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Multidisciplinary Research in Transportation

Project Level Performance Database for Rigid Pavements in Texas, II

Sung Woo Ryu, Sureel Saraf, Moon C. Won

Texas Department of Transportation

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Over the years, the Texas Department of Transportation (TxDOT) has built a number of CRCP (continuously reinforced concrete pavement) experimental sections to investigate the effects of design, materials, and construction variables on CRCP structural responses and performance. Design variables include longitudinal reinforcement (percent steel and bar diameter) and slab thickness. Materials variables include coarse aggregate type in concrete. Construction variables include differences in concrete placement season and curing method. Field evaluations were conducted and findings include; (1) spalling occurred in sections with siliceous coarse aggregate 14 years after construction, (2) more cracking was observed at sections with a larger amount of steel, and (3) overall, the performance is excellent. Forensic evaluation conducted on IH40 in the Amarillo District shows deficient slab support caused punchout distress. Load transfer efficiency (LTE) at transverse cracks at punchout distress area was maintained at quite a high level. It appears that LTE is not a good indicator for structural condition of CRCP. Deflection testing using falling weight deflectometer (FWD) continued for Level I test sections. The findings are consistent with those in the past – LTEs are quite high regardless of crack spacing, pavement age, slab thickness and season of testing (winter vs. summer). Variability in deflections evaluated at 50-ft intervals indicates variations in slab support. TxDOT developed a mechanistic-empirical pavement design software, called TxCRCP-ME, under research project 0-5832. A critical software element that determines the accuracy of the software is a transfer function. Extensive field evaluations were conducted to obtain reasonable distress information, since not all distresses are structural-deficiency related. It was found that the majority of distresses that are classified as punchouts in the TxDOT PMIS (pavement management information system) are not distresses caused by structural deficiency. They were, rather, caused by issues related to design details, and materials and construction quality. Efforts were made to estimate traffic information for the development of a transfer function. The traffic information in TxDOT PMIS was utilized; however, there were challenges in estimating accurate traffic.</td></tr>
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Project Level Performance Database for Rigid Pavements in Texas, Phase II

Sungwoo Ryu

Sureel Saraf

Moon Won

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This report contains no products.

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CHAPTER 1 INTRODUCTION

Over the years, the Texas Department of Transportation (TxDOT) built a number of CRCP (continuously reinforced concrete pavement) experimental sections to investigate the effects of design, materials, and construction variables on CRCP structural responses and performance. Follow-up monitoring of those sections is quite important, since these sections provide rare opportunities to investigate the effects of selected variables. Design variables include longitudinal reinforcement (percent steel and bar diameter) and slab thickness. Materials variables include coarse aggregate type in concrete. Construction variables include differences in concrete placement season and curing method. Some test and experimental sections are more than 20 years old under heavy traffic. Monitoring the performance of those sections will provide invaluable information that can enhance our knowledge of CRCP behavior and performance, and ultimately will help improve CRCP design, materials, and construction practices.

As of FY 2010, TxDOT had 12,345 lane miles of CRCP. This is by far the most CRCP in the nation. Some of the sections are quite old, as old as 50 years, and show distresses in the form of punchouts. Old sections have design features that are no longer employed, such as placing concrete directly on top of subgrade and using asphalt shoulders. Understanding how CRCPs with different design features eventually fail will also advance understanding of CRCP behavior and develop better design method and construction practices.

Detailed structural evaluations of CRCP have been conducted at 27 Level I test sections in the state since 2005 using falling weight deflectometer (FWD). Evaluations include load transfer efficiency (LTE) testing at cracks with small, medium, and large crack spacing, at different ambient temperature conditions (winter vs. summer), and deflection testing at 50-ft. intervals for average deflection evaluations. This evaluation provided valuable information that could be used to estimate structural condition of CRCP, such as back-calculated subgrade modulus of reaction, or to validate closed form solution results, such as Westergaard's equations for deflections.

TxDOT developed mechanistic-empirical pavement design software, called TxCRCP-ME, under research project 0-5832. A critical software element that determines the accuracy of the software is a transfer function. To develop an accurate transfer function, both traffic and distress (punchouts per mile) information is quite important. Extensive field evaluations are needed to get reasonable distress information, since not all distresses are structural deficiency related. Field evaluations were conducted to classify punchouts, and it was found that the majority of distresses classified as punchouts in TxDOT PMIS (pavement management information system) are not distresses caused by structural deficiency. They were, rather, caused by issues related to design details, and materials and construction quality. Efforts were made to estimate traffic information for the development of a transfer function. The traffic information in TxDOT PMIS was utilized; however, there were challenges in estimating accurate traffic.

SCOPE OF THE REPORT

Chapter 2 describes findings of field evaluations for experimental sections.

Chapter 3 summarizes the forensic evaluations conducted to identify the causes of punchouts.

Chapter 4 discusses the results of LTE evaluations.

Chapter 5 presents the results of effort to improve the accuracy of a transfer function.
Chapter 6 presents conclusions and recommendations.

CHAPTER 2 EVALUATIONS OF EXPERIMENTAL SECTIONS

Over the years, TxDOT has built a number of experimental CRCP sections to investigate and enhance understanding of CRCP behavior, with the ultimate objective of improving CRCP design, materials, and construction practices for a better performing pavement system. Follow-up evaluations were conducted in this project to evaluate the structural behavior and long-term performance.

2.1 Experimental Sections Built Under TxDOT Research Project 0-1244

The objectives of project 0-1244 included the identification of coarse aggregate and steel reinforcement effects on CRCP performance. Experimental sections were constructed at four locations in the Houston District – IH 45 in Spring Creek, BW 8 frontage road and two locations on SH 6. Figure 2.1 shows the locations of experimental sections.

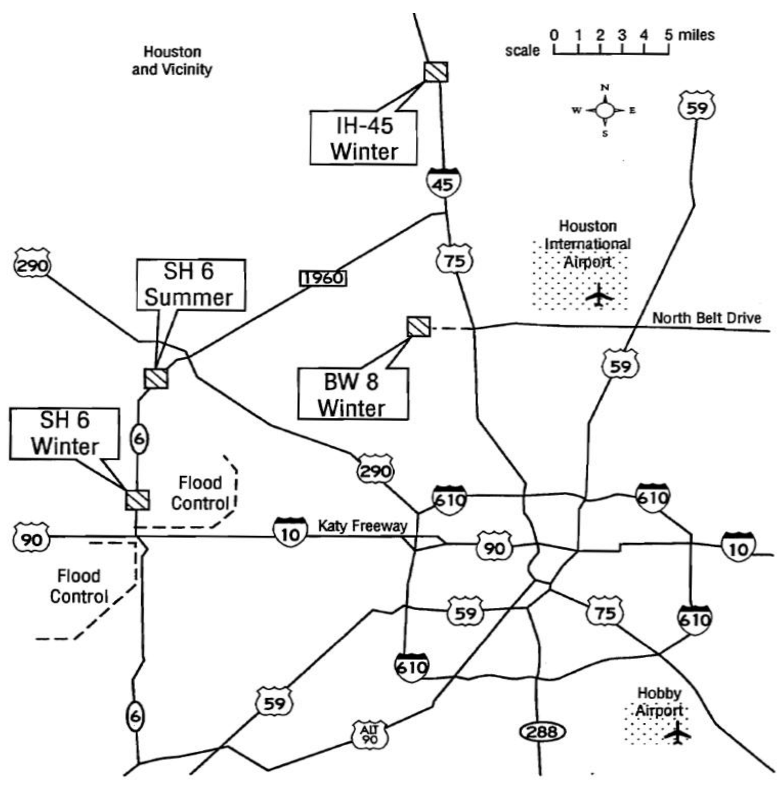

Figure 2.1 Experimental Sections in Houston Built Under Project 0-1244

Sections on SH 6 were overlaid with hot mix asphalt in the middle of the 2000s due to severe spalling problems. Roadway users complained to TxDOT of poor ride due to spalling. TxDOT decided to place the asphalt overlay. Sections in the other two locations are in good condition, except for spalling distresses on sections containing siliceous river gravel (SRG) as coarse aggregate. Sections with limestone (LS) coarse aggregate are in excellent condition, with not a single distress after more than 20 years of service. Field evaluations were conducted in the remaining two locations.

2.1.1 BW 8 Section

This section is located in the frontage road of Beltway 8 eastbound, just east of Antoine Rd. This section was placed on November 24 (SRG section) and on November 25 (LS section), 1989. Figure 2.2 shows the plan layout of the test sections. It is 10-in. CRCP with 1-in. asphalt concrete base (bond breaker) over 6-in. cement stabilized subbase.

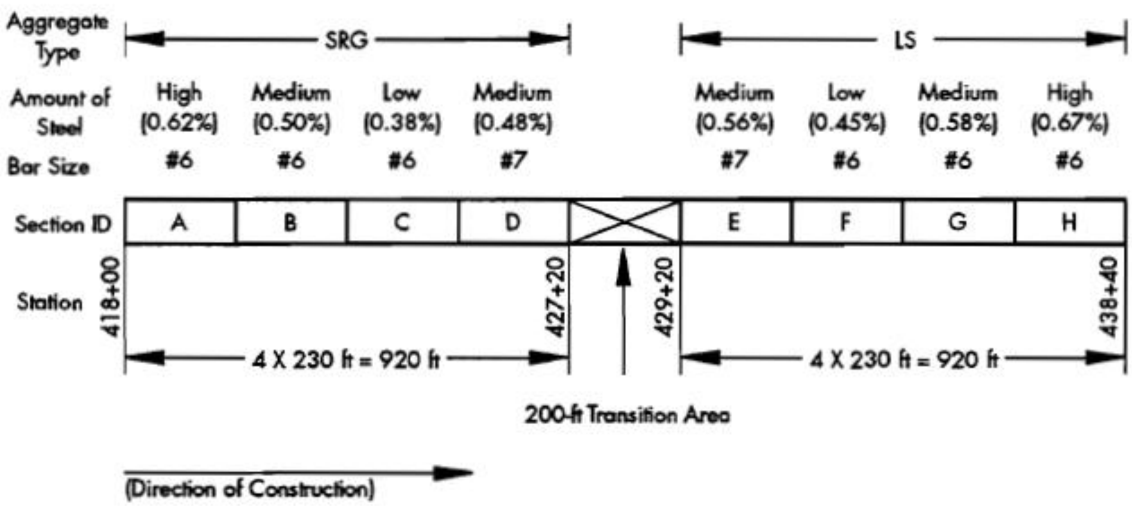

Figure 2.2 Layout of Test Sections in BW 8 Frontage Road

Two coarse aggregate types, SRG and LS, and four different reinforcement designs were used. This section provided an excellent opportunity to investigate the effects of coarse aggregate type and steel percentages. At the writing of this report, the section is 21 years old. Sufficient environmental loading (temperature and moisture variations) was applied. It is quite difficult to investigate the effects of longitudinal steel amounts on CRCP performance in typical CRCP projects, because CRCPs are constructed with a fixed steel amount per design standards, resulting in little variations in the amount of longitudinal steel among projects. Figure 2.3 illustrates crack spacing distribution for all eight sections. The findings on crack spacing are summarized as follows:

1. Crack spacing varies with the amount of longitudinal steel – the more the steel, the smaller the crack spacing, which is consistent with the findings in other studies and with theoretical analysis. The exceptions are crack spacing in Sections G and H. Section H has a larger amount of steel than Section G, but similar crack spacing is observed for both sections.
2. Coarse aggregate type doesn't appear to affect crack spacing. This is somewhat different from the findings elsewhere. Normally, concrete with LS has larger crack spacing than

concrete with SRG. On the other hand, environmental conditions during and right after construction have substantial effects on cracking development. Cracking information at early ages (up to 30 days after construction) shows no difference between the SRG and LS section for high steel and #7 bar sections, and for low steel percentage sections, the LS section had larger crack spacing than the SRG section. It appears that the trend still continues after 21 years.

Deflection testing was conducted to evaluate the effect of percent steel on overall deflections and load transfer efficiency (LTE) using falling weight deflectometer (FWD). This testing was conducted on Sections E thru H. Deflections were measured in the middle 100 feet of each section. FWD testing was conducted at the middle between two adjacent transverse cracks, upstream and downstream. Figure 2.4 shows testing results. Sections with #7 bar size are denoted in green while sections with #6 bar size are denoted in blue. Deflections here are the average of deflections at the middle between two adjacent transverse cracks.

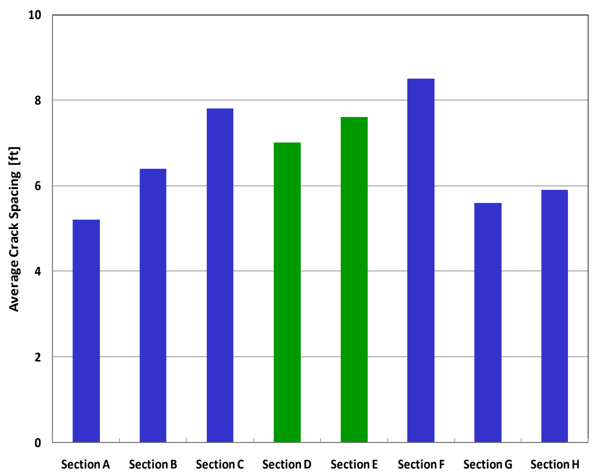

Figure 2.3 Average Crack Spacing of All Sections

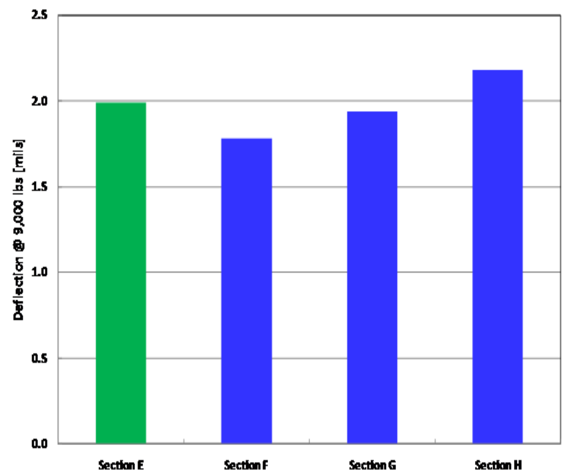

Figure 2.4 Average Mid-Crack Deflections of Each Section

Figure 2.4 shows that Section F has the lowest deflections, and Section H has the largest. Section F has the lowest steel percentage and Section H has the highest. It was anticipated that Section H would have the lowest deflections, since a larger amount of steel will keep the cracks tighter and reduce deflections even at the mid-point between two transverse cracks. The testing results do not support this idea. Figure 2.5 shows crack spacing and deflection relations for the four sections. Deflections here are those at mid-point between two transverse cracks. It shows that deflections at four locations on Section H (largest steel percentage) are particularly high, which led to the highest average deflections as shown in Figure 2.4. Without those four points, deflections could be fairly comparable in all sections. There is no clear trend between crack spacing and deflections, even though it appears that deflections decrease somewhat with increased crack spacing. Considering the precision of FWD deflections and the variability in slab thickness and subbase support, the effect of crack spacing on deflections is negligible.

Figures 2.6 and 2.7 show deflections in Sections G and H, respectively. Black squares represent deflections at the mid-point between cracks, and red diamonds represent deflections at upstream and downstream of cracks. Figure 2.6 shows that deflections at mid-point between cracks are lower than those at nearby cracks. On the other hand, Figure 2.7 shows that deflections at the mid-point between cracks register in between the deflections at nearby cracks. This might indicate that the support condition or slab thickness in Section H is not uniform. According to Westergaard equations, slab thickness has a substantial effect on slab deflections, and unless slabs are constructed with quite uniform thickness, the effects of steel percentages on slab deflections would be quite difficult to evaluate, as shown here.

The effect of steel percentages on CRCP performance is an important topic; however, there is not much information available on the topic. One of the reasons is that most of the CRCPs are built with an adequate amount of steel, and it's rare to find CRCP sections with much lower or much higher than normal amounts of steel. Since sections in this location have quite low steel percentages, it was hoped that this section would provide valuable information on steel percentage and performance.

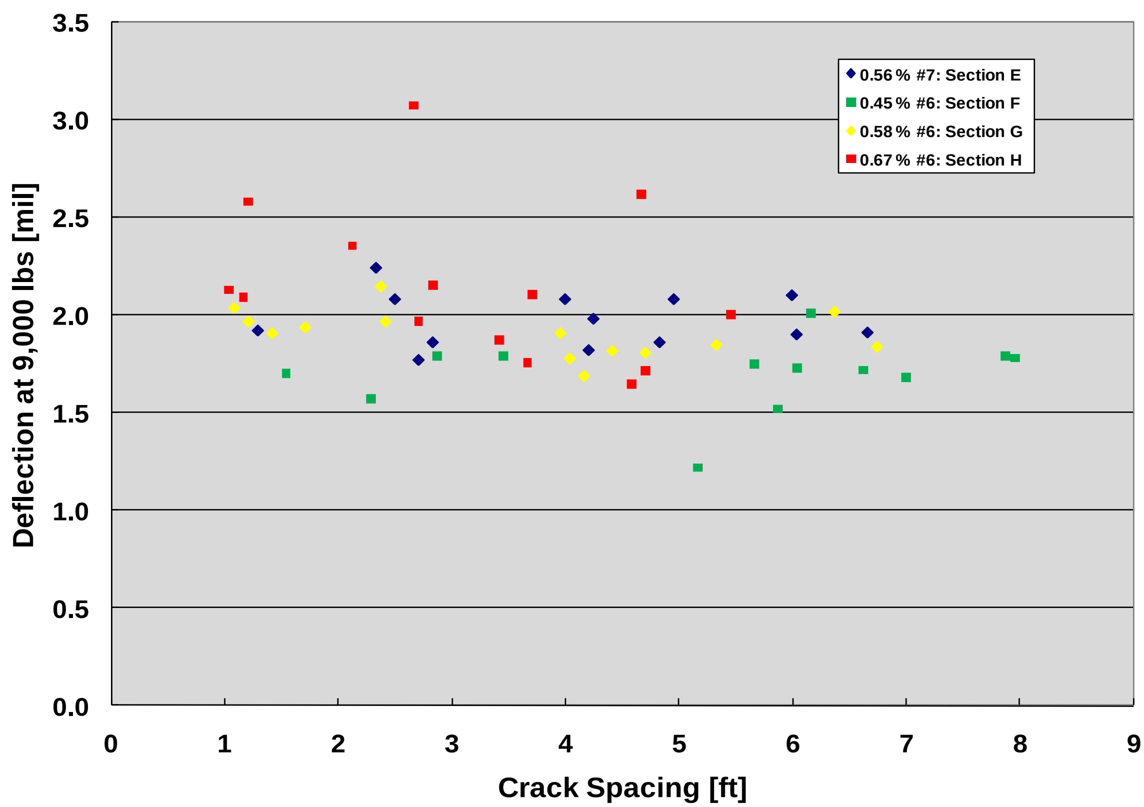

Figure 2.5 Crack Spacing and Deflections for Various Steel Percentages

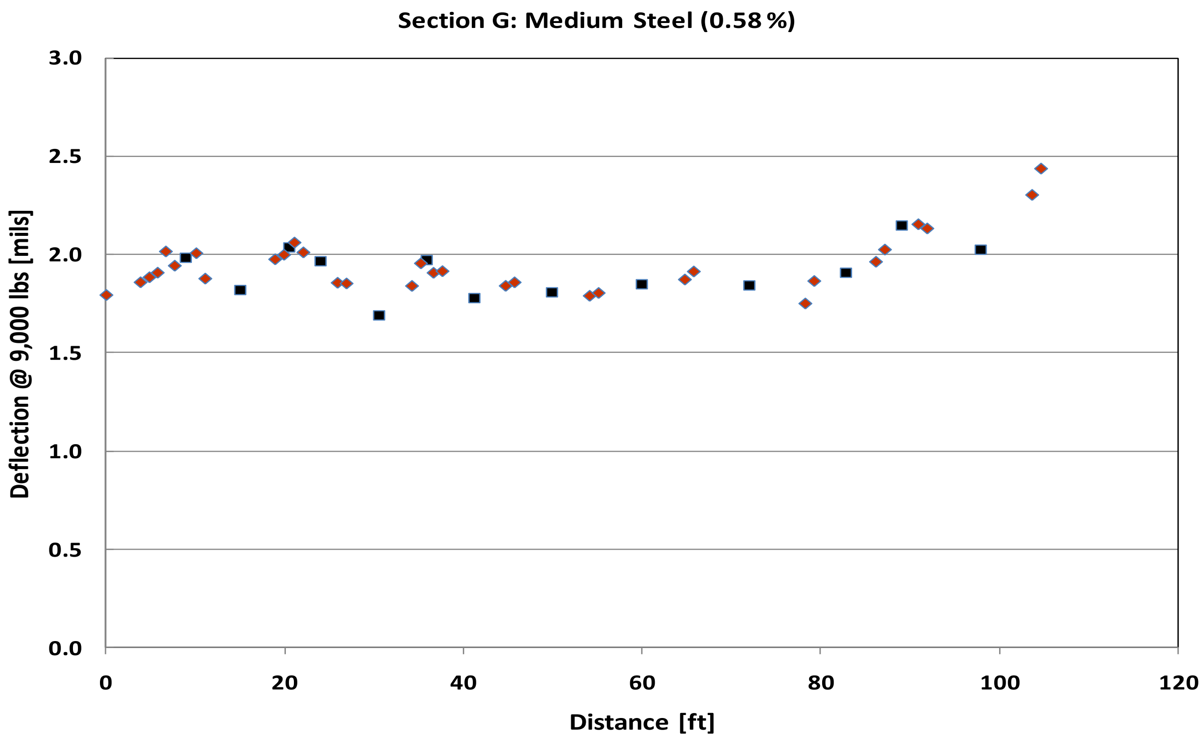

Figure 2.6 Deflections at Section G

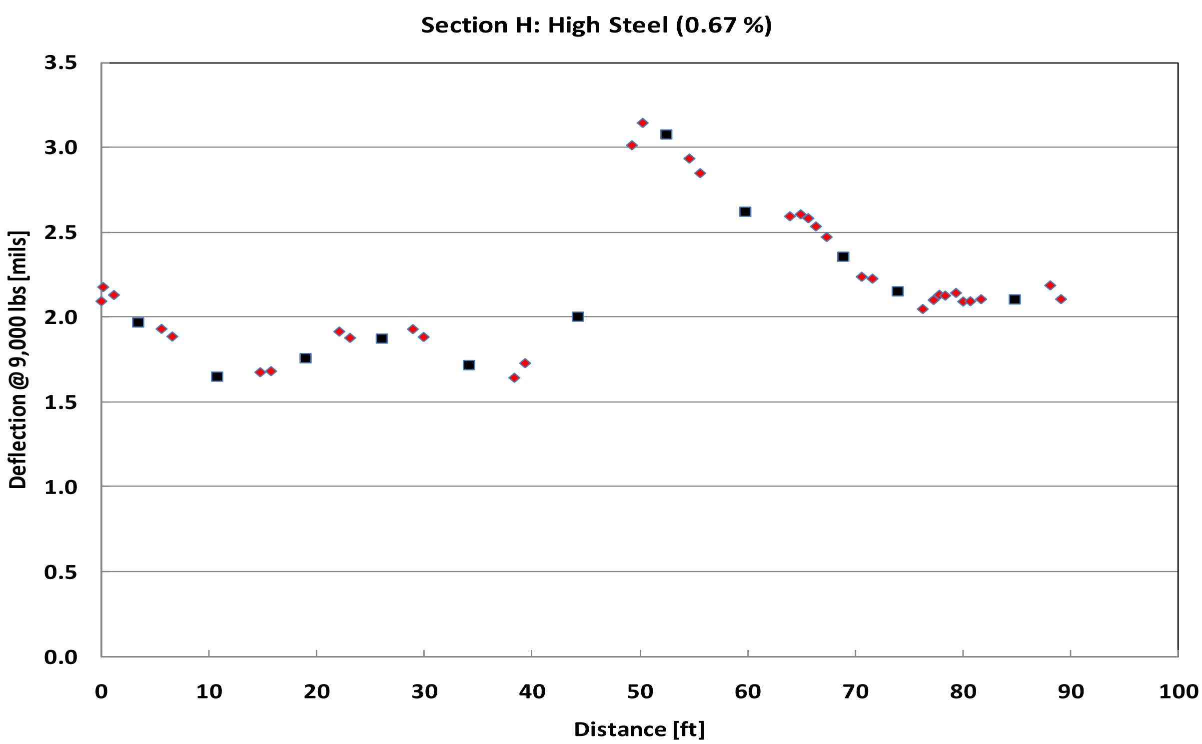

Figure 2.7 Deflections at Section H

There was a large difference in the surface defects between sections containing SRG and those containing LS. There was no distress, either punchout or spalling, in sections with LS coarse aggregate. On the other hand, there were numerous spalling and spalling repairs in the SRG sections. Figure 2.8 shows deep spalling in the SRG section. Figure 2.9 shows a close-up view of spall repairs. The number of spalling and spall repairs are: 17 in Section A, 22 in Section B, 34 in Section C, and 19 in Section D. Section A had the highest steel percentage, and Section C had the lowest steel percentage among SRG sections. This finding strongly indicates that there is a good correlation between steel percentage and spalling potential in concrete with spalling-susceptible coarse aggregate.

Figure 2.8 Deep Spalling in SRG Section

Figure 2.9 Close-Up View of Spall Repairs

It appears that a larger amount of steel restrains concrete movement at transverse cracks, thus reducing spalling occurrence. On the other hand, a lower steel amount does not restrain concrete volume changes well, and larger concrete displacements at cracks eventually lead to spalling. This finding is supported by the fact that spalling in this section did not take place at early ages; it took a long while before deep spalling took place. It appears that large concrete displacements at cracks in sections with a low steel percentage accumulate fatigue damage due to temperature and moisture variations, and eventually with traffic wheel loading applications, resulting in spalling. Observation of spalling in this section indicates the problem has been getting worse over the years. This section will be monitored periodically to document the spalling progress. The finding in this section suggests that a larger amount of steel may be needed for concrete with SRG as a coarse aggregate type. One of the distinctive properties of concrete containing SRG in the Houston area is a high coefficient of thermal expansion (CoTE). The potential issue would be that, with a larger steel percentage for concrete with high CoTE, there will be more transverse cracks. From a theoretical standpoint, short crack spacing shouldn't be a problem. This finding does not indicate that concrete with a high CoTE can be used safely with a larger steel amount. There were 17 spall repairs or spalls in Section A, where a high steel percentage was used. There was no spalling in LS section. As for the performance in terms of punchouts, there was no punchout in either SRG or LS sections after more than 20 years.

2.1.2 IH 45 Section

This section is located in the main lanes of IH 45 northbound, just north of Spring Creek. The test sections are the inside two lanes. This section was placed on January 14 (SRG section) and on January 21 (LS section), 1990. Figure 2.10 shows the layout of the test sections. It is 15-in. CRCP with 1-in. asphalt concrete base (bond breaker) over 6-in. cement stabilized subbase. When the design of the experimental sections was developed, TxDOT was considering the use of less steel in concrete with high CoTE. The idea was that, by doing so, comparable average crack spacing would be achieved and CRCP sections with comparable average crack spacing would provide approximately the same performance.

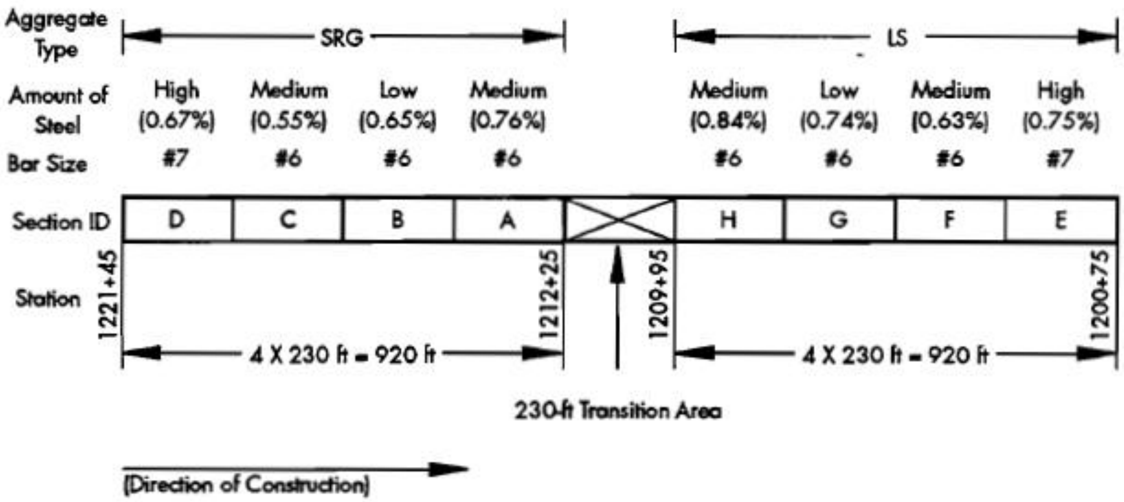

Figure 2.10 Layout of Test Sections in IH 45 in Spring Creek

Figure 2.11 shows the crack spacing distribution for the eight sections. It shows that the effect of steel percentage on crack spacing is not consistent. Also, there is no difference in crack spacing between sections with SRG and LS, as in the BW 8 section discussed earlier. It is noted that, even though the slab is 15-in., crack spacings are rather small. This indicates that cracks in this section are primarily due to temperature and moisture variations, because the slab is 15-in. thick and concrete stresses due to wheel loading are quite small.

Deflection testing was conducted every 50 ft. for both SRG and LS sections. Figure 2.12 shows that the deflections are quite small, and there is little difference in deflections between the SRG and LS sections. The average deflections at 9,000 lb are 1.01 mils and 0.99 mils for SRG and LS sections, respectively. The coefficients of variations (CV) of deflections were 10.8 percent and 12.3 percent for SRG and LS sections, respectively. The deflection testing started at the beginning of the test sections for each coarse aggregate type from the SRG section. The effects of steel percentages on deflections are evident. In the SRG sections, there is little variation in deflections throughout the four test sections. In the LS sections, the lowest deflections were observed at sections with medium (#6) bar and low steel percentages. According to Westergaard equations, slab thickness has a substantial effect on slab deflections. Also, as shown in Figure 2.6, small variations are observed in deflections at cracks and mid-slabs. The variability in deflections here might be due to the relative locations of the FWD loading from transverse cracks or variability in slab thickness. The sections with #6 bar are denoted in blue and #7 bar sections are denoted in green.

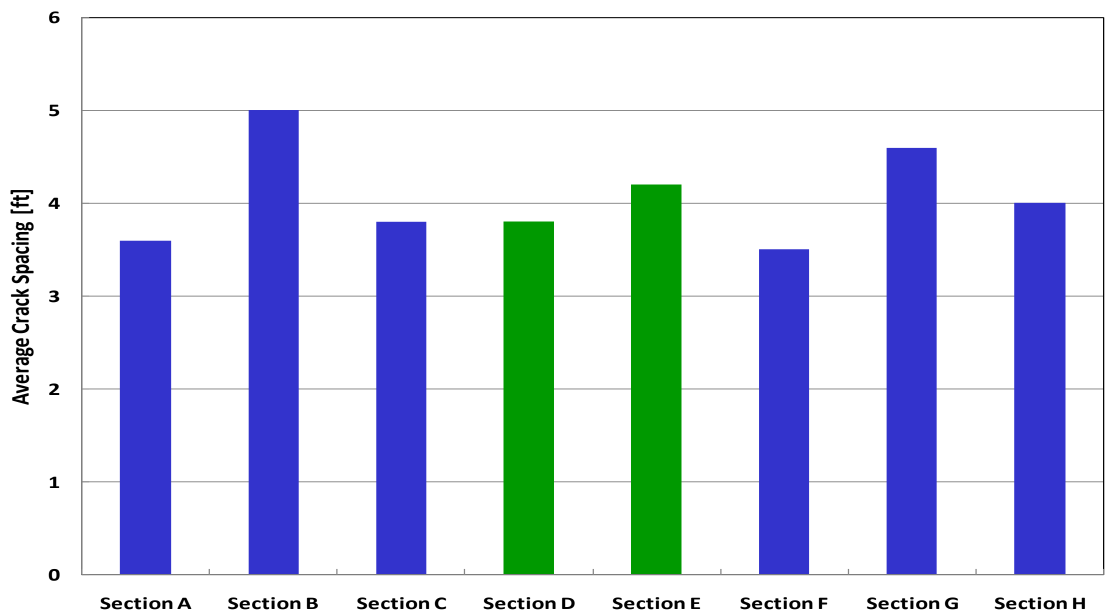

Figure 2.11 Average Crack Spacing in Various Sections

There was no punchout in the sections studied. The only distress observed was spalling in the SRG sections. There was no spalling observed in this section until 2004, when the director of construction at the Houston District first observed spalling 14 years after construction. Since then, more spalling occurred and spalling conditions worsened. Figure 2.13 shows the spalling repairs. The spalling in the SRG sections is solely due to a materials issue, or possibly contributed by construction quality. However, since the same construction crew built both the LS and SRG test sections and there is no spalling in the LS sections, it is most likely that the spalling is due to coarse aggregate used in the SRG section. As shown in Figure 2.12, deflections are small, and the spalling is not due to excessive deflections. Crack spacing is relatively small, as shown in Figure 2.11, and the spalling is not due to large crack spacing. The number of spalling at sections with various steel percentages was not quantified, even though it appeared that spalling was uniformly distributed throughout the sections. The information will be collected in the next round of field evaluations and included in the final report.

