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STRUCTURAL PERFORMANCE OF ULTRA-THIN WHITETOPPING ON ILLINOIS ROADWAYS AND PARKING LOTS

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16. Abstract

A performance evaluation of ultra-thin whitetopping (UTW) pavements in Illinois was undertaken in 2012–2014 to evaluate current design procedures and to determine design life criteria for future projects. The two main components of this evaluation were (1) visual distress surveys of 20 existing UTW projects across the state to document and quantify distresses and (2) falling weight deflectometer (FWD) testing of eight of these UTW projects to evaluate structural performance.

The findings of the surveys are detailed in this report. Deflection data collected during FWD testing were used to directly calculate load transfer efficiency and assess joint performance, but there was no existing method to assess the in situ structural properties of UTW pavements. To better characterize structural performance, a backcalculation procedure for UTW pavements was derived and applied to the deflection data obtained from FWD testing. The backcalculated effective concrete thickness quantifies the load carrying capacity of the UTW pavement, variation of the structural capacity as a function of distance along the roadway, and potentially the condition of the concrete–asphalt bond interface and the underlying asphalt concrete layer.

The findings of the visual distress surveys and the FWD data analysis largely agreed with each other and were studied to help provide a greater understanding of factors that affect UTW performance. From this analysis, a number of conclusions and recommendations were made regarding UTW pavement design and construction.

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EXECUTIVE SUMMARY

A performance evaluation of ultra-thin whitetopping (UTW) pavements in Illinois was undertaken in 2012–2014 to evaluate current design procedures and to determine design life criteria for future projects. The two main components of this evaluation were (1) visual distress surveys of 20 existing UTW pavements across the state and (2) falling weight deflectometer (FWD) testing of eight of these UTW projects.

The main goal of the visual distress surveys was to evaluate the projects by documenting all observed distresses and to determine the design features that provide favorable and unfavorable performance. The complete findings of the surveys are detailed in this report. The surveys provided significant insight about whether the existing design methodology used in Illinois was sufficient in areas such as slab thickness, panel size, the use of macro-fibers, and construction choices.

The main goal of the FWD testing was to evaluate structural performance of the projects. Because UTW layer thicknesses and support conditions can vary greatly as a function of distance along a project and may not be accurately known in the first place, it can be difficult to characterize the load carrying capacity of a UTW pavement at a given point. Deflection data collected during FWD testing were used to directly calculate load transfer efficiency to characterize joint performance, but there was no existing method to assess the in situ structural properties of UTW pavements.

To assess the structural performance of UTW, a backcalculation procedure was derived on the basis of two-dimensional finite element modeling of UTW pavements. The procedure established a backcalculated effective concrete thickness as a metric to quantify the load carrying capacity of UTW pavements. The average backcalculated effective thickness values provided a reasonable assessment of load carrying capacity that agreed with observed section performances and estimated layer thicknesses. Backcalculating the effective thickness also demonstrated variation of the structural capacity as a function of distance along the roadway, and it potentially provided a way to evaluate the condition of the concrete–asphalt bond interface and the underlying asphalt concrete layer.

The findings of the visual distress surveys and the FWD data analysis largely agreed with each other and were studied to help provide a greater understanding of factors that affect UTW performance. From this analysis, a number of conclusions and recommendations were made regarding UTW pavement design and construction.

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CHAPTER 1 INTRODUCTION

Ultra-thin whitetopping pavements (UTW), also known as bonded concrete overlays of asphalt (BCOA), consist of a 3 to 6 inch concrete inlay or overlay bonded to the surface of an existing asphalt or composite pavement structure (Harrington 2008). In 1991, the first ultra-thin whitetopping project in the United States was constructed in Louisville, Kentucky, with Illinois building its first UTW in 1998 (Winkelman 2005). UTW has seen major expansion across the state over the past 15 years, with more than 40 new projects in Illinois (Riley 2010). Currently, the Illinois Department of Transportation (IDOT) provides UTW design standards and guidelines in Chapter 53 of its *Bureau of Design and Environment Manual* (IDOT 2010).

With increasing design and construction of UTW pavements, there is a need to assess the performance of existing UTW projects to help determine design life criteria for future projects and to re-evaluate the performance of various design features, such as panel size, slab thickness, use of macro-fibers, joint spacing, underlying asphalt condition, and asphalt–concrete interface, as well as different construction techniques. To gain insight about UTW structural performance in Illinois, visual distress surveys were performed on 19 UTW projects during the summer of 2012. To complement the surveys, seven of these projects also underwent falling weight deflectometer (FWD) testing in fall 2012 and spring 2013. The resulting deflection data were analyzed to calculate joint load transfer efficiency and to backcalculate layer properties to characterize the load carrying capacity and to assess the interface bond condition of the UTW pavements. Finally, in summer 2014, distress surveys and FWD testing were carried out to investigate the cause of early-age distresses on a UTW project constructed in 2012.

This report details the findings of the visual distress surveys and the FWD testing. The results of the surveys and the analysis of the FWD deflection data are used to draw conclusions and recommendations for IDOT's existing UTW thickness design procedure and material and construction standards.

CHAPTER 2 VISUAL DISTRESS SURVEYS

Visual distress surveys of 19 UTW pavements in Illinois were conducted in summer 2012. A map of Illinois with the project locations highlighted is featured in Figure 2.1. The projects that were surveyed ranged from parking lots to intersections to mainline pavements, including both low-volume rural or residential streets and busy roads with heavy truck traffic. Several projects were 14 years old while others had just recently been completed. In Figure 2.1, purple markers indicate projects that incorporated macro-fibers, while green markers indicate projects that did not.

The main goal of these surveys was to evaluate the projects by documenting all observed distresses and to determine the design features that provide favorable and unfavorable performance. Several projects evaluated did not fit the exact definition of ultra-thin whitetopping (i.e., unbonded interface, overlay of concrete rather than bonded to the HMA substrate) but were evaluated anyway because they fell into the category of thin concrete overlays.

2.1 SURVEY METHODS

Guidelines for the distresses that were recorded during the surveys are found in Appendix A. For each project, the number of panels that were cracked, patched, or replaced was expressed as a percentage of the total number of slabs. For many projects surveyed, this analysis was further divided into different sections or stages. The total number of panels cracked was also expressed as a percentage of the total number of slabs surveyed for the project. The survey method allowed for one panel to feature multiple types of cracking, so the sum of columns containing the different categories of cracking listed separately may exceed the total number of panels cracked. Also, a comparison of the 2012 surveys with past survey data showed that some old, cracked slabs might now be patched, replaced, or counted as shattered slabs. These crack data are provided together for all projects in Table 2.1.

Concrete mixture design and other hardened concrete properties were not known for all projects, but when available this information is listed in Appendix A.

Figure 2.1. Locations of the 19 Illinois UTW projects that were surveyed in 2012.

Table 2.1. Survey Crack Data

Project/Section		Percent Slabs Cracked (2012)	Age at Time of Survey (years)	Macro-Fibers	Project Type
Decatur: Intersection of U.S. Highway 36 and Oakland Avenue		35.8	14	—	UTW
University of Illinois: Talbot Lab Parking Lot		22.0	14	—	UTW
Clay County: Sailor Springs Road		0.0	14	—	UTW (5-6 in)[b]
Tuscola: U.S. Highway 36		26.3	13	—	UTW on composite with some widening
Piatt County: County Highway 4	5.5 foot panels	1.4[a]	12	—	UTW (5 in)[b]
	11 foot panels	17.8[a]			
Cumberland County: County Highway 2		0.3	11	—	UTW (5.75 in)[b]
Oak Park: Marion Street		29.3	11	Steel	Geotextile-separated (unbonded)
Lombard: Grace Street		8.1	9	—	UTW
Chicago: Western Avenue Bus Pads	Hubbard NB	10.7	9	Synthetic	Geotextile-separated (unbonded) over concrete; bonded over asphalt
	Iowa SB	13.3			
	Ohio SB	9.3			
	Washington SB	20.0			
	Warren SB	13.3			
Chicago: Michigan Avenue Bus Pad (124th Place)		71.6	8	Synthetic	UTW on composite
Kane County: North Lorang Road		2.0	8	Synthetic	UTW with some widening
Mundelein: Schank Avenue		8.1	7	Synthetic	UTW on composite with some widening
University of Illinois: McKinley Health Center Parking Lot		0.4	6	Synthetic	UTW
University of Illinois: E-15 Parking Lot		0.3	6	Synthetic	UTW
Clay County: Bible Grove Road		0.3	4	—	UTW (5 in)[b]
Richland County: County Highway 9		0.0	3	Synthetic	UTW (5.5 in)[b]
Clay County: Xenia-Iola Road		0.4	2	Synthetic	UTW (5 in)[b]

[a]Source: ERI Inc. (2012).
[b]Overlay thickness exceeds some classic definitions of UTW (Winkelman 2005; Riley 2010).

2.2 SURVEYS

2.2.1 Decatur: Intersection of U.S. Highway 36 and Oakland Avenue

Completed in the spring of 1998, the intersection of U.S. 36 and Oakland Avenue was one of the first experimental ultra-thin whitetopping projects built in Illinois. The UTW portion of the project was a 3.5 inch inlay of a milled HMA surface constructed in the eastbound lane of U.S. 36. As of 2003, 6% to 9% of traffic was classified as heavy commercial (Winkelman 2005). Although no more recent traffic data were available, a high number of trucks passed through the intersection during the June 2012 survey, which was conducted at about 11:00 a.m. on a weekday. Project details are provided in Table 2.2. An overview and the location of the intersection are pictured in Figures 2.2 and 2.3.

Figure 2.2. Overview of U.S. 36 and Oakland Avenue in Decatur (June 2012).

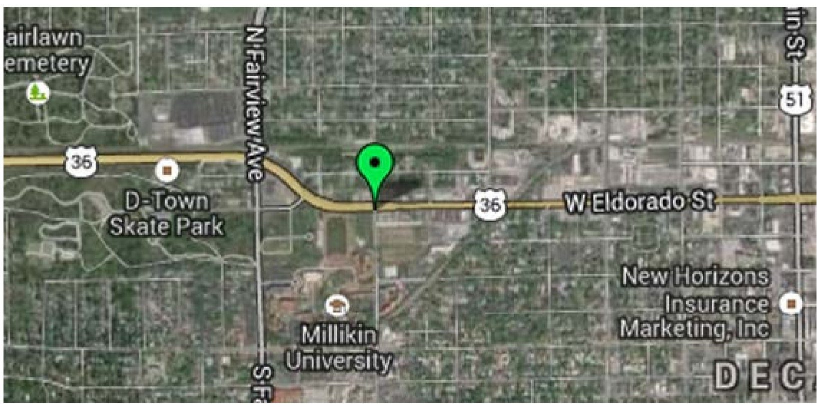

Figure 2.3. Project location of U.S. 36 and Oakland Avenue in Decatur.

Table 2.2. Decatur Project Details

Completion Date	Spring 1998[a]
Inlay Thickness	3.5 inches[a]
Underlying Thickness/ Condition	6 inches of milled HMA[a]
Slab Size	Varied between 3 by 3 feet and 4 by 4 feet
ADT	17,500 (as of 2003)[b]
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).
[b]Source: Winkelman (2005).

Overall, the project was in poor condition. As seen in Table 2.3, 35.8% of panels at this intersection featured cracking or were considered completely shattered slabs. Included in the number of panels with longitudinal cracks was a continuous longitudinal crack extending through 14 slabs. At 3 to 4 feet wide, the joints fell in the wheel path, likely contributing to the distresses. As noted in Table 2.4, the number of cracked slabs almost doubled between 2003 and 2012.

Table 2.3. Decatur Distress Survey

	Total	%
Total # Slabs	201	—
# Slabs Corner Breaks	23	11.4
# Slabs Longitudinal Cracks	36	17.9
# Slabs Transverse Cracks	9	4.5
# Slabs Diagonal Cracks	0	0.0
# Shattered Slabs	9	4.5
# Slabs Patched	5	2.5
Total # Slabs Cracked	72	35.8

Table 2.4. 2003 to 2012 Decatur Distress Survey Comparison

	2003[a]	2012
Total # Slabs	188	201
Total # Slabs Cracked	34	72
% Slabs Cracked	18.8	35.8

[a]Source: Winkelman (2005).

There were also three to five instances of partial slab blow-ups in panels caused by slab migration toward the intersection going eastbound. The inside five rows of slabs had migrated into the intersection from 1 inch to as much as 6 inches at the end of that section, where it transitions to asphalt, as seen in Figure 2.4. As hypothesized by Winkelman (2005), the slab migration is likely caused by vehicles stopping or slowing as they approach the intersection, thus “shoving” the UTW surface eastward. Faulting was observed in the transverse and longitudinal joints for a significant number of slabs, visible in Figure 2.5.

It is possible that if structural fibers had been present they may have been able to improve the performance of the project, especially in regard to slab migration and vertical alignment. The movement may have been limited if macro-fibers had been present to tie adjacent panels together. Truck traffic through the intersection likely contributed to the distresses and exacerbated the slab migration.

Five slabs had been patched with asphalt, as seen in Figure 2.6. Also, the outermost panels in a small section in the intersection were missing. It was not clear from the original construction notes whether the project was built this way or whether the panels had been removed or patched at some point.

Figure 2.4. Slab migration (June 2012).

Figure 2.5. Longitudinal cracking and visible faulting (June 2012).

Figure 2.6. Asphalt patching (June 2012).

2.2.2 University of Illinois: Talbot Lab Parking Lot

Completed in 1998, the Talbot Lab parking lot was the first UTW parking lot on the University of Illinois campus. It consisted of a 3 inch overlay of a very thin, very distressed asphalt surface. Project details are provided in Table 2.5. An overview of the parking lot and its location are pictured in Figures 2.7 and 2.8.

Table 2.5. Talbot Lab Project Details

Completion Date	1998[a]
Overlay Thickness	3 inches[a]
Underlying Thickness/ Condition	Thin, distressed asphalt[a]
Slab Size	6 by 6 feet
ADT	n/a
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).

Figure 2.7. Overview of Talbot Lab parking lot (February 2013).

Figure 2.8. Project location of Talbot Lab parking lot.

The lot was in good condition overall in 2012, especially considering its age. Although 22% of panels were cracked, as noted in Table 2.6, the majority of the distresses appeared to be the result of use of the lot as a staging area for heavy vehicles and equipment during renovation projects on Talbot Lab rather than repeated loading from parking lot traffic. Cracks were most prevalent near the entrance to the lot, as seen in Figure 2.9, but none exceeded medium severity. Spalling of cracks and joints was minimal, and there was no faulting. The pavement was still in good serviceable condition.

Table 2.6. Talbot Lab Distress Survey

	Total	%
Total # Slabs	514	—
# Slabs Corner Breaks	32	6.2
# Slabs Longitudinal Cracks	75	14.6
# Slabs Transverse Cracks	6	1.2
# Slabs Diagonal Cracks	26	5.1
# Shattered Slabs	0	0.0
# Slabs Patched	0	0.0
Total # Slabs Cracked	113	22.0

Figure 2.9. Distresses near the entrance to the Talbot lot (August 2013).

2.2.3 Clay County: Sailor Springs Road

An 8.2 mile section of Sailor Springs Road in a rural area just outside of Louisville, Illinois, was finished in 1998 as a PCC overlay with three different thickness/panel designs. The first (and longest) section was a 5 inch overlay with 11 foot by 11 foot slabs and skewed transverse joints, the second section was a 6 inch overlay featuring 15 foot long by 11 foot wide slabs with skewed transverse joints, and the third section was a 6 inch overlay with 5.5 foot by 5.5 foot slabs and skewed transverse joints. Project details are provided in Table 2.7. The project location is highlighted in Figure 2.10.

Table 2.7. Sailor Springs Road Project Details

Completion Date	1998[a]
Overlay Thickness	Varied (see description in the preceding paragraph)[a]
Underlying Thickness/ Condition	Scarified HMA[a]
Slab Size	Varied (see description in the preceding paragraph)[a]
ADT	1,200 (as of 1995)[a]
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).

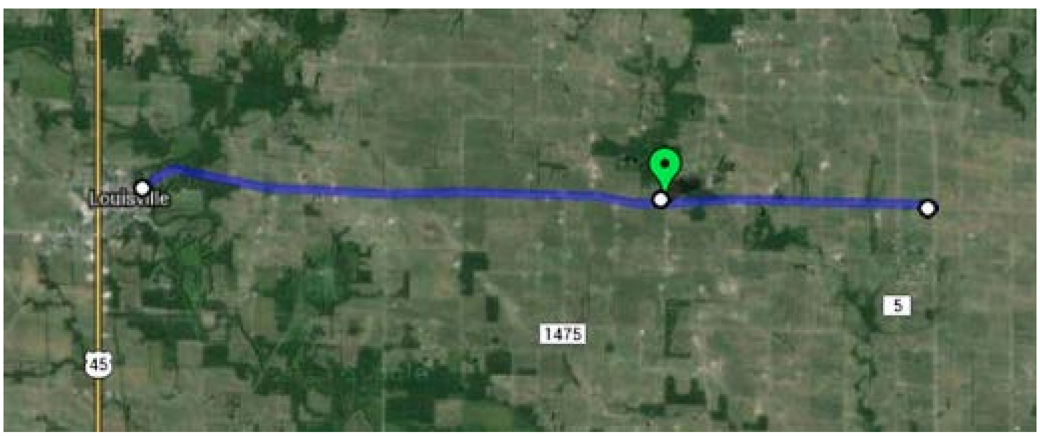

Figure 2.10. Project location of Sailor Springs Road in Clay County.

Sailor Springs Road was in good condition overall. As seen in Table 2.8, no cracked slabs were discovered during the survey. Faulting was the main problem with the project. Sections 1 and 3 provided a smooth ride with only minor faulting, but the faulting was much more problematic in Section 2 (15 foot joint spacing). The increased faulting was likely a result of the larger panel sizes and was most noticeable at the acute joint intersection angles. Otherwise, no major distresses were observed in the project, including no cracked, patched, or replaced panels. In Section 3 (5.5 foot joint spacing), there was one location with scaling and loss of support at the edge of the pavement near a mailbox. There was also some plant growth in the longitudinal joints in Section 3.

Table 2.8. Sailor Springs Road Distress Survey

Section	1	2	3	Total	%
Total # Slabs	200	132	732	1064	—
# Slabs Corner Breaks	0	0	0	0	0.0
# Slabs Longitudinal Cracks	0	0	0	0	0.0
# Slabs Transverse Cracks	0	0	0	0	0.0
# Slabs Diagonal Cracks	0	0	0	0	0.0
# Shattered Slabs	0	0	0	0	0.0
# Slabs Patched	0	0	0	0	0.0
Total # Slabs Cracked	0	0	0	0	0.0

2.2.4 Tuscola: U.S. Highway 36

A 1 mile stretch of U.S. 36 in Tuscola east of I-57 near an aggregate quarry was completed in 1999. In July 2012, it was removed and replaced with asphalt. This survey was completed in June 2012, before the reconstruction of the section. Panel sizes varied from section to section in the project. In Sections 1 (eastbound) and 2 (westbound), the longitudinal joint spacings varied because the pavement was widened during construction; the exact slab widths are given in Figure 2.11. Transverse joints were every 5 feet in these sections. Sections 3 (westbound) and 4 (eastbound) of the project both featured identical 4 by 4 foot slab sizes. Although the most recent ADT data were from 2003, a high number of trucks were still observed during the 2012 survey, which was completed on two separate weekday mornings in June. Project details are provided in Table 2.9. An overview of the project and its location are pictured in Figures 2.12 and 2.13.

Table 2.9. Tuscola Project Details

Completion Date	1999[a]
Overlay Thickness	Varied between 4 and 7 inches[a]
Underlying Thickness/ Condition	Composite pavement with top HMA layer 3 to 4.25 inches thick[a]
Slab Size	Stages 1 and 2: 4 by 4 feet, Stages 3 and 4 varied (Figure 2.11)
ADT	5,600 (1,200 trucks)[b]
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).
[b]Source: Winkelman (2005).

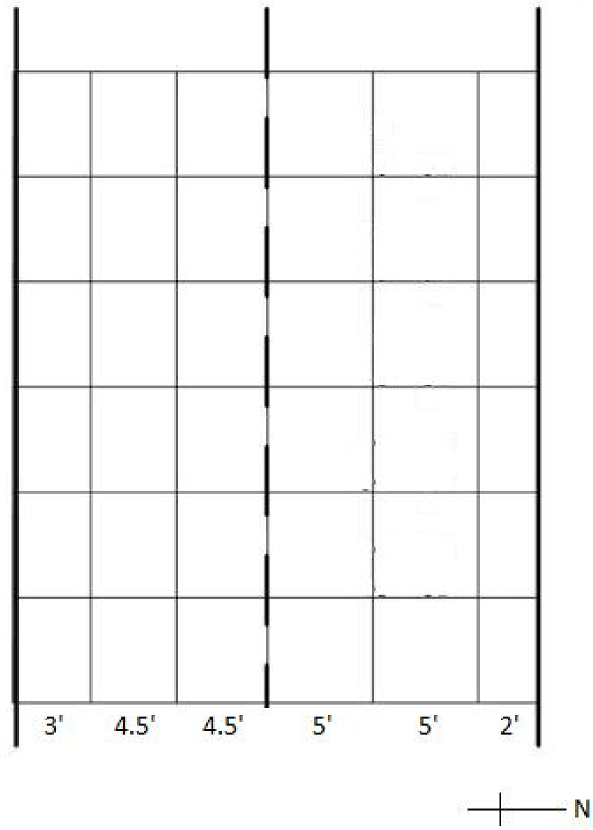

Figure 2.11. Slab dimensions for Sections 1 and 2 of Tuscola project.

