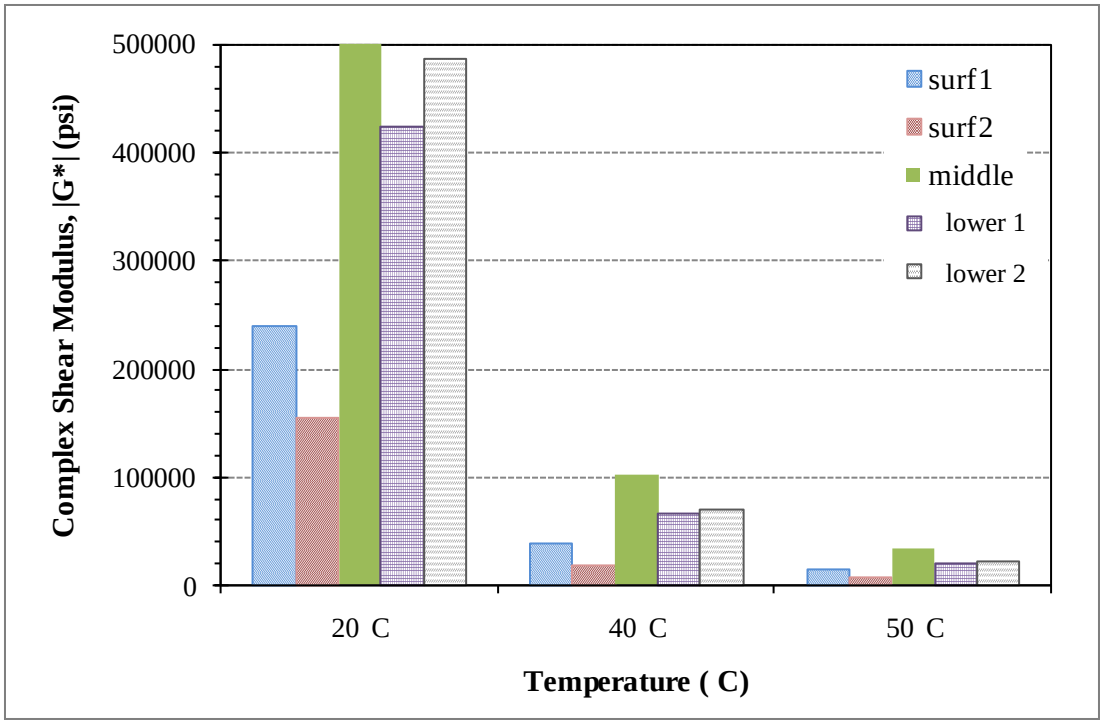


Figure 20. Average complex shear moduli (|G*|) at 10 Hz test frequency. From NC Superpave Report (see Appendix E).

7. Pavement Performance Data Analysis

7.1 Strain Measurement Results

Strain information was collected by Mr. D. Newman and analyzed by Dr. J. Crovetti of the Marquette University Department of Civil and Environmental Engineering. Data was collected in several ways from the strain and temperature sensors installed in the perpetual pavement test sections. The different data collection and analysis approaches are described in the following sections. This information is summarized from the final report published by Crovetti for the strain data collection and analysis. [*15*]

7.1.1 Falling Weight Deflectometer Tests Over Strain Sensors

To obtain precise strain sensor output readings, localized falling weight deflectometer (FWD) tests were performed directly over the functioning strain sensors at stations 3+037 and 3+600. These tests were conducted in September and October 2003 at FWD load levels of 5, 9 and 12 kips. Pavement temperatures in September were 110°F (43°C), and ranged from 48 to 57°F (9 to 14°C) in October. [*15*]

Strain readings from the pavement sensors are provided in Table 10. [*15*] Strain measured in the HMA layers increased with FWD loading. Strain was higher at the bottom of the lower HMA layer than at the interface of the middle and lower layers. This is expected, as strain profiles increase with increasing

depth into the pavement. Strain is therefore critical at the bottom of pavement layers. Measured strain was approximately 50% lower in the October analysis, when pavement temperatures were lower.

Table 10. Pavement Strain Sensor Output Readings. Adapted from [*15*]

Strain Sensor Station and Position	**FWD Load (kips)**	**Strain Output x10^{-6}**	
		September	**October**
3+037 Between middle and lower layers	5	16	9
	9	25	15
	12	37	20
3+600 Between middle and lower layers	5	No Data	5
	9		9
	12		11
3+600 Bottom of lower layer	5	30	13
	9	60	21
	12	85	29

7.1.2 Loaded Truck-Induced Strains

Strain readings were also collected as loaded trucks drove over the sensors. For these measurements, output for individual axle loads were recorded from the weigh station scales. In June 2004, induced strain was measured from four FHWA Class 9 trucks that legally passed through the weigh station. The trucks' gross weights ranged from 42,000 to 78,920 lbs, and their heaviest tandem axle loads ranged from 21,280 to 38,420 lbs. The trucks were travelling at approximately 40 miles per hour (mph) when they reached station 3+600, where strain measurements were recorded at the bottom of the HMA pavement layer. [*15*] Pavement temperatures were not measured; the average air temperature on the test day was 62°F (17°C).

Maximum tensile bottom-of-HMA strain readings induced by the trucks' tandem axles are shown in Figure 21. All strains were less than 50x10^{-6}. A relatively linear response was noted between load and induced strain. [*15*] The Truck 3 strain readings were proportionately lower than other trucks' readings. This could be due to the fact that Truck 3 traveled over the sensors at a slightly higher speed; loads applied more quickly tend to induce lower strain in a material.

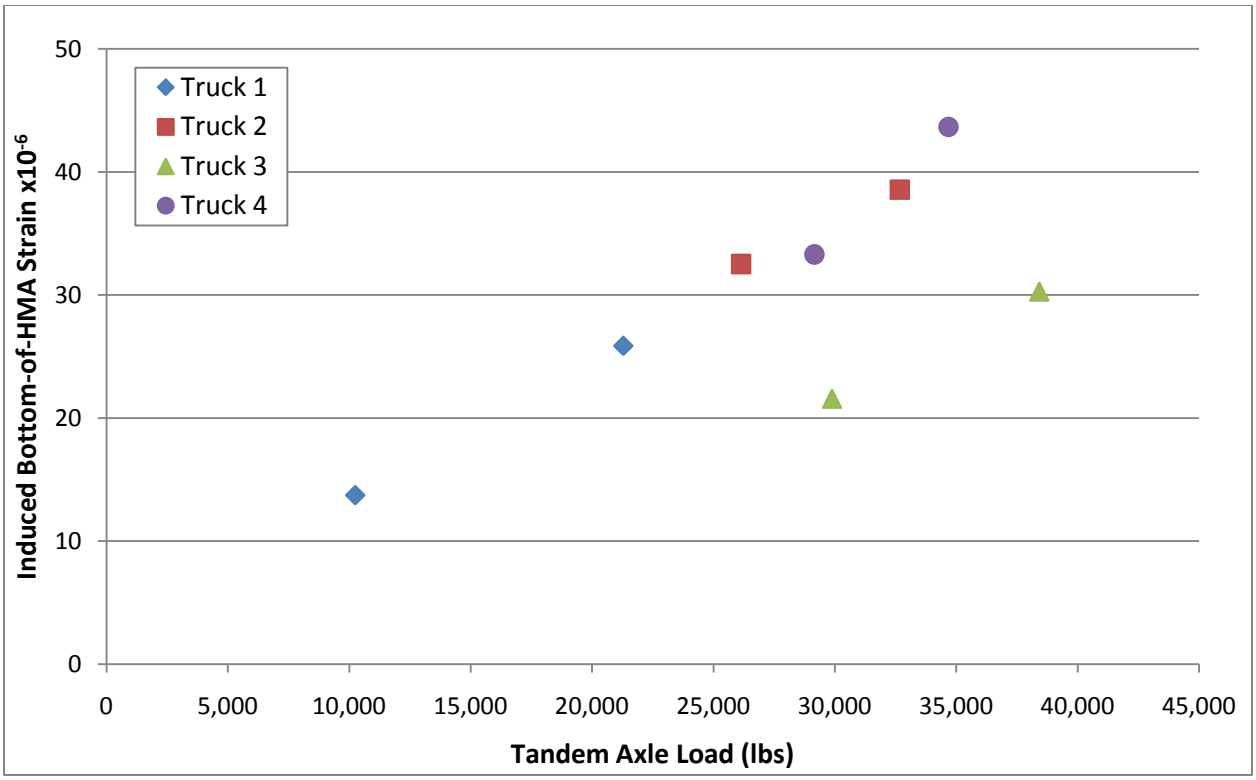


Figure 21. Induced strain at the bottom of HMA pavement. Adapted from [*15*]

Two additional loaded truck experiments were conducted in April and July 2005. In these cases, a loaded dump truck with a gross weight of 72,700 lbs drove over the strain sensors at station 3+600. Tests were conducted at truck speeds ranging from 26 to 55 mph. In addition, trucks were tested with pusher wheels up and down, resulting in steering axle/load axle weights of 19,000/15,000/38,600 lbs (wheels down) and 25,400/47,300 lbs (wheels up). Pavement surface temperatures in April and July were 80 to 91°F (27 to 33°C) and 90 to 103°F (32 to 39°C), respectively. [*15*]

Several conclusions were drawn from this portion of the strain sensor testing: [*15*]

1. **Pavement temperature.** In April (cooler pavement temperatures), strain readings at the bottom of the HMA layer were approximately $15x10^{-6}$ with pushers down and truck speed of 55 mph. With the same speed and axle configuration, the maximum strain was $50x10^{-6}$ in July (higher pavement temperatures). Strain never exceeded $25x10^{-6}$ during testing in April, regardless of truck speed and axle configuration.
2. **Axle configuration.** Holding temperature and speed constant, bottom-of-HMA strains were higher with pusher wheels up (heavier loads) than with wheels down. In April, wheels up versus wheels down resulted in strains of 23 and $15x10^{-6}$, respectively. In July, wheels up versus wheels down resulted in strains of approximately 53 and $48x10^{-6}$, respectively. The difference in induced strain was not as great in July, possibly because in the wheels up configuration, the truck did not drive directly over the strain sensor.

3. **Truck speed.** Slow speed tests (26 mph) were conducted in July. In the wheels down configuration, the truck induced strains of 69 and 48×10^{-6} at 26 and 54 mph, respectively. Slow travel speeds resulted in higher strain at the bottom of the HMA pavement.
4. **Layer interface strain.** The sensor located between the middle and lower HMA layers was functioning in the loaded dump truck tests. Strain readings at this location were generally lower than at the bottom of the HMA pavement.
5. **Overall strain values.** Strain readings did not exceed 100×10^{-6} in any of the tests. In the worst-case test scenario, with high pavement temperatures, pusher wheels up (heavy loads), and slow speed (32 mph), the maximum bottom-of-HMA strain reading was 100×10^{-6}. In more typical loading cases (wheels-down loads and/or faster travel speeds), strain levels were less than 70×10^{-6}. This indicates that the perpetual pavement system functioned adequately, keeping strains low at the bottom of the pavement layer, and thereby protecting the base and subgrade layers from excessive strain.

7.1.3 Service Life Prediction

The final portion of the analysis by Crovetti included a service life prediction based on a mechanistic appraisal. In this investigation, pavement failure was indicated by bottom-up fatigue cracking covering 50% of the HMA pavement surface. Cumulative fatigue damage was calculated based on average monthly pavement temperatures and assumed traffic levels of 520,000 ESALs per month (6,240,000 ESALs per year). To evaluate the effect of air void variability, the pavement system was analyzed with 4, 5 and 6 percent voids in the fatigue-resistant middle and lower (25 mm) HMA layers. [*15*]

The results from this analysis are shown in Figure 22. In the mechanistic evaluation, increased air voids in the 25 mm layers resulted in a shorter service life. The predicted service life was 94 years for 4 percent voids and only 13 years for 6 percent voids. [*15*] The average measured air voids for the middle layer were 5.4 and 5.8 percent for test sections 1 and 2, respectively; for the lower layer, the average measured air voids were 6.0 and 6.9 percent for test sections 1 and 2, respectively (Section 4.3). Based on these averages, neither test section would be expected to perform well in the bottom-up fatigue cracking category. Although only top-down cracking was noted in the forensic coring analysis (Section 7.4), the alligator cracking observed in the wheel paths during the time since coring could be bottom-up fatigue cracking.

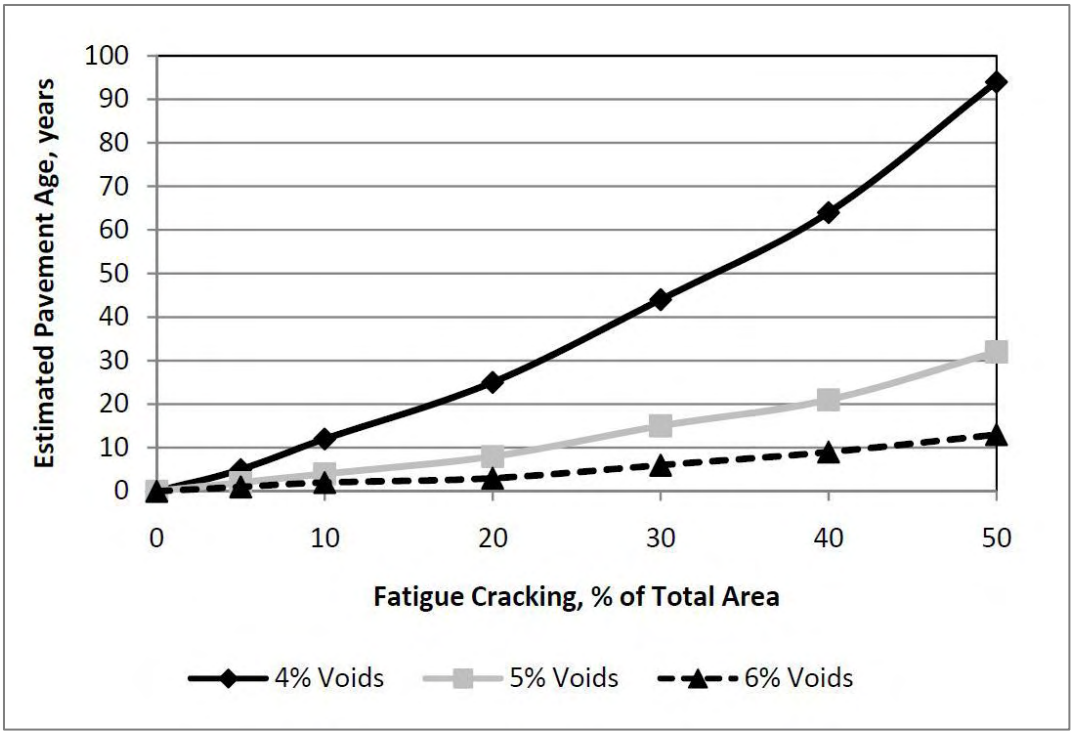


Figure 22. Service life prediction for 4, 5 and 6 percent air voids in the middle and lower HMA pavement layers. From [*15*]

7.2 Falling Weight Deflectometer Results

To obtain information on the individual pavement layers, FWD testing was performed along the entire length of the perpetual pavement test sections in 2003, 2004, and 2007. Testing was conducted by WisDOT using the Department's KUAB FWD equipment, a tow-behind device capable of measuring pavement deflection during heavy loads imposed on the pavement by dropping weights from calibrated heights. Load tests were typically conducted at 5, 9, 12, and 20 kips.

FWD testing was conducted in August 2003, June 2004, and April 2007. The daily average temperatures for the test days were 76.0, 62.1 and 33.5°F (24.4, 16.7 and 0.8°C), respectively. Testing was conducted twice in August 2003: once in the morning, when pavement temperatures ranged from 97.0 to 102°F (36.1 to 38.9°C), and once in the afternoon, when pavement temperatures ranged from 118 to 126°F (47.7 to 52.2°C). Test data were backcalculated using EVERCALC, and pavement layer moduli were determined. (Base layer moduli were not calculated in 2003.) These results are shown in Figures 23 through 25. Average moduli for all layers in test sections 1 and 2 are provided in Table 11.