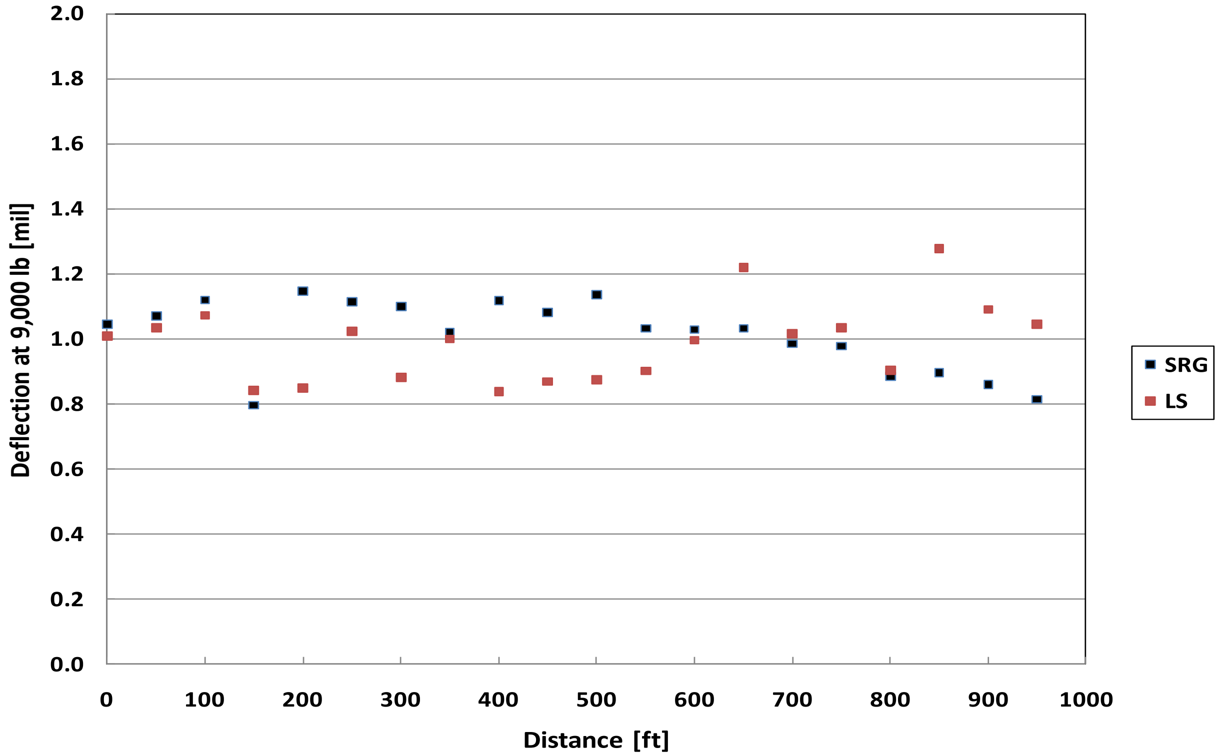

Figure 2.12 Deflection at SRG and LS Sections

Figure 2.13 Repaired Spalls in SRG Section

2.2 Experimental Sections Built Under TxDOT Research Project 0-1700

One of the objectives of project 0-1700 included the identification of temperature effects on CRCP performance. Experimental sections were constructed at five locations in the state – Austin, Cleveland, Houston, Baytown and El Paso. Figure 2.14 shows the locations, along with slab thicknesses and construction dates.

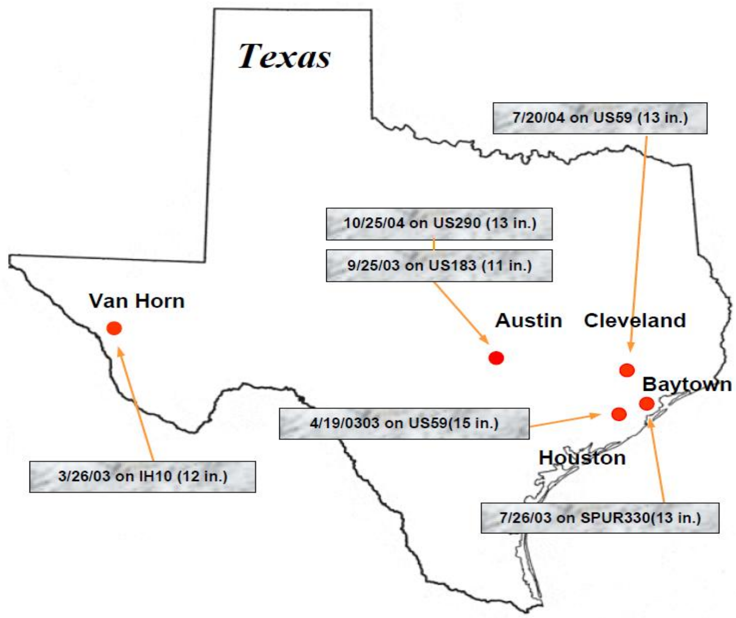

Figure 2.14 Locations of Experimental Section under Project 0-1700

2.2.1Experimental Sections in Houston Area

Field evaluations were made in three sections constructed in the Houston area –Spur 330 in Baytown, US 59 in Houston, and US 59 in Cleveland. The Baytown section is located at the inside main lane of the 700 ft. long test section on SPUR 330 between Baker Rd. and Little Rd. The Houston test section is located between Bissonnet St. and S. Gessner Dr. on US 59 at the outside shoulder of the north and south bound lanes. The Cleveland test section is 3.668 miles long on US 59 at the outside shoulder and extends from FM 2090 south to Fosteria Road.

Concrete mix designs and pavement thickness for each of the three sections are listed in Table 2.1. In all sections, crushed limestone aggregate was used as a coarse aggregate.

Table 2.1 Mix Design for Test Sections in Houston, Baytown, and Cleveland

	US 59 - Houston	SPUR 330 - Baytown	US 59 - Cleveland
Pavement Thickness (in.)	15	13	13
Cement (lbs/yd^3)	406	362	362
Fly Ash (lbs/yd^3)	135	129	131
Coarse Aggregate (lbs/yd^3)	1836	1695	1848
Fine aggregate (lbs/yd^3)	1277	1413	1265
Air Content (%)	5	5	5
Water (lbs/yd^3)	198	220	215
SCM Type (Replacement Rate)	Class C (25%)	Class C (30%)	Class F (30%)
Cement Type	Type I	Type I/II	Type I/II
Coarse Aggregate Type	Limestone	Limestone	Limestone

The construction dates for each section were: Houston section on April 19, 2003, Baytown section on July 26, 2003, and Cleveland section on July 20, 2004. Concrete temperatures for all three sections were measured using i-buttons. I-buttons were installed at various longitudinal and transverse locations of the pavement as well as various depths of the pavement to comprehensively identify the temperature patterns in the concrete pavement. Air temperatures were also measured using i-buttons. The temperatures were collected every 30 minutes during at least the first 72 hours after concrete placement. After 72 hours, the temperatures were collected every 2 hours. Detailed temperature information is available elsewhere (Nam, 2005) and not included in this report.

Figure 2.15 shows the average crack spacing observed in 2003 for the Baytown and Cleveland sections versus the age. The average crack spacing patterns are very similar; cracking developed at a rapid rate within 3 weeks after construction, and the rate slowed down significantly afterwards.

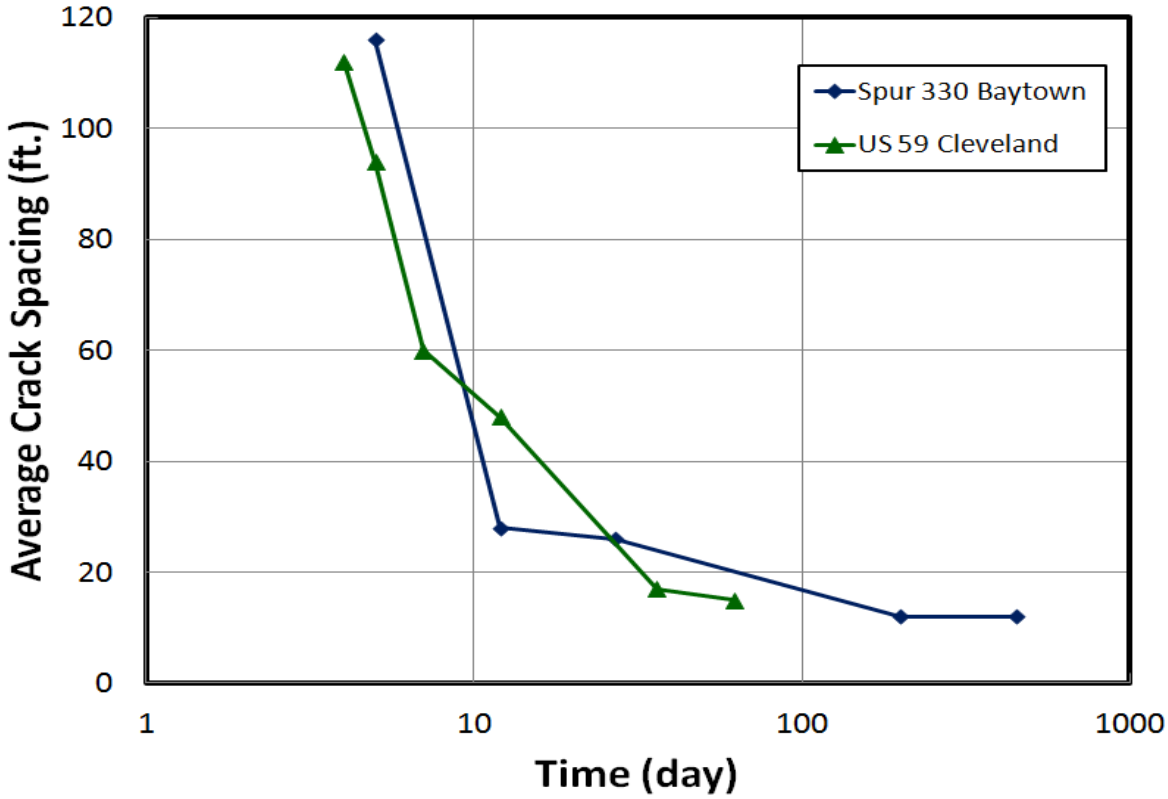

Figure 2.15 Average Crack Spacing Variations in Baytown and Cleveland (Nam, 2005)

Crack spacing was evaluated in 2010 for the Baytown and Cleveland sections, but not for the Houston section. It was quite difficult to measure crack spacing in the Houston section due to the heavy traffic. Figure 2.16 shows crack spacing distribution of the Cleveland section. The average crack spacing was 6.3 ft., with a standard deviation of 3.0 ft. Compared to the information in Figure 2.15, there were additional transverse cracks developed after 4 months after construction. About 23 percent of cracks had more than 8 ft. crack spacing; still, no spalling was observed after six years of service. The idea that large crack spacing could cause spalling was not verified in this project. Recall that the coarse aggregate type used in this project was limestone. No punchouts or other distresses were observed.

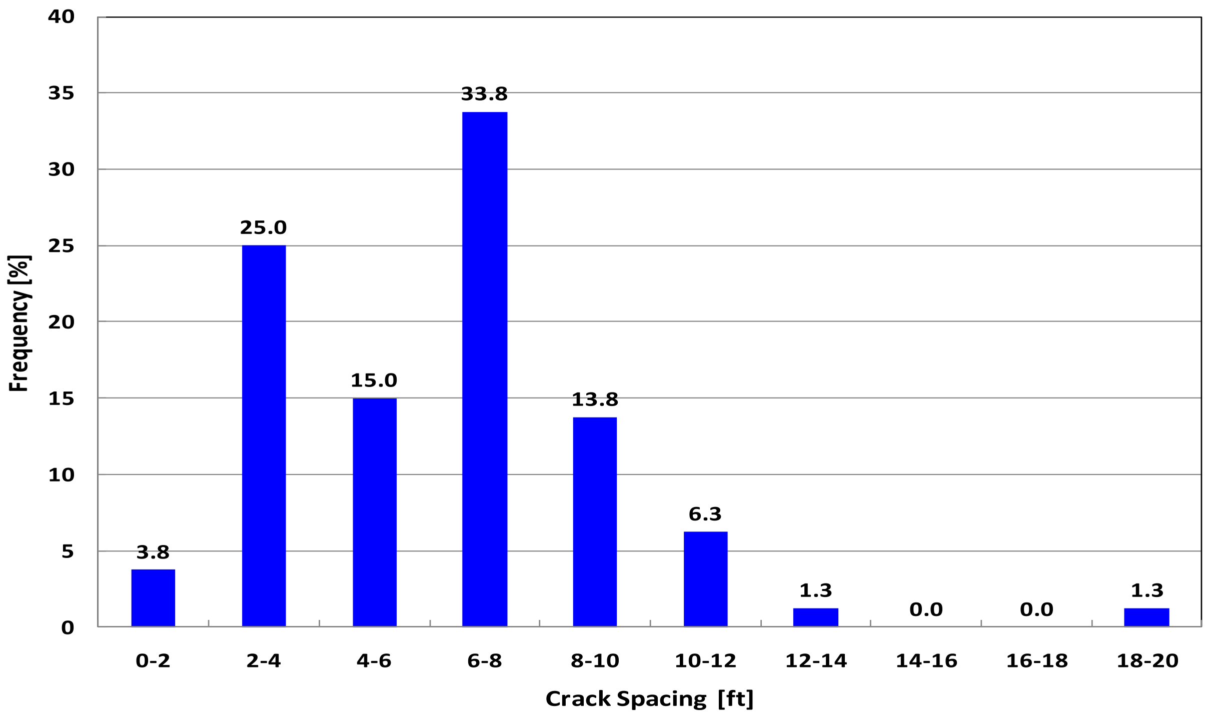

Figure 2.16 Crack Spacing Distribution in Cleveland section in 2010

Figure 2.17 shows the crack spacing distribution of the Baytown section. The average crack spacing was 10.0 ft., with a standard deviation of 4.5 ft. Compared to the information in Figure 2.15, there were additional transverse cracks developed after three months of construction. Sixty-four percent of cracks were more than 8 ft. crack spacing; still, no spalling was observed after seven years of service. Again, the idea that large crack spacing could cause spalling was not verified in this project. In Texas, most of the spalling problems occur when coarse aggregates with a high CoTE are used. Coarse aggregates with high CoTE might have other properties that make them more prone to spalling. Identifying those other properties is not an easy task. Over the years, TxDOT has sponsored a number of research projects to improve CRCP performance when spalling-prone aggregates are used. So far, no solution has been found. In this project, no punchouts or other distresses were observed.

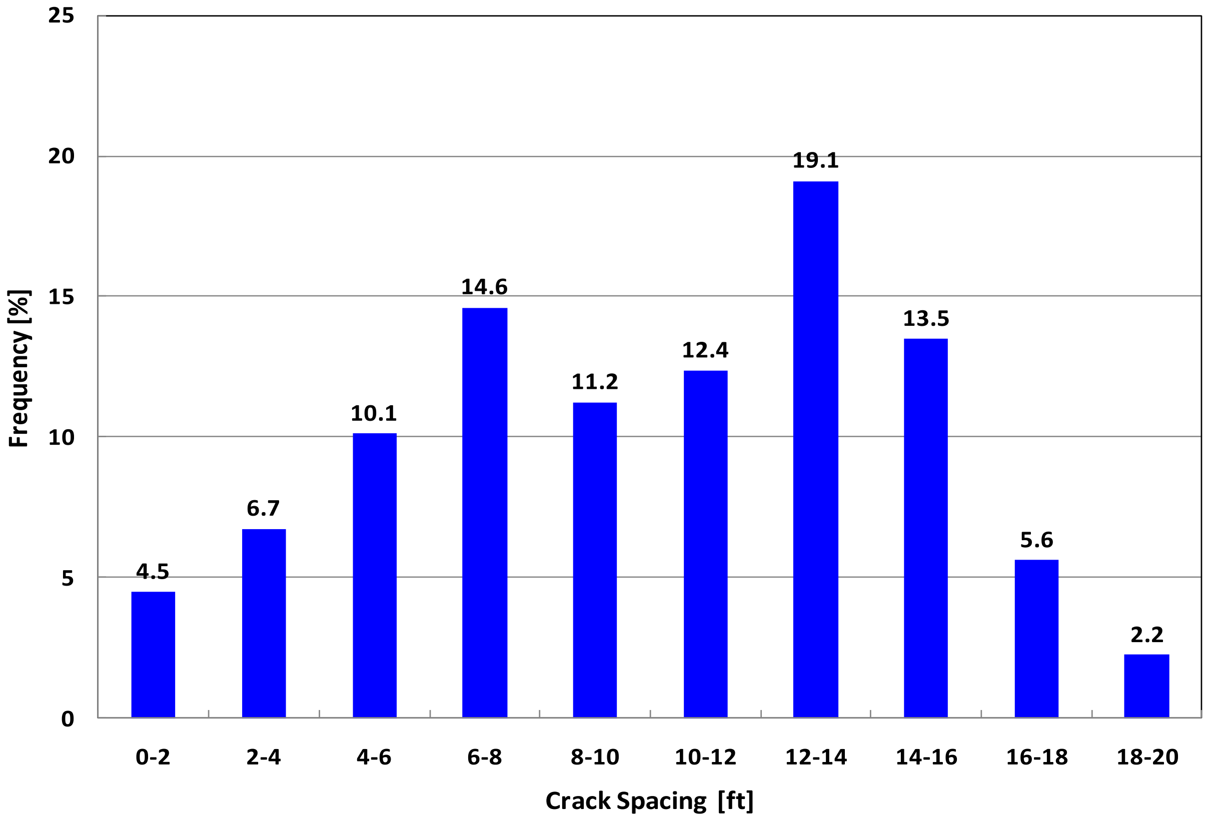

Figure 2.17 Crack Spacing Distribution in Baytown Section in 2010

2.3 Summary

The findings described in this chapter provide valuable information on the effect of coarse aggregate type and longitudinal steel amount on the behavior and performance CRCP. The value of the experimental sections with various amounts of longitudinal steel is quite substantial, since most of the CRCP sections in place in actual highways nationwide have a narrow range in longitudinal steel amount. Findings can be summarized as follows:

1) There is no clear trend between crack spacing and deflections, which indicates the efficiency of longitudinal steel in providing the continuity of slabs at transverse cracks.
2) For concrete with spalling susceptible coarse aggregate, there is a strong correlation between steel percentage and spalling potential. The larger the steel amount, the lower the frequency of spalling. However, this does not imply that TxDOT can utilize spalling-susceptible coarse aggregate in CRCP with a larger amount of longitudinal steel. The frequency of spalling in CRCP with spalling-prone coarse aggregate and larger longitudinal steel is still much higher than that in CRCP with less spalling-prone coarse aggregate.
3) For concrete with coarse aggregate with lower spalling potential, the effect of transverse crack spacing or longitudinal steel amount on spalling is non-existent.

CHAPTER 3 FORENSIC EVALUATION OF CRCP DISTRESS ON IH 40

Punchout is the structural distress in CRCP, and the frequency of real punchout – caused by structural deficiency of CRCP – in Texas is quite low. Evaluations of punchout and understanding what caused the punchout distress will provide valuable information that can be used to improve CRCP designs, material selection, and construction practices. Forensic evaluations were conducted to identify the causes and mechanisms of punchouts on IH 40 in the Amarillo District.

3.1 IH 40 Section in the Amarillo District

A section of CRCP from 2 miles west of Groom to 2 miles of east of Groom was completed in 1979. It is a 9-in. CRCP slab on 600 lb/sy. of asphalt stabilized base over lime treated subgrade. Prime coat was applied on top of lime treated subgrade. Longitudinal steel of #6 bar was placed at approximately 8-in. spacing at the mid-depth, which was 0.61 percent steel. Transverse steel and tie bars were placed at 3-ft. spacing. The size of tie bars and transverse steel was #4. There are two lanes in each direction, with a 4-ft. wide inside shoulder and 10-ft. outside shoulder. Traffic projection from 2010 to 2030 is estimated at about 27 million ESALs, which indicates that the truck traffic on this section is not high. The average rainfall is about 19.7 inches per year, and average monthly high and low temperatures are 91 ºF in July and average monthly low temperature is 23 ºF in January.

Figure 3.1 Distress on IH 40 in Amarillo District

Figure 3.1 shows a typical distress type observed in the CRCP section. In areas where punchouts occurred, faulting was observed at longitudinal joints. Faulting varied from almost negligible to a half inch. Lane separations were also observed at longitudinal joints, whether there were longitudinal construction joints or warping joints. Most of the distresses observed were located along longitudinal joints where lane separations and/or faulting were observed. From visual observations, it was clear that poor load transfer at longitudinal joints and poor subbase support were related to the distresses.

Distresses were observed in several locations and the area office staff repaired the distresses. The distress shown in Figure 3.1 was repaired on May 4, 2010. Repair work was performed by the area office staff and forensic evaluations were conducted during the repair. Before the repair work began, deflection testing using falling weight deflectometer (FWD) was conducted. Figure 3.2 shows the locations of FWD testing. IL stands for inside lane, OL for outside lane and OS for outside shoulder. L1 through L5 indicate longitudinal line number for FWD testing. Most of the testing was done in the outside lane, with four locations in the outside shoulder.

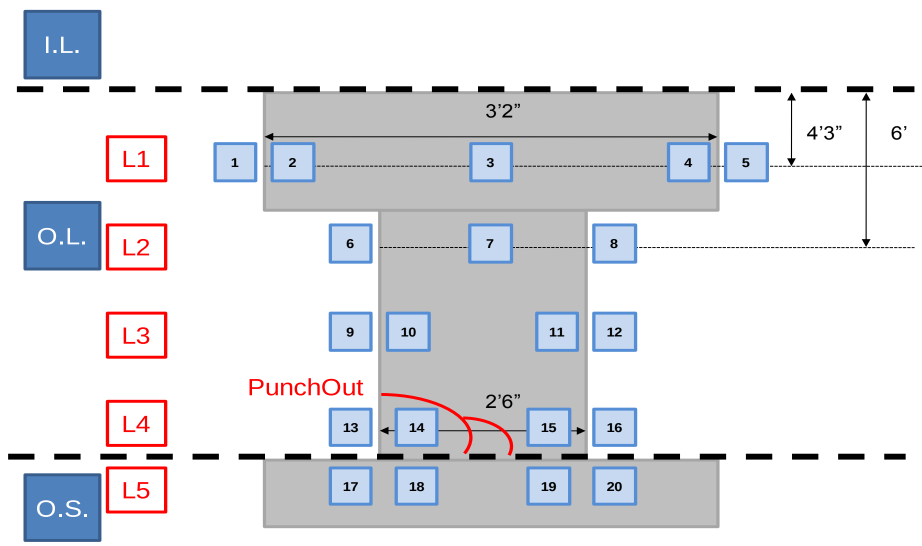

Figure 3.2 FWD Testing Location

The deflections along L1 are shown in Figure 3.3. Deflection bowls are illustrated at four locations: upstream at transverse crack between loading locations #1 and #2 in Figure 3.2 (loading location #1), downstream at the same crack (loading location #2), upstream at transverse crack between loading locations #4 and #5 (loading location #4),and downstream at the same crack (loading location #5). The notation for slab is that Slab 1 is the CRCP on the left side of the left transverse crack with asphalt sealing in Figure 3.1, Slab 2 is the portion of concrete within two closely spaced transverse cracks, which includes punchouts, and Slab 3 is the CRCP on the right side of the right transverse crack with asphalt sealing. There were asphalt patches in this area, and efforts were made to place the loading plate and all sensors away from asphalt patches. Figure 3.3 shows that the deflections at 9,000 lb are about 9 mils. The deflections are quite large, considering the average deflection for 9-in. CRCP sections in Texas (according to the rigid pavement database) is about 3 mils. This large deflection is an indication of ruptures of either longitudinal steel or tie bars. In this testing, sensor #4 was placed on the other side of the loading plate for LTE evaluations. LTE at both transverse cracks were over 90 percent, even though there was a distress. As will be discussed later, high LTE values at transverse cracks that are undergoing punchout distress question the value of LTE in CRCP evaluations. Maximum deflections with respect to the statewide average deflections for the same slab thickness might be a better indicator of the structural condition of CRCP.

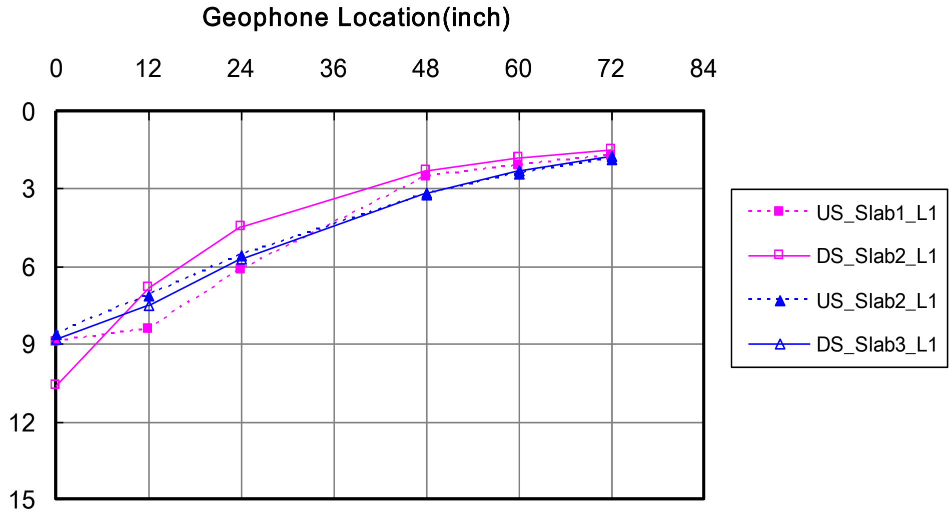

Figure 3.3 Deflections along L1

Figure 3.4 illustrates deflections along L4, which is at the longitudinal warping joint. Large deflection of about 30 mils is shown in the loading location #16 (see Figure 3.2). This large deflection is possible only if there is a steel rupture, whether it's longitudinal steel and transverse steel, and voids or weak subbase are present under the slab. The deflection at loading location #14 (DS_Slab2_L4 in Figure 3.4) is comparable to those of loading locations #13 and #15, even though the loading was applied on the slab segment isolated by cracks (Figure 3.1). This indicates that cracks themselves in CRCP, whether they are longitudinal or transverse, do not necessarily cause large deflections or distresses.

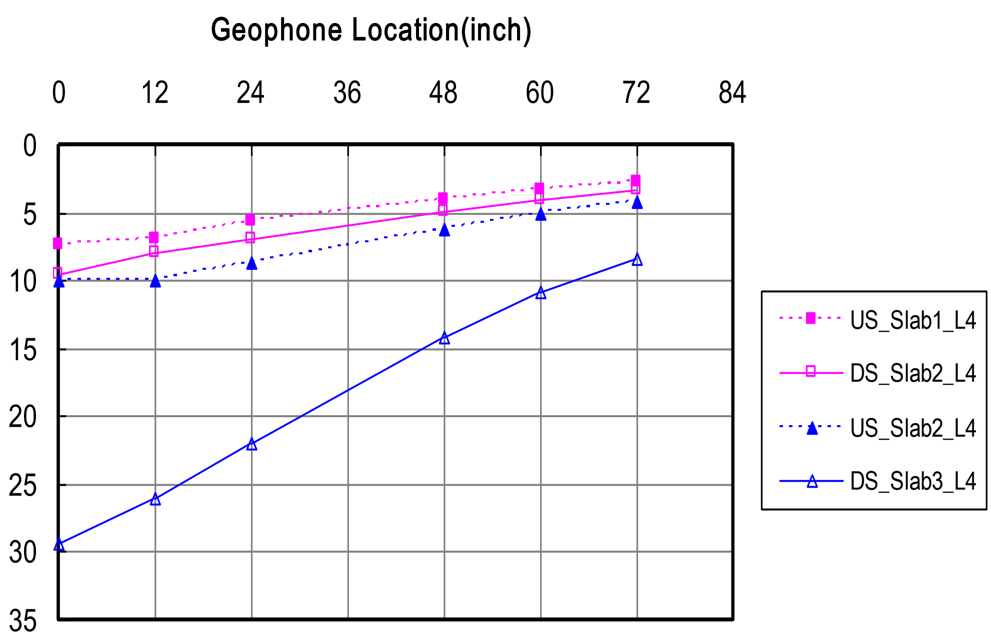

Figure 3.4 Deflections along L4

It's quite interesting to note that even though the deflection at loading location #16 was quite high, that at loading location #15 was much smaller. As will be discussed later, both longitudinal and transverse steel were broken. The large difference between deflections in loading locations #15 and #16 indicates that there was a void under the slab at loading location #16, while no void existed under loading location #15. The direction of traffic is from left to right in Figure 3.2. In jointed plain concrete pavement (JCP), at transverse contraction joints with faulting, leave ends of the slabs are always depressed and approach ends of the slabs are always lifted. Figure 3.5 shows a typical faulting. When a vehicle is approaching a joint and there is water in the subbase material, it pushes materials under the slab in the approach side of the joint into and under the slab at the other side of the joint. When a vehicle passes the joint, it pushes the materials under the slab back to the other side of the joint, causing voids under the leave side of the slab at the joint. It appears that the same mechanism was at work at a transverse crack between load locations #15 and #16 in Figure 3.2.

Figure 3.5 Severe Faulting Observed on US 287 in Quanah

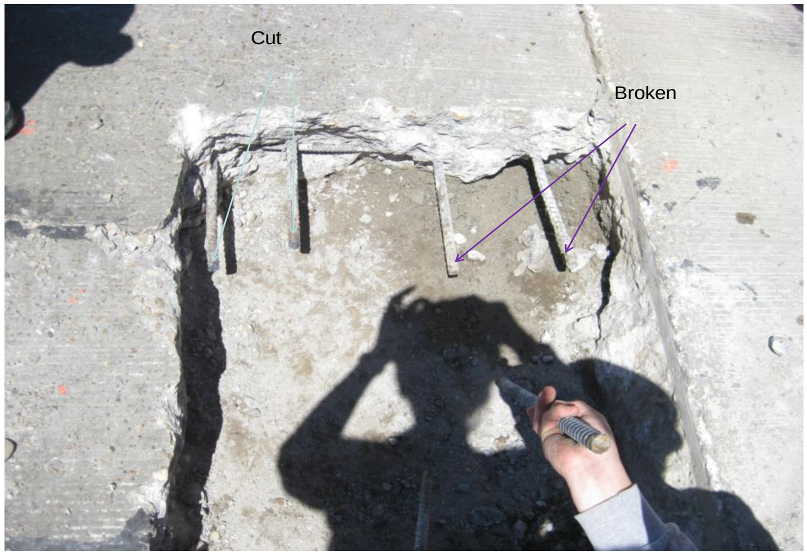

Figure 3.6 Fractured Longitudinal Steel at Punchout Location

Concrete was removed and it was observed that both longitudinal and transverse steel were fractured as shown in Figure 3.6. Figure 3.6 shows that the warping joint did not crack under the joint saw-cut; however, it appears that at the loading location #16, full-depth crack occurred at the warping joint. Water was observed in the leave side of the joint. Since the concrete was removed by a jackhammer and manually, not by saw-cutting, no water was used in the concrete removal operation. The water observed had apparently been there for a while, since weather information shows that it had not rained for several months prior to the pavement repair. Due to the way concrete was removed, it was not possible to observe whether there was a void under the slab. Based on the FWD deflections and the faulting mechanism in JCP, it was concluded that there was a void in loading location #16 or the support condition was much weaker. Westergaard equation for corner condition shows that the deflection for 9-in. slab with modulus of subgrade reaction of 300 psi/in would be 25.5 mils. Since the deflection was 30 mils at loading location #16, the support condition was much weaker or there was a void under the slab. Also, according to Westergaard equation for edge loading condition, the deflection for 9-in. slab with 300 psi/in modulus of subgrade reaction would be 10.7 mils. The deflection at location #15 was 10.2 mils, which is quite close to Westergaard prediction at edge condition with 300 psi/in modulus of subgrade reaction. It implies that, even though longitudinal steel was fractured at this location, the support provided by tied concrete shoulder (with joint not cracked), put the slab in the edge condition. Dynamic cone penetration (DCP) testing was conducted after the concrete removal. Visual observation showed that there was no cohesiveness in the asphalt materials, and the stiffness of the asphalt material was quite low. It appeared that stripping occurred in the asphalt stabilized materials. DCP testing results indicate that the average of the estimated modulus values for asphalt layer was about 40,000 psi, which is very low. The average estimated modulus of subgrade material from DCP testing was 17,500 psi, which is larger than the modulus of subgrade material in many parts of Texas. It appears that the water in the asphalt base degraded asphalt materials and support conditions.

Evaluations of punchouts in Texas show that interactions between longitudinal steel and surrounding concrete cause horizontal cracking at the depth of longitudinal steel and subsequent longitudinal crack. In this punchout, longitudinal steel were broken and no longitudinal crack was observed at the loading locations #15 and #16 (See Figures 3.1 and 3.2). It is noted in Figure 3.6 that two longitudinal steel bars near the longitudinal warping joint were ruptured. The ruptured longitudinal steel, as shown in Figure 3.7, shows that the failure was primarily due to shear, not longitudinal stress. Quite limited corrosion was also observed. Evidence of necking and corrosion were noted in transverse steel. Figure 3.7 shows that the ruptured shape of the longitudinal steel, minimum corrosion and little evidence of necking indicate that shear played a major role, with tensile force playing a minor role. Even though horizontal cracking was observed at loading location #2, as shown in Figure 3.8, due to the way the concrete was removed it was not feasible to determine whether the horizontal cracking was pre-existing or it was caused by concrete removal operations. It is postulated that high deflections due to poor subbase support condition resulted in high deflections due to wheel loading applications at loading location #16, which caused the rupture of longitudinal steel without causing longitudinal cracking.

