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16. Abstract

This project was conducted to determine the correlation of field performance to Hamburg Wheel Tracking Device (HWTD) testing results. The HWTD measures the combined effects of rutting and moisture damage by rolling a steel wheel across the surface of an asphalt concrete specimen that is immersed in hot water. The test results from this laboratory equipment have been promising in regard to evaluating the moisture susceptibility of hot mix asphalt mixtures. This five-year research project will be an important step in validating the test and ensuring that the test results could be used reliably to predict performance. Three designs (Superpave, CMHB-C, and Type C) and three aggregate sources (siliceous gravel, sandstone, and quartzite) were used for this study. The test sections, including nine different mixture designs, were constructed on IH 20 in Harrison County to observe the performance of the overlays under real traffic conditions. Field performance will be observed through visual pavement condition surveys and nondestructive tests for four years. This research report summarizes the nondestructive test results and visual pavement condition surveys in the fourth year of this study.

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Pavement Performance Evaluation by Using Field Data

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Preface

This is the fourth report from the Center for Transportation Research (CTR) on Project 0-4185. To evaluate the laboratory-field correlation for the Hamburg Wheel Tracking Device (HWTD), nine test sections were constructed on IH 20 in Harrison County. This research includes monitoring the construction of these test sections, collection of construction data, performance data through a five-year period, performance of laboratory tests using the HWTD, and analysis of the collected information. This report presents the results and findings of the information collected from the test sections for the fourth year of a five-year project.

Acknowledgments

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Disclaimers

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1. Introduction

1.1 Objective

The objective of this study is to determine the relationship between hot mix asphalt (HMA) field performance and Hamburg Wheel Tracking Device (HWTD) test results. The project will be completed in a total of five years. Test sections were built on IH 20 in Harrison County. Nine different types of overlay on continuously reinforced concrete pavement (CRCP) were placed in December 2001. Test sections are being monitored for four years by the Center for Transportation Research (CTR) at The University of Texas at Austin.

Three mix design methods (Superpave, CMHB-C, and Type C) and three aggregate sources (siliceous gravel, sandstone, and quartzite) were used for this study. The test sections, including all mixture designs, were constructed on IH 20 in Harrison County to observe the performance of the overlays under real traffic conditions. Type B mixture was used for all overlays as a base layer.

The HWTD was utilized to determine the laboratory performance of samples. Field performance will be observed through visual pavement condition surveys and nondestructive tests (NDTs) for four years. NDTs include falling weight deflectometers (FWD), portable seismic pavement analyzers (PSPA), and rolling dynamic deflectometers (RDD). In addition, visual pavement condition surveys are being performed at the end of each year. Field performance is being monitored every year until 2005. The HWTD results and the field performance of the overlays will be gathered and compared at the end of the project to determine the behavior of the mixture types, and a guideline will be developed to correlate HWTD results and field performance.

1.2 Background

The HWTD is a wheel-tracking device used to simulate field traffic effects on HMA in terms of rutting and moisture-induced damage (Yildirim and Kennedy 2002). This equipment measures the combined effects of rutting and moisture damage by rolling a steel wheel across the surface of an asphalt concrete slab that is immersed in hot water.

In the first year of Project 0-4185, specimens were prepared and tested using the HWTD. The results of the tests were analyzed and are included in Research Report 4185-1 (Yildirim and Kennedy 2001). In the second year of this project, samples from the plant mixes and cores from the test sections were taken for each mixture type. The samples were tested using the HWTD in the Texas Department of Transportation (TxDOT) asphalt laboratory. The results of these tests are summarized in Research Report 4185-2 (Yildirim and Kennedy 2002). Research Report 4185-3 mainly includes field performance data collected 1 ½ years after construction (Yildirim, Culfik, Lee, Smit, and Stokoe 2003).

This research report summarizes the visual pavement condition survey and nondestructive test results in the fourth year of this study. Chapter 2 reviews the visual pavement condition survey, Chapter 3 reviews the International Roughness Index measurements, Chapter 4 reviews the field rut depth measurements, Chapter 5 reviews the FWD measurements, Chapter 6 reviews the rolling dynamic deflectometer measurements, and Chapter 7 reviews the portable seismic pavement analyzer measurements.

2. Visual Pavement Condition Survey for 0-4185

This chapter summarizes the visual pavement condition survey results conducted on the eastbound and westbound test sections on IH 20 in the Atlanta District on November 18 and 19, 2003, respectively. The survey was conducted according to the Strategic Highway Research Program (SHRP) Distress Identification Manual for the Long-Term Pavement Performance Studies (SHRP 1990).

2.1 Classification of Distresses According to Strategic Highway Research Program Distress Identification Manual

The manual classifies distresses in pavements into four general modes: cracking, joint deficiencies, surface defects, and miscellaneous distresses. Cracking distresses include corner breaks, longitudinal cracking, and transverse cracking. Joint deficiencies are considered joint seal damage of transverse joints, longitudinal joints, and transverse joints. Surface defects include map cracking and scaling, polished aggregate, and popouts. Finally, miscellaneous distresses include blowups, faulting of transverse joints and cracks, lane-to-shoulder drop-off and separation, patch/patch deterioration, water bleeding, and pumping.

In this survey, observed distress types were described with the associated severity levels. In addition, photographs of distresses that occurred are provided to aid in quantifying their severity levels. The severity levels of transverse cracks are recorded. Detected distresses are mostly transverse cracks, which are the cracks relatively perpendicular to the pavement centerline. Longitudinal cracks, fatigue cracks, potholes, and patching, which were rarely observed, are defined, classified, and measured according to the SHRP distress identification manual as follows.

2.1.1 Transverse Cracking

Transverse cracks are relatively perpendicular to the pavement centerline.

Low: Cracks with low severity or no spalling; mean unsealed as width of ¼” or less. (See Figure 2.1)

Moderate: Cracks with moderate severity spalling; mean unsealed crack width of greater than ¼”; low severity random cracking near the crack. (See Figure 2.2)

High: Cracks with high severity spalling; moderate or high severity random cracking near the crack. (See Figure 2.3)

How to measure: Number and linear feet of transverse cracks at each severity level.

Figure 2.1. Low-level transverse crack

Figure 2.2. Moderate-level transverse crack

Figure 2.3. High-level transverse crack

2.1.2 Fatigue Cracking

Fatigue cracking is a series of interconnected cracks. Fatigue cracks are many-sided, sharp-angled pieces, and are usually less than 1” on the longest side. They occur in a chicken wire/alligator pattern. Fatigue cracks occur only in areas subjected to repeated traffic loadings (usually in wheelpaths). They initially appear as longitudinal cracks.

Low: Longitudinal disconnected hairline cracks running parallel to each other; may be a single crack in wheelpath; crack not spalled.

Moderate: A pattern of articulated pieces formed by cracks that may be lightly spalled; cracks may be sealed.

High: Pieces more severely spalled at edges and loosened until the pieces rock under traffic; pumping may exist.

How to measure: Square feet of surface area at each severity level. If different severity levels existing within an area cannot be distinguished, rate entire area at highest severity present.

2.1.3 Longitudinal Cracking

Longitudinal cracks are relatively parallel to the pavement centerline.

Low: Cracks with low severity or no spalling; mean unsealed crack width of ¼” or less; sealant material in good condition.

Moderate: Cracks with moderately severe spalling; mean unsealed crack width of greater than ¼”; sealant material in bad condition; low severity random cracking near the crack.

High: Cracks with high severity spalling; moderate or high severity random cracking near the crack.

How to measure: Linear feet at each severity level.

2.1.4 Reflection Cracking at Joints

Reflection cracking at joints is cracks in asphalt concrete (AC) overlay surfaces over jointed concrete pavements at original joints. Knowing the slab dimensions beneath the AC surface helps identify these cracks.

Low: Cracks with low severity or no spalling; mean unsealed crack width of ¼" or less; sealant material in good condition.

Moderate: Cracks with moderate severity spalling; mean unsealed crack width of greater than ¼"; sealant material in bad condition; low severity random cracking near the crack.

High: Cracks with high severity spalling; moderate or high severity random cracking near the crack.

How to measure: Number and linear feet of longitudinal and transverse cracks at each severity level. Measurements for longitudinal and transverse cracks shall be recorded separately.

2.1.5 Patching

Patching is a portion of pavement surface that has been removed or replaced.

Low: Patch is in very good condition or has low severity distress of any type.

Moderate: Patch has moderate severity distress of any type.

High: Patch has high severity distress of any type.

How to measure: Square feet of surface area and number of patches at each severity level.

2.1.6 Potholes

Potholes are bowl-shaped holes of various sizes in the pavement surface. Table 2.1 shows severity levels for potholes.

Table 2.1. Severity levels of potholes

	Area (Square Feet)		
Depth (Inches)	<1	1-3	>3
<1	Low	Low	Moderate
1-2	Moderate	Moderate	High
>2	Moderate	High	High

How to measure: Number of potholes at each severity level.

2.2 Westbound Outside Lane

The visual pavement condition survey was conducted on the westbound outside lane on November 18, 2003. A mixture of transverse cracks and patches were detected. Visual condition survey results on the westbound outside lane are given in Tables 2.2 and 2.3. The beginning and the end of the test sections and corresponding mixture and aggregate types are given in Table 2.4. Pictures of each distress are included in Appendix A.

2.3 Eastbound Outside Lane

The visual pavement condition survey was conducted on the eastbound outside lane on November 19, 2003. Distresses detected were mostly transverse cracks. Cracks were at low and moderate levels, so they were considered to be insignificant. Distresses are summarized in Tables 2.6 and 2.7. The beginning and the end of the test sections and corresponding mixture and aggregate types are given in Table 2.5. Pictures of every distress observed are available in Appendix A.

2.4 Comparison of Changes in the Number of Cracks for Different Test Sections

Table 2.8 shows a summary of cracks for different test sections in November 2003, and Table 2.9 shows the changes in the number of transverse cracks for different test sections between December 2001, January 2002, November 2002 and November 2003.

The aggregate type that was used in different sections is expected to affect the pavement performance. The aggregate types that were used in different sections are as follows:

- Sections 2, 5, and 8 – sandstone
- Sections 3, 6, 9 – quartzite
- Sections 1, 4, 7 - gravel

The initial condition of the continuously reinforced concrete pavement (CRCP) can affect the formation of distresses on asphalt pavement. Table 2.10 shows the existing number of cracks that include both transverse cracks and patchings on the CRCP before the asphalt pavement was placed on it. The existing transverse cracks and the edges of the patchings on the CRCP are expected to affect the crack formation in asphalt pavement. Table 2.8 shows us that the maximum number of distresses occurred in Sections 2, 6, 7, and 8.

Table 2.2. Visual pavement condition survey results on westbound outside lane

Station Numbers	Distresses	Dimension (feet)	Photo #
1321-1320	1 Transverse Crack, Low	3 ft	WBP1TC
1319-1318	1 Patch Deterioration	8x12 sq ft	WBP2PD
1318-1317	1 Patch Deterioration	8x12 sq ft	WBP3PD
1313-1312	1 Patch Deterioration	8x12 sq ft	WBP4PD
1309-1308	1 Transverse Crack, Moderate	12 ft	WBP5TC
1308-1307	1 Patch Deterioration	1x2 sq ft	WBP6PD
1308-1307	1 Patch Deterioration	1x2 sq ft	WBP6PD
1308-1307	1 Patch Deterioration	18x12 sq ft	WBP7PD
1306-1305	1 Transverse Crack, Low	6 ft	WBP8TC
1306-1305	1 Patch Deterioration	9x12 sq ft	WBP9PD
1305-1304	1 Transverse Crack, Low	6 ft	WBP10TC
1305-1304	1 Transverse Crack, Low	12 ft	WBP10TC
1303-1302	1 Transverse Crack, Low	4 ft	WBP11TC
1302-1301	1 Transverse Crack, Low	3 ft	WBP12TC
1301-1300	1 Transverse Crack, Low	4 ft	WBP13TC
1301-1300	1 Patch Deterioration	6x12 sq ft	WBP13TC
1301-1300	1 Patch Deterioration	12x12 sq ft	WBP14PD
1300-1299	1 Transverse Crack, Low	3 ft	WBP15TC
1300-1299	1 Transverse Crack, Moderate	12 ft	WBP16TC
1298-1297	1 Transverse Crack, Low	6 ft	WBP17TC
1297	1 Patch Deterioration	21x12 sq ft	WBP18PD
1297-1296	1 Transverse Crack, Moderate	3 ft	WBP19TC
1293-1292	1 Patch Deterioration	7x12 sq ft	WBP20PD
1292-1291	1 Transverse Crack, Low	6 ft	WBP21TC
1291-1290	1 Transverse Crack, Low	5 ft	WBP22TC
1290-1289	1 Patch Deterioration	7x12 sq ft	WBP23PD
1287-1286	1 Transverse Crack, Low	6 ft	WBP24TC
1287-1286	1 Transverse Crack, Low	6 ft	WBP24TC
1285-1284	1 Patch Deterioration	6x12 sq ft	WBP25PD
1253-1252	1 Patch Deterioration	15x12 sq ft	WBP26PD
1250	1 Patch Deterioration	11x12 sq ft	WBP28PD
1250-1249	1 Patch Deterioration	6x12 sq ft	WBP29PD
1249-1248	1 Transverse Crack, Moderate	4 ft	WBP30TC
1240-1239	1 Transverse Crack, Low	12 ft	WBP31TC

Table 2.3. Visual pavement condition survey results on westbound outside lane

Station Numbers	Distresses	Dimension (feet)	Photo #
1236-1235	1 Transverse Crack, High	4 ft	WBP32TC
1235-1234	1 Patch Deterioration	6x12 sq ft	WBP33PD
1229-1228	1 Patch Deterioration	18x12 sq ft	WBP34PD
1227-1226	1 Transverse Crack, Moderate	12 ft	WBP35TC
1224-1223	1 Patch Deterioration	6x12 sq ft	WBP36PD
1223-1222	1 Transverse Crack, Moderate	12 ft	WBP37TC
1221-1220	1 Transverse Crack, Low	3 ft	WBP38TC
1221-1220	1 Transverse Crack, Moderate	12 ft	WBP39TC
1221-1220	1 Transverse Crack, Moderate	12 ft	WBP39TC
1218-1217	1 Transverse Crack, Low	3 ft	WBP40TC
1215+62	1 Patch Deterioration	9x12 sq ft	WBP41PD
1213+69	1 Patch Deterioration	18x12 sq ft	WBP42PD
1213-1212	1 Transverse Crack, Moderate	12 ft	WBP43TC
1213-1212	1 Transverse Crack, Moderate	12 ft	WBP43TC
1211+45	1 Patch Deterioration	9x12 sq ft	WBP44PD
1206+54	1 Patch Deterioration	6x12 sq ft	WBP45PD
1205-1204	1 Transverse Crack, Low	3 ft	WBP46TC
1204-1203	1 Transverse Crack, Low	12 ft	WBP47TC
1203-1202	1 Transverse Crack, Moderate	12 ft	WBP48TC
1203-1202	1 Transverse Crack, Moderate	12 ft	WBP48TC
1201-1200	1 Transverse Crack, Moderate	12 ft	WBP49TC
1998-1997	1 Transverse Crack, Moderate	12 ft	WBP50TC
1197-1996	1 Transverse Crack, Moderate	4 ft	WBP51TC
1996-1995	1 Patch Deterioration	4x12 sq ft	WBP52PD
1194	1 Transverse Crack, Moderate	5 ft	WBP53TC
1194	1 Transverse Crack, High	5 ft	WBP54TC
1182-1181	1 Patch Deterioration	6x12 sq ft	WBP55PD
1151-1150	3 small potholes		WBP56PH

Table 2.4. Beginning and end of the test sections on westbound outside lane

Section	Section Name	Station Numbers	Mixture Type	Aggregate
W1	2	1278 – 1321	Superpave	Sandstone
W2	5	1235 – 1278	CMHB–C	Sandstone
W3	8	1193 – 1235	Type C	Sandstone
W4	3	1135 – 1188	Superpave	Quartzite

Table 2.5. Beginning and end of the test sections on eastbound outside lane

Section	Section Name	Station Numbers	Mixture Type	Aggregate
E1	6	1135 – 1185	CMHB–C	Quartzite
E2	9	1190 – 1218	Type C	Quartzite
E3	1	1218 – 1245	Superpave	Gravel
E4	4	1245 - 1282	CMHB–C	Gravel
E5	7	1282 - 1321	Type C	Gravel

Table 2.6. Visual pavement condition survey results on eastbound outside lane

Station Numbers	Distresses	Dimension (feet)	Photo #
1135	1 Transverse Crack, Low	3 ft	EBP1TC
1135	1 Transverse Crack, Moderate	3 ft	EBP2TC
1135	1 Transverse Crack, Moderate	3 ft	EBP3TC
1135	1 Transverse Crack, Low	4 ft	EBP4TC
1135	1 Transverse Crack, Moderate	4 ft	EBP5TC
1135	1 Transverse Crack, Low	2 ft	EBP5TC
1136	1 Transverse Crack, Low	4 ft	EBP5TC
137-1138	1 Transverse Crack, Low	4 ft	EBP6TC
1138	1 Transverse Crack, Low	3 ft	EBP7TC
1138-1139	1 Transverse Crack, Low	3 ft	EBP8TC
1138-1139	1 Transverse Crack, Low	3 ft	EBP9TC
1139-1140	1 Transverse Crack, Low	4 ft	EBP10TC
1140	1 Transverse Crack, Low	4 ft	EBP11TC
1140-1141	1 Transverse Crack, Low	3 ft	EBP12TC
1148-1149	1 Transverse Crack, Moderate	12 ft	EBP13TC
1171-1172	1 Transverse Crack, Low	12 ft	EBP14TC
1190-1191	1 Transverse Crack, High	12 ft	EBP15TC
1203	1 Transverse Crack, Low	12 ft	EBP16TC
1209-12110	1 Transverse Crack, Low	12 ft	EBP17TC
1212-1213	1 Transverse Crack, Low	4 ft	EBP18TC
1214-1215	1 Transverse Crack, Low	4 ft	EBP19TC
1218-1219	1 Transverse Crack, Low	1 ft	EBP20TC
1218-1219	1 Transverse Crack, Low	4 ft	EBP20TC
1220-1221	1 Transverse Crack, Low	12 ft	EBP21TC
1221-1222	1 Transverse Crack, High	6 ft	EBP22TC
1223-1224	1 Transverse Crack, Low	3 ft	EBP23TC
1223-1224	1 Patch Deterioration	7x12 sq ft	EBP24PD
1225-1226	1 Patch Deterioration	12x12 sq ft	EBP25PD
1228	1 Transverse Crack, Low	4 ft	EBP26TC
1230-1231	1 Transverse Crack, Low	5 ft	EBP27TC
1230-1231	1 Transverse Crack, Low	2 ft	EBP27TC
1249-1250	1 Transverse Crack, Low	4 ft	EBP28TC
1249-1250	1 Transverse Crack, Moderate	4 ft	EBP29TC
1250-1251	1 Transverse Crack, Low	4 ft	EBP30TC
1258-1259	1 Transverse Crack, Low	12 ft	EBP31TC