On each test date and time, average moduli were approximately equal in both test sections (Table 11). The time of day that pavement was tested had a large effect on HMA modulus, as seen in the August 2003 test data (Figure 23). In the afternoon, when the sun had heated the pavement to higher temperatures than in the morning, the HMA modulus was reduced by nearly half. The April 2007 test

data were collected at the end of the spring thaw, when the base and subgrade could still have been saturated; the April 2007 base and subgrade moduli were in the same range as the 2003 and 2004 values, but results were more variable along the length of the test sections (Figure 25). Average HMA moduli were highest in April 2007, when the air and pavement temperatures were lowest. The range of moduli ($0.3x10^6$ to $2.5x10^6$ psi) were typical for uncracked HMA pavement at the various temperatures tested. Base layer moduli (approx. 20,000 psi) were somewhat low for a well-drained crushed aggregate base course. Subgrade moduli were higher than expected for a silty-clay soil, which are usually in the 12,000 to 20,000 psi range.

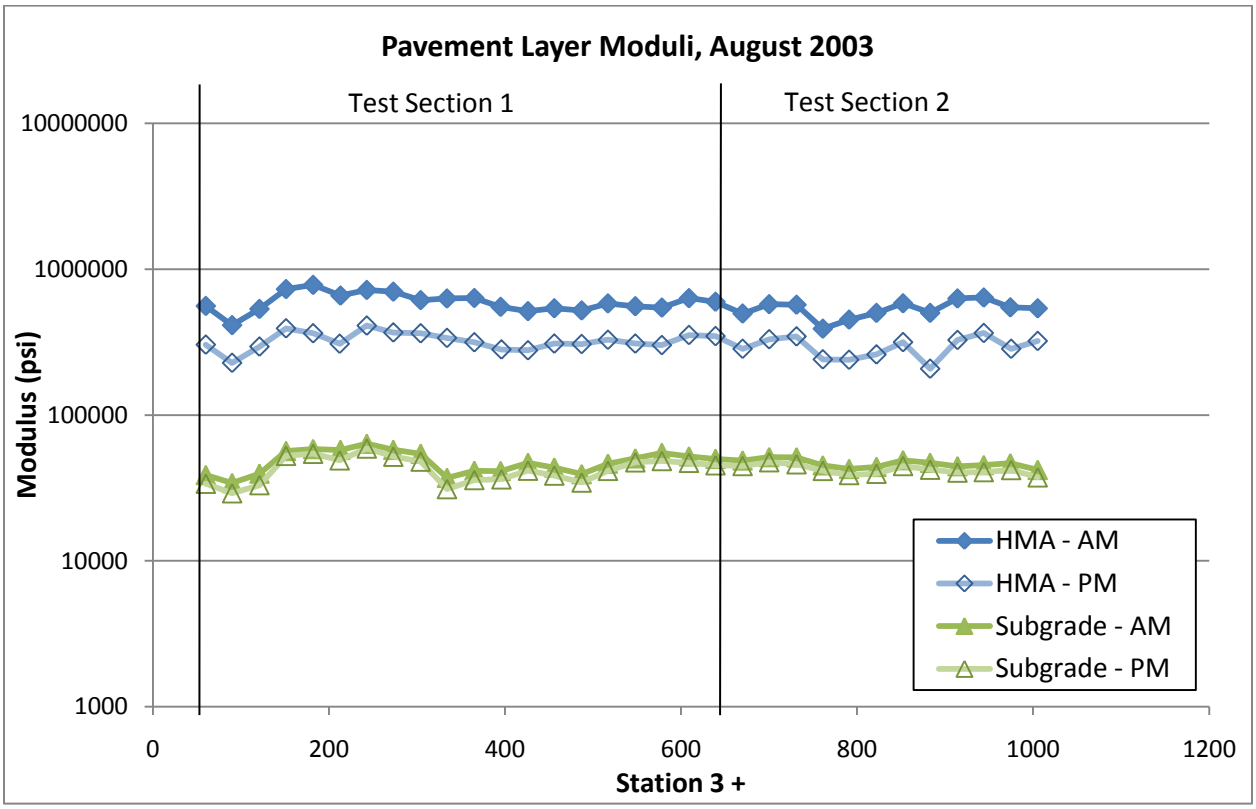


Figure 23. Pavement layer moduli from FWD testing, August 2003 morning and afternoon.

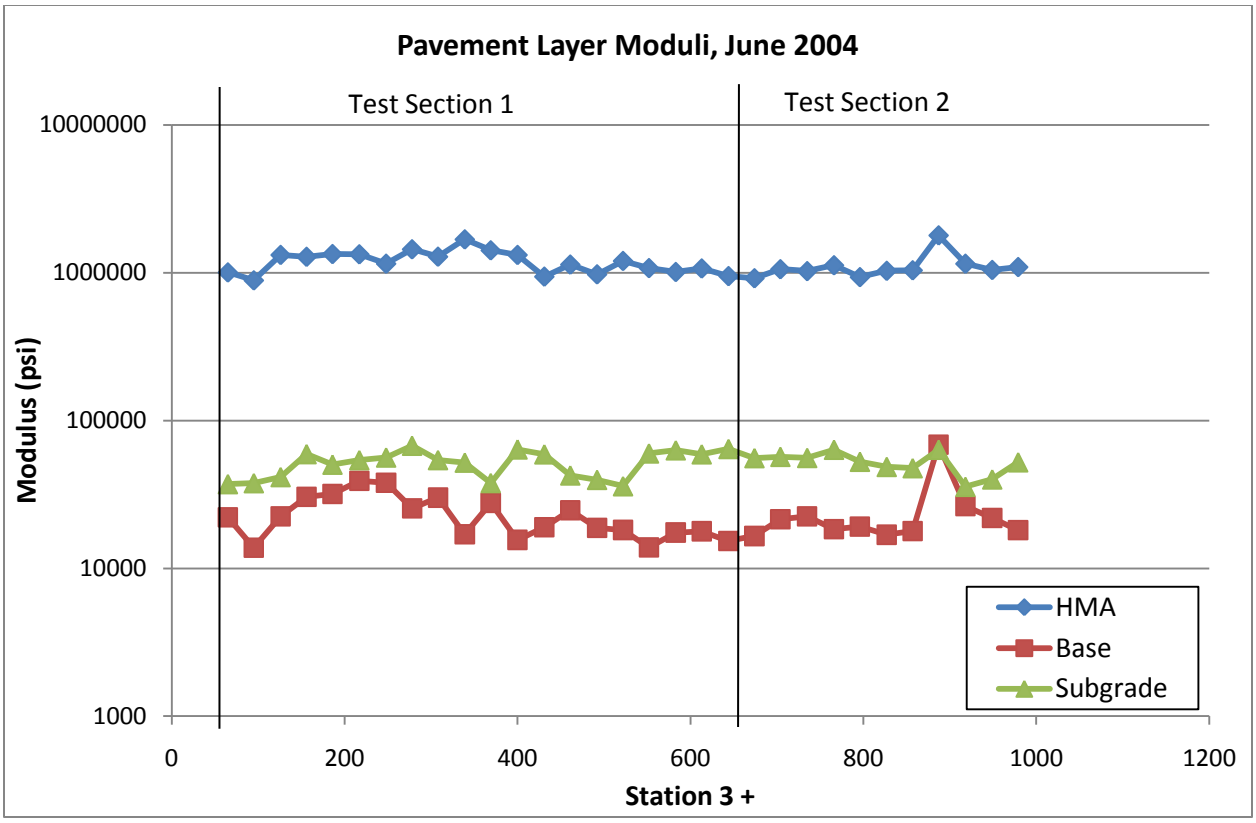


Figure 24. Pavement layer moduli from FWD testing, June 2004.

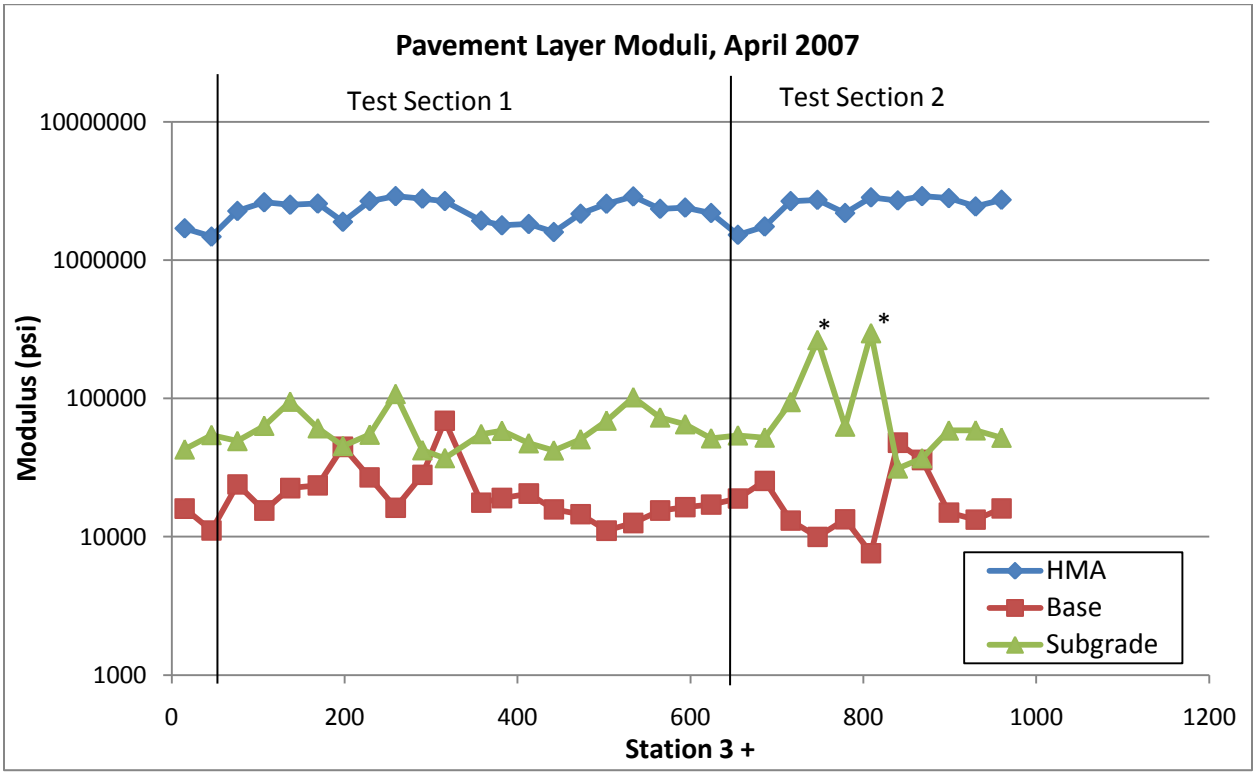


Figure 25. Pavement layer moduli from FWD testing, April 2007. * Denotes outliers.

Table 11. Average Pavement Layer Moduli Values

Pavement Layer		HMA	Base	Subgrade
August 2003 AM Testing	TS 1	601,100	--	48,100
	TS 2	540,200	--	46,800
August 2003 PM Testing	TS 1	323,700	--	42,800
	TS 2	297,700	--	42,300
June 2004	TS 1	1,178,700	22,600	52,000
	TS 2	1,129,300	25,100	51,700
April 2007	TS 1	2,358,900	23,000	62,100
	TS 2	2,456,200	19,400	55,200*

* Excludes outliers shown in Figure 25.

7.3 Distress Survey Results

WisDOT staff conducted detailed distress surveys of the perpetual pavement test sections in 2007, 2008, and 2009. Results of these surveys are provided pictorially in Appendix F. Using automated profiling equipment, international roughness index (IRI), and rutting data were also collected in 2007 and 2009. These results are shown in Table 12. The pavement distress index (PDI) is often collected along with IRI and rut data, but for a research study such as this, the visual surveys reported in Appendix F are more useful for evaluating pavement performance.

The most frequent distresses noted were early alligator cracking in the wheel paths, and longitudinal cracking at the center of the pavement. Other distresses included transverse and longitudinal cracking in the wheel paths, and areas where surface aggregate was missing (aggregate popouts). The presence of these distresses were surprising, as the pavement had only been in service for four years when many were first noted.

Test section 1 exhibited slightly more distress than test section 2, although the latter section still had several areas of cracking. An area of particularly poor performance was noted in test section 1, between stations 3+360 and 3+440 (see Appendix F). In this area, extensive alligator cracking was present in the wheel paths, and several center-of-pavement longitudinal cracks were developing. Photographs of these areas in 2008 and 2009 are shown in Figures 26 and 27, respectively.

It is possible that the noted distresses initiated and worsened during spring thaws, when base and subgrade strength tend to be critical. However, based on the FWD data presented in the previous section, there was no correlation between areas with greater distress and areas with lower springtime foundation moduli.

Although the early distress formation was of concern, the pavement sections still performed well in terms of rutting and ride quality. Wheel path rutting was approximately one-tenth of an inch after six years in service (Table 12). In addition, IRI measurements of 1.34 to 1.67 indicated that the pavement

sections had good ride quality (Table 12). In terms of the rutting and IRI measurements, both test sections performed at approximately equal levels.

Table 12. IRI and Rut Depth Measurements

Test Section	2007		2009	
	IRI (m/km)	Rut depth (in)	IRI (m/km)	Rut depth (in)
1	1.34	0.02	1.50	0.11
2	1.66	0.03	1.67	0.12

Figure 26. Distress in test section 1, station 3+362, 2008.

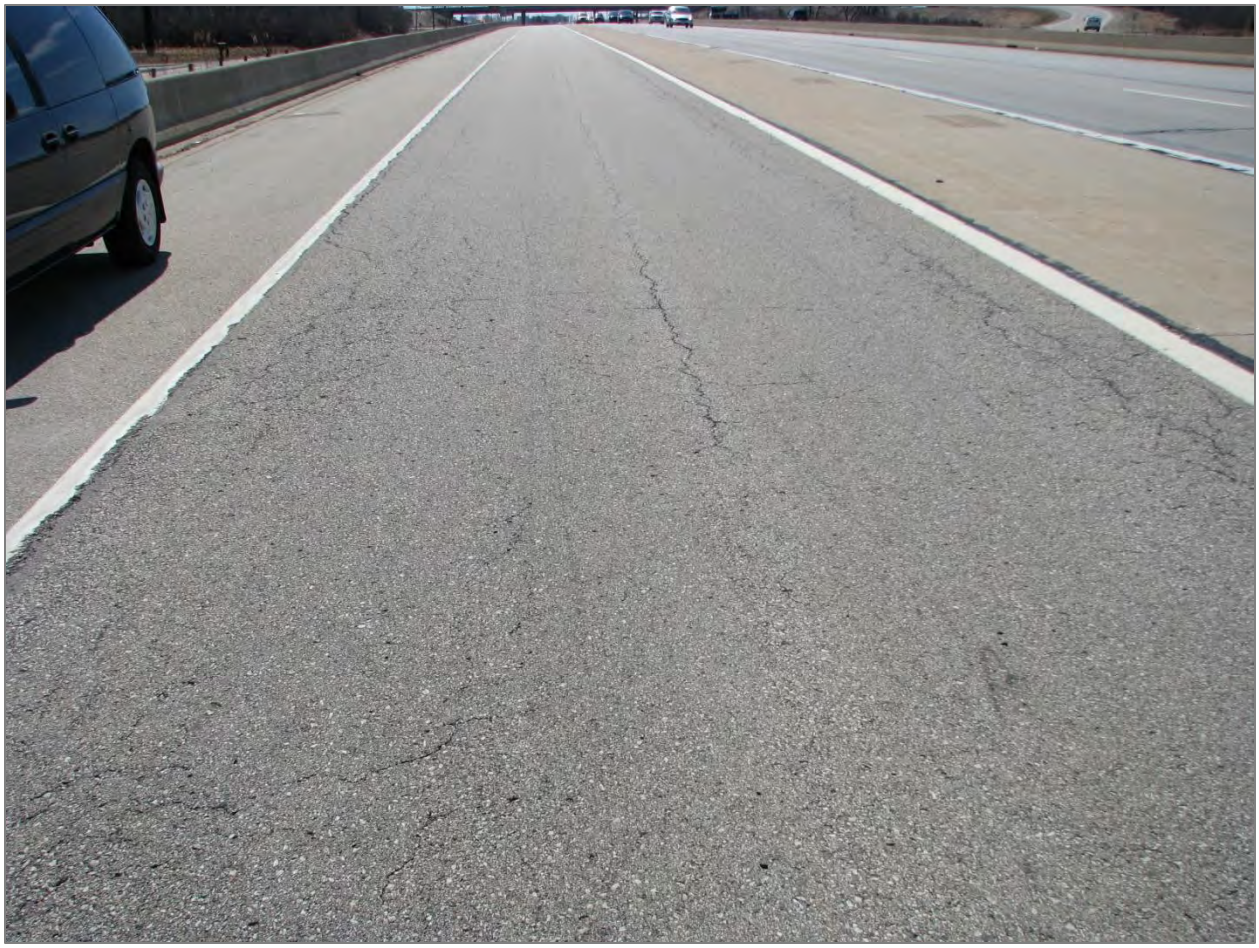


Figure 27. Distress in test section 1, approximate station 3+375, 2009.