Figure 2.12. Overview of U.S. 36 in Tuscola (June 2012).

Figure 2.13. Project location of U.S. 36 in Tuscola.

Overall, U.S. 36 was in very poor condition. As shown in Table 2.10, 20.8% of all panels were cracked, which was substantially more than had been found in surveys performed between 1999 and 2004 (Tables 2.11 and 2.12). Issues with the longitudinal joints were the primary cause of distress. The longitudinal joints fell in the wheel path in each section of the project. This problem, in conjunction with the fact that the joints had been saw-cut too wide during construction, left the longitudinal joints very susceptible to opening and spalling. In turn, extensive corner cracking (15.3% of all panels) developed along the longitudinal joints. Figure 2.14 shows the opened joints and corner cracking along a longitudinal joint. As seen in the figure, some of the corner cracks had been patched with asphalt. Use of a thinner saw blade would have reduced this opening and slowed the deterioration.

Table 2.10. Tuscola Distress Survey

Section	1	2	3	4	Total	%
Total # Slabs	1410	1401	1002	996	4809	—
#Slabs Corner Breaks	392	75	189	79	735	15.3
# Slabs Longitudinal Cracks	19	8	3	8	38	0.8
# Slabs Transverse Cracks	36	21	28	35	120	2.5
# Slabs Diagonal Cracks	30	9	16	24	79	1.6
# Shattered Slabs	26	1	11	16	54	1.1
# Slabs Patched	68	18	16	22	124	2.6
# Slabs Replaced	0	15	0	0	15	0.3
Total # Slabs Cracked	480	112	243	166	1001	20.8

Table 2.11. Previous Tuscola Survey Data (Roesler and Bordelon 2008)

Date of Survey	8/25/1999						4/20/2001						6/29/2004					
Time of Survey (days)	96						700						1866					
Stage	1	2	3	4			1	2	3	4			1	2	3	4		
Length of Survey Stage	470	467	334	332			470	467	334	332			470	467	334	332		
Width (# Slabs)	3	3	3	3			3	3	3	3			3	3	3	3		
Slab Sizes (feet)	—	—	—	—		%	—	—	—	—		%	—	—	—	—		%
# Slabs	1410	1401	1002	996	**3813**		1410	1401	1002	996	**3813**		1410	1401	1002	996	**3813**	
# Slabs Corner Breaks	17	0	5	5	**27**	**0.71**	31	0	8	19	**58**	**1.52**	76	3	20	46	**145**	**3.80**
# Slabs Diagonal Cracks	3	0	0	0	**3**	**0.08**	22	1	1	1	**25**	**0.66**	43	2	1	2	**48**	**1.26**
# Slabs Debonding	0	0	0	0	**0**	**0.00**	0	0	0	0	**0**	**0.00**	34	35	2	27	**98**	**2.57**
# Slabs Longitudinal Cracks	0	0	0	0	**0**	**0.00**	2	0	0	1	**3**	**0.08**	7	2	2	4	**15**	**0.39**
# Slabs Transverse Cracks	0	0	0	0	**0**	**0.00**	6	0	2	2	**10**	**0.26**	32	4	6	8	**50**	**1.31**
# Slabs Patched	0	0	0	0	**0**	**0.00**	0	0	0	0	**0**	**0.00**	11	2	6	3	**22**	**0.58**

Table 2.12. Tuscola Distress Survey Comparison

	1999[a]	2001[a]	2004[a]	2012
Total # Slabs	3813	3813	3813	4809
Total # Slabs Cracked	30	96	258	1001
% Slabs Cracked	0.79	2.52	6.77	20.8

[a]Source: Roesler et al. (2008).

Although corner cracking was the most prevalent distress, other types of cracking and shattered slabs were frequent as well, especially at the ends of the project. Faulting was a major issue throughout the entire section, contributing to a very rough ride. Additionally, panels in the eastbound and westbound lanes were migrating in opposite directions, as seen in Figure 2.15. In this case, the migration may have been due to thermal movements. As with the Decatur project, application of macro-fibers would likely have helped slow the faulting and slab migration.

Water ponded in the outer longitudinal joint, as shown in Figure 2.16. The pavement—shoulder joint, shown in Figure 2.16, also contributed to the pavement distresses observed. The asphalt concrete shoulders were elevated higher than the pavement edge in places, trapping water in the longitudinal joints instead of directing the runoff laterally from the shoulder. This water along the shoulder joints may have caused a loss of support along the edges of the pavement, contributing to the distresses.

Figure 2.14. Corner cracking along a longitudinal joint (June 2012).

Figure 2.15. Slab migration (June 2012).

Figure 2.16. Water buildup between the shoulder and pavement edge (June 2012).

2.2.5 Piatt County: County Highway 4

Ultra-thin whitetopping along a 4.5 mile long stretch of County Highway 4 in Piatt County near Monticello was completed in 2000. The project had two distinct sections featuring 5.5 by 5.5 foot and 11 by 11 foot panels with skewed transverse joints. Project details are provided in Table 2.13. The location of County Highway 4 is pictured in Figure 2.17.

Table 2.13. Piatt County Project Details

Completion Date	2000[a]
Overlay Thickness	5 inches[a]
Underlying Thickness/ Condition	4 inches of milled HMA[a]
Slab Size	5.5 by 5.5 feet and 11 by 11 feet
ADT	2,150 (7.2% trucks)[a]
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).

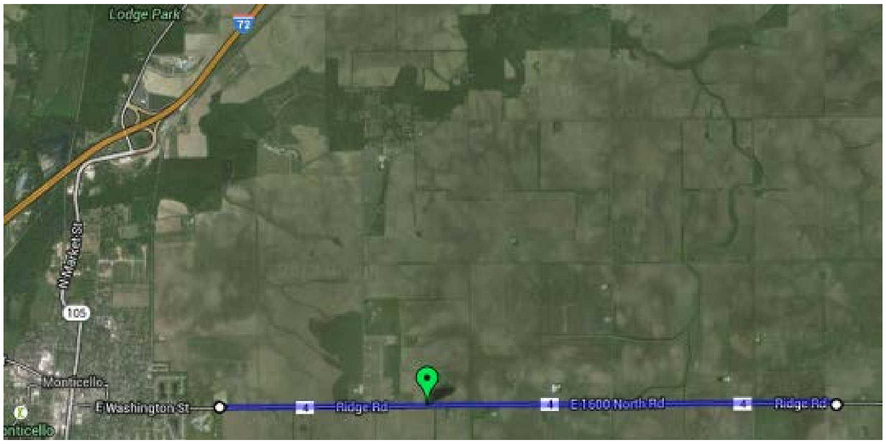

Figure 2.17. Project location of County Highway 4.

An informal survey was performed in June 2012 because of adverse traffic and weather conditions. From this survey, it was obvious that there was severe faulting and a high number of cracked panels, especially in the 11 foot section. Longitudinal cracking down the middle of the 11 foot slabs and at the edges of the pavement was particularly noticeable, which can be seen in Figure 2.18.

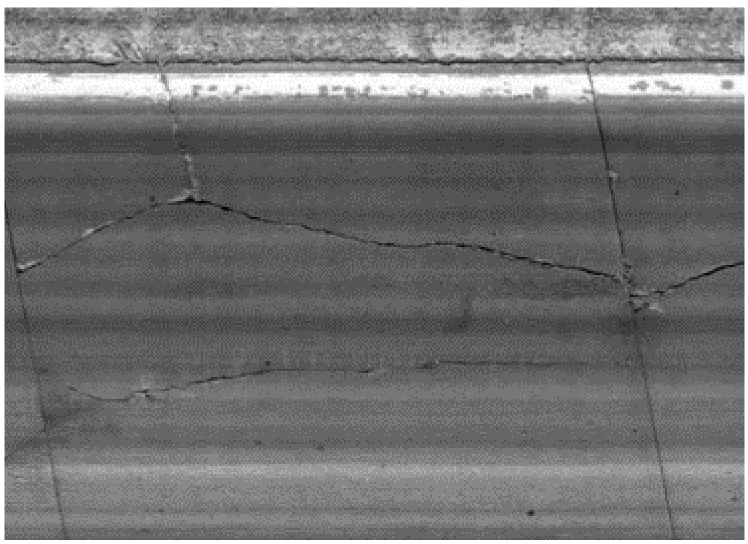

Figure 2.18. Longitudinal cracking in the middle of the 11 foot panels (ERI 2012).

A complete report on the pavement condition of County Highway 4 was completed by ERI Inc. in September 2012. The report indicated that longitudinal cracking, corner breaks, cracking near the edge of the pavement, and shattered slabs were the primary distresses found in the project. Only 1.36% of all panels in the 5.5 foot section were cracked, compared with 17.8% of the 11 foot panels, with as many as 58% of the panels cracked in portions of the 11 foot section (ERI 2012). This part of County Highway 4 is being prepared for rehabilitation in the next few years.

2.2.6 Cumberland County: County Highway 2

A 3.5 mile section of County Highway 2 in rural Cumberland County was completed in 2001. The project featured skewed transverse joints and 5 foot aggregate shoulders. Project details are provided in Table 2.14. An overview of County Highway 2 and its location are pictured in Figures 2.19 and 2.20.

Table 2.14. Cumberland County Project Details

Completion Date	2001[a]
Overlay Thickness	5.75 inches[a]
Underlying Thickness/ Condition	3.5 inches of milled HMA[a]
Slab Size	6 by 5.5 feet
ADT	3,100[a]
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).

Figure 2.19. Overview of Cumberland County Highway 2 (June 2012).

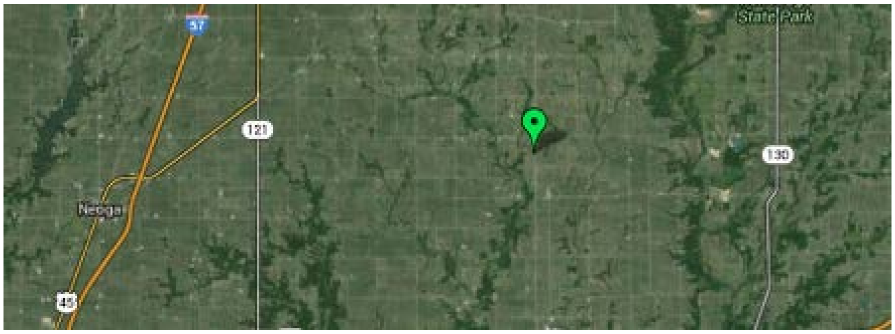

Figure 2.20. Project location of Cumberland County Highway 2.

County Highway 2 was in excellent condition. There were two transverse cracks that covered a total of four panels, as noted in Table 2.15, but they appeared to be just shrinkage cracks resulting from an uncracked transverse joint. Dominant joints were not apparent. There was some low-severity joint spalling scattered throughout the project, with the greatest approximately 4 inches.

Table 2.15. Cumberland County Distress Survey

	Total	%
Total # Slabs	1440	—
# Slabs Corner Breaks	0	0.0
# Slabs Longitudinal Cracks	0	0.0
# Slabs Transverse Cracks	4	0.3
# Slabs Diagonal Cracks	0	0.0
# Shattered Slabs	0	0.0
# Slabs Patched	0	0.0
Total # Slabs Cracked	4	0.3

2.2.7 Oak Park: Marion Street

Completed in 2001 and about 1 mile in length, Marion Street is a thin, unbonded concrete overlay. This section is 4 inches of concrete placed on a woven geotextile layer over an old concrete surface. The key feature of interest of Marion Street is that it was the oldest project visited that used macro-fibers. Transverse joints were hand-tooled, longitudinal joints were saw-cut, and all joints were sealed. Project details are provided in Table 2.16. An overview of Marion Street and its location are pictured in Figures 2.21 and 2.22.

Table 2.16. Marion Street Project Details

Completion Date	2001[a]
Overlay Thickness	4 inches[a]
Underlying Thickness/ Condition	Geotextile on top of old concrete[a]
Slab Size	6 feet, 10 inches by 5.5 feet
ADT	3,469 (5% trucks)[a]
Fiber Reinforcement	4 lb/yd^3 crimped steel[a]

[a]Source: Riley (2010).

Figure 2.21. Overview of Marion Street and longitudinal cracking (August 2012).

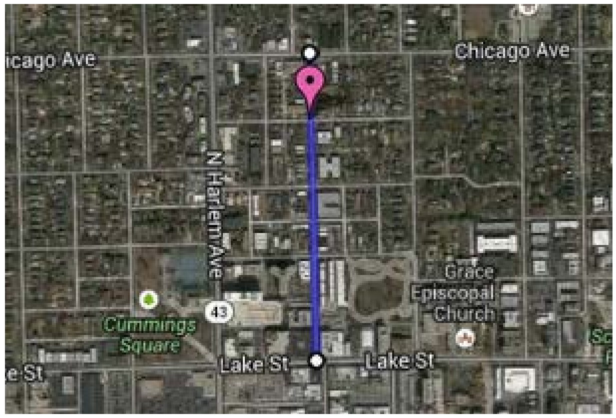

Figure 2.22. Project location of Marion Street in Oak Park.

Marion Street was in good condition overall despite the high number of cracked slabs, 29.3%, as noted in Table 2.17. The most common distress was longitudinal cracking. Nearly the entire project length had one extended longitudinal crack that was caused by the extra-wide 6 foot, 10 inch panels. The longitudinal cracking can be seen in Figures 2.21 and 2.23, and it accounted for cracking in roughly 25% of all panels. Square 5 or 6 foot panels likely would have performed much better. These longitudinal cracks occurred in the middle two rows of slabs because parking was allowed on the west side of the street; therefore, traffic generally traveled in the middle of the street.

Table 2.17. Marion Street Distress Survey

	Total	%
Total # Slabs	300	—
# Slabs Corner Breaks	9	3.0
# Slabs Longitudinal Cracks	68	22.7
# Slabs Transverse Cracks	5	1.7
# Slabs Diagonal Cracks	11	3.7
# Shattered Slabs	2	0.7
# Slabs Patched	0	0.0
Total # Slabs Cracked	88	29.3

Corner and diagonal cracking also occurred near the pavement edges. Most of the corner cracks had been sealed, pictured in Figure 2.24, at the time of this survey. However, with a few exceptions, the macro-fibers appeared to have done a good job keeping the joints and cracks tight. The pavement was still in good serviceable condition overall. Reflective cracking from the old concrete street did not appear to be a problem in the current Marion Street performance.

Many of the steel macro-fibers lying horizontal to the surface popped out of the pavement, although this did not cause any distress except to affect the visual appearance of the street. The fibers themselves were corroded and popping out of the surface of the pavement in certain places but had not led to additional distresses or damage to vehicles. The ride down the street was loud, which was likely due to the hand-tooled transverse joints and possibly the unbonded interface. There was no faulting noted during the survey and no abnormal surface roughness. There were some areas with vertical misalignment between the pavement and the curb, pictured in Figure 2.25, but this did not cause any other pavement distress.

Figure 2.23. Continuous longitudinal cracking (August 2012).

Figure 2.24. Corner cracking that had been sealed (August 2012).

Figure 2.25. Vertical misalignment between the pavement edge and curb (August 2012).

2.2.8 Lombard: Grace Street

A roughly 0.25 mile section of Grace Street in a residential area of Lombard, Illinois, was constructed with a 4 inch UTW in 2003. The bonded concrete overlay was not tied to the adjacent curbs. Project details are provided in Table 2.18. An overview of Grace Street and the project location are pictured in Figures 2.26 and 2.27. The project was in very good condition overall in terms of serviceability. Although 8.1% of the slabs were cracked, as seen in Table 2.19, almost all of these were concentrated along the eastern edge of the pavement adjacent to residential driveways, as in Figure 2.28. These distresses did not appear on the western edge of the street, where there were not driveways.

Table 2.18. Grace Street Project Details

Completion Date	2003[a]
Overlay Thickness	4 inches[a]
Underlying Thickness/ Condition	11 inches of HMA[a]
Slab Size	5.5 by 5.5 feet
ADT	Unknown[a]
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).

Table 2.19. Grace Street Distress Survey

	Total	%
Total # Slabs	813	%
# Slabs Corner Breaks	58	7.1
# Slabs Longitudinal Cracks	14	1.7
# Slabs Transverse Cracks	0	0
# Slabs Diagonal Cracks	2	0.3
# Shattered Slabs	0	0
# Slabs Patched	0	0
# Slabs Replaced	24	3.0
Total # Slabs Cracked	66	8.1

Figure 2.26. Overview of Grace Street (August 2012).

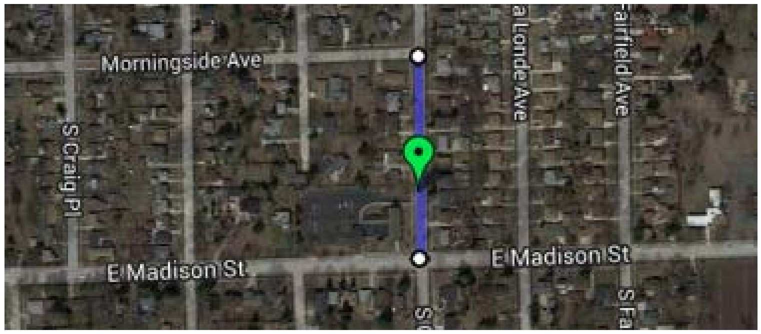

Figure 2.27. Project location of Grace Street in Lombard.


Figure 2.28. Cracking in slabs at the east pavement edge (August 2012).

Some low-severity slab migration (less than 0.25 inch) was seen in the outer slabs along the west side of the project—a common problem with projects that did not contain fibers. The cause of the migration was not immediately clear because Grace Street had a low amount of traffic, including trucks. The migration may have been a result of thermal expansion/contraction. Little to no faulting was observed, and the ride was smooth.

Debonding was detected along both the outside and inside longitudinal joints throughout the project in both lanes and along the edges. Only the debonding on the eastern edge of the pavement was accompanied by visual distress. Two large sections of panels had been replaced, but the replacement was a result of sidewalk or utility work rather than poor UTW performance.

2.2.9 Schaumburg: IDOT District 1 Parking Lot

The IDOT District 1 office parking lot in Schaumburg was built as a demonstration project with varying concrete thicknesses, panel sizes, and fiber dosages (Riley 2010). The location of the lot is pictured in Figure 2.29. No crack survey was performed because exact data for the various pavement features were not available, including where the macro-fiber dosages changed. General information about construction conditions, thickness of the overlay at certain locations, and panel sizes was available; thus, observations were still made. Other pertinent project details include the facts that curbs were finished before the concrete pavement was cast and a laser screed was used to pave the lot.

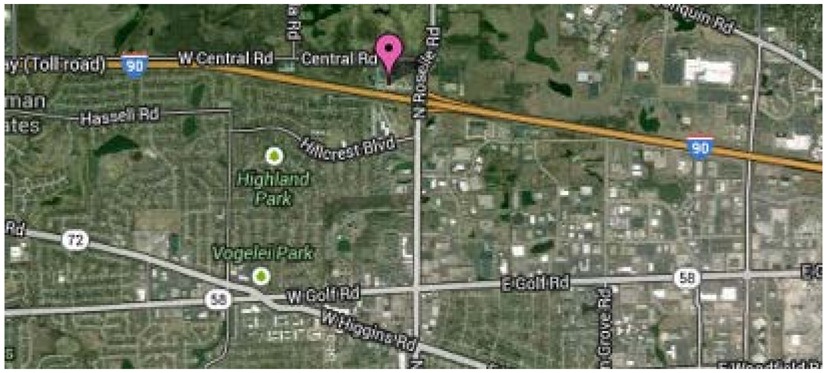

Figure 2.29. Location of IDOT District 1 parking lot in Schaumburg.

Project performance varied greatly across the lot. Most of the lot featured concrete between 4 and 6 inches thick over either stone or a 6 inch asphalt layer and was in good serviceable condition. However, some areas of the lot performed very poorly. There were severe cracks of all types and shattered slabs at the front entrance to the lot, which were likely due to the inadequate slab thickness (2 to 3 inches). There was also a cold milling mistake in this section where the HMA layer had been milled down to about 2.5 inches (it was intended to be 6 inches). These concrete distresses were accompanied by faulting, spalled cracks, and debonding. A shattered slab from this area of the lot is pictured in Figure 2.30.

Figure 2.30. Shattered slab near the front entrance (August 2012).