FWD testing was also conducted along L5, near the longitudinal warping joint in the outside shoulder (Figure 3.2). Figure 3.9 shows the deflection bowls at four locations (loading locations #17, #18, #19 and #20) (Figure 3.2). It shows that the deflections are not excessive. It is interesting to note that the maximum deflection at "Shoulder 4," which is at loading location #20 and next to loading location #16, is about 7 mils. The continuity of longitudinal steel itself doesn't explain this low deflection value. Recall that Westergaard equation shows 10.5 mils of deflections if it is in edge condition, with 300 psi/in modulus of subgrade reaction. The maximum deflection of 7 mils indicates that modulus of subgrade reaction is larger than 300 psi/in. Even when there is a void under the slab near a longitudinal joint, it was observed on US 75 in Sherman in the Paris District that the subbase condition on the other side of the joint is in good condition. It is believe that the same phenomenon occurred here.

Figure 3.7 Ruptured Longitudinal and Transverse Steel

Figure 3.8 Distressed Slab Showing Horizontal Cracking at the Mid-Depth of Slab

Load transfer efficiency (LTE) was evaluated at two transverse cracks with asphalt seal shown in Figure 3.1 along L1, L3, and L4 (Figure 3.2). The testing followed the same protocol used in the rigid pavement database or LTPP. Sensor #4 was located 12-in. on the other side of the loading plate, and the FWD van was located so that a crack was positioned between loading plate and sensor #2 (upstream) or between sensor #4 and the loading plate (downstream). LTE was evaluated as a ratio of sensor #2 deflection to sensor #4 deflection for upstream and a ratio of sensor #4 deflection to sensor #2 deflection for downstream loading. Figure 3.10 shows the LTE evaluated in six locations.

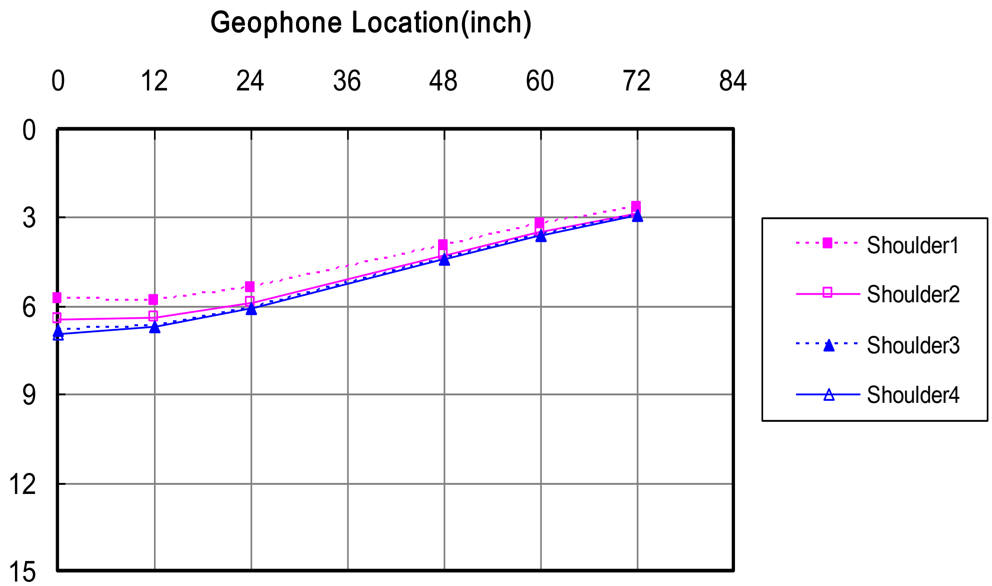

Figure 3.9 Deflections along L5

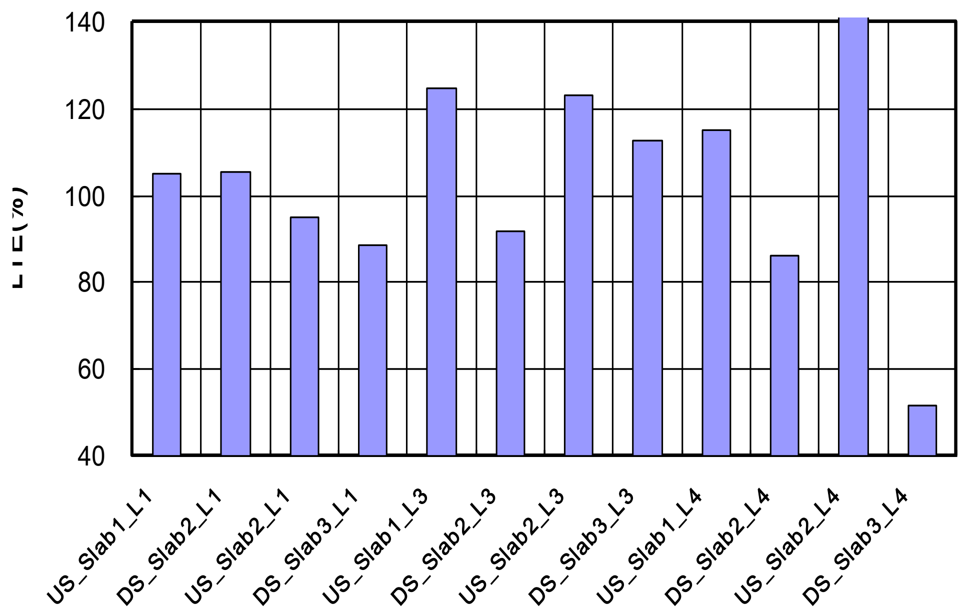

Figure 3.10 Load Transfer Efficiency at Transverse Cracks

The observations can be summarized as follows:

1) Even though a punchout distress was in progress in the two transverse cracks, LTEs are maintained at a high level except at one location.
2) In general, LTE values at upstream were higher than those at downstream. It could be because of voids or weakened subbase support in the downstream side, as discussed above. In the upstream loading, sensor #4 deflection was relatively small compared to sensor #2 due to inadequate slab support in sensor #2 area, making the LTE at upstream larger. In the downstream loading, sensor #2 deflection was relatively high due to inadequate slab support and sensor #4 deflection was relatively low, resulting in smaller LTE at the downstream. This trend was not observed in the testing done in the rigid pavement database project. If subbase support condition at both sides of a crack is comparable, the LTEs at upstream and downstream should be similar.
3) There is a large difference in LTE values at loading locations #15 and #16 (US_Slab2_L4 and DS_Slab_3_L4). The deflection information in Figure 3.4, and the descriptions in above 2) explain the large difference in LTE.
4) Average LTE values at each location are all above 90 percent, even though there is non-uniform subbase support and the cracks are undergoing punchout distress. The current way of computing crack LTE may not be a good indicator of pavement condition. The ratio of upstream and downstream LTE, along with deflection of sensor #1, might be a better indicator.

To obtain reference information on deflections in this area, cracks with no distresses were evaluated. Three slab segments with short, medium, and large crack spacings were selected. This testing was done in two locations. Table 3.1 shows the crack spacing information.

Table 3.1 Crack Spacing Information for Two Locations

Crack spacing	Location #1	Location #2
Short	1-ft. 7-in.	1-ft. 10-in.
Medium	4-ft. 0-in.	4-ft. 6-in.
Large	6-ft. 1-in.	8-ft. 5-in.

Figures 3.11 and 3.12 show the loading location numbers for Locations #1 and #2, respectively.

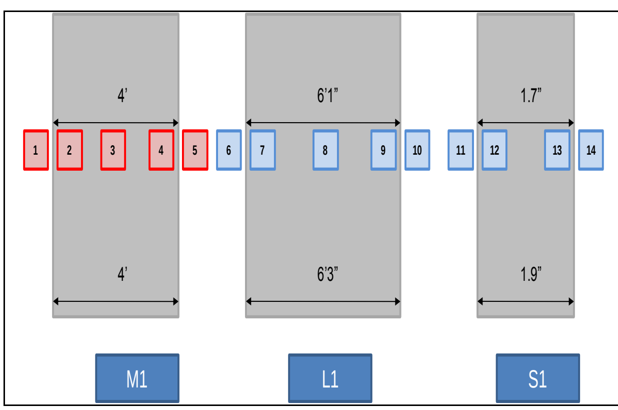

Figure 3.11 Location #1

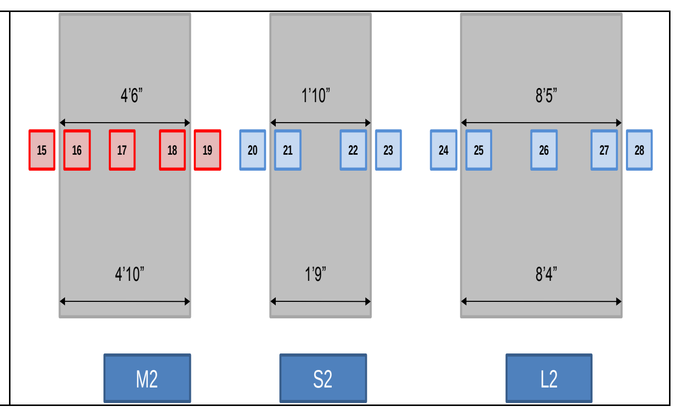

Figure 3.12 Location #2

Deflection testing was conducted at each location. One of the objectives was to evaluate average deflection values so that the deflections at the distressed area (Figure 3.1) could be compared. There was no distress in either location. Figure 3.13 shows the deflections obtained at locations #1 and #2 for large crack spacing. The observations can be summarized as follows:

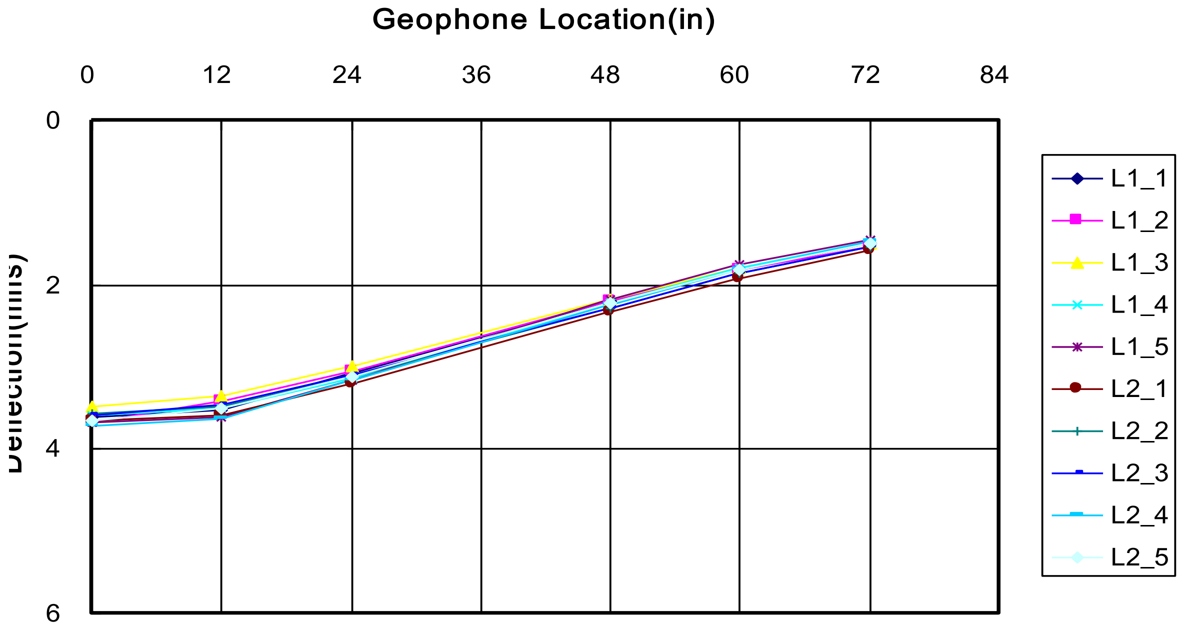

Figure 3.13 Deflections at Slabs with Large Crack Spacing at Both Locations

1) Maximum deflections were about 3.5 mils, which is much smaller than the values in the distressed area. This value is still a little larger than the statewide average for 9-in. CRCP.

2) Deflections were quite consistent, regardless of whether they were at cracks or the middle of the slab.

3) There was little difference in deflections between upstream and downstream locations.

Comparisons of Figures 3.3 (distressed area) and 3.13 indicate that poor subbase support condition in the distressed area is responsible for the distress. It appears that there was a rather large variability in the support conditions in this project, and distresses occur where support conditions are poor.

Figure 3.14 illustrates the deflections obtained at Locations #1 and #2 for medium crack spacing. The findings can be summarized as follows:

1) Maximum average deflections were about 4.0 mils, which is a little larger than the values obtained for large crack spacing. However, this value is still much smaller than the values in the distressed area.
2) The consistency of the deflections at various locations was not as good as the one for large crack spacing.

Figure 3.15 shows the deflections obtained at Locations #1 and #2 for small crack spacing. The deflections were quite similar to those for large crack spacing. Also, the maximum average deflection was smaller than that for medium crack spacing. In other words, there was no trend between crack spacing and maximum deflections. This is important because it confirms the findings from the rigid pavement database project that transverse crack spacing does not have an effect on deflections and LTEs. The fact that there was little difference in deflections between cracks and mid-slab indicates that cracks in CRCP are not necessarily weak elements in the pavement. They are there to relieve environmental stresses due to temperature and moisture variations. Also, this finding justifies the use of Westergaard's interior condition for stress and deflection analysis if the deflections are measured sufficiently away from longitudinal joints. One assumption needed for back-calculations of modulus values in pavement layers is the continuity of the concrete slab in both transverse and longitudinal directions, and CRCP will meet the assumption, as long as steel is not ruptured at transverse cracks.

The most important variable that affects CRCP distress development is subbase support, and more attention and effort need to be paid to the support condition. Up to this point, in CRCP research, too much emphasis has been placed on the effect of crack spacing on distress. On the other hand, the insensitivity of subgrade support condition on stresses in concrete slab in Westergaard's equations led to the belief that slab support condition is not as important as crack spacing. Field evaluations conducted in this project and others show the importance of slab support for good performance of CRCP.

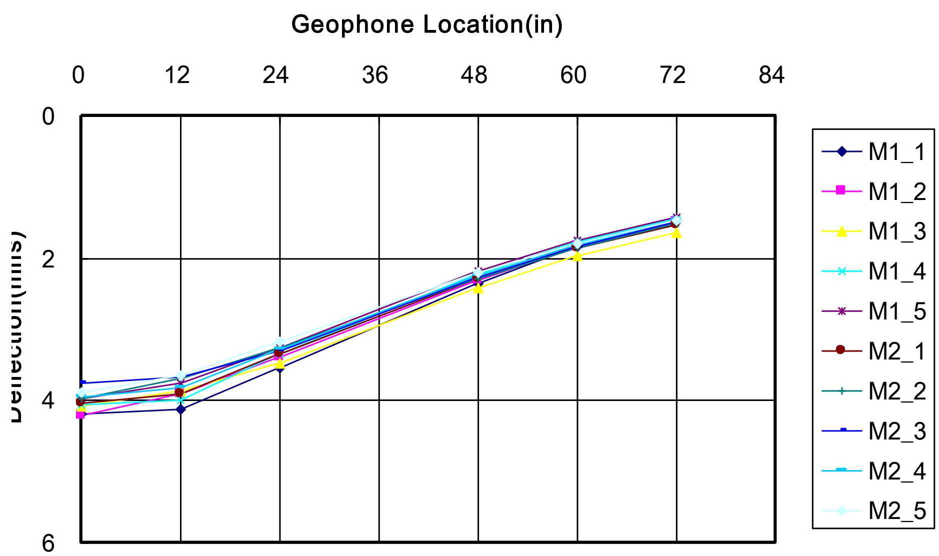

Figure 3.14 Deflections at Slabs with Medium Crack Spacing at Both Locations

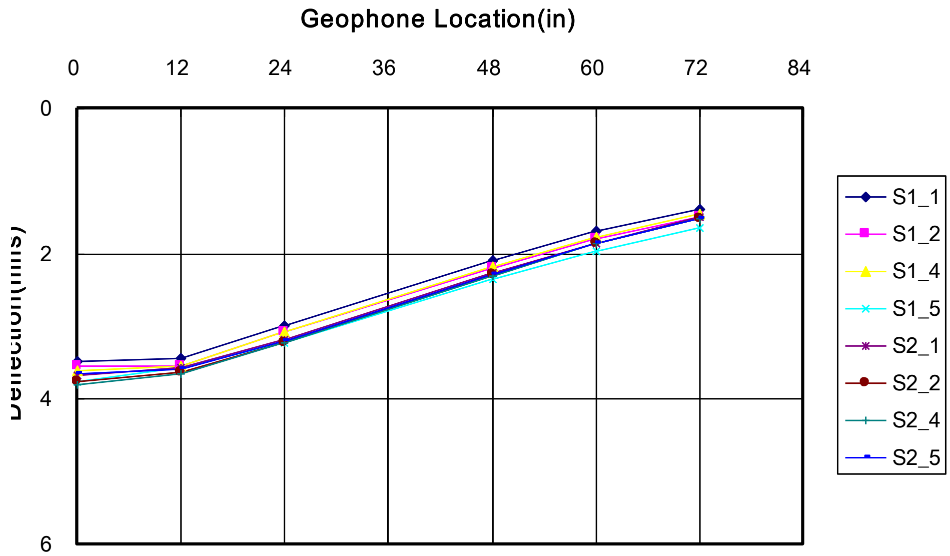

Figure 3.15 Deflections at Slabs with Small Crack Spacing at Both Locations

3.2 Summary

The findings in this investigation provide valuable information on one of the failure mechanisms of CRCP. It could be that the pavement section was designed with a design life of 20 years. It served more than 30 years of traffic and is still in good condition except for punchout distress in a few locations. Forensic evaluations were conducted to identify the causes of the distress. The evaluations included (1) deflection testing using FWD at the distressed area as well as at non-distressed areas and (2) DCP testing at the distressed area. Findings can be summarized as follows:

1) The overall deflections in the distressed area were much higher than those in non-distressed areas. This indicates that there was a large variability in subbase support in this project, and one of the causes of the distress was poor slab support.
2) Average load transfer efficiency (LTE) at transverse cracks was maintained at a high level at the distressed area. On the other hand, LTE at upstream was higher than that at downstream. Average LTE of upstream and downstream may not be a good indicator of the shear stiffness of transverse cracks.
3) Transverse crack spacing doesn’t appear to affect LTE or overall deflections. This finding is consistent with the results of extensive evaluations made in this project.
4) Punchout mechanism adopted in MEPDG – increased crack width due to continued drying shrinkage of concrete reduces LTE at transverse cracks, which again increases transverse concrete stresses on top of slab and causes punchout – doesn’t seem to apply to the distress investigated. It appears that inadequate subbase support might degrade the conditions of cracks, not vice versa.
5) When concrete was removed from the distressed area, water was observed even though no water was used during concrete removal, and there was no rain for several months prior to the repair.

6) Even though the plan called for the use of asphalt stabilized base, visual observation showed that there was no cohesiveness in the asphalt materials, and the stiffness of the asphalt material was quite low. It appeared that stripping occurred in the asphalt stabilized materials.
7) Longitudinal and transverse steel were ruptured at a transverse crack at a distressed area. Corrosion was quite limited in the ruptured longitudinal steel. It appears that longitudinal steel was ruptured due to shear. Water was observed in the subbase where steel rupture occurred.

The findings in this investigation indicate that for good CRCP performance, the following conditions should be met.

1) Adequate and uniform slab support needs to be provided. Just increasing slab thickness might not provide satisfactory performance if the slab support is deficient.
2) The quality and durability of the subbase needs to be maintained throughout the life of CRCP.
3) Adequate load transfer needs to be provided at longitudinal joints. In 2009, TxDOT revised CRCP Standards to ensure an adequate amount of tie bars and transverse steel at longitudinal joints, and this may not be an issue for projects built under this Standard.

CHAPTER 4 DATABASE SYSTEM AND LEVEL I FIELD EVALUATIONS

This research project has two broad objectives. One is to evaluate overall performance of rigid pavements in Texas, and the other is to conduct detailed structural evaluations of CRCP. Level I evaluations include deflection testing at 50-ft. intervals for average slab deflections and LTE evaluations of selected transverse cracks – cracks with small, medium, and large spacing. The testing was done in the summer and in the winter to evaluate the seasonal effects. This chapter provides information on the structure of the latest GIS-based database and presents field testing results conducted in FY2010.

4.1 Rigid Pavement Database System

One of the essential deliverables of this project is the development of an efficient system to store the rigid pavement field performance data and also develop a user-friendly, GIS-oriented web portal for easy access to the database. The field data is stored on a Windows Server 2008 R2 for Enterprise which is being maintained by Technology Operations & Systems Management (TOSM) at Texas Tech University. The storage structure of all the Level I data files on the server is given in Figure 4.1.

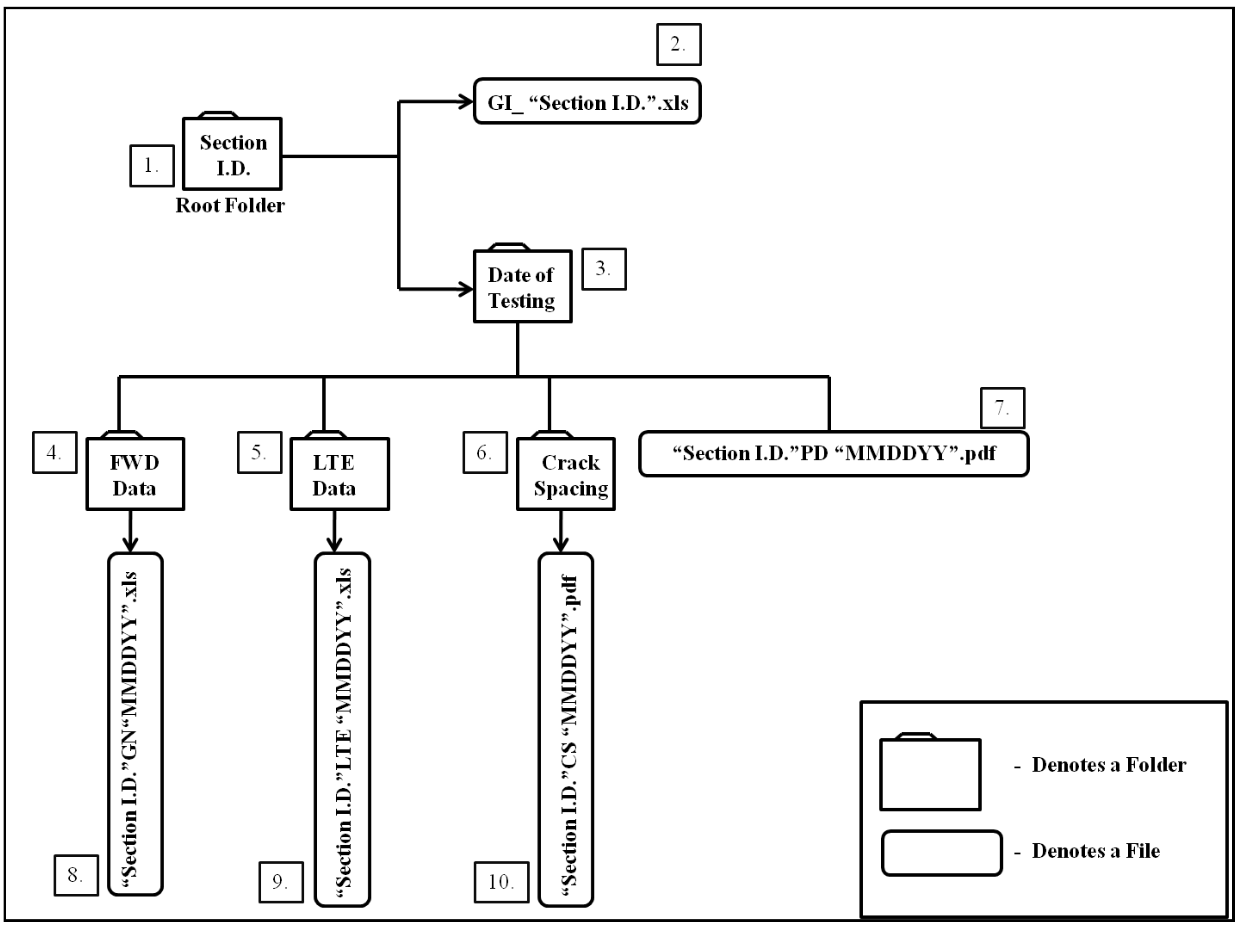

Figure 4.1 Structure of Data Storage

Each test section has a prescribed root folder named as per the Section I.D. The root folder contains a file containing the general information such as the district, county, year of construction, slab thickness, highway, reference marker, GPS coordinates and type of shoulder. The root folder also contains folders named as per the date of field data collection. The dated folders contain individual folders for each data type such as FWD Data, LTE Data, and Crack Spacing as well as a .pdf file containing the photos for the particular section collected on the specific day of testing. The individual folders contain the respective raw field data files. Table 4.1 lists the names of files and folders and the data they contain.

Table 4.1 File and Folder Nomenclature

Name of File or Folder	**Type of Data**
GI_"Section I.D."	Test section general information file
"Section I.D."GN"MMDDYY"	FWD deflection data file at every 50 feet
"Section I.D."LTE"MMDDYY"	LTE at transverse cracks data file
"Section I.D."CS"MMDDYY"	Crack spacing data file
"Section I.D."PD"MMDDYY"	Photos

The data for Level II and Level III sections contains on-site photos taken during periodic visual field surveys. Visual survey evaluation data in the form of photos for various special sections constructed by TxDOT over the past few years are also included in the database under the folder Special Sections. Various test sections built under previous TxDOT research projects are being monitored visually under this project and the related photo database is stored under the Experimental Sections folder.

A GIS-based web application has been developed using ESRI's ArcGIS Desktop 9.3 and ArcGIS Server 9.3.1 software that displays graphically the geographical location of the Level I test section on a map. Each test section appearing on the graphical interface of the application is also connected to its root data file on the server, thus providing easy access to all field data associated with the particular section. Apart from data access, the application also carries out query and search functions based on inputs provided by the user. The query functions can be carried out based on a combination of attributes such as the year of construction, highway, district, and the thickness of the test section. The search attribute function enables the user to directly input and search for a given value of the previously mentioned attributes. Figure 4.2 shows the interface of the GIS-based service.

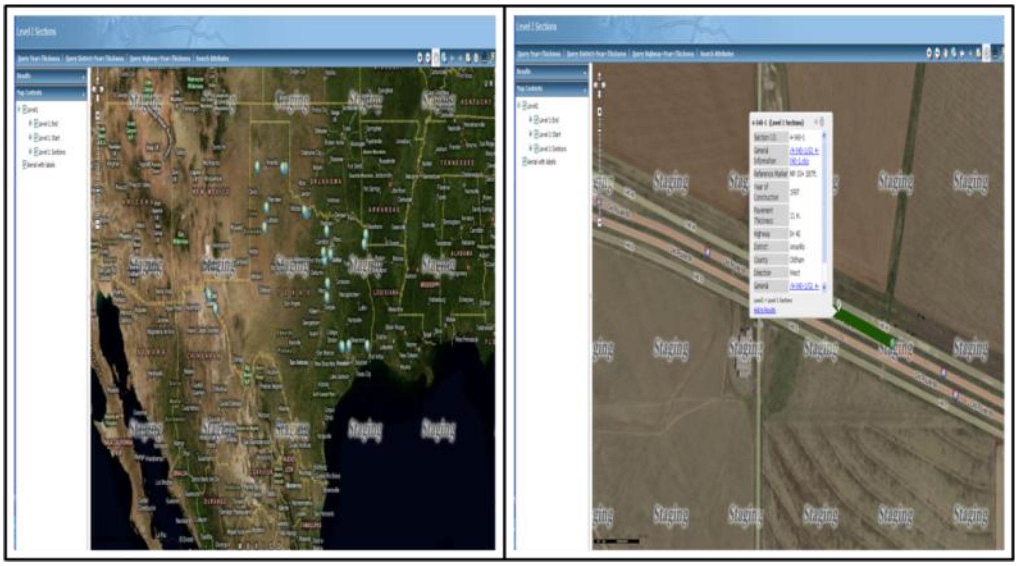

Figure 4.2 GIS-Based Web Application

Since the database comprises a large number of test sections and various combinations of field data collection, a web-based portal was developed to ensure centralized access to the various forms of data and various data collection levels. This website was developed using www.wix.com, which is a flash web design creation website that enables users to create interactive websites using a drag and drop interface. The Texas Rigid Pavement Database website can be accessed at www.wix.com/sarafss83/rpdb. Once all essential components of the web portal are developed, the web address will be personalized by hosting it from the domain host at Texas Tech University. The web-portal is currently password protected using a pre-set six digit password. Figure 4.3 provides a snap shot of the Texas Rigid Pavement Database web-portal.

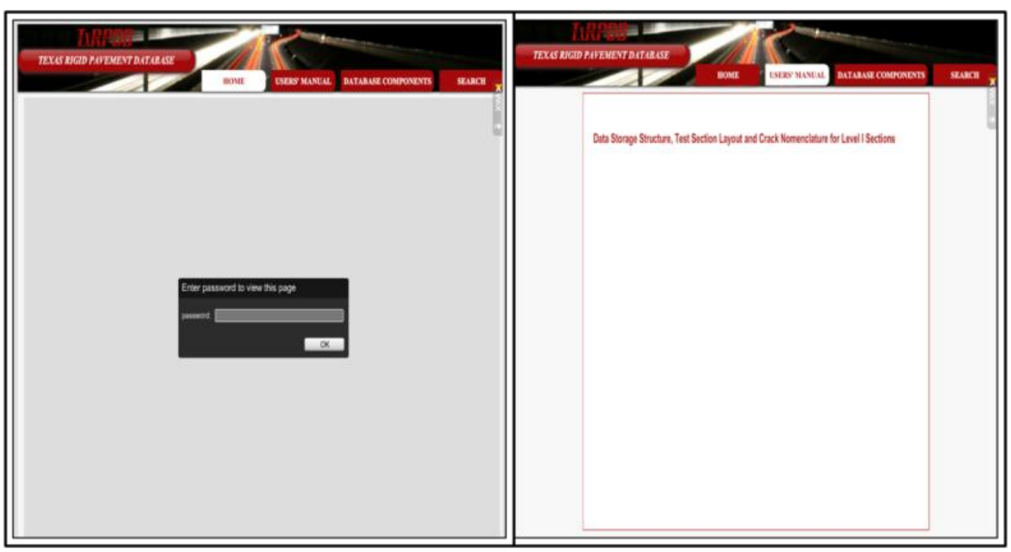

Figure 4.3 Texas Rigid Pavement Database Web Portal

As shown in Figure 4.4, the Texas Rigid Pavement Database website contains four main tabs on the top right of the screen i.e. *Home*, *Users' Manual*, *Database Components* and *Search*. The user is guided to the *Home* page upon logging-in. to the website. This page will contain general background regarding the Texas Rigid Pavement Database project and its importance. The *Users' Manual* page currently contains a guide to the Data Storage Structure, Test Section Layout and Crack Nomenclature for Level I sections. Upon inclusion of all necessary components to the website, a comprehensive Users' Manual and website guide will be published onto the Users' Manual page.

The Database Components tab allows access to the page enlisting the various types of field data collected during this project. For Level I sections, the primary field for accessing data is the district. Once the user clicks on a specific district, the slab thicknesses of the test sections in the particular district are displayed. Upon clicking the desired slab thickness, the highway and the Section I.D. pertaining to the specific section are displayed. Detailed information associated with a particular section such as general information, design plan sets, average deflection trends, load transfer efficiency trends, on-site pictures and field data files can be accessed by clicking on the test section I.D. Figure 4.4 provides a preview of the Level I Sections webpage.

Figure 4.4 Level I Sections Web Page

As can be seen in Figure 4.4, the Level I section webpage has a distinct tab to access the GIS Database associated with the Level I sections. In this way, the web-portal provides the user with the flexibility to either access data files directly via the click-and-dropdown menu, or view the test section graphically, access the associated data, and perform queries using the *Access GIS Database* tab.

The data collection for Level II and Level III sections in the database were focused on periodic visual evaluation of the test sections and the respective tabs on the webpage allow access to on-site pictures related to these sections. Pictures from periodic visual evaluations of Special and Experimental sections can also be accessed through the respective tabs on the left-hand side of the Database Components page menu.

Since TxDOT has upgraded to the latest release of ArcGIS software (ArcGIS 10), efforts will be made to transfer and reproduce the GIS-based web application on the new platform. Recommendations made by TxDOT – Technology Services Division will be followed for development of this application.

4.2 Level I Data Analysis

The details of Level I test sections and data collection scheme are described in TxDOT Report 0-5445-R2, and are not repeated here. In this section, deflection data collected in FY10 is discussed. Deflection testing results from FY10 field testing are included in the Appendices as follows: (1) numerical information on deflections for 50-ft. interval testing in Appendix A, (2) graphical presentation of the information on deflections for 50-ft. interval testing in Appendix B, (3) numerical load transfer efficiency (LTE) information in Appendix C, and (4) graphical presentation of the information on LTE in Appendix D.

4.2.1 Data Collected at 50-ft. Intervals

The objective of the deflection measurements at every 50-ft. for a 1,000-ft. long section is to estimate average deflection for each slab thickness and the variability in the section. In Chapter 3, the variability of subbase support and its effect on CRCP distress development was discussed. Even though there was a rather tight control on slab thickness in part due to a penalty in Item 360 for deficient slab thickness, quality control in the subgrade preparation and subbase construction was not as stringent. Field evaluations of CRCP distresses recorded as punchouts in TxDOT PMIS show that a substantial amount of distresses are caused by a non-structural element, such as quality control issues in materials and construction. Among the distresses caused by structural deficiency, the majority is due to deficient subbase support. If CRCP develops distresses due to fatigue of concrete on uniform subbase support, punchouts should be observed randomly with little evidence of the deficiency of subbase support. Field evaluations show otherwise. The current pavement design methods do not consider the effect of non-uniformity in slab support directly. On the other hand, the effect of non-uniformity in slab support is indirectly included in a transfer function, since the transition from cumulative damage to punchout includes the effect of non-uniform slab support. As the quality of subgrade preparation and subbase construction improves, pavement performance will be enhanced. As discussed in Chapter 3, deficiency in slab support capacity and uniformity will lead to distresses and shorten the pavement life.