7.4 Forensic Coring Analysis

On April 4, 2007, a series of 6-inch diameter cores were taken to further investigate the distresses noted in the pavement. Cores were taken in three areas in test section 1 and from two areas in test section 2. Cores were taken directly through distresses to determine how far below the surface the cracking extended. Two cores were also taken in non-distressed areas as a comparison. A summary of the cores removed for pavement analysis is provided in Table 13. Soil information was also obtained by augering through the base and subgrade. Separate core holes were used for this purpose - a second soil analysis core was taken approximately 5 feet north of each core described in Table 13.

Table 13. Pavement Cores Taken for Forensic Analysis

Core ID	Test Section	Station	Distress Type	Pavement Location
1	1	3+346	Longitudinal crack	Center of pavement
2	1	3+399	Early alligator cracking	Right edge of pavement
3	1	3+474	None	1' left of right wheel path
4	2	3+698	Banded longitudinal crack	Center of pavement
5	2	3+738	None	Center of pavement

Upon analysis of the cores taken through distresses, it was determined that cracking initiated at the surface and extended downward into the surface HMA layer. This was encouraging, as bottom-up cracking would have indicated excessive strains at the bottom of the pavement layer. Top down cracking is expected in a perpetual pavement system, as the surface layer is intended to eventually be milled and replaced. Observations from individual cores are discussed below.

Results from the soils analysis are provided in Appendix G. The results did not show any soil anomaly that would be a cause of the distresses noted.

7.4.1 Core 1

Core 1 was taken directly over a longitudinal crack in test section 1. The crack initiated at the surface and extended halfway through the surface HMA layer on one side of the core and less than an inch downward on the other side of the core. Large pockets of asphalt binder were present in the middle HMA layer; most of the pockets were located at the bottom of this layer (Figure 28). Coarse and fine aggregate distribution was uniform in all layers.

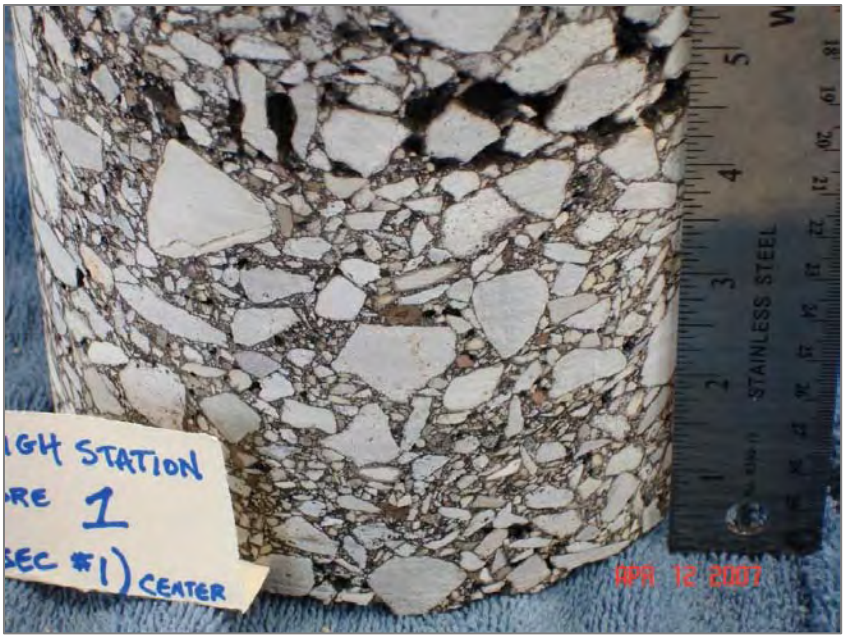


Figure 28. Concentrated asphalt binder pockets noted in the middle HMA layer of core 1.

7.4.2 Core 2

Core 2 was extracted over an area of early alligator cracking in test section 1. Cracking initiated at the surface and did not extend far into the top layer of HMA. Voids were present in the surface layer (Figure 29). Aggregate distribution was not as uniform as in core 1.

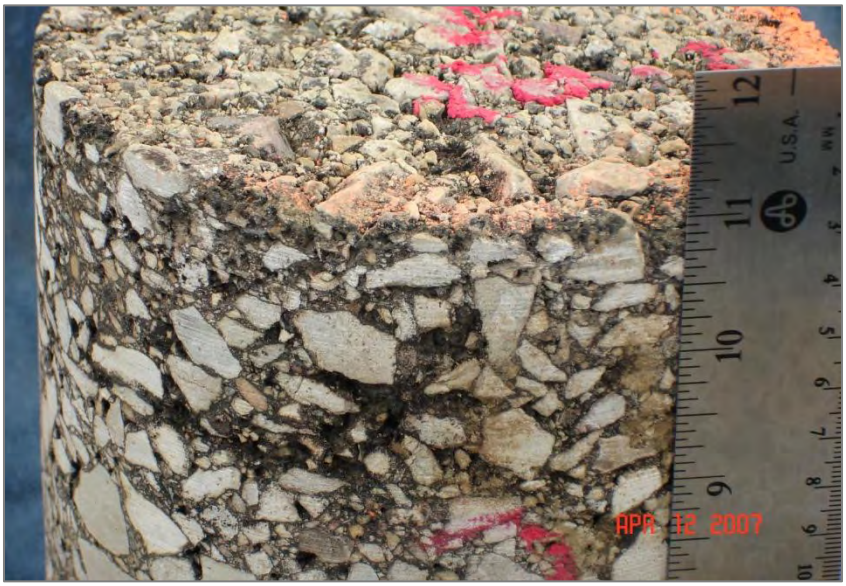


Figure 29. Void noted in surface HMA layer of core 2.

7.4.3 Core 3

Core 3 was taken from an area in test section 1 with no visible distress. As with core 1, core 3 had concentrated pockets of asphalt binder present in the middle HMA layer. Aggregate distribution was good, although the surface layer appeared to have a higher concentration of fine aggregate than cores 1 and 2.

7.4.4 Core 4

Core 4 was taken from a location in test section 2 with banded longitudinal cracking. The cracks initiated at the surface and did not extend far into the pavement. Pockets of asphalt binder were noted at the bottom of the middle HMA layer, although not as concentrated as in cores 1 and 3. Coarse and fine aggregate distribution was good.

7.4.5 Core 5

Core 5 was taken from an area in test section 2 with no visible distress. This core had uniform distribution of coarse and fine aggregates and no concentrated spaces of asphalt binder.

7.5 Forensic Site Review

In July 2007, the pavement was observed by a team of WisDOT, Payne and Dolan, and WAPA personnel. Possible causes of the two most frequently occurring distresses were discussed. Potential solutions for future perpetual pavement systems were also proposed.

It was suggested that the longitudinal center-lane cracking was caused by segregation of the HMA mix during construction. Segregation was noted during construction, as mentioned in Section 4.4. Mixture segregation is often prevented with the use of shuttle buggies, which remix the HMA prior to transfer to the paver, resulting in a uniform delivery of material.

A proposed cause of the alligator cracking was over-compaction of the top HMA layer. This might have caused shattering of the aggregate, which would allow moisture to seep into the pavement. Indeed, evidence of shattered aggregate was noted upon further review of distressed areas (Figure 30). Target densities were higher for the perpetual pavement than for typical WisDOT pavement mixtures. Slight changes to the mixture design could result in a mixture that is more easily compacted to the higher target density. A reduction of the target densities of the top layers could also help prevent over-compaction.

It was also noted that the low rut levels were encouraging, as this indicated that the pavement system was in good structural condition. That is, the pavement layers had adequate foundation support, which would allow the perpetual pavement system to perform and be rehabilitated as designed.

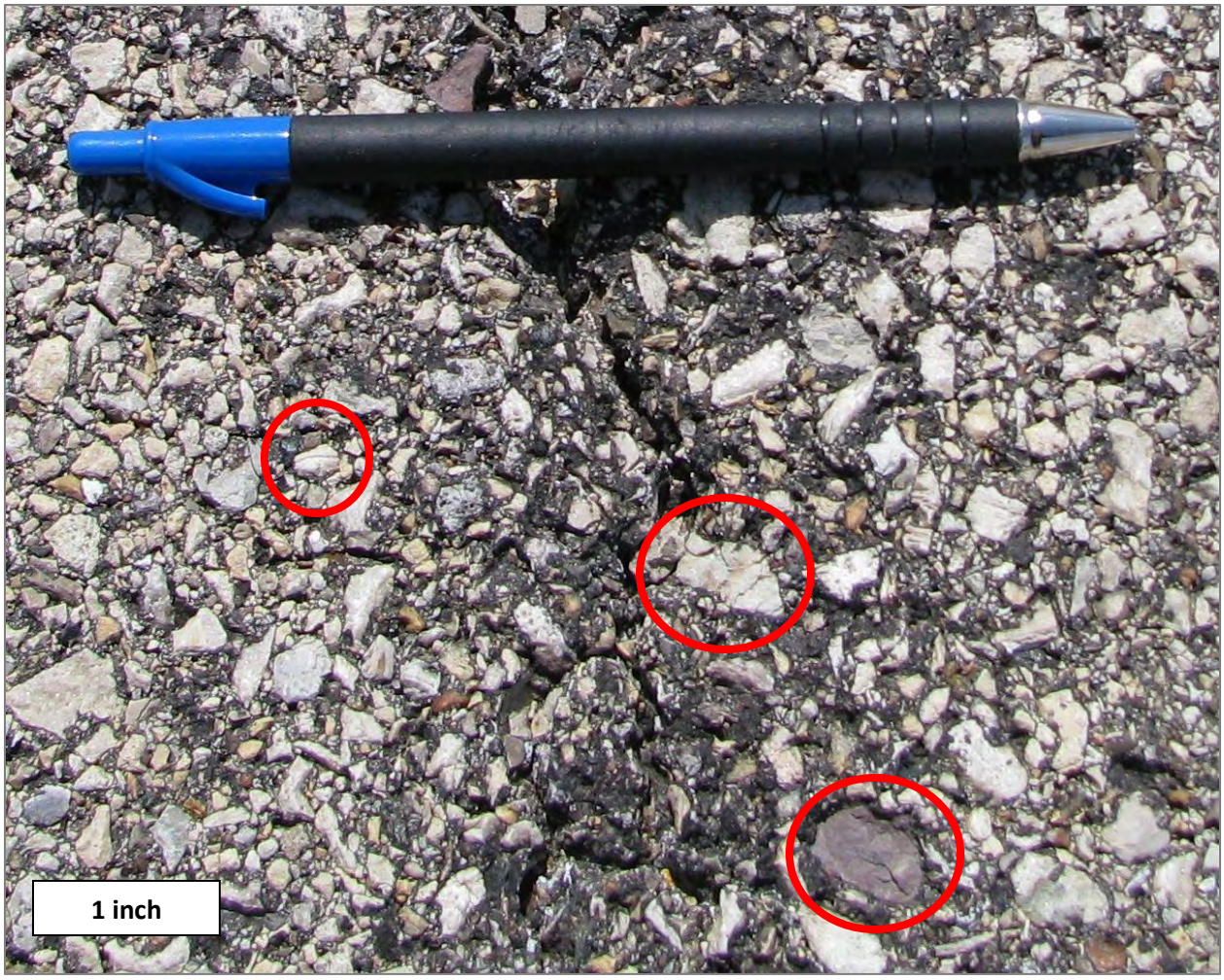


Figure 30. Shattered aggregate noted in a distressed area.

8. Summary and Conclusions

Two perpetual pavement test sections were constructed on the entrance ramp to I-94 westbound at the Kenosha Safety and Weigh Station Facility in Southeastern Wisconsin. Performance surveys after four, five and six years in service showed many instances of alligator and longitudinal cracking in the wheel paths. These distresses occurred more frequently in test section 1. Forensic core investigation showed that the distresses were top-down, and cracks had not propagated into the middle HMA layer. It was determined that segregation and over-compaction may have led to these premature distresses. Very little rutting was noted (approximately 0.1 in), and pavement ride quality was good in both test sections.

Strain gages were installed in the pavement layers during construction. While only 3 of the 16 sensors survived after construction, good-quality strain data was collected using loaded trucks with known axle weights. Bottom-of-pavement strain was dependent on induced load, pavement temperature, and vehicle speed. Higher temperatures and lower speeds resulted in greater strain in the bottom HMA layer. The maximum strain noted was 100×10^{-6} and occurred with high pavement temperatures (approximately 100°F/38°C), slow speeds (32 mph), and heavy tandem axle loads (47-kip). More average tandem axle loads (up to 40 kips) and higher speeds (45 to 55 mph) resulted in bottom-of-pavement strains that were less than 70×10^{-6}. [*15*]

Material properties tested with the Superpave Shear Tester and Simple Performance Tests indicated that the HMA layers used in the test sections had adequate permanent deformation (rutting) resistance. The middle and lower layers performed at approximately the same level. The surface layer used in test section 1 had better rutting properties than the test section 2 surface layer.

FWD testing showed little difference in pavement layer moduli between the two test sections. HMA and base layer moduli were in appropriate ranges for the temperatures at which they were tested. Subgrade moduli were higher than expected. Ambient air temperature affected HMA modulus values, with higher temperatures (76°F/24.4°C) resulting in average moduli of approximately 0.6×10^{6} psi and lower temperatures (33.5°F/0.8°C) producing stiffer moduli of 2.5×10^{6} psi.

It was difficult to estimate an expected service life prior to the first replacement of the perpetual pavement's surface layer. Premature distresses were noted in both test sections but were likely a result of construction issues. In addition, the weigh station ramp is scheduled to be completely replaced in 2010 as part of the I-94 corridor upgrade between Milwaukee and the Illinois state line. It will not be possible to monitor the test sections after seven years in service.

Current WisDOT policies indicate that in life cycle cost analyses, an initial service life of 16 years should be used for the upper HMA layer. The same service life of 16 years is also designated for the upper layer replacement. [*21*]

Overall, Wisconsin and other states have noted good performance with perpetual pavements. The concept of designing pavement layers based on a limiting strain value is rational as pavement design moves towards mechanistic procedures. The ability to rejuvenate the pavement system by periodically

removing and replacing a thin surface layer is also ideal, as agency costs, material requirements, and user delays will be reduced.

9. Recommendations

It is recommended that, as with all pavement placement, strict attention be paid to construction details and the quality of the mixture as it is being paved. These factors are critical to the success of perpetual pavement systems.

Because the Kenosha Safety and Weigh Station Facility ramp is scheduled to be replaced prior to its intended service life, there will be an opportunity for additional forensic evaluation. If time and resources permit, it is recommended that the pavement be observed as it is removed. It can then be determined how far surface cracks have propagated into the pavement, and also whether any fatigue (bottom-up) cracking is evident in the lower layer.

In conclusion, the following specific observations and recommendations are presented based on the objectives outlined in Section 3.2.

1. Evaluate the theory of perpetual pavements

Based on the results from this study, the theory of a perpetual pavement system is solid. The lower HMA layer was designed to resist bottom-up fatigue cracking. This was true in the test sections. There were only several instances (high temperature, high load, and low travel speed) where measured strains were significantly higher than the currently-accepted 70×10^{-6} strain endurance limit.

The top HMA layer was designed to resist rutting and wear. Low rut levels (approximately 0.1 in) were noted in both test sections. Although significant early distresses were noted in both test sections, these distresses were found to originate at the surface and extend downward, as the sacrificial layer of a perpetual pavement is designed. It is expected that the surface layer will reach the end of its service life prior to the 16-year design life, but the middle and lower layers were protected and can remain in place.

2. Analyze different pavement layer materials

The HMA mixtures used in the perpetual pavement test sections were tested for properties that predict permanent deformation (rutting). Tests included Simple Performance Tests (SPT) for flow number (F_N) and complex dynamic modulus (|E*|), and Superpave shear tests (SST) for repeated shear (RSCH) and frequency sweep (FSCH). These tests all had similar results: the middle and lower layers performed at adequate and approximately equal levels, and the test section 1 surface layer outperformed the test section 2 surface layer.