There were also distresses at the other entrances to the lot, as shown in Figure 2.31, but the cracks and joints held together much better than in the thinner-pavement area, and there were fewer instances of debonding. This improved performance may indicate that a higher-than-normal macro-fiber dosage was used in these areas or could be attributed to a sufficiently thick overlay and a good base. The area featured in Figure 2.31, with low-severity distresses, did not appear to contain macro-fibers.

Figure 2.31. Less severe cracking near a different lot entrance (August 2012).

2.2.10 Chicago: Western Avenue Bus Pads

The Western Avenue bus pads in Chicago were constructed in 2003. Each bus pad measured approximately 10 by 100 feet and featured a 4 inch concrete inlay with 40 by 48 inch joint spacing. The thin concrete inlay was technically considered a bonded/unbonded hybrid, with the support layer conditions beneath the concrete varying between bus stops. A higher-than-normal macro-fiber dosage (0.5%) was used on this project (Riley 2010). Project details are provided in Table 2.20. Figure 2.32 shows the stop at Western and Iowa as an overview of a typical Western Avenue UTW bus pad. The general corridor of stops along Western Avenue is highlighted in Figure 2.33.

Table 2.20. Western Avenue Project Details

Completion Date	2003[a]
Inlay Thickness	4 inches[a]
Underlying Thickness/ Condition	Varied[a]
Slab Size	40 by 48 inches
ADT	200+ buses per day[a]
Fiber Reinforcement	7.5 to 8.5 lb/yd^3 synthetic[a]

[a]Source: Riley (2010).

Figure 2.32. Typical UTW bus pad at Western and Iowa (August 2012).

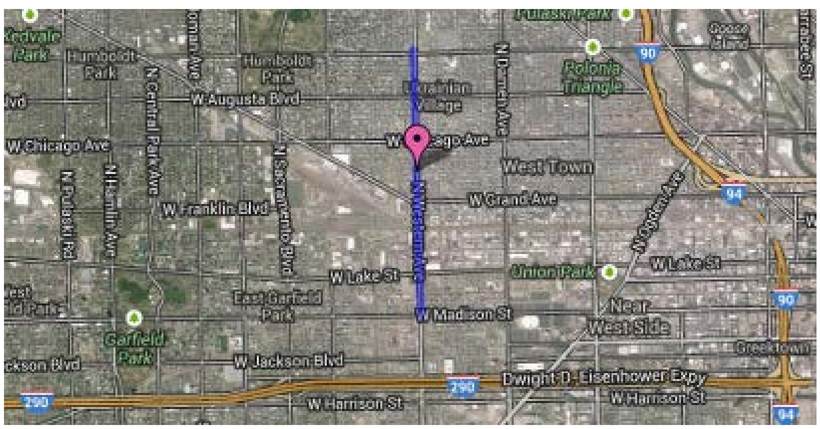

Figure 2.33. Location of the Western Avenue bus pads.

Overall, the bus pads were in very good condition from a functional performance perspective. Although the pads had cracked panels, noted in Table 2.21, the joints and cracks appeared to have been kept tight by the fibers, and little faulting was observed. The main issue that likely led to cracking in many sections was a debonded interface between the concrete slabs and foundation, especially along the curb.

Table 2.21. Western Avenue Distress Survey

Stop (Intersection and Direction of Traffic)	Hubbard NB	Iowa SB	Ohio SB	Washington SB	Warren SB
Total # Slabs	75	60	75	75	75
Total # Slabs Cracked	8	8	7	15	8
% Slabs Cracked	10.7	13.3	9.3	20.0	13.3

Certain observations unique to particular stops included faulting in a longitudinal joint at the beginning of the Washington stop that may have resulted from a distress in the adjacent asphalt, pictured in Figure 2.34; joint opening and faulting near a patch at a manhole at the Hubbard stop, pictured in Figure 2.35; and some minor slab migration on the approach panels of the Warren stop. The pad with the highest percentage of cracked panels, at Washington, was noted to have had the toughest construction conditions. Also at this bus stop, a geotextile had been placed between the concrete and base layers toward the beginning of the pad but had not led to any observable distresses.

Figure 2.34. Joint opening and faulting at Western and Washington (August 2012).

Figure 2.35. Joint opening and faulting near a manhole patch at Western and Hubbard (August 2012).

2.2.11 Chicago: South Michigan Avenue and 124th Place Bus Pad

The South Michigan Avenue bus pads in Chicago, completed in 2004, were designed similarly to those on Western Avenue, with 10 by 100 foot inlays with 40 by 48 inch joint spacing. However, a more typical fiber dosage was used, as opposed to the elevated dosage used on Western Avenue. At the time of the 2012 survey, only one of the original bus pads (at 124th Place) remained. The rest of the pads had been replaced. Project information is provided in Table 2.22. An overview of the last remaining pad and its location can be seen in Figures 2.36 and 2.37.

Table 2.22. South Michigan Avenue Project Details

Completion Date	2004[a]
Inlay Thickness	4 inches[a]
Underlying Thickness/ Condition	Varied[a]
Slab Size	40 by 48 inches
ADT	Around 50 buses per day[a]
Fiber Reinforcement	4 lb/yd^3 synthetic[a]

[a]Source: Riley (2010).

Figure 2.36. UTW bus pad at South Michigan and 124th Place (August 2012).

Figure 2.37. Project location of the UTW bus pad at South Michigan and 124th Place.

As seen in Table 2.23, the remaining pad experienced significant distresses. Although the other pads had been replaced, a 2007 report from the University of Illinois at Chicago detailed their early performance. The report indicated that all of the pads exhibited similar distresses to those observed at 124th Place and that the majority of the pads experienced early failure. It was concluded in the 2007 report that a lack of consideration of the underlying layer in design and poor quality control during construction were the main causes of distress (Issa 2007). Despite the extensive cracking in the last remaining pad, the cracks and joints were tight and the pavement was still in serviceable condition.

Table 2.23. South Michigan Avenue Distress Survey

Total # Slabs	81
Total # Slabs Cracked	58
% Slabs Cracked	71.6

2.2.12 Kane County: North Lorang Road

North Lorang Road, a short (0.5 mile) road serving a quarry, was completed in 2004. Slabs measured 4 by 4 feet except for the panels on the east edge of the pavement, which were widened to 5 feet. Joints were single cut with no sealant. At the time of the 2012 survey, an average of 30 trucks per day traveled down this section to the quarry and back, compared with as many as 120 per day at the time of construction (Riley 2010). Project details are provided in Table 2.24. An overview of North Lorang Road and its location are shown in Figures 2.38 and 2.39.

Table 2.24. North Lorang Road Project Details

Completion Date	2004[a]
Overlay Thickness	4.25 to 4.5 inches[a]
Underlying Thickness/ Condition	3 to 3.5 inches of HMA over stone[a]
Slab Size	4 by 4 feet
ADT	30 trucks per day
Fiber Reinforcement	4 lb/yd^3 synthetic[a]

[a]Source: Riley (2010).

Figure 2.38. Overview of North Lorang Road (August 2012).

Figure 2.39. Project location of North Lorang Road in Kane County.

Overall, the project was in excellent condition. As noted in Table 2.25, there was a low percentage of cracked panels. The majority of the cracked slabs occurred on the widened east edge of the pavement on the south end of the project, which was adjacent to residential driveways. An example of this cracking is shown in Figure 2.40. Debonding was detected near the joints by these cracks. There were no driveways on the west pavement edge, and no similar distresses were observed on that side.

Table 2.25. North Lorang Road Distress Survey

End	South	North	Total	%
Total # Slabs	2250	900	3150	—
# Slabs Corner Breaks	25	0	25	0.8
# Slabs Longitudinal Cracks	21	0	21	0.7
# Slabs Transverse Cracks	3	0	3	0.1
# Slabs Diagonal Cracks	4	0	4	0.1
# Shattered Slabs	0	0	0	0
# Slabs Patched	0	0	0	0
Total # Slabs Cracked	44	0	44	1.4

The macro-fibers appeared to have kept the joints tight and reduced the rate of crack deterioration. There were no instances of slab migration. However, at the south end of the project, the second longitudinal joint from the western edge of the pavement opened up and exhibited spalling over a stretch of 50 to 55 panels, shown in Figure 2.41. This widening was accompanied by growth in the cracks, minor faulting, and rotation of the pavement around the joint. It appeared the macro-fibers were not effective in keeping the pavement together after the joint had opened up.

The joint opening and rotation could be due to several factors. One possibility was loss of shoulder or edge support. Another possible cause was truck traffic—this joint was in the "loaded" lane, traveled by the trucks after leaving the quarry with a full load. Over the remainder of the project, the ride was very smooth.

Figure 2.40. Cracked panels on the widened pavement edge (August 2012).

Figure 2.41. Longitudinal joint opening and spalling (August 2012).

2.2.13 Mundelein: Schank Avenue

Schank Avenue, a suburban, 0.5 mile concrete overlay of a composite (asphalt over concrete) pavement, was completed in 2005. The pavement was widened at intersections at both ends of the project. During the survey in August 2012, a high truck volume was observed. Project details are provided in Table 2.26. An overview of Schank Avenue and its location can be seen in Figures 2.42 and 2.43.

Table 2.26. Schank Avenue Project Details

Completion Date	2005[a]
Overlay Thickness	4 inches[a]
Underlying Thickness/ Condition	HMA over previously existing concrete[a]
Slab Size	4 by 4 feet
ADT	11,700[a]
Fiber Reinforcement	4 lb/yd^3 synthetic[a]

[a]Source: Riley (2010).

Figure 2.42. Overview of Schank Avenue (August 2012).

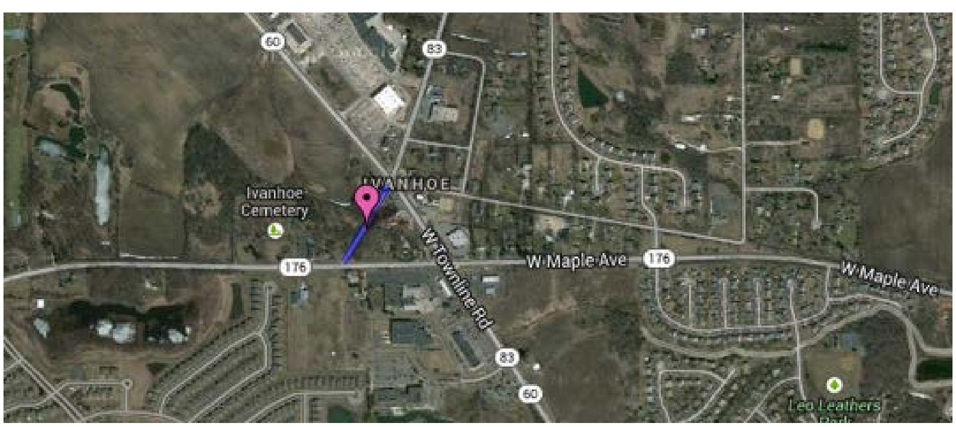

Figure 2.43. Project location of Schank Avenue in Mundelein.

Overall, the project was in good condition, with roughly 8% of all slabs cracked, as seen in Table 2.27. However, serious support problems were developing. The main issue was in the middle of the roadway section, which lay on an embankment that appeared to be settling. A major loss of shoulder support at the edges of the pavement resulted from the settlement, causing the opening of the outermost longitudinal joints in both lanes and the rotation of the outside panels away from the rest of the pavement. Faulting, debonding, and low-severity slab migration (less than 5 mm) were all prevalent at these edge areas and may worsen as the embankment continues to settle. A location with obvious loss of shoulder support is shown in Figure 2.44, and examples of the resulting distresses are found in Figures 2.45 through 2.47.

Table 2.27. Schank Avenue Distress Survey

	Total	%
Total # Slabs	1019	—
# Slabs Corner Breaks	44	4.3
# Slabs Longitudinal Cracks	3	0.3
# Slabs Transverse Cracks	16	1.6
# Slabs Diagonal Cracks	9	0.9
# Shattered Slabs	0	0.0
# Slabs Patched	16	1.6
Total # Slabs Cracked	82	8.1

Figure 2.44. Loss of shoulder support (August 2012).

Figure 2.45. Faulting at the outside longitudinal joint and asphalt patching (August 2012).

Figure 2.46 Close-up of faulting in the outside longitudinal joint (August 2012).

Figure 2.47. Slab migration at the outside longitudinal joint (August 2012).

Schank Avenue was more stable toward the centerline of the pavement but was still prone to some of the same issues. The longitudinal joint in the centerline in the section on the embankment also opened up, resulting in spalling, debonding, slab migration, and some faulting as well. Corner and transverse cracking were located primarily in the wide areas of the pavement near the intersections, as shown in Figure 2.48. Debonding was also detected in the widened sections.

Figure 2.48. Transverse and corner cracking at widened pavement section (August 2012).

2.2.14 University of Illinois: McKinley Health Center Parking Lot

The McKinley Health Center parking lot was completed in 2006 as the second UTW parking lot at the University of Illinois. Project details are provided in Table 2.28, and the project overview and location are shown in Figures 2.49 and 2.50.

Table 2.28 McKinley Lot Project Details

Completion Date	2006[a]
Overlay Thickness	3.5 inches[a]
Underlying Thickness/ Condition	3.5 to 4.5 inches of untreated HMA[a]
Slab Size	4 by 4 feet
ADT	n/a
Fiber Reinforcement	3 lb/yd^3 synthetic[a]

[a]Source: Roesler et al. (2008).

Figure 2.49. Overview of the McKinley parking lot (May 2013).

Figure 2.50. Project location of the McKinley parking lot.

The McKinley parking lot was in excellent functional and structural condition overall. Less than 0.5% of all panels were cracked, as noted in Table 2.29. Of the longitudinal cracks found in the lot, the two in the northwest section propagated from a curb joint and the six in the northeast section propagated from a manhole. The corner breaks were graded low to medium severity.

Table 2.29. McKinley Lot Distress Survey

Section	NW	NE	SE	Total	%
Total # Slabs	1140	1240	607	2987	—
# Slabs Corner Breaks	2	0	1	3	0.1
# Slabs Longitudinal Cracks	2	6	0	8	0.3
# Slabs Transverse Cracks	0	0	0	0	0.0
# Slabs Diagonal Cracks	0	0	0	0	0.0
# Shattered Slabs	0	0	0	0	0.0
# Slabs Patched	0	0	0	0	0.0
Total # Slabs Cracked	4	6	1	11	0.4

Several distresses were found in the northwest section of the lot including scaling, joint spalling, and plastic shrinkage cracking, which can be seen in Figure 2.51. In total, 35 panels showed shrinkage cracks, which were easier to identify than normal because it rained the morning of the survey. These distresses were likely the result of a combination of hot and windy conditions on the day of the pour. These distributed shrinkage cracks were not counted in Table 2.29.

Figure 2.51. Plastic shrinkage cracking (June 2012).

Areas of debonding were found mainly near the construction joints and in low spots where water is directed to the drainage inlets. Fiber balls near the surface were seen in all sections of the parking lot and measured about 1 to 2 inches in diameter. An example of a fiber ball pop-out is pictured in Figure 2.52. There did not appear to be any other distresses associated with this pop-out.

Figure 2.52. Fiber ball pop-out (June 2012).

Originally, it was possible to see dominant joints in this lot, but now these joints are filled with debris. It could not be determined which joints were the dominant joints, except for a few. There was no link between dominant joints and distresses. In the end, the majority of the distresses that were found in the McKinley parking lot were not serious and appeared to be due to construction problems.

2.2.15 University of Illinois: E-15 Parking Lot

One half of the E-15 parking lot at the University of Illinois was rehabilitated in 2006 with a bonded concrete overlay of an existing, untreated HMA surface. The general condition of the underlying pavement can be seen in the overview of the lot pictured in Figure 2.53, where the dividing line between the original section and the overlay is clear. The remainder of the lot was completed as a UTW in August 2012. This survey was performed in June 2012 before the new construction began. Project details are provided in Table 2.30. The project location is marked in Figure 2.54.

Figure 2.53. Overview of E-15 parking lot (June 2012).

Table 2.30. E-15 Lot Project Details[a]

Completion Date	2006[b]
Overlay Thickness	3.5 inches[b]
Underlying Thickness/ Condition	2.5 inches of untreated HMA[b]
Slab Size	4 by 4 feet
ADT	n/a
Fiber Reinforcement	3 lb/yd^3 synthetic[b]

[a]Project details are for the 2006 section. Overlay details for the 2012 section are unknown.
[b]Source: Roesler et al. (2008).

Figure 2.54. Project location of the E-15 parking lot.

Overall, the E-15 lot was in excellent condition. As shown in Table 2.31, just 0.25% of all panels were cracked. All of the cracking was graded as low severity. The corner breaks and diagonal cracks in Bays 1 and 2 were either near islands or propagated from drains, as shown in Figure 2.55.

Areas of debonding were found mainly near the construction joints. In Bay 2, there was some high-severity spalling at the construction joints. Low-severity joint spalling and some scaling were observed in areas of Bay 3. As in the McKinley parking lot, fiber balling at the surface occurred throughout the project.

Table 2.31. E-15 Lot Distress Survey[a]

Bay	1	2	3	Total	%
Total # Slabs	832	1165	1198	3195	—
# Slabs Corner Breaks	2	1	2	5	0.2
# Slabs Longitudinal Cracks	0	0	0	0	0.0
# Slabs Transverse Cracks	0	0	0	0	0.0
# Slabs Diagonal Cracks	1	2	0	3	0.1
# Shattered Slabs	0	0	0	0	0.0
# Slabs Patched	0	0	0	0	0.0
Total # Slabs Cracked	3	3	2	8	0.3

[a]All three bays of the lot cited in this table are in the 2006 section.

Figure 2.55. Corner cracking (June 2012).

2.2.15 Clay County: Bible Grove Road

Bible Grove Road, a 7.5 mile long rural highway, was completed in 2008. The project was divided into two sections. West of and through the town of Bible Grove, the project featured 5.5 by 5.5 foot panels. East of town, the joints were cut into 11 by 11 foot panels. There were no shoulders along the road, although there were curbs in the short section that ran through town. Project details are provided in Table 2.32. An overview of the project and its location are shown in Figures 2.56 and 2.57.

Table 2.32. Bible Grove Road Project Details

Completion Date	2008[a]
Overlay Thickness	5 inches[a]
Underlying Thickness/ Condition	Existing HMA of unknown thickness[a]
Slab Size	5.5 by 5.5 feet (west), 11 by 11 feet (east)
ADT	550[a]
Fiber Reinforcement	None[a]

[a]Source: Riley (2010).

Figure 2.56. Overview of Bible Grove Road (June 2012).

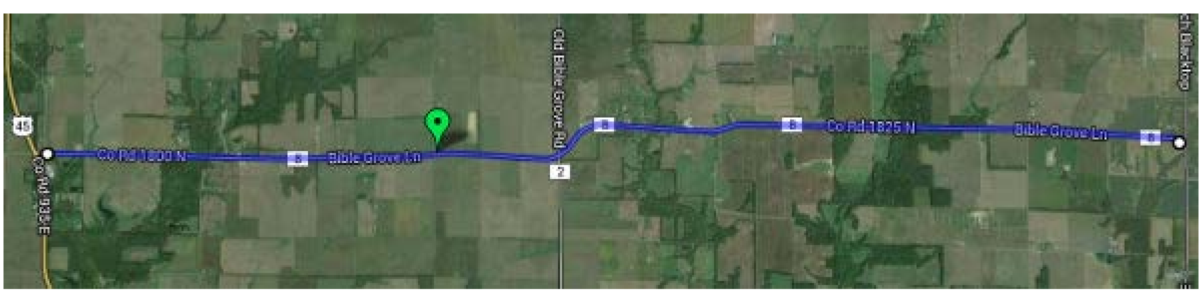

Figure 2.57. Project location of Bible Grove Road in Clay County.

Overall, the project was in good condition. There were few cracked slabs, as noted in Table 2.33, but there were a number of distresses that were likely caused by construction issues. In the 5.5 foot panel section, there were several instances of misalignment of transverse joints in the eastbound lane. In three cases, this irregularity led to transverse cracks across the adjacent panels in the westbound lane, as demonstrated in Figure 2.58. A panel had also been replaced at a misaligned joint.

Table 2.33. Bible Grove Road Distress Survey

Section	West	East	Total	%
Total # Slabs	520	118	638	—
# Slabs Corner Breaks	0	8	8	1.3
# Slabs Longitudinal Cracks	0	0	0	0.0
# Slabs Transverse Cracks	8	0	8	1.3
# Slabs Diagonal Cracks	0	0	0	0.0
# Shattered Slabs	0	0	0	0.0
# Slabs Patched	3	3	4	0.5
Total # Slabs Cracked	8	8	16	0.3

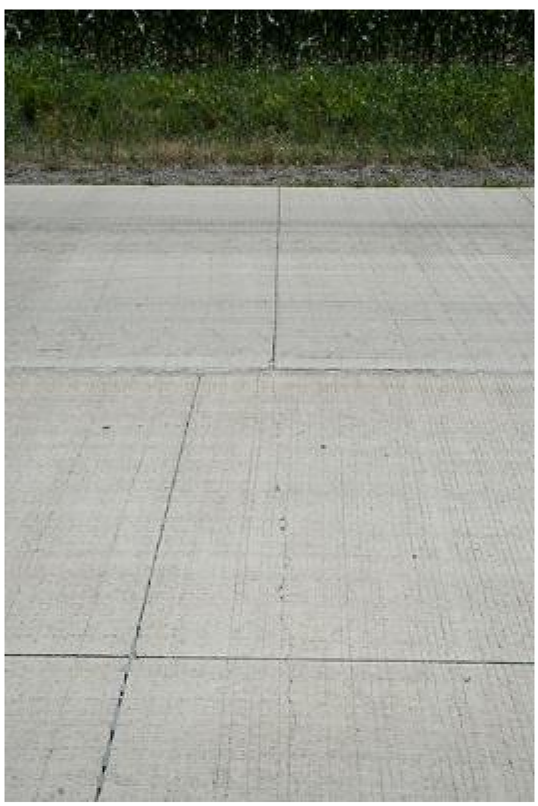

Figure 2.58. Transverse joint misalignment and cracking (June 2012).