Figure 4.5 shows deflection data in the 12-US290-1 section in the Houston district. This section has a 10-in. slab and was built in 1992. The average deflection is 2.18 mils. There was a rather large variability in deflections. In this project, it was confirmed that the repeatability of well-calibrated FWD is excellent, and the variability in this section could be due to the non-uniformity in the slab support. In Chapter 3, it was shown that in CRCP, Westergaard's interior condition equation can be used to estimate subgrade modulus. Figure 4.6 shows deflections at the interior condition as affected by modulus of subgrade reaction (k). It shows that the k value varies from about 400 psi/in at the 300-ft. location to 900 psi/in at the 450-ft. location, with the average of about 650 psi/in. Even though there was no punchout distress in this section, eventually punchouts will develop and it could be at the 300-ft. to 350-ft. location. This shows the need for better quality control during construction for slab support. Development and implementation of intelligent compaction could help achieve more uniform slab support.

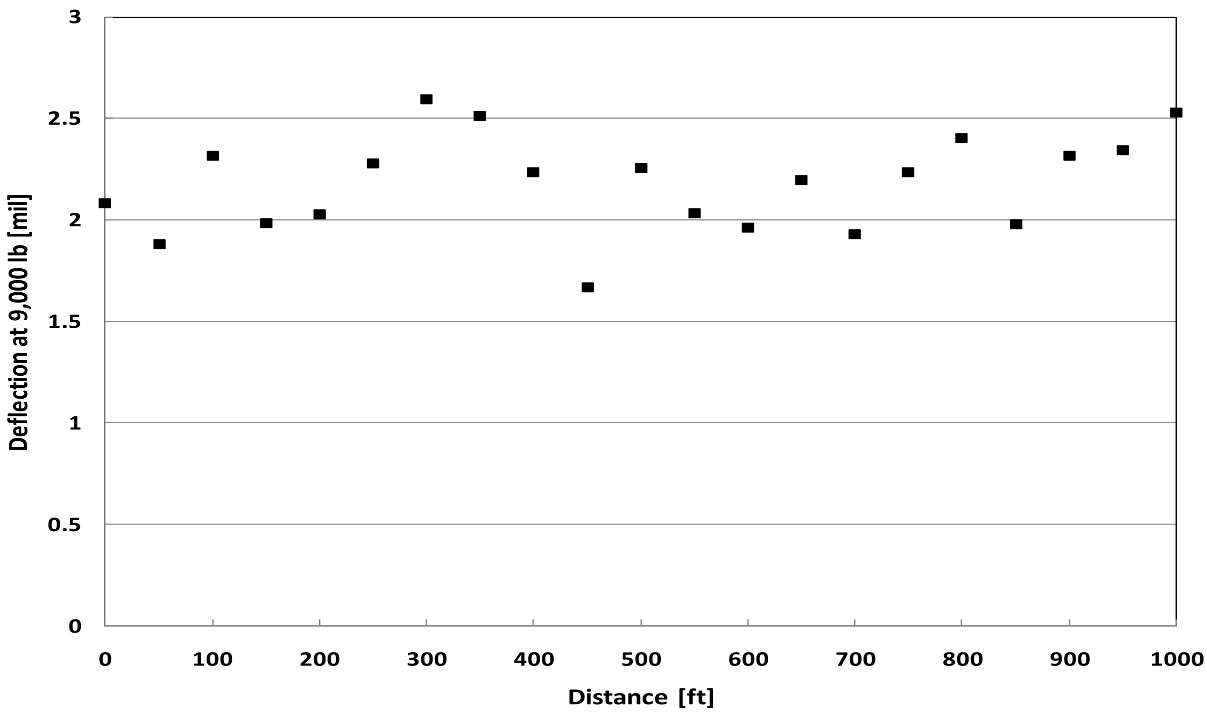

Figure 4.5 Deflection at 50-ft. Intervals at 12-US 290-1 Section in Houston

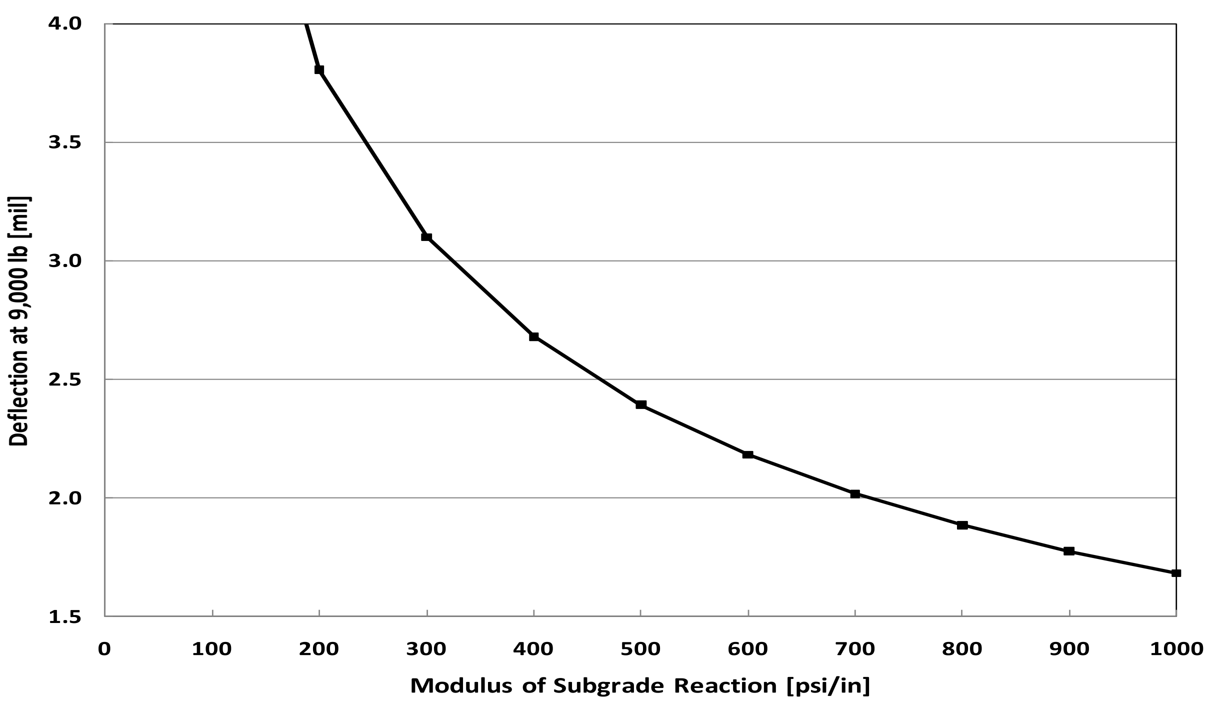

Figure 4.6 Deflections at Interior Condition for Various Modulus of Subgrade Reaction

4.2.2 Data Collected for Load Transfer Efficiency (LTE)

It has been reported that LTE is an important structural response that has a significant effect on CRCP performance (ERES, 2004). It also has been reported that crack spacing should not be large, since large crack spacing will lead to large crack width and lower LTE (ERES, 2004). A hypothesis was made in the development of MEPDG for CRCP under NCHRP 1-37(A) that there is a good correlation between crack spacing and crack width, and between crack spacing and LTE. It was decided at the beginning of this research study that, based on the assumptions made in MEPDG, LTEs will be evaluated at three different transverse crack spacings. A primary objective was to evaluate the effect of crack spacing on LTE and to collect information on overall LTE values in CRCP in Texas. At each Level I section, 12 cracks were selected; 4 each for small, medium, and large crack spacing. Data collected so far could not validate the hypothesis in MEPDG. In other words, there was no good correlation between crack spacing and LTE. Figure 4.7 shows the correlation between crack spacing and LTE from the LTPP database (FHWA, 1999). No correlations were observed. It was also noted that more than 96 percent of the cracks had LTE values higher than 90 percent, and the other 4 percent (2 cracks) had more than 80 percent LTE. LTE values obtained in the TxDOT database project also confirm the findings from LTPP. There is no correlation between crack spacing and LTE.

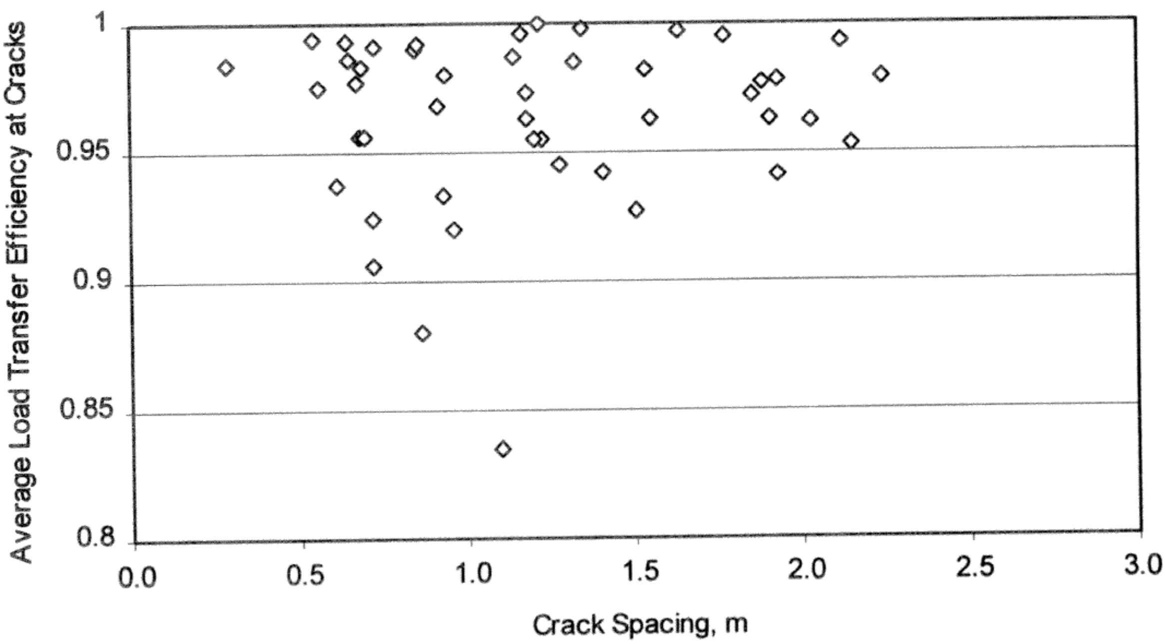

Figure 4.7 Effect of Crack Spacing on LTE from LTPP

Figure 4.8 shows the LTE values obtained in FY10 at the El Paso Section (24-I10-4). S stands for “short” crack spacing, M for “medium” crack spacing, and L for “large” crack spacing. Blue bars indicate LTE values in the winter and red in the summer. In this project, LTE testing was conducted in the summer and in the winter. The objective of the testing in two seasons was to evaluate the temperature effect on LTE. Figure 4.8 shows that LTEs are all quite high, and there was no practical difference between winter and summer testing. The testing results obtained so far show no temperature effect. Also, no correlations were observed between crack spacing and LTE. This trend has been consistent from the beginning of the project and in all test sections.

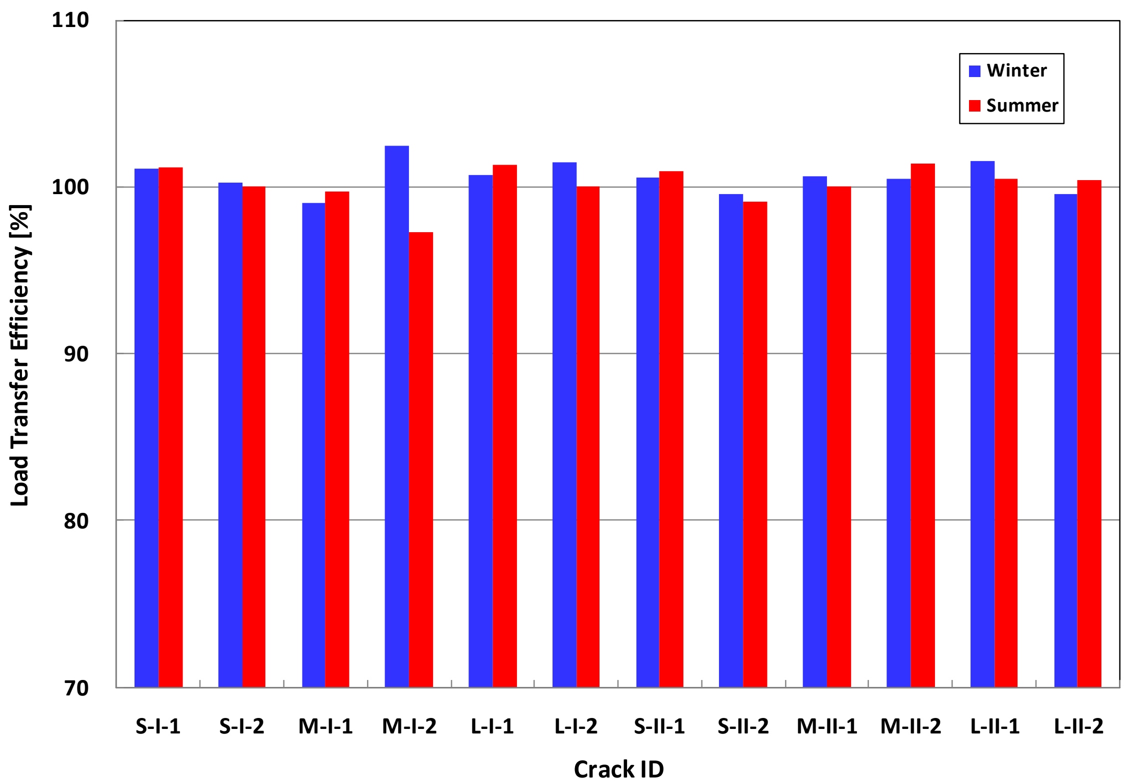

Figure 4.8 LTE Values from 24-I 10-4 in El Paso District

Figure 4.9 shows the LTE output from MEPDG. It shows that LTE was high in the summer and low in the winter. This is because MEPDG assumes that crack width varies with temperature. In the summer, concrete volume expands and crack width becomes smaller, which leads to higher LTE. On the other hand, in the winter, concrete volume contracts and crack width becomes larger, leading to lower LTE. Figure 4.9 also shows that LTE decreased to as low as 40 to 50 percent at the end of design life. It appears that the crack width and LTE models in MEPDG over-estimate the effect of temperature and crack spacing. Extensive deflection testing conducted in this project over the years produced a consistent trend – crack spacing and temperature do not have any effect on LTE. LTPP also confirms no effect of crack spacing on LTE.

As discussed in Chapter 3, it appears that LTE is not a good indicator of CRCP performance. LTE is an important variable if the punchout mechanism involves widening of crack, and resulting decrease in LTE. Evaluations of punchouts in Texas do not support this punchout mechanism. Punchouts occur when there is deficient slab support, either by voids or degradation of subbase materials. As long as there is adequate slab support, LTE values are maintained quite high. All the LTE information collected in this study shows that almost all the cracks have LTE values higher than 90 percent regardless of crack spacing, slab thickness, age of the pavement or season of testing. LTE variations shown in Figure 4.8 could not be found.

The reason extensive testing was conducted in this project for LTE was based on the assumption that LTE has a significant effect on punchout development in CRCP. The usefulness of LTE in

CRCP structural condition is quite limited. Since it was discovered that LTE is not as significant a structural response as once thought, a decision may need to be made concerning whether extensive field testing for LTE should continue. Deflection value itself might be a better indicator of the pavement condition. Or some statistic of the deflection testing values, such as average and coefficient variation, might have a better value than LTE itself.

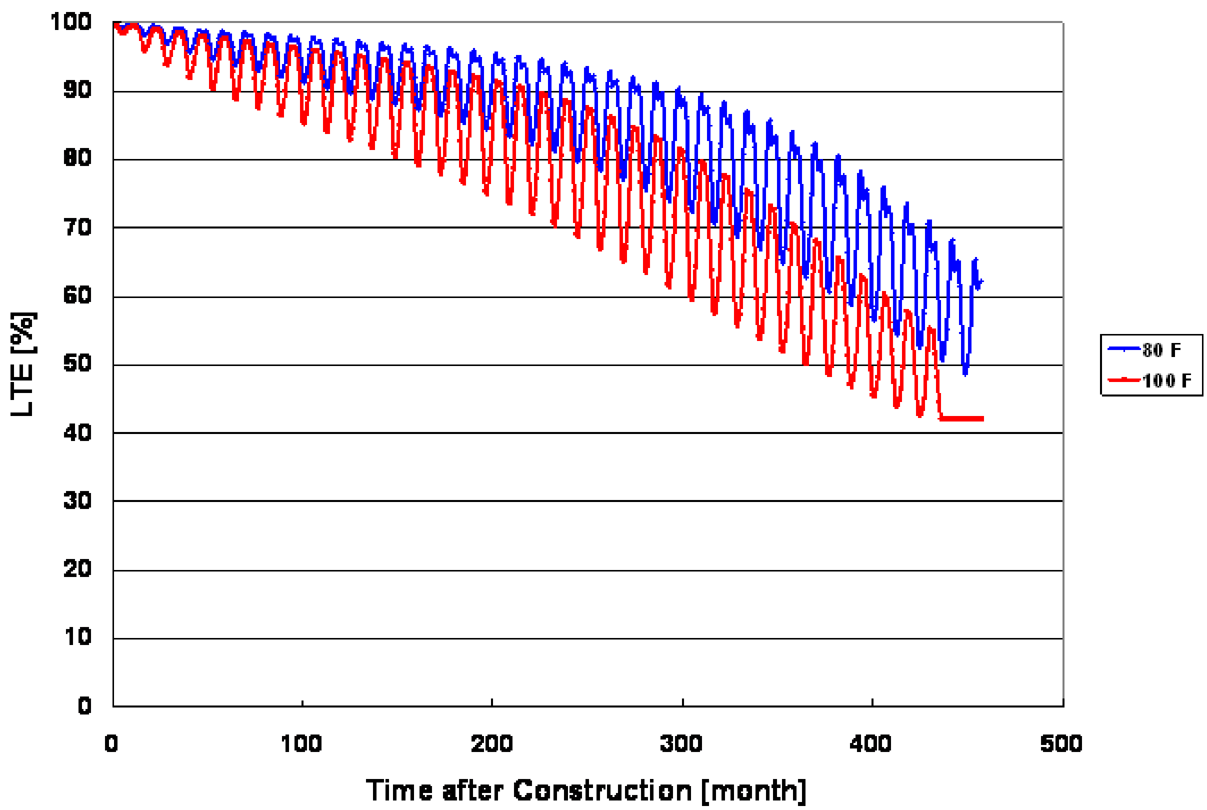

Figure 4.9 LTE Variations over Time from MEPDG

Research effort could be better utilized for more important aspects of CRCP design and performance, such as the evaluation of the effects of slab support on punchout. An investigation of the cause for the variability in deflections would be a good example. Another example would be the field confirmation of the back-calculated modulus of subgrade reaction from deflection testing. The findings from these efforts will enhance the capabilities of pavement engineers in correctly and accurately evaluating pavement conditions and determining what needs to be done to preserve the pavement system with the least cost.

4.3 Summary

Deflection testing using FWD has been conducted in Level I sections and the findings so far can be summarized as follows:

1) There is variability in deflections in the 1,000-ft. test section. At this point, it is postulated that non-uniform slab support is responsible for the variability.

2) LTE values are quite high in all the 324 cracks evaluated in this project, regardless of crack spacing, slab thickness, or age of the pavement.
3) LTE itself might not be a good indicator of the pavement condition. With adequate steel percentage and slab support, LTE values will always be quite high, even when cracks are experiencing punchout distress, as discussed in Chapter 3. Other statistics, such as an average and coefficient of variation of a series of deflection values, might be a better indicator of CRCP structural condition.
4) Since the usefulness of LTE in evaluating structural condition of CRCP is questionable, research effort could be better utilized for more important aspects of CRCP design and performance, such as the evaluation of the effects of slab support on punchout.

CHAPTER 5 DEVELOPMENT OF A TRANSFER FUNCITON

One of the objectives of this project is to collect information on CRCP structural responses that can be used to calibrate a mechanistic-empirical CRCP design program developed under TxDOT research study 0-5832, called TxCRCP-ME. A transfer function correlates accumulated damage in the concrete slab to punchout development. The importance of a transfer function in any mechanistic-empirical pavement design programs cannot be overstated. In this project, efforts were made to develop an accurate transfer function. There are only three inputs needed for the development of a transfer function – punchout rate, traffic, and pavement damage. Accordingly, developing an accurate transfer function requires (1) accurate estimation of punchout frequency, (2) reliable traffic information from the construction of a section, and (3) reasonableness of accumulated damage information. This chapter discusses the efforts made so far and to be made in the future that are essential for the development of an accurate transfer function.

5.1 Accurate Estimation of Punchout Frequency

Since there are only two distress types identified so far in CRCP, which are spalling and punchouts, all the distresses that are not obvious spalling have been classified as punchouts in this research. Spalling is a functional distress and punchout is a structural distress. In Texas, spalling occurs when certain coarse aggregate type is used. Otherwise, spalling is quite rare. Spalling is fairly well defined. Since all the distresses that are not spalling are classified as punchouts, it can be stated that punchout has been loosely defined, at least for those who collect data on CRCP distresses. If all the distresses other than spalling in CRCP have only one development mechanism, the practice of punchout being loosely defined does not cause a serious problem. On the other hand, if different mechanisms are at work for distresses currently classified as punchout, a more precise definition of punchout is badly needed. Field evaluations conducted in this study to collect punchout information revealed that there are several different distress mechanisms for CRCP distresses. In this chapter, the term "punchout" is used as a distress identified and recorded as punchout in TxDOT PMIS, unless another description is provided.

Efforts were made to evaluate all the punchouts in Texas that were recorded in the TxDOT PMIS. The goal was that the research team visits every single punchout in Texas, visually evaluate the distress and potential distress mechanisms identified. It was not possible to evaluate every single punchout, because some punchouts are in inside lanes where traffic is heavy. There are also sections where an outside shoulder does not exist or is quite narrow. With those limitations, efforts were made to collect as complete information as possible. According to 2009 PMIS, the Houston district has the most punchouts, with about 60 percent of all the punchouts in the state, followed by the Lubbock district, which has about 9 percent. The two districts have about 70 percent of all punchouts in the state. Field evaluations of punchouts in Houston revealed that most of the distresses recorded as punchout in PMIS are not actual punchout. Some of them are distresses in thin bonded overlays, and many of them were actually surface defects where siliceous river gravel was used as coarse aggregate. Most of the distresses recorded as punchouts in the Lubbock district are large surface defects along longitudinal joints, which were not caused by structural deficiency of CRCP. A decision was made to concentrate on Dallas, Fort Worth, Wichita Falls, and Childress districts for punchout evaluations. Table 5.1 shows the punchout information collected for the above four districts. The explanations of punchout types are provided below the Table. “Repair or Not Found” indicates the punchouts recorded in PMIS might have been repaired with asphalt or concrete patches. “Not Investigated” indicates it was not possible to evaluate due to the reasons described above. “Incorrect” indicates that the research team was not able to locate the distresses. Example pictures of each category are included in Appendix E. The punchout numbers in the first 6 columns were the distresses the research team was able to classify. It shows that one out of four was the distress at transverse construction joints. About one out of five was distress at the repair joints. Another quarter of the distresses were surface defects probably caused by poor concrete surface finish. Distresses caused by structural deficiency were about 33 percent of the distresses observed. This information is quite valuable.

Table 5.1 Punchout Information for Four Districts

	Punchout									
	PCH	**E-PCH**	**E-PCH-PTB**	**PCH-CJ**	**PCH-RJ**	**BS-PCW**	**Repair or Not Found**	**Not Investigated**	**Incorrect**	**TOTAL**
Dallas	10	8	2	7	8	6	9	19	15	84
Fort Worth	0	1	0	6	10	11	26	12	2	68
Wichita Falls	4	0	4	10	0	5	34	2	3	62
Childress	2	0	0	1	0	0	1	1	0	5
Sub Total	16	9	6	24	18	22	70	34	20	219
Ratio	**16.8%**	**9.5%**	**6.3%**	**25.3%**	**18.9%**	**23.2%**				

PCH: punchout due to structural deficiency
E-PCH: edge punchout
E-PCH-PTB: edge punchout with poor tie bar
PCH-CJ: punchout at transverse construction joint
PCH-RJ: punchout at repair joint
BS-PCW: big spalling with poor concrete work

The punchout information from TxDOT PMIS cannot be used for the development of a transfer function or calibration of the model. The punchout data in LTPP is not adequate for the development of a transfer function either. In LTPP, the length of the test section is 500-ft. The number of punchouts observed in the 500-ft. section is multiplied by 10 to get the number of punchouts per mile. This is an over-simplification of punchout information. Currently, 10 punchouts per mile is considered the terminal condition of CRCP. The way LTPP data was collected and used in the development of a transfer function for MEPDG is not realistic. In Texas, no CRCP sections were identified by the research team where more than 10 punchouts were observed. Table 5.1 shows that in the Dallas district, there are a total 20 punchouts in the whole district that were due to the structural deficiency, even though the research team was not able to look at all of the distresses. In the Fort Worth district, there was only one. Efforts will continue to collect as accurate information on punchouts as possible.

5.2 Reliable Traffic Information

Traffic information is another important input needed for the development of an accurate transfer function. To develop a transfer function used in TxCRCP-ME, traffic information in PMIS was used. PMIS provides 20-year future traffic information in terms of ESALs for each segment of highway. The methodology used was as follows:

1) Estimate current annual ESAL assuming that there will be a 4 % annual growth rate.
2) Assuming that there was 4 % annual traffic growth rate from the construction of the sections, estimate the past traffic that was applied to the pavement section.

The selection of 4 % was arbitrary. Unfortunately, traffic growth rate has a substantial effect on traffic data. Figure 5.1 illustrates the effect of traffic growth rate on estimated accumulated ESALs.

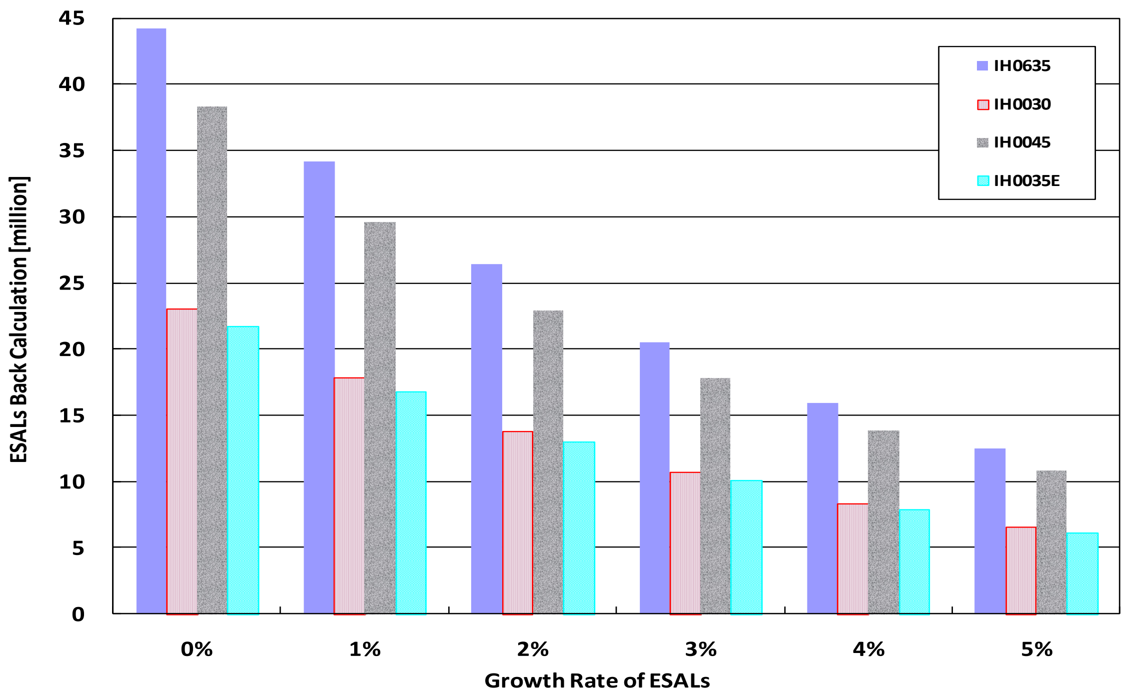

Figure 5.1 Estimated Accumulated ESALs for Various Growth Rates

In the development of Figure 5.1, 20-year ESALs were obtained from PMIS and the procedure described above was followed. It shows a substantial effect on the past ESALs. For example, for IH 635, with 0 percent growth assumed in the future and in the past, the accumulated traffic from the construction of the pavement is about 44 million ESALs. On the other hand, with a three percent growth rate, the value becomes less than half of the value with a 0 percent growth rate. Similar trends were observed in other highways as well. In other words, arbitrarily selecting traffic growth rate could result in over-estimation or under-estimation of past ESALs. This could be a problem when developing a transfer function. Unless there is a reasonable traffic growth rate available, traffic back-calculation from PMIS should not be used. For a transfer function to be reasonably accurate, all three elements – punchout rate, traffic, and estimated damage in pavement – must be reasonably accurate. Out of these three, the estimates for punchout rate and estimated damage in pavement can be made with certain accuracy. Pavement engineers and researchers have some control on those two variables. On the other hand, obtaining reasonably accurate traffic information is beyond pavement engineers or researchers for various reasons. Efforts should be made to obtain valid traffic data.

5.3 Reasonableness of Accumulated Damage from ME Models

The primary distress type observed in CRCP with deficient slab thickness but with tied concrete shoulder is related to deteriorations of concrete at the depth of longitudinal steel. For the estimation of concrete stress near the longitudinal steel in the development of TxCRCP-ME program, 3-dimensional finite element analysis was conducted. It is believed that the analysis is reasonable. The last element for a reasonable transfer function is the accuracy of accumulated damage from CRCP ME models. There is no fatigue equation available that considers the concrete stresses near longitudinal steel. However, the selection of a fatigue equation is not critical, since what's more important is the ratio of strength to stress. Damages will depend on the fatigue equation used; however, a transfer function will address the issue of a fatigue equation, as long as the same fatigue equation that was used to develop a transfer function is used in the program.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

There are two primary objectives for this study; one is to evaluate overall performance of rigid pavements in Texas and the other is to gather detailed CRCP structural behavior that can be used for the calibration of a now-developed mechanistic-empirical CRCP design program, called TxCRCP-ME. Field evaluations were conducted to collect accurate information on CRCP distress and structural behavior. Forensic evaluations were also conducted to identify distress mechanisms. Based on the work conducted in this study, the following conclusions are made:

A. Evaluations of Experimental Sections

1) There is no clear trend between crack spacing and deflections, which indicates the efficiency of longitudinal steel in providing the continuity of slabs at transverse cracks.
2) For concrete with spalling-susceptible coarse aggregate, there is a strong correlation between steel percentage and spalling potential. The larger the steel amount, the lower the frequency of spalling. However, this does not imply that TxDOT can utilize spalling-susceptible coarse aggregate in CRCP with a larger amount of longitudinal steel. The frequency of spalling in CRCP with spalling-prone coarse aggregate and larger longitudinal steel is still much higher than that in CRCP with less spalling-prone coarse aggregate.
3) For concrete with coarse aggregate with lower spalling potential, the effect of transverse crack spacing or longitudinal steel amount on spalling is non-existent.

B. Forensic Evaluations of CRCP Distress on IH 40

1) The overall deflections in the distressed area were much higher than those in non-distressed areas. This indicates that there was a large variability in subbase support in this project, and one of the causes of the distress was poor slab support.
2) Average load transfer efficiency (LTE) at transverse cracks was maintained at a high level at the distressed area.
3) Transverse crack spacing doesn't appear to affect LTE or overall deflections. This finding is consistent with the results of extensive evaluations made in this project.
4) Even though the plan called for the use of asphalt stabilized base, visual observation showed that there was no cohesiveness in the asphalt materials, and the stiffness of the asphalt material was quite low.
5) Longitudinal and transverse steel were ruptured at a transverse crack at one distressed area. Corrosion was quite limited in the ruptured longitudinal steel. It appears that longitudinal steel was ruptured due to shear.

The findings in this investigation indicate that for good CRCP performance, the following conditions should be met.

1) Adequate and uniform slab support needs to be provided. Just increasing slab thickness might not provide satisfactory performance if the slab support is deficient.
2) The quality and durability of the subbase needs to be maintained throughout the life of CRCP.

3) Limiting deflections to an acceptable value is a key to the good performance of CRCP. Providing quality slab support and adequate slab thickness and load transfer at longitudinal joints is essential to reducing deflections.

C. Level I Field Evaluations

1) There is variability in deflections in the 1,000-ft. test section. At this point, it is postulated that non-uniform slab support is responsible for the variability.
2) LTE values are quite high in all the 324 cracks evaluated in this project, regardless of crack spacing, slab thickness, or age of the pavement.
3) LTE itself might not be a good indicator of the pavement condition. With adequate steel percentage and slab support, LTE values will always be quite high, even when cracks are experiencing punchout distress. Other statistics, such as an average and coefficient of variation of a series of deflection values, might be a better indicator of CRCP structural condition.
4) Since the usefulness of LTE in evaluating structural condition of CRCP is in question, research effort could be better utilized for more important aspects of CRCP design and performance, such as the evaluation of the effects of slab support and deflections on punchout.