The test section 1 and 2 surface layers differed in their binder grade; test sections 1 and 2 had PG 76-28 and 70-28 binders, respectively. In the material tests mentioned above, the stiffer binder (PG 76-

28) had better rutting properties. Although stiffer binders are more costly, they would provide greater rutting resistance in the long term. During the first six years in service, however, neither test section had appreciable rut levels.

The lower layers of the two test sections differed in their target air content: 4 and 6 percent for test sections 1 and 2, respectively. Although these materials performed approximately equally in the SPT and SST tests, the air void level was found to make a great difference in service life based on the mechanistic analysis performed by Crovetti. A perpetual pavement under the Kenosha weigh station conditions was estimated to fail by fatigue damage after 94 and 13 years for a lower layer with 4 and 6 percent voids, respectively. [*15*] While this result should be reviewed with additional perpetual pavement systems, it appears that a lower target air void design in the lower HMA layers would provide the perpetual pavement system with greater fatigue durability.

3. Provide data that can be used to develop and/or validate perpetual pavement guidelines

Current guidelines on when to consider using a perpetual pavement are suitable. These guidelines state that a deep-strength or perpetual pavement design alternative must be included in the pavement type selection process during the design phase for major highways. [*22*] The guidelines go on to state that perpetual pavements be designed using a mechanistic design procedure and based on a maximum allowable strain ($\varepsilon_{allowable}$) value at the bottom of the HMA pavement. [*23*]. The $\varepsilon_{allowable}$ value should be 70×10^{-6}, which is a conservative value for the HMA endurance limit. In addition, the fatigue performance of the test sections was excellent (no bottom-up cracking), and measured strains were typically lower than 70×10^{-6}. [*15*] Research on endurance limit should be monitored, however, and the $\varepsilon_{allowable}$ value should be modified accordingly.

4. Determine the preferred design methodology for perpetual pavements in Wisconsin

Based on the results from this study, the following design methodology recommendations are proposed:

1. <u>Overall pavement structure design</u>: Use mechanistic design tools (such as the Mechanistic-Empirical Pavement Design Guide (MEPDG) and software) to determine a pavement thickness that limits bottom-of-pavement tensile strains to 70×10^{-6}.
2. <u>Lower layer</u>: Select a relatively low target air void content, such as 4 percent. Use a moderate-stiffness asphalt binder grade appropriate for temperature conditions at the lower layer depth.
3. <u>Middle layer</u>: Adjust the thickness of this layer to meet overall structural requirements. Use rut-resistant mixture design properties.
4. <u>Surface layer</u>: Select a stiff, high performance asphalt binder appropriate for the environmental conditions. Design this layer to be rut- and wear-resistant.

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Appendix A Production Volumetrics

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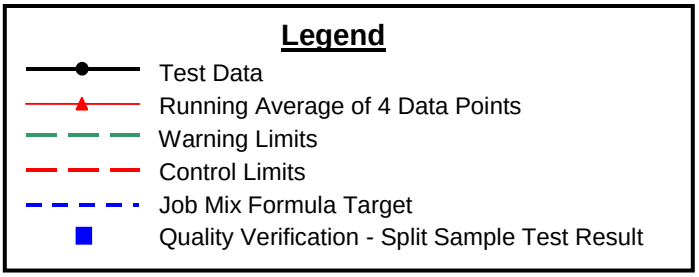
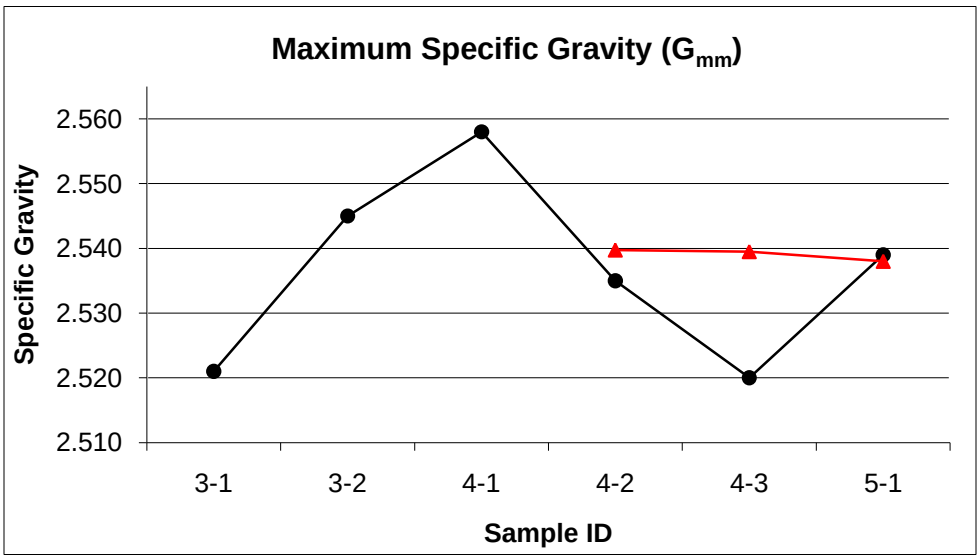
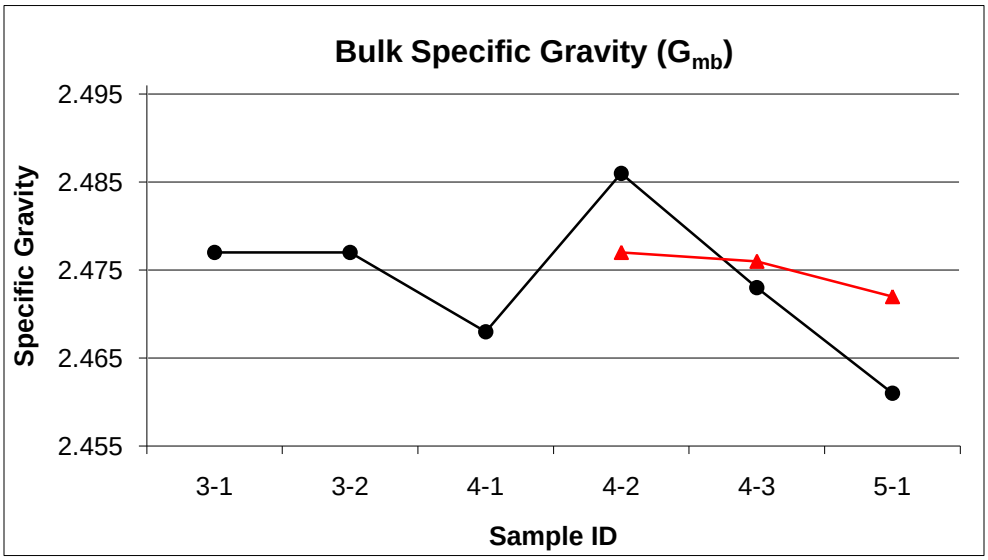
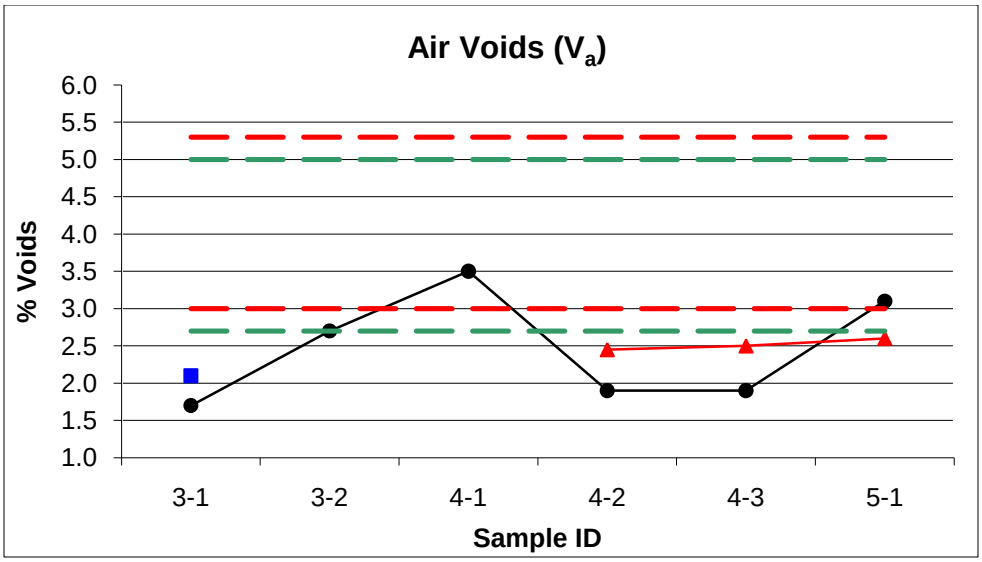


25.0 mm Mix



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5.0
4.5
4.0
3.5
3.0
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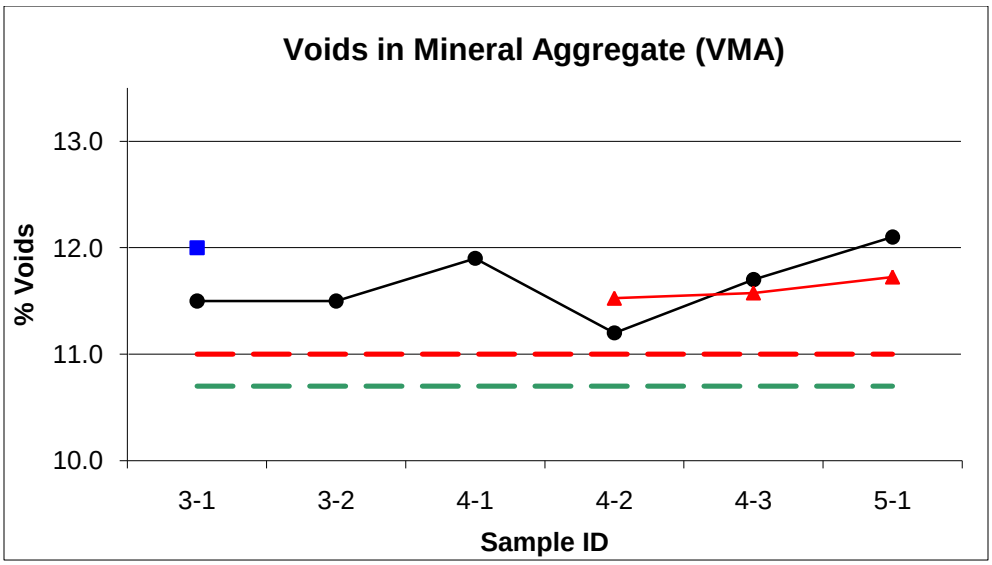
3-1
3-2
4-1
4-2
4-3
5-1
Sample ID


Voids in Mineral Aggregate (VMA)
% Voids
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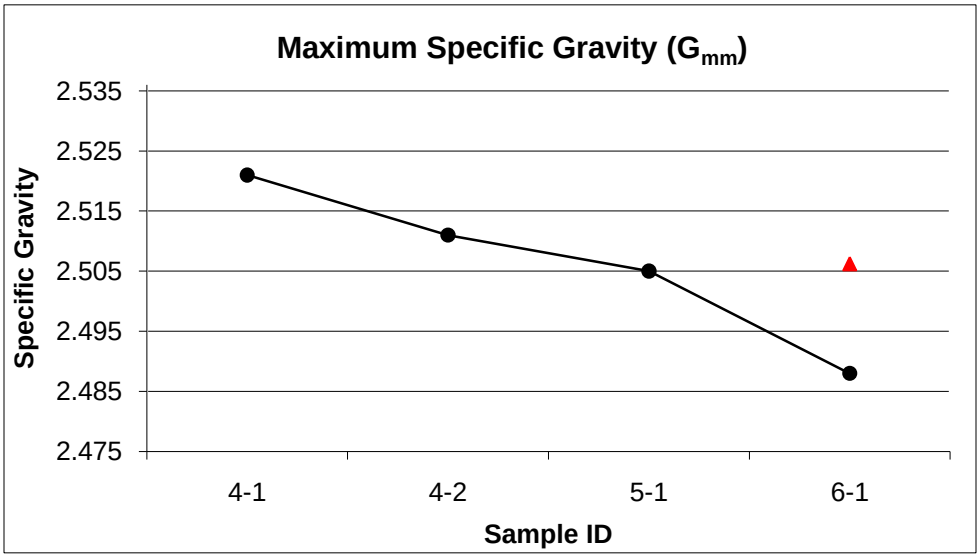
12.0
11.0
10.0
3-1
3-2
4-1
4-2
4-3
5-1
Sample ID

12.5 mm Mix


Maximum Specific Gravity (G_{mm})
Specific Gravity
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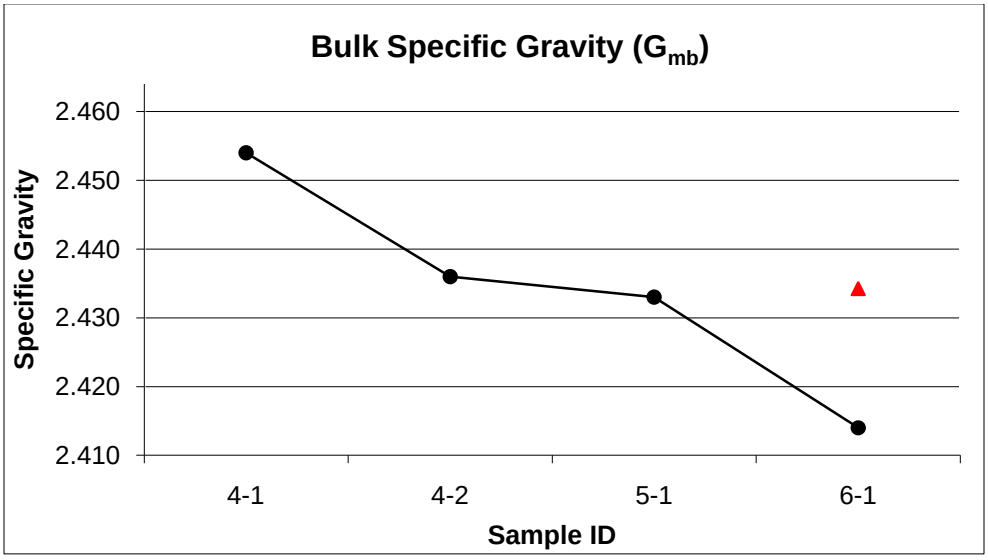
2.515
2.505
2.495
2.485
2.475
4-1
4-2
5-1
6-1
Sample ID


Bulk Specific Gravity (G_{mb})
Specific Gravity
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2.420
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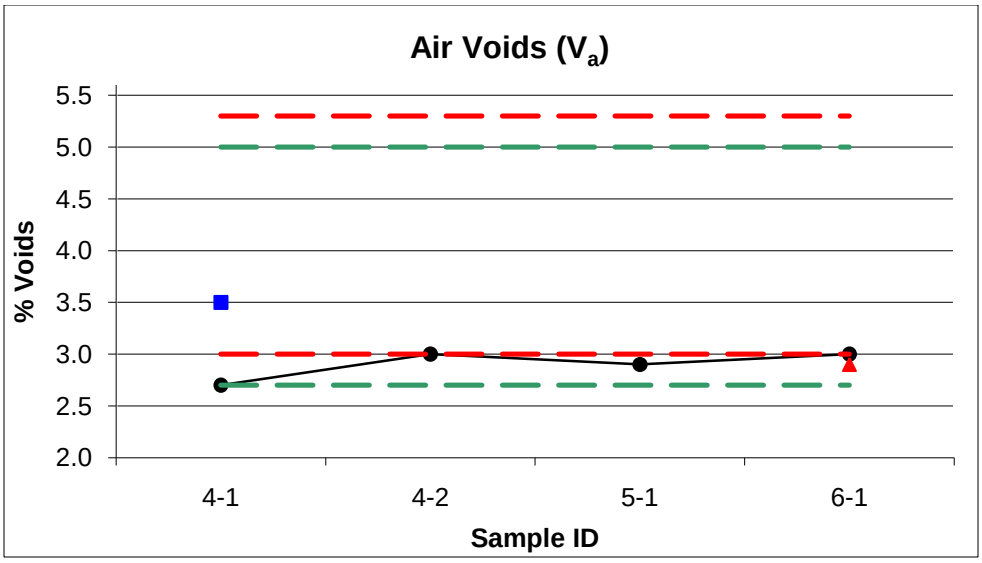
4-2
5-1
6-1
Sample ID