The 11 foot panel section had problems as well. First, there was a loss of support on the edge of the eastbound lane, resulting in corner cracking and fracturing of the edge of the pavement. The eastbound lane appeared to have been constructed first and the westbound lane second, so this problem may have resulted from trucks driving on the edge of the pavement during the second stage of construction. Additionally, there was some low-severity spalling in the longitudinal and transverse joints. Finally, there was up to 2 mm of faulting in this section, resulting in a rough ride. The cause of the faulting was unclear, however. Sounding tests did not indicate debonding at the faulted sections.

2.2.16 Richland County: County Highway 9

County Highway 9 was completed in 2010 in rural Richland County. One unique feature of this project was a cross slope greater than 1.5%. Owing to the taper in the cross-section, in some areas the overlay was placed directly over cement-stabilized soil. Project details are provided in Table 2.34. An overview of County Highway 9 and the project location are shown in Figures 2.59 and 2.60.

Table 2.34. County Highway 9 Project Details

Completion Date	2009[a]
Overlay Thickness	5.5 inches[a]
Underlying Thickness/ Condition	Existing, milled HMA of unknown thickness over cement-stabilized soil[a]
Slab Size	5.5 by 5.5 feet
ADT	550[a]
Fiber Reinforcement	4 lb/yd^3 synthetic[a]

[a]Source: Riley (2010).

Figure 2.59. Overview of Richland County Highway 9 (June 2012).

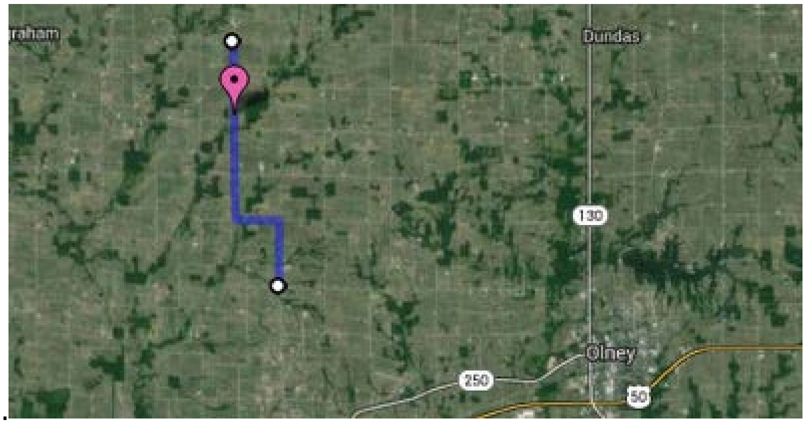

Figure 2.60. Project location of Richland County Highway 9.

Overall, the project was in excellent condition. There was no cracking in the slabs, as shown in Table 2.35. There were a few instances of joint spalling, which was likely due to saw-cutting. There was no fiber balling evident at the surface similar to what was observed in other pavements that featured macro-fibers.

Table 2.35. County Highway 9 Distress Survey

	Total	%
Total # Slabs	840	—
# Slabs Corner Breaks	0	0.0
# Slabs Longitudinal Cracks	0	0.0
# Slabs Transverse Cracks	0	0.0
# Slabs Diagonal Cracks	0	0.0
# Shattered Slabs	0	0.0
# Slabs Patched	0	0.0
Total # Slabs Cracked	0	0.0

2.2.17 Lombard: North Industrial Park

In 2010, a UTW inlay was placed on 0.5 mile to 1 mile stretches of four roads serving an industrial park in Lombard. Project details are provided in Table 2.36. Figure 2.61 pictures DuPage Avenue as a typical example of the four roadways in the project. All four roads are highlighted in Figure 2.62.

Table 2.36. North Industrial Park Project Details

Completion Date	2010[a]
Inlay Thickness	4 inches[a]
Underlying Thickness/ Condition	3 inches of milled HMA[a]
Slab Size	4 by 4 feet
ADT	3,100[a]
Fiber Reinforcement	4 lb/yd^3 synthetic[a]

[a]Source: Riley (2010).

Figure 2.61. DuPage Avenue in the North Industrial Park project (August 2012).

Figure 2.62. Location of the North Industrial Park in Lombard.

No crack survey of the North Industrial Park project was performed because the project was relatively new and there were few instances of distress. Most of the distresses that were observed appeared to have resulted from construction, which was done on a cold day in November with highs around 40°F (Riley 2010).

The most common distress was joint spalling resulting from too-early saw-cutting, as demonstrated in Figure 2.63. There were also instances of hairline corner cracking on slabs at the edge of the pavement where transverse joints were not finished all the way to the curb. Out of the four streets in the project, DuPage Avenue appeared to have the fewest surface distresses, while Lombard Avenue had the most. All of these distresses were construction issues that were not related to traffic. The macro-fibers appeared to be keeping the joints tight, there was no faulting, and the ride was very smooth. The macro-fibers were clearly visible on the surface of the pavement, and there were some instances of fiber balling.

Figure 2.63. Joint spalling on DuPage Avenue (August 2012).

2.2.18 Clay County: Xenia-Iola Road

Xenia-Iola Road in rural Clay County was completed in 2010. Though the panels were square (roughly 5.5 by 5.5 feet), there were small variations in the longitudinal joint spacing. The northbound lane measured 10 feet, 11 inches wide, while the southbound lane was 11 feet, 2 inches wide. The project featured 5 foot aggregate shoulders. Project details are provided in Table 2.37. An overview of Xenia-Iola Road and its location are found in Figures 2.64 and 2.65.

Table 2.37. Xenia-Iola Road Project Details

Completion Date	2010[a]
Inlay Thickness	5 inches[a]
Underlying Thickness/ Condition	Existing HMA of unknown thickness[a]
Slab Size	Roughly 5.5 by 5.5 feet
ADT	700[a]
Fiber Reinforcement	4 lb/yd^3 synthetic[a]

[a]Source: Riley (2010).

Figure 2.64. Overview of Xenia-Iola Road (June 2012).

Figure 2.65. Project location of Xenia-Iola Road in Clay County.

Overall, the project was in excellent condition. There were few cracked panels, as noted in Table 2.38. The only major observed distresses were surface cracks that may have resulted from shrinkage or restraint. There was also slight deterioration of the edge of the northbound lane.

Table 2.38. Xenia-Iola Road Distress Survey

	Total	%
Total # Slabs	480	—
# Slabs Corner Breaks	0	0
# Slabs Longitudinal Cracks	2	0.4
# Slabs Transverse Cracks	0	0
# Slabs Diagonal Cracks	0	0
# Shattered Slabs	0	0
# Slabs Patched	0	0
Total # Slabs Cracked	2	0.4

2.3 DATA ANALYSIS OF DISTRESS SURVEYS

On the basis of on these visual distress surveys, a number of observations were made about certain design choices and features and how they potentially related to UTW performance.

First, it appeared that 5.5 to 6 foot panel sizes were ideal for UTW roadway projects both with and without structural fibers. Where the panels were smaller, such as in the Decatur or Tuscola projects (both 4 feet), the joints were left in one of the wheel paths, contributing greatly to joint deterioration and subsequent cracking and faulting. Where the panels were larger than 6 feet, such as Marion Street in Oak Park (6 feet, 10 inches), longitudinal cracking developed down the middle of many slabs. On Bible Grove Road and Piatt County Highway 4, there were two distinct sections that featured 5.5 foot and 11 foot joints. The sections with the 11 foot panels exhibited faulting, while faulting was minimal in the 5.5 foot sections. For parking lots with mostly passenger car traffic, 4 foot joints appeared to be fine.

Skewed joints were occasionally used on past UTW pavements, but they did not appear to provide a tangible benefit and may have hurt performance. On Sailor Springs Road and Piatt County Highway 4, the combination of skewed joints and too-large panel sizes appeared to lead to very noticeable faulting at the edges of the pavement near the acute joint intersections. In Piatt County, increased stresses at the acute joint intersections may also have been at least partially responsible for the extensive longitudinal cracking near the edges of the pavement.

With less than 10 years of experience constructing UTW pavements with synthetic macro-fibers, the performance indicators suggest an improvement over UTW with plain concrete, especially because the more recent designs have reduced thickness when incorporating fibers. Macro-fibers also provide post-cracking benefits because they keep joints tight and greatly reduce the risk of faulting. Where cracking had already occurred, fibers helped keep the cracks tight, maintaining serviceability of the pavement. For example, although Marion Street and the Western Avenue bus pads featured a relatively high number of cracked panels, the joints and cracks themselves stayed tight, keeping the pavement smooth. More severe stresses, such as faulting or shattered slabs, did not develop to anywhere near the same extent that they did in Tuscola or Decatur. Projects without macro-fibers were also highly susceptible to slab migration, a distress that eventually led directly to blow-ups in Decatur.

UTW pavements performed well with the addition of macro-fibers, but the fibers' effectiveness was limited when large displacements occurred in the pavement cross-section. For example, the combination of settlement of the embankment and heavy truck traffic caused joint opening and faulting in places on Schank Avenue in Mundelein despite the use of macro-fibers on the project.

Finally, from a construction standpoint, cutting the joints with a thinner saw blade appeared to be beneficial in extending UTW performance. Wide joints were a major contributing factor to distresses in several projects, especially on U.S. 36 in Tuscola, where severe spalling, corner cracking, and faulting resulted from excessive joint opening. Cutting the joints with a thinner saw blade could have helped keep them together longer, which would have been especially helpful given that fibers were not used in the project.

CHAPTER 3 FALLING WEIGHT DEFLECTOMETER TESTING

Falling weight deflectometer (FWD) testing was performed in October through December 2012 and in May 2013 across seven of the previously surveyed UTW pavements: the Talbot, McKinley, and E-15 parking lots at the University of Illinois; Bible Grove Road, Richland County Highway 9, North Lorang Road, and Schank Avenue. Also included in this chapter are FWD test results collected in fall 2008 for the three University of Illinois parking lots.

The main goal of the FWD testing on these projects was to evaluate their in situ structural performance. UTW layer thicknesses and support conditions can vary greatly as a function of distance along a project and may not be accurately known in the first place, making it difficult to characterize the load carrying capacity of a UTW pavement at a given point. Deflection data resulting from FWD tests were used to determine joint load transfer efficiency and to backcalculate structural properties with a procedure that was developed specifically for UTW pavements.

3.1 TEST PLANS

3.1.1 General Test Procedure

All of the FWD tests were conducted under the same general procedure outlined in this section. Variations in testing are covered in the subsections dedicated to each individual project.

At each project site, testing was done in a series of consecutive slabs, in a number of single slabs at set distances apart, or both. Each slab was tested in four or five locations, as seen in the drop pattern figures provided for the projects. The drops at test locations 1 and 2 (or just 1, depending on the project) were used to evaluate joint performance. Drops at location 3 (or 2) were used to assess center slab deflection and backcalculate the effective thickness and k-value of the subgrade. Data collected at test locations 4 and 5 (or 3 and 4) were gathered to be considered in future studies on slab curling and corner support.

At each test location, target loads of 6, 9, and 12 kips were applied and the resulting slab deflections were measured with velocity transducers offset from the loading plate. One transducer was located directly below the loading plate, with the remaining transducers positioned radially from the loading plate. As seen in Figure 3.1, the transducer spacings were: d_0 = 0 inches, d_1 = 12 inches, d_2 = 24 inches, d_3 = 36 inches, d_4 = 12 inches forward, d_5 = 12 inches to the right, and d_6 = 12 inches to the left.

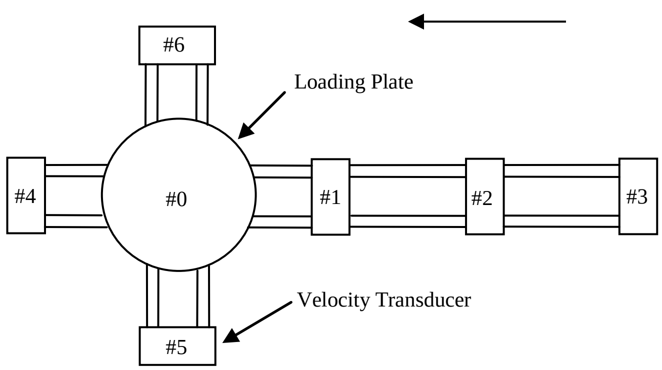

Figure 3.1. Location of transducers around FWD plate.

3.1.2 University of Illinois Parking Lots

There are three UTW parking lots on the campus of the University of Illinois: the Talbot Lab parking lot, the McKinley Health Center parking lot, and the E-15 parking lot. All three lots were tested in October 2012 using the same drop pattern, as shown in Figure 3.2.

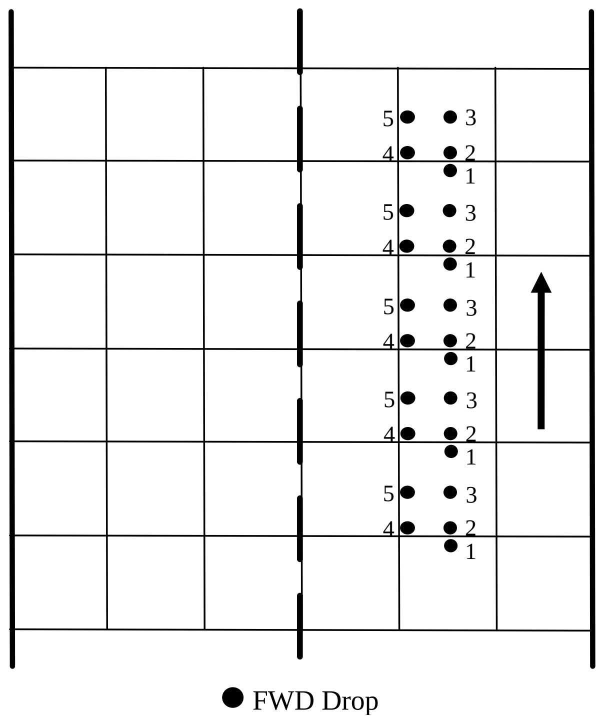

Figure 3.2. FWD drop pattern, University of Illinois parking lots.

3.1.2.1 E-15 Lot

The E-15 lot was previously tested in 2006 and 2008. Originally, only half of the lot was overlaid in 2006. The other half was completed in August 2012, creating an excellent opportunity to compare current results from the old part of the lot with those of the brand new one. Three passes of 15 consecutive slabs were done over the old part of the lot (as in 2008) and one pass of 15 was completed over the new section of the lot, as seen in Figure 3.3.

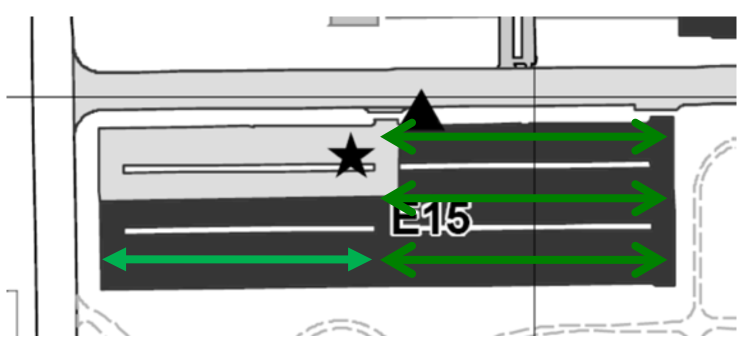

Figure 3.3. Test plan, E-15 parking lot.

3.1.2.2 McKinley Health Center Parking Lot

The McKinley lot was previously tested in 2008. The survey in June 2012 (Section 2.2.14) indicated some debonding across the lot, especially in low areas near drains. FWD testing near those areas may be able to indicate debonding as well. The test plan, seen in Figure 3.4, was identical to the 2008 test: performing three passes of 15 consecutive slabs over distinct sections of the lot labeled Northwest (NW), Northeast (NE), and Southeast (SE).

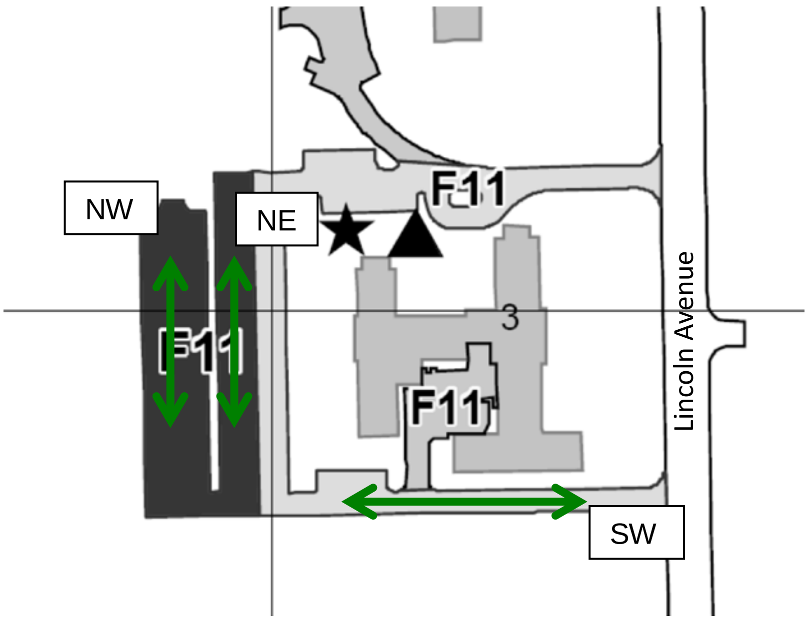

Figure 3.4. Test plan, McKinley parking lot.

3.1.2.3 Talbot Lab Parking Lot

The Talbot lot was previously tested in 2008. It was the first UTW parking lot at the U of I and as of fall 2012 was 14 years old, making it valuable as one of the oldest UTW projects available for testing. Another goal of testing was to potentially provide insight about the future behavior of the E-15 and McKinley lots. Just one pass of 15 consecutive slabs over the lot was performed, as seen in Figure 3.5.

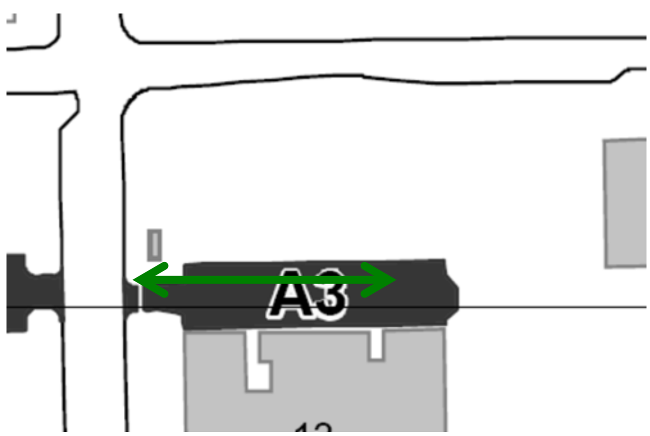

Figure 3.5. Test plan, Talbot Lab parking lot.

3.1.3 Downstate Projects

3.1.3.1 Bible Grove Road, Clay County

As noted in the June 2012 survey (Chapter 2), Bible Grove Road had already begun to exhibit some faulting despite being constructed in 2008. The faulting was limited to the 11 foot panel section. The cause of the faulting was unclear at the time of the survey; sounding tests did not seem to indicate debonding. The FWD testing could provide insight about the mechanism of the faulting.

FWD testing was performed in December 2012 on the 5.5 foot and 11 foot panel sections according to the proposed drop patterns shown in Figures 3.6 and 3.7, respectively. Intensive testing was carried out in a single pass over 15 consecutive slabs, and periodic testing was performed over 25 slabs at 100 foot intervals.