Table 2.7. Visual pavement condition survey results on eastbound outside lane

Station Numbers	Distresses	Dimension (feet)	Photo #
1259-1260	1 Patch Deterioration	7x12 sq ft	EBP32PD
1273-1274	1 Transverse Crack, Low	8 ft	EBP33TC
1274-1275	1 Transverse Crack, Low	4 ft	EBP34TC
1274-1275	1 Transverse Crack, Low	4ft	EBP35TC
1277-1278	1 Transverse Crack, Low	4 ft	EBP38TC
1285-1286	1 Transverse Crack, Low	4 ft	EBP39TC
1286-1287	1 Transverse Crack, Low	3 ft	EBP40TC
1287-1288	1 Transverse Crack, Low	6 ft	EBP41TC
1287-1288	1 Transverse Crack, Low	2 ft	EBP41TC
1288-1289	1 Patch Deterioration	9x12 sq ft	EBP42PD
1289-1290	1 Transverse Crack, Low	4 ft	EBP43TC
1289-1290	1 Transverse Crack, Low	4 ft	EBP44TC
1290	1 Transverse Crack, Low	4 ft	EBP45TC
1290-1291	1 Transverse Crack, Moderate	6 ft	EBP46TC
1290-1291	1 Transverse Crack, Moderate	6 ft	EBP47TC
1291-1292	1 Transverse Crack, Low	6 ft	EBP48TC
1291-1292	1 Transverse Crack, Low	4 ft	EBP49TC
1292-1293	1 Patch Deterioration	9x12 sq ft	EBP50PD
1293-1294	1 Transverse Crack, Low	4 ft	EBP51TC
1295-1296	1 Transverse Crack, Low	3 ft	EBP52TC
1296-1297	1 Transverse Crack, Low	12 ft	EBP53TC
1296-1297	1 Transverse Crack, High	3 ft	EBP54TC
1300-1301	1 Transverse Crack, Low	6 ft	EBP55TC
1303-1304	1 Transverse Crack, Moderate	6 ft	EBP56TC
1303-1304	1 Transverse Crack, High	12 ft	EBP57TC
1306-1307	1 Transverse Crack, Low	8 ft	EBP58TC
1307-1308	1 Transverse Crack, Low	6 ft	EBP59TC
1309-1310	1 Transverse Crack, Low	12 ft	EBP60TC
1310-1311	1 Transverse Crack, Low	12 ft	EBP61TC
1316-1317	1 Transverse Crack, Low	4 ft	EBP62TC
1317-1318	1 Transverse Crack, High	8 ft	EBP63TC
1317-1318	1 Transverse Crack, High	6 ft	EBP64TC
1318-1319	1 Transverse Crack, Moderate	6 ft	EBP65TC
1320-1321	1 Transverse Crack, Low	6 ft	EBP66TC

Table 2.8. Summary of cracks for different test sections in November 2003

Section	Total	Transverse Cracks Low	Transverse Cracks Moderate	Transverse Cracks High	Patch Deterioration
2	18	15	3	0	13
5	3	1	1	1	3
8	17	4	12	1	8
3	0	0	0	0	1
6	16	12	4	0	0
9	5	4	0	1	0
1	8	7	0	1	2
4	8	7	1	0	1
7	27	19	3	5	2

Table 2.9. Number of transverse cracks for different test sections for December 2001, January 2002, November 2002, and November 2003

Sec.	Number of Transverse Cracks in December 2001	Number of Transverse Cracks in January 2002				Number of Transverse Cracks in November 2002				Number of Transverse Cracks in November 2003			
	Total	Low	Mod.	High	Total	Low	Mod.	High	Total	Low	Mod.	High	Total
2	0	2	2	0	4	1	5	2	8	15	3	0	18
5	0	2	1	0	3	1	1	0	2	1	1	1	3
8	0	5	0	0	5	3	4	2	9	4	12	1	17
3	0	0	0	0	0	0	0	0	0	0	0	0	0
6	0	0	0	0	0	2	0	0	2	12	4	0	16
9	0	0	0	0	0	1	0	0	1	4	0	1	5
1	0	0	0	0	0	1	2	0	3	7	0	1	8
4	0	0	0	0	0	2	1	0	3	7	1	0	8
7	0	1	0	0	1	3	2	0	5	19	3	5	27

Table 2.10. Existing number of cracks on CRCP before the construction of the overlays

Section	Low Transverse Crack	Moderate Transverse Crack	Patching	Total Number of Cracks
2	30	33	28	119
5	12	66	27	132
8	15	115	39	208
3	8	15	10	43
6	190	0	29	248
9	219	0	37	293
1	129	0	31	191
4	141	6	39	225
7	89	1	30	150

3. International Roughness Index Measurements

A second pavement condition survey was conducted on the outside lanes of eastbound and westbound test sections on IH 20 in the Atlanta District on November 14, 2003. There are four test sections in the westbound lane and five test sections in the eastbound lane. Each test section has a different mixture design or aggregate type. Three different mix designs (CMBH-C, Type C, and Superpave) and three different aggregates (quartzite, gravel, and sandstone) were combined, resulting in a total factorial of nine tests. The location of the test sections is given in Figure C.1 in Appendix C. The section names and properties for the eastbound and westbound lanes are given in Tables C.1 and C.2 in Appendix C.

The International Roughness Index (IRI) is a widely used profile index where the analysis method is intended to work with different types of profilers. It is defined as a property of the true profile, and therefore it can be measured with any valid profiler. The analysis equations were developed and tested to minimize the effects of some profiler measurement parameters such as sample interval. Example computer programs were published and have been used by profiler developers and others to test new software that computes IRI (UMTRI, 2004).

Both on eastbound and westbound lanes the IRI(Left) and IRI(Right) values were estimated separately. The data is collected only for the outside lanes. IRI-Nov2003 (IRI values obtained in November 2003) and IRI-Finished (IRI values obtained just after the asphalt concrete pavement was constructed, in December 2001) values are given in Appendix B, through Tables B.1 and B.6.

The objective of this study is to present the IRI-Finished and IRI-Nov2003 values and to perform a statistical test for each section. The test shows on which sections IRI values changed significantly from December 2001 to November 2003. In this study three sets of IRI values are presented and compared from those collected both during Nov-2003 and Dec-2001: IRI(Left), IRI values collected from the left wheelpaths, IRI(Right), IRI values collected from right wheelpaths, and IRI(Average), average of IRI(left) and IRI(right) values. Each dataset is analyzed separately.

3.1 Statistical Analysis of Data

In order to determine whether or not the IRI values changed significantly between December 2001 and November 2003, a t-test for each section was conducted. Because IRI-Finished and IRI-Nov2003 values are estimated at the same locations, the estimates are dependent; therefore it is appropriate to use a paired t-test.

d = (IRI-Finished) – (IRI-Nov2003)

From d values, t-statistics values were calculated, where

t-statistics = d(ave)/ (S_D(d)/√n)

d(ave) = mean of d values in each section
n= number of IRI values in each section (sample size)
S_D= sample standard deviation of d
Dt: degree of freedom=n-1

Then t-statistics values are compared with t_α values, which are found from t-test tables. Because we chose a 95 percent significance level,

t_α is found where α=0.05

Tests of hypothesis were measured out according to the following:

Null Hypothesis: For a given section IRI-Finished = IRI-Nov2003

Alternate Hypothesis: For a given section IRI-Finished > IRI-Nov2003

Criteria: Reject null hypothesis and accept alternate hypothesis if t-statistics > t_α

The t-test was used to determine whether or not the IRI-Finished and IRI-Nov2003 values were changed with a significance level of 5 percent. The value 0.05 represents the 5 percent error area under the t distribution curve. In the t-test, a one tail method was used in order to establish if the IRI-Nov2003 values are not smaller than the IRI-Finished values. For each test section, the t-statistics value was compared with t_α value. If the t-statistics value is smaller than t_α t-test confirms, the IRI-Finished and IRI-Nov2003 values are not different with a significance of 95 percent.

Another way of comparing the IRI-Finished and IRI-Nov2003 values with the t-test is to calculate the p-value for each test section. Because in the t-test the significance level is 5 percent, if the p-value is greater than 0.05, it can be said that IRI-Finished and IRI-Nov2003 values are not different at a 5 percent level.

3.1.1 Results for International Roughness Index (Right) Data:

When we compare the IRI(Right) values measured just after the construction and the ones measured in November 2003, the values seem to differ significantly from each other for some of the eastbound outside lane sections. The averages of the IRI(Right) values and their standard deviations for each section are shown in Table 3.1. In addition to the IRI(Right) values, the mean of the differences between them, d(ave), and their standard deviations are also given in Table 3.1.

The t-statistics, $t_{\alpha,}$ and p-value are shown in Table 3.2 for each test section. As we see, for all sections on the westbound outside lane p-values are higher than 0.05. However, for Section 9 and Section 4 on the eastbound lane, p-values are less than 0.05. This shows that for Sections 9 and 4, IRI(Right) values significantly decreased (at a 5% significance level) from the date of the asphalt concrete pavement placement to November 2003, which is approximately two years. Table 3.1 also shows that the mean of IRI(Right)-Nov 2003 values for these two sections is significantly lower than the mean of IRI(Right)-Finished values in comparison with the other test sections.

Table 3.1. IRI(Right) values of the test sections

	Section	IRI(Right)-FINISHED, Average	IRI(Right)-FINISHED, STDEV	IRI(Right)-NOV.03, Average	IRI(Right)-NOV.03, STDEV	d(average)	SDEV(d)
westbound outside lane	2	73.159	11.608	72.748	17.972	0.411	16.807
	5	68.129	8.435	66.756	12.498	1.373	8.892
	8	64.187	14.224	68.778	17.473	-4.591	16.955
	3	52.892	6.488	48.206	10.268	4.686	11.206
eastbound outside lane	6	65.183	14.205	61.091	27.941	4.091	15.346
	9	62.903	15.255	55.863	14.465	7.040	4.171
	1	58.387	7.461	59.190	8.829	-0.803	6.988
	4	54.933	5.726	47.593	10.959	7.340	9.918
	7	67.160	12.038	67.577	22.411	-0.417	17.992

Table 3.2. t_α, t-statistics and p-values for each test sections for IRI(Right)

	Section	d (average)	SDEV(d)	tα	t-statistics	p-Value
westbound outside lane	2	0.411	16.807	1.895	0.069	0.473
	5	1.373	8.892	1.895	0.437	0.338
	8	-4.591	16.955	1.860	-0.812	0.780
	3	4.686	11.206	1.833	1.322	0.109
eastbound outside lane	6	4.091	15.346	1.943	0.705	0.254
	9	7.040	4.171	1.943	4.466	0.002
	1	-0.803	6.988	1.943	-0.304	0.614
	4	7.340	9.918	1.943	1.958	0.049
	7	-0.417	17.992	1.943	-0.061	0.523

3.1.2 Results for International Roughness Index (Left) Data:

When compared, the IRI(Left) values measured just after the construction and the ones measured on November 2003 seem to be very close for all sections. The averages of the IRI(Left) values and their standard deviations for each section are shown in Table 3.3. In addition to the IRI(Left) values, the mean of the differences between them, d(ave), and their standard deviations are also given in Table 3.3.

Without a statistical test, the existence of a decreasing trend in IRI(Left) values over time is not obvious because the values are very close . In some cases there are even some increases in the IRI(Left)-Nov2003 values in comparison with the IRI(Left)-Finished values, which are not expected and may stem from some measurement errors.

The t-statistics, $t_{\alpha,}$ and p-value are shown in Table 3.4 for each test section. As we see from these figures, p-values for all sections are higher than 0.05. This shows that IRI(Left) values did not decrease significantly (at a 5 percent significance level) from the date of the asphalt concrete pavement placement to November 2003, which is approximately two years.

Table 3.3. IRI(Left) values of the test sections

	Section	IRI(Left)-FINISHED, Average	IRI(Left)-FINISHED, STDEV	IRI(Left)-NOV.03, Average	IRI(Left)-NOV.03, STDEV	d(average)	SDEV(d)
westbound outside lane	2	57.769	8.320	64.816	13.143	-7.048	10.243
	5	60.581	5.064	56.131	7.326	4.450	9.491
	8	52.249	11.673	59.170	9.791	-6.921	10.785
	3	53.461	3.755	51.143	6.365	2.318	5.294
eastbound outside lane	6	57.561	8.961	53.651	15.518	3.910	12.144
	9	61.474	14.273	59.964	11.139	1.510	11.369
	1	55.946	10.066	55.514	10.450	0.431	7.019
	4	50.867	7.926	44.577	8.275	6.290	9.509
	7	55.349	10.784	54.301	14.391	1.047	7.880

Table 3.4. $t_{\alpha,}$, t-statistics, and p-values for each test sections for IRI(Left)

	Section	d (average)	SDEV(d)	tα	t-statistics	p-Value
west bound outside lane	2	-7.048	10.243	1.895	-1.946	0.954
	5	4.450	9.491	1.895	1.326	0.113
	8	-6.921	10.785	1.860	-1.925	0.955
	3	2.318	5.294	1.833	1.385	0.100
east bound outside lane	6	3.910	12.144	1.943	0.852	0.213
	9	1.510	11.369	1.943	0.351	0.369
	1	0.431	7.019	1.943	0.163	0.438
	4	6.290	9.509	1.943	1.750	0.065
	7	1.047	7.880	1.943	0.352	0.369

3.1.3 Results for International Roughness Index (Average) Data:

IRI(Average) values are calculated by taking the average of IRI(Left) and IRI(Right) values. The averages of the IRI(Average) values and their standard deviations for each section are shown in Table 3.5. In addition to the IRI(Average) values, the mean of the differences between them, d(ave), and their standard deviations are also given in Table 3.5.

IRI(Average) values are very similar to the IRI(Right) values. The t-statistics, t_α and p-value are shown in Table 3.6 for each test section. As we see from these figures, as in the IRI(Right) case for Section 4, the p-value is less than 0.05. Therefore, for this section, IRI(Average) values are significantly decreased (at a 5 percent significance level) from the date of the asphalt concrete pavement placement to November 2003. It can also be seen in Table 3.5 that the mean of IRI(Average)-Nov 2003 values is significantly lower than the mean of IRI(Average)-Finished values for Section 4 in comparison with the other sections.

Table 3.5. IRI(Average) values of the test sections

	Section	IRI FINISHED, Average	IRI FINISHED, SDEV	IRI NOV.02, Average	IRI NOV.02, SDEV	d (average)	SDEV(d)
westbound outside lane	2	65.464	9.043	68.782	14.879	-3.318	12.538
	5	64.355	5.705	61.444	9.549	2.911	8.239
	8	58.218	12.461	63.974	13.107	-5.756	12.834
	3	53.177	4.416	49.675	8.043	3.502	7.774
eastbound outside lane	6	61.372	10.399	57.371	21.514	4.001	13.576
	9	62.189	14.567	57.914	12.413	4.275	7.655
	1	57.166	8.447	57.352	9.544	-0.186	6.706
	4	52.900	4.997	46.085	9.192	6.815	8.593
	7	61.254	10.490	60.939	17.505	0.315	12.113

Table 3.6. t_α, t-statistics, and p-values for each test section for IRI(Average)

	Section	d (average)	SDEV(d)	tα	t-statistics	p-Value
westbound outside lane	2	-3.318	12.538	1.895	-0.749	0.761
	5	2.911	8.239	1.895	0.999	0.175
	8	-5.756	12.834	1.860	-1.345	0.892
	3	3.502	7.774	1.833	1.425	0.094
eastbound outside lane	6	4.001	13.576	1.943	0.780	0.233
	9	4.275	7.655	1.943	1.478	0.095
	1	-0.186	6.706	1.943	-0.073	0.528
	4	6.815	8.593	1.943	2.098	0.040
	7	0.315	12.113	1.943	0.069	0.474

4. Field Rut Depth Measurements

4.1 Field Rutting Data

Rutting data was collected using the dipstick profilometer from each test section on November 18 and 19, 2003, approximately two and a half years after the completion of construction. This data was collected along the profile of the roads in order to get an estimate of the in-place rutting of the asphalt pavement. The data was collected on one lane length in each measurement. For each profile, two rut depths were found that correspond to the inside and the outside wheelpaths. For the outside lanes, the right rut depth corresponds to the outside wheelpath and the left rut depth corresponds to the inside wheelpath.

The final depth of the rutting was found using American Association of State Highway and Transportation Officials (AASHTO) Designation PP38-00, and the equation to find the perpendicular distance from a point to a line made by two points was used to calculate the rut depth. Using AASHTO Designation PP38-00, focus is on five points (A, B, C, D, and E) in analyzing the profiler data. Two points, A and C, that create a line were chosen as the two highest points across the first half of the data for the outside wheelpath and the two highest points on the second half of the data, C and E, were chosen for the inside wheelpath. Points B and D were the deepest points across A and C, and C and E, respectively across the profile, and thus provided the depth of the rut for the outside and inside wheelpaths. An example of how the rutting depths were found is given in Figure 4.1.

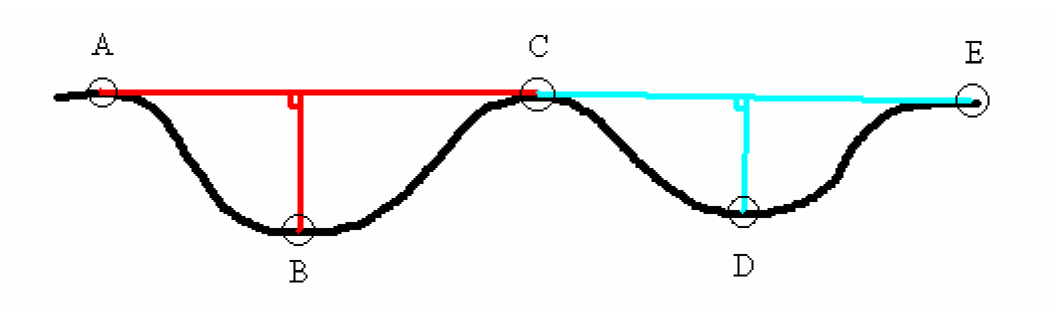

Figure 4.1. Rut depth profile

Table 4.1 shows the right and left rutting value for each section. The average of right and left rut depths for each section that was found are shown in Figure 4.2. As can be seen from Figure 4.2, overall rutting observed in the tests sections is very low. The highest rutting data was observed from the mixes produced by gravel.

Table 4.1. Average right and left rutting values for each section

	Sections	Right	Left	Average
6	**CMHB Quartize**	1.19	0.76	0.97
9	**Type C Quartize**	1.67	0.80	1.23
1	**Superpave Gravel**	1.62	1.19	1.40
4	**CMHB Gravel**	2.07	1.41	1.74
7	**Type C Gravel**	1.84	0.95	1.40
2	**Superpave Sandstone**	1.60	1.16	1.38
5	**CMHB Sandstone**	1.44	0.80	1.12
8	**Type C Sandstone**	0.95	0.95	0.95
3	**Superpave Quartize**	1.05	0.83	0.94

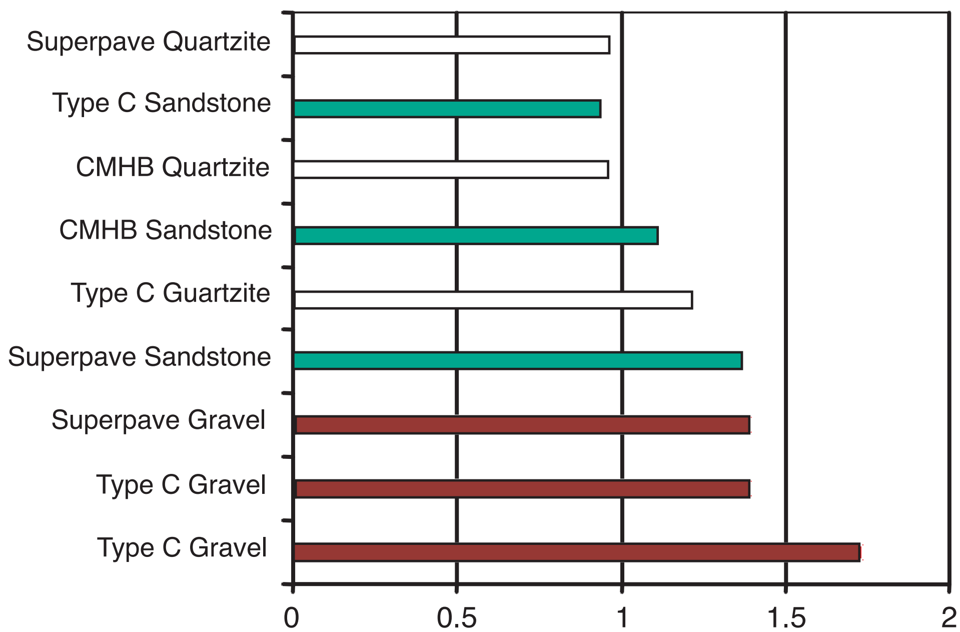

Figure 4.2. Average rutting approximately 2 ½ years after construction (units in mm)

5. Falling Weight Deflectometer Measurements

5.1 Introduction

This chapter reports the results of falling weight deflectometer (FWD) tests done on the outside lanes of the various sections evaluated on IH 20 in Harrison County. The reader is referred to Appendix C for orientation of the different sections evaluated. Appendix C also outlines the different mixes used on these sections.