Air Voids (V_a)
% Voids
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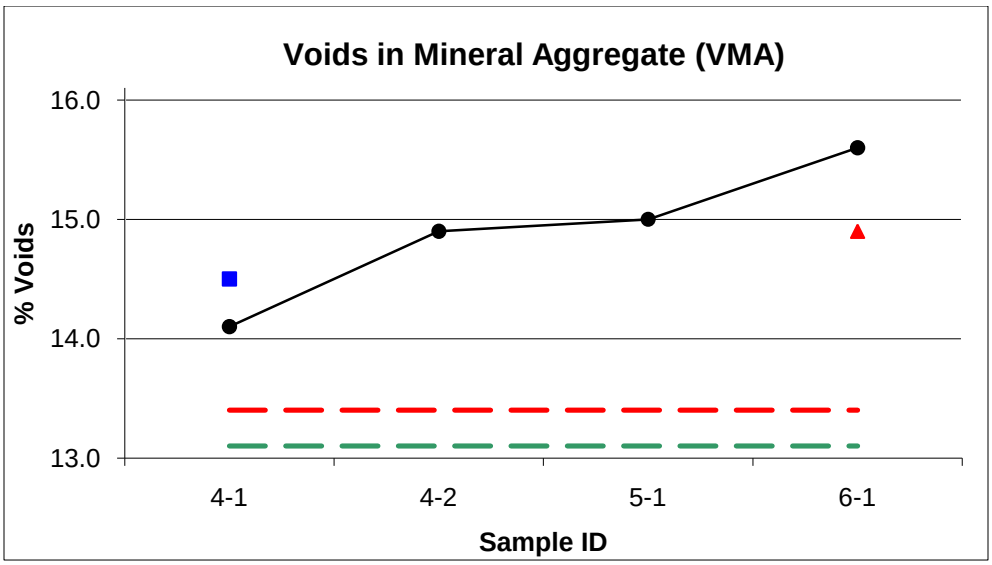
4.5
4.0
3.5
3.0
2.5
2.0
4-1
4-2
5-1
6-1
Sample ID


Voids in Mineral Aggregate (VMA)
% Voids
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15.0
14.0
13.0
4-1
4-2
5-1
6-1
Sample ID

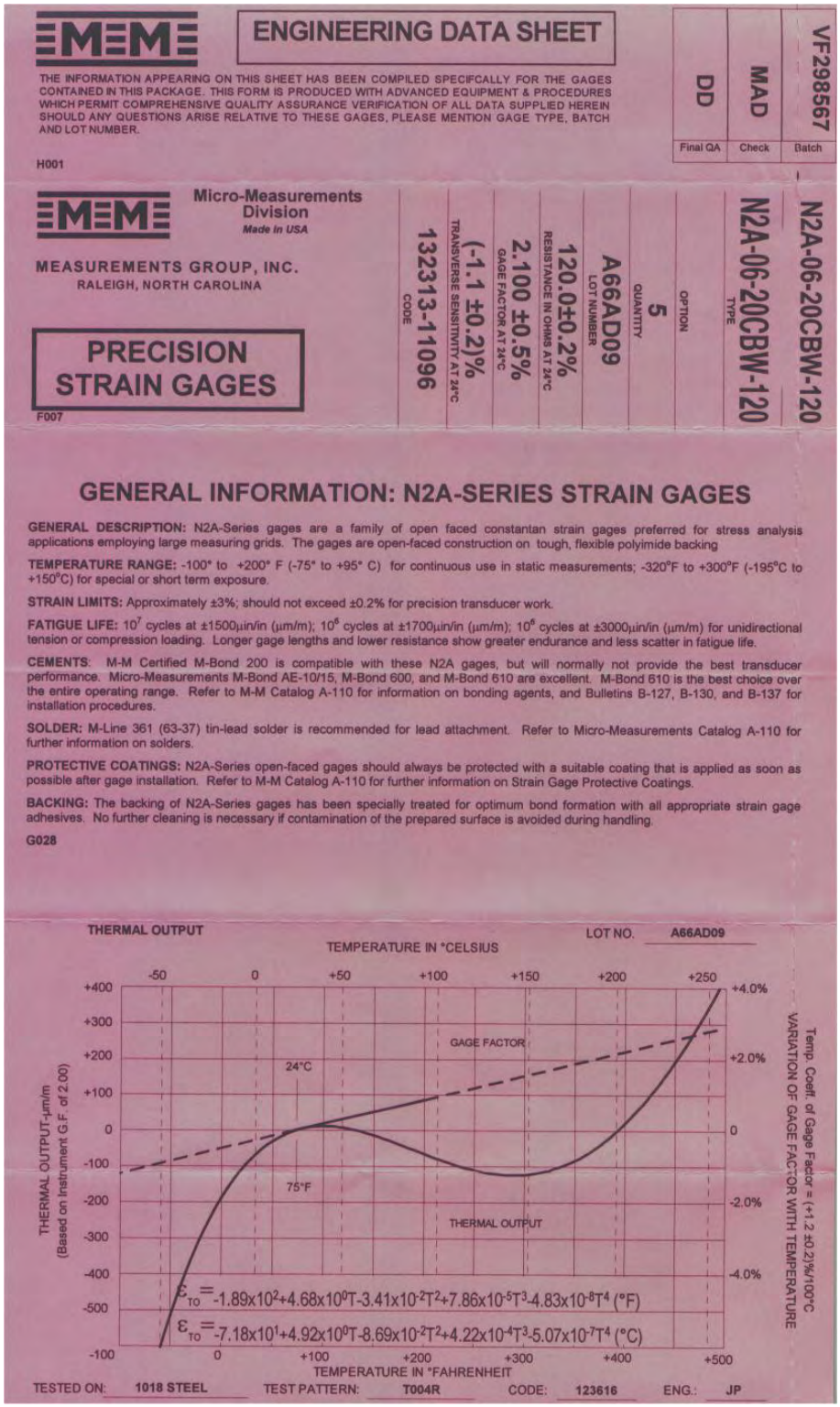
Appendix B Density Measurements

	Test Lot (by station)	Lower Layer			Middle Layer			Upper Layer		
		QC Density (%)	QV Density (%)	Target Density (%)	QC Density (%)	QV Density (%)	Target Density (%)	QC Density (%)	QV Density (%)	Target Density (%)
Test Section 1	3+020 to 3+050				95.4*			93.2		
	3+050 to 3+100				95.3			93.4	91.1	
	3+100 to 3+150	95.1			95.6	97.0*			91.3	
	3+150 to 3+200		96.7		96.6*	97.3				
	3+200 to 3+250		94.9		95.8	96.0*		92.4*	93.3	
	3+250 to 3+300	94.2*	93.4		94.7*				92.4	
	3+300 to 3+350			96		94.7	94	91.7	92.9	94
	3+350 to 3+400		94.7			94.7				
	3+400 to 3+450					94.0*				
	3+450 to 3+500	91.9	92.2		92.3				92.7	
	3+500 to 3+550	93.7	95.3		91.7				91.4	
	3+550 to 3+600				92.5	92.7*		92.8	91.1	
	3+600 to 3+620	92.4	93.9		90.8	96.1*				
Test Section 2	3+620 to 3+650	92.1*	94.1		92.7	95.2			93.8	
	3+650 to 3+700		90.7		90.6*	95.0		91.5*		
	3+700 to 3+750	93.4			95.1	97.2			89.7	
	3+750 to 3+800		95.4*	94	95.0	93.4	94		89.6	94
	3+800 to 3+850				93.8	95.3		90.9		
	3+850 to 3+900				92.7			90.9		
	3+900 to 3+950							92.7		

*Average of multiple measurements.

Note: metric stationing.

Appendix C Strain Gage and Temperature Sensor Specifications



ENGINEERING DATA SHEET

THE INFORMATION APPEARING ON THIS SHEET HAS BEEN COMPILED SPECIFCALLY FOR THE GAGES CONTAINED IN THIS PACKAGE. THIS FORM IS PRODUCED WITH ADVANCED EQUIPMENT & PROCEDURES WHICH PERMIT COMPREHENSIVE QUALITY ASSURANCE VERIFICATION OF ALL DATA SUPPLIED HEREIN SHOULD ANY QUESTIONS ARISE RELATIVE TO THESE GAGES, PLEASE MENTION GAGE TYPE, BATCH AND LOT NUMBER.

H001

Batch	Check	Final QA
VF298567	MAD	DD

Micro-Measurements Division
Made in USA

MEASUREMENTS GROUP, INC.
RALEIGH, NORTH CAROLINA

PRECISION STRAIN GAGES

F007

TYPE	N2A-06-20CBW-120
OPTION	
QUANTITY	5
LOT NUMBER	A66AD09
RESISTANCE IN OHMS AT 24°C	120.0±0.2%
GAGE FACTOR AT 24°C	2.100 ±0.5%
TRANSVERSE SENSITIVITY AT 24°C	(-1.1 ±0.2)%
CODE	132313-11096

N2A-06-20CBW-120

GENERAL INFORMATION: N2A-SERIES STRAIN GAGES

GENERAL DESCRIPTION: N2A-Series gages are a family of open faced constantan strain gages preferred for stress analysis applications employing large measuring grids. The gages are open-faced construction on tough, flexible polyimide backing

TEMPERATURE RANGE: -100° to +200° F (-75° to +95° C) for continuous use in static measurements; -320°F to +300°F (-195°C to +150°C) for special or short term exposure.

STRAIN LIMITS: Approximately ±3%; should not exceed ±0.2% for precision transducer work.

FATIGUE LIFE: 10^7 cycles at ±1500μin/in (μm/m); 10^6 cycles at ±1700μin/in (μm/m); 10^6 cycles at ±3000μin/in (μm/m) for unidirectional tension or compression loading. Longer gage lengths and lower resistance show greater endurance and less scatter in fatigue life.

CEMENTS: M-M Certified M-Bond 200 is compatible with these N2A gages, but will normally not provide the best transducer performance. Micro-Measurements M-Bond AE-10/15, M-Bond 600, and M-Bond 610 are excellent. M-Bond 610 is the best choice over the entire operating range. Refer to M-M Catalog A-110 for information on bonding agents, and Bulletins B-127, B-130, and B-137 for installation procedures.

SOLDER: M-Line 361 (63-37) tin-lead solder is recommended for lead attachment. Refer to Micro-Measurements Catalog A-110 for further information on solders.

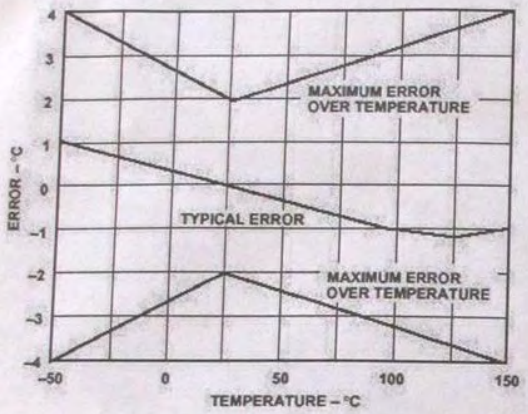
PROTECTIVE COATINGS: N2A-Series open-faced gages should always be protected with a suitable coating that is applied as soon as possible after gage installation. Refer to M-M Catalog A-110 for further information on Strain Gage Protective Coatings.

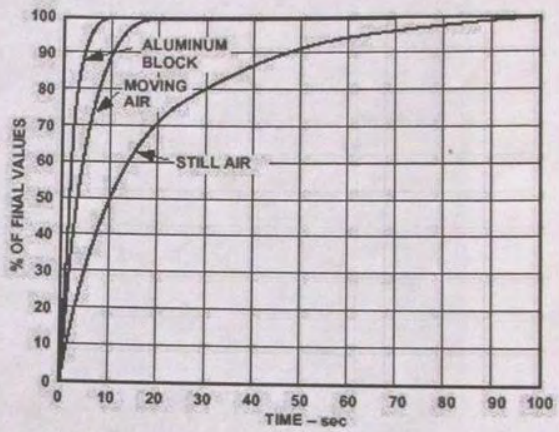
BACKING: The backing of N2A-Series gages has been specially treated for optimum bond formation with all appropriate strain gage adhesives. No further cleaning is necessary if contamination of the prepared surface is avoided during handling.

G028

Figure C-1. Strain gage engineering data sheet.

Temperature Sensor Data Sheet





Parameter		Units
Transfer Function	V_{OUT}=(V+/5 V) x [1.375 V + (22.5mV/°C x T_A]	V
Coefficient	(V+/5 V) x 22.5	mV/°C
Initial Error T_A =+25°C	±1.0	°C
Nonlinearity	1.0	%FS
Output V+=5.0 V, T_A =-50°C V+=5.0 V, T_A =+150°C	 0.250 4.750	 V V
Temperature Range	-50 to +150	°C

Figure C-2. Temperature sensor data sheet.

Appendix D Asphalt Binder Test Results

Abbreviations:

PAV	Pressure Aging Vessel
RTFO	Rolling Thing Film Oven

Surface Layer, Test Section 1

Listed Temperature Grade			76-28
Actual Temperature Grade			75.2-29.0
AASHTO T 316			
Rotational Viscosity			1.185 Pa.s
AASHTO T 315			
Dynamic Sheer	Original at 70°C		1.13 kPa
	RTFO at 70°C		2.00 kPa
	PAV at 28°C		1523 kPa
AASHTO T 240			
RTFO Mass Loss			-0.435%
AASHTO T 313			
Creep Stiffness at -18°C		Stiffness	208 MPa
		m-value	0.307

Verification testing result: Satisfactory within tolerance

Surface Layer, Test Section 2

Listed Temperature Grade			70-28
Actual Temperature Grade			69.1-33.1
AASHTO T 316			
Rotational Viscosity			0.875 Pa.s
AASHTO T 315			
Dynamic Sheer	Original at 70°C		1.07 kPa
	RTFO at 70°C		1.97 kPa
	PAV at 28°C		1180 kPa
AASHTO T 240			
RTFO Mass Loss			-0.356%
AASHTO T 313			
Creep Stiffness at -18°C		Stiffness	166 MPa
		m-value	0.341

Verification testing result: Satisfactory within tolerance

Middle Layer, Shipment A Sample

Listed Temperature Grade		70-22
Actual Temperature Grade		69.2-22.8
AASHTO T 316		
Rotational Viscosity		0.628 Pa.s
AASHTO T 315		
Dynamic Sheer	Original at 70°C	0.98 kPa
	RTFO at 70°C	1.97 kPa
	PAV at 28°C	2180 kPa
AASHTO T 240		
RTFO Mass Loss		-0.182%
AASHTO T 313		
Creep Stiffness at -12°C	Stiffness	198 MPa
	m-value	0.305

Verification testing result: Unsatisfactory

Middle Layer, Shipment B Sample

Listed Temperature Grade		70-22
Actual Temperature Grade		69.8-22.5
AASHTO T 316		
Rotational Viscosity		0.615 Pa.s
AASHTO T 315		
Dynamic Sheer	Original at 70°C	1.01 kPa
	RTFO at 70°C	2.08 kPa
	PAV at 28°C	2704 kPa
AASHTO T 240		
RTFO Mass Loss		-0.176%
AASHTO T 313		
Creep Stiffness at -12°C	Stiffness	201 MPa
	m-value	0.304

Verification testing result: Satisfactory within tolerance

Middle Layer, Shipment C Sample

Listed Temperature Grade		70-22
Actual Temperature Grade		69.7-23.0
AASHTO T 316		
Rotational Viscosity		0.618 Pa.s
AASHTO T 315		
Dynamic Sheer	Original at 70°C	1.02 kPa
	RTFO at 70°C	2.08 kPa
	PAV at 28°C	2969 kPa
AASHTO T 240		
RTFO Mass Loss		-0.203%
AASHTO T 313		
Creep Stiffness at -12°C	Stiffness	185 MPa
	m-value	0.299

Verification testing result: Satisfactory within tolerance

Middle Layer, Shipment D Sample

Listed Temperature Grade		70-22
Actual Temperature Grade		69.6-22.1
AASHTO T 316		
Rotational Viscosity		0.619 Pa.s
AASHTO T 315		
Dynamic Sheer	Original at 70°C	1.01 kPa
	RTFO at 70°C	2.04 kPa
	PAV at 28°C	2987 kPa
AASHTO T 240		
RTFO Mass Loss		-0.189%
AASHTO T 313		
Creep Stiffness at -12°C	Stiffness	185 MPa
	m-value	0.301