D. Development of Transfer Function

1) The accuracy of a transfer function is a key to the reasonableness of any mechanistic-empirical pavement design procedures.
2) All three input variables for transfer function development – distress rate, traffic, and accumulated damage – are important. Among these 3, obtaining reasonable traffic data is the most challenging.
3) Efforts should be made to collect reasonably accurate traffic information.

Based on the findings of this study, the following recommendations are made:

1) LTE doesn't seem to be a good indicator of structural condition of transverse cracks and CRCP condition as long as an adequate amount of steel is used. So far, sufficient data have been collected on LTE. Consideration could be given to a different direction for this study, such as the effect of slab support on CRCP distresses.
2) The definition of punchout in TxDOT PMIS Rater's Manual can be further refined so that better information can be collected.

APPENDIX A

Deflections at 50-ft Interval Measured in FY10

Distance	3-US287-1	4-I40-1	5-I27-1		5-LP289-1		12-US290-1	12-US290-2		12-US290-3		19-US59-1		19-US59-2		24-I10-1		24-I10-2		24-I10-3		24-I10-4		25-I40-1		25-I40-2	
	Winter	Summer	Winter	Summer	Winter	Summer	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer
0	4.854	3.093	2.937	3.118	2.795	3.728	2.082	2.441	2.656	2.441	2.327	1.861	1.935	6.493	1.943	1.156	1.440	2.436	1.959	2.004	2.128	1.902	1.773	1.463	1.393	1.273	1.343
50	4.252	3.074	2.945	3.463	3.019	3.880	1.880	2.394	2.728	2.681	2.691	1.580	2.041	4.055	2.080	1.239	1.349	1.958	1.745	1.913	2.027	1.516	1.350	1.149	1.341	1.058	1.564
100	4.829	2.540	3.235	3.538	3.826	4.374	2.316	2.681	2.763	2.375	2.136	1.840	2.068	4.197	2.053	1.080	1.290	1.932	1.625	1.846	1.766	1.712	1.317	1.285	1.386	1.226	1.422
150	5.598	2.882	3.474	3.706	2.819	3.708	1.983	2.375	2.328	3.750	2.403	1.637	2.166	2.729	1.922	1.051	1.263	1.709	1.786	1.876	1.879	1.783	1.387	1.350	1.515	0.947	1.453
200	4.067	2.990	2.708	3.168	3.331	3.738	2.027	3.750	2.606	2.795	2.480	1.772	2.348	6.742	1.855	1.026	1.426	1.903	1.831	1.769	1.780	1.926	1.372	1.181	1.524	1.257	1.451
250	4.379	3.021	3.412	2.654	3.304	3.714	2.278	2.795	2.704	2.816	2.871	1.822	2.091	4.073	2.212	1.055	1.466	2.139	2.073	1.760	1.825	2.467	1.636	1.099	1.564	1.166	1.474
300	3.790	2.563	3.153	3.092	3.617	2.794	2.596	2.816	2.786	2.915	2.854	1.872	2.167	3.600	1.724	1.075	1.463	1.837	1.815	1.885	1.996	1.964	1.735	1.092	1.311	1.065	1.549
350	4.147	2.974	2.897	3.610	2.746	2.745	2.515	2.915	2.460	2.693	2.661	1.926	2.141	2.829	1.859	1.051	1.276	2.004	1.912	1.627	1.592	1.752	1.422	1.196	1.319	1.160	1.453
400	3.412	2.900	3.087	3.164	2.874	3.351	2.233	2.693	1.981	3.375	3.088	1.742	2.083	3.272	1.953	1.085	1.334	2.244	1.705	1.714	1.746	1.568	1.379	1.155	1.277	1.173	1.483
450	3.801	3.085	2.906	3.721	3.542	3.570	1.667	3.375	2.806	3.044	2.260	2.205	2.214	2.454	1.923	1.055	1.222	1.919	1.799	1.756	1.883	1.694	1.526	1.306	1.587	1.144	1.464
500	4.521	2.655	3.610	3.726	3.289	3.695	2.255	3.044	2.718	3.748	2.682	2.056	2.313	2.969	1.904	1.191	1.756	3.032	2.385	2.374	2.158	1.710	1.608	1.100	1.420	1.113	1.491
550	4.108	3.280	3.782	3.377	3.331	3.370	2.032	3.748	2.380	3.725	3.087	1.779	2.130	2.392	2.240	1.063	1.296	1.995	2.014	1.379	1.937	1.672	2.039	1.390	1.412	1.287	1.665
600	4.666	2.819	2.665	3.980	3.188	3.604	1.963	3.725	3.023	3.342	2.219	2.196	2.267	3.642	2.203	1.004	1.146	2.141	2.065	1.550	2.112	1.770	1.679	1.576	1.339	1.092	1.513
650	4.669	2.369	3.270	3.191	3.364	4.351	2.199	3.342	3.281	2.825	2.643	1.853	3.158	1.869	1.880	1.084	1.331	1.947	1.993	1.554	1.904	2.084	1.737	1.389	1.342	1.803	1.614
700	3.997	2.990	2.719	3.162	2.639	2.977	1.930	2.825	2.407	2.580	2.311	1.951	2.287	1.942	1.874	1.076	1.317	2.141	1.874	1.518	1.902	1.385	1.375	1.273	1.352	1.177	1.298
750	3.663	2.937	2.662	3.966	2.682	3.136	2.237	2.580	2.915	2.662	3.251	1.932	2.717	2.605	1.842	1.052	1.373	2.397	2.074	1.750	2.021	1.421	1.471	1.243	1.322	1.093	1.393
800	4.387	2.748	2.773	2.964	2.510	2.925	2.401	2.662	3.283	2.349	2.961	1.855	2.177	1.794	1.767	1.136	1.383	2.313	2.022	1.529	1.887	1.477	1.767	1.232	1.355	1.113	1.476
850	4.111	2.373	3.205	3.793	2.488	2.862	1.979	2.349	3.106	2.680	2.611	1.658	2.283	2.906	1.960	1.090	1.284	2.136	2.001	1.573	1.847	1.953	1.880	1.303	1.396	1.374	1.378
900	3.638	2.960	3.249	2.687	2.334	2.735	2.315	2.680	2.893	2.780	2.747	1.684	2.173	1.838	1.667	1.120	1.302	2.275	2.401	1.643	1.935	1.864	2.088	1.168	1.307	1.164	1.473
950	4.529	3.009	3.238	2.937	2.302	2.853	2.342	2.780	2.625	3.303	2.921	2.186	2.236	2.202	1.844	1.051	1.306	2.084	2.153	1.767	2.107	2.042	2.163	1.214	1.343	1.404	1.494
1000	3.898	2.886	3.212	3.116	2.731	3.047	2.529	3.303	3.366		2.380	1.636	1.929		1.886	1.066	1.266	1.173	2.098	1.654	1.935	2.390	2.211	1.132	1.422	1.115	1.509

APPENDIX B

Deflections at Transverse Cracks: FY 2010

SECTION I.D.	3-US287-1	WINTER TESTING				DATE		12/10/09		SUMMER TESTING				DATE		6/18/10	
LOCATION	US 287, N , MP 330					TIME								TIME			
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	4.3	4.2	3.5	4.0	1.9	1.7	1.4	107								
	DOWNSTREAM	4.3	3.8	3.2	4.1	1.7	1.5	1.2									
S-I-2	UPSTREAM	4.4	4.4	3.6	4.0	2.1	1.8	1.4	106								
	DOWNSTREAM	4.3	4.1	3.6	4.3	2.1	1.8	1.3									
M-I-1	UPSTREAM	4.2	4.2	3.5	3.9	2.0	1.7	1.3	109								
	DOWNSTREAM	4.2	3.9	3.2	4.2	1.9	1.7	1.4									
M-I-2	UPSTREAM	4.7	4.5	3.7	4.2	2.2	1.9	1.5	107								
	DOWNSTREAM	4.6	4.2	3.5	4.5	2.2	1.9	1.5									
L-I-1	UPSTREAM	4.4	4.1	3.5	4.5	2.1	1.9	1.5	93								
	DOWNSTREAM	5.8	4.5	3.7	4.2	2.2	1.9	1.5									
L-I-2	UPSTREAM	4.2	3.9	3.3	3.7	2.0	1.7	1.4	107								
	DOWNSTREAM	4.1	3.7	3.2	3.9	1.9	1.7	1.3									
S-II-1	UPSTREAM	4.7	4.5	3.8	4.4	2.3	2.1	1.7	103								
	DOWNSTREAM	4.7	4.4	3.8	4.5	2.3	2.1	1.7									
S-II-2	UPSTREAM	4.5	4.2	3.5	4.2	2.0	1.8	1.4	103								
	DOWNSTREAM	4.3	4.0	3.4	4.2	2.0	1.7	1.4									
M-II-1	UPSTREAM	4.9	4.7	3.9	4.4	2.3	2.0	1.6	108								
	DOWNSTREAM	4.9	4.4	3.7	4.8	2.3	2.0	1.7									
M-II-2	UPSTREAM	4.7	4.5	3.8	4.2	2.2	1.9	1.5	107								
	DOWNSTREAM	4.6	4.2	3.6	4.5	2.1	1.8	1.5									
L-II-1	UPSTREAM	4.9	4.7	4.0	4.5	2.6	2.4	1.9	106								
	DOWNSTREAM	4.9	4.5	3.9	4.8	2.5	2.3	1.9									
L-II-2	UPSTREAM	4.0	3.9	3.3	3.7	2.0	1.8	1.4	105								
	DOWNSTREAM	3.9	3.7	3.2	3.9	2.0	1.8	1.5									

SECTION I.D.	4-I40-1	WINTER TESTING				DATE				SUMMER TESTING				DATE		6/24/10	
LOCATION	IH 40, W, MP 33+287					TIME								TIME		11:10 A.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM									5.5	5.4	4.8	5.1	3.9	3.4	3.0	103
	DOWNSTREAM									5.4	5.3	4.9	5.3	3.9	3.4	3.0	
S-I-2	UPSTREAM									5.9	5.7	5.1	5.4	3.9	3.4	2.9	101
	DOWNSTREAM									5.9	5.6	5.0	5.4	3.9	3.1	2.8	
M-I-1	UPSTREAM									7.2	7.1	6.3	6.3	4.7	4.1	3.5	108
	DOWNSTREAM									7.3	6.8	6.1	6.9	4.7	4.1	3.5	
M-I-2	UPSTREAM									5.7	5.6	5.2	5.4	4.1	3.6	3.2	101
	DOWNSTREAM									5.7	5.5	5.1	5.5	4.2	3.6	3.2	
L-I-1	UPSTREAM									5.2	5.2	4.8	5.0	3.8	3.3	2.9	102
	DOWNSTREAM									5.4	5.2	4.8	5.1	3.8	3.4	3.0	
L-I-2	UPSTREAM									5.3	5.4	4.9	5.1	3.9	3.4	3.0	102
	DOWNSTREAM									5.6	5.3	4.9	5.3	3.9	3.4	3.0	
S-II-1	UPSTREAM									5.6	5.5	4.9	5.3	3.8	3.2	2.8	102
	DOWNSTREAM									5.6	5.4	5.0	5.4	3.8	3.2	2.8	
S-II-2	UPSTREAM									5.8	5.7	5.1	5.4	4.0	3.4	2.9	101
	DOWNSTREAM									5.9	5.7	5.3	5.5	4.0	3.4	2.9	
M-II-1	UPSTREAM									7.0	7.0	6.5	6.8	5.1	4.3	3.7	102
	DOWNSTREAM									7.0	6.9	6.3	6.9	4.9	4.2	3.7	
M-II-2	UPSTREAM									5.9	6.1	5.3	5.3	3.9	3.1	2.7	108
	DOWNSTREAM									6.2	5.7	5.1	5.8	3.8	3.2	2.7	
L-II-1	UPSTREAM									6.5	6.3	5.7	6.3	4.4	3.8	3.4	101
	DOWNSTREAM									5.9	5.9	5.4	6.1	4.3	3.8	3.4	
L-II-2	UPSTREAM									5.5	5.4	4.8	5.1	3.8	3.3	2.8	102
	DOWNSTREAM									5.9	5.3	4.8	5.3	3.8	3.3	2.9	

SECTION I.D.	5-I27-1	WINTER TESTING				DATE		12/16/09		SUMMER TESTING				DATE		6/17/10	
LOCATION	IH 27, S , MP 43+.22					TIME								TIME		2:13 P.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	3.1	3.1	2.7	3.1	1.8	1.9	1.7	100	3.4	3.2	2.8	3.22	1.8	1.9	1.7	100
	DOWNSTREAM	3.2	3.1	2.8	3.1	1.8	1.9	1.6		3.2	3.1	2.8	3.19	1.8	1.9	1.7	
S-I-2	UPSTREAM	2.9	2.6	2.3	2.6	1.4	1.4	1.3	101	3.2	2.9	2.5	2.98	1.5	1.5	1.3	100
	DOWNSTREAM	2.7	2.5	2.2	2.6	1.3	1.4	1.2		2.9	2.8	2.4	2.89	1.5	1.5	1.3	
M-I-1	UPSTREAM	3.3	3	2.7	3	1.7	1.9	1.7	100	3.5	3.3	2.9	3.28	1.8	2	1.7	100
	DOWNSTREAM	3.2	3	2.7	3.1	1.8	1.8	1.6		3.4	3.3	3	3.31	1.9	2	1.7	
M-I-2	UPSTREAM	3.3	3	2.7	3	1.6	1.7	1.5	100	3.7	3.4	2.9	3.4	1.7	1.8	1.5	100
	DOWNSTREAM	3.4	3	2.7	3.1	1.6	1.7	1.4		3.7	3.4	3	3.44	1.7	1.8	1.5	
L-I-1	UPSTREAM																
	DOWNSTREAM																
L-I-2	UPSTREAM																
	DOWNSTREAM																
S-II-1	UPSTREAM	2.9	2.7	2.4	2.7	1.4	1.5	1.3	100	3.3	3	2.6	2.99	1.6	1.6	1.4	100
	DOWNSTREAM	2.9	2.7	2.4	2.7	1.4	1.5	1.3		3.2	3	2.7	3.05	1.6	1.6	1.4	
S-II-2	UPSTREAM	2.7	2.5	2.2	2.5	1.4	1.4	1.2	100	2.8	2.6	2.4	2.63	1.5	1.5	1.3	98
	DOWNSTREAM	2.8	2.5	2.2	2.5	1.4	1.4	1.3		2.8	2.8	2.5	2.67	1.5	1.5	1.3	
M-II-1	UPSTREAM	3.2	3	2.5	2.9	1.5	1.5	1.3	103	3.5	3.2	2.7	3.25	1.6	1.6	1.4	101
	DOWNSTREAM	3.1	2.9	2.5	3	1.5	1.5	1.3		3.5	3.1	2.7	3.2	1.6	1.6	1.3	
M-II-2	UPSTREAM	3.2	3	2.5	2.9	1.5	1.5	1.3	103	3.6	3.3	2.9	3.32	1.7	1.7	1.4	101
	DOWNSTREAM	3.2	2.9	2.5	3	1.5	1.5	1.3		3.7	3.3	2.9	3.37	1.7	1.7	1.4	
L-II-1	UPSTREAM																
	DOWNSTREAM																
L-II-2	UPSTREAM																
	DOWNSTREAM																

SECTION I.D.	5-LP289-1	WINTER TESTING				DATE		12/16/09		SUMMER TESTING				DATE		6/17/10	
LOCATION	LOOP 289, N					TIME								TIME		10:06 A.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	4.1	3.8	3.3	4.0	2.0	2.0	1.8	100	4.0	3.8	3.5	3.9	2.1	2.2	1.9	100
	DOWNSTREAM	3.9	3.6	3.2	3.8	2.0	2.0	1.8		4.0	3.8	3.3	3.8	2.2	2.2	1.8	
S-I-2	UPSTREAM	3.7	3.3	3.0	3.3	1.9	2.0	1.8	102	3.1	2.9	2.7	3.0	1.9	2.1	1.8	100
	DOWNSTREAM	3.5	3.3	2.9	3.4	1.9	2.0	1.8		3.1	2.9	2.7	3.0	1.9	2.1	1.9	
M-I-1	UPSTREAM	3.6	3.5	3.2	3.6	2.1	2.1	1.8	100	4.3	4.0	3.6	4.0	2.2	2.3	2.0	100
	DOWNSTREAM	3.7	3.6	3.2	3.6	2.0	2.0	1.8		4.1	3.9	3.6	4.0	2.3	2.3	1.9	
M-I-2	UPSTREAM	3.7	3.6	3.2	3.6	1.9	2.0	1.8	101	4.0	3.8	3.4	3.8	2.0	2.1	1.9	101
	DOWNSTREAM	3.8	3.5	3.1	3.6	1.9	2.0	1.8		3.9	3.7	3.3	3.8	2.0	2.1	1.8	
L-I-1	UPSTREAM	3.0	2.7	2.5	2.9	1.7	1.8	1.6	100	2.7	2.7	2.4	2.7	1.6	1.8	1.7	100
	DOWNSTREAM	2.8	2.6	2.4	2.8	1.7	1.8	1.6		2.8	2.7	2.4	2.7	1.7	1.9	1.7	
L-I-2	UPSTREAM	3.1	2.9	2.6	2.9	1.8	1.8	1.6	102	2.9	2.7	2.4	2.7	1.6	1.9	1.7	101
	DOWNSTREAM	3.4	2.8	2.5	2.9	1.7	1.8	1.6		2.8	2.6	2.4	2.7	1.7	1.8	1.7	
S-II-1	UPSTREAM	3.4	3.1	2.8	3.1	1.9	1.8	1.6	100	3.5	3.4	3.1	3.4	1.9	2.0	1.7	100
	DOWNSTREAM	3.4	3.1	2.8	3.1	1.9	1.8	1.5		3.6	3.4	3.0	3.4	1.9	2.0	1.6	
S-II-2	UPSTREAM	2.7	2.5	2.2	2.5	1.6	1.5	1.3	101	2.8	2.7	2.4	2.8	1.5	1.6	1.3	100
	DOWNSTREAM	2.6	2.5	2.2	2.5	1.6	1.5	1.3		2.8	2.7	2.4	2.7	1.5	1.5	1.4	
M-II-1	UPSTREAM	3.3	3.1	2.8	3.2	1.9	1.9	1.7	99	3.0	2.8	2.5	2.9	1.6	1.6	1.4	100
	DOWNSTREAM	3.3	3.1	2.8	3.1	1.9	1.9	1.7		3.0	2.8	2.5	2.9	1.6	1.6	1.4	
M-II-2	UPSTREAM	2.7	2.5	2.3	2.6	1.6	1.5	1.3	100	2.7	2.5	2.3	2.6	1.6	1.5	1.3	100
	DOWNSTREAM	2.7	2.5	2.3	2.6	1.6	1.5	1.3		2.7	2.5	2.3	2.6	1.6	1.5	1.3	
L-II-1	UPSTREAM	3.0	3.1	2.8	3.1	1.9	1.9	1.7	99	3.6	3.5	3.2	3.5	2.1	2.1	1.8	100
	DOWNSTREAM	3.1	3.1	2.8	3.1	2.0	2.0	1.7		3.8	3.5	3.2	3.5	2.1	2.2	1.9	
L-II-2	UPSTREAM	2.9	2.7	2.4	2.7	1.7	1.6	1.4	101	2.9	2.7	2.4	2.7	1.6	1.6	1.4	100
	DOWNSTREAM	2.9	2.7	2.4	2.7	1.6	1.6	1.4		2.9	2.7	2.5	2.8	1.5	1.6	1.4	

SECTION I.D.	12-US290-1	WINTER TESTING				DATE		1/17/10		SUMMER TESTING				DATE		8/19/10	
LOCATION	US 290, E					TIME								TIME			
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
J-1	UPSTREAM	4.6	4.3	3.8	2.9	2.4	1.9	1.6	109	3.5	3.2	3.0	2.3	1.9	1.6	1.4	105
	DOWNSTREAM	4.7	4.2	4.0	2.9	2.4	1.9	1.6		3.6	3.2	3.0	2.4	2.0	1.6	1.4	
C-1	UPSTREAM	4.5	4.0	3.9	2.9	2.4	2.0	1.7	106	3.6	3.2	3.1	2.4	2.0	1.6	1.4	105
	DOWNSTREAM	4.5	4.0	3.8	2.9	2.5	2.0	1.7		3.5	3.2	3.0	2.3	2.0	1.6	1.4	
J-2	UPSTREAM	4.9	4.4	4.0	3.0	2.5	2.0	1.7	109	3.7	3.3	3.1	2.4	2.1	1.7	1.5	105
	DOWNSTREAM	4.9	4.3	4.2	2.9	2.4	2.0	1.7		3.8	3.4	3.2	2.5	2.1	1.8	1.6	
C-2	UPSTREAM	4.5	4.1	3.9	3.0	2.5	2.1	1.9	107	3.8	3.4	3.3	2.5	2.3	1.9	1.7	105
	DOWNSTREAM	4.4	4.0	3.8	2.9	2.5	2.1	1.8		3.7	3.4	3.2	2.6	2.2	1.9	1.7	
J-3	UPSTREAM	3.7	3.3	3.1	2.2	1.8	1.4	1.1	108	3.1	2.8	2.6	1.9	1.6	1.3	1.1	106
	DOWNSTREAM	3.7	3.2	3.0	2.1	1.7	1.4	1.1		3.2	2.8	2.6	2.0	1.6	1.3	1.1	
C-3	UPSTREAM	3.5	3.1	2.8	2.2	1.8	1.4	1.2	107	3.1	2.8	2.5	2.0	1.6	1.3	1.2	107
	DOWNSTREAM	3.7	3.2	3.0	2.2	1.8	1.5	1.2		3.2	2.8	2.6	2.0	1.7	1.4	1.2	
J-4	UPSTREAM	3.8	3.4	3.1	2.3	1.9	1.6	1.3	108	3.3	3.0	2.9	2.2	1.9	1.5	1.3	105
	DOWNSTREAM	3.8	3.3	3.2	2.3	2.0	1.6	1.4		3.4	3.0	2.9	2.3	1.9	1.5	1.3	
C-4	UPSTREAM	3.4	3.1	2.9	2.2	1.8	1.5	1.3	106	3.1	2.7	2.6	2.0	1.7	1.4	1.2	105
	DOWNSTREAM	3.4	3.0	2.8	2.1	1.8	1.5	1.3		3.0	2.7	2.6	2.0	1.7	1.4	1.2	
J-5	UPSTREAM	3.2	2.7	2.5	1.8	1.5	1.2	1.1	109	2.7	2.4	2.3	1.8	1.4	1.1	1.1	103
	DOWNSTREAM	3.1	2.7	2.6	1.9	1.6	1.3	1.1		2.8	2.5	2.3	1.8	1.5	1.2	1.1	
C-5	UPSTREAM	3.6	3.3	3.1	2.4	2.0	1.7	1.4	106	3.6	3.3	3.1	2.4	2.1	1.8	1.5	105
	DOWNSTREAM	3.6	3.3	3.0	2.4	2.1	1.7	1.5		3.6	3.3	3.1	2.5	2.2	1.7	1.5	

SECTION I.D.	12-US290-2	WINTER TESTING				DATE		1/17/10		SUMMER TESTING				DATE		8/19/10	
LOCATION	US 290, W					TIME								TIME			
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	4.2	3.9	3.5	2.7	2.3	1.8	1.5	106	4.9	4.3	4.1	3.1	2.5	2.0	1.7	105
	DOWNSTREAM	4.2	3.8	3.6	2.8	2.3	1.8	1.5		4.7	4.3	4.1	3.1	2.6	2.0	1.7	
S-I-2	UPSTREAM	5.0	4.6	4.1	3.2	2.7	2.2	1.8	106	4.8	4.4	4.1	3.3	2.7	2.1	1.7	104
	DOWNSTREAM	5.0	4.5	4.2	3.2	2.7	2.2	1.8		4.9	4.5	4.2	3.3	2.7	2.1	1.7	
M-I-1	UPSTREAM	3.6	3.4	3.0	2.4	2.0	1.7	1.4	107	4.8	4.4	4.1	3.2	2.7	2.1	1.7	105
	DOWNSTREAM	3.7	3.3	3.1	2.4	2.1	1.7	1.4		4.9	4.5	4.2	3.3	2.7	2.1	1.7	
M-I-2	UPSTREAM	5.6	5.3	4.7	3.4	2.7	2.1	1.7	110	3.9	3.6	3.4	2.6	2.1	1.7	1.4	107
	DOWNSTREAM	5.7	4.9	5.0	3.3	2.7	2.0	1.6		3.9	3.6	3.4	2.6	2.1	1.7	1.4	
L-I-1	UPSTREAM	3.7	3.4	3.1	2.4	2.1	1.7	1.4	106								
	DOWNSTREAM	3.7	3.4	3.1	2.5	2.1	1.7	1.4									
L-I-2	UPSTREAM	4.5	4.2	3.9	2.9	2.5	2.0	1.7	107								
	DOWNSTREAM	4.5	4.0	3.9	2.9	2.5	2.0	1.7									
S-II-1	UPSTREAM									5.1	4.7	4.3	3.4	2.8	2.2	1.8	105
	DOWNSTREAM									5.2	4.8	4.4	3.4	2.8	2.2	1.8	
S-II-2	UPSTREAM									6.0	5.5	5.1	4.0	3.3	2.5	2.1	106
	DOWNSTREAM									6.0	5.6	5.2	4.0	3.3	2.6	2.1	
M-II-1	UPSTREAM	5.3	4.9	4.4	3.6	3.1	2.4	2.0	105	4.2	3.9	3.6	2.8	2.3	1.8	1.5	106
	DOWNSTREAM	5.4	5.0	4.6	3.7	3.0	2.4	2.0		4.2	3.8	3.6	2.8	2.3	1.8	1.4	
M-II-2	UPSTREAM									5.1	4.7	4.4	3.4	2.9	2.3	1.8	105
	DOWNSTREAM									5.1	4.7	4.4	3.4	2.8	2.2	1.8	
L-II-1	UPSTREAM	5.6	5.1	4.9	3.6	3.0	2.3	1.9	106	4.5	4.1	3.9	2.9	2.3	1.9	1.6	107
	DOWNSTREAM	5.5	5.0	4.8	3.6	3.0	2.3	1.9		4.5	4.1	3.8	2.9	2.4	1.9	1.6	
L-II-2	UPSTREAM									4.7	4.3	4.1	3.1	2.6	2.0	1.7	106
	DOWNSTREAM									4.7	4.3	4.0	3.1	2.6	2.0	1.7	

SECTION I.D.	19-US59-1	WINTER TESTING				DATE		3/29/10		SUMMER TESTING				DATE		9/14/10	
LOCATION	US 59, S, MP 218-.4					TIME								TIME		11:13 A.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	2.0	1.6	1.4	1.7	1.1	0.9	0.8	99	2.3	1.9	1.6	1.9	1.2	1.1	0.9	103
	DOWNSTREAM	2.1	1.5	1.4	1.6	1.1	0.9	0.8		2.4	1.8	1.6	2.0	1.2	1.0	0.9	
S-I-2	UPSTREAM	2.5	1.6	1.5	1.6	1.0	0.9	0.8	100	2.2	1.8	1.6	1.9	1.2	1.1	0.9	101
	DOWNSTREAM	2.0	1.6	1.4	1.7	1.0	0.9	0.8		2.4	1.8	1.6	1.9	1.2	1.1	1.0	
M-I-1	UPSTREAM	2.0	1.5	1.3	1.5	1.0	0.8	0.8	102	2.2	1.9	1.6	1.9	1.3	1.1	0.9	104
	DOWNSTREAM	1.9	1.4	1.3	1.5	1.0	0.9	0.8		2.3	1.8	1.6	2.0	1.2	1.1	0.9	
M-I-2	UPSTREAM	1.8	1.4	1.2	1.3	0.9	0.8	0.7	102	2.5	1.9	1.6	2.0	1.3	1.1	0.9	106
	DOWNSTREAM	2.6	1.3	1.2	1.3	0.9	0.8	0.7		2.8	1.9	1.6	2.1	1.2	1.1	0.9	
L-I-1	UPSTREAM	2.0	1.3	1.2	1.3	0.9	0.8	0.7	102	2.3	1.7	1.5	1.8	1.2	1.1	1.0	105
	DOWNSTREAM	2.1	1.3	1.2	1.3	0.9	0.8	0.8		2.4	1.7	1.6	1.9	1.2	1.1	1.0	
L-I-2	UPSTREAM	2.1	1.6	1.4	1.6	1.1	0.9	0.8	102	2.5	1.8	1.6	2.0	1.2	1.1	0.9	106
	DOWNSTREAM	2.0	1.6	1.4	1.6	1.1	0.9	0.8		2.4	1.7	1.6	2.0	1.3	1.1	0.9	
S-II-1	UPSTREAM	3.6	1.7	1.5	3.1	1.1	0.9	0.8	78	2.7	2.1	1.9	2.2	1.5	1.3	1.1	96
	DOWNSTREAM	2.1	1.7	1.5	1.7	1.1	0.9	0.8									
S-II-2	UPSTREAM	2.0	1.6	1.4	1.5	1.0	0.9	0.8	102	2.2	1.6	1.5	1.8	1.1	1.0	0.8	100
	DOWNSTREAM	1.9	1.5	1.4	1.6	1.0	0.9	0.8		2.2	1.8	1.6	1.9	1.1	0.9	0.8	
M-II-1	UPSTREAM	1.9	1.5	1.3	1.5	1.0	0.9	0.8	101	2.6	2.0	1.8	2.2	1.4	1.2	1.0	103
	DOWNSTREAM	2.6	1.5	1.3	1.5	1.0	0.9	0.8		2.5	1.9	1.8	2.1	1.3	1.1	1.0	
M-II-2	UPSTREAM	2.1	1.6	1.4	1.5	1.0	0.9	0.8	102	2.4	1.9	1.7	2.0	1.3	1.2	0.9	102
	DOWNSTREAM	2.0	1.5	1.4	1.6	1.0	0.9	0.8		2.4	1.9	1.7	2.0	1.4	1.2	1.1	
L-II-1	UPSTREAM	2.2	1.7	1.4	1.6	1.1	0.9	0.7	104	2.2	1.5	1.4	1.8	1.1	0.9	0.8	97
	DOWNSTREAM	3.0	1.6	1.4	1.6	1.1	0.9	0.8		2.1	1.7	1.5	1.8	1.1	0.9	0.8	
L-II-2	UPSTREAM	2.3	1.5	1.4	1.5	1.1	0.9	0.8	103								
	DOWNSTREAM	2.1	1.5	1.3	1.6	1.0	0.9	0.8									

SECTION I.D.	19-US59-2	WINTER TESTING				DATE		3/29/10		SUMMER TESTING				DATE		9/14/10	
LOCATION	US 59, S, MP 218-.4					TIME								TIME		4:00 P.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	2.2	1.5	1.4	1.6	1.1	0.9	0.8	101	2.0	1.5	1.4	1.7	0.9	0.9	0.7	88
	DOWNSTREAM	3.8	1.4	1.4	1.6	1.1	0.9	0.8									
S-I-2	UPSTREAM	2.4	1.0	1.3	1.4	0.9	0.8	0.7	96	1.8	1.4	1.2	1.55	0.9	0.7	0.6	87
	DOWNSTREAM	2.3	1.2	1.3	1.5	1.0	0.8	0.7									
M-I-1	UPSTREAM	3.6	1.3	1.4	1.4	0.9	0.8	0.7	101	1.9	1.4	1.3	1.5	1.0	0.9	0.8	101
	DOWNSTREAM	2.6	1.2	1.3	1.4	0.9	0.8	0.7		1.8	1.3	1.2	1.4	1.0	0.9	0.7	
M-I-2	UPSTREAM	2.5	1.4	1.5	1.5	1.1	0.9	0.7	100	2.0	1.5	1.4	1.6	1.1	1.0	0.8	101
	DOWNSTREAM	2.4	1.5	1.4	1.6	1.1	1.0	0.9		2.1	1.6	1.4	1.7	1.1	1.0	0.8	
L-I-1	UPSTREAM	2.7	1.3	1.4	1.6	1.0	0.8	0.7	97	2.0	1.5	1.3	1.6	1.0	0.8	0.6	104
	DOWNSTREAM	2.2	1.4	1.4	1.6	1.0	0.9	0.8		2.2	1.5	1.3	1.7	1.0	0.8	0.7	
L-I-2	UPSTREAM	2.3	0.9	1.3	1.3	0.8	0.7	0.6	106	1.7	1.3	1.2	1.4	0.9	0.8	0.7	101
	DOWNSTREAM	2.0	1.0	1.2	1.4	0.9	0.8	0.7		1.8	1.3	1.1	1.4	0.9	0.7	0.7	
S-II-1	UPSTREAM	2.3	1.8	1.7	1.8	1.3	1.0	0.9	99	2.2	1.7	1.5	1.9	1.1	1.0	0.8	99
	DOWNSTREAM	2.3	1.9	1.7	1.8	1.2	1.0	0.9		2.2	1.7	1.5	1.8	1.2	1.0	0.8	
S-II-2	UPSTREAM	1.7	1.3	1.2	1.3	0.9	0.7	0.6	97	1.8	1.3	1.2	1.4	0.9	0.8	0.7	95
	DOWNSTREAM	1.8	1.3	1.2	1.2	0.8	0.7	0.6		1.8	1.4	1.2	1.4	0.8	0.8	0.7	
M-II-1	UPSTREAM	2.7	1.7	1.5	1.6	1.1	0.9	0.8	102	2.1	1.6	1.4	1.7	1.1	0.9	0.8	101
	DOWNSTREAM	2.5	1.7	1.5	1.7	1.1	0.9	0.7		2.1	1.5	1.4	1.7	1.0	0.9	0.8	
M-II-2	UPSTREAM	2.1	1.3	1.2	1.4	0.9	0.8	0.7	99	1.7	1.3	1.2	1.4	0.9	0.7	0.6	100
	DOWNSTREAM	1.9	1.4	1.2	1.4	0.9	0.7	0.6		1.7	1.3	1.2	1.4	0.9	0.8	0.6	
L-II-1	UPSTREAM	2.5	1.5	1.3	1.5	0.9	0.8	0.7	100	2.0	1.5	1.3	1.6	1.0	0.8	0.7	100
	DOWNSTREAM	2.2	1.5	1.3	1.5	1.0	0.8	0.7		1.9	1.4	1.3	1.5	1.0	0.8	0.7	
L-II-2	UPSTREAM	2.3	1.4	1.3	1.4	1.0	0.8	0.7	100	1.7	1.3	1.2	1.4	0.9	0.7	0.6	102
	DOWNSTREAM	1.6	1.4	1.3	1.4	0.9	0.8	0.7		1.7	1.3	1.2	1.4	0.9	0.7	0.6	