FWD testing typically is used to evaluate the structural performance of pavement. This point is emphasized, given that the total thickness of asphalt surfacing overlaid on the continually reinforced concrete pavement (CRCP) in question was about 100 mm (4 inches). Thin asphalt layers (less than 5 inches in thickness) overlaid on concrete pavements do not contribute significantly to the structural capacity of these pavements. The benefit of an asphalt concrete overlay is that it improves the riding quality of the pavement. It provides smoother pavement that attenuates the effects of dynamic wheel loading under heavy traffic. This may extend the structural life of the pavement, a benefit not necessarily associated with the actual performance of the asphalt concrete mixture in terms of rutting and/or fatigue.

Given the above, FWD analyses were done in order to identify possible trends indicating performance contributions or respective benefits associated with the different mixes placed on the various sections of IH 20. This chapter addresses the analyses toward this objective.

5.1.1 Falling Weight Deflectometer Testing Completed

The results of five separate instances of FWD testing are reported. The first of these occurred toward the end of March and early April 2001. These FWD tests were done on top of a 4-inch asphalt overlay (placed over an 8-inch CRCP), which was subsequently removed by milling. After milling of the old overlay, a second round of FWD testing was done directly on top of the milled concrete pavement toward the end of August 2001. The milled concrete pavement was overlaid with a 2-inch Type B asphalt mix, which served as a base layer for the various mixes evaluated as part of the study, placed in 2-inch lifts on top thereof. After construction of the various mixes, a third round of FWD testing was done on each of the newly constructed sections during January 2002. The fourth round of FWD testing was done during November 2002. The fifth round of tests was conducted in November 2003. It should be noted that between the fourth and fifth rounds, parts of all sections were patched. Thus, some measurements were taken from patched pavement, which may affect statistical analyses, particularly in Sections 2 and 8. Table 5.1 summarizes the FWD testing done on IH 20 as reported.

Table 5.1. Summary of FWD testing

FWD Series	Date Tested	Pavement Structure
1	April 2001	Old overlay
2	August 2001	Concrete
3	January 2002	New overlays
4	November 2002	New overlays
5	November 2003	New overlays

Because the different FWD series were performed on the same locations, one is able to track the deflection response of the pavement structure and specific sections during the different stages of rehabilitation. An obvious question is how the deflections on the new overlay compare to those on the old and to what extent the asphalt overlays are influencing FWD deflections.

5.2 Falling Weight Deflectometer Testing

5.2.3 Normalization of Falling Weight Deflectometer Deflections

FWD deflections resulting from load drops in the vicinity of 9,000 lb were converted directly to standard deflections at 9,000 lb. In order to compare the FWD deflections of tests done at different times of the day and year, it was deemed necessary to apply a temperature correction. Air temperature measurements were consistently collected at each FWD drop. Figures 5.1 and 5.2 show the means and standard deviations of these air temperatures for the different sections. Temperatures ranged from 45 °F to 86 °F, the highest standard deviations apparent during the November 2002 FWD testing.

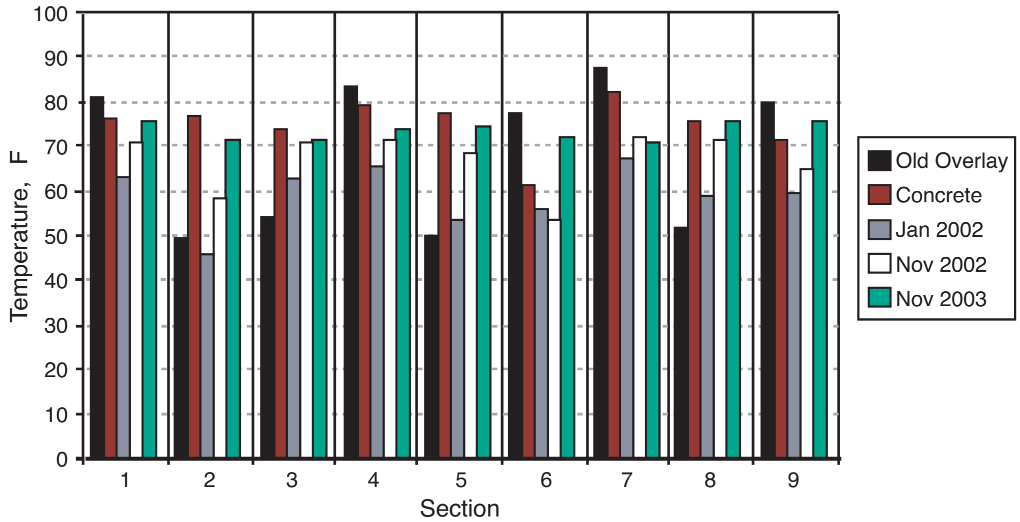

Figure 5.1. Mean air temperatures during FWD testing

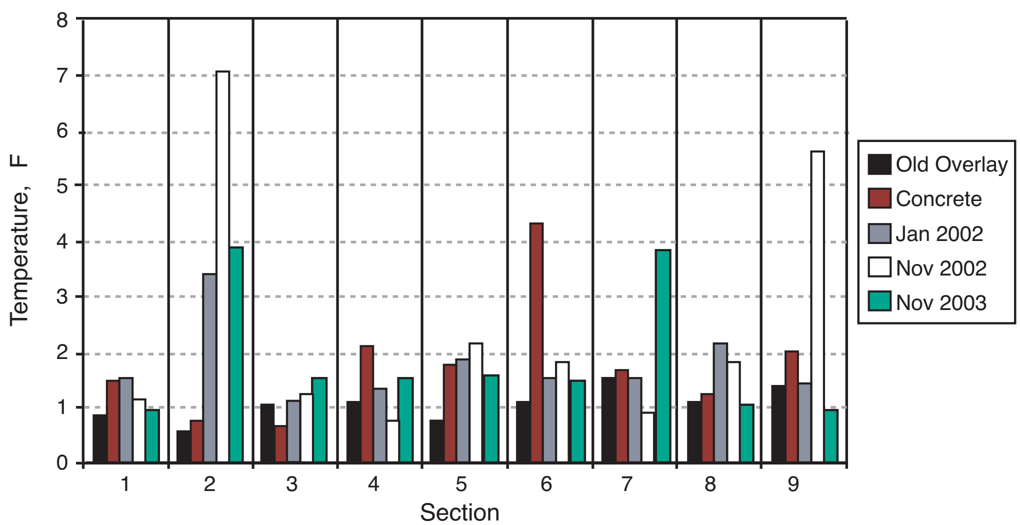

Figure 5.2. Standard deviation of air temperatures during FWD testing

Using these temperatures, the deflections measured on the asphalt sections (only) were normalized to those at a standard temperature of 20 °C (68 °F), using a correction factor based on that developed at Delft (Molenaar 1997):

$$TNF = 1 + \left(a_1 + \frac{a_2}{h_1}\right)(T_A - 20) + \left(a_3 + \frac{a_4}{h_1}\right)(T_A - 20)^2$$

where:

TNF	=	Temperature normalization factor
T_A	=	Air temperature (°C)
h_1	=	Thickness of the asphalt layer = 100 mm

TNF takes on values smaller than one if the measurements are taken below the reference temperature of 20 °C and larger than one if the measurements were taken above 20 °C. For FWD base plates having a diameter of 300 mm, the constants a_1 to a_4 in the above equation take on the following values:

$a_1 = 0.05398$ °C^{-1} $a_2 = -2.6113$ mm/°C
$a_3 = 0.00128439$ °C^{-1} $a_4 = -0.07493$ mm/°C

The deflection measured at a specific temperature is normalized to that at 20 °C by dividing it by *TNF*.

5.3 Falling Weight Deflectometer Deflection Results

FWD tests were done on the outside eastbound and westbound lanes of IH 20. The collected data were divided into subsets representing the various sections tested indicating the normalized deflection parameters determined for each separate section before removal of deflection outliers.

From the data, it can be seen that the deflections along the individual sections are fairly uniform but are characterized by sporadic jumps and irregularities indicating regions where repairs had been conducted or regions of potential structural weakness. These may be due to localized cracking within the structure and are not necessarily indicative of the integrity of the section as a whole. In general, the very high W1 deflections apparent at irregular intervals along the sections on the old overlay and concrete pavement appear to have corresponding lower W1 deflections on the new overlay indicating that the overlay was influential in decreasing the deflections on the pavement.

5.3.1 Outliers

Given that one of the objectives of the study is to identify the relative performance of the specific mixes used on the different sections, it was decided to identify and eliminate deflection outliers using a statistical approach to prevent these from overly influencing the mean and standard deviation of the deflection parameters apparent on a particular section. This was done by standardizing the deflection data and defining outliers as data points greater or less than three times the standard deviation of the sample population for a particular section. This slightly decreased the number of records used to determine statistical means and standard deviations for the deflections on a particular section, as shown in Table 5.2.

Table 5.2 indicates the number of FWD deflection records collected on each of the sections for the different series of FWD tests completed. The number of outliers identified on a particular section provides an indication of its uniformity, i.e., the greater the number of outliers, the greater the number of abnormalities apparent.

Table 5.2. Number of FWD deflection records after (and before) eliminating outliers

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov-2003
1	23 (24)	24 (26)	24 (24)	22 (24)	22 (24)
2	41 (44)	37 (40)	38 (40)	38 (40)	44 (46)
3	54 (56)	47 (49)	49 (50)	43 (46)	45 (46)
4	35 (37)	36 (40)	36 (37)	35 (37)	34 (36)
5	39 (41)	42 (44)	44 (45)	43 (44)	40 (42)
6	40 (42)	46 (50)	42 (44)	38 (40)	42 (43)
7	38 (40)	37 (39)	37 (39)	33 (37)	37 (40)
8	41 (42)	38 (41)	39 (41)	39 (41)	43 (45)
9	27 (29)	27 (29)	28 (29)	26 (28)	25 (26)

Figure 5.3 illustrates and ranks the number of outliers apparent on each of the nine sections evaluated for the different FWD series. From this figure it is clear that the greatest number of irregular deflections were apparent from the FWD tests on the concrete pavement after milling the old overlay. It is interesting to note that there was a marked decrease in the number of irregularities after the construction of the new overlay (January 2002), but that the number of irregularities started to be apparent again after November 2002. Note that there were no outliers identified for Section 1 in January 2002. Figure 5.4 compares the results in each section for January 2002, November 2002, and November 2003.

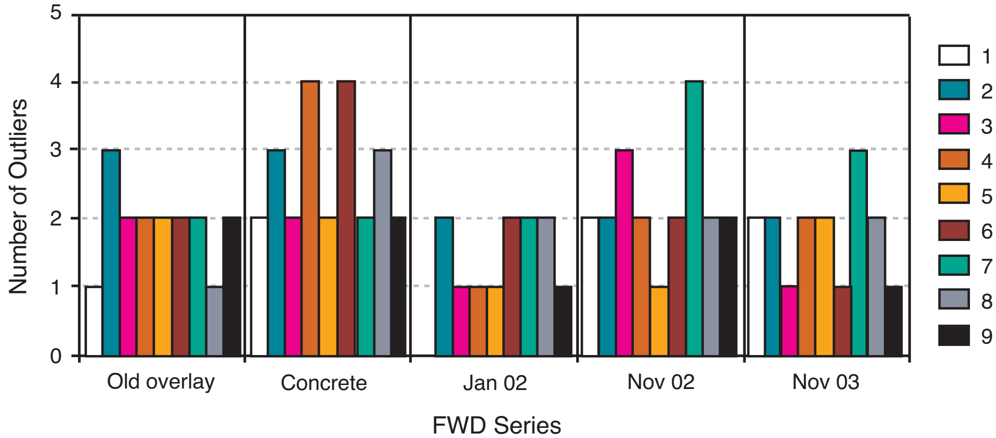

Figure 5.3. Number of outliers identified on the nine sections

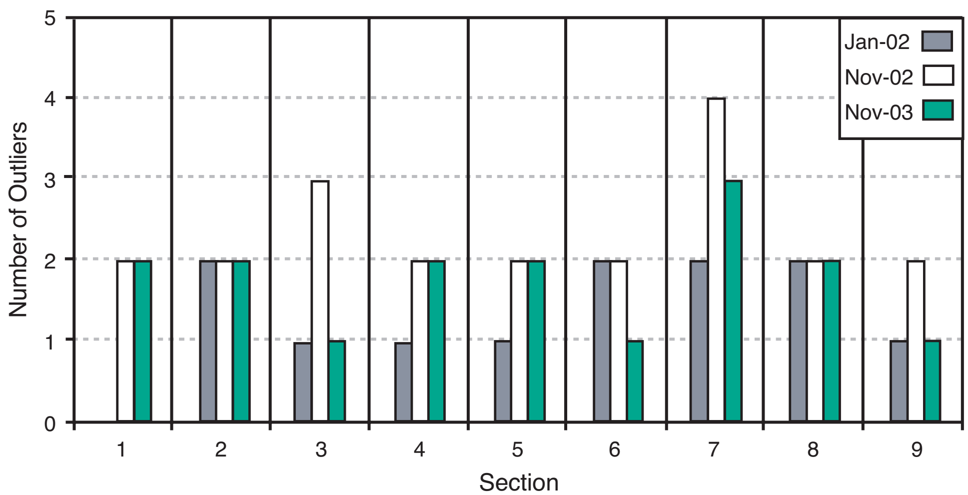

Figure 5.4. Number of outliers identified on the nine sections between November 2002 and November 2003

5.3.2 Summary Means of Falling Weight Deflectometer Deflection Parameters

Tables 5.3 through 5.6 indicate the mean FWD deflection parameters (W1, W7, SCI, and BCI, respectively) determined for each of the sections during each FWD testing series. The mean defection parameters for each of the sections (roadway means) are also given. These means are used later in the chapter to investigate whether the deflection on a specific section differs significantly from that on others. The results are discussed later in the chapter.

Table 5.3. Mean W1 deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	2.99	3.93	3.80	3.60	2.86
2	3.72	4.49	4.66	4.01	3.09
3	3.38	3.57	3.44	3.05	2.91
4	2.62	4.48	3.32	3.10	2.66
5	3.02	3.17	3.85	3.06	2.66
6	2.62	4.53	3.54	3.50	2.93
7	2.23	4.04	3.00	2.83	3.03
8	3.75	4.12	3.98	3.53	3.30
9	2.53	4.09	3.92	3.55	3.45
Mean	2.98	4.05	3.72	3.36	2.99

Table 5.4. Mean W7 deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	1.19	1.21	1.16	1.14	0.82
2	1.24	1.22	1.45	1.26	0.84
3	1.05	0.98	0.88	0.80	0.74
4	0.88	1.05	0.91	0.86	0.72
5	1.10	0.96	1.17	0.92	0.75
6	1.35	1.11	1.02	1.09	0.73
7	0.73	1.01	0.83	0.83	0.82
8	1.29	1.17	1.20	1.20	1.20
9	1.16	1.23	1.26	1.20	0.95
Mean	1.11	1.11	1.10	1.03	0.84

Table 5.5. Mean SCI deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	0.20	0.36	0.65	0.59	0.55
2	0.44	0.46	0.65	0.69	0.48
3	0.41	0.41	0.69	0.68	0.63
4	0.25	0.56	0.66	0.57	0.49
5	0.41	0.32	0.64	0.61	0.51
6	0.30	0.56	0.65	0.60	0.63
7	0.22	0.41	0.57	0.49	0.47
8	0.40	0.41	0.64	0.67	0.65
9	0.19	0.41	0.64	0.53	0.69
Mean	0.31	0.43	0.64	0.60	0.57

Table 5.6. Mean BCI deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	0.47	0.51	0.43	0.39	-0.27
2	0.39	0.57	0.54	0.45	0.01
3	0.40	0.47	0.41	0.34	-0.09
4	0.45	0.61	0.39	0.36	-0.14
5	0.28	0.40	0.43	0.32	-0.19
6	0.46	0.57	0.40	0.39	-0.11
7	0.37	0.57	0.35	0.32	0.04
8	0.41	0.56	0.47	0.40	-0.25
9	0.46	0.51	0.45	0.39	-0.29
Mean	0.41	0.53	0.43	0.37	-0.14

Figures 5.5 through 5.8 illustrate the mean deflection parameter data as tabulated. These results are discussed later in the chapter.