Verification testing result: Satisfactory within tolerance

Middle Layer, Shipment E Sample

Listed Temperature Grade		70-22
Actual Temperature Grade		71.5-22.9
AASHTO T 316		
Rotational Viscosity		0.635 Pa.s
AASHTO T 315		
Dynamic Sheer	Original at 70°C	1.01 kPa
	RTFO at 70°C	2.21 kPa
	PAV at 28°C	3321 kPa
AASHTO T 240		
RTFO Mass Loss		-0.186%
AASHTO T 313		
Creep Stiffness at -12°C	Stiffness	206 MPa
	m-value	0.306

Verification testing result: Satisfactory

Bottom Layer, Shipment A Sample

Listed Temperature Grade		64-22
Actual Temperature Grade		67.0-23.9
AASHTO T 316		
Rotational Viscosity		0.439 Pa.s
AASHTO T 315		
Dynamic Sheer	Original at 70°C	1.30 kPa
	RTFO at 70°C	3.04 kPa
	PAV at 28°C	3800 kPa
AASHTO T 240		
RTFO Mass Loss		-0.179%
AASHTO T 313		
Creep Stiffness at -12°C	Stiffness	195 MPa
	m-value	0.314

Verification testing result: Satisfactory

Bottom Layer, Shipment B Sample

Listed Temperature Grade			64-22
Actual Temperature Grade			67.2-23.3
AASHTO T 316			
Rotational Viscosity			0.455 Pa.s
AASHTO T 315			
Dynamic Sheer	Original at 70°C		1.35 kPa
	RTFO at 70°C		3.09 kPa
	PAV at 28°C		3929 kPa
AASHTO T 240			
RTFO Mass Loss			-0.199%
AASHTO T 313			
Creep Stiffness at -12°C		Stiffness	204 MPa
		m-value	0.319

Verification testing result: Satisfactory

Bottom Layer, Shipment C Sample

Listed Temperature Grade			64-22
Actual Temperature Grade			67.0-22.7
AASHTO T 316			
Rotational Viscosity			0.438 Pa.s
AASHTO T 315			
Dynamic Sheer	Original at 70°C		1.35 kPa
	RTFO at 70°C		2.89 kPa
	PAV at 28°C		3698 kPa
AASHTO T 240			
RTFO Mass Loss			-0.194%
AASHTO T 313			
Creep Stiffness at -12°C		Stiffness	209 MPa
		m-value	0.327

Verification testing result: Satisfactory

Appendix E NC Superpave Center Material Analysis Report

WISCONSIN PERPETUAL PAVEMENT PROJECT

SUMMARY OF MIX DATA ANALYSES

Submitted by

Ayesha Shah

Becky McDaniel

NORTH CENTRAL SUPERPAVE CENTER

WEST LAFAYETTE, IN

JANUARY 2008

WISDOT DATA ANALYSIS

This report summarizes the results from some of the tests that were conducted on hot mix asphalt (HMA) samples that were collected from different layers of a pavement section in Wisconsin. The two test sections were constructed as a part of the Kenosha Weigh Station Perpetual Pavement Project. Each section consisted of three layers; surface, middle and base. The only difference between the two surface layers was the binder grade used in the mix. The middle HMA layer was common to both test sections, while the two base layers were compacted to different densities. Figure 1 below shows a cross-section schematic of the pavement.

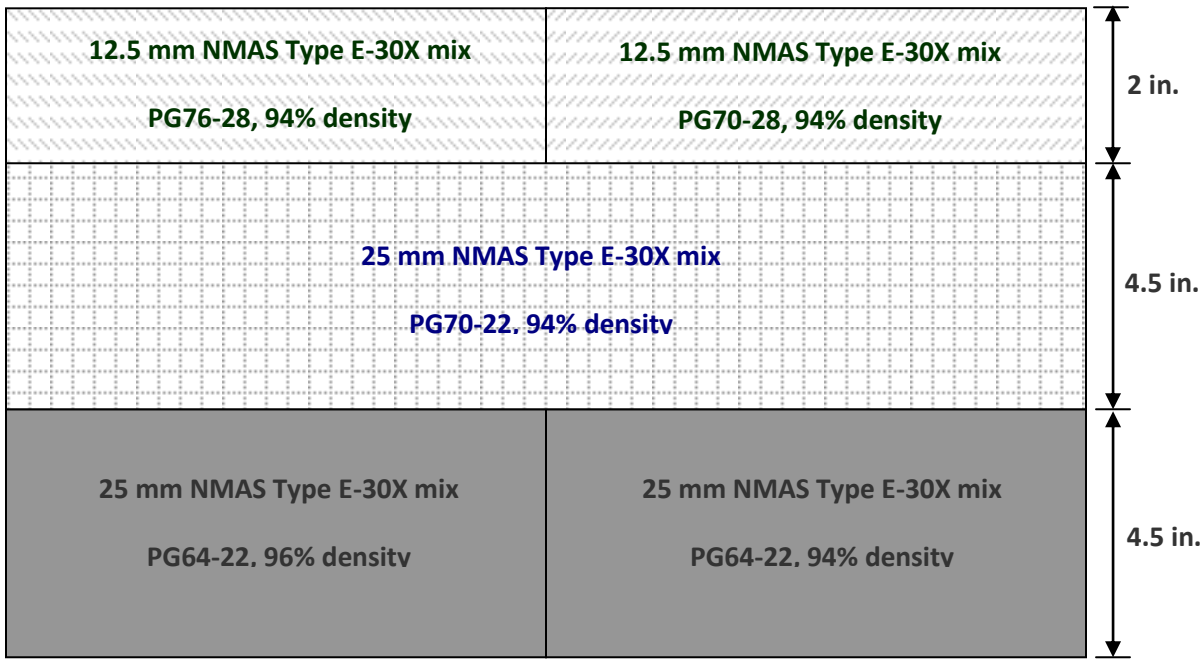


Figure 1 Cross-sectional layout of the pavement

REPEATED SHEAR TEST DATA

This test is conducted to assess the rut resistance of HMA when subjected to repeated shear load, in accordance with AASHTO T320. In other words, it used to identify mixtures that are likely to exhibit tertiary flow (plastic flow) due to mixture instability. In this test, the HMA sample is held between two platens and subjected to repeated shear stress of 69 ± 5 kPa for a period of 0.1 s followed by a rest period of 0.6 s to allow the sample to recover between the applied load pulses. Constant specimen height is maintained (within ± 0.013 mm) by adjusting the vertical load through a feedback loop. The

test is run until 5000 cycles are completed or 5% permanent strain is reached, whichever occurs earlier. The permanent strain as a function of load cycles is recorded throughout the test duration. The cumulative shear deformation (and hence the shear strain) of the test specimen at the end of 5000 load cycles is used to assess the expected rut resistance in the field. This test is typically conducted at the effective pavement temperature for permanent deformation; 58°C in this case.

Figure 2 shows the average plots of the cumulative permanent strain for the mixes studied. Four to five replicates per mix were tested. The maximum permanent cumulative strains of the samples, along with mean and standard deviation are shown in Table 1. No data points were deleted in calculations.

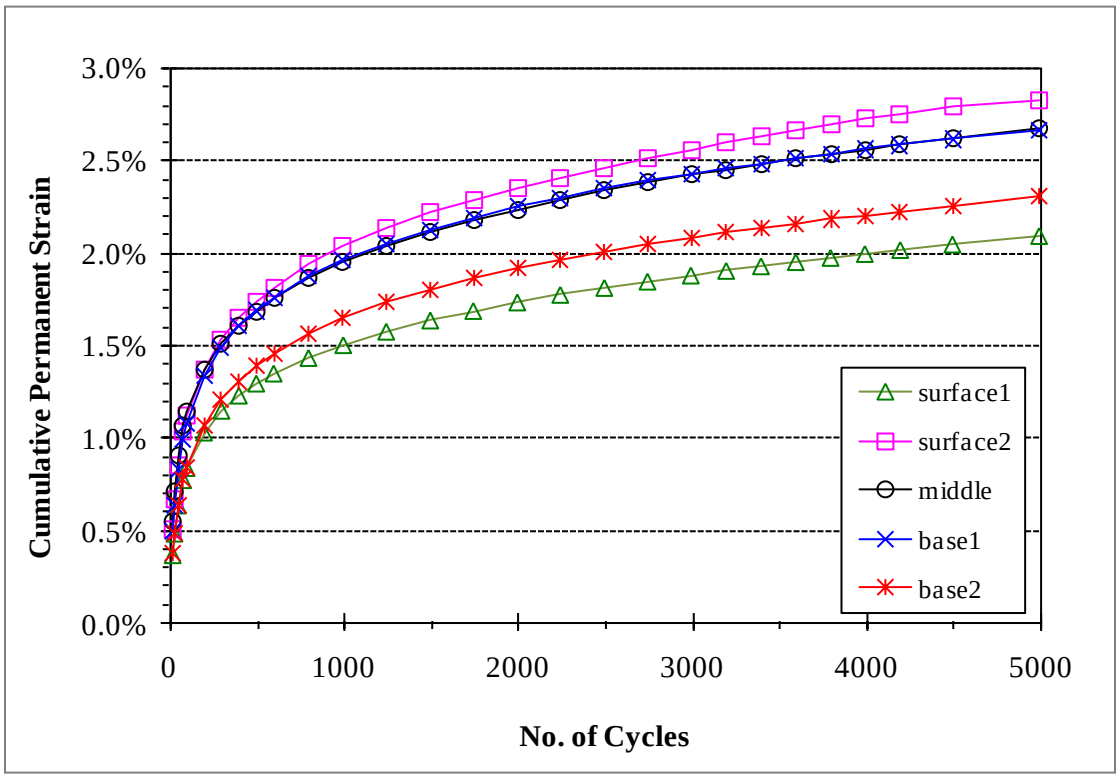


Figure 2 Cumulative permanent strain at 58°C

Table 1 Percent maximum cumulative permanent strain of the mixes at 58°C

Layer	Surface1	Surface2	Base1	Base2	Middle
Rep 1	1.86	2.07	2.91	2.16	2.22
Rep 2	1.87	3.20	2.76	2.10	3.08
Rep 3	1.62	2.56	2.39	2.37	3.09
Rep 4	2.72	3.13	2.92	2.24	2.31
Rep 5	2.35	3.16	2.32	2.65	
Mean	2.09	2.82	2.66	2.30	2.67
Std. Dev.	0.445	0.497	0.286	0.218	0.471
C. V., %	21.4	17.6	10.8	9.5	17.6
RANKING	5	1	3	4	2

This test is typically conducted on samples compacted to low air void content (3 - 4%) when the mix is particularly prone to rutting. The percent air-void of the samples tested was not known. It may be noted that in accordance with the general design procedures, surface mixes are typically designed to withstand higher loads and be more rut resistant than the underlying layers. The percent cumulative strain of the mixes ranged between 2.09% (Surface1) and 2.82% (Surface2). Based on the Asphalt Institute's recommendations, mixes with cumulative strain between 2% and 3% are expected to show fair performance, while mixes with strains greater than 3% are expected to show poor performance. Mixes with strains less than 1% and between 1% and 2% are expected to show excellent and good performance, respectively. However, these guidelines apply to low void content mixes, so may or may not be applicable here.

Single factor ANOVA on all the data points indicated that the mixes were statistically different. Further analysis was conducted on the surfaces mixes and the base mixes separately. Statistical comparison of means of the two surface layers indicated that the mean cumulative strains in the two mixes were significantly different. The lower cumulative strain observed in Surface1 may be attributed to the stiffer binder (PG76-28) used in Surface1 compared with that used in Surface2 (PG70-28). Accordingly, Surface1 may be expected to show minimal or lower in-service rutting. There is a degree of uncertainty associated with this conclusion due to the relatively high coefficient of variation (and standard deviation) in the test results, which is further compounded by the unknown air content of the samples tested.

The two base mixes had the same binder grade and NMAS, but were compacted to slightly different percent density. No significant differences were found between the two base mixes. The coefficient of variation of the base mixes was lower than that observed in the Surface mixes. Although the Middle layer had a slightly stiffer binder grade than Base2 mix (with the same percent density and NMAS), it showed lower rut resistance than the Base2 mix. No aggregate gradation data were available for any of the mixes tested.

FREQUENCY SWEEP TEST DATA

Frequency Sweep at Constant Height is a performance-related test conducted on field cores or lab-compacted HMA samples to determine the shear stiffness at a given test temperature, in accordance with AASHTO T320. Shear loads are applied at different frequencies simulating the different traffic speeds that occur at the pavement surface during its service life. The results of this test are indicative of the rut resistance of the HMA used in the pavement. In the frequency sweep at constant height test, the test sample is held between two platens and subjected to sinusoidal shear strain cycles of 0.0001 mm/mm amplitude at different frequencies. As the sample tends to deform under the applied shear strain, the axial load is controlled through a feedback loop to maintain constant height of the specimen. The shear load required to apply the desired strain level is recorded, along with the phase lag between applied shear strain and shear load and used to determine the complex shear modulus (|G*|) of the mix.

Five samples per each mix were tested. Table 2 shows the average complex shear moduli (|G*|) of the mixes, along with the coefficients of variation. Based on the classification scheme suggested by the Asphalt Institute, mixes may be expected to show bad, fair or excellent performance under shear loading at 40ºC if their moduli were less than 22,000 psi, 22,000 psi - 35,000 psi and 35,000 psi - 50,000 psi, respectively. Accordingly, Surface2 mix is expected to show poor performance, Surface1 mix to show fair performance and the remaining three mixes to show excellent performance. Figure 3 shows the same data in graphical format. Asphalt Institute recommendations were based on tests conducted on samples compacted to 7 ± 0.5% air voids. Since the actual percent air voids of the samples tested is not known, the conclusions drawn may not agree with observed field performance.

ANOVA tests indicated that the mixes were statistically different at each test temperature. Further testing using a two-sample t-test indicated that there were no significant differences between Base1 and Base2 mixes, at all three test temperatures. However, the mean |G*| of mixes from Surface1 and Surface2 were found to be statistically different. Surface1 was consistently stiffer than Surface2, as expected based on the stiffer binder it contains. Similar results were observed in the case of Repeated Shear test results as well. Mix from the middle layer was found to have the highest stiffness at all temperatures.

Table 2 Average complex shear modulus of the mixes

Layer	20ºC		40ºC		50ºC	
	|G*|, psi	c. v., %	|G*|, psi	c. v., %	|G*|, psi	c. v., %
Surface1	238770	8.9	38614	10.1	14075	9.3
Surface2	154707	3.9	17031	6.4	6353	8.7
Base1	423682	9.9	65009	10.4	19234	1.6
Base2	487005	11.0	68649	3.1	21094	3.8
Middle	499370	10.3	102172	17.6	32328	26.0

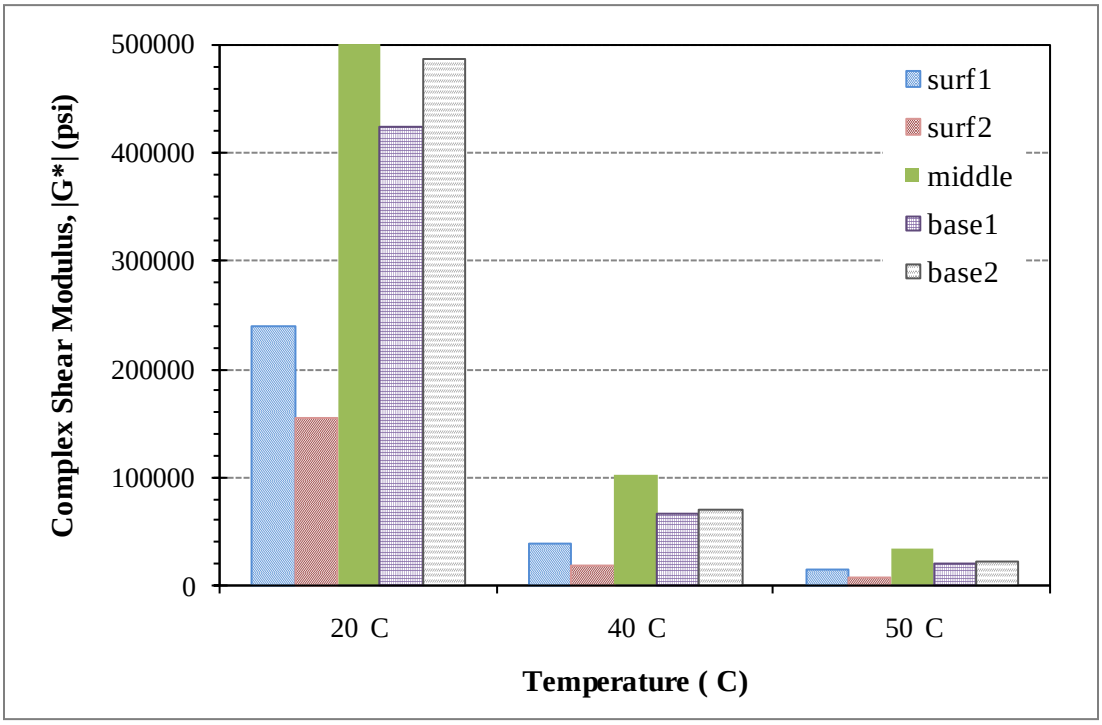


Figure 3 Complex shear modulus at different test temperatures

DYNAMIC MODULUS TEST DATA

Dynamic modulus testing is similar to frequency sweep testing, with the exception that the applied loading is axial compression in the former case, whereas the loading direction is shear (diametric) in the latter case. Repeated loading is applied to the test specimen at different frequencies and the resultant vertical deformation (recoverable and permanent) is measured, along with the lag between the time of peak load and peak strain, in accordance with AASHTO TP62. These data are then used to determine the complex dynamic modulus (|E*|) of the material tested. Testing is required at a minimum of three temperatures to generate a master curve (see Figure 4). A master curve represents the behavior (|E*|, in this case) of a mix over a range of temperatures and frequencies. Using a master curve, it is possible to estimate the modulus of the mix at different test temperatures for a given traffic speed (in terms of reduced frequency). Only the mixes from the Surface and Middle layers were tested at three temperatures. Master curves could not be generated for mixes from the Base layers as they were tested at only two temperatures.