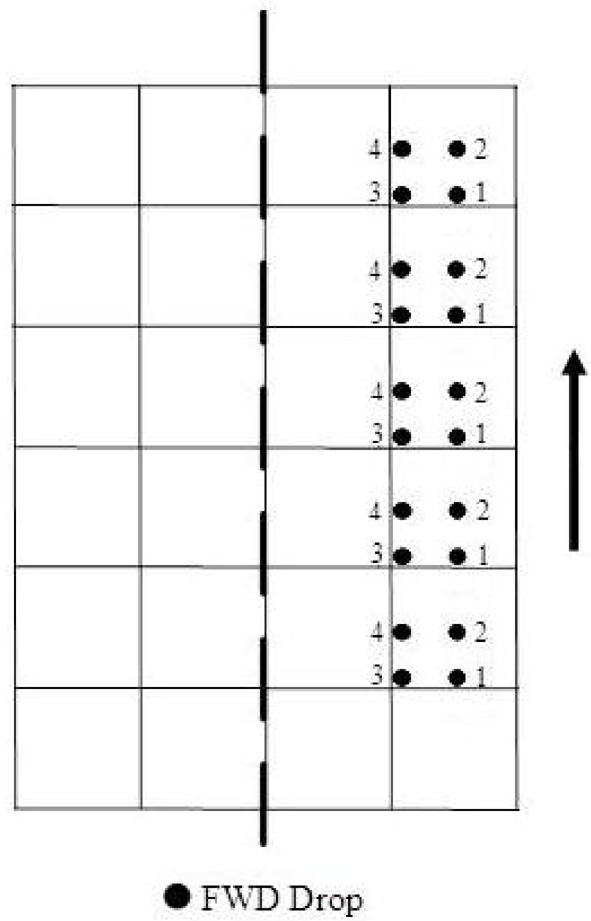

Figure 3.6. FWD drop pattern, 5.5 foot panel section,
Bible Grove Road and Richland County Highway 9.

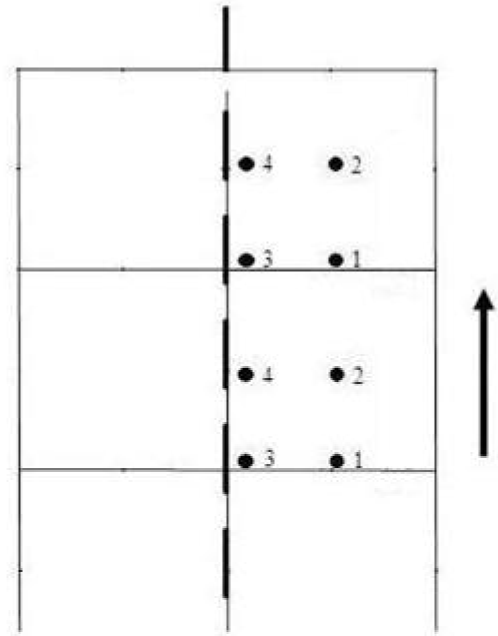

Figure 3.7. FWD drop pattern, 11 foot
panel section, Bible Grove Road.

3.1.3.2 Highway County 9, Richland County

Richland County Highway 9 was one of the newest projects (Section 2.2.17). It was completed in 2010 and used macro-synthetic fibers. It was a good project to test to compare with Bible Grove Road because of their closeness in age and because County Highway 9 used macro-fibers while Bible Grove Road did not. County Highway 9 does not have an 11 foot panel section.

FWD testing was performed in November 2012 according to the proposed drop pattern in Figure 3.6, the same as the 5.5 foot section on Bible Grove Road. Intensive testing was carried out in a single pass over 15 consecutive slabs, and periodic testing was performed over 25 slabs at 100 foot intervals.

3.1.4 Chicago-Area Projects

3.1.4.1 North Lorang Road, Kane County

Completed in 2004, North Lorang Road was one of the oldest UTW projects in the state to use synthetic macro-fibers, making it a valuable test site. As of the August 2012 survey (Section 2.2.12), it was still in very good shape except for some distresses found on the south end of the project.

FWD testing was performed in May 2013 according to the drop pattern in Figure 3.8. Intensive testing was performed in three passes of 15 consecutive slabs over different parts of the project. The project areas that were tested included the south end adjacent to the residential homes, in the middle of the project, and the north end in front of the quarry entrance.

3.1.4.2 Schank Avenue, Mundelein

Schank Avenue was previously tested in 2006 and was a good project to test and compare with North Lorang Road because it was completed one year after North Lorang Road, featured a similar panel size

(4 feet) and mix design (macro-fibers), and experienced significant truck traffic. At the time of the August 2012 surveys, both the truck traffic and the passenger car traffic on Schank Avenue were higher than on North Lorang Road. There were many more distresses, such as longitudinal joint opening, faulting, and embankment/pavement rotation. These problems were especially apparent in the middle section of the project, which was on an embankment that appeared to be settling. The main goal of FWD testing was to provide insight about whether the settlement and ensuing loss of support were the primary cause of the distresses or whether debonding may have been involved as well.

FWD testing was done in May 2013 according to the drop pattern in Figure 3.8, the same as on North Lorang Road, for 25 consecutive slabs in each direction near the middle of the roadway section, away from the intersections at either end.

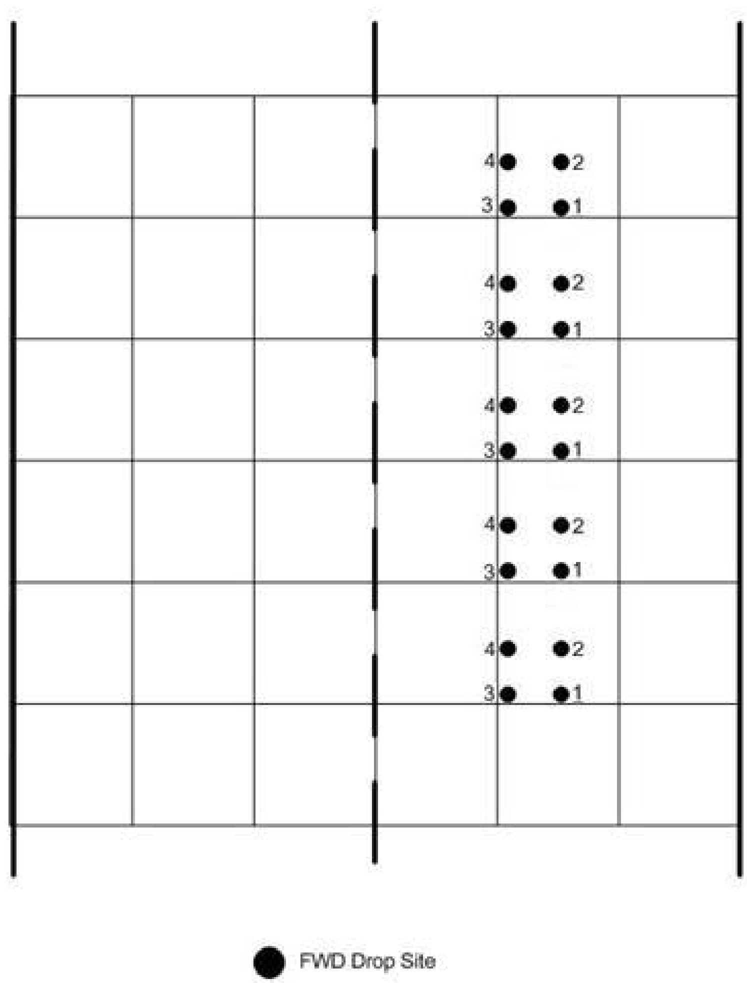

Figure 3.8. FWD drop pattern, North Lorang Road and Schank Avenue.

3.2 TEST RESULTS

Before the results of the FWD testing can be analyzed, the effect of temperature must be considered. Concrete surface deflections are affected by the asphalt concrete layer, which can vary with pavement temperature. However, without accurate measurement of the asphalt concrete thickness, it is not possible to normalize the deflections or the calculated values for joint LTE for temperature. The initial temperature conditions at the commencement of each FWD loading are listed in Table 3.1 for reference.

Though the University of Illinois parking lots were tested in both 2008 and 2012, the exact testing locations from 2008 are unknown. The tests were performed in the same general areas of each parking lot and allow for a good general comparison between 2008 and 2012, but the tests were not necessarily conducted on the same slabs.

Table 3.1. FWD Testing Conditions

Project/Section		Date/Time	Average Air Temp (°F)	Average Pavement Surface Temp (°F)	Average Pavement Temp at 2 in Depth (°F)
McKinley Lot	NE	10-17-08 11:00	57	63	60
		10-19-12 9:34	45	52	50
	NW	10-17-08 9:55	54	63	60
		10-16-12 11:48	69	67	67
	SE	10-17-08 12:48	57	58	58
		10-19-12 10:47	46	49	50
E-15 Lot	1	10-16-08 9:36	53	68	70
		10-22-12 9:36	63	60	65
	2	10-16-08 10:51	55	63	70
		10-22-12 10:46	59	51	64
	3	10-16-08 12:36	58	58	70
		10-24-12 9:24	63	58	64
	4	10-24-12 10:31	67	62	67
Talbot Lot		10-20-08 9:33	57	59	58
		10-16-12 9:43	61	69	64
Bible Grove Road		12-4-12 10:15	54	54	59
Richland County Hwy 9		11-28-12 10:23	44	48	47
North Lorang Road		5-16-13 8:49	72	82	80
Schank Avenue		5-15-13 11:20	80	108	94

3.2.1 Determination of Joint Load Transfer Efficiency

Joint load transfer efficiency (LTE) was calculated at the longitudinal (LTE_x) and transverse (LTE_y) joints for each slab that was tested, according to Equation 3.1, where d_{UL} and d_L are the deflections in the unloaded and loaded slabs, respectively.

$$LTE = \frac{d_{UL}}{d_L} \tag{3.1}$$

The average values of longitudinal joint LTE (LTE_x) and transverse joint LTE (LTE_y) for each project at the time and temperature of testing are provided in Table 3.2. In addition, deflection and transverse joint LTE (LTE in the direction of traffic) calculated from the drops at each slab and target weight for all projects, were plotted as a function of distance. These plots are found in Appendix B and include a center slab LTE calculation, which represents the theoretical LTE of a concrete pavement with 100% functioning joints (i.e., shear and moment transfer). The center slab LTE can be compared with the joint LTE to determine whether a full-depth joint crack has occurred (Roesler et al. 2008). Example plots of local, transverse joint LTE for consecutive slabs are provided in Figures 3.9 and 3.10.

Table 3.2. Average Joint Load Transfer Efficiencies for Seven Illinois UTW Projects

Project/Section		Date	Transverse Joint Average LTE (%)	Longitudinal Joint Average LTE (%)
McKinley Lot	Northeast	2008	79.4	83.5
		2012	79.7	76.6
	Northwest	2008	86.9	91.6
		2012	85.7	83.6
	Southeast	2008	82.5	78.8
		2012	74.7	82.3
E-15 Lot	1	2008	78.3	88.0
		2012	80.7	91.0
	2	2008	82.9	88.2
		2012	78.0	90.0
	3	2008	83.8	88.9
		2012	81.6	90.2
	4	2012	87.7	83.3
Talbot Lot		2008	70.7	74.8
		2012	64.2	71.9
Bible Grove Road	5.5 ft Intensive	2012	91.5	77.2
	5.5 ft Periodic		87.5	n/a[1]
	11 ft Intensive		30.8	73.3
	11 ft Periodic		39.9	n/a[1]
Richland County Hwy 9	Intensive	2012	80.5	92.2
	Periodic		84.0	n/a[1]
North Lorang Road	1	2013	81.0	77.3
	2		91.8	80.7
	3		92.3	81.1
Schank Avenue	Northbound	2013	81.9	77.0
	Southbound		90.5	81.2

[1]Drops at the longitudinal joints were not performed, so LTE could not be calculated.

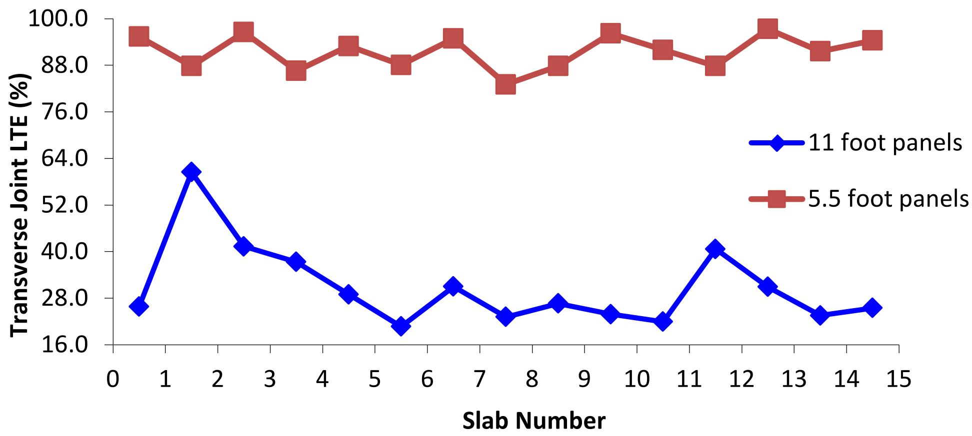

Figure 3.9. Variation in transverse joint LTE for consecutive slabs, Bible Grove Road.

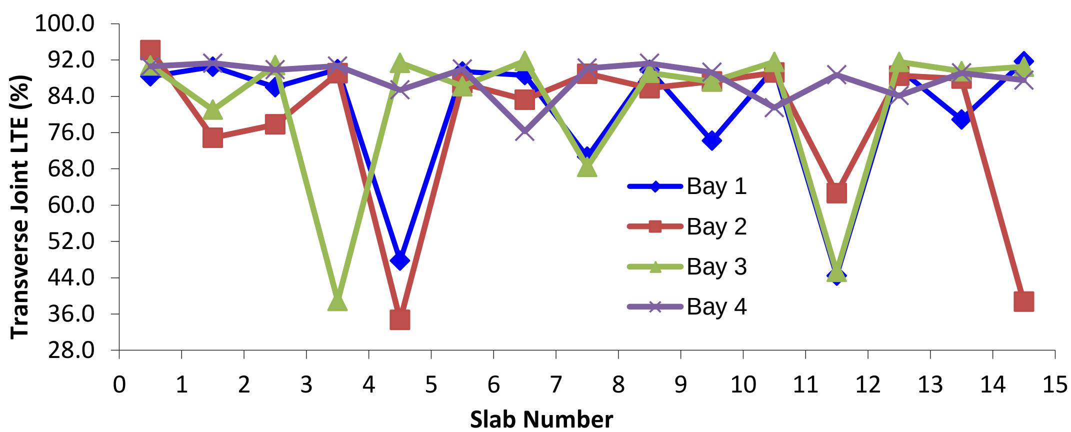

Figure 3.10. Variation in transverse joint LTE for consecutive slabs, E-15 lot 2012.

3.2.2 Backcalculation of UTW Structural Properties

For many years, existing flexible and rigid pavement support conditions and layer stiffnesses have been successfully assessed through falling weight deflectometer (FWD) testing and analyses of the deflection data. Several published FWD studies of UTW have looked at variation in deflection (Armaghani and Tu 1999; Cable et al. 2001; Vandenbossche 2004; Saeed and Hammons 2005) and joint load transfer efficiency (Roesler et al. 2008) along project sections. Layered elastic backcalculation has been used to determine layer modulus values of a UTW airport runway in Tennessee (Saeed and Hammons 2005) and state highway in Iowa (Cable and Hart 1998). Finally, the radius of relative stiffness was backcalculated using the AASHTO 1998 method for an accelerated test section of UTW with established values for both the thickness and modulus of the asphalt and concrete layers (Newbolds and Olek 2008). Because UTW have finite-sized slabs and variable joint load transfer efficiency (LTE), backcalculations using layered elastic theory or plate theory with infinite slab dimensions are not expected to give accurate results, especially over the range of expected support conditions, slab thicknesses, joint stiffness, and slab geometry.

Most UTW projects occur on lower-volume roads where the pavement layer thicknesses and support conditions may vary as a function of distance along the project. Furthermore, the existing asphalt pavement is often distressed before construction of the bonded concrete overlay and may have been cold milled, so the asphalt layer thickness and stiffness cannot be known accurately without extensive destructive testing. To backcalculate accurate foundation and layer stiffnesses for UTW pavement systems, the pavement layer thicknesses and interface conditions must be known, and the finite-sized slab condition with variable joint LTE must be accounted for as well.

3.2.2.1 Backcalculation of Concrete Slabs

Hoffman and Thompson (1981) first developed a method for transforming deflection data obtained from flexible pavement FWD testing into a deflection basin area term, known as AREA. This term, defined as $AREA_{36}$, is calculated from normalized surface deflections measured at 0 (d_0), 12 (d_{12}), 24 (d_{24}), and 36 (d_{36}) inch offsets from the center of the loaded plate as follows:

$$AREA_{36} = 6\left(1 + 2\frac{d_{12}}{d_0} + 2\frac{d_{24}}{d_0} + \frac{d_{36}}{d_0}\right) \tag{3.2}$$

For an infinite concrete slab (plate) on a Winkler foundation layer, Ioannides (1990) demonstrated that there was a unique relationship between $AREA_{36}$ and the radius of relative stiffness (ℓ) for a given radius of loaded area (a). The regression equation for the relationship, presented by Hall (1991), is shown in Equation 3.3:

$$\ell = \left[\frac{ln\left(\frac{36 - AREA_{36}}{1812.279133}\right)}{-2.559340}\right]^{4.387009} \tag{3.3}$$

With this unique relationship between ℓ for a slab system and $AREA_{36}$, Barenberg and Ioannides (1989) outlined a procedure to backcalculate the dimensionless Westergaard's (1926) maximum interior deflection (W_{int}) from the known a/ℓ ratio using Equation 3.4. After W_{int} is calculated from Equation 3.4, the modulus of subgrade reaction (k) can be calculated by re-arranging Equation 3.5:

$$W_{int} = \frac{1}{8}\left[1 + \left(\frac{1}{2\pi}\right)\left(\ln\left(\frac{1.7810725a}{2\ell}\right) - \frac{5}{4}\right)\left(\frac{a}{\ell}\right)^2\right] \tag{3.4}$$

where

$$W_{int} = \frac{k d_0 \ell^2}{P} \tag{3.5}$$

Finally, the thickness (h) or elastic modulus (E) of the slab can be directly determined using Equation 3.6 by assuming E or h. Normally, the elastic modulus of concrete is determined by assuming that the concrete thickness is known from the design or construction quality control/assurance tests.

$$\ell = \left[\frac{Eh^3}{12(1-\nu^2)k}\right]^{1/4} \quad (3.6)$$

Over the years, researchers have modified the slab on grade backcalculation procedure to account for more offset deflection sensors up to 60 inches (Hall et al. 1997; Khazanovich et al. 2001), finite-sized slabs and varying joint load transfer (Crovetti 1994), interface bond conditions and layer equivalencies (Khazanovich et al. 2001), and even built-in temperature curling (Vandenbossche 2003; Rao and Roesler 2005; Lederle et al. 2011).

3.2.2.2 Backcalculation Challenges for UTW Pavement Systems

The closed-form backcalculation process for rigid pavements described above is well-established and has successfully been used for assessing the structural response and layer stiffness of conventional rigid pavements. Unfortunately, varying slab geometry, varying joint LTE, and interface bond condition of UTW pavement systems limit the accuracy and usefulness of the existing backcalculation procedures. Equation 3.3 assumes a center-loaded slab condition without influence of the edges or joints on the deflection basin, which is violated for UTW pavement systems that have small values for the ratio of slab length to radius of relative stiffness (L/ℓ). Crovetti (1994) first developed correction factors for finite-sized slabs in ILLI-SLAB for a range of L/ℓ values. However, the L/ℓ values for many UTW pavement systems fall outside the dataset considered by Crovetti, which ranged from roughly 2 to 12. Additionally, UTW pavements have slab dimensions shorter than 6 feet and thus it is not possible to calculate $AREA_{36}$ accurately for slabs because the offset sensor 36 inches away from the load plate will be on the adjacent slab.

The main objective of the FWD testing is to characterize the in situ structural capacity of the UTW pavement. Several assumptions must be made in order to control the number of unknown variables in this multi-layered, finite-sized slab system, which is accomplished through backcalculating the effective modulus of subgrade reaction (k) and an effective concrete thickness (h_{eff}) by assuming the elastic modulus (E) and Poisson's ratio (v). Backcalculating k and h_{eff} allows for an evaluation of the roadway's capacity to handle traffic, even without detailed knowledge of individual layer stiffnesses and thicknesses and interface conditions. Effective thickness can also be plotted as a function of distance along the roadway to determine how the effective pavement structure varies along a UTW section.

Backcalculating h_{eff} does provide an indirect method to evaluate the condition of the bond interface between the concrete and asphalt layers. If h_{eff} for a UTW pavement is the same as or close to the best-known thickness of the concrete layer, it suggests that the interface bond has deteriorated and the concrete layer is primarily providing the structural support. If h_{eff} exceeds the best-known concrete thickness, it suggests that there is at least partial bond between the concrete and asphalt layer. The backcalculated k-value for UTW is not the soil stiffness, but a composite stiffness of the unbound layers below the asphalt concrete layer.

3.2.2.3 UTW Pavement Modeling

To have a backcalculation method for the expected UTW design features, changes must be made to the existing backcalculation method to address the previously discussed limitations. These changes were accomplished by modeling the behavior of UTW pavements with the two-dimensional finite element program ILLI-SLAB (Khazanovich 1994), which is based on medium-thick plate theory over a Winkler foundation.