SECTION I.D.	24-I10-1	WINTER TESTING				DATE		2/17/10		SUMMER TESTING				DATE		8/30/10	
LOCATION	IH 10, E, MP 36+.3					TIME		10:42 A.M.						TIME		9:00 A.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	1.7	1.6	1.4	1.6	1.1	1.0	0.9	99	1.9	1.7	1.6	1.7	1.1	1.2	1.0	100
	DOWNSTREAM	1.7	1.6	1.4	1.6	1.2	1.0	0.9		2.0	1.8	1.6	1.8	1.3	1.2	1.0	
S-I-2	UPSTREAM	1.6	1.5	1.3	1.4	1.1	1.0	0.8	100	1.9	1.7	1.6	1.76	1.3	1.3	1.1	100
	DOWNSTREAM	1.6	1.5	1.4	1.5	1.1	1.0	0.9		1.8	1.7	1.6	1.8	1.3	1.3	1.1	
M-I-1	UPSTREAM	1.7	1.6	1.4	1.6	1.1	1.0	0.9	100	2.0	1.8	1.6	1.8	1.3	1.3	1.1	105
	DOWNSTREAM	1.8	1.6	1.4	1.6	1.1	1.0	0.9		1.9	1.5	1.6	1.7	1.2	1.1	1.0	
M-I-2	UPSTREAM	1.6	1.5	1.4	1.4	1.1	1.0	0.9	99	1.7	1.6	1.5	1.7	1.2	1.2	1.0	100
	DOWNSTREAM	1.6	1.5	1.4	1.4	1.1	1.0	0.8		1.8	1.7	1.6	1.7	1.2	1.2	1.0	
L-I-1	UPSTREAM	1.6	1.4	1.3	1.4	1.1	1.0	0.9	99	1.8	1.6	1.6	1.7	1.2	1.2	1.0	99
	DOWNSTREAM	1.6	1.5	1.4	1.5	1.1	1.0	0.8		1.9	1.7	1.6	1.7	1.2	1.2	1.0	
L-I-2	UPSTREAM	1.6	1.5	1.3	1.5	1.1	1.0	0.9	99	1.9	1.7	1.6	1.7	1.2	1.2	1.0	100
	DOWNSTREAM	1.7	1.5	1.4	1.5	1.1	1.0	0.9		1.8	1.7	1.6	1.7	1.2	1.2	1.0	
S-II-1	UPSTREAM	1.6	1.5	1.4	1.5	1.1	1.0	0.8	101	1.8	1.6	1.5	1.8	1.2	1.2	1.1	97
	DOWNSTREAM	1.7	1.5	1.3	1.5	1.1	1.0	0.8		1.7	1.7	1.6	1.7	1.2	1.2	1.0	
S-II-2	UPSTREAM	1.6	1.4	1.4	1.5	1.0	0.9	0.8	98	1.8	1.6	1.5	1.7	1.2	1.2	1.0	100
	DOWNSTREAM	1.6	1.4	1.3	1.5	1.1	0.9	0.9		1.7	1.6	1.5	1.7	1.2	1.2	1.0	
M-II-1	UPSTREAM	1.5	1.5	1.3	1.5	1.1	1.0	0.8	99	1.7	1.6	1.5	1.7	1.1	1.2	0.9	99
	DOWNSTREAM	1.6	1.5	1.4	1.5	1.2	1.1	1.0		1.7	1.7	1.6	1.7	1.2	1.2	1.0	
M-II-2	UPSTREAM	1.6	1.4	1.3	1.4	1.1	1.0	0.8	98	1.8	1.5	1.5	1.6	1.1	1.2	0.9	98
	DOWNSTREAM	1.6	1.4	1.3	1.4	1.1	1.0	0.8		1.7	1.6	1.5	1.6	1.1	1.2	1.0	
L-II-1	UPSTREAM	1.6	1.4	1.3	1.4	1.0	0.9	0.8	100	1.6	1.6	1.5	1.7	1.2	1.2	1.0	100
	DOWNSTREAM	1.6	1.4	1.4	1.4	1.1	1.0	0.8		1.8	1.6	1.5	1.7	1.2	1.2	1.0	
L-II-2	UPSTREAM	1.6	1.4	1.4	1.5	1.1	0.9	0.8	100	1.7	1.6	1.5	1.7	1.1	1.2	1.0	99
	DOWNSTREAM	1.6	1.4	1.3	1.5	1.0	1.0	0.8		1.7	1.6	1.5	1.6	1.1	1.2	1.0	

SECTION I.D.	24-I10-2	WINTER TESTING				DATE		2/17/10		SUMMER TESTING				DATE		8/30/10	
LOCATION	IH 10, E, MP 39+.3					TIME		3:06 P.M.						TIME		11:00 A.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM																
	DOWNSTREAM																
S-I-2	UPSTREAM																
	DOWNSTREAM																
M-I-1	UPSTREAM	4.1	3.9	3.5	3.7	2.6	2.3	1.9	105	3.4	3.4	3.4	3.5	2.6	2.6	2.2	101
	DOWNSTREAM	4.1	3.7	3.3	3.9	2.6	2.2	1.9		3.6	3.4	3.3	3.5	2.5	2.5	2.2	
M-I-2	UPSTREAM	3.9	3.8	3.3	3.6	2.4	2.1	1.7	106	3.2	3.0	2.9	3.1	2.3	2.3	1.9	102
	DOWNSTREAM	3.9	3.5	3.1	3.8	2.3	2.0	1.7		3.1	3.0	2.9	3.2	2.3	2.3	1.9	
L-I-1	UPSTREAM	3.3	3.2	2.8	3.0	2.1	1.8	1.5	107	2.6	2.5	2.4	2.5	1.8	1.8	1.6	98
	DOWNSTREAM	3.4	3.0	2.7	3.2	2.0	1.7	1.4		2.7	2.4	2.3	2.4	1.8	1.9	1.6	
L-I-2	UPSTREAM	3.6	3.4	2.9	3.2	2.2	1.9	1.5	107	2.7	2.5	2.4	2.6	1.8	1.8	1.5	102
	DOWNSTREAM	3.6	3.1	2.8	3.3	2.1	1.8	1.5		2.5	2.5	2.3	2.6	1.8	1.7	1.4	
S-II-1	UPSTREAM																
	DOWNSTREAM																
S-II-2	UPSTREAM																
	DOWNSTREAM																
M-II-1	UPSTREAM	2.8	2.6	2.3	2.5	1.7	1.5	1.2	104	2.9	2.7	2.7	2.7	2.0	2.0	1.6	102
	DOWNSTREAM	2.7	2.5	2.2	2.6	1.7	1.5	1.2		2.9	2.7	2.6	2.8	2.0	2.0	1.6	
M-II-2	UPSTREAM	3.4	3.3	3.0	3.1	2.3	2.1	1.7	102	3.1	2.9	2.8	2.9	2.2	2.2	1.8	101
	DOWNSTREAM	3.6	3.3	3.1	3.3	2.4	2.1	1.7		3.0	2.9	2.8	3.0	2.2	2.1	1.7	
L-II-1	UPSTREAM	3.3	3.0	2.8	3.2	2.1	1.8	1.5	94	2.7	2.5	2.5	2.6	1.8	1.9	1.5	99
	DOWNSTREAM	3.3	3.1	2.7	2.9	2.1	1.8	1.5		2.7	2.5	2.4	2.6	1.9	1.8	1.6	
L-II-2	UPSTREAM	3.3	3.1	2.7	3.0	2.1	1.8	1.5	103	3.1	2.9	2.8	3.2	2.1	2.0	1.7	97
	DOWNSTREAM	3.2	2.9	2.7	3.0	2.0	1.8	1.5		3.3	2.9	2.7	3.0	2.1	2.0	1.7	

SECTION I.D.	24-I10-3	WINTER TESTING				DATE		2/18/10		SUMMER TESTING				DATE		8/30/10	
LOCATION	IH 10, W, MP 45+.1					TIME		10:07 A.M.						TIME		2:00 P.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	2.8	2.5	2.2	2.5	1.6	1.3	1.1	100	2.8	2.5	2.3	2.6	1.7	1.6	1.2	100
	DOWNSTREAM	2.7	2.4	2.2	2.4	1.6	1.3	1.1		2.8	2.5	2.3	2.6	1.6	1.5	1.2	
S-I-2	UPSTREAM	2.4	2.3	2.1	2.3	1.6	1.4	1.2	99	2.3	2.3	2.1	2.31	1.6	1.6	1.3	101
	DOWNSTREAM	2.5	2.4	2.1	2.3	1.7	1.4	1.2		2.4	2.3	2.2	2.3	1.6	1.6	1.3	
M-I-1	UPSTREAM	2.8	2.6	2.3	2.6	1.7	1.4	1.2	100	2.7	2.6	2.4	2.7	1.8	1.7	1.4	100
	DOWNSTREAM	2.7	2.5	2.3	2.5	1.6	1.4	1.2		2.8	2.6	2.4	2.7	1.8	1.7	1.4	
M-I-2	UPSTREAM	2.3	2.1	1.9	2.0	1.3	1.2	1.0	98	2.2	2.1	1.9	2.1	1.4	1.4	1.2	101
	DOWNSTREAM	2.4	2.2	1.9	2.1	1.4	1.2	1.0		2.2	2.1	1.9	2.1	1.4	1.4	1.1	
L-I-1	UPSTREAM	2.4	2.1	1.9	2.1	1.4	1.2	1.1	100	2.0	2.0	1.8	2.0	1.4	1.3	1.1	101
	DOWNSTREAM	2.4	2.2	1.9	2.2	1.4	1.2	1.0		2.2	2.0	1.8	2.0	1.3	1.3	1.1	
L-I-2	UPSTREAM	2.4	2.2	2.0	2.2	1.5	1.3	1.1	102	2.3	2.2	2.0	2.2	1.5	1.4	1.2	101
	DOWNSTREAM	2.4	2.2	1.9	2.2	1.4	1.2	1.0		2.3	2.1	2.0	2.2	1.5	1.4	1.2	
S-II-1	UPSTREAM																
	DOWNSTREAM																
S-II-2	UPSTREAM																
	DOWNSTREAM																
M-II-1	UPSTREAM	2.3	2.2	2.0	2.2	1.5	1.3	1.1	101	2.5	2.4	2.3	2.5	1.7	1.7	1.4	100
	DOWNSTREAM	2.4	2.2	2.0	2.2	1.5	1.3	1.1		2.6	2.5	2.4	2.5	1.8	1.8	1.5	
M-II-2	UPSTREAM	2.9	2.8	2.6	2.7	2.0	1.7	1.4	103	2.8	2.8	2.7	2.9	2.1	2.0	1.7	100
	DOWNSTREAM	3.1	2.8	2.5	2.8	1.9	1.7	1.4		2.7	2.8	2.7	2.9	2.1	2.0	1.7	
L-II-1	UPSTREAM	2.5	2.3	2.1	2.3	1.6	1.4	1.2	101	2.8	2.7	2.6	2.7	2.0	1.9	1.6	101
	DOWNSTREAM	2.5	2.3	2.0	2.3	1.6	1.3	1.1		2.7	2.7	2.6	2.8	1.9	1.9	1.6	
L-II-2	UPSTREAM	2.7	2.4	2.2	2.4	1.7	1.4	1.2	100	2.6	2.4	2.3	2.5	1.8	1.7	1.4	100
	DOWNSTREAM	2.7	2.4	2.2	2.5	1.7	1.4	1.2		2.5	2.5	2.3	2.5	1.8	1.8	1.5	

SECTION I.D.	24-I10-4	WINTER TESTING				DATE		2/18/10		SUMMER TESTING				DATE		8/31/10	
LOCATION	IH 10, W, MP 85					TIME		1:42 P.M.						TIME		10:04 A.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	2.6	2.4	2.1	2.4	1.4	1.2	1.0	101	2.0	1.7	1.6	1.8	1.1	1.0	0.9	101
	DOWNSTREAM	2.6	2.3	2.0	2.4	1.5	1.2	0.9		2.1	1.7	1.5	1.8	1.1	1.1	0.9	
S-I-2	UPSTREAM	2.2	2.0	1.8	2.0	1.2	1.1	0.9	100	2.0	1.6	1.4	1.6	1.0	1.0	0.8	100
	DOWNSTREAM	2.2	2.0	1.7	2.0	1.3	1.1	0.9		1.9	1.6	1.5	1.6	1.0	1.0	0.8	
M-I-1	UPSTREAM	2.6	2.3	2.2	2.3	1.5	1.4	1.0	99	1.9	1.7	1.6	1.8	1.2	1.1	0.9	100
	DOWNSTREAM	2.6	2.5	2.2	2.4	1.6	1.3	1.1		2.0	1.8	1.6	1.8	1.2	1.2	1.0	
M-I-2	UPSTREAM	2.5	2.3	2.1	2.2	1.4	1.2	0.9	102	1.9	1.6	1.5	1.8	1.1	1.1	0.8	97
	DOWNSTREAM	2.5	2.2	1.9	2.2	1.4	1.1	0.9		1.9	1.6	1.5	1.7	1.0	1.0	0.8	
L-I-1	UPSTREAM	2.3	2.0	1.7	2.1	1.2	1.0	0.9	101	1.7	1.6	1.4	1.6	1.0	1.0	0.9	101
	DOWNSTREAM	2.2	1.9	1.6	2.0	1.2	1.0	0.8		1.7	1.5	1.4	1.6	1.0	1.0	0.9	
L-I-2	UPSTREAM	2.3	2.1	1.8	2.1	1.3	1.1	0.9	101	1.9	1.7	1.5	1.7	1.1	1.1	1.0	100
	DOWNSTREAM	2.4	2.2	1.9	2.2	1.3	1.1	1.0		2.0	1.6	1.5	1.7	1.1	1.1	1.0	
S-II-1	UPSTREAM	2.9	2.7	2.4	2.6	1.7	1.4	1.1	101	2.5	2.2	2.0	2.3	1.5	1.4	1.2	101
	DOWNSTREAM	2.9	2.7	2.3	2.7	1.6	1.3	1.1		2.5	2.2	2.0	2.3	1.5	1.4	1.2	
S-II-2	UPSTREAM	2.5	2.3	2.0	2.3	1.4	1.3	1.0	100	2.3	2.1	1.9	2.1	1.4	1.4	1.2	99
	DOWNSTREAM	2.5	2.3	2.0	2.3	1.5	1.2	1.1		2.2	2.0	1.9	2.1	1.4	1.4	1.1	
M-II-1	UPSTREAM	2.5	2.3	2.0	2.3	1.5	1.3	1.0	101	2.4	2.1	1.9	2.1	1.4	1.4	1.2	100
	DOWNSTREAM	2.5	2.3	2.0	2.3	1.5	1.3	1.1		2.2	2.0	1.9	2.1	1.4	1.4	1.2	
M-II-2	UPSTREAM	3.0	2.8	2.6	2.6	2.0	1.8	1.4	100	3.4	3.2	3.0	3.2	2.4	2.3	2.0	101
	DOWNSTREAM	3.0	2.8	2.5	2.7	1.9	1.8	1.4		3.6	3.2	3.0	3.3	2.3	2.3	2.0	
L-II-1	UPSTREAM	3.3	3.1	2.7	3.1	1.9	1.6	1.3	102	2.7	2.5	2.3	2.6	1.7	1.7	1.4	100
	DOWNSTREAM	3.3	2.9	2.5	3.0	1.8	1.5	1.3		2.7	2.5	2.3	2.6	1.7	1.6	1.4	
L-II-2	UPSTREAM	2.6	2.4	2.2	2.4	1.6	1.4	1.2	100	2.6	2.4	2.2	2.4	1.7	1.6	1.4	100
	DOWNSTREAM	2.7	2.5	2.2	2.5	1.7	1.4	1.1		2.6	2.4	2.2	2.4	1.7	1.7	1.4	

SECTION I.D.	25-I40-1	WINTER TESTING				DATE		3/16/10		SUMMER TESTING				DATE		6/23/10	
LOCATION	IH 40, W, MP 158.52					TIME								TIME		9:49 A.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	1.7	1.6	1.4	1.3	1.3	1.1	1.1	102	2.2	2.0	1.7	1.9	1.3	1.1	1.0	101
	DOWNSTREAM	1.8	1.7	1.4	1.3	1.3	1.1	1.2		2.2	1.9	1.7	1.9	1.3	1.1	1.0	
S-I-2	UPSTREAM	1.9	2.0	1.7	1.5	1.4	1.2	1.2	105	2.3	2.2	2.0	2.1	1.5	1.3	1.2	101
	DOWNSTREAM	2.2	2.0	1.7	1.5	1.4	1.2	1.2		2.3	2.2	2.0	2.1	1.5	1.3	1.2	
M-I-1	UPSTREAM	1.8	1.6	1.4	1.3	1.2	1.1	1.1	102	2.2	2.1	1.9	2.0	1.5	1.3	1.2	100
	DOWNSTREAM	1.8	1.6	1.4	1.3	1.3	1.1	1.1		2.1	2.0	1.9	2.0	1.5	1.3	1.2	
M-I-2	UPSTREAM	2.0	1.8	1.5	1.4	1.3	1.1	1.2	103	2.4	2.2	2.0	2.2	1.5	1.3	1.2	100
	DOWNSTREAM	1.9	1.8	1.5	1.4	1.3	1.1	1.2		2.3	2.2	1.9	2.2	1.5	1.3	1.2	
L-I-1	UPSTREAM	2.1	1.9	1.6	1.5	1.3	1.1	1.2	102	2.1	2.0	1.8	1.9	1.5	1.4	1.2	101
	DOWNSTREAM	2.1	1.9	1.7	1.5	1.3	1.2	1.2		2.1	2.0	1.8	1.9	1.5	1.3	1.2	
L-I-2	UPSTREAM	1.9	1.7	1.5	1.3	1.3	1.1	1.1	102	2.1	2.0	1.8	2.0	1.4	1.2	1.1	100
	DOWNSTREAM	1.9	1.7	1.5	1.4	1.2	1.1	1.1		2.0	2.0	1.8	2.0	1.4	1.3	1.1	
S-II-1	UPSTREAM	1.9	1.7	1.4	1.3	1.2	1.0	1.0	105	2.2	2.0	1.8	1.9	1.4	1.2	1.1	101
	DOWNSTREAM	1.8	1.7	1.4	1.3	1.2	1.1	1.1		2.2	2.0	1.8	2.0	1.4	1.3	1.1	
S-II-2	UPSTREAM	2.8	2.4	2.0	1.7	1.5	1.2	1.2	108	2.0	1.9	1.7	1.8	1.3	1.2	1.1	101
	DOWNSTREAM	2.6	2.4	2.0	1.7	1.5	1.2	1.2		2.0	1.9	1.7	1.9	1.4	1.2	1.1	
M-II-1	UPSTREAM	2.2	2.0	1.7	1.5	1.3	1.1	1.1	105	1.9	1.8	1.6	1.7	1.2	1.1	1.0	100
	DOWNSTREAM	2.2	2.0	1.7	1.5	1.3	1.1	1.1		1.9	1.8	1.6	1.7	1.3	1.1	1.0	
M-II-2	UPSTREAM	2.6	2.2	1.7	1.5	1.3	1.1	1.1	108	2.2	2.1	1.9	2.0	1.4	1.2	1.0	101
	DOWNSTREAM	2.3	2.1	1.7	1.4	1.3	1.1	1.1		2.3	2.1	1.9	2.1	1.4	1.2	1.1	
L-II-1	UPSTREAM	2.4	2.1	1.7	1.6	1.4	1.2	1.2	105	2.2	2.0	1.8	1.9	1.4	1.3	1.1	101
	DOWNSTREAM	2.4	2.1	1.7	1.5	1.4	1.2	1.2		2.1	2.1	1.8	1.9	1.5	1.3	1.1	
L-II-2	UPSTREAM	2.2	1.9	1.6	1.4	1.2	1.1	1.1	105	2.2	2.0	1.8	2.0	1.4	1.2	1.1	100
	DOWNSTREAM	2.1	1.8	1.5	1.4	1.2	1.0	1.1		2.1	2.0	1.8	2.0	1.4	1.2	1.1	

SECTION I.D.	25-I40-2	WINTER TESTING				DATE		3/16/10		SUMMER TESTING				DATE		6/23/10	
LOCATION	IH 40, W, MP 147-.15					TIME								TIME		2:02 P.M.	
CRACKS		W1	W2	W3	W4	W5	W6	W7	LTE	W1	W2	W3	W4	W5	W6	W7	LTE
S-I-1	UPSTREAM	1.9	1.7	1.5	1.4	1.4	1.2	1.3	102	2.5	2.3	2.1	2.3	1.5	1.3	1.2	101
	DOWNSTREAM	1.9	1.7	1.5	1.4	1.4	1.2	1.3		2.5	2.3	2.1	2.3	1.5	1.3	1.2	
S-I-2	UPSTREAM	1.8	1.8	1.6	1.5	1.4	1.2	1.3	102	2.6	2.4	2.1	2.3	1.6	1.3	1.2	100
	DOWNSTREAM	1.8	1.8	1.6	1.5	1.4	1.2	1.3		2.5	2.4	2.1	2.3	1.5	1.3	1.2	
M-I-1	UPSTREAM	1.5	1.6	1.4	1.4	1.3	1.2	1.3	101	2.3	2.1	1.8	2.0	1.3	1.2	1.0	100
	DOWNSTREAM	1.8	1.6	1.4	1.4	1.3	1.2	1.2		2.3	2.0	1.8	2.0	1.3	1.2	1.0	
M-I-2	UPSTREAM	1.8	1.7	1.5	1.4	1.4	1.2	1.3	101	2.4	2.2	1.9	2.1	1.4	1.2	1.1	100
	DOWNSTREAM	1.8	1.7	1.5	1.4	1.4	1.2	1.3		2.4	2.2	1.9	2.1	1.4	1.2	1.1	
L-I-1	UPSTREAM	1.9	1.7	1.5	1.5	1.4	1.3	1.4	100	2.2	2.1	1.9	2.0	1.4	1.2	1.1	101
	DOWNSTREAM	1.8	1.7	1.6	1.5	1.5	1.3	1.4		2.3	2.1	1.9	2.0	1.4	1.2	1.1	
L-I-2	UPSTREAM	1.7	1.6	1.4	1.4	1.3	1.1	1.2	102	2.3	2.2	1.9	2.1	1.4	1.2	1.1	102
	DOWNSTREAM	1.9	1.6	1.4	1.4	1.3	1.1	1.2		2.3	2.2	1.9	2.1	1.4	1.2	1.1	
S-II-1	UPSTREAM	2.0	1.7	1.6	1.5	1.4	1.3	1.3	101	2.5	2.3	2.0	2.2	1.4	1.2	1.1	102
	DOWNSTREAM	1.9	1.7	1.6	1.5	1.4	1.3	1.3		2.4	2.2	1.9	2.2	1.4	1.3	1.1	
S-II-2	UPSTREAM	1.7	1.7	1.5	1.4	1.4	1.2	1.3	101	2.8	2.7	2.4	2.5	1.7	1.5	1.2	102
	DOWNSTREAM	1.9	1.7	1.5	1.5	1.4	1.2	1.3		2.8	2.7	2.4	2.6	1.7	1.4	1.2	
M-II-1	UPSTREAM	1.7	1.7	1.5	1.5	1.4	1.3	1.4	100	2.3	2.2	1.9	2.1	1.4	1.2	1.0	101
	DOWNSTREAM	1.8	1.6	1.5	1.4	1.4	1.2	1.3		2.3	2.2	1.9	2.2	1.4	1.2	1.0	
M-II-2	UPSTREAM	1.8	1.7	1.5	1.4	1.4	1.2	1.3	102	2.5	2.3	2.0	2.2	1.5	1.2	1.1	102
	DOWNSTREAM	2.0	1.7	1.5	1.4	1.4	1.2	1.3		2.6	2.3	2.0	2.2	1.4	1.2	1.0	
L-II-1	UPSTREAM	1.8	1.7	1.5	1.5	1.4	1.2	1.3	101	2.3	2.2	2.0	2.1	1.5	1.3	1.2	100
	DOWNSTREAM	1.9	1.7	1.6	1.5	1.4	1.2	1.3		2.4	2.3	2.1	2.2	1.6	1.4	1.2	
L-II-2	UPSTREAM	2.0	1.8	1.6	1.5	1.4	1.3	1.3	101	2.5	2.3	1.9	2.1	1.4	1.2	1.0	103
	DOWNSTREAM	1.9	1.8	1.6	1.5	1.4	1.3	1.3		2.4	2.2	1.9	2.2	1.4	1.2	1.0	

APPENDIX C

Graph of Deflections at 50-ft Interval

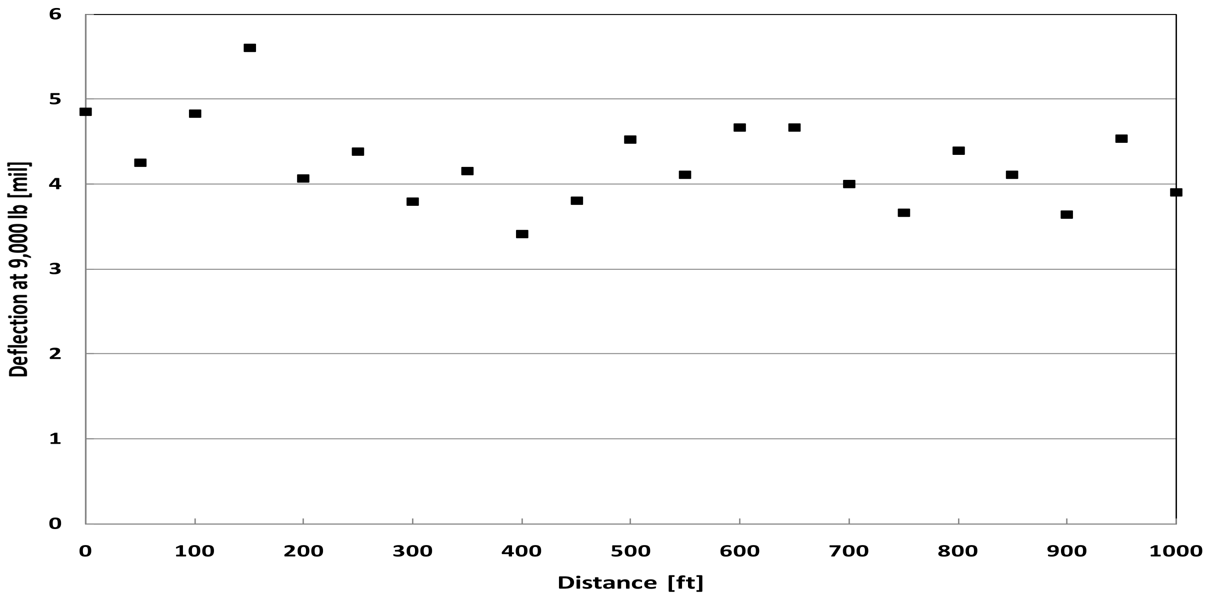

Figure C.1 Deflections at 50-ft. interval [3-US 287-1, winter]

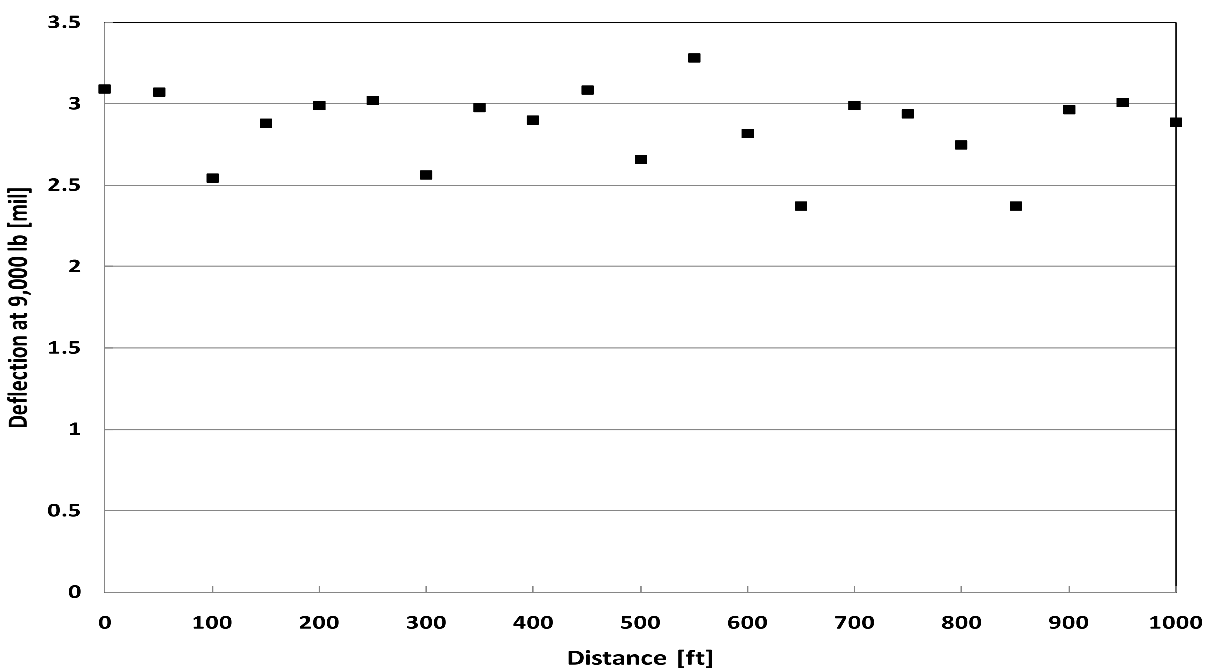

Figure C.2 Deflections at 50-ft. interval [4-I 40-1, summer]

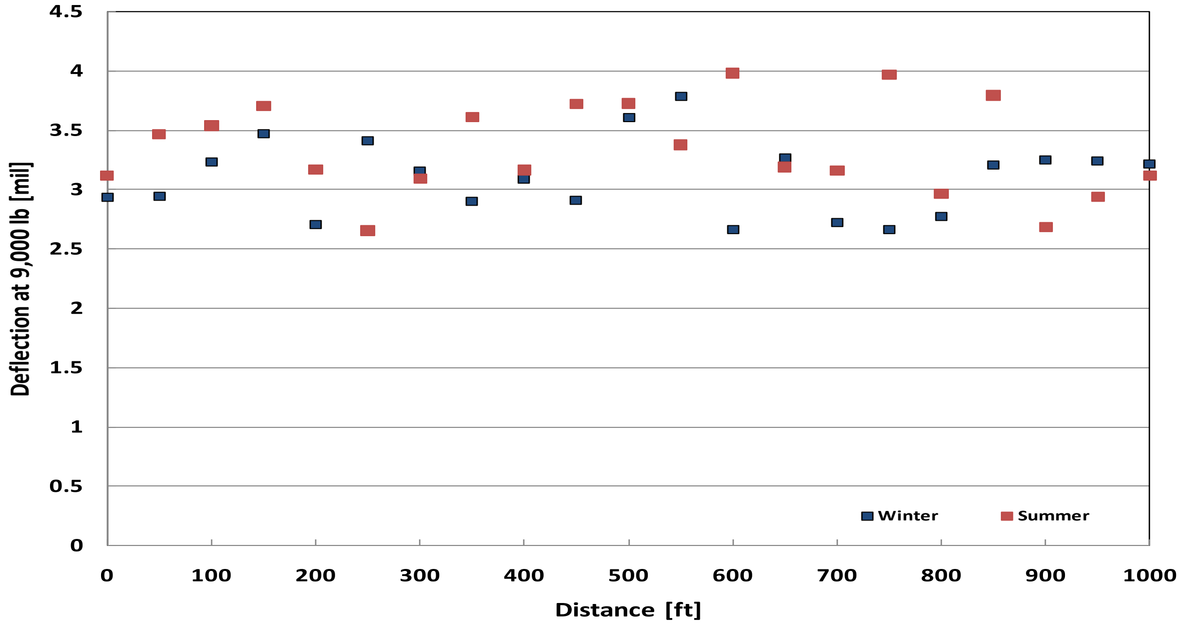

Figure C.3 Deflections at 50-ft. interval [5-I 27-1]