The moduli of the Middle layer were found to be higher than those of the two Surface mixes at all frequencies and temperatures. Between the two Surface layers, the mix from the Surface 1 was found to be stiffer than the mix from Surface 2, as expected, due to the stiffer binder grade used in Surface 1.

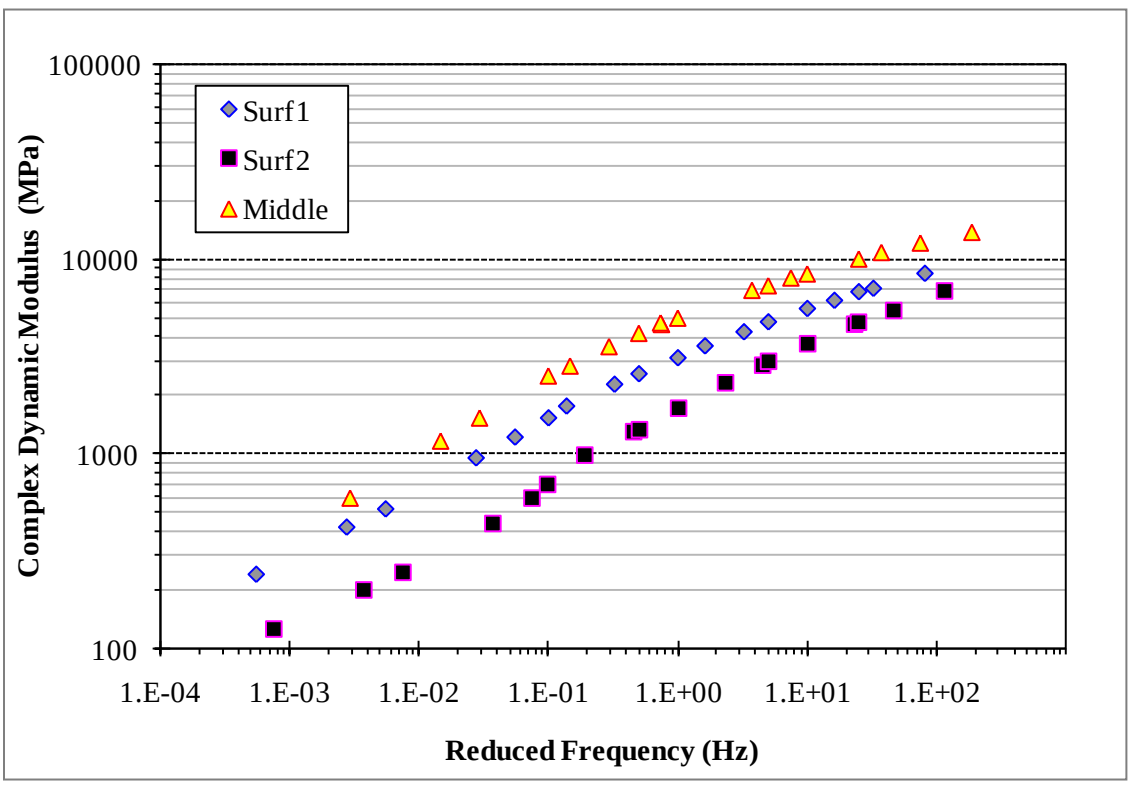


Figure 4 Master curves for complex dynamic modulus of the Surface and Middle layers

Four to seven samples were tested for each mix. Table 3 shows the mean and coefficient of variation of the samples tested. The mixes from the Base and the Middle layers had higher moduli than the Surface layers, at all test temperatures. This may be attributed to the predominant influence of the larger aggregate size (higher NMAS) used in the Base and Middle layer mixes in spite of the softer binder grade used in these layers in comparison with that used in the Surface mixes. Statistical analysis (two sample t-test) was conducted to look for differences between the mean complex dynamic moduli of the two Surface layers and the two Base layers. While the mean moduli of the Surface layers were found to be significantly different at the three test temperatures, no significant differences were found between the Base layers at the two temperatures tested.

Table 3 Average complex dynamic moduli of the mixes

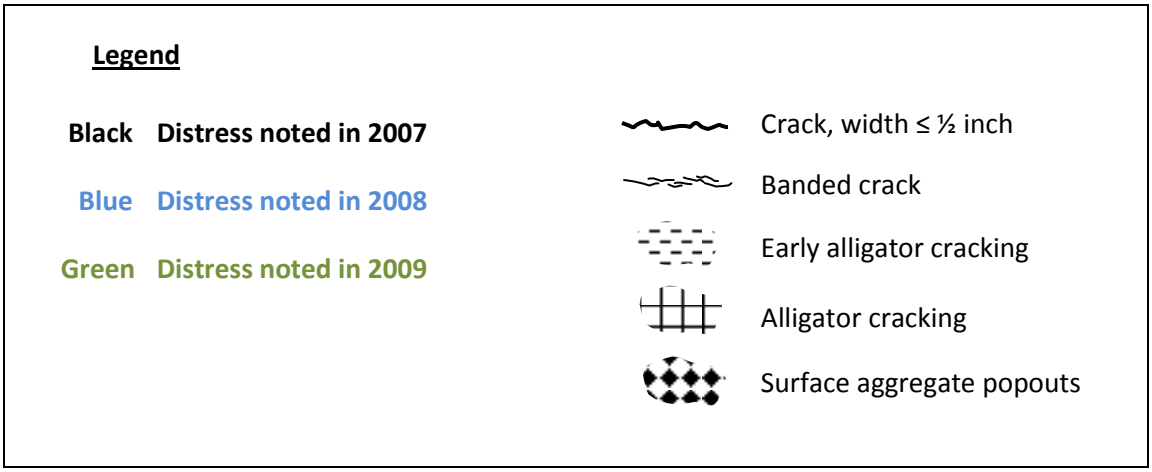
Layer	**15.9ºC**		**21.6ºC**		**40.6ºC**	
	|E*|, MPa	**c. v., %**	**|E*|, MPa**	**c. v., %**	**|E*|, MPa**	**c. v., %**
Surface1	8472	7.1	6819	8.7	1764	16.4
Surface2	6801	5.5	4757	9.4	973	14.5
Base1	15089	9.3	11392	8.7	Not tested	
Base2	12550	10.5	10396	15.4	Not tested	
Middle	13717	5.5	9998	13.1	4692	15.4

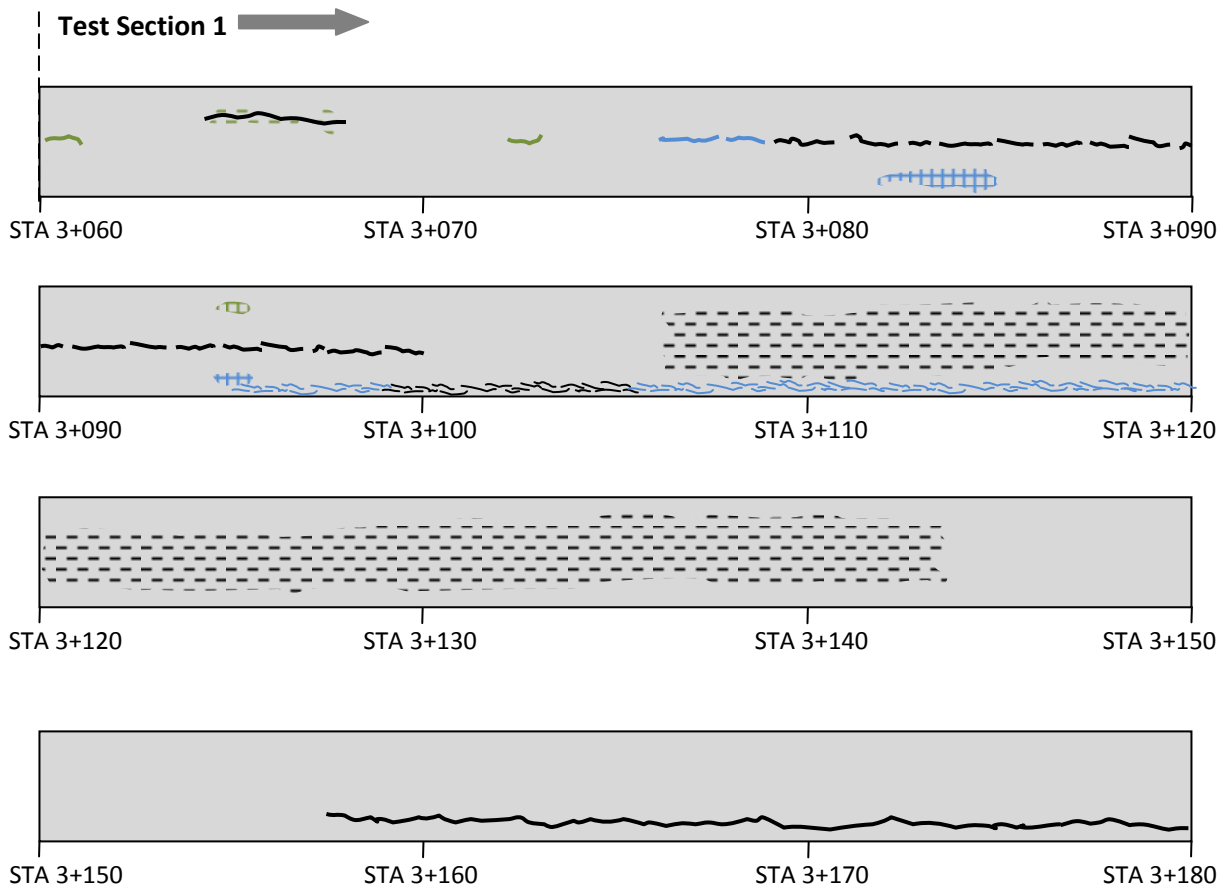
CONCLUSIONS

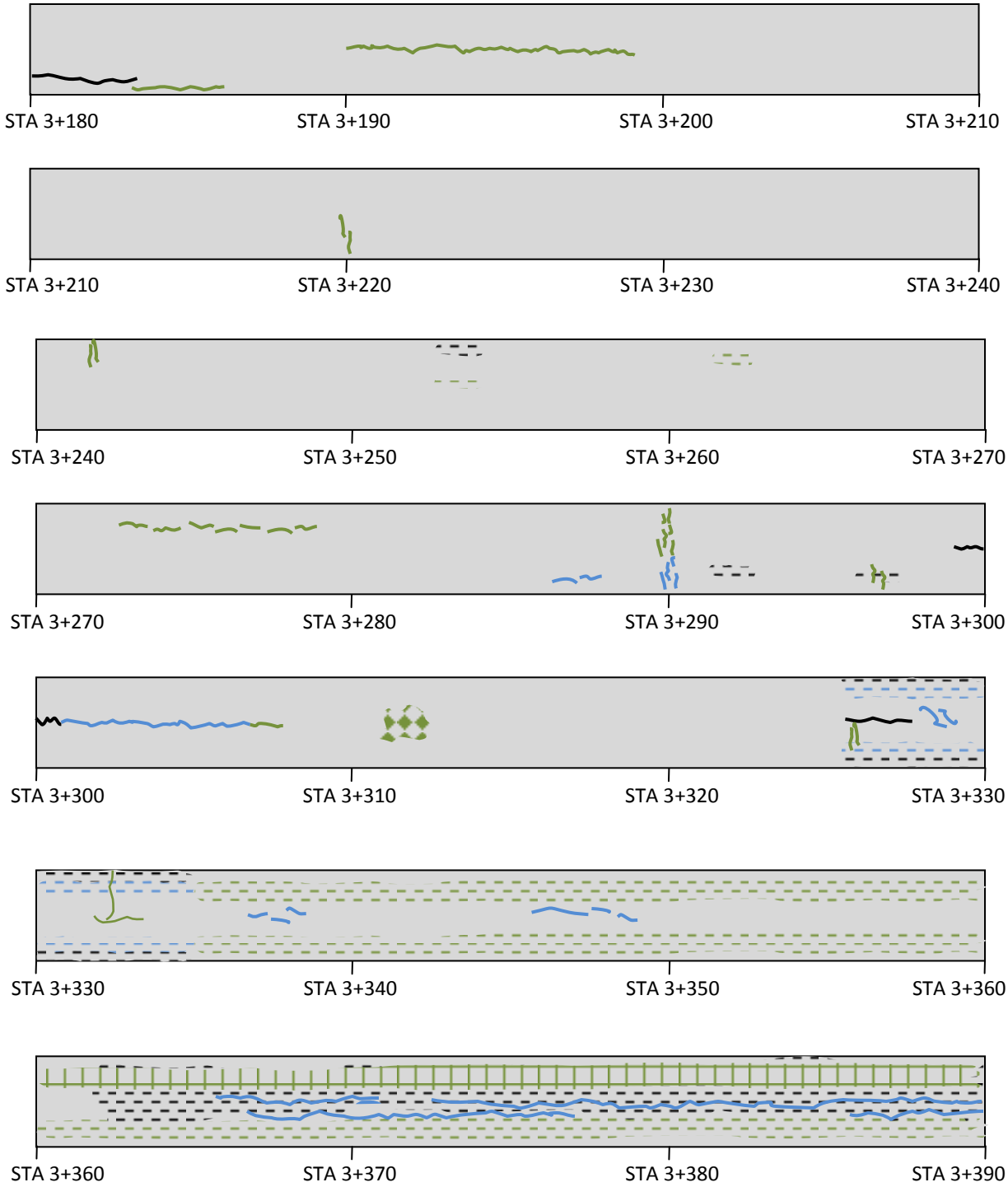
Analysis of results obtained from HMA testing indicated that, in general, the Middle layer had the highest complex dynamic and shear moduli. Of the two surface layers, Surface 1 may be expected to show better performance (in the three properties discussed) than Surface 2, which may be attributed to the stiffer binder grade used in Surface 1 (assuming all other volumetric properties, aggregate types and gradations used in these layers are similar). The two Base layers showed average rut resistance, but performed very well in the modulus tests (shear and dynamic). Surface 2 mix consistently showed poor performance (lowest rank) in all the tests summarized here.

Any inconsistencies in the conclusions observed between the tests may be attributed to the differences in percent air voids and other volumetric properties of the mixes. It was assumed that the percent air voids of the replicate samples of all the mixes were similar (within ±0.5%).