As illustrated in Figure 3.11, a UTW pavement system was modeled in ILLI-SLAB as nine square concrete slabs over subgrade with an element size ranging from 1.5 to 2 inches. The slab at the center of the grid was subject to the FWD plate load test. The fixed input parameters for the FWD test and the concrete pavement properties are listed in Table 3.3.

Table 3.3. Finite Element Analysis Fixed Input Parameters

Load, P	9,000 lb
Plate Radius, a	6.0 in
Modulus of Subgrade Reaction, k	100 psi/in
Poisson's Ratio, v	0.15
Modulus of Elasticity, E	5,000,000 psi

Multiple finite element runs were made by varying the slab thickness between 2 and 9 inches at half-inch increments for different combinations of slab length and load transfer efficiencies. The slab lengths considered were 4, 5, and 6 feet, which corresponded to L/ℓ ranging from 1.1 to 5.3. Load transfer efficiencies were 100%, 80%, 50%, and 0% in the longitudinal (LTE_x) and transverse (LTE_y) joints, with LTE_x and LTE_y kept the same rather than varied independently (e.g., $LTE_x = LTE_y$ for each input scenario). From each simulation, offset deflections were extracted from ILLI-SLAB to derive the new backcalculation equations for UTW pavement systems.

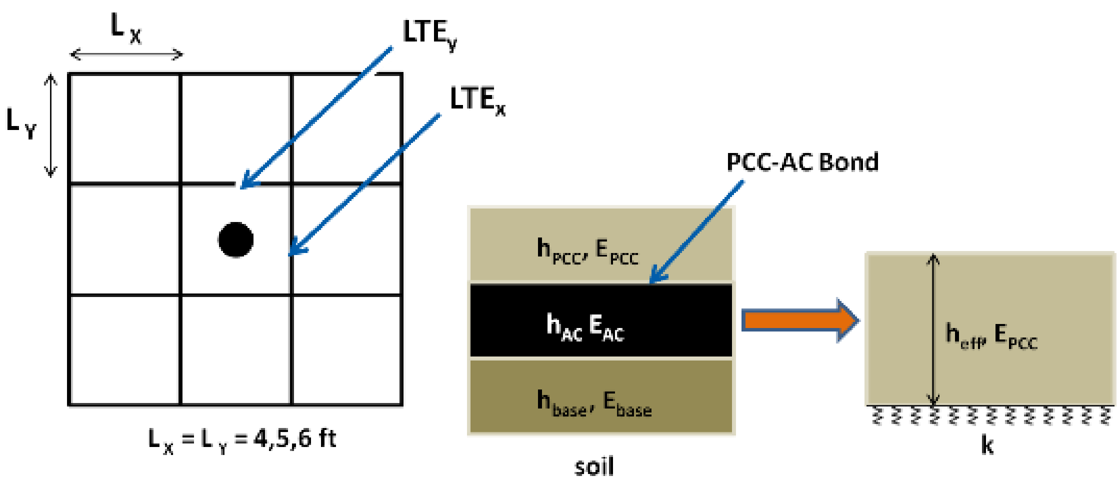

Figure 3.11. UTW pavement modeled with 2-D finite element analysis.

3.2.2.4 Relationship Between Radius of Relative Stiffness, ℓ, and $AREA_{24}$

To develop a backcalculation process for UTW, a new relationship between AREA and the radius of relative stiffness must be defined. From the ILLI-SLAB output data, a new deflection basin quantity for UTW pavement systems called $AREA_{24}$ is defined through Equation 3.7 to account for the smaller slab size (i.e., 4 to 6 foot panel sizes). $AREA_{24}$ is calculated from deflection values at 0 (d_0), 12 (d_{12}), and 24 (d_{24}) inch offsets from the load plate. Although one fewer deflection is used in the $AREA_{24}$ calculation relative to $AREA_{36}$, there is a unique relationship between ℓ and AREA as long as at least two offset deflection values are used (Ioannides 1990).

$$AREA_{24} = 6\left(1 + 2\frac{d_{12}}{d_0} + \frac{d_{24}}{d_0}\right) \tag{3.7}$$

Figures 3.12 through 3.14 provide the variation of $AREA_{24}$ with radius of relative stiffness for each panel size and several joint LTE. A mathematical relationship between ℓ vs. $AREA_{24}$ was produced using an exponential function (Hall 1991). Table 3.4 lists the functional form, the values of the regression coefficients for each given slab length and LTE, and a statistical analysis of the functions.

Table 3.4. Regression Equations for Radius of Relative Stiffness Versus $AREA_{24}$ for Three Slab Sizes

Equation Form: $\ell = \left(\frac{1}{C}\right)\ln\left[\frac{(AREA_{24} - A)}{B}\right]$ (in)				
	4 x 4 ft Slabs			
	LTE = 100	LTE = 80	LTE = 50	LTE = 0
A	24.31087	24.17607	24.018	23.909
B	–26.00771	–26.59701	–30.54813	–56.09284
C	–0.08457	–0.08888	–0.10403	–0.16102
Adj. R^2	0.999	0.99934	0.99945	0.99984
	5 x 5 ft Slabs			
	LTE = 100	LTE = 80	LTE = 50	LTE = 0
A	24.60319	24.39142	24.23217	24.04759
B	–18.45349	–19.07068	–21.29594	–34.69015
C	–0.061	–0.06597	–0.07671	–0.11602
Adj. R^2	0.99977	0.99983	0.99965	0.99882
	6 x 6 ft Slabs			
	LTE = 100	LTE = 80	LTE = 50	LTE = 0
A	24.21624	24.09991	24.26183	24.27216
B	–15.84026	–16.41619	–17.40423	–24.22541
C	–0.05707	–0.06101	–0.06409	–0.08703
Adj. R^2	0.99946	0.99962	0.99866	0.99881

3.2.2.5 Relationship Between a/ℓ and Non-Dimensional Deflection, W_{int}

Once ℓ is known and adjusted for a finite slab length, a/ℓ is used to solve for Westergaard's non-dimensional interior deflection (W_{int}). However, for UTW pavement systems, the relationship between a/ℓ and W_{int} for UTW slab geometries must be re-derived using ILLI-SLAB. Equation 3.8 was used to calculate W_{int} given ℓ, k, d_0, and load (P). With the load plate radius (a) of 6 inches, the relationships between a/ℓ and W_{int} were plotted as shown in Figures 3.15 through 3.17 for each slab size and multiple joint LTE. The data were fitted with an exponential function to produce regression coefficients to predict W_{int} from a/ℓ as shown in Table 3.5.

$$W_{int} = \frac{d_0 k l^2}{P} = f\left(\frac{a}{\ell}\right) \tag{3.8}$$

Table 3.5. Regression Equations for Non-Dimensional Deflection (W_{int}) Versus a/ℓ for Three Slab Sizes

Equation Form: $W_{int} = A + B\exp\left[C\left(\frac{a}{\ell}\right)\right]$				
	4 x 4 ft Slabs			
	LTE = 100	LTE = 80	LTE = 50	LTE = 0
A	120.87846	126.72553	133.15737	143.54028
B	3427.0594	2782.2579	2504.9307	5295.6693
C	–21.73887	–19.49571	–16.63839	–14.99493
Adj. R^2	0.99969	0.99997	0.99975	0.99934
	5 x 5 ft Slabs			
	LTE = 100	LTE = 80	LTE = 50	LTE = 0
A	124.23289	128.38665	132.29779	134.03373
B	5389.1597	2728.2911	1996.9598	3953.5161
C	–30.46372	–24.3336	–19.24581	–16.66974
Adj. R^2	0.99754	0.99964	0.99921	0.99968
	6 x 6 ft Slabs			
	LTE = 100	LTE = 80	LTE = 50	LTE = 0
A	126.03401	129.04035	130.97818	131.30063
B	19132.207	2405.1605	1255.4924	3210.6486
C	–45.37175	–28.39154	–19.90633	–18.57444
Adj. R^2	0.98295	0.98821	0.99189	0.99954

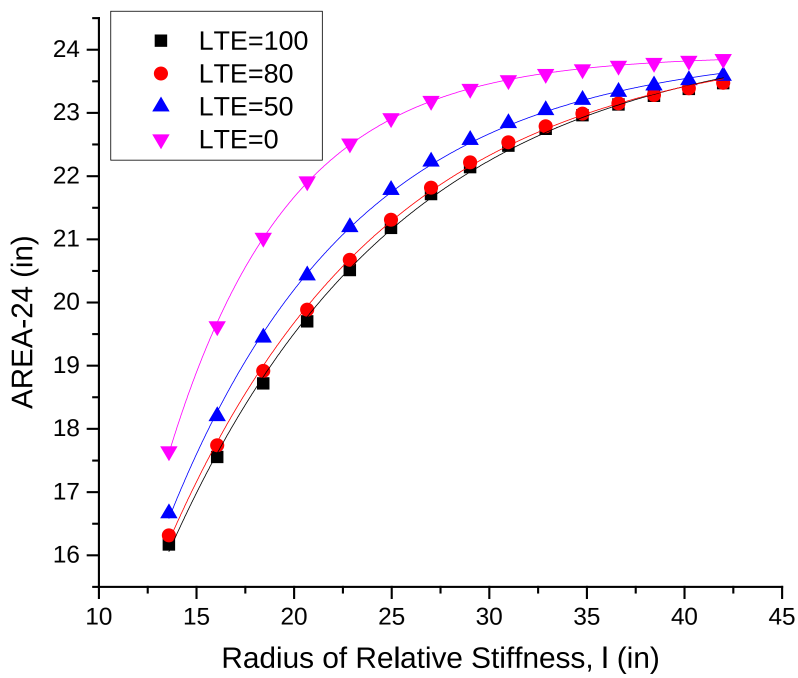

Figure 3.12. Radius of relative stiffness versus $AREA_{24}$ for several joint LTE and 4 by 4 foot slabs.

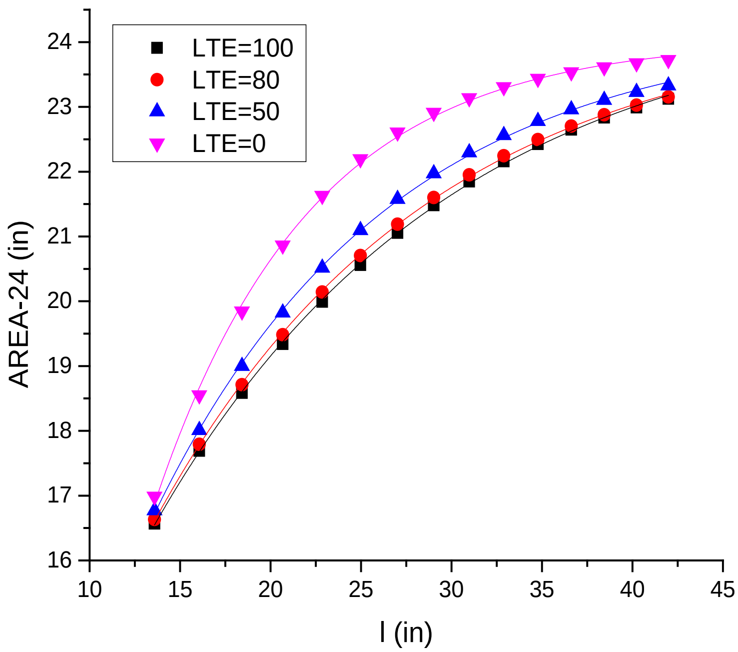

Figure 3.13. Radius of relative stiffness versus $AREA_{24}$ for several joint LTE and 5 by 5 foot slabs.

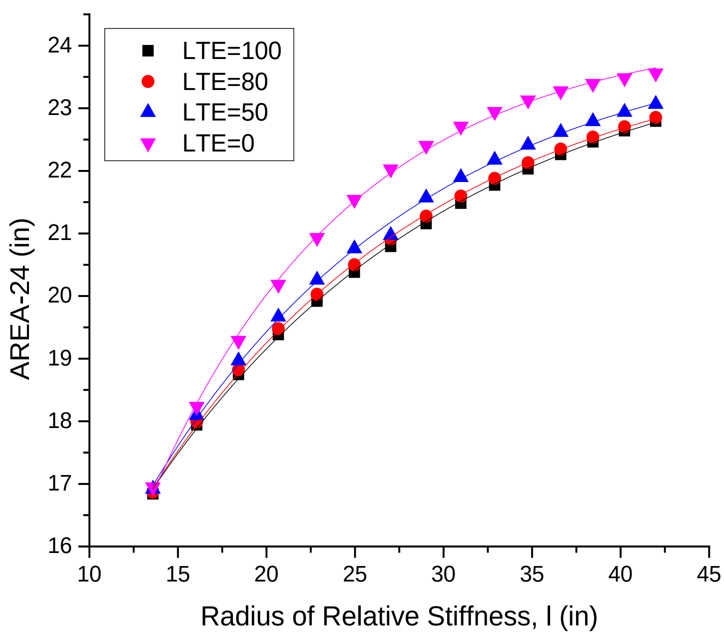

Figure 3.14. Radius of relative stiffness versus AREA$_{24}$ for several joint LTE and 6 by 6 foot slabs.

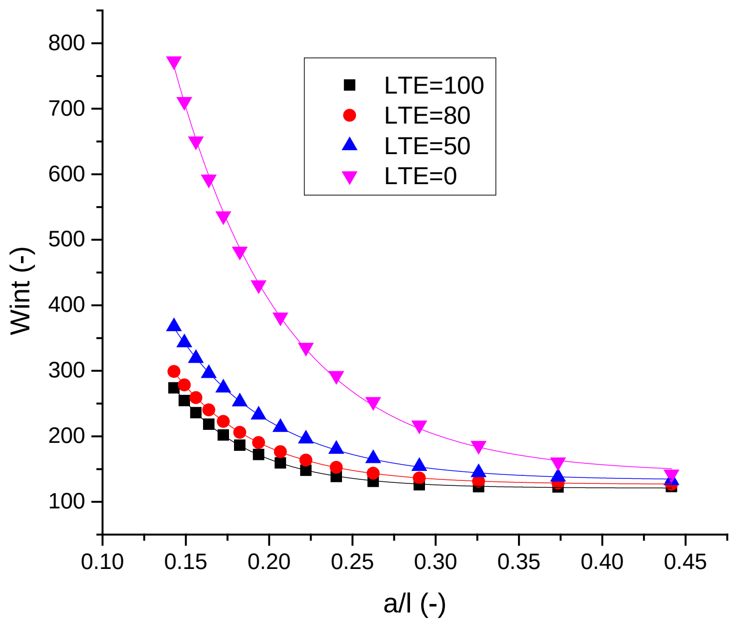

Figure 3.15. Non-dimensional deflection (W_{int}) versus a/ℓ for several joint LTE and 4 by 4 foot slabs.

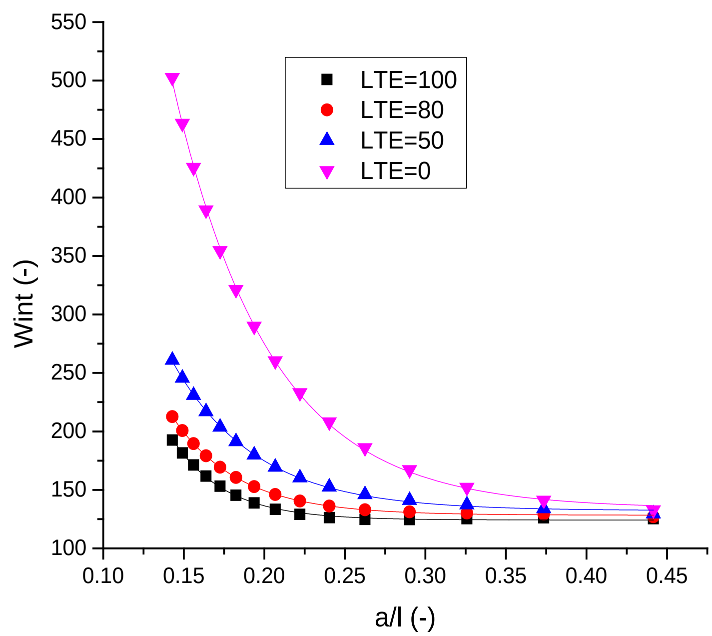

Figure 3.16. Non-dimensional deflection (W_{int})
versus a/ℓ for several joint LTE and 5 by 5 foot slabs.

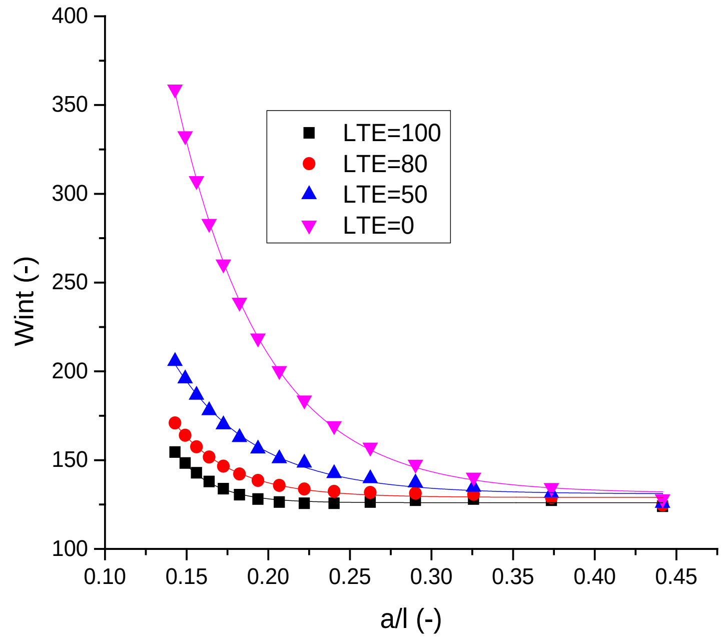

Figure 3.17. Non-dimensional deflection (W_{int}) versus
a/ℓ for several joint LTE and 6 foot by 6 foot slabs.

3.2.2.6 Application of the UTW Backcalculation Procedure

On the basis of the new regression equations derived from finite element analysis of equivalent UTW pavement structures, the following steps outline the process to backcalculate the effective slab thickness (h_{eff}) and modulus of subgrade reaction (k) from field-collected FWD data normalized to 9,000 pounds.

1. Using normalized deflections d_0, d_{12}, and d_{24}, calculate $AREA_{24}$ from Equation 3.7.
2. With the calculated $AREA_{24}$, solve for ℓ using the equation in Table 3.4 with the coefficients that most closely match the slab size and average joint LTE. Linear interpolation between two equations is acceptable, and further modeling can always be performed to determine the exact coefficients for any combination of slab size and LTE. The average joint LTE value is determined from FWD drops at various longitudinal and transverse joints in the UTW pavement of interest. An average value for LTE should be determined for each project on the basis of field measurements of the transverse and longitudinal joint LTEs. This average joint LTE is then used in steps 2 and 4 to minimize error in the backcalculated properties.
3. Divide the radius of the loading plate (a) by ℓ to determine a/ℓ.

4. With the calculated a/ℓ value, solve for non-dimensional deflection (W_{int}) using the equation in Table 3.5 with the coefficients that most closely match the slab size and average load transfer efficiency.
5. With values for ℓ and W_{int} calculated in steps 2 and 4—9,000 pounds for P and normalized d_0 from the FWD test, respectively—use Equation 3.5 to backcalculate the effective modulus of subgrade reaction (k).
6. Finally, using the same calculated values for ℓ and k, and assuming a best estimate of the concrete modulus of elasticity (E) and Poisson's ratio (v), use Equation 3.6 to determine the effective slab thickness (h_{eff}).

3.2.2.7 Application of Backcalculation Procedure to Illinois UTW Sections

The new backcalculation procedure was applied to FWD test data from the projects that were tested. Average values for LTE_x and LTE_y calculated from all of the FWD drops for a given project using Equation 3.1 were used to select the correct coefficients for the UTW backcalculation procedure. Coefficients for 80% LTE were chosen for all projects. Backcalculated values for k and h_{eff} for each center slab drop were determined by assuming the concrete had an elastic modulus of 5×10^6 psi and a Poisson's ratio of 0.15, which are typical values for concrete in Illinois with dolomite/limestone aggregates.

Table 3.6 lists the average backcalculated k and h_{eff} for the evaluated projects along with the standard deviation of h_{eff}. Estimated concrete and asphalt layer thicknesses at the time of construction for each project, when available, are provided in Table 3.6. As shown in Figures 3.18 through 3.30, h_{eff} was plotted as a function of distance for each project to capture how the underlying pavement structure varied on consecutive slabs or at periodic intervals (e.g., every 100 feet) along the section. The plots of h_{eff} in Figures 3.20, 3.27, and 3.28 are accompanied by joint load transfer efficiency as well.

As mentioned previously, the concrete surface deflections are affected by the behavior of the underlying asphalt layer, which can vary with pavement temperature, so it is not possible to normalize backcalculated h_{eff} values for temperature. Table 3.1 lists the initial temperature conditions at the commencement of each FWD loading as a reference.