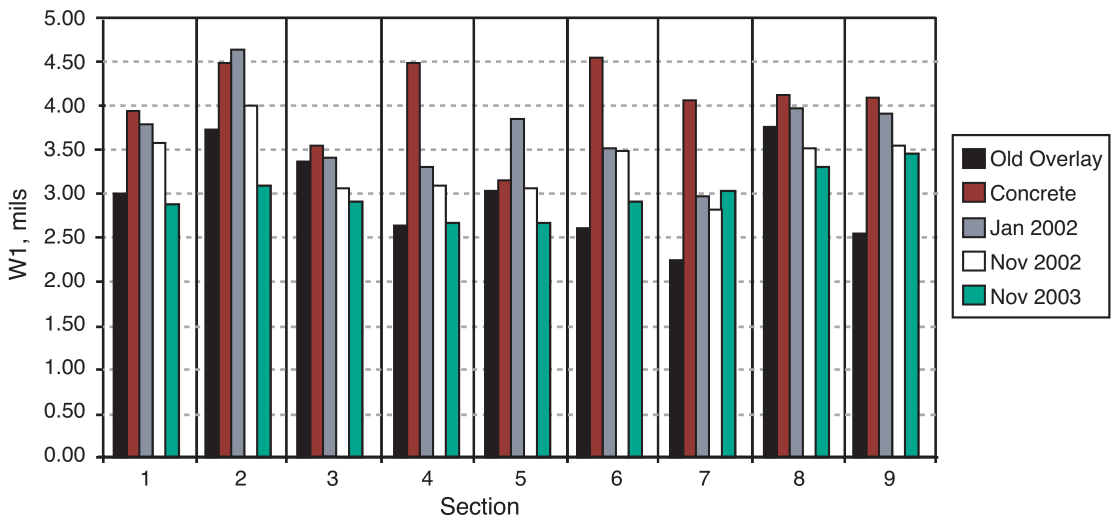

Figure 5.5. Mean W1 FWD deflections for sections evaluated

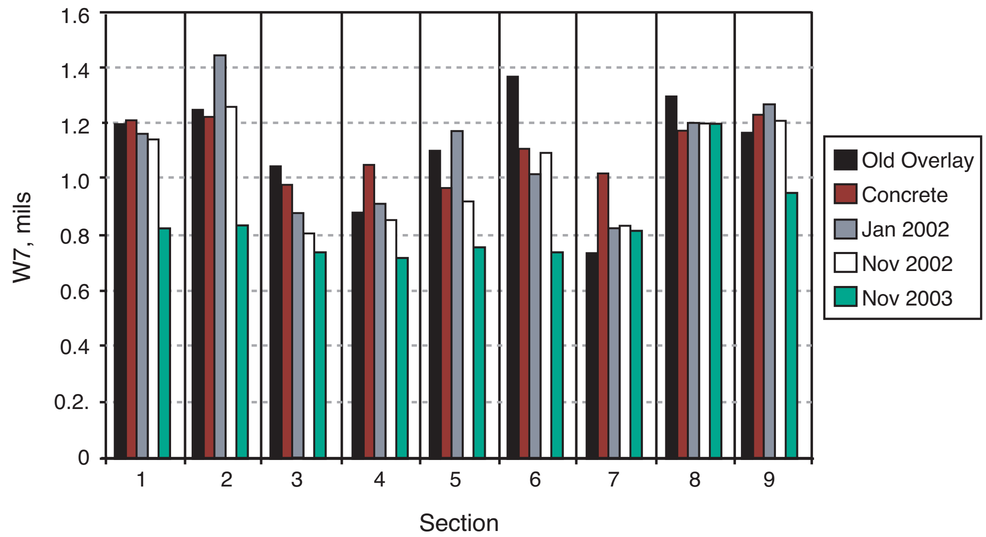

Figure 5.6. Mean W7 FWD deflections for sections evaluated

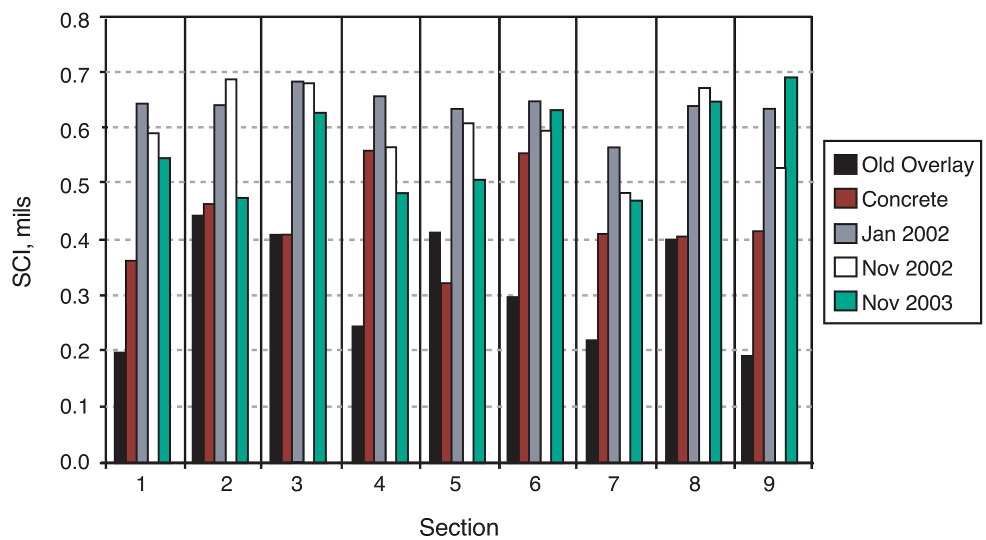

Figure 5.7. Mean SCI for sections evaluated

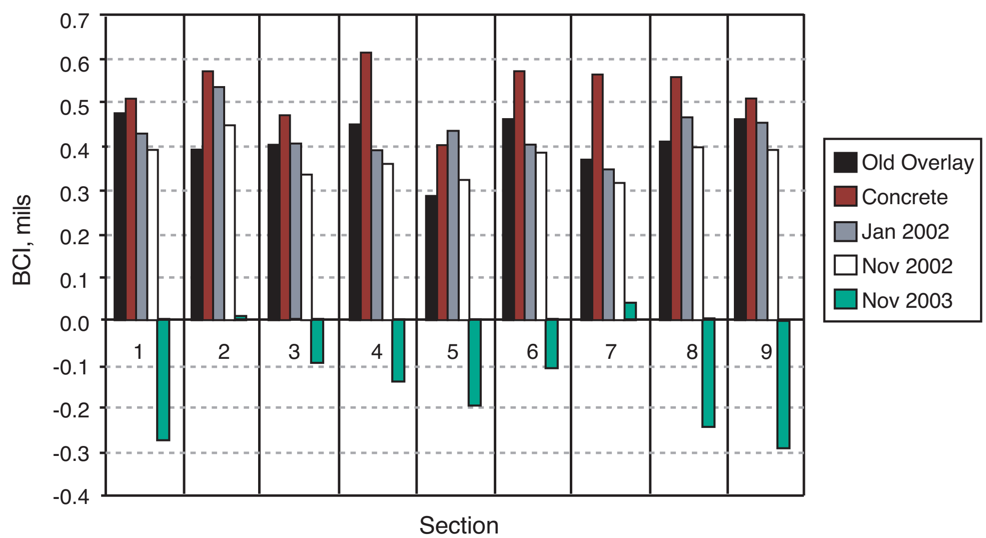

Figure 5.8. Mean BCI for sections evaluated

5.3.3 Standard Deviations

Tables 5.7 through 5.10 indicate the standard deviations of the FWD deflection parameters (W1, W7, SCI, and BCI, respectively) determined for each of the sections during each FWD testing series. The results are discussed later in the chapter.

Table 5.7. Standard deviation of W1 deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	1.13	0.67	1.19	0.94	0.41
2	1.07	1.61	1.20	1.24	0.75
3	0.68	1.01	0.45	0.37	0.40
4	0.97	1.49	0.54	0.58	0.47
5	0.46	0.71	0.56	0.52	0.46
6	0.80	1.66	0.43	0.48	0.40
7	0.66	1.10	0.63	0.68	0.62
8	0.68	1.19	0.60	0.54	0.61
9	0.51	0.85	0.63	0.82	0.63

Table 5.8. Standard deviation of W7 deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	0.42	0.30	0.25	0.28	0.19
2	0.48	0.55	0.55	0.51	0.28
3	0.25	0.35	0.21	0.20	0.20
4	0.37	0.33	0.22	0.22	0.18
5	0.25	0.32	0.27	0.24	0.19
6	0.24	0.44	0.26	0.28	0.21
7	0.22	0.31	0.22	0.27	0.27
8	0.31	0.36	0.27	0.27	0.27
9	0.22	0.35	0.28	0.41	0.16

Table 5.9. Standard deviation of SCI deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	0.14	0.09	0.24	0.22	0.16
2	0.27	0.30	0.20	0.27	0.10
3	0.16	0.35	0.14	0.13	0.12
4	0.16	0.36	0.09	0.13	0.11
5	0.12	0.12	0.13	0.11	0.11
6	0.22	0.49	0.13	0.12	0.12
7	0.19	0.15	0.14	0.10	0.10
8	0.15	0.36	0.12	0.16	0.19
9	0.11	0.16	0.14	0.12	0.12

Table 5.10. Standard deviation of BCI deflections

Section	Overlay	Concrete	Jan 2002	Nov 2002	Nov 2003
1	0.10	0.10	0.15	0.13	0.13
2	0.15	0.20	0.16	0.16	0.16
3	0.14	0.16	0.06	0.06	0.06
4	0.15	0.21	0.08	0.10	0.10
5	0.08	0.11	0.08	0.07	0.07
6	0.09	0.20	0.07	0.10	0.10
7	0.11	0.17	0.09	0.11	0.11
8	0.13	0.18	0.10	0.10	0.10
9	0.09	0.12	0.12	0.11	0.11

Figures 5.9 through 5.11 illustrate the standard deviations of the deflection parameter data as tabulated. From these it is clear that the highest standard deviations are associated with the FWD tests directly on the concrete pavement. The results are discussed later in the chapter.

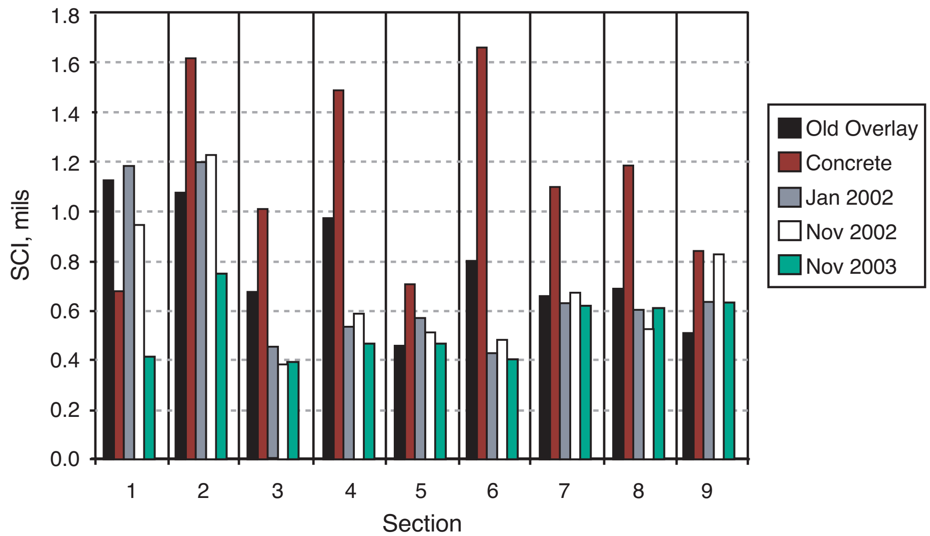

Figure 5.9. Standard deviations of W1 FWD deflections of sections as evaluated

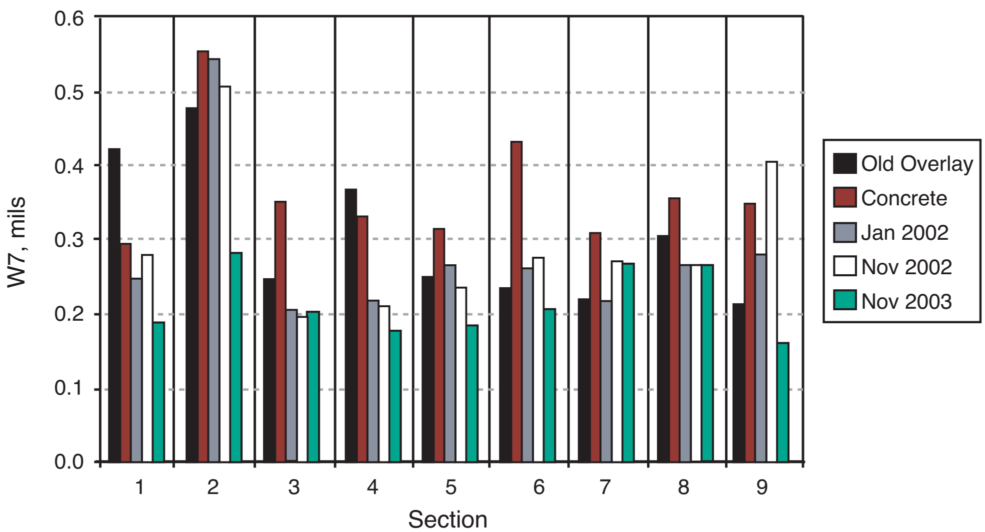

Figure 5.10. Standard deviations of W7 FWD deflections of sections as evaluated

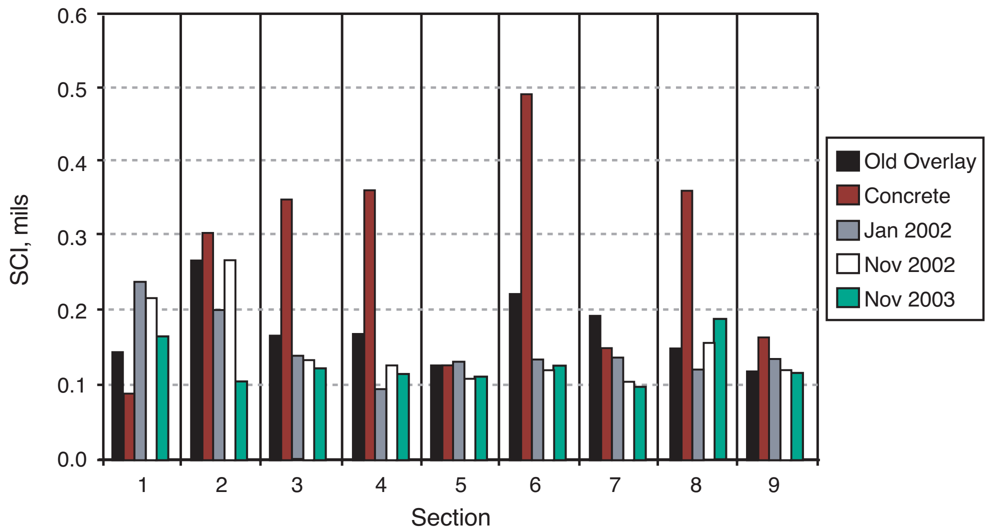

Figure 5.11. Standard deviations of SCI of sections as evaluated

5.4 Discussion of Deflection Results

The FWD results are expressed in terms of means and standard deviations of the deflection parameters W1, W7, SCI, and BCI. The reason for evaluating these deflection parameters is addressed, followed by a discussion of the results in the context of ranking the performance of the different sections.

5.4.1 Deflection Parameters

The deflection of a pavement beneath an FWD load may be used as an indicator of the structural integrity of the pavement. The greater the deflection, the weaker the pavement structure and vice versa. The maximum (W1) deflection indicates the deflection of the entire pavement structure under the load. The W1 deflection includes the collective deflection of the surfacing, base, and subbase layers, as well as the subgrade. Use is made of other deflection parameters such as W7, SCI, and BCI to differentiate between the deflections of the respective layers of the pavement structure. The W7 deflection, for example, although measured on the surface of the pavement, is commonly used as an indicator of subgrade stiffness. Subgrade deflection is influenced predominantly by the stress on the subgrade and hence the integrity or load-spreading ability of the overlying pavement layers and is also influenced to a lesser extent by seasonal variations in moisture content. The surface curvature index (SCI=W1-W2) indicates the curvature of the upper 300 mm (12 in.) of the pavement. Low SCI values indicate that the W1 and W2 deflections are very similar and that the upper pavement structure is not deflecting much relative to the underlying structure under the load. The SCI value alone cannot provide information regarding the strength of the upper pavement structure. It is possible that the upper pavement structure is very weak, which would result in load punching and consequently low SCI values. Hence, in order to assess the pavement's structural integrity, it is necessary to evaluate other parameters such as the base curvature index (BCI=W4-W5). BCI is an indicator of the relative base and subbase layer deflections. Deflection parameters allow an evaluation of the relative deflections and integrity of the respective pavement layers.

5.4.2 Paired Student's t-Test Analyses (January 2002–November 2003)

Paired sample comparisons were done to evaluate the significance of differences between the deflection parameters determined during the January 2002 and November 2003 FWD tests. The null hypothesis assumed that there was no difference between the January 2002 and November 2003 deflections. The statistical student's t-test was applied to the data for the different FWD parameters, the results of which are indicated in Tables 5.11 through 5.14, respectively. Sections with significantly different deflections at the 95 percent confidence level (between January 2002 and November 2003) are shaded in the tables. The numbers of paired sample records evaluated are also indicated.

Table 5.11. Student's t-analyses of W1 deflections

Section	1	2	3	4	5	6	7	8	9
N	22	27	42	33	39	40	33	37	24
t Stat	4.01	7.28	7.01	5.15	10.82	7.83	-0.73	8.47	2.37
t Critical two-tail	2.08	2.06	2.02	2.04	2.02	2.02	2.04	2.03	2.07
Reject Null?	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes

Table 5.12. Student's t-analyses of W7 deflections

Section	1	2	3	4	5	6	7	8	9
N	22	27	42	33	39	40	33	37	24
t Stat	4.48	7.00	3.93	3.64	9.79	6.26	0.30	8.73	3.98
t Critical two-tail	2.08	2.06	2.02	2.04	2.02	2.02	2.04	2.03	2.07
Reject Null?	Yes	Yes	Yes	Yes	Yes	Yes	No	Yes	Yes

Table 5.13. Student's t-analyses of SCI deflections

Section	1	2	3	4	5	6	7	8	9
N	22	27	42	33	39	40	33	37	24
t Stat	1.62	3.93	1.81	6.61	5.12	0.47	3.34	0.67	-1.83
t Critical two-tail	2.08	2.06	2.02	2.04	2.02	2.02	2.04	2.03	2.07
Reject Null?	No	Yes	No	Yes	Yes	No	Yes	No	No

Table 5.14. Student's t-analyses of BCI deflections

Section	1	2	3	4	5	6	7	8	9
N	22	27	42	33	39	40	33	37	24
t Stat	11.90	8.95	14.16	11.94	13.30	13.86	6.87	14.98	13.36
t Critical two-tail	2.08	2.06	2.02	2.04	2.02	2.02	2.04	2.03	2.07
Reject Null?	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

As previously discussed, the deflection parameters provide an indication of the relative deflection of the layers within the pavement structure. These parameters are inter-related; a decrease in one parameter may be associated with a decrease in another deflection parameter. This is emphasized because a decrease in SCI, for example, may be related to stiffening or densification of the asphalt layer or upper pavement structure, which is to be expected for newly constructed asphalt layers after 10 months in the field. Based on the statistical analyses, the following observations are made regarding the deflections on the different sections.

Discussions

The statistical analyses indicated a significant difference in the W1, W7, and BCI deflection parameters between January 2002 and November 2003. Each of these parameters decreased in magnitude between January 2002 and November 2003. No significant difference in SCI was apparent. Given the large number of factors influencing the deflections of pavement structure, it is difficult to identify the exact reason for the decrease in FWD deflection. The fact that the SCI did not decrease significantly, however, may indicate that the stiffening of the pavement structure is not directly related to the nature of the surfacing layer. The lower BCI may be an indicator of densification within the base/subbase layers or strengthening of the subgrade. The latter may be related to moisture conditions within the subgrade. Pavement Sections 3, 6, 8, and 9 exhibited similar behavior.

A significant decrease in each of the deflection parameters is apparent in Sections 2, 4, and 5. The decrease in SCI indicates a relative stiffening or densification of the surfacing layer or upper pavement structure. This may in turn be the reason for the lower W1, W7, and BCI deflection parameters. Traffic-related densification of the asphalt layers is expected. This tends to stiffen the asphalt layer, which could be the reason for the lower deflections apparent in the section.

Significant decreases in SCI and BCI are apparent on Section 7. The higher t-statistic determined for the SCI deflections may indicate that the corresponding decrease in BCI is consequential. It is interesting to note that this is the only section in which neither W1 nor W7 decreased significantly. It may be concluded that the strengthening of the upper structure of Section 7 did not contribute to the overall deflection of the pavement structure as a whole.

No specific trends are evident from the FWD deflection data that may be used to infer the relative performance of the mixes on the different sections evaluated. It was found that construction of the new overlay resulted in a decrease in the magnitude and extent of deflections apparent on the old pavement structure, but that it does not appear to significantly contribute to the structural capacity of the pavement.

6. Rolling Dynamic Deflectometer Measurements: Overview of the Rolling Dynamic Deflectometer

6.1 Introduction

Researchers at The University of Texas at Austin first developed the rolling dynamic deflectometer (RDD) in the late 1990s. A comprehensive description of the RDD is given in *Development of a Rolling Dynamic Deflectometer for Continuous Deflection Testing of Pavements* (Bay and Stokoe 1998). The RDD as described in this report is a research prototype device that was converted from a Vibroseis, a geophysical exploration tool. A schematic diagram of the RDD is shown in Figure 6.1.

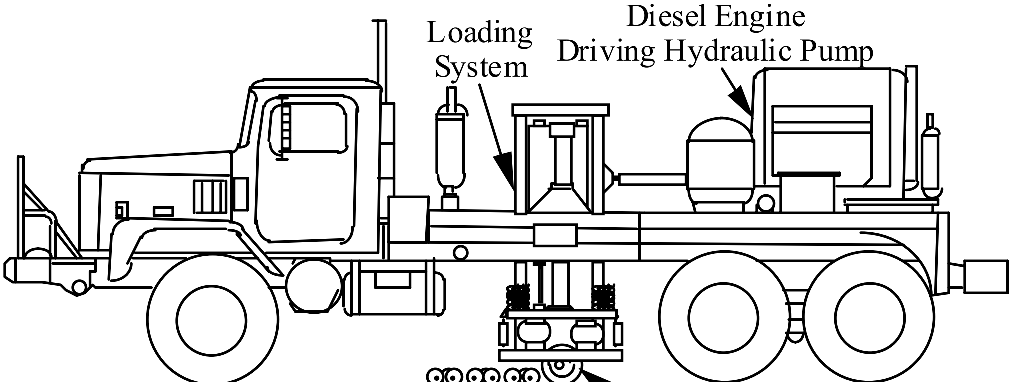

Figure 6.1. Schematic diagram of the major components of the RDD (after Bay 1997)

6.2 Rolling Dynamic Deflectometer Continuous Deflection Profiles

RDD testing was carried out along Interstate Highway 20 near Marshall, Texas, at different stages of the asphalt overlay project. Until now, RDD continuous deflection profiles were collected at five different stages, which are: Stage 1 – before milling off the old asphalt surface; Stage 2 – after milling off the old asphalt layer; Stage 3 – 1 month after the new overlay; Stage 4 – 11 months after the new overlay; and Stage 5 –23 months after the new overlay. RDD profiles were obtained at these different stages of the overlay project so that a baseline could be established prior to the overlay, and the pavement response was monitored at subsequent stages of the project. The schedule of the RDD testing is shown in Table 6.1.

Table 6.1. Schedule of the RDD testing along Interstate Highway 20

	Westbound Lane	**Eastbound Lane**
Stage 1	March 2, 2001	April 5, 2001
Stage 2	August 30, 2001	September 28, 2001
Stage 3	January 8, 2002	January 9, 2002
Stage 4	November 13, 2002	November 14, 2002
Stage 5	November 18, 2003	November 19, 2003

To date, the RDD testing has been focused on the westbound and eastbound outside lanes. The RDD continuous deflection profiles were collected along the outside lanes at all five stages of the overlay project. Furthermore, the continuous deflection profile along the westbound inside lane was also collected at Stage 1 of the project.

During testing, the RDD applies a static hold-down force and a dynamic force to the pavement with two polyurethane-coated loading rollers. A nominal peak-to-peak dynamic force of 10 kips (44.5 kN) at 35 Hz was used at all stages. However, the nominal static hold-down force varies from 10–15 kips (44.5 – 66.7 kN).