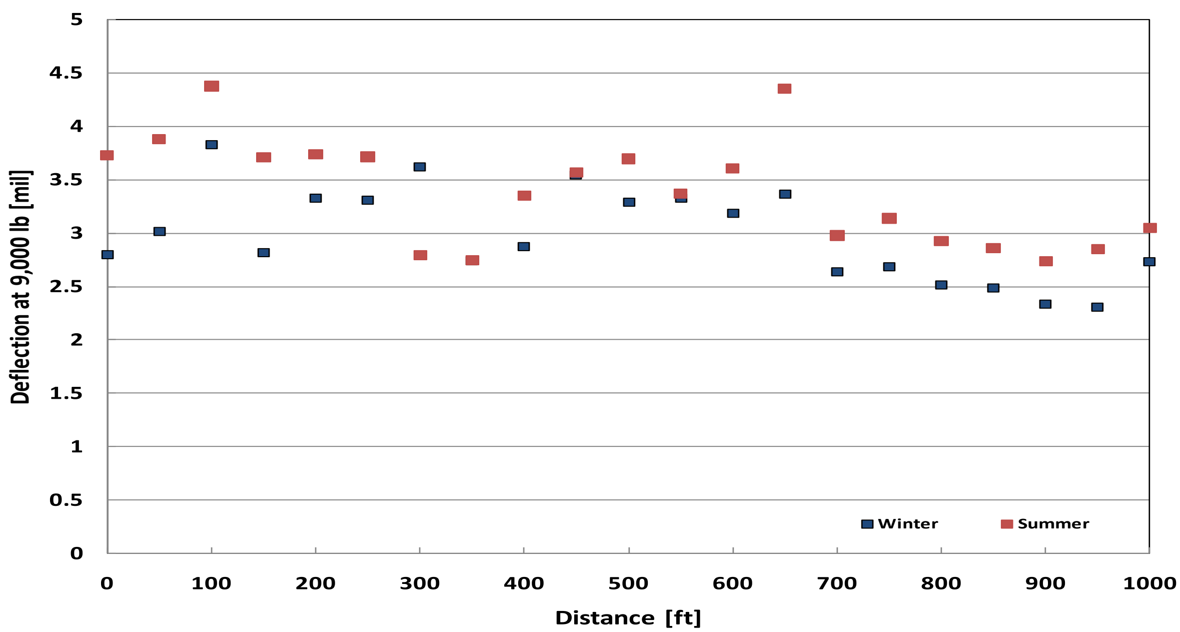

Figure C.4 Deflections at 50-ft. interval [5-LP 289-1]

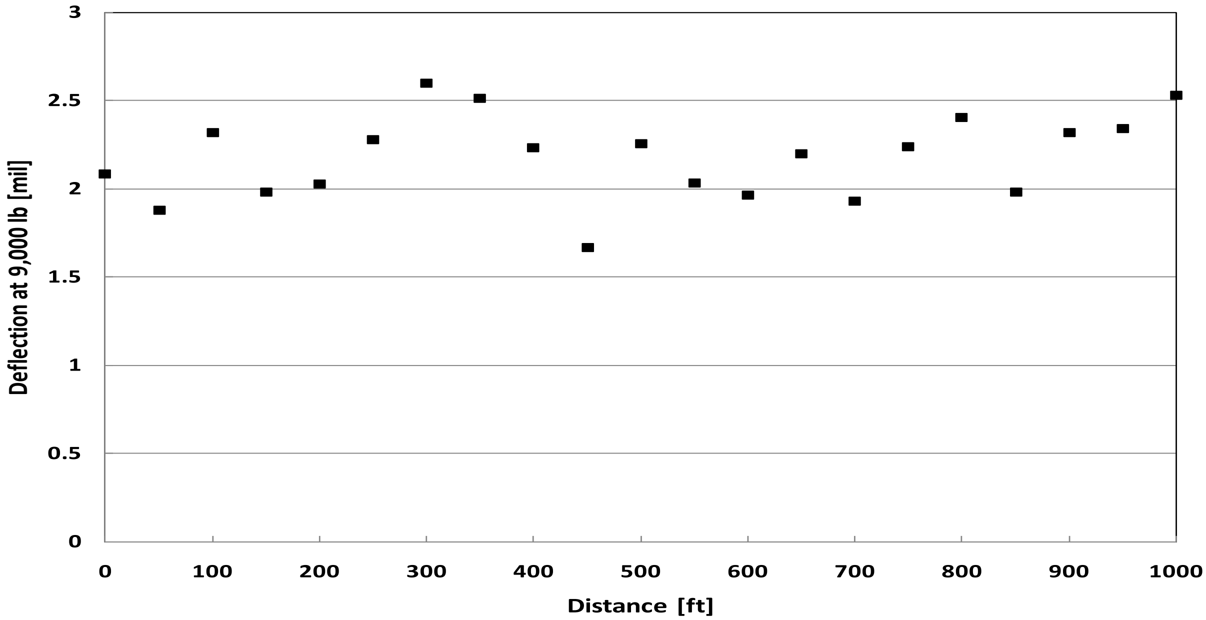

Figure C.5 Deflections at 50-ft. interval [12-US 290-1, summer]

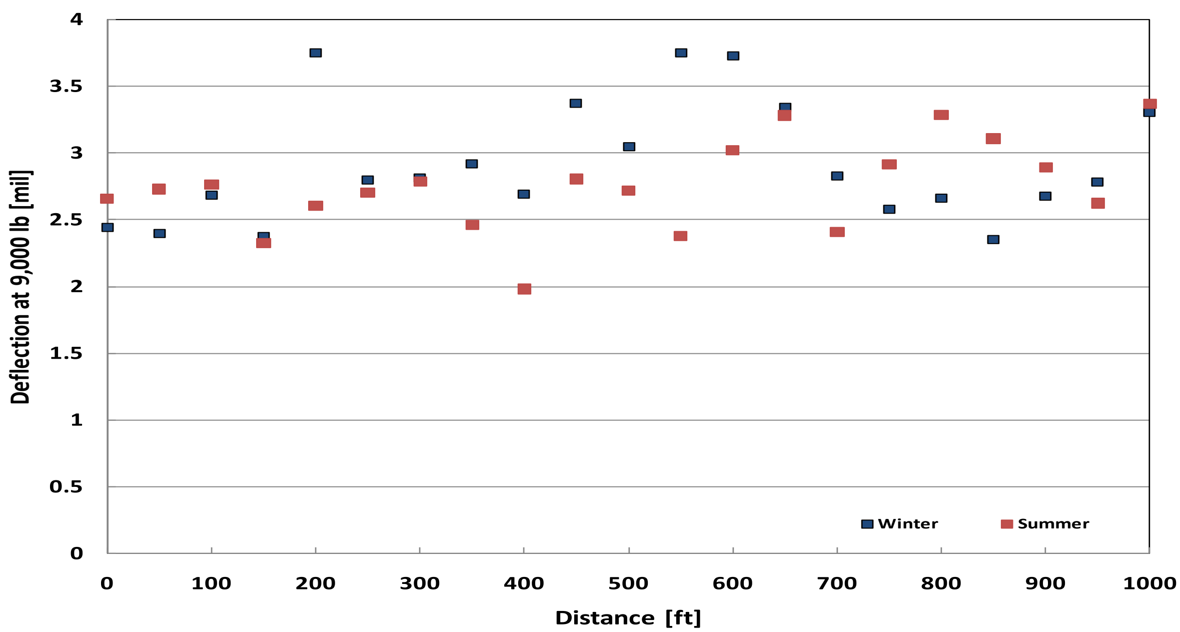

Figure C.6 Deflections at 50-ft. interval [12-US 290-2]

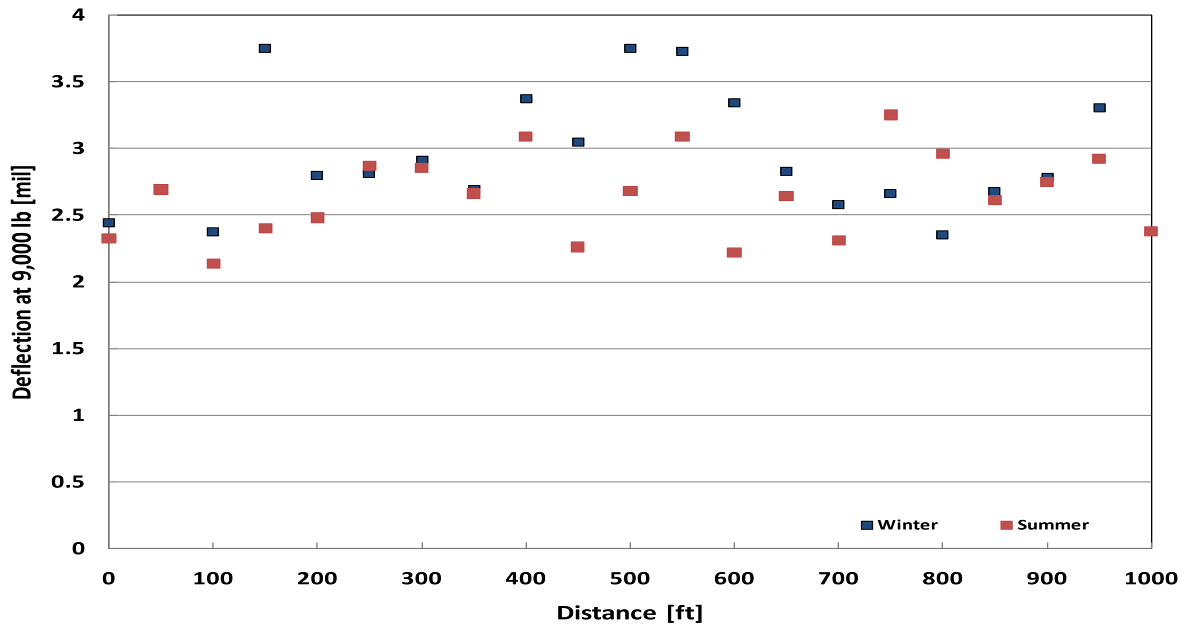

Figure C.7 Deflections at 50-ft. interval [12-US 290-3]

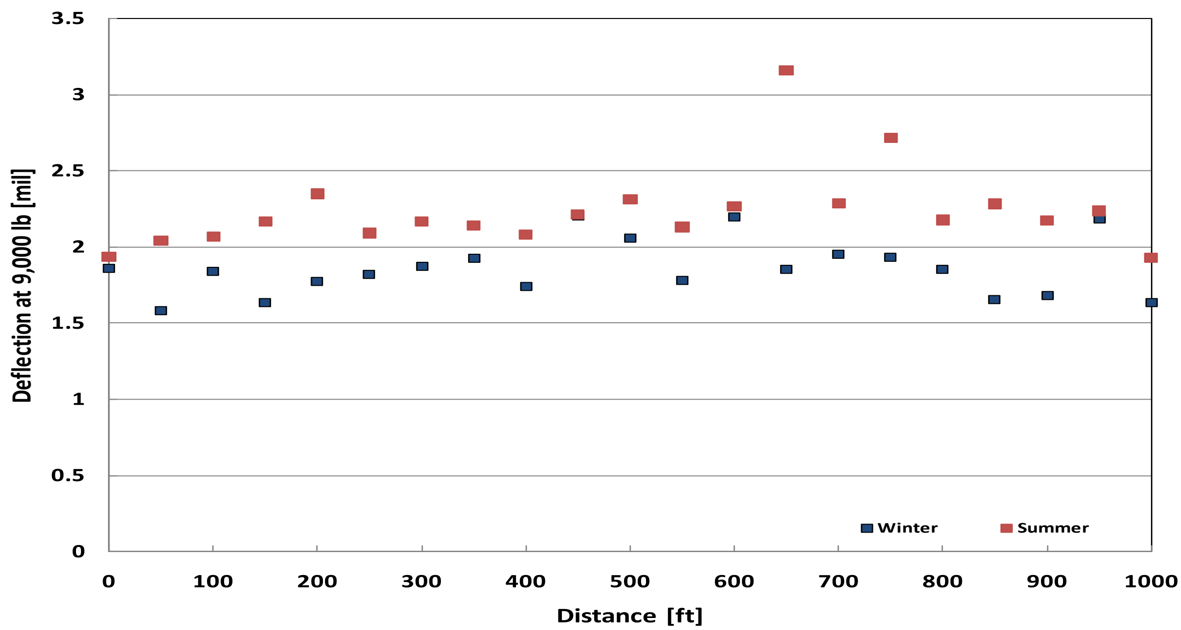

Figure C.8 Deflections at 50-ft. interval [19-US 59-1]

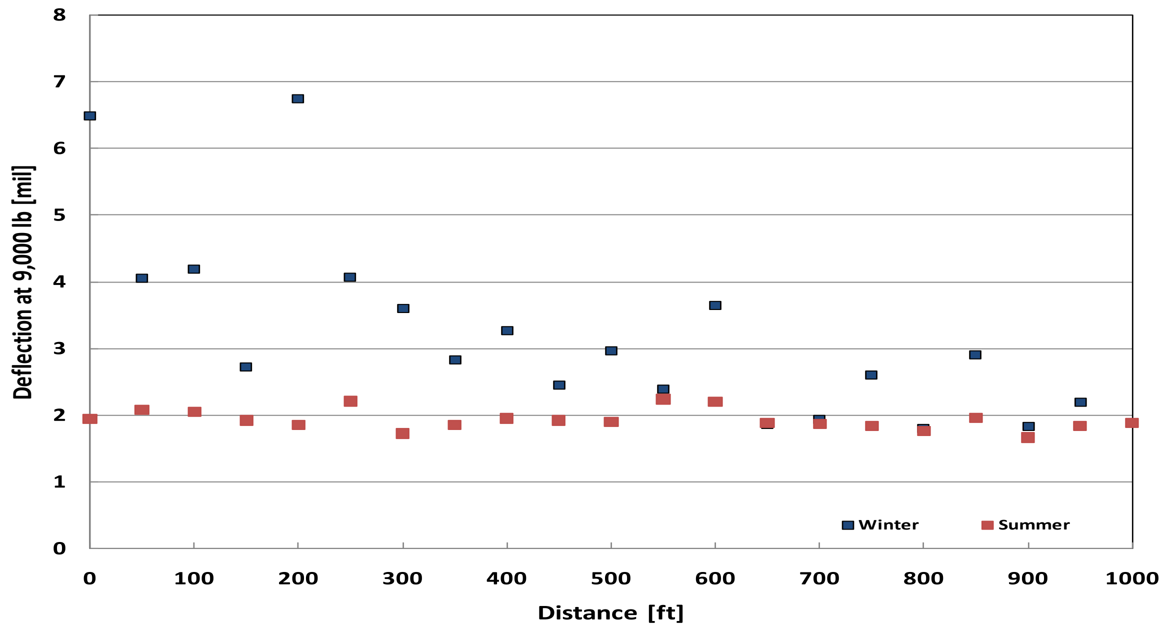

Figure C.9 Deflections at 50-ft. interval [19-US 59-2]

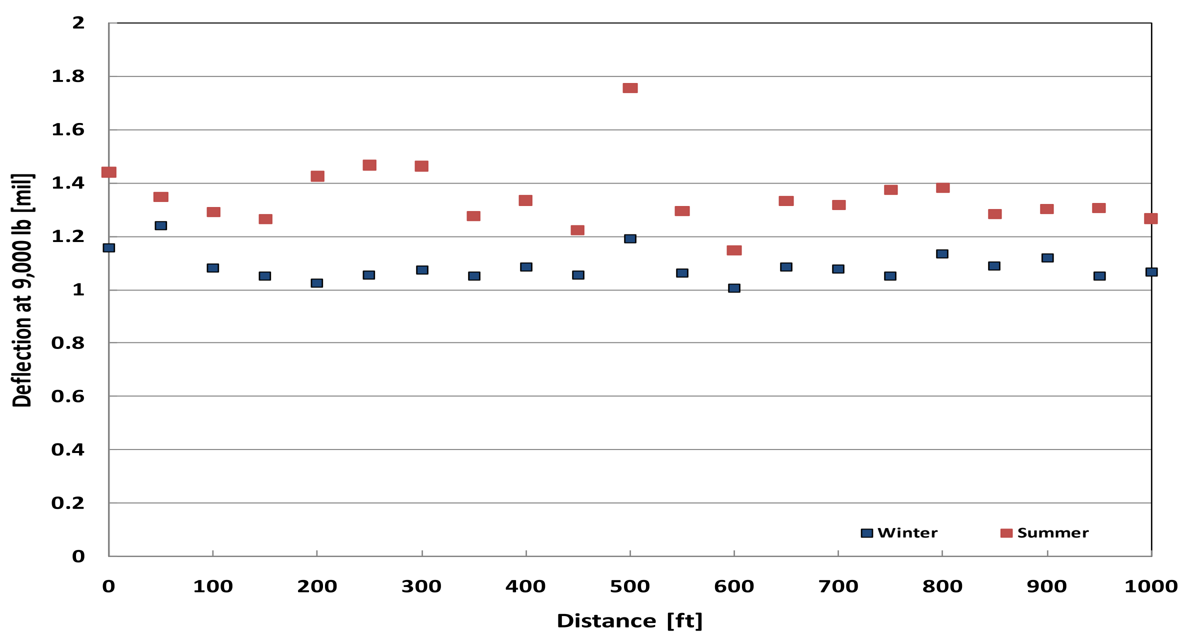

Figure C.10 Deflections at 50-ft. interval [24-I 10-1]

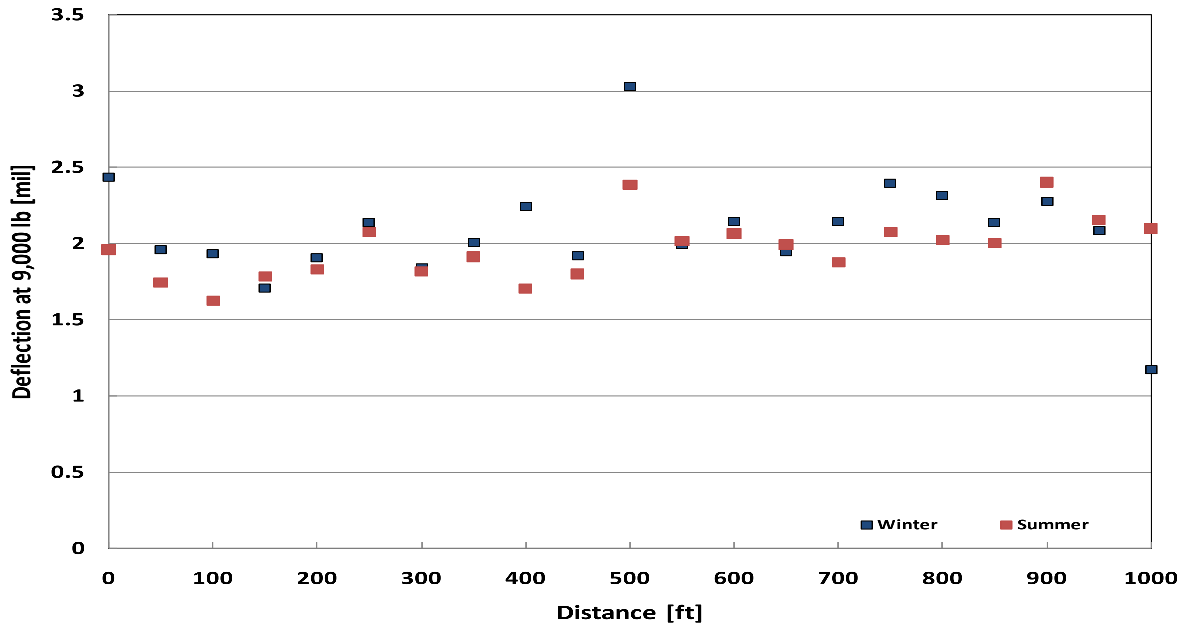

Figure C.11 Deflections at 50-ft. interval [24-I 10-2]

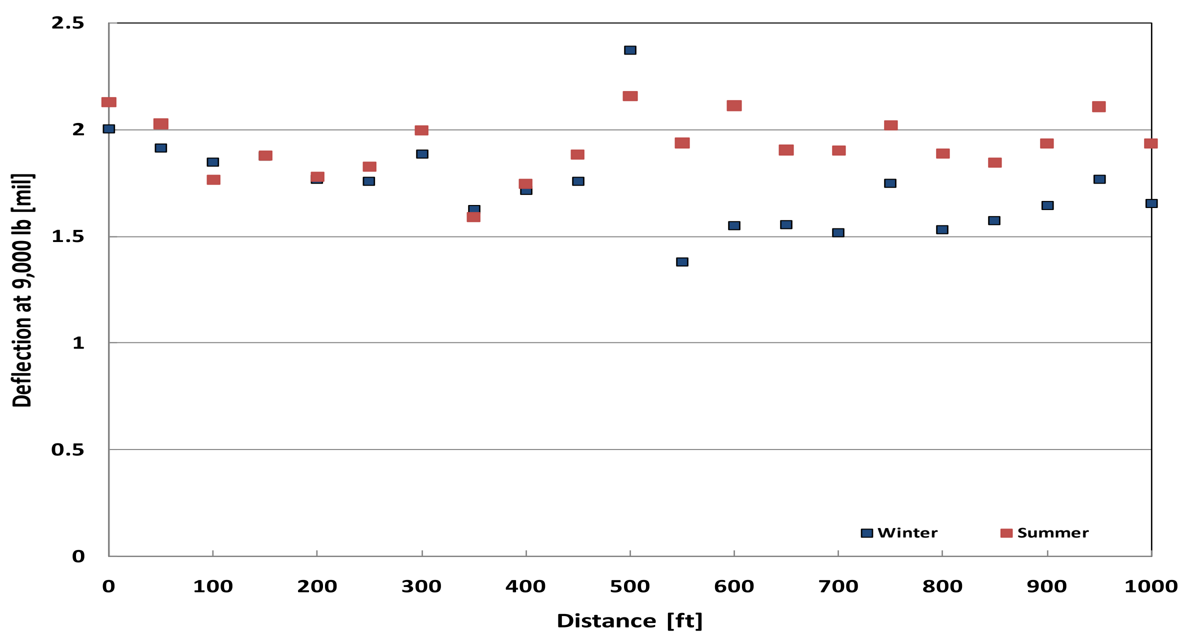

Figure C.12 Deflections at 50-ft. interval [24-I 10-3]

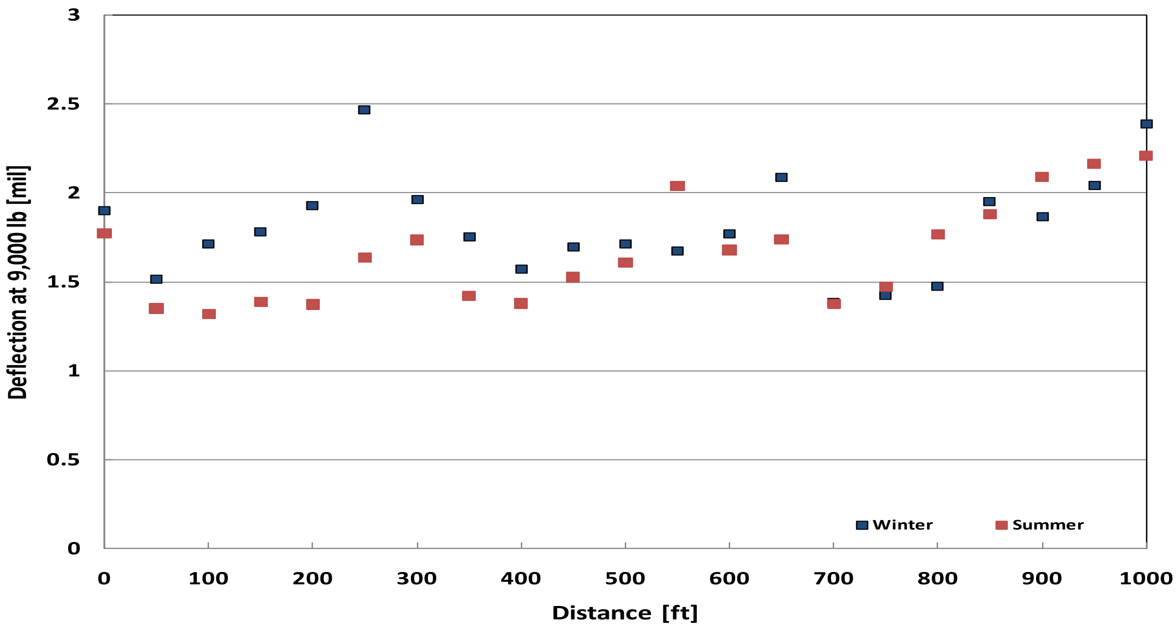

Figure C.13 Deflections at 50-ft. interval [24-I 10-4]

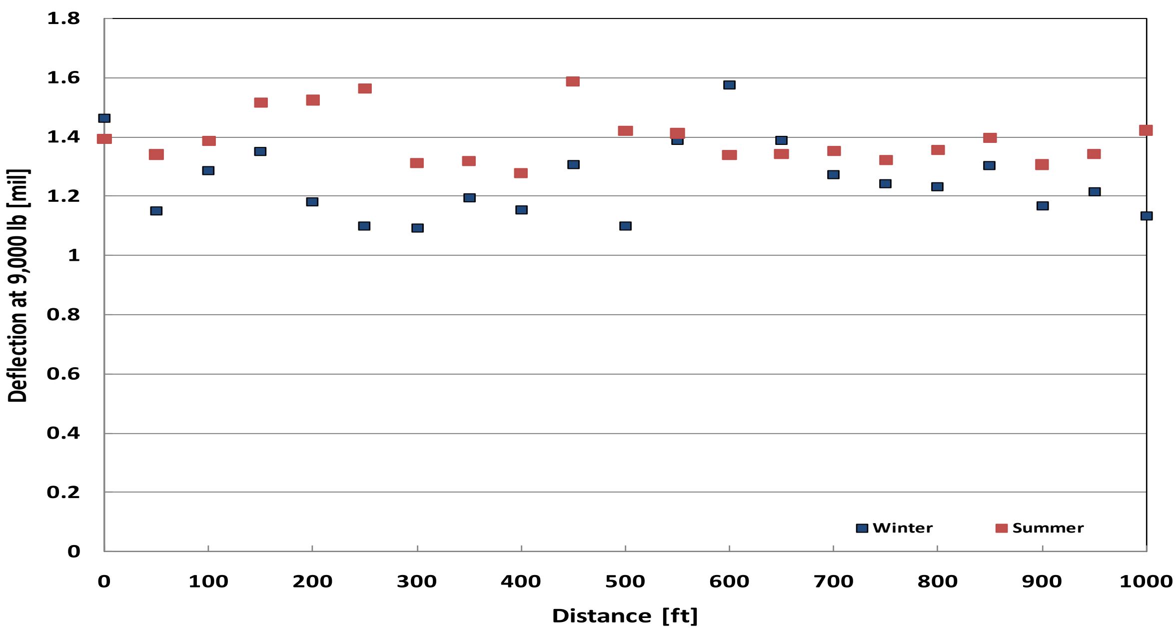

Figure C.14 Deflections at 50-ft. interval [25-I 40-1]

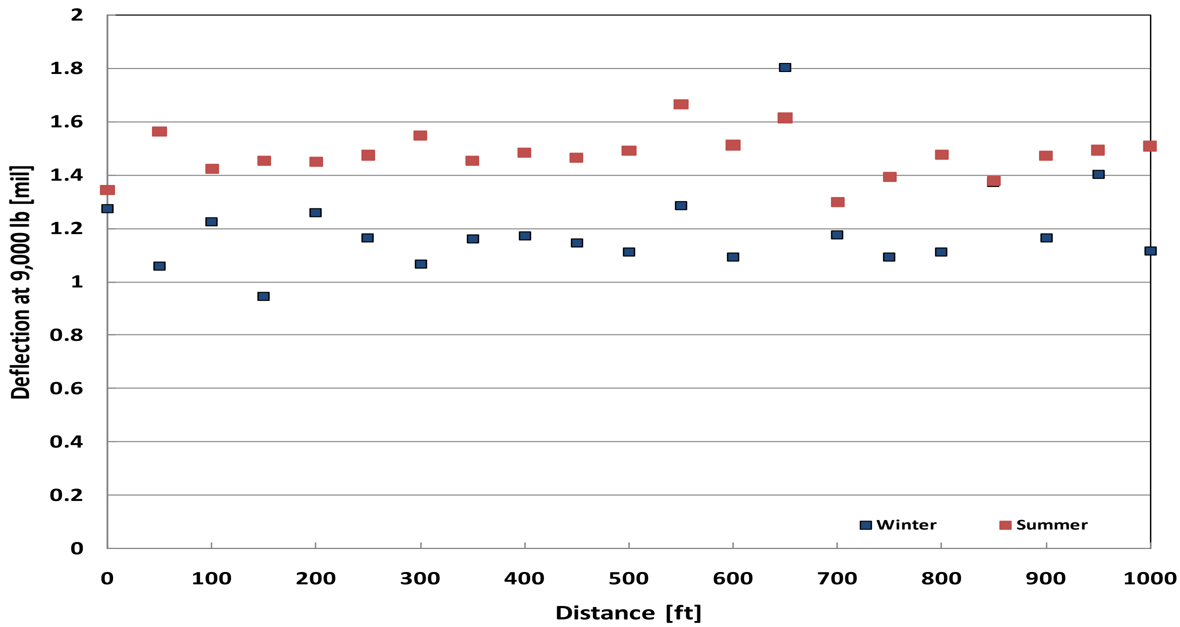

Figure C.15 Deflections at 50-ft. interval [25-I 40-2]

APPENDIX D

LTE Evaluations in FY10

Crack ID	12-US290-1	
	Winter	Summer
J1	109	105
C1	106	105
J2	109	136
C2	107	105
J3	108	106
C3	107	107
J4	108	105
C4	106	105
J5	109	103
C5	106	134

Crack ID	4-I40-1
	Summer
S-I-1	107
S-I-2	102
M-I-1	107
M-I-2	102
L-I-1	101
L-I-2	102
S-II-1	102
S-II-3	101
M-II-1	102
M-II-2	106
L-II-1	101
L-II-2	102

Crack ID	3-US287-1	5-I27-1		5-LP289-1		12-US290-2		19-US59-1		19-US59-2		24-I10-1		24-I10-2		24-I10-3		24-I10-4		25-I40-1		25-I40-2	
	Winter	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer	Winter	Summer
S-I-1	107	100	100	100	100	106	105	96	103	101	88	99	100			100	100	101	101	102	101	102	101
S-I-2	106	101	100	112	100	106	104	100	101	96	87	100	100			99	101	100	100	105	101	102	100
M-I-1	109	100	100	100	100	107	105	102	104	101	101	100	105	105	101	100	100	99	100	102	100	101	100
M-I-2	107	100	100	101	101	110	107	102	106	100	101	99	100	106	102	98	101	102	97	103	100	101	100
L-I-1	93			100	100	106		102	105	97	104	99	99	107	98	100	101	101	101	102	101	100	101
L-I-2	107			102	101	107		102	106	106	101	99	100	107	102	102	101	101	100	102	100	102	102
S-II-1	103	100	100	100	100		105	78	96	99	99	101	97					101	101	105	101	101	102
S-II-2	103	100	98	101	100		106	102	100	97	95	98	100					100	99	108	101	101	102
M-II-1	108	103	101	109	101	105	106	101	98	102	101	99	99	104	102	101	100	101	100	105	100	100	101
M-II-2	107	103	101	100	100		105	102	102	99	100	98	98	102	101	103	100	100	101	108	101	102	102
L-II-1	106			99	100	106	107	104	103	100	100	100	100	94	99	101	101	102	100	105	101	101	100
L-II-2	105			101	98		106	103		100	102	100	99	103	103	95	100	100	100	105	100	101	103

APPENDIX E

Graph of LTE Evaluations in FY10

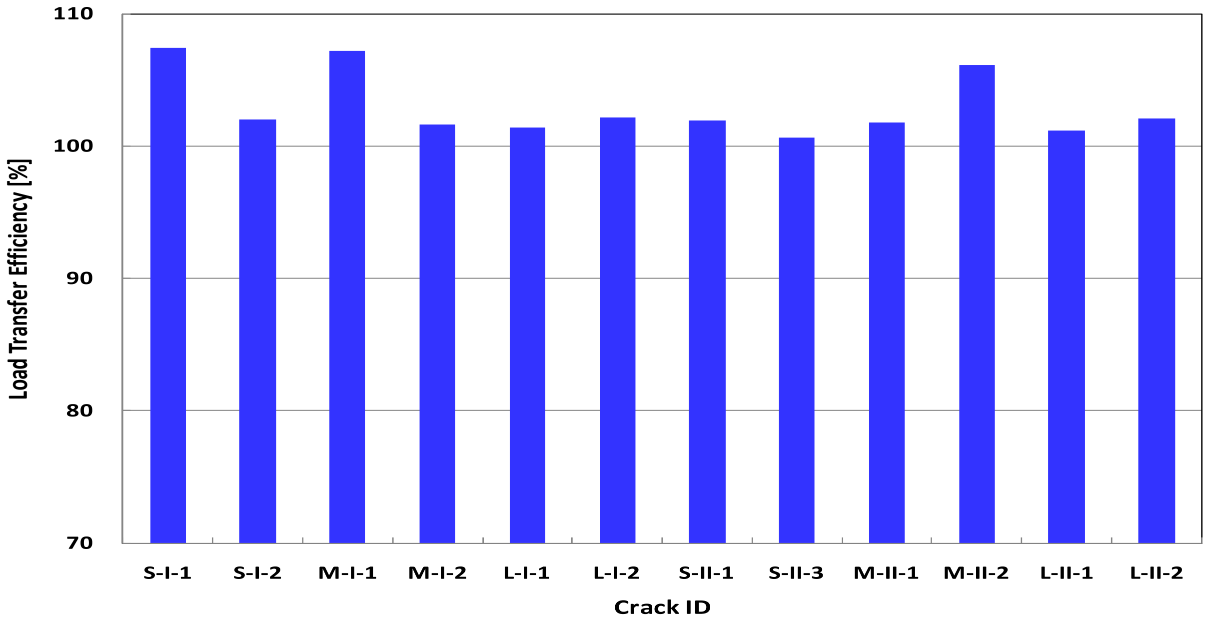

Figure E.1 LTE Evaluations in FY10 [4-I40-1, summer]