The modeling of LTE in ILLI-SLAB for UTW systems may result in some error when the asphalt layer beneath the joint is continuous, allowing shear and moment transfer. However, any error in measuring joint LTE in the field should not be significant to the backcalculated effective thickness. Analysis of the sensitivity of backcalculated h_{eff} to the selection of a 50% or 100% LTE (rather than 80%) using the 2008 deflection data from the McKinley parking lot demonstrated less than a 2% difference in the final h_{eff} value, as shown in Table 3.7.

The assumed value for concrete elastic modulus (E) may not match the actual layer modulus of the concrete overlay that was tested, and the E-value assumption can affect the backcalculated value of effective thickness. The h_{eff} is not intended to represent the exact composite thickness of the pavement but to provide an estimated structural capacity of the bonded overlays of concrete asphalt in terms of an equivalent concrete thickness with the assumed concrete elastic modulus and backcalculated modulus of subgrade reaction. Again, using the 2008 deflection data from the McKinley parking lot, the sensitivity of backcalculated h_{eff} to the assumed value of E indicated that increasing E to 6 to 7×10^6 psi resulted in a 5% to 10% decrease in the h_{eff} value, while lowering E to 3 to 4×10^6 psi resulted in a 7% to 18% increase in h_{eff}. This sensitivity analysis is also shown in Table 3.7.

Table 3.6. Average Values of k and h_{eff}

Project/Section		Date	Average Modulus of Subgrade Reaction, k (psi/in)	Average Effective Slab Thickness (in)	Thickness Standard Deviation (in)	Estimated Thickness: $h_{PCC} + h_{AC} = h_{total}$ (in)
McKinley Lot	Northeast	2008	197	5.06	0.475	3.5 + 4 = 7.5
		2012	241	5.52	0.938	
	Northwest	2008	279	6.63	0.603	
		2012	291	7.23	1.30	
	Southeast	2008	421	6.89	0.583	
		2012	324	7.45	2.35	
E-15 Lot	1	2008	182	5.71	0.603	3.5 + 2.5 = 6
		2012	152	5.81	0.963	
	2	2008	241	6.20	0.617	
		2012	202	5.80	0.810	
	3	2008	200	6.66	2.07	
		2012	186	6.62	1.46	
	4	2012	202	5.94	0.666	
Talbot Lot		2008	176	4.56	0.632	3 + 2.5 = 5.5
		2012	160	4.20	0.454	
Bible Grove Road (5.5 by 5.5 ft only[b])	Intensive	2012	179	7.08	1.07	5[a]
	Periodic		230	6.19	0.795	
Richland County Hwy 9	Intensive	2012	275	8.26	2.87	5.5[a]
	Periodic		206	7.81	2.24	
North Lorang Road	1	2013	156	5.89	1.85	4.5[a]
	2		394	8.39	1.32	
	3		379	7.47	0.801	
Schank Avenue	Northbound	2013	387	5.32	0.506	4[a]
	Southbound		300	6.52	1.30	

[a]Concrete thickness only. Thickness of the underlying asphalt layer is unknown.
[b]Backcalculations were not performed on the 11 foot panel section of Bible Grove Road.

Table 3.7. McKinley Lot 2008 Sensitivity Analysis

Effective Thickness Values					% Difference				
NE Section					NE Section				
	LTE (%)					LTE (%)			
E (psi)	0	50	80	100	E (psi)	0	50	80	100
7×10^6	4.09	4.44	4.52	4.47	7×10^6	19.1	12.2	10.6	11.6
6×10^6	4.31	4.67	4.76	4.71	6×10^6	14.8	7.58	5.90	6.93
5×10^6	4.58	4.97	5.06[a]	5.00	5×10^6	9.46	1.78	0[a]	1.10
4×10^6	4.93	5.35	5.45	5.39	4×10^6	2.47	5.8	7.72	6.54
3×10^6	5.43	5.89	5.99	5.93	3×10^6	7.35	16.5	18.6	17.3
NW Section					NW Section				
	LTE (%)					LTE (%)			
E (psi)	0	50	80	100	E (psi)	0	50	80	100
7×10^6	5.31	5.82	5.93	5.84	7×10^6	19.9	12.2	10.6	11.9
6×10^6	5.59	6.13	6.24	6.15	6×10^6	15.7	7.57	5.90	7.24
5×10^6	5.94	6.51	6.63[a]	6.54	5×10^6	10.4	1.78	0[a]	1.43
4×10^6	6.4	7.02	7.14	7.04	4×10^6	3.50	5.80	7.72	6.19
3×10^6	7.04	7.72	7.86	7.75	3×10^6	6.22	16.5	18.6	16.9
SE Section					SE Section				
	LTE (%)					LTE (%)			
E (psi)	0	50	80	100	E (psi)	0	50	80	100
7×10^6	5.55	6.05	6.16	6.08	7×10^6	19.4	12.2	10.6	11.7
6×10^6	5.85	6.37	6.48	6.4	6×10^6	15.1	7.58	5.90	7.03
5×10^6	6.21	6.77	6.89[a]	6.81	5×10^6	9.81	1.79	0[a]	1.20
4×10^6	6.69	7.29	7.42	7.33	4×10^6	2.85	5.79	7.72	6.4
3×10^6	7.37	8.02	8.17	8.07	3×10^6	6.93	16.4	18.6	17.1

[a]Highlighted cells indicate the E and LTE values assumed when performing original backcalculations for the 2008 McKinley parking lot data

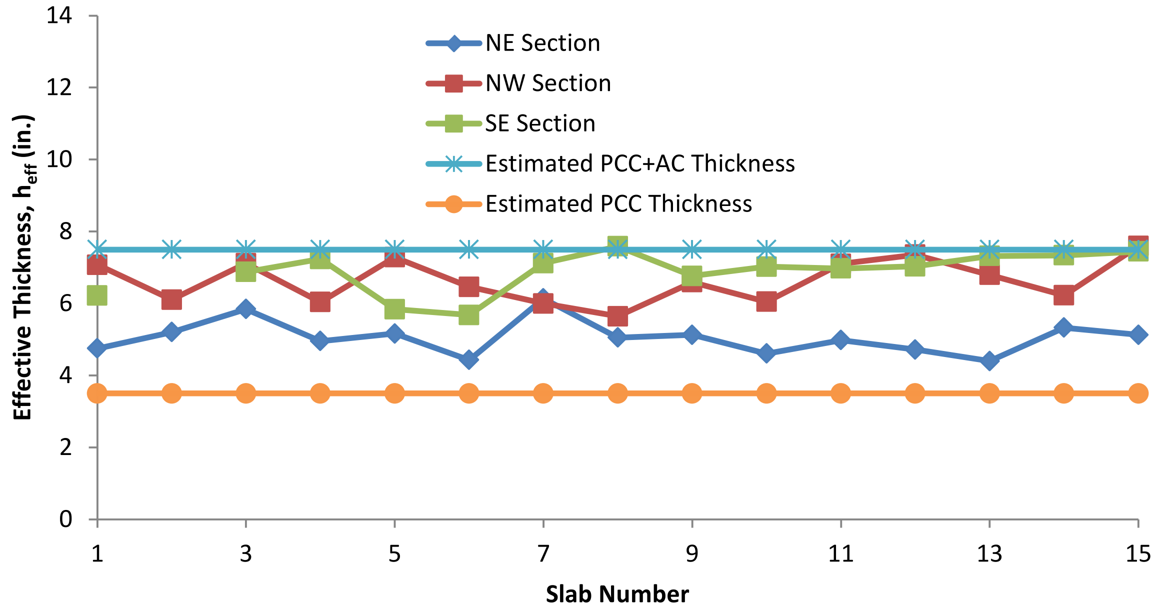

Figure 3.18. Effective thickness, McKinley lot, 2008.

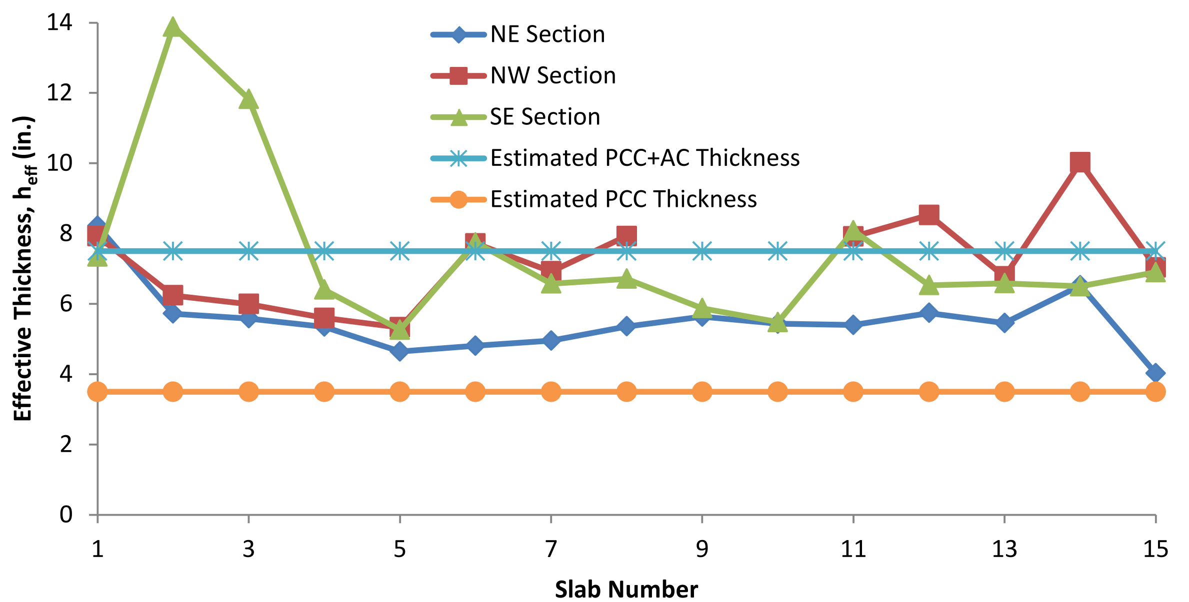

Figure 3.19. Effective thickness, McKinley lot, 2012.

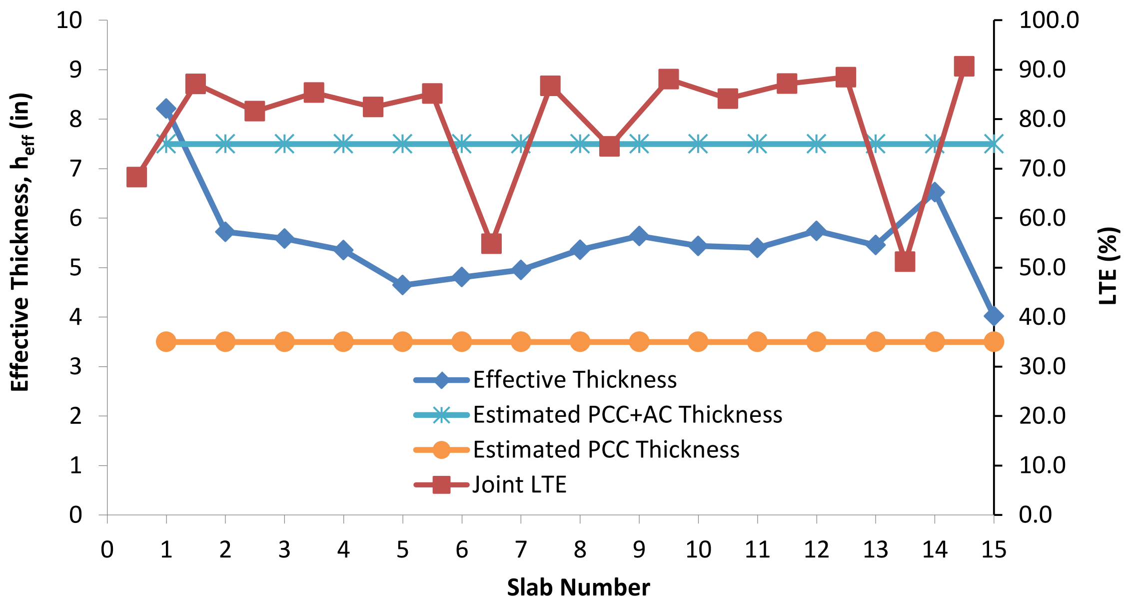

Figure 3.20. Comparison of local transverse joint LTE and effective slab thickness, northeast section, McKinley lot, 2012.

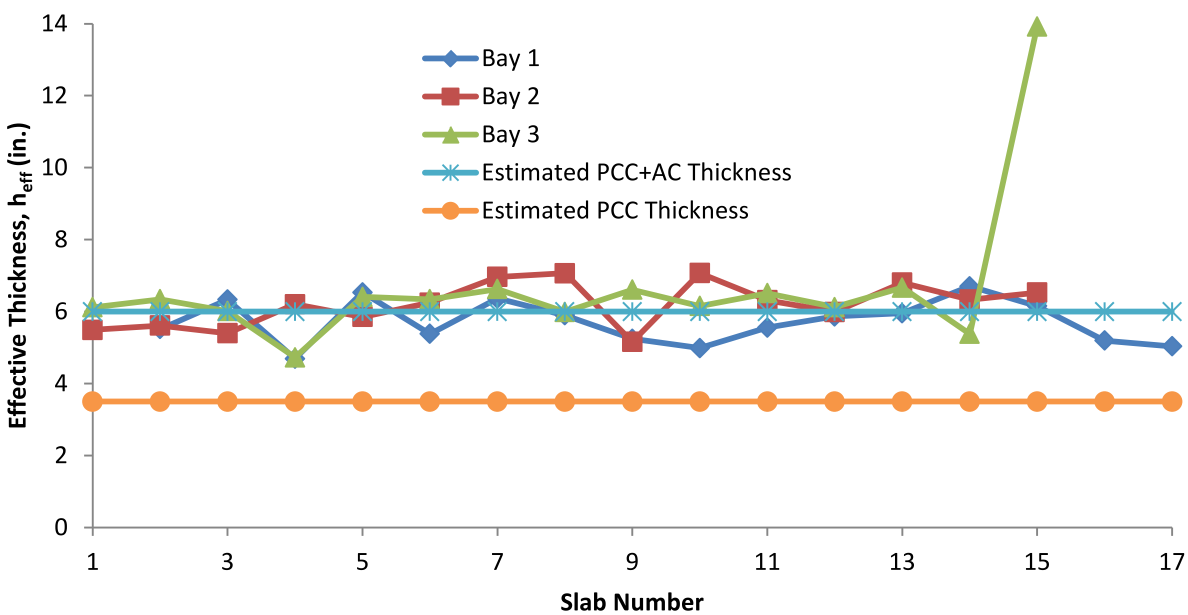

Figure 3.21. Effective thickness, E-15 lot, 2008.

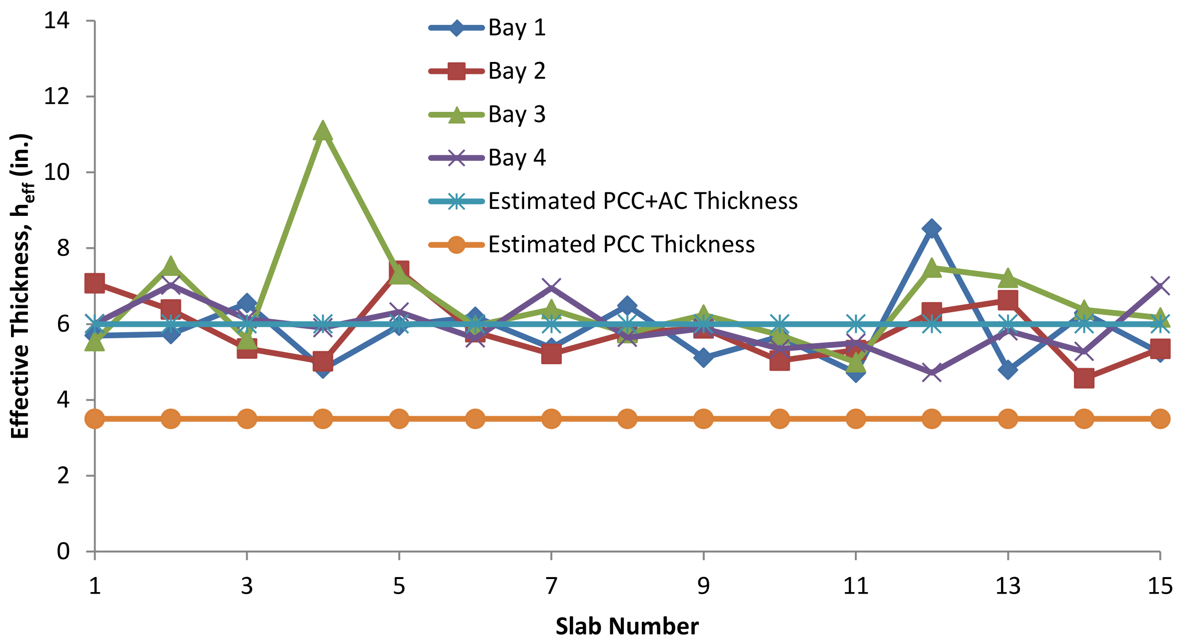

Figure 3.22. Effective thickness, E-15 lot, 2012.

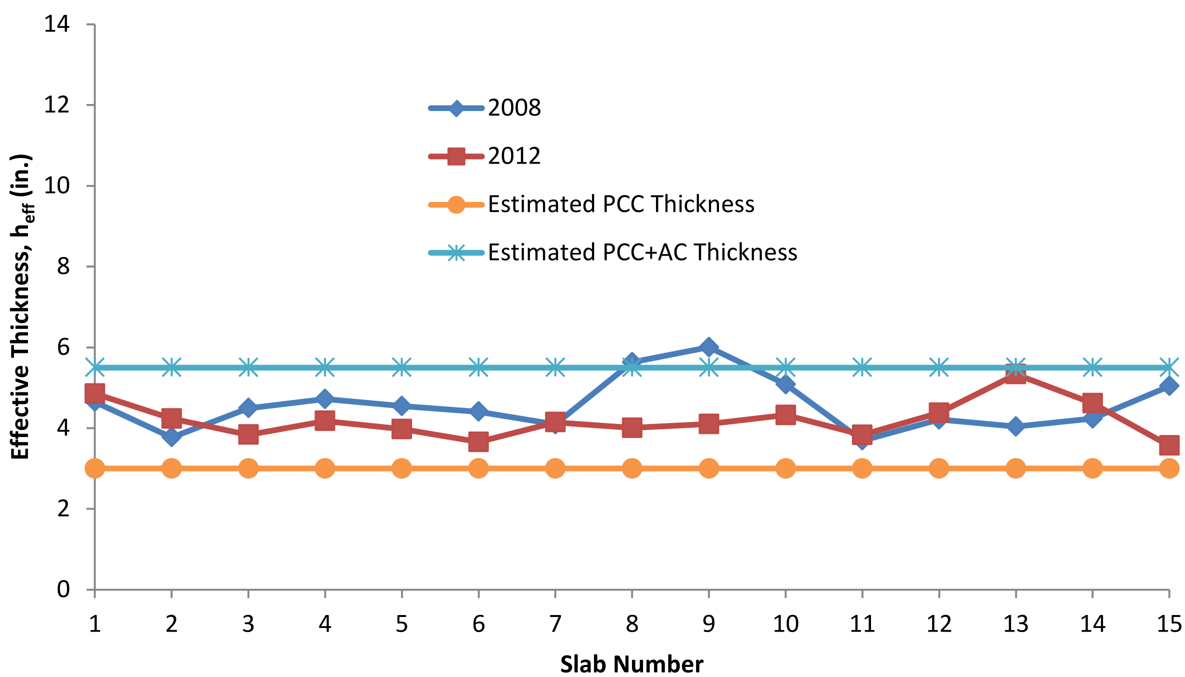

Figure 3.23. Effective thickness, Talbot lot, 2008 and 2012.

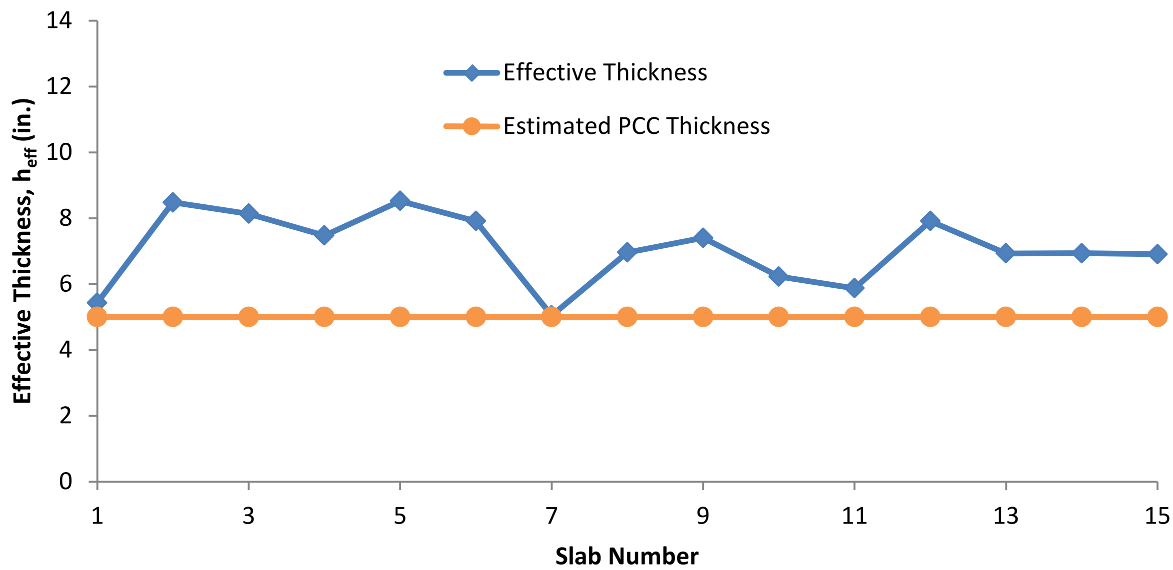

Figure 3.24. Effective thickness, intensive testing on 5.5 foot panels, Bible Grove Road, 2012.