The test section under investigation lies between stations 1135+00 and 1321+00 on the eastbound and westbound lanes of Interstate Highway 20 near Marshall, Texas. The test section is divided into nine different subsections, and a different asphalt overlay mix design was used for each subsection. Four of these sections are located on the westbound side, and the remaining five are located on the eastbound side. A summary of the different mix designs and the station limits for each subsection can be seen in Tables C.1 and C.2 in Appendix C.

The RDD continuous deflection profiles were collected at all five stages of the overlay project. For each subsection, sensor #1 deflection readings for each stage are shown in Figures 6.2 to 6.10.

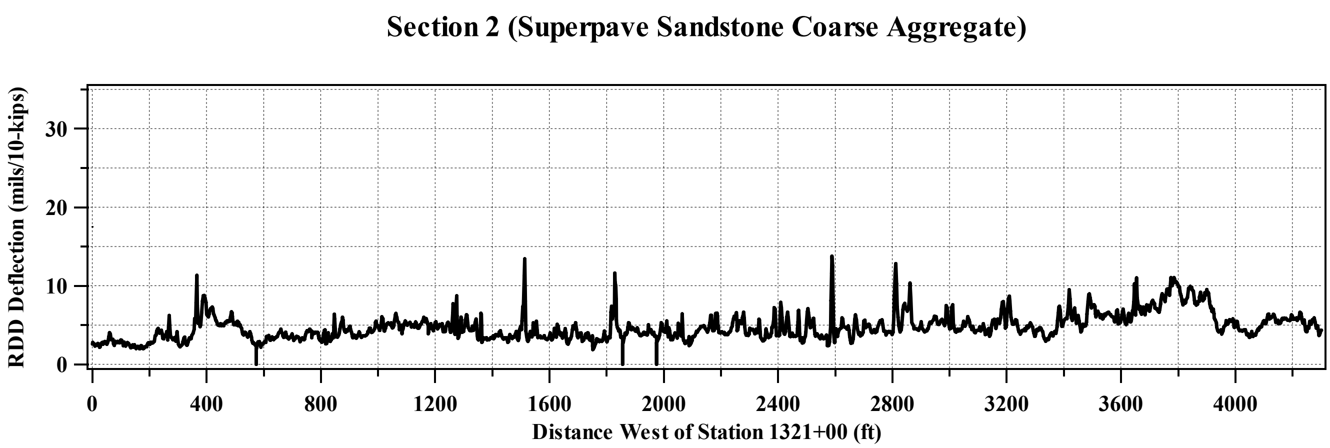

Figure 6.2. RDD deflection profile for Section 2 along Interstate Highway 20

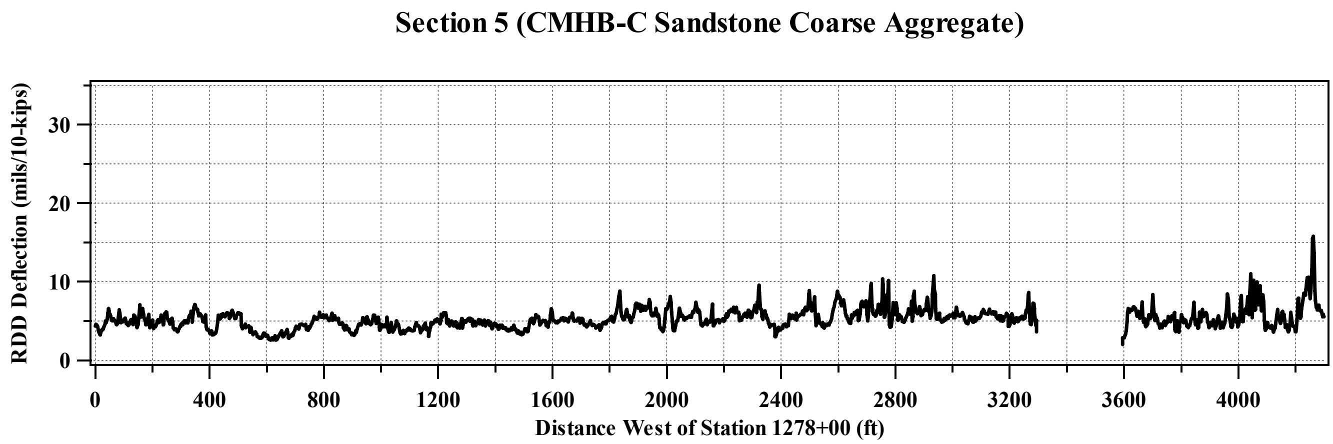

Figure 6.3. RDD deflection profile for Section 5 along Interstate Highway 20

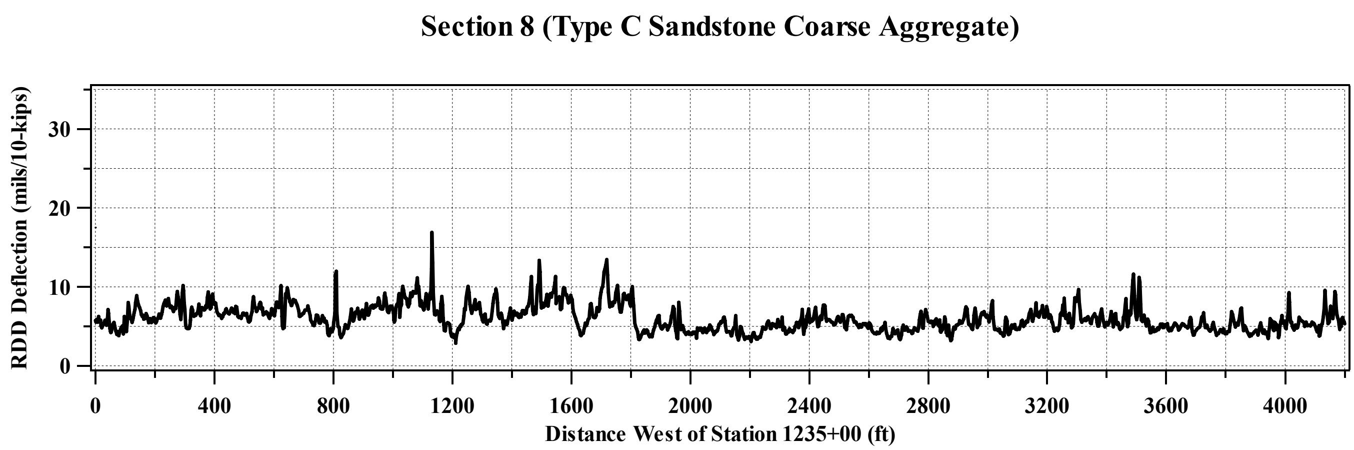

Figure 6.4. RDD deflection profile for Section 8 along Interstate Highway 20

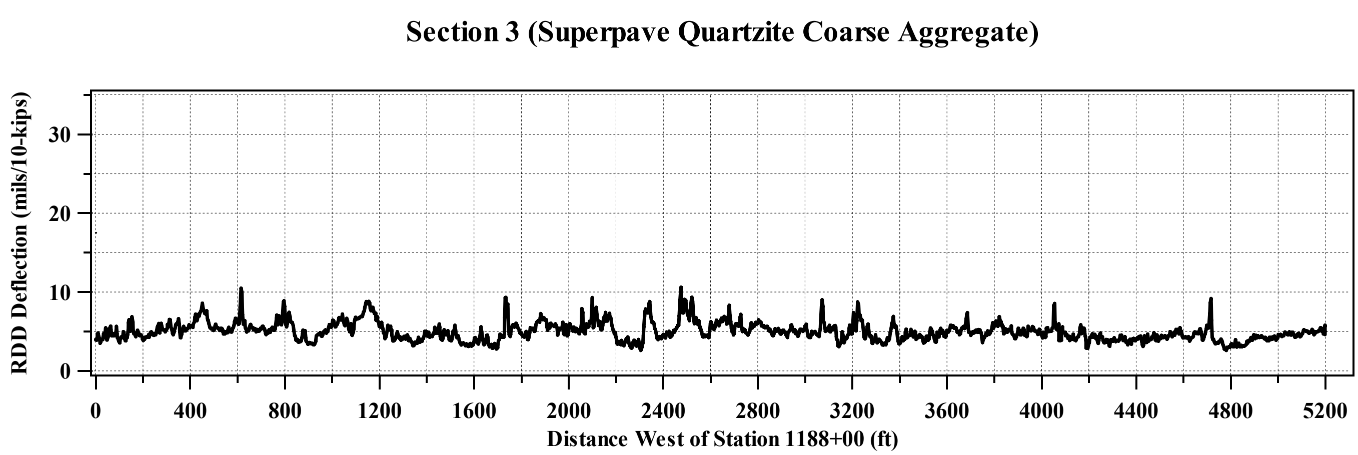

Figure 6.5. RDD deflection profile for Section 3 along Interstate Highway 20

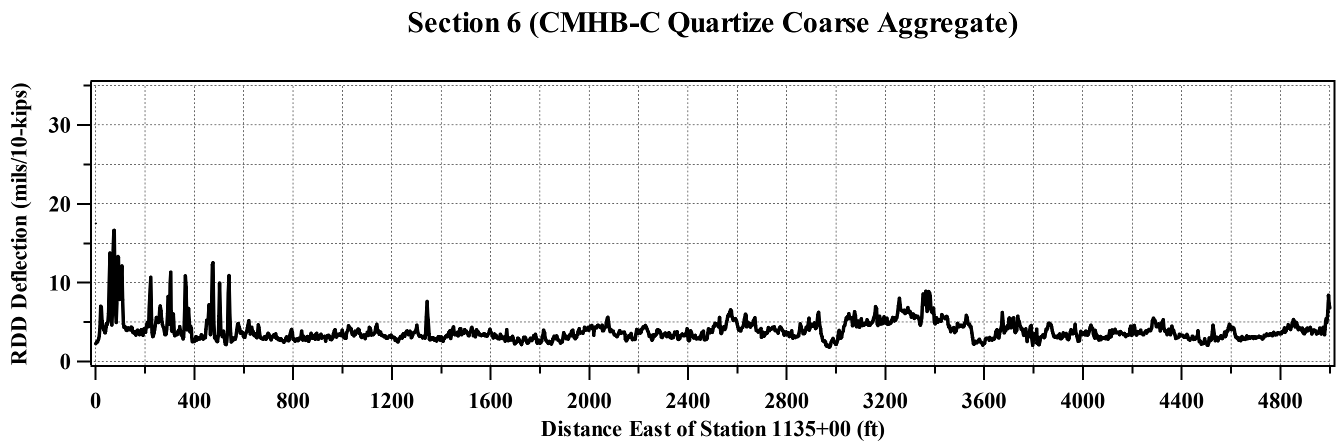

Figure 6.6. RDD deflection profile for Section 6 along Interstate Highway 20

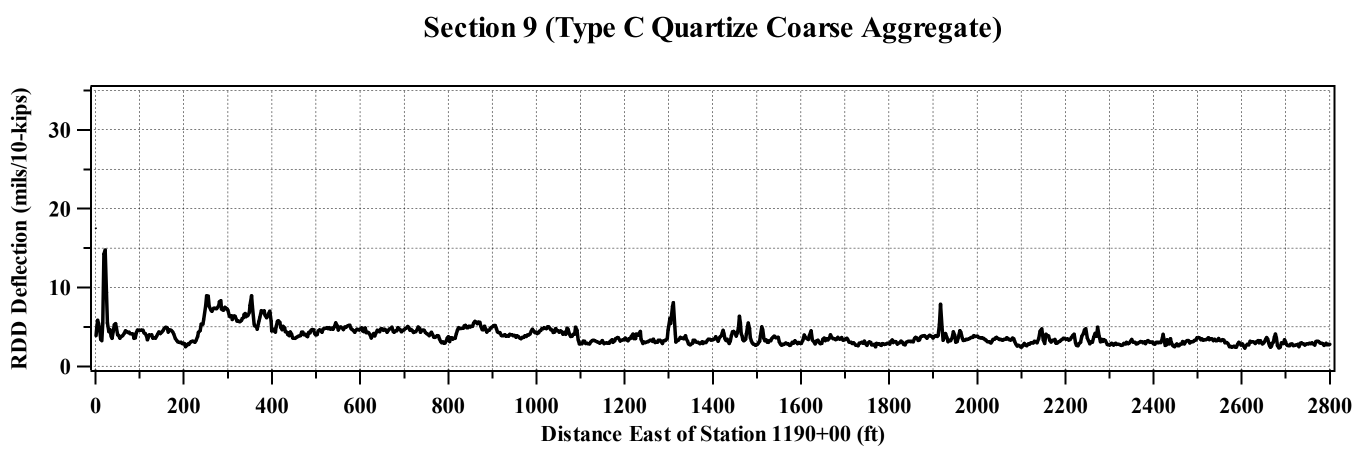

Figure 6.7. RDD deflection profile for Section 9 along Interstate Highway 20

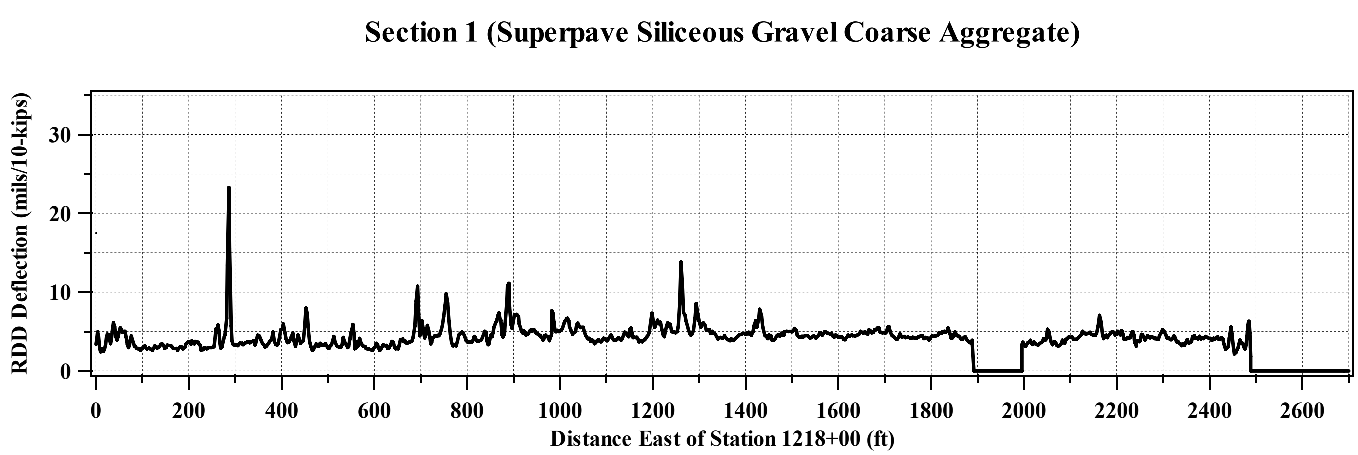

Figure 6.8. RDD deflection profile for Section 1 along Interstate Highway 20

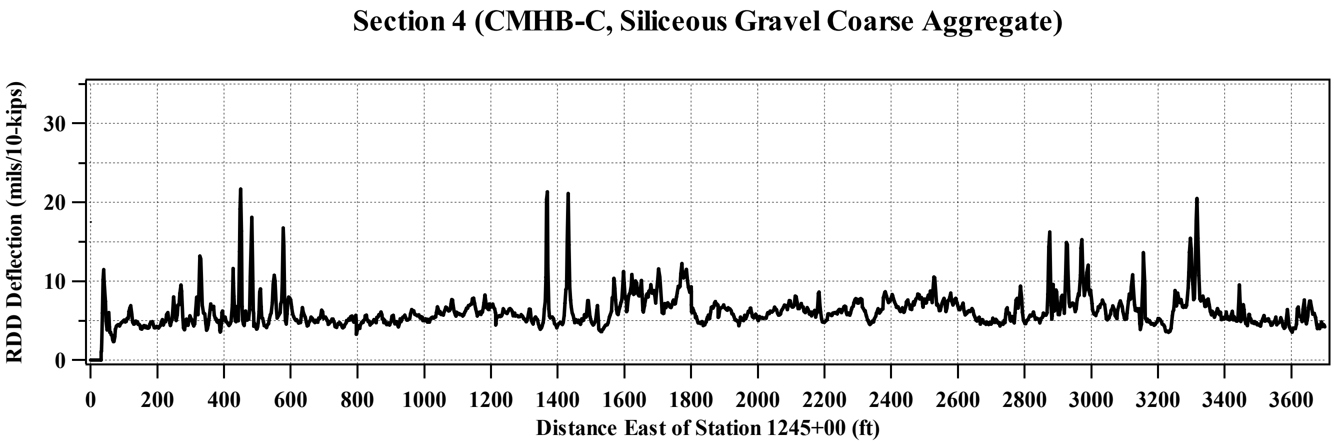

Figure 6.9. RDD deflection profile for Section 4 along Interstate Highway 20

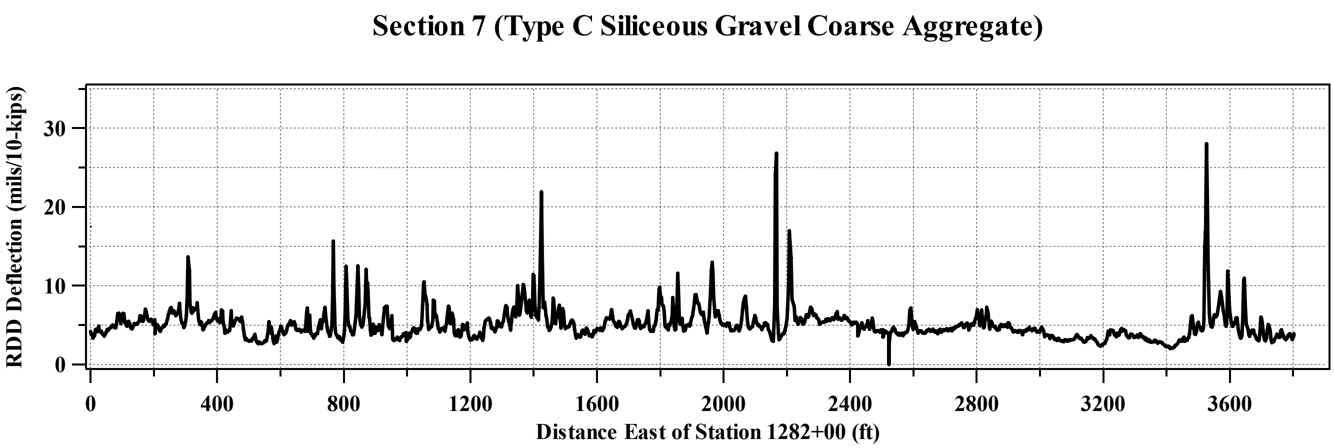

Figure 6.10. RDD deflection profile for Section 7 along Interstate Highway 20

Based on the sensor #1 deflection profiles shown in Figures 6.2 through 6.10, the summary statistics for the different mix designs were calculated and are shown in Table 6.2. The same information is also shown graphically in Figure 6.11.