Appendix F Distress Survey Results



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STA 3+210
STA 3+210
STA 3+220
STA 3+230
STA 3+240
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STA 3+270
STA 3+270
STA 3+280
STA 3+290
STA 3+300
STA 3+300
STA 3+310
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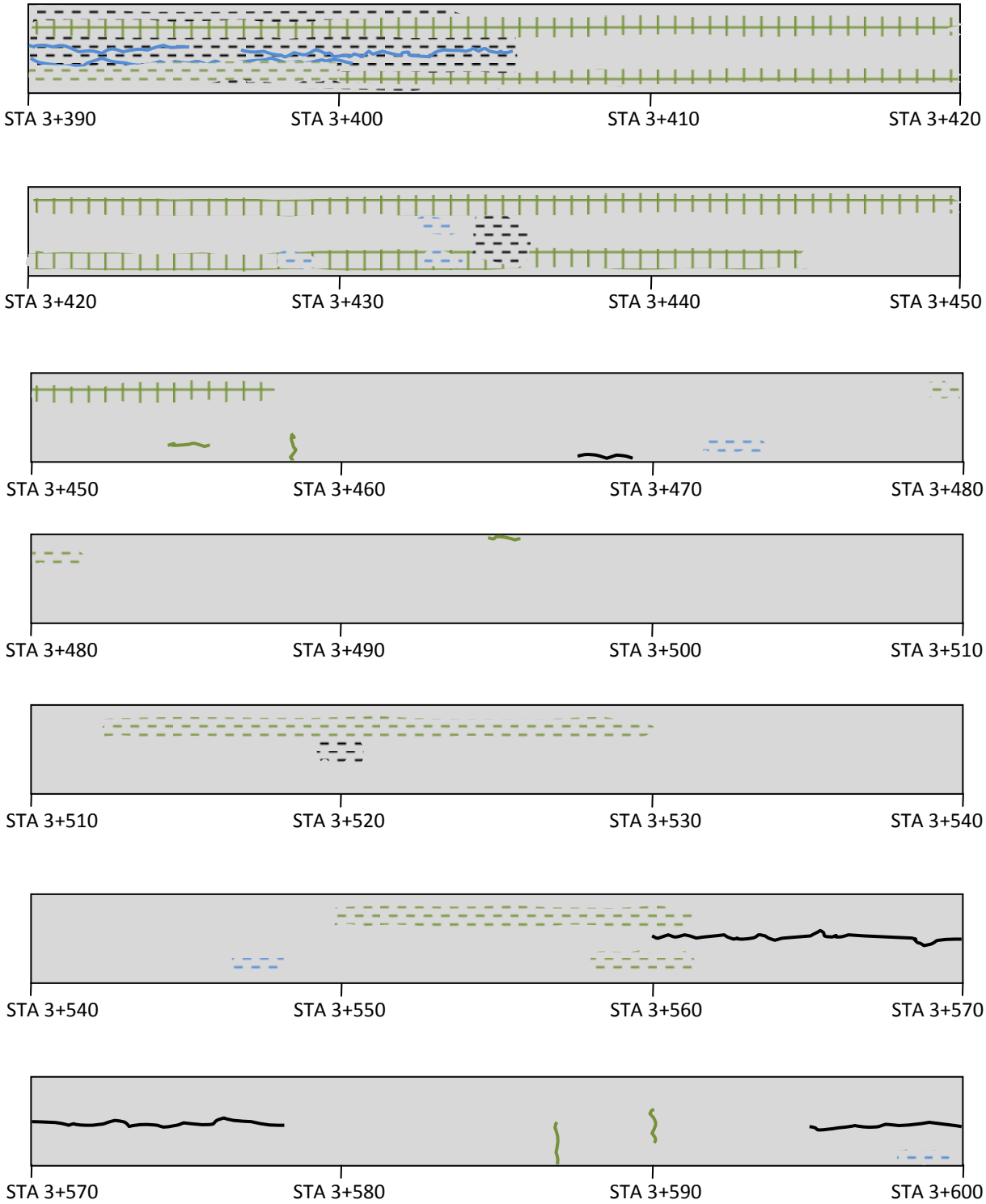
STA 3+360
STA 3+360
STA 3+370
STA 3+380
STA 3+390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STA 3+480
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STA 3+500
STA 3+510
STA 3+510
STA 3+520
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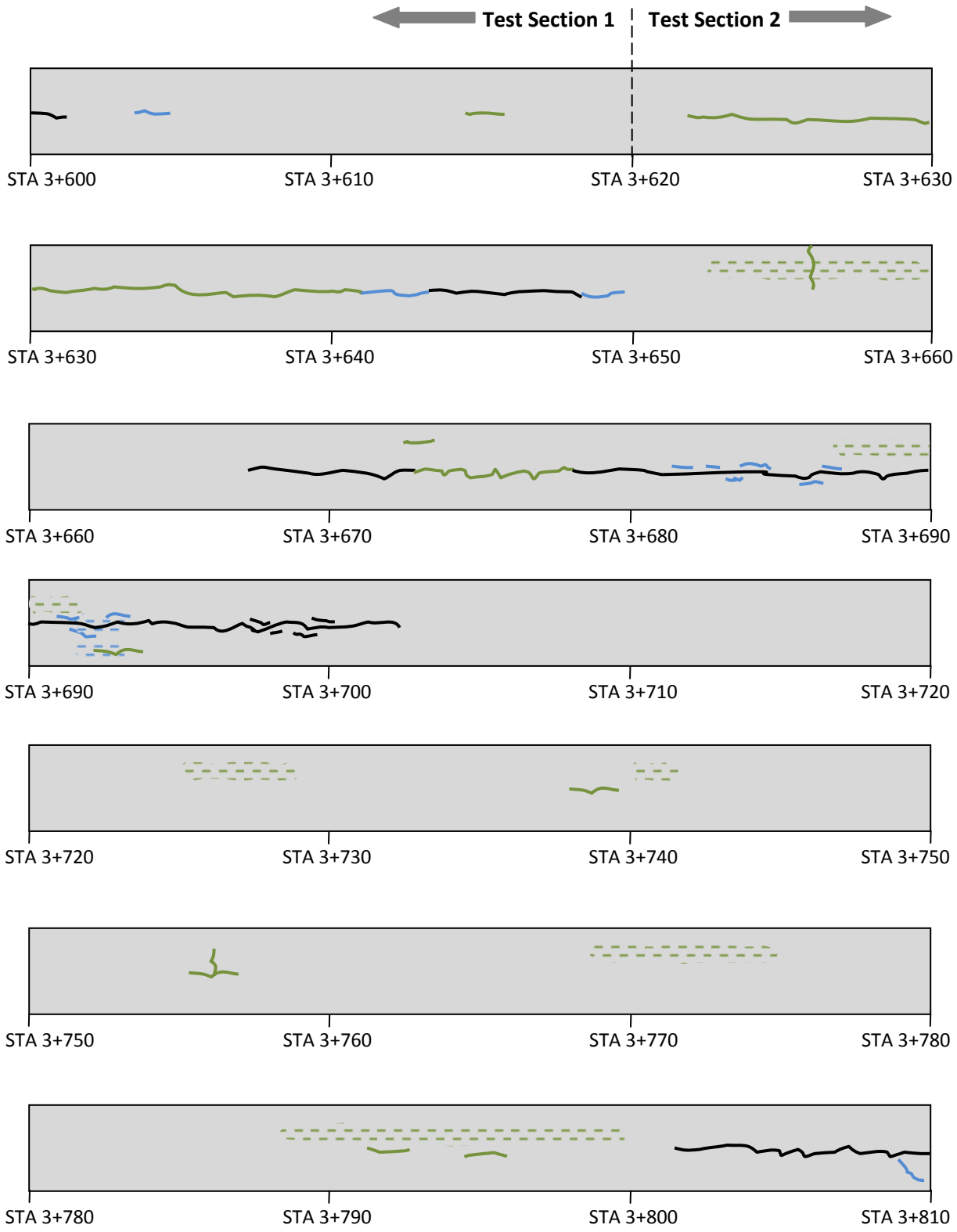
STA 3+570
STA 3+580
STA 3+590
STA 3+600


Test Section 1
Test Section 2
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STA 3+620
STA 3+630
STA 3+630
STA 3+640
STA 3+650
STA 3+660
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STA 3+670
STA 3+680
STA 3+690
STA 3+690
STA 3+700
STA 3+710
STA 3+720
STA 3+720
STA 3+730
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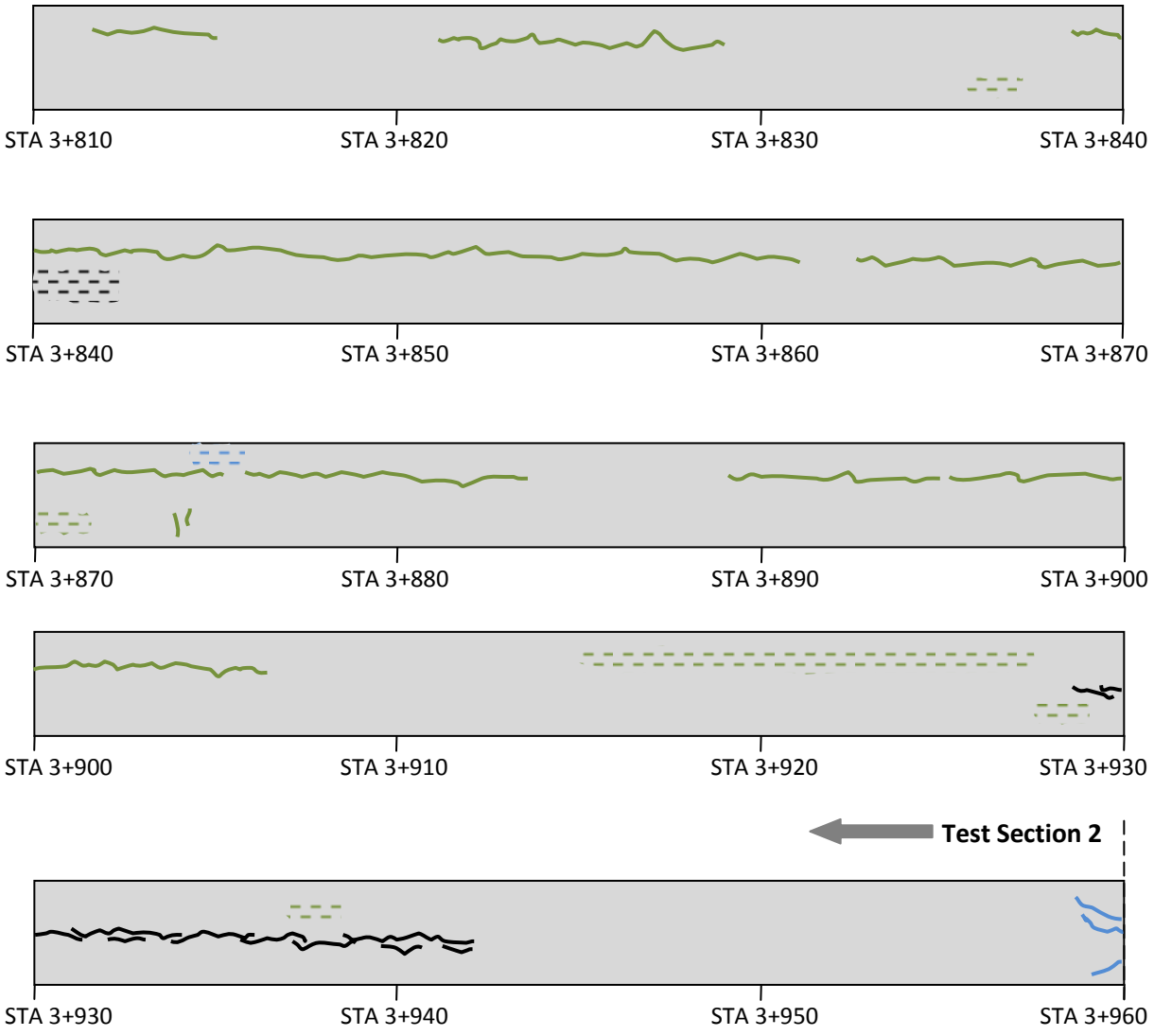
STA 3+790
STA 3+800
STA 3+810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STA 3+840
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STA 3+850
STA 3+860
STA 3+870
STA 3+870
STA 3+880
STA 3+890
STA 3+900
STA 3+900
STA 3+910
STA 3+920
STA 3+930
Test Section 2
STA 3+930
STA 3+940
STA 3+950
STA 3+960

Appendix G Soil Analysis Summaries from Forensic Coring Investigation

REPORT ON TESTS ON SOILS
EL140(L) 1-78

Wisconsin Department of Transportation
Div. of Transportation Infrastructure Development
Bureau of Highway Construction
3502 Kinsman Blvd., Madison, WI 53704 - 2507

Subgrade Analysis

Page 1 of 1

TEST NUMBER 230-4-07

	MO.-DAY-YR.			PROJECT ID	TEST CODE		QUANTITY
	4	17	07	0092-43-03	190	2	4 jars & 5 bags

County	Kenosha
Project Name	Kenosha Weigh Station
Contractor	
Material	Soil – subgrade samples from split-spoon
Source	ramp
Tests Requested By	Foundation & Pavements
Submitted by:	Irene LaBarca
Date	4/9/07

BORING NUMBER	B-1	B-2	B-4	B-5		
SAMPLE NUMBER	2A &2B	3 & 4	2,3 & 4	2 & 3		
DEPTH, FT.	4 - 6.5	6 - 10	3.5 - 10	3.5- -8		
% PASSING (AASHTO T-11, T-27 & T-248)						
2"						
1 1/2"						
1"						
3/4"	100		100			
1/2"	98	100	97	100		
3/8"	97	99	96	99		
# 4	89	98	95	97		
# 10	83	96	92	95		
# 40	75	90	86	90		
# 100	64	82	78	83		
# 200	59.5 ✓	77.4 ✓	72.7 ✓	78.3 ✓		
LIQUID LIMIT (AASHTO T-89)	28	27	29	27		
PLASTICITY INDEX (AASHTO T-90)	13	10	13	11		
UNIFIED CLASSIFICATION (ASTM D 2487)	CL	CL	CL	CL		
AASHTO CLASSIFICATION (AASHTO M-145)	A-6 (5) ✓	A-4 (6) ✓	A-6 (7) ✓	A-6 (6) ✓		
FAA CLASSIFICATION						
LOSS ON IGNITION, % (AASHTO T-267)						
MOISTURE CONTENT, % (AASHTO T-265)	13.8	16.6	16.2	15.4		
COMPACTION TEST						
AASHTO T -99, METHOD						
OPTIMUM MOISTURE, %						
MAXIMUM DENSITY, PCF						
D_{50}, mm						
D_{90}, mm						

Remarks

DISTRIBUTION: Geotechnical Unit
District

By WRK

REPORT ON TESTS ON SOILS
EL140(L) 1-78

Wisconsin Department of Transportation
Div. of Transportation Infrastructure Development
Bureau of Highway Construction
3502 Kinsman Blvd., Madison, Wi 53704 - 2507

Base Course Analysis

TEST NUMBER 230-4-07

	MO.-DAY-YR.			PROJECT ID	TEST CODE		QUANTITY
	4	17	07	0092-43-03	190	2	4 jars & 5 bags

County	Kenosha
Project Name	Kenosha Weigh Station
Contractor	
Material	Soil – Crushed aggregate base course (CABC)
Source	ramp
Tests Requested By	Foundation & Pavements
Submitted by:	Irene LaBarca
Date	4/9/07

BORING NUMBER	1A	2A	3A	4A	5A
SAMPLE NUMBER	1	1	1	1	1
DEPTH, FT.	1 - 3	1 - 3	1 - 3	1 - 3	1 - 3
% PASSING (AASHTO T-11, T-27 & T-248)					
2"					
1 1/2"					
1"	100	100	100		100
3/4"	99	98	99	100	98
1/2"	93	89	92	94	91
3/8"	85	81	82	87	85
# 4	69	68	66	70	70
# 10	53	54	51	54	56
# 40	31	35	31	33	35
# 100	18	21	20	20	20
# 200	13.9	16.4	15.4	14.4	15.2
LIQUID LIMIT (AASHTO T-89)	Non-Coh	Non-Coh	Non-Coh	Non-Coh	Non-Coh
PLASTICITY INDEX (AASHTO T-90)	NP	NP	NP	NP	NP
UNIFIED CLASSIFICATION (ASTM D 2487)					
AASHTO CLASSIFICATION (AASHTO M-145)					
FAA CLASSIFICATION					
LOSS ON IGNITION, % (AASHTO T-267)					
MOISTURE CONTENT, % (AASHTO T-265)	6.2	6.7	6.5	8.1	6.5
COMPACTION TEST					
AASHTO T-99, METHOD					
OPTIMUM MOISTURE, %					
MAXIMUM DENSITY, PCF					
D_{50}, mm					
D_{90}, mm					

Remarks

DISTRIBUTION: Geotechnical Unit
District

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