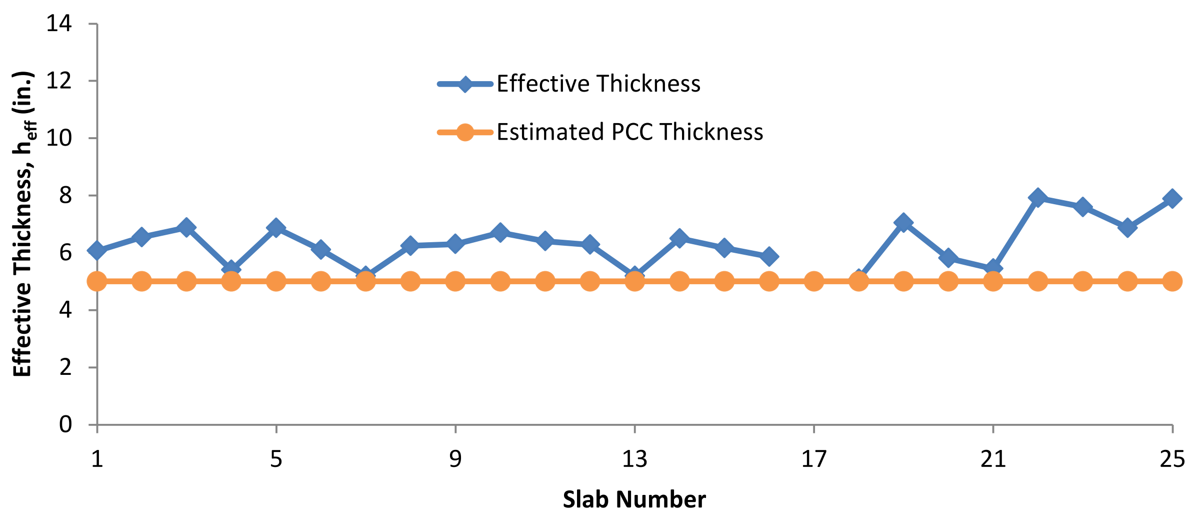

Figure 3.25. Effective thickness, periodic testing on 5.5 foot panels, Bible Grove Road, 2012.

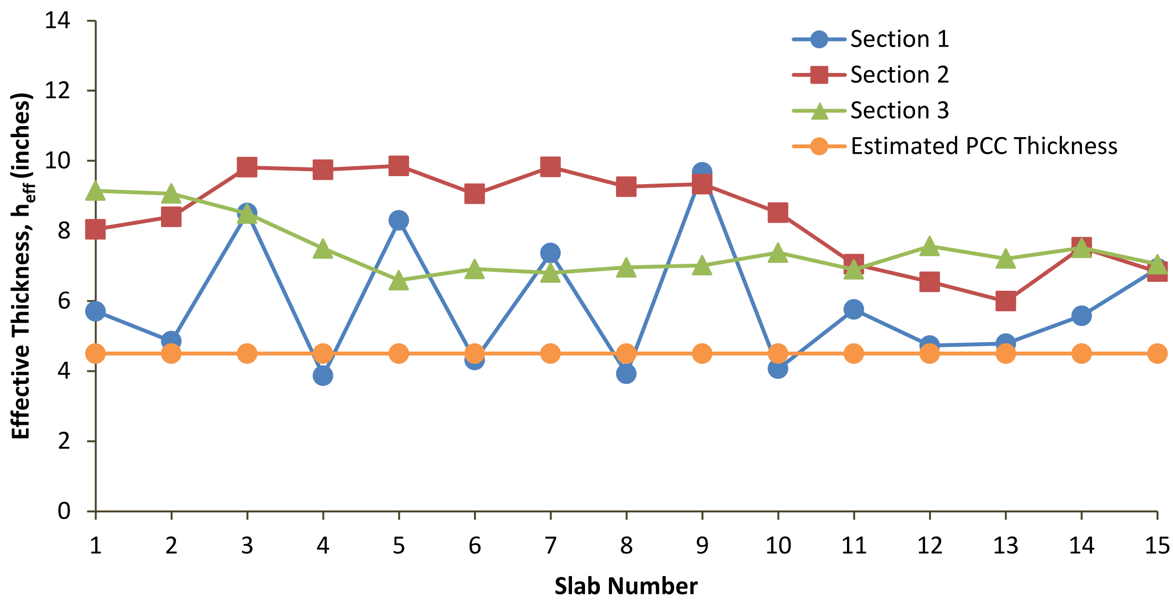

Figure 3.26. Effective thickness, North Lorang Road, 2013.

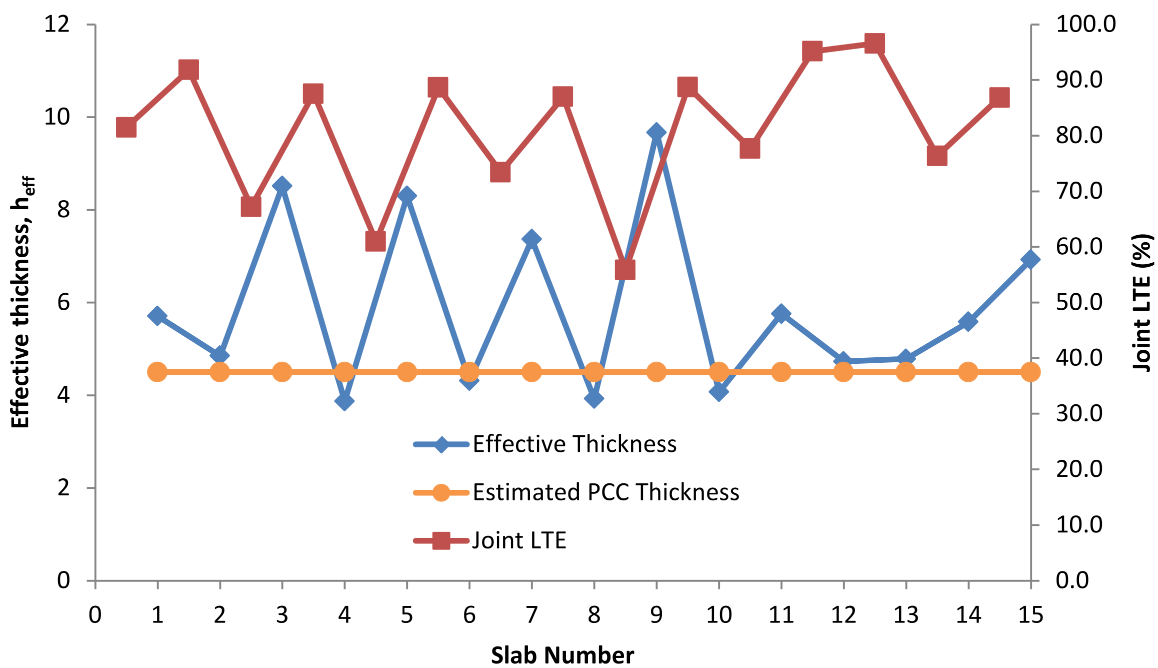

Figure 3.27. Comparison of local transverse joint LTE and effective slab thickness, Section 1, North Lorang Road, 2013.

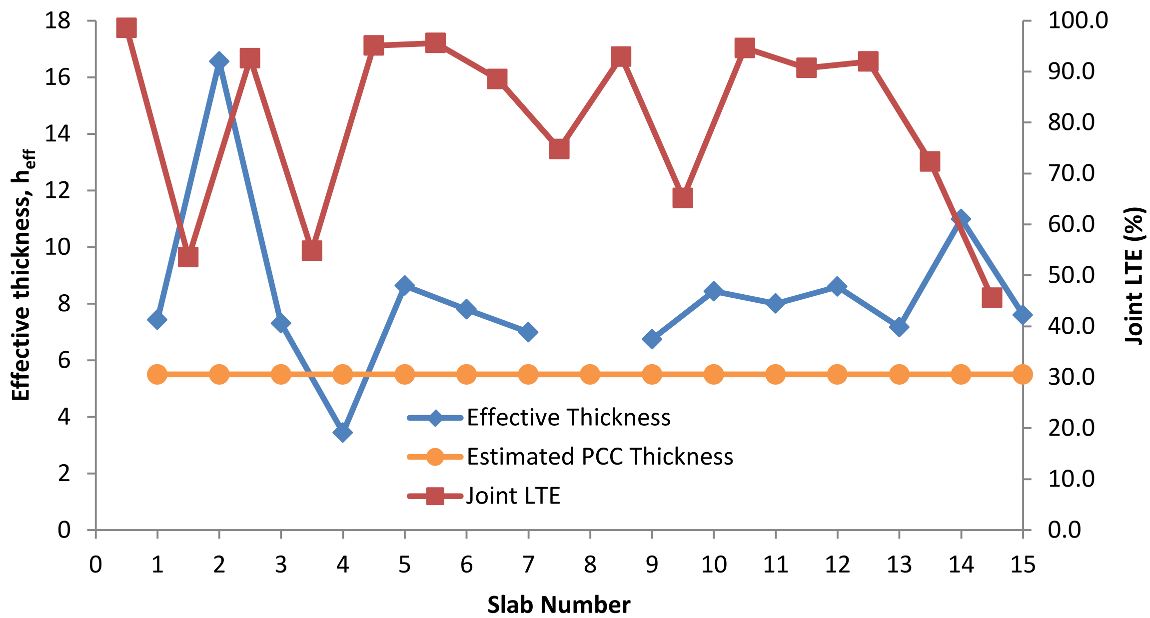

Figure 3.28. Comparison of local transverse joint LTE and effective slab thickness, intensive testing, Richland County Highway 9, 2012.

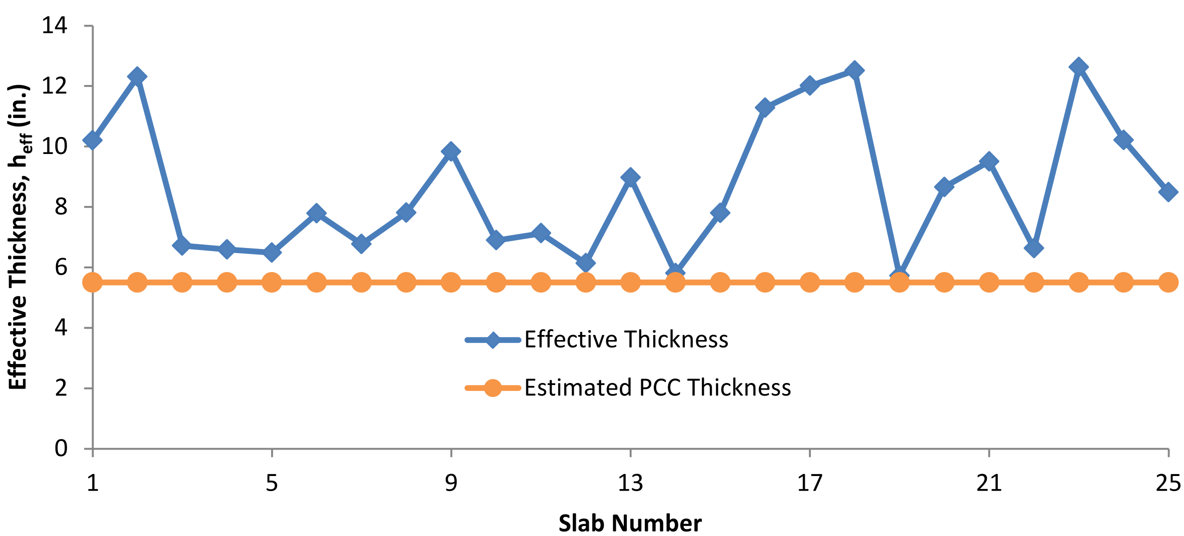

Figure 3.29. Effective thickness, periodic testing, Richland County Highway 9, 2012.

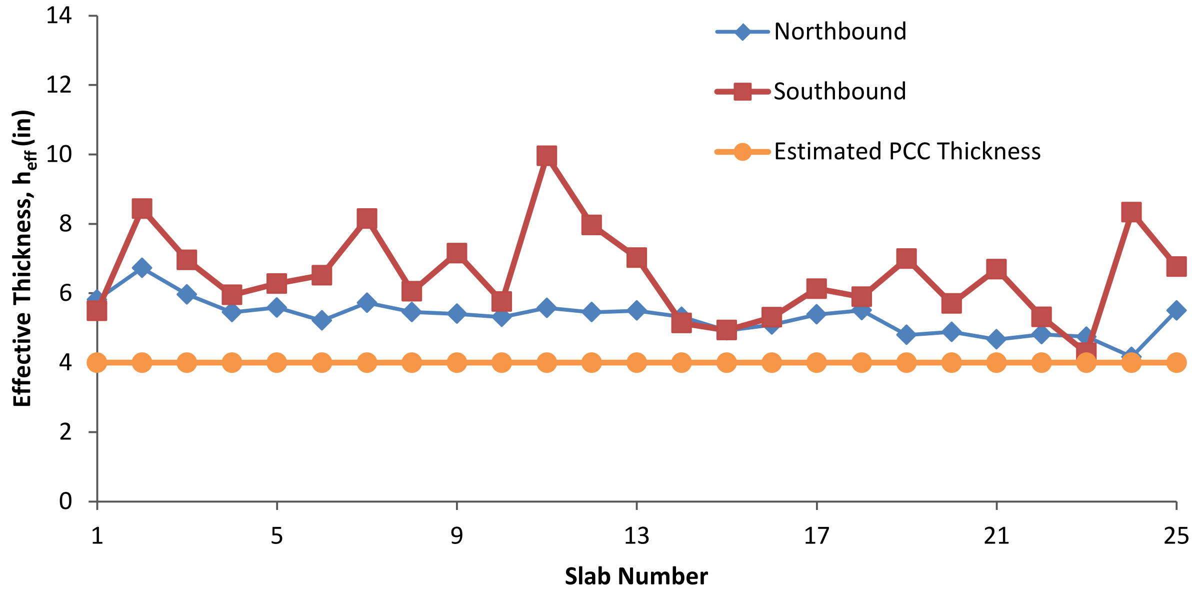

Figure 3.30. Effective thickness, Schank Avenue, 2013.

3.3 ANALYSIS

With the load transfer efficiency and backcalculated effective thickness values that were determined from the FWD test data, it was possible to gain insight about the structural performance and load carrying capacity of the UTW projects that were tested. The results were combined with the findings of the visual distress surveys in Chapter 2 to relate observed behavior to the data.

3.3.1 Joint Performance

Across most of the projects that were tested, average joint load transfer efficiencies (Table 3.2) ranged from about 70% to 90%, which indicates good joint performance and is consistent with the findings of the surveys. The average transverse joint LTE for the 11 foot panel section of Bible Grove Road, which was substantially lower (ranging from 20% to 60%) than those of the rest of the projects, was compared directly with the 5.5 foot panel section, as shown in Figure 3.9. The LTE differences suggest that the faulting development noted in the 11 foot section was a key factor, but it was not seen in the 5.5 foot panel section during the June 2012 survey.

For Piatt County Highway 4, another project that featured different sections with both 5.5 and 11 foot panels, the joint LTE data are less conclusive. The report from ERI (2012) did not find a clear trend in LTE between the 5.5 and 11 foot panel sections. However, the range of average LTE values reported for portions of the 11 foot section featured lower values (73% to 92%) than those of the 5.5 foot section (88% to 90%).

A common finding when plotting transverse joint load transfer efficiency as a function of distance was local variation in LTE from slab to slab. Numerous local reductions in transverse joint LTE are shown in Figures 3.10, 3.20, 3.27, and 3.28. These reductions are hypothesized as either dominant joints or locations where full-depth joint crack has propagated through both the concrete and asphalt layers (Cervantes et al. 2009). These reductions in joint LTE may have affected the backcalculated h_{eff} results in some cases, which will be discussed in the next section (3.3.2).

3.3.2 Structural Performance

The average values for k and h_{eff} (Table 3.6) for each project indicate sufficient support stiffness and equivalent concrete thickness for the intended facility (i.e., parking lot, street, or highway). These findings match the surveys, where each of the projects was observed to be in good condition with less than 8% panels cracked, except for the Talbot parking lot, which had 22% panels cracked at year 14 but still had good serviceability. In some sections of the E-15 parking lot, h_{eff} exceeded the combined estimated concrete and asphalt pavement thickness. In theory, h_{eff} should always be less than the total thickness because the interface is not fully bonded and the asphalt stiffness is less than the concrete stiffness. However, the effective thickness results simply suggest a deviation in the in situ concrete overlay thickness, asphalt layer thickness, or both, relative to design thickness. This finding is consistent with UTW parking lot pavements, which typically have variable concrete thickness to meet the desired grades.

Searching for trends between the 2008 and 2012 test results for the University of Illinois parking lots proved largely inconclusive. The average joint LTEs for the McKinley and E-15 parking lots remained mostly unchanged between 2008 and 2012, and the differences in average h_{eff} for the two lots were not statistically significant to a 90% confidence level. The Talbot parking lot was an exception—there was a decrease in h_{eff} between 2008 and 2012 that was significant to a 90% confidence interval. This decrease may be due to deterioration in the underlying asphalt layer or a slight difference in the FWD testing location on the site. This asphalt layer was already very thin and distressed when the lot was constructed (Riley 2010), and now there is panel cracking (22%) in the overlay. For the time being, however, the lot is still highly functional.

A comparison of the backcalculated h_{eff} of the newly completed section (Bay 4) of the E-15 lot with the sections that were six years old at the time of the 2012 testing is provided in Figures 3.10 and 3.21 for LTE and h_{eff}, respectively. The primary difference appears to be that there is not as much variation in both LTE and h_{eff} in Bay 4 compared with the older sections of the lot and no instances of local spikes or reductions. This uniform behavior may indicate that full-depth cracks did not occur, which may have been due to the weather during construction or to a combination of reasons, including a difference in the concrete material hydration and initial cooling.

Effective thickness results for intensive and periodic testing of the 5.5 foot panel section of Bible Grove Road are presented in Figures 3.24 and 3.25. The h_{eff} values for the periodic testing were stable at each successive measurement and consistent with the h_{eff} values for the intensive testing, indicating that the structural conditions did not vary much across the section. Though the underlying HMA thickness of Bible Grove Road was unknown, the average h_{eff} values (Table 3.6) that were calculated were only about 1 to 2 inches thicker than the estimated PCC thickness, suggesting a relatively thin asphalt support layer.

On North Lorang Road, the h_{eff} that was backcalculated in Section 1 (6.24 inches) was about 1 to 2 inches lower than the h_{eff} measured in Sections 2 (8.36 inches) and 3 (7.47 inches) (Figure 3.26, Table 3.6), and the difference was significant to a 90% confidence interval. These results seem to correspond with the findings of the visual distress survey because Section 1 of the FWD testing of North Lorang Road was the southernmost portion of the project. That section of the project experienced the greatest degree of distress and apparent loss of structural support in some areas, which would be expected to result in a lower h_{eff}. The backcalculated h_{eff} values in Section 1 were also significantly more volatile than in the other two sections, with spikes occurring at Slabs 3, 5, 7, and 9, shown in Figure 3.27. These spikes appeared to correspond to local reductions in LTE at the joint with the preceding slab (i.e., the spike in h_{eff} at Slab 3 corresponded with a local reduction in LTE at the joint between Slabs 2 and 3). As discussed in Section 3.3.1 of this paper, the local reductions in joint LTE may indicate dominant or cracked joints, and it appears that every other transverse joint in this location had cracked.

There were several other cases where spikes in h_{eff} corresponded to local reductions in LTE at the preceding transverse joint, including at slabs 1 and 14 in the northeast section of the McKinley lot (Figure 3.20) and at slab 2 on Richland County Highway 9 (Figure 3.28). To unravel the cause of this phenomenon, one of the transverse joints was assumed not to crack at the expected 4 foot interval, thus providing 100% moment and shear transfer, as seen in Figure 3.31. This resulted in a 4 by 8 foot concrete slab with the outer joints at 50% LTE. In the ILLI-SLAB analysis, the FWD plate was placed at the apparent center of a 4 by 4 foot slab to analyze the deflection behavior and compare it to the case where all joints were 50% LTE.