Table 6.2. Summary statistics for the RDD deflection profile on Interstate Highway 20

	Station Limits (ft)		Stage 1 (March 2001)		Stage 2 (August 2001)		Stage 3 (January 2002)		Stage 4 (November 2002)		Stage 5 (November 2003)	
Section	**Begin**	**End**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**
3	1135+00	1188+00	5.39	2.38	5.82	2.73	3.64	1.26	5.62	1.98	4.96	1.16
8	1193+00	1235+00	6.53	2.72	6.59	2.42	4.40	1.18	-	-	5.85	1.68
5	1235+00	1278+00	7.73	2.66	6.52	2.05	4.99	1.01	3.97	0.90	5.05	1.33
2	1278+00	1321+00	8.91	2.99	7.14	2.09	5.14	1.15	4.51	0.94	4.96	1.16

	Station Limits (ft)		Stage 1 (April 2001)		Stage 2 (September 2001)		Stage 3 (January 2002)		Stage 4 (November 2002)		Stage 5 (November 2003)	
Section	**Begin**	**End**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**	**Mean (μ)**	**St. Dev (δ)**
6	1135+00	1185+00	7.29	2.76	5.89	2.32	3.60	0.89	3.39	1.12	3.78	1.38
9	1190+00	1218+00	5.96	1.76	4.85	1.32	3.54	0.79	3.67	0.88	3.77	1.19
1	1218+00	1245+00	9.16	3.86	5.65	1.86	3.96	0.87	4.56	1.62	4.10	1.46
4	1245+00	1282+00	9.90	3.96	7.20	2.15	5.15	1.12	5.63	1.63	5.95	2.06
7	1282+00	1321+00	10.98	4.10	6.23	2.65	4.50	1.15	4.61	1.81	4.83	2.05

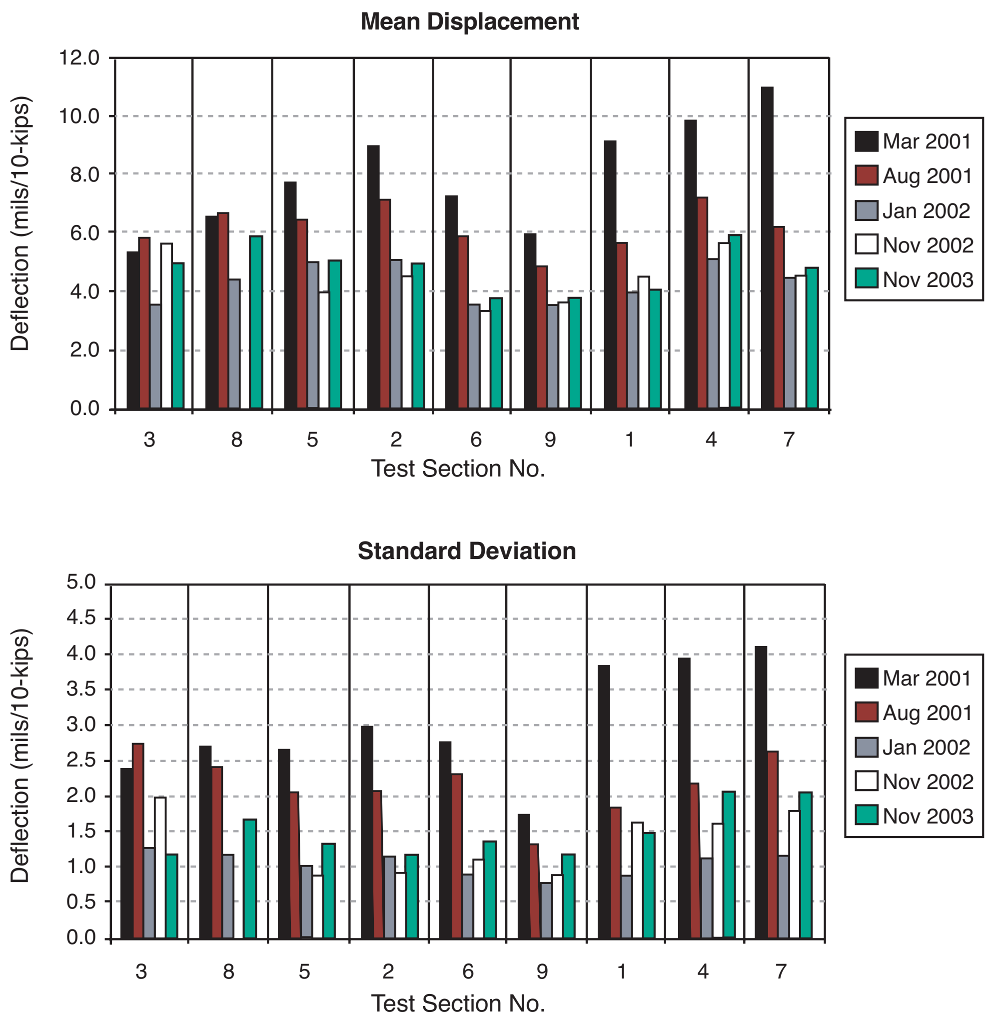

Figure 6.11. Summary statistics of the RDD continuous deflection profile

7. Portable Seismic Pavement Analyzer Measurements

Three series of portable seismic pavement analyzer (PSPA) measurements were done on the IH 20 sections being evaluated in Harrison County after the first series of PSPA tests were done directly on top of the concrete pavement, after the old overlay had been milled off. Section details as well as the different mixes used on the sections are outlined in Appendix C. These three series of tests were done after construction of the new pavement sections in January 2002, November 2002, and November 2003, respectively. In addition, laboratory V-meter tests were done on cores removed from the pavement sections in March 2002. This chapter reports and discusses the results of the different V-meter and PSPA tests.

Tables 7.1 and 7.2 summarize the V-meter modulus measurements done on the cores taken from the different sections as well the PSPA measurements done in the field during January 2002, November 2002, and November 2003. The average (Avg), minimum (min) and maximum (max) range of the moduli values, as well as the coefficients of variation (C.V.) for the PSPA tests on the different sections, are shown. Modulus values shown have been adjusted to a temperature of 77 °F and frequency of 30 Hz.

Figure 7.1 shows the difference in the average moduli measurements from the different sections. Changes in the modulus values from January 2002 to November 2002 are presented in Report 4185-3. In this report, we examine the changes in the modulus values from January 2002 to November 2003. In Figure 7.1, it can be seen that on average for all the sections evaluated the moduli values increased from January 2002 to November 2003.

To explore this finding further, a statistical analysis of the difference between the modulus measurements in January 2002 and November 2003 was done applying a t-test with the null hypothesis that there was no difference between the mean moduli in January 2002 and November 2003 at the 95 percent confidence level and assuming unequal variances. Results of these analyses are shown in Table 3. From the table it can be seen that the null hypothesis is rejected on all sections except Section 3, which consists of Superpave mix design with quartz aggregate. Therefore, for all sections except Section 3, mean moduli values increased from January 2002 to November 2003. For Section 3, mean moduli did not change through this period.

Based on the results of the PSPA tests, it may be concluded that with the exception of Section 3, there was a significant increase in the asphalt modulus of the sections evaluated between January 2002 and November 2003.

Table 7.1. Summary of V-meter and PSPA measurements in March 2002 and January 2002

Section Number	Mix	LAB (Cores) - Mar. 2002				PSPA - Jan. 2002			
		Average	Min	Max	C. V.	**Average**	Min	Max	C. V.
		ksi	ksi	ksi	%	**ksi**	ksi	ksi	%
1	Superpave Siliceous	**575**	518	630	9.2	**577**	470	659	10.8
2	Superpave Sandstone	**593**	563	626	5.2	**560**	487	660	5.9
3	Superpave Quartz	**625**	591	669	10.7	**622**	545	832	7.7
4	CMHB-C Siliceous	**662**	618	688	4.8	**683**	515	799	12.0
5	CMHB-C Sandstone	**516**	501	539	3.2	**515**	487	660	8.6
6	CMHB-C Quartz	**507**	432	567	11.2	**608**	395	704	13.4
7	Type-C Siliceous	**637**	632	645	0.9	**572**	381	698	11.5
8	Type-C Sandstone	**542**	508	565	4.8	**531**	437	633	8.0
9	Type-C Quartz	**589**	574	606	2.7	**566**	460	618	7.2

Table 7.2. Summary of PSPA measurements in November 2002 and November 2003

Section Number	Mix	PSPA - Nov. 2002				PSPA - Nov. 2003			
		Average	Min	Max	C. V.	**Average**	Min	Max	C. V.
		ksi	ksi	ksi	%	**ksi**	ksi	ksi	%
1	Superpave Siliceous	**583**	469	733	11.1	**728**	478	963	**11.8**
2	Superpave Sandstone	**564**	412	725	11.8	**619**	466	791	**12.2**
3	Superpave Quartz	**563**	409	792	16.0	**608**	406	916	**15.9**
4	CMHB-C Siliceous	**659**	471	851	14.0	**771**	488	1006	**15.2**
5	CMHB-C Sandstone	**513**	394	634	10.8	**582**	416	816	**16.0**
6	CMHB-C Quartz	**549**	397	651	12.3	**713**	538	922	**12.5**
7	Type-C Siliceous	**656**	505	743	8.9	**769**	606	966	**14.1**
8	Type-C Sandstone	**510**	421	662	13.0	**566**	378	809	**14.8**
9	Type-C Quartz	**517**	369	622	11.4	**695**	594	947	**11.2**

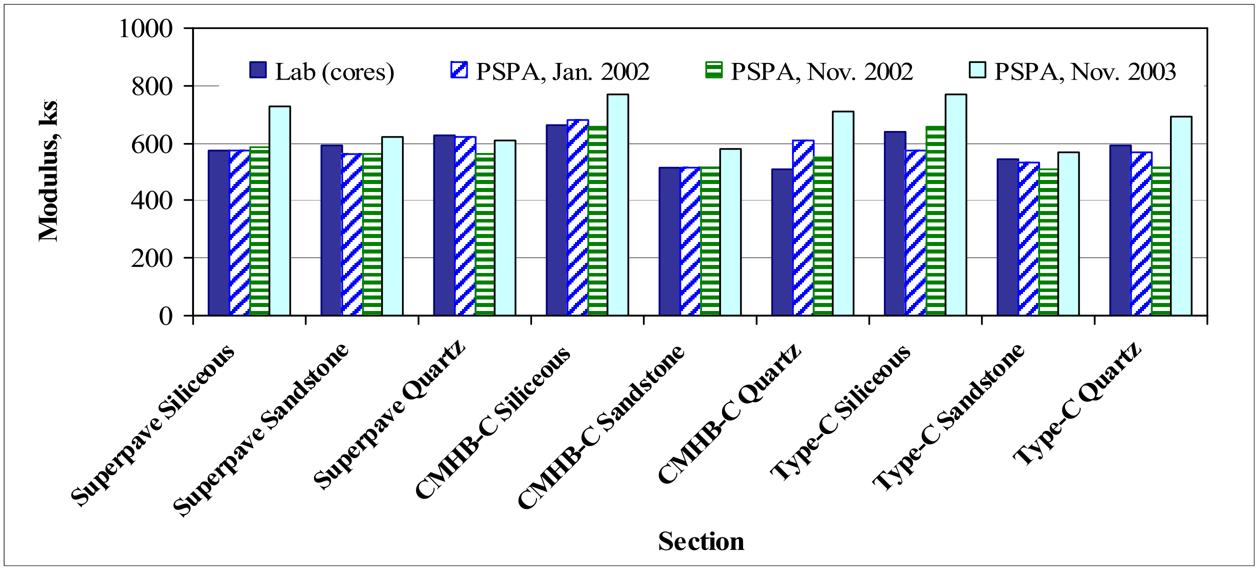

Figure 7.1. Comparison of average moduli measurements done on the different sections

Table 7.3. Statistical analyses results for PSPA modulus means between January 2002 and November 2003

Section	Mix	PSPA - Jan. 2002		PSPA - Nov. 2003					
		Average	Standard Deviation	Average	Standard Deviation	Degree of Freedom	t-statistics	t_{α}	Null Hypothesis
		ksi		ksi					
1	Superpave Siliceous	577	62.3	728	86.0	34	8.576	1.74	Rejected
2	Superpave Sandstone	560	33.0	619	75.5	68	6.045	1.69	Rejected
3	Superpave Quartz	622	47.9	608	96.6	48	0.983	1.711	Accepted
4	CMHB-C Siliceous	683	81.9	771	117.2	56	4.711	1.701	Rejected
5	CMHB-C Sandstone	515	44.3	582	93.2	72	5.626	1.684	Rejected
6	CMHB-C Quartz	608	81.5	713	89.1	54	6.488	1.703	Rejected
7	Type-C Siliceous	572	65.8	769	108.4	64	12.615	1.697	Rejected
8	Type-C Sandstone	531	42.5	566	83.8	50	2.729	1.708	Rejected
9	Type-C Quartz	566	40.7	695	77.8	34	8.800	1.734	Rejected

8. Conclusions

In this project, nine asphalt mixes with underlying Type B base mixture were placed on the test sections on IH 20 in Harrison County. Superpave, CMHB-C, and Type C mix designs and siliceous gravel, sandstone, and quartzite aggregates were used for the construction of the test sections. PG 76-22 asphalt binder was used for all mixtures.

The project is scheduled to continue for one more year, for a total of five years. During this period field performances will be monitored using nondestructive devices, and visual surveys will be carried out. The laboratory tests already have been completed and the data was presented in Research Reports 4185-1 and 4185-2. This report summarizes the visual pavement condition survey and the International Roughness Index, rut depth, falling weight deflectometer, rolling dynamic deflectometer, and portable seismic pavement analyzer measurements collected at the test sections in the fourth year of the study. At the end of five years, all information from field and laboratory tests will be assembled and compared. It will be determined if the Hamburg Wheel Tracking Device (HWTD) can properly predict the performance of the overlays under field conditions. Correlations will be developed between the HWTD and the field performance data.

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Appendix A:
Crack Pictures for Eastbound and Westbound Lanes

Figures A1–A62: Westbound

Figure A1. WBP2PD

Figure A2. WBP2PD

Figure A3. WBP3PD

Figure A4. WBP4PD

Figure A5. WBP5TC

Figure A6. WBP6PD

Figure A7. WBP7PD

Figure A8. WBP8TC

Figure A9. WBP9PD

Figure A10. WBP10TC

Figure A11. WBP11TC

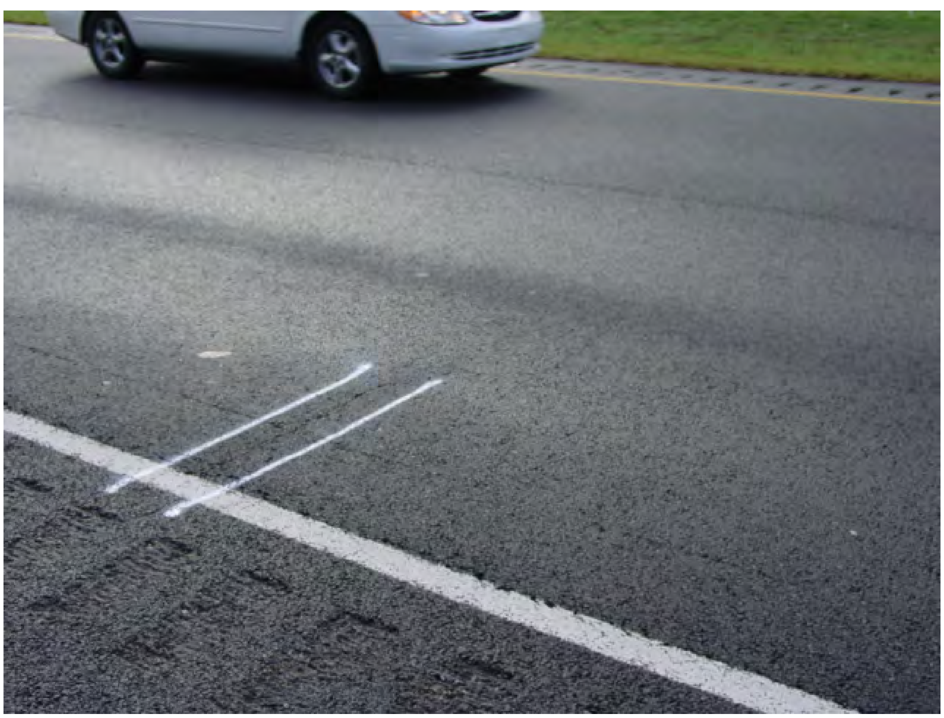

Figure A12. WBP12TC

Figure A13. WBP13TC

Figure A14. WBP14PD

Figure A15. WBP15TC

Figure A16. WBP16TC

Figure A17. WBP17TC

Figure A18. WBP18TC

Figure A19. WBP19TC

Figure A20. WBP20PD(a)

Figure A21. WBP20PD(b)