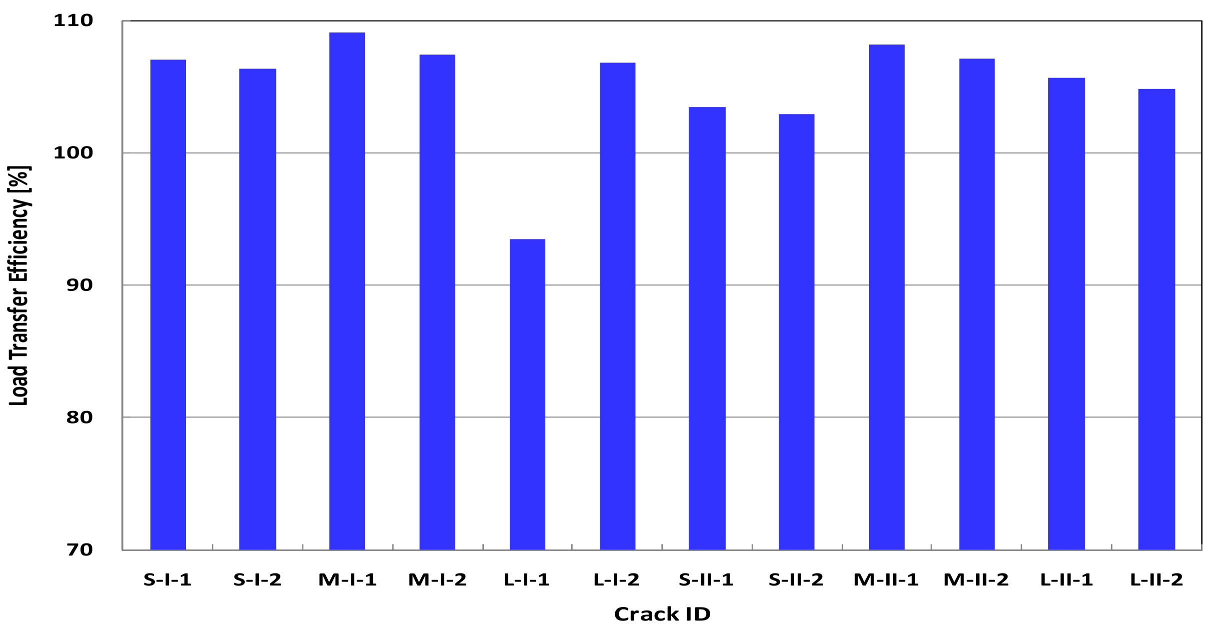

Figure E.2 LTE Evaluations in FY10 [3-US287-1, winter]

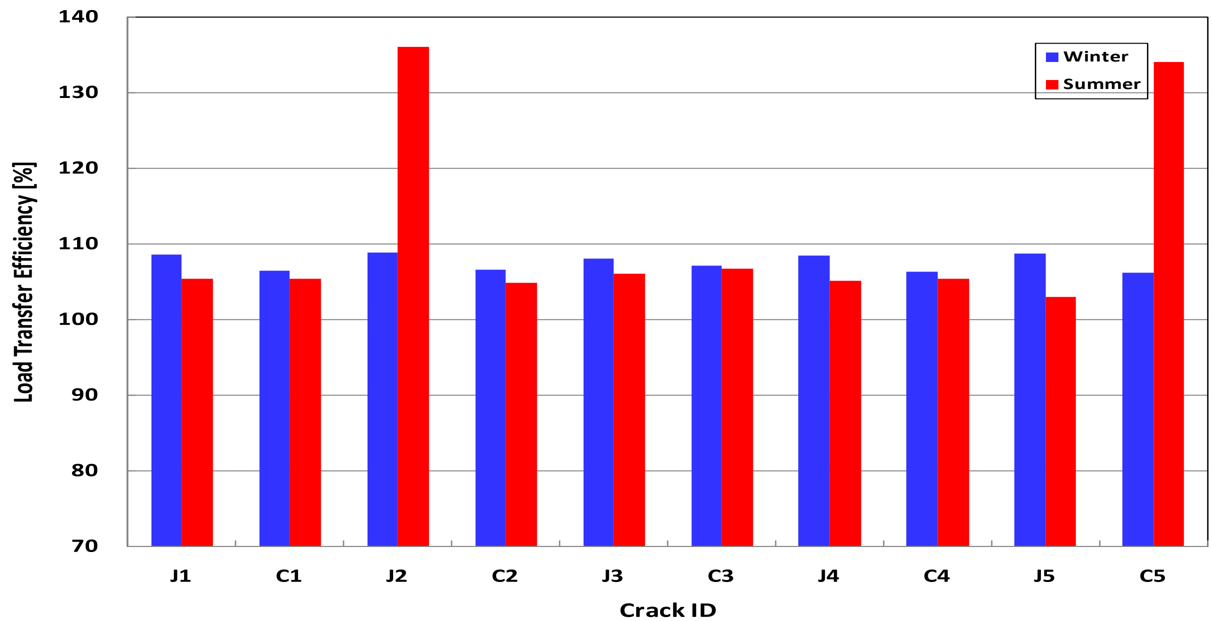

Figure E.3 LTE Evaluations in FY10 [12-US290-1]

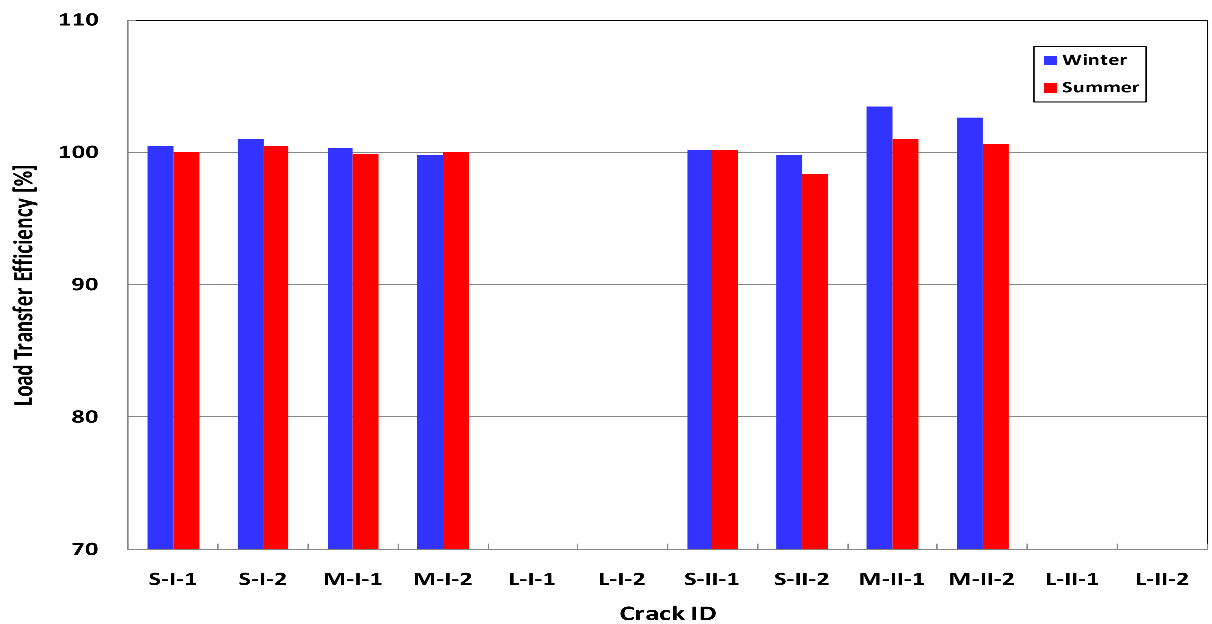

Figure E.4 LTE Evaluations in FY10 [5-I27-1]

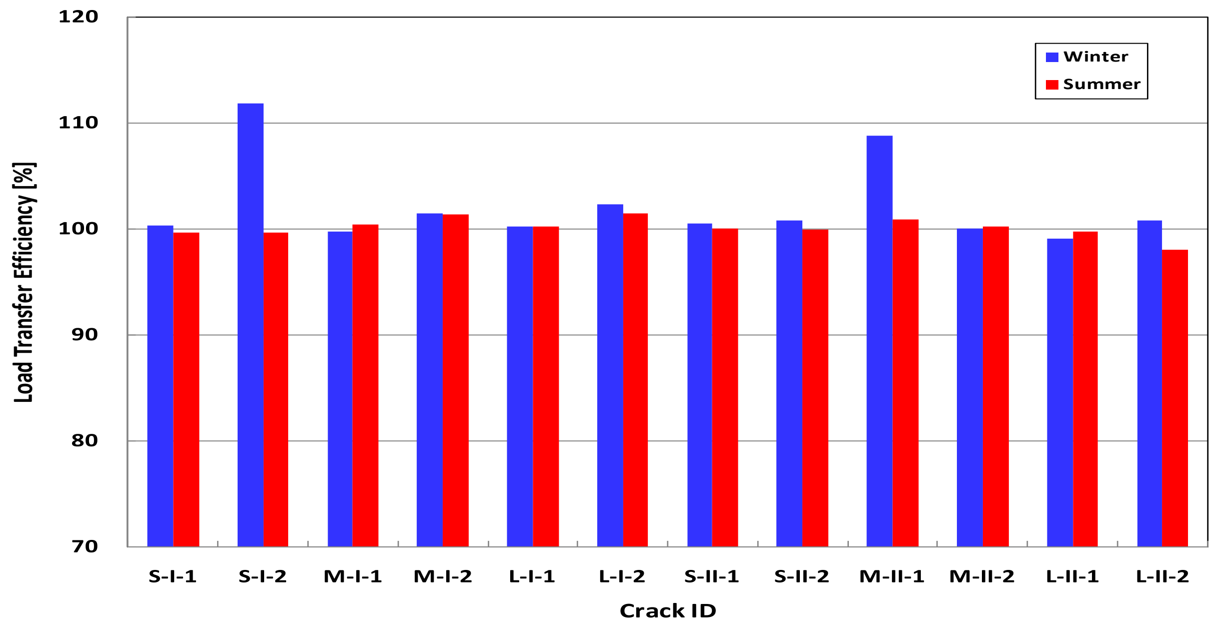

Figure E.5 LTE Evaluations in FY10 [5-LP289-1]

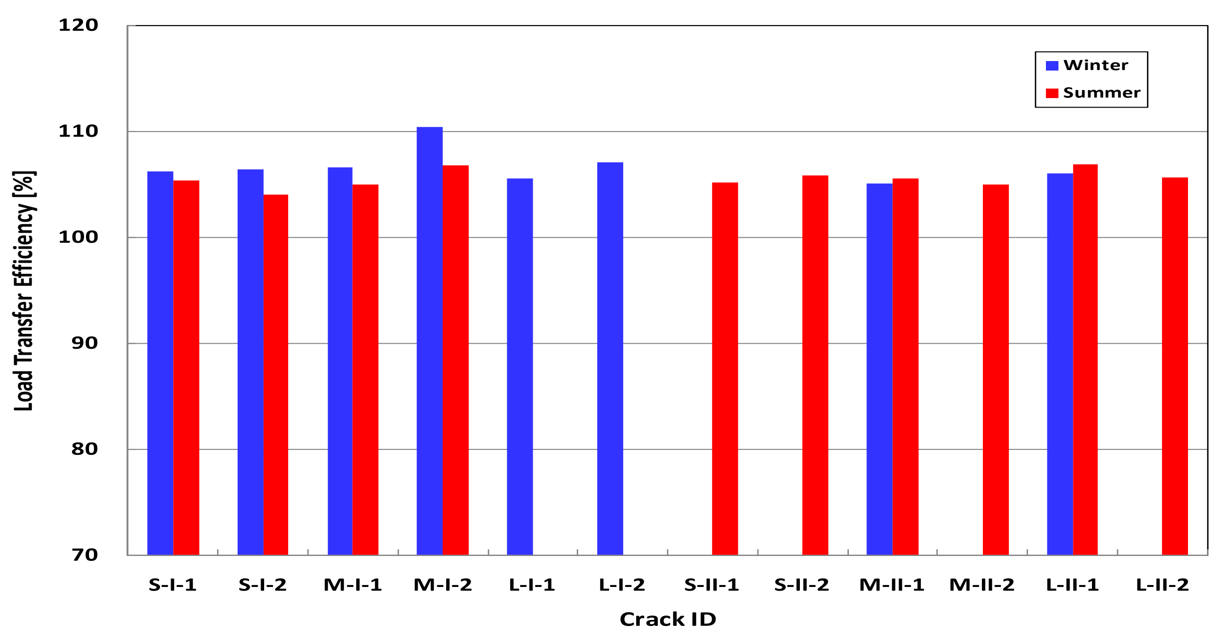

Figure E.6 LTE Evaluations in FY10 [12-US290-2]

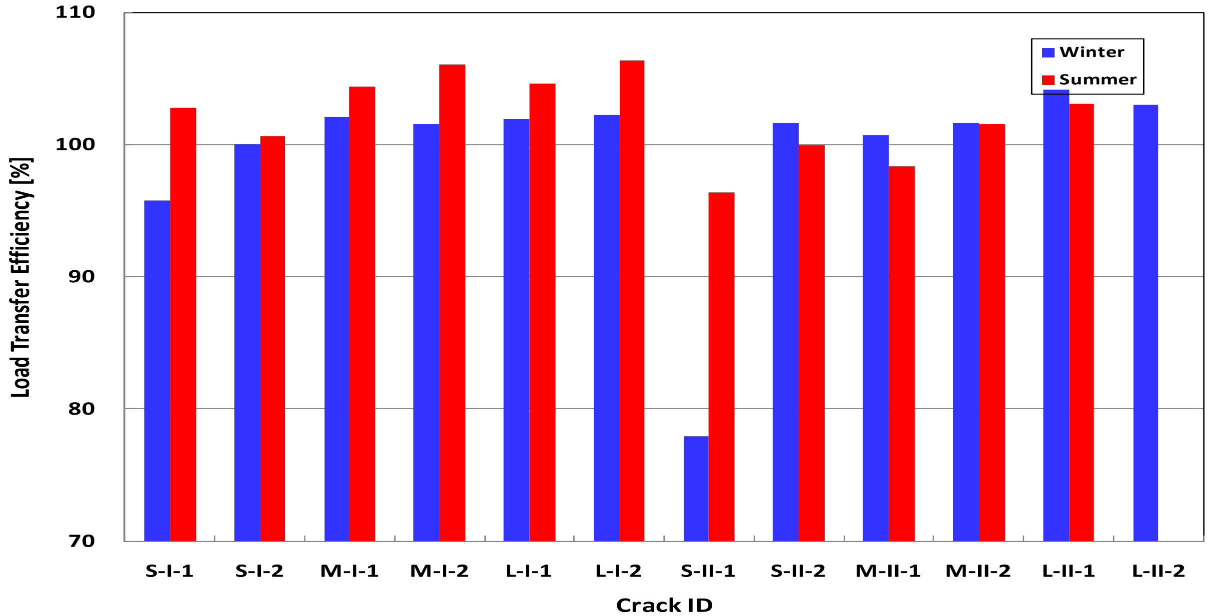

Figure E.7 LTE Evaluations in FY10 [19-US59-1]

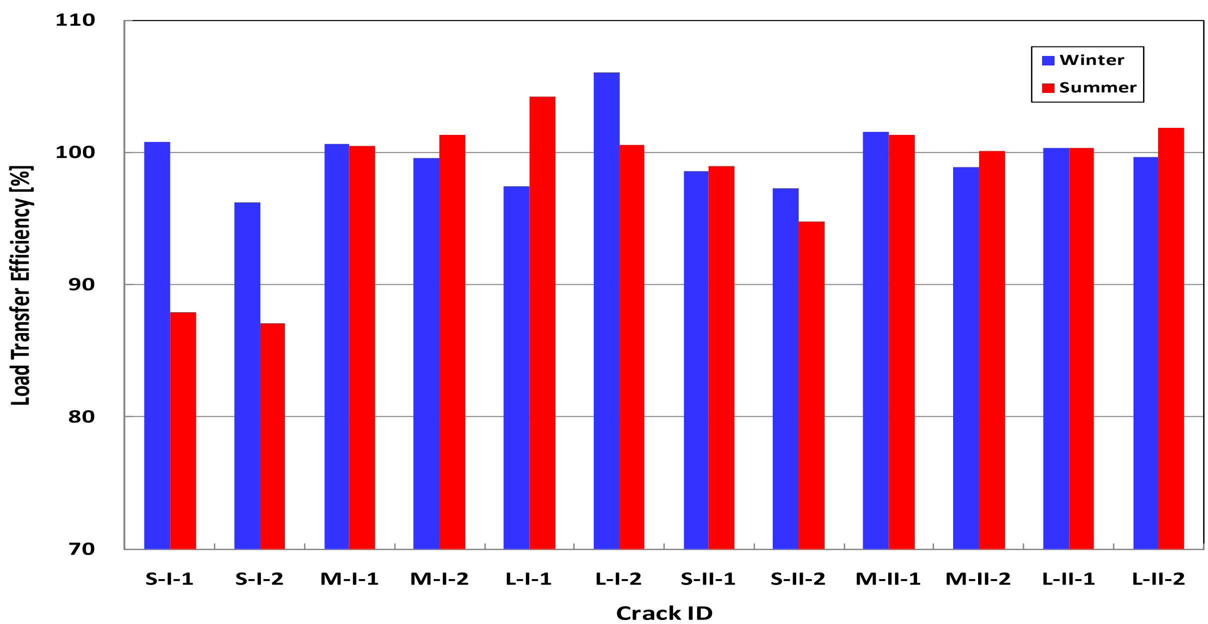

Figure E.8 LTE Evaluations in FY10 [19-US59-2]

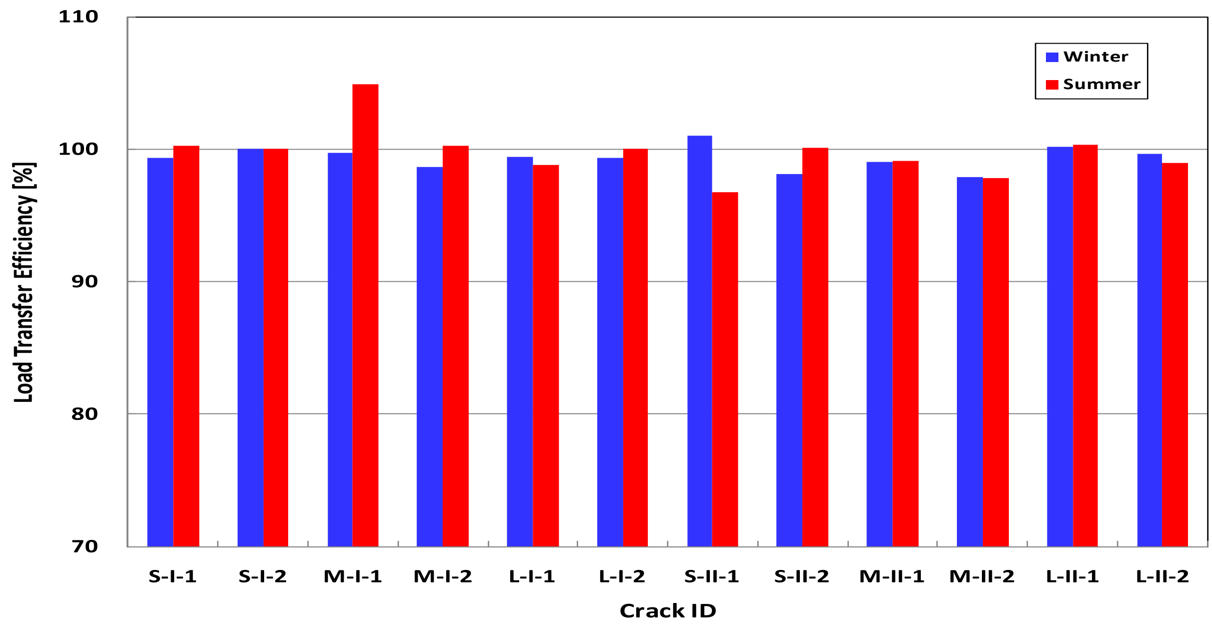

Figure E.9 LTE Evaluations in FY10 [24-I10-1]

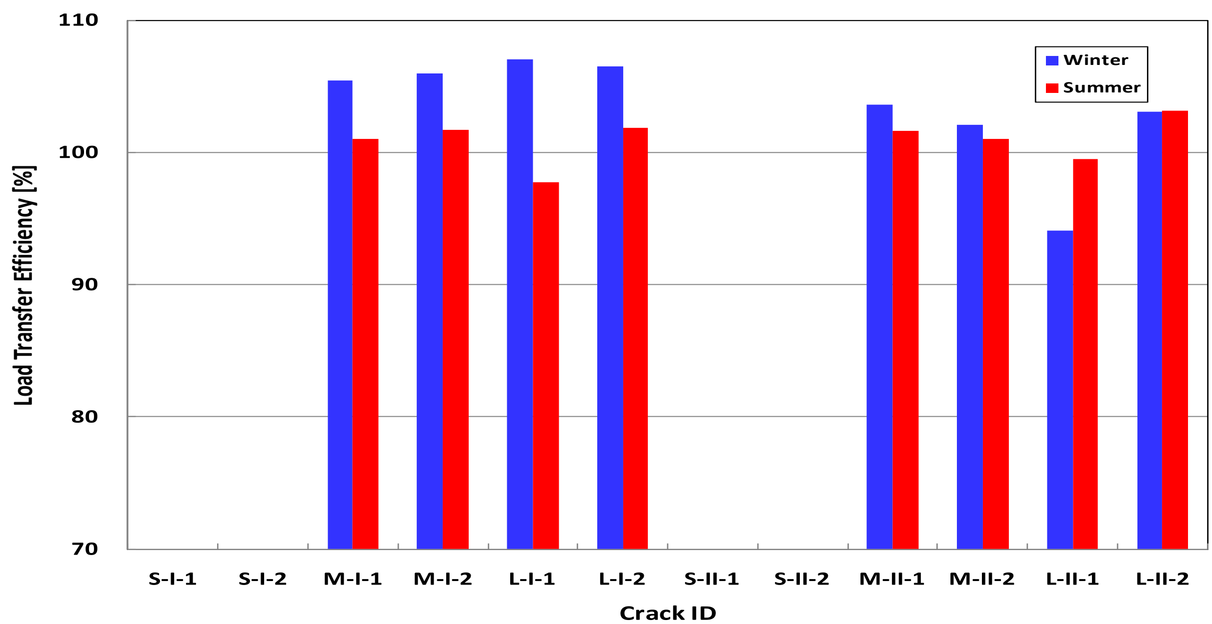

Figure E.10 LTE Evaluations in FY10 [24-I10-2]

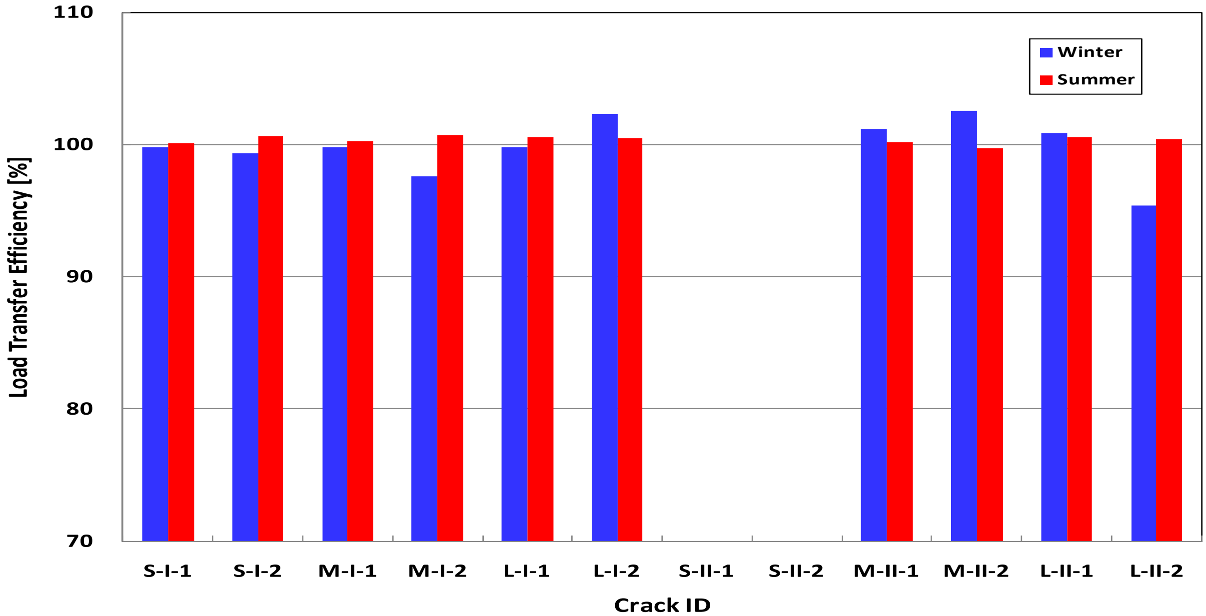

Figure E.11 LTE Evaluations in FY10 [24-I10-3]

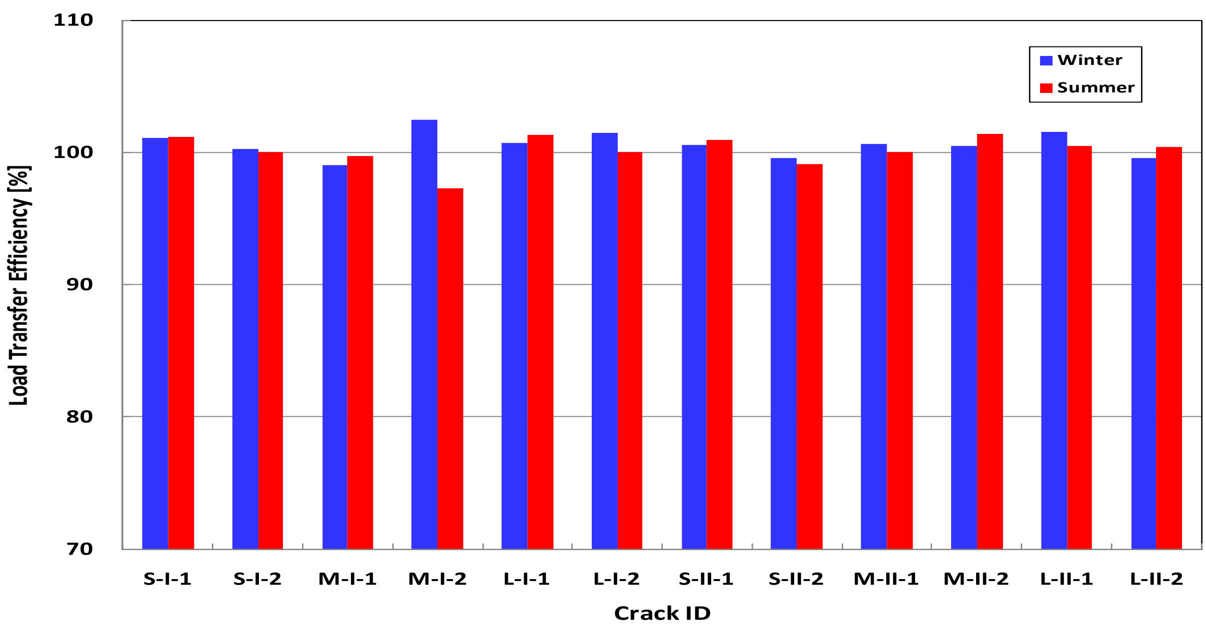

Figure E.12 LTE Evaluations in FY10 [24-I10-4]

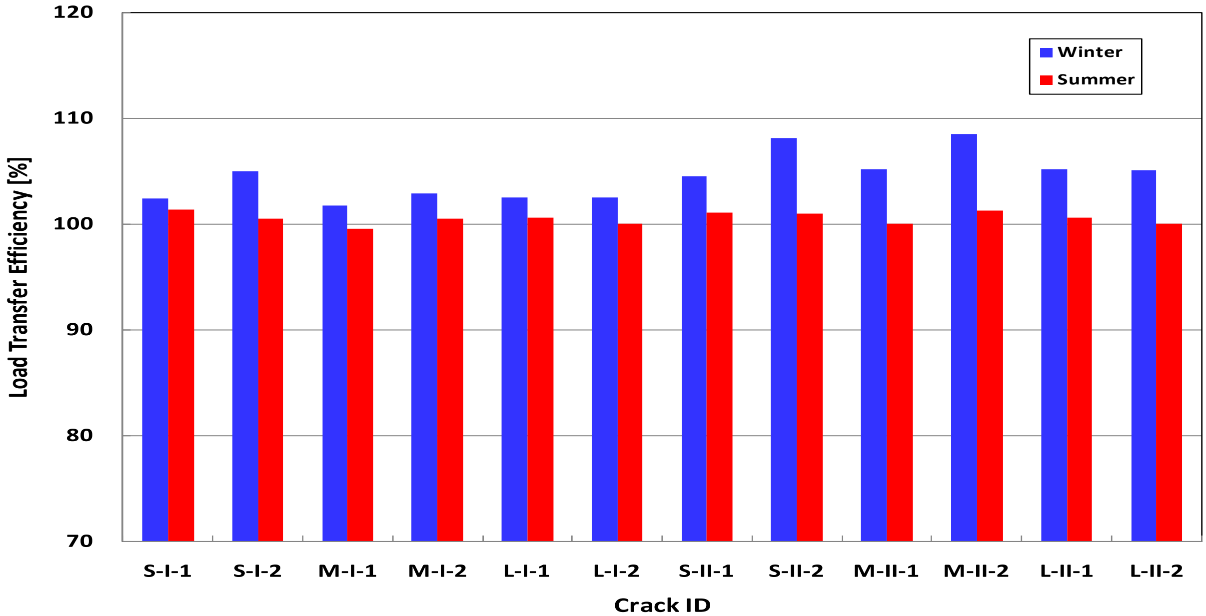

Figure E.13 LTE Evaluations in FY10 [25-I40-1]

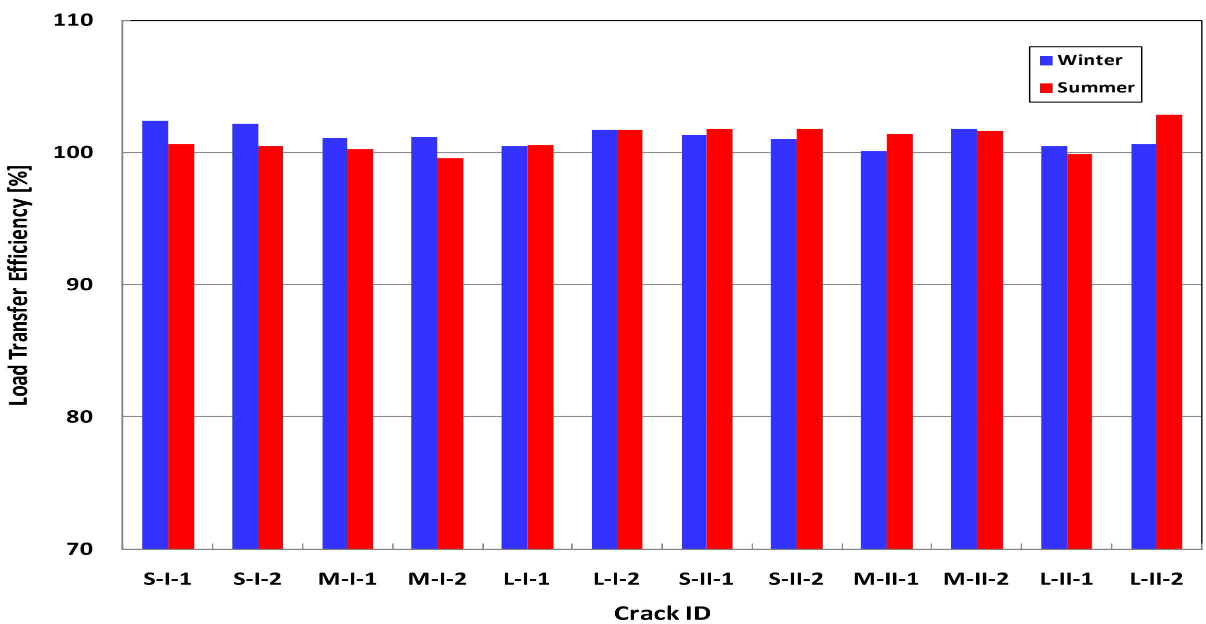

Figure E.14 LTE Evaluations in FY10 [25-I40-2]

APPENDIX F

Punchout Types in Chapter 5

- PCH ; PunCHout
- E-PCH ; Edge PunCHout
- E-PCH-PTB ; Edge PunCHout with Poor Tie-Bar
- PCH-CJ ; PunCHout in Construction Joint
- PCH-RJ ; PunCHout in Repair Joint
- BS-PCW ; Big Spalling with Poor Concrete Work

PCH

PCH

E-PCH (US81 Wichita Falls)

E-PCH (US81 Wichita Falls)

E-PCH (US81 Wichita Falls)

E-PCH-PTB (IH35-Wichita Falls)

E-PCH-PTB (IH30-Dallas)

PCH-CJ (IH35-Wichita Falls)

PCH-CJ (IH35-Wichita Falls)

PCH-CJ

PCH-RJ

PCH-RJ (IH30 Dallas)

PCH-RJ (IH30-Dallas)

BS-PCW (IH35-Wichita Falls)

BS-PCW (SH114-Dallas)

BS-PCW (SH360-Fort Worth)

TEXAS TECH UNIVERSITY
Multidisciplinary Research in Transportation

Texas Tech University | Lubbock, Texas 79409
P 806.742.3503 | F 806.742.4168