Figure A22. WBP21TC

Figure A23. WBP22TC

Figure A24. WBP23PD

Figure A25. WBP24TC

Figure A26. WBP25PD

Figure A27. WBP26PD

Figure A28. WBP27SC

Figure A29. WBP28PD

Figure A30. WBP29PD

Figure A31.   WBP30TC

Figure A32.   WBP31TC

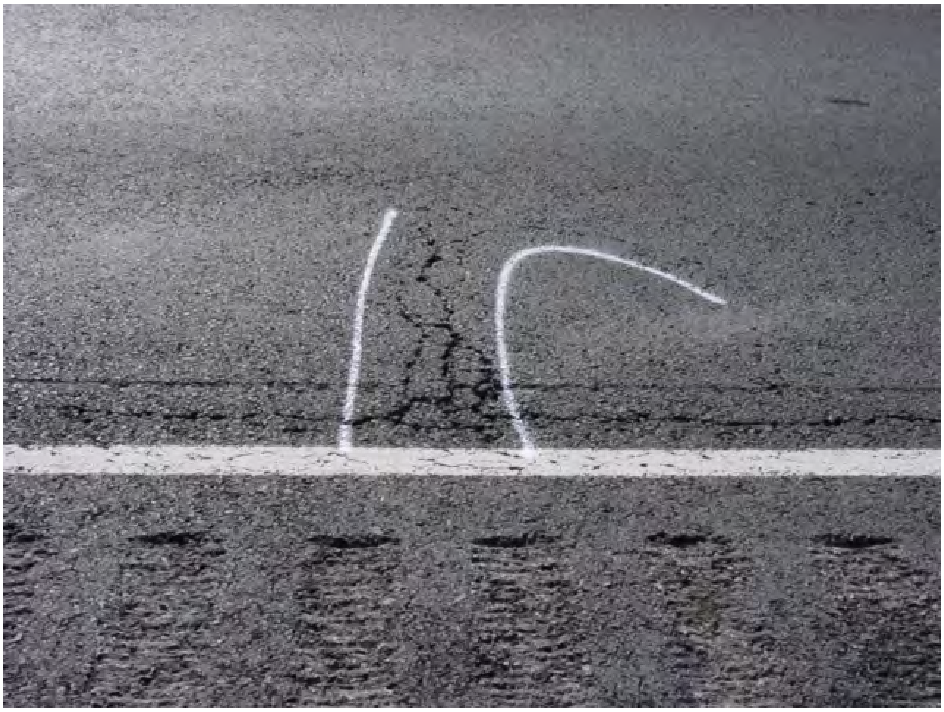

Figure A33. WBP32TC

Figure A34. WBP33PD

Figure A35.   WBP34PD

Figure A36.   WBP35TC

Figure A37. WBP36PD

Figure A38. WBP37TC

Figure A39. WBP38TC

Figure A40. WBP39TC

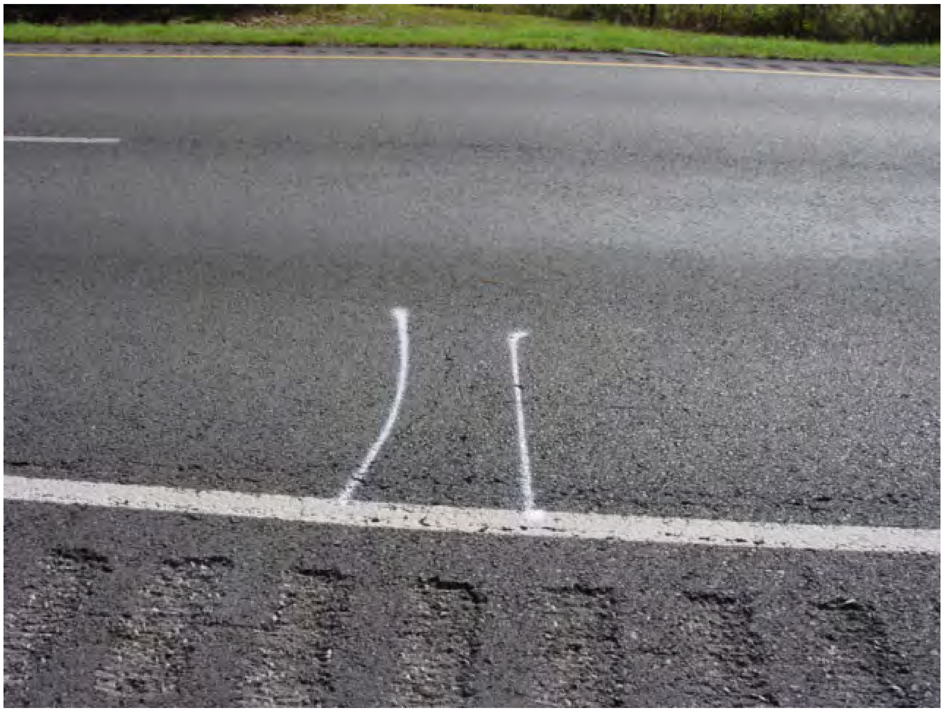

Figure A41. WBP40TC

Figure A42. WBP41PD

Figure A43. WBP42PD

Figure A44. WBP43TC

Figure A45. WBP44PD

Figure A46. WBP45PD

Figure A47. WBP46TC

Figure A48. WBP47TC

Figure A49. WBP48TC

Figure A50. WBP49TC

Figure A51. WBP50TC

Figure A52. WBP51TC

Figure A53. WBP52PD

Figure A54. WBP53SC

Figure A55. WBP54TC

Figure A56. WBP55TC

Figure A57. WBP56WS

Figure A58. WBP57PD

Figure A59. WBP58PH

Figure A60. WBP59PH

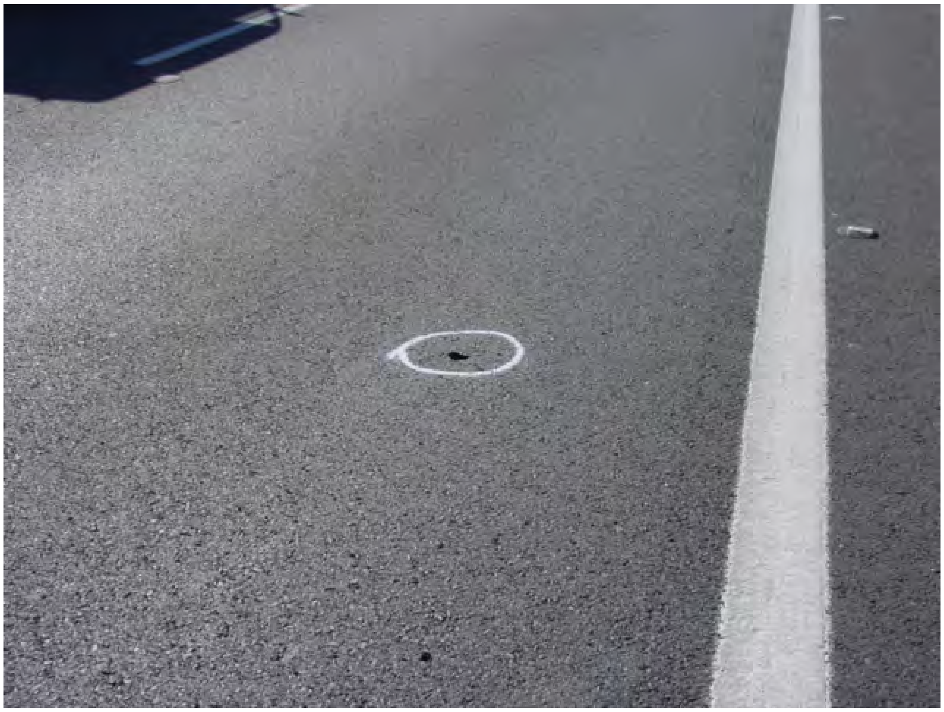

Figure A61. WBP60PH

Figure A62. WBP61TYPE-C

Figures A63–A125: Eastbound

Figure A63. EBP1TC

Figure A64. EBP2TC

Figure A65. EBP3TC

Figure A66. EBP4TC

Figure A67. EBP5TC

Figure A68. EBP6TC

Figure A69. EBP7TC

Figure A70. EBP8TC

Figure A71. EBP9TC

Figure A72. EBP10TC

Figure A73. EBP11TC

Figure A74. EBP12TC

Figure A75. EBP13TC

Figure A76. EBP14TC

Figure A77. EBP15TC

Figure A78. EBP16TC

Figure A79. EBP17TC

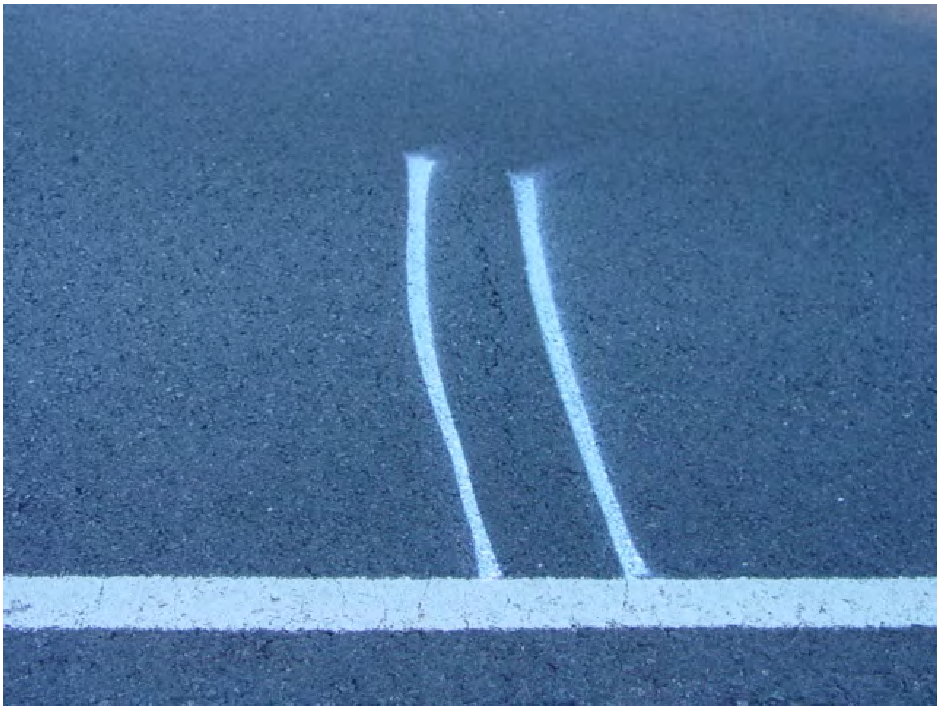

Figure A80. EBP18TC

Figure A81. EBP19TC

Figure A82. EBP20TC

Figure A83. EBP21TC

Figure A84. EBP22TC

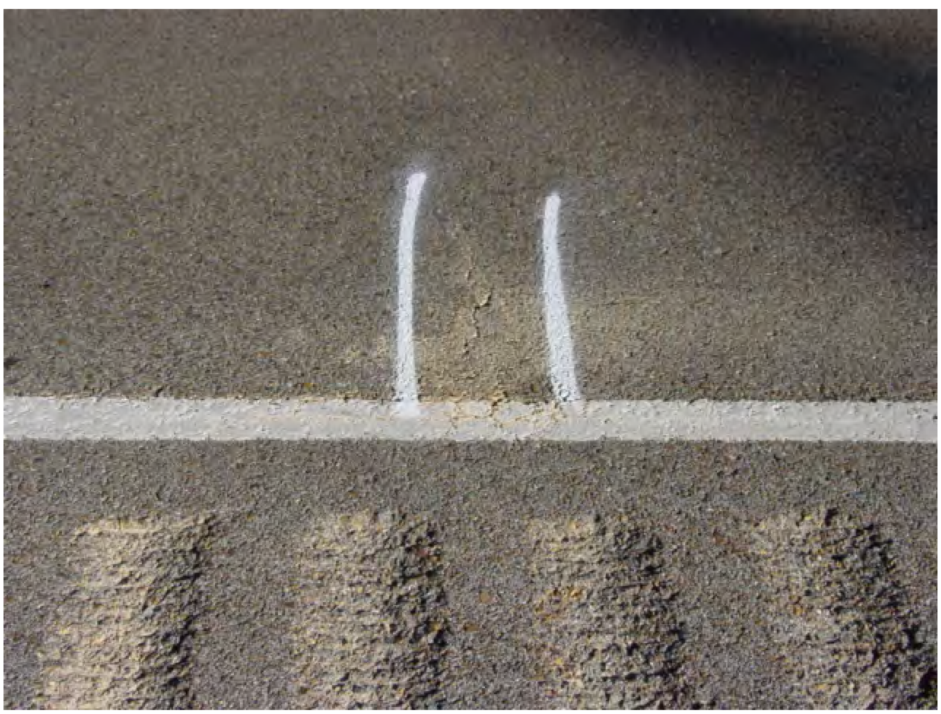

Figure A85. EBP23TC

Figure A86. EBP24PD

Figure A87. EBP25PD

Figure A88. EBP26TC

Figure A89. EBP27TC

Figure A90. EBP28TC

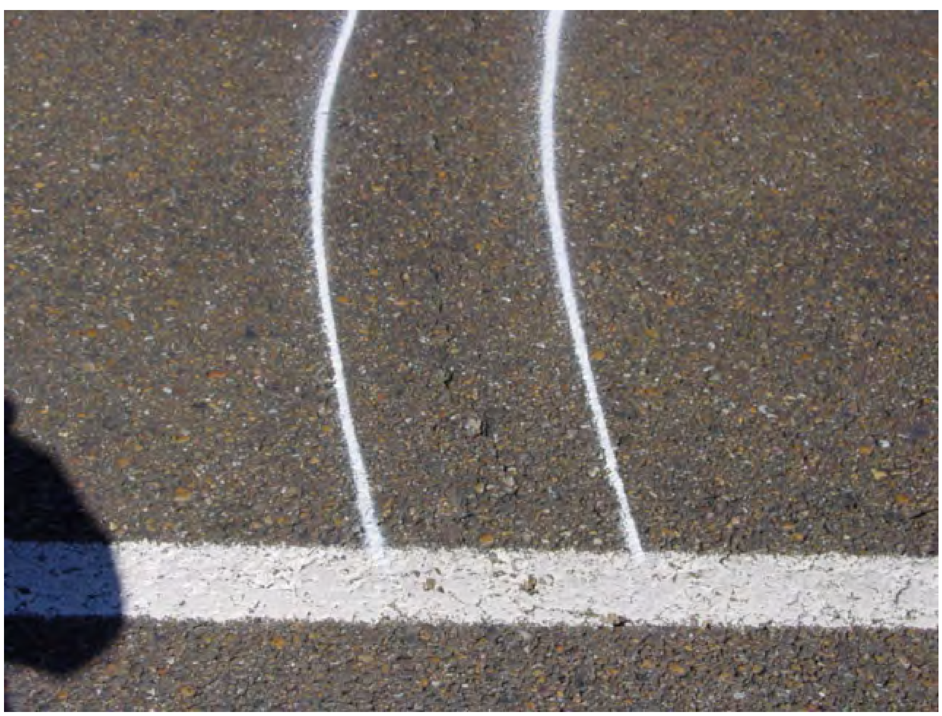

Figure A91. EBP29TC

Figure A92. EBP30TC

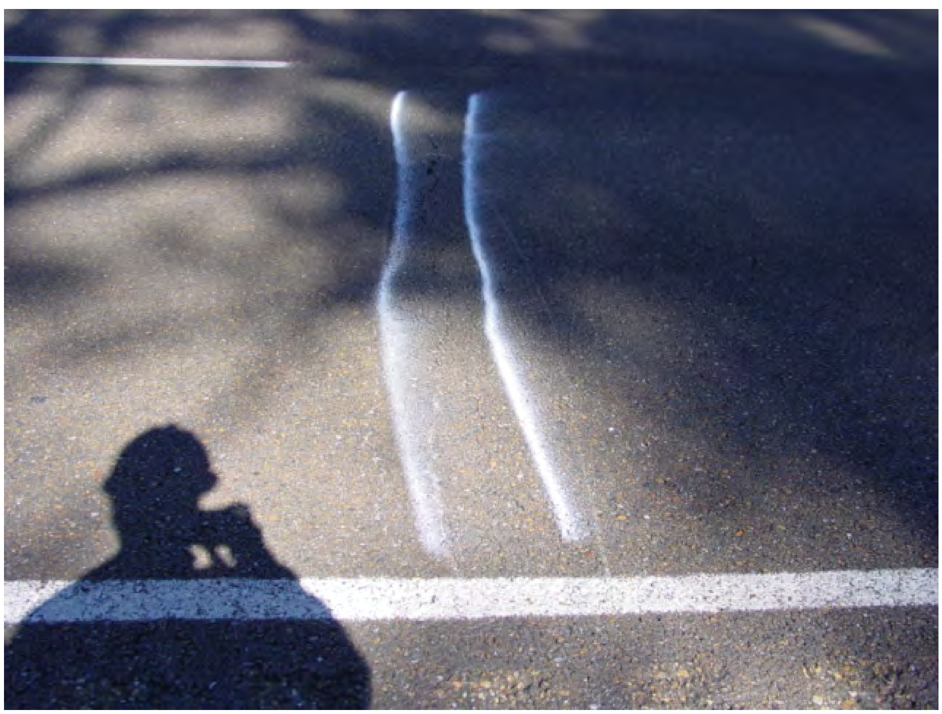

Figure A93. EBP31TC

Figure A94. EBP32PD

Figure A95. EBP33TC

Figure A96. EBP34TC

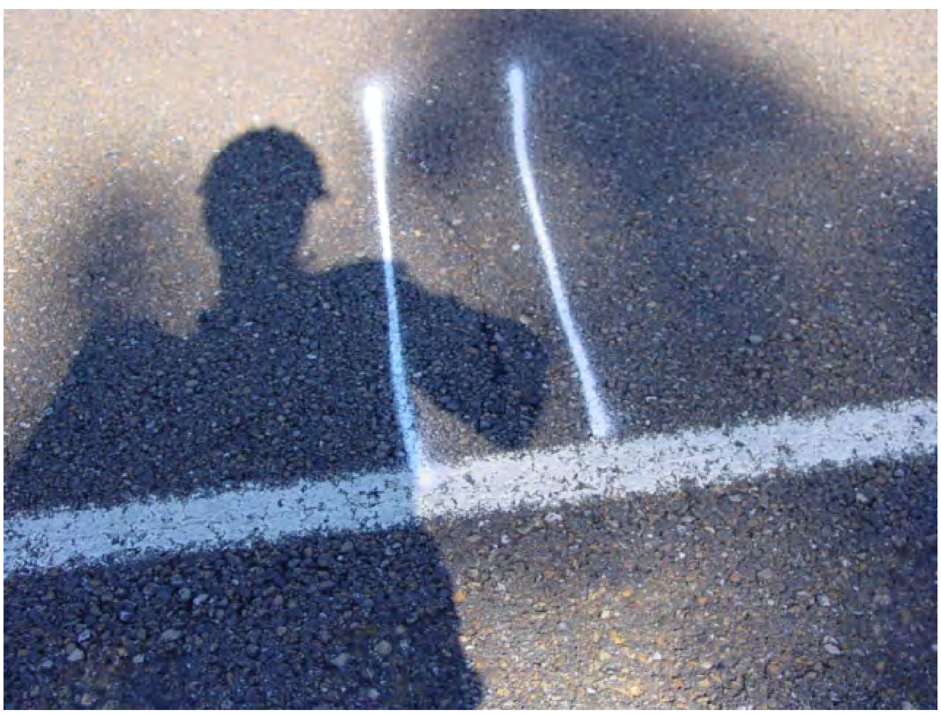

Figure A97. EBP35TC

Figure A98. EBP36TC

Figure A99. EBP37TC

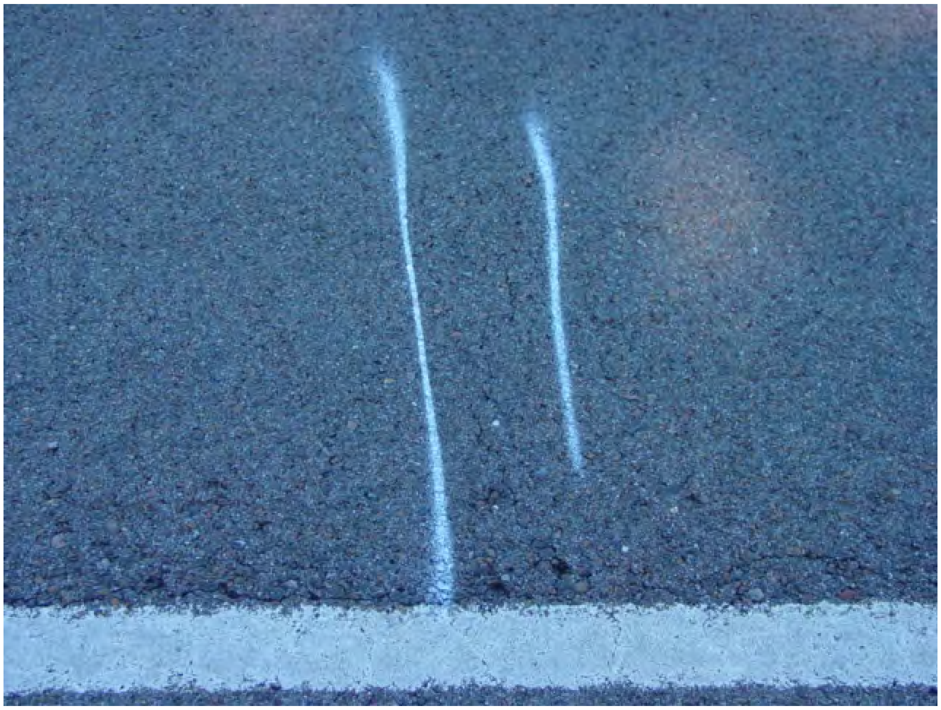

Figure A100. EBP38TC

Figure A101. EBP39TC

Figure A102. EBP40PD

Figure A103. EBP41TC

Figure A104. EBP42TC

Figure A105. EBP43TC

Figure A106. EBP44TC

Figure A107. EBP45TC

Figure A108. EBP46TC

Figure A109. EBP47TC

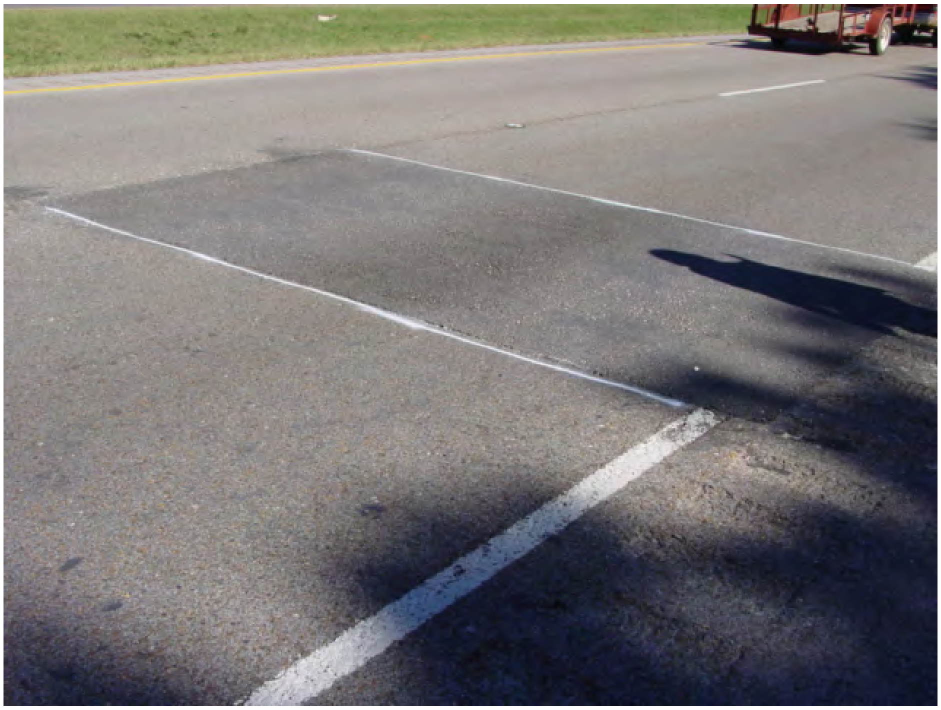

Figure A110. EBP48PD

Figure A111. EBP49TC

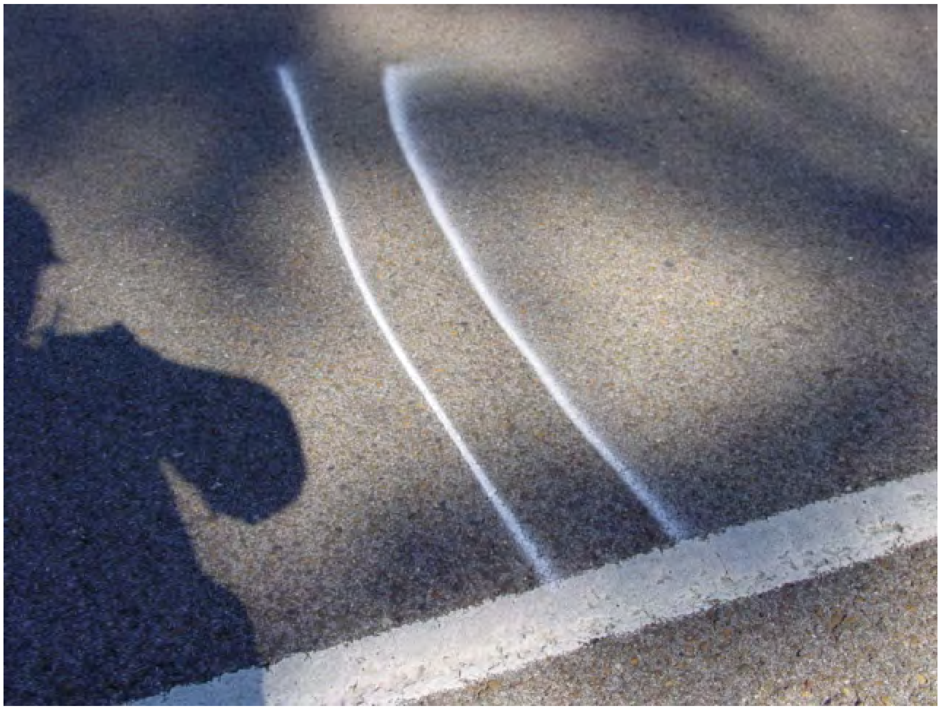

Figure A112. EBP50TC

Figure A113. EBP51TC

Figure A114. EBP52TC

Figure A115. EBP53TC

Figure A116. EBP54TC

Figure A117. EBP55TC

Figure A118. EBP56TC

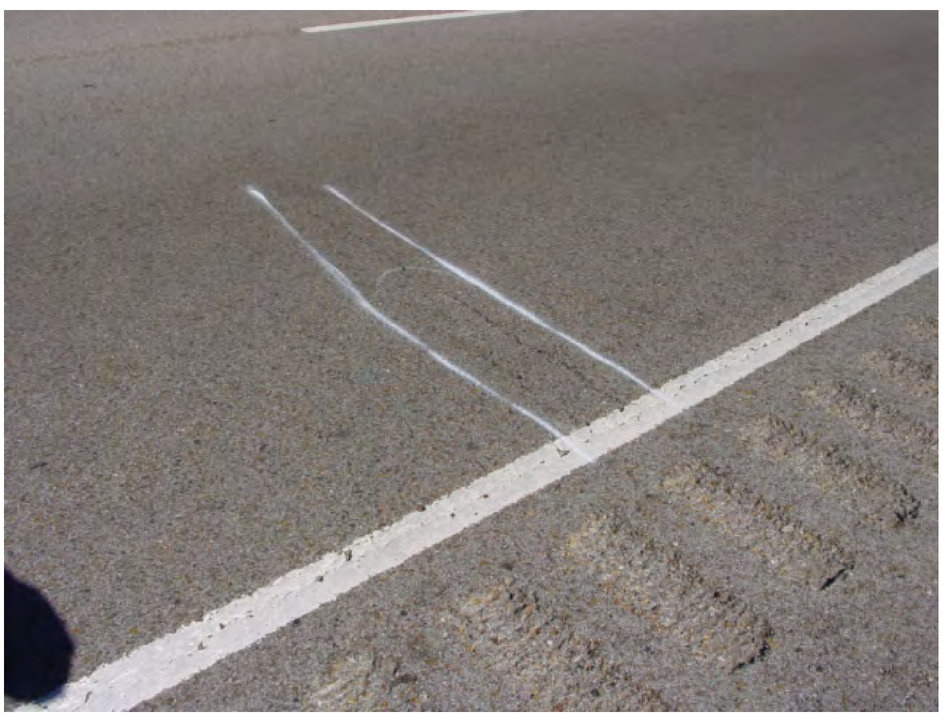

Figure A119. EBP57TC

Figure A120. EBP58TC

Figure A121. EBP59TC

Figure A122. EBP60TC

Figure A123. EBP61TC

Figure A124. EBP62TC

Figure A125. EBP63TC

Appendix B:
International Roughness Index Values

Table B.1 IRI(Right) values on westbound outside lane

Distance (mi)	Milepost	Station	IRI(Right)-Finished	IRI(Right)-Nov2003	
0.1	613.9	1326.28	70.97		
0.2	613.8	1321	81.68		
0.3	613.7	1315.72	87.82	102.45	SECTION 2
0.4	613.6	1310.44	68.36	65.85	
0.5	613.5	1305.16	89.63	75.46	
0.6	613.4	1299.88	54.9	78.66	
0.7	613.3	1294.6	74.6	92.49	
0.8	613.2	1289.32	63.24	53.98	
0.9	613.1	1284.04	73.36	59.84	
1	613	1278.76	73.36	53.25	
1.1	612.9	1273.48	57.63	62.65	SECTION 5
1.2	612.8	1268.2	56.86	49.8	
1.3	612.7	1262.92	70.13	53.61	
1.4	612.6	1257.64	60.68	58.92	
1.5	612.5	1252.36	77.94	73.95	
1.6	612.4	1247.08	72.78	83.83	
1.7	612.3	1241.8	73.48	80.82	
1.8	612.2	1236.52	75.53	70.47	
1.9	612.1	1231.3	54.84	65.47	SECTION 8
2	612	1226.02	62.95	52.68	
2.1	611.9	1220.74	65.29	59.9	
2.2	611.8	1215.46	75.44	63.86	
2.3	611.7	1210.18	59.19	99.65	
2.4	611.6	1204.9	54.83	59.15	
2.5	611.5	1199.62	51.12	71.52	
2.6	611.4	1194.34	57.14	51.32	
2.7	611.3	1189.06	96.88	95.45	
2.8	611.2	1183.78	57.37	70.32	SECTION 3
2.9	611.1	1178.5	45.92	52.69	
3	611	1173.22	63.87	37.87	
3.1	610.9	1167.94	45.87	45.47	
3.2	610.8	1162.66	54.11	39.27	
3.3	610.7	1157.38	53.27	39.96	
3.4	610.6	1152.1	58.96	54.78	
3.5	610.5	1146.82	55.87	52.63	
3.6	610.4	1141.54	49.99	51.16	
3.7	610.3	1136.26	43.69	37.91	

Table B.2 IRI(Left) values on westbound outside lane

Distance (mi)	Milepost	Station	IRI(Left)-Finished	IRI(Left)-Nov2003	
0.1	613.9	1326.28	75.35		
0.2	613.8	1321	72.55		
0.3	613.7	1315.72	74.96	86.14	SECTION 2
0.4	613.6	1310.44	55.04	70.71	
0.5	613.5	1305.16	59.3	72.9	
0.6	613.4	1299.88	54.71	71.18	
0.7	613.3	1294.6	49.65	64.41	
0.8	613.2	1289.32	48.21	45.52	
0.9	613.1	1284.04	60.95	54.16	
1	613	1278.76	59.33	53.51	
1.1	612.9	1273.48	57.51	56.89	SECTION 5
1.2	612.8	1268.2	57.34	44.51	
1.3	612.7	1262.92	62.37	53.84	
1.4	612.6	1257.64	60.54	48.81	
1.5	612.5	1252.36	59.73	61	
1.6	612.4	1247.08	52.92	68.32	
1.7	612.3	1241.8	64.73	56.59	
1.8	612.2	1236.52	69.51	59.09	
1.9	612.1	1231.3	50.02	68.61	SECTION 8
2	612	1226.02	45.08	52.07	
2.1	611.9	1220.74	48.96	48.43	
2.2	611.8	1215.46	49.41	53.98	
2.3	611.7	1210.18	49.73	75.51	
2.4	611.6	1204.9	52.39	58.3	
2.5	611.5	1199.62	45.65	54.11	
2.6	611.4	1194.34	46.27	51.1	
2.7	611.3	1189.06	82.73	70.42	
2.8	611.2	1183.78	59.38	64.57	SECTION 3
2.9	611.1	1178.5	55.7	51.43	
3	611	1173.22	54.95	45.26	
3.1	610.9	1167.94	49.03	47.78	
3.2	610.8	1162.66	54.92	46.39	
3.3	610.7	1157.38	51.85	49.17	
3.4	610.6	1152.1	51.47	47.98	
3.5	610.5	1146.82	58.11	58.86	
3.6	610.4	1141.54	48.04	54.45	
3.7	610.3	1136.26	51.16	45.54	

Table B.3 IRI(Average) values on westbound outside lane

Distance (mi)	Milepost	Station	IRI(Average)-Finished	IRI(Average)-Nov2003	
0.1	613.9	1326.28	73.16		
0.2	613.8	1321	77.115		
0.3	613.7	1315.72	81.39	94.295	SECTION 2
0.4	613.6	1310.44	61.7	68.28	
0.5	613.5	1305.16	74.465	74.18	
0.6	613.4	1299.88	54.805	74.92	
0.7	613.3	1294.6	62.125	78.45	
0.8	613.2	1289.32	55.725	49.75	
0.9	613.1	1284.04	67.155	57	
1	613	1278.76	66.345	53.38	
1.1	612.9	1273.48	57.57	59.77	SECTION 5
1.2	612.8	1268.2	57.1	47.155	
1.3	612.7	1262.92	66.25	53.725	
1.4	612.6	1257.64	60.61	53.865	
1.5	612.5	1252.36	68.835	67.475	
1.6	612.4	1247.08	62.85	76.075	
1.7	612.3	1241.8	69.105	68.705	
1.8	612.2	1236.52	72.52	64.78	
1.9	612.1	1231.3	52.43	67.04	SECTION 8
2	612	1226.02	54.015	52.375	
2.1	611.9	1220.74	57.125	54.165	
2.2	611.8	1215.46	62.425	58.92	
2.3	611.7	1210.18	54.46	87.58	
2.4	611.6	1204.9	53.61	58.725	
2.5	611.5	1199.62	48.385	62.815	
2.6	611.4	1194.34	51.705	51.21	
2.7	611.3	1189.06	89.805	82.935	
2.8	611.2	1183.78	58.375	67.445	SECTION 3
2.9	611.1	1178.5	50.81	52.06	
3	611	1173.22	59.41	41.565	
3.1	610.9	1167.94	47.45	46.625	
3.2	610.8	1162.66	54.515	42.83	
3.3	610.7	1157.38	52.56	44.565	
3.4	610.6	1152.1	55.215	51.38	
3.5	610.5	1146.82	56.99	55.745	
3.6	610.4	1141.54	49.015	52.805	
3.7	610.3	1136.26	47.425	41.725	

Table B.4 IRI(Right) values on eastbound outside lane

Distance (mi)	Milepost	Station	IRI(Right)-Finished	IRI(Right)-Nov2003	
0.10	610.10	1125.64			
0.20	610.20	1130.92			
0.30	610.30	1136.20			
0.40	610.40	1141.48	91.69	122.07	SECTION 6
0.50	610.50	1146.76	71.05	62.20	
0.60	610.60	1152.04	59.73	53.08	
0.70	610.70	1157.32	47.99	39.41	
0.80	610.80	1162.60	53.86	43.35	
0.90	610.90	1167.88	62.91	51.73	
1.00	611.00	1173.16	69.05	55.80	
1.10	611.10	1178.44	64.53	56.64	SECTION 9
1.20	611.20	1183.72	55.78	50.15	
1.30	611.30	1189.00	91.97	80.15	
1.40	611.40	1194.28	68.39	67.21	
1.50	611.50	1199.56	43.60	39.72	
1.60	611.60	1204.84	53.14	40.31	
1.70	611.70	1210.12	62.91	56.86	
1.80	611.80	1215.40	53.43	53.81	SECTION 1
1.90	611.90	1220.63	62.46	71.76	
2.00	612.00	1225.91	62.64	70.70	
2.10	612.10	1231.19	56.35	50.80	
2.20	612.20	1236.47	46.59	51.72	
2.30	612.30	1241.75	57.43	54.77	
2.40	612.40	1247.03	69.81	60.77	
2.50	612.50	1252.31	51.42	61.44	SECTION 4
2.60	612.60	1257.59	57.57	35.15	
2.70	612.70	1262.87	64.13	61.79	
2.80	612.80	1268.15	46.61	38.67	
2.90	612.90	1273.43	51.76	38.45	
3.00	613.00	1278.71	54.46	47.07	
3.10	613.10	1283.99	58.58	50.58	
3.20	613.20	1289.27	67.61	44.46	SECTION 7
3.30	613.30	1294.55	67.19	97.50	
3.40	613.40	1299.83	86.44	84.28	
3.50	613.50	1305.11	77.46	88.43	
3.60	613.60	1310.39	58.40	64.55	
3.70	613.70	1315.67	49.88	48.70	
3.80	613.80	1320.95	63.14	45.12	

Table B.5 IRI(Left) values on eastbound outside lane

Distance (mi)	Milepost	Station	IRI(Left)-Finished	IRI(Left)-Nov2003	
0.10	610.10	1125.64			
0.20	610.20	1130.92			
0.30	610.30	1136.20			
0.40	610.40	1141.48	63.82	85.66	SECTION 6
0.50	610.50	1146.76	53.91	50.74	
0.60	610.60	1152.04	57.13	48.09	
0.70	610.70	1157.32	39.48	36.26	
0.80	610.80	1162.60	59.56	45.59	
0.90	610.90	1167.88	63.66	57.02	
1.00	611.00	1173.16	65.37	52.20	
1.10	611.10	1178.44	57.97	58.13	SECTION 9
1.20	611.20	1183.72	56.05	64.66	
1.30	611.30	1189.00	91.59	73.80	
1.40	611.40	1194.28	60.77	73.39	
1.50	611.50	1199.56	45.22	49.66	
1.60	611.60	1204.84	59.56	45.32	
1.70	611.70	1210.12	59.16	54.79	
1.80	611.80	1215.40	47.26	49.75	SECTION 1
1.90	611.90	1220.63	71.49	71.66	
2.00	612.00	1225.91	61.56	69.74	
2.10	612.10	1231.19	51.03	47.75	
2.20	612.20	1236.47	43.33	49.45	
2.30	612.30	1241.75	52.85	48.75	
2.40	612.40	1247.03	64.10	51.50	
2.50	612.50	1252.31	43.84	47.28	SECTION 4
2.60	612.60	1257.59	42.14	35.02	
2.70	612.70	1262.87	50.92	60.19	
2.80	612.80	1268.15	53.52	40.93	
2.90	612.90	1273.43	56.64	41.94	
3.00	613.00	1278.71	44.90	38.59	
3.10	613.10	1283.99	64.11	48.09	
3.20	613.20	1289.27	62.45	57.98	SECTION 7
3.30	613.30	1294.55	69.95	74.07	
3.40	613.40	1299.83	64.09	66.78	
3.50	613.50	1305.11	54.70	57.69	
3.60	613.60	1310.39	45.09	51.09	
3.70	613.70	1315.67	40.05	38.37	
3.80	613.80	1320.95	51.11	34.13	

Table B.6 IRI(Average) values on eastbound outside lane

Distance (mi)	Milepost	Station	IRI(Average)-Finished	IRI(Average)-Nov2003	
0.10	610.10	1125.64			
0.20	610.20	1130.92			
0.30	610.30	1136.20			
0.40	610.40	1141.48	77.76	103.87	SECTION 6
0.50	610.50	1146.76	62.48	56.47	
0.60	610.60	1152.04	58.43	50.59	
0.70	610.70	1157.32	43.74	37.84	
0.80	610.80	1162.60	56.71	44.47	
0.90	610.90	1167.88	63.29	54.38	
1.00	611.00	1173.16	67.21	54.00	
1.10	611.10	1178.44	61.25	57.39	SECTION 9
1.20	611.20	1183.72	55.92	57.41	
1.30	611.30	1189.00	91.78	76.98	
1.40	611.40	1194.28	64.58	70.30	
1.50	611.50	1199.56	44.41	44.69	
1.60	611.60	1204.84	56.35	42.82	
1.70	611.70	1210.12	61.04	55.83	
1.80	611.80	1215.40	50.35	51.78	SECTION 1
1.90	611.90	1220.63	66.98	71.71	
2.00	612.00	1225.91	62.10	70.22	
2.10	612.10	1231.19	53.69	49.28	
2.20	612.20	1236.47	44.96	50.59	
2.30	612.30	1241.75	55.14	51.76	
2.40	612.40	1247.03	66.96	56.14	
2.50	612.50	1252.31	47.63	54.36	SECTION 4
2.60	612.60	1257.59	49.86	35.09	
2.70	612.70	1262.87	57.53	60.99	
2.80	612.80	1268.15	50.07	39.80	
2.90	612.90	1273.43	54.20	40.20	
3.00	613.00	1278.71	49.68	42.83	
3.10	613.10	1283.99	61.35	49.34	
3.20	613.20	1289.27	65.03	51.22	SECTION 7
3.30	613.30	1294.55	68.57	85.79	
3.40	613.40	1299.83	75.27	75.53	
3.50	613.50	1305.11	66.08	73.06	
3.60	613.60	1310.39	51.75	57.82	
3.70	613.70	1315.67	44.97	43.54	
3.80	613.80	1320.95	57.13	39.63	

Appendix C: Orientation of the Test Sections

Mix Design Summary (Surface)

WESTBOUND

Table C.1 Summary of test section, westbound

STATIONS	SECTION	MIX DESIGN	SY	TONS
1135 to 1188	3	SUPERPAVE ½", Quartzite Coarse Aggregate (MARTIN MARIETTA JONES MILL)	24482	2693
1193 to 1235	8	TY C, Sandstone Coarse Aggregate (MERIDIAN SAWYER)	18037	1984
1235 to 1278	5	CMHB-C, Sandstone Coarse Aggregate (MERIDIAN SAWYER)	18037	1984
1278 to 1321	2	SUPERPAVE ½", Sandstone Coarse Aggregate (MERIDIAN SAWYER)	18040	1984
		SUBTOTAL	78596	8645

EASTBOUND

Table C.2 Summary of test section, eastbound

STATION LIMITS	SECTION	MIX DESIGN	SY	TONS
1135 to 1185	6	CMHB-C, Quartize Coarse Aggregate (MARTIN MARIETTA JONES MILL)	15530	1708
1190 to 1218	9	TY C, Quartize Coarse Aggregate (MARTIN MARIETTA JONES MILL)	15197	1672
1218 to 1245	1	SUPERPAVE ½", Siliceous Gravel Coarse Aggregate (HANSON EAGLE MILLS, PRESCOTT, OR LITTLE RIVER)	15956	1755
1245 to 1282	4	CMHB-C, Siliceous Gravel Coarse Aggregate (HANSON EAGLE MILLS, PRESCOTT, OR LITTLE RIVER)	15956	1755
1282 to 1321	7	TY C, Siliceous Gravel Coarse Aggregate (HANSON EAGLE MILLS, PRESCOTT, OR LITTLE RIVER)	15958	1755
		SUBTOTAL	78597	8645
		TOTAL	157193	17290

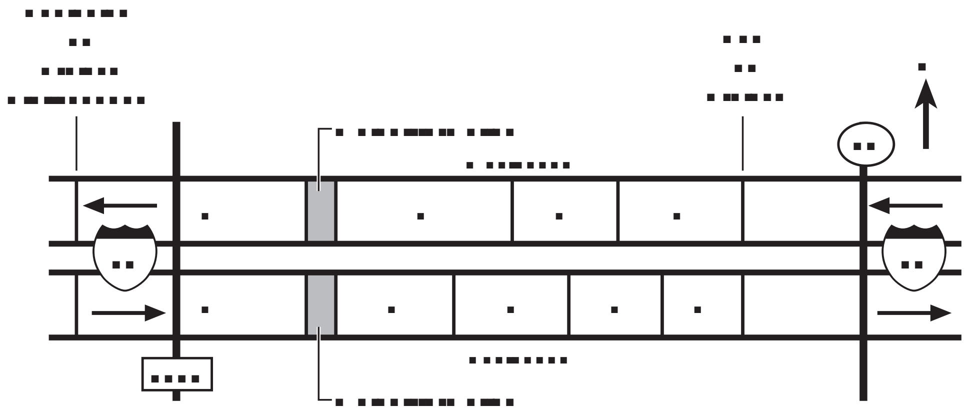

Figure C.1 Layout of the test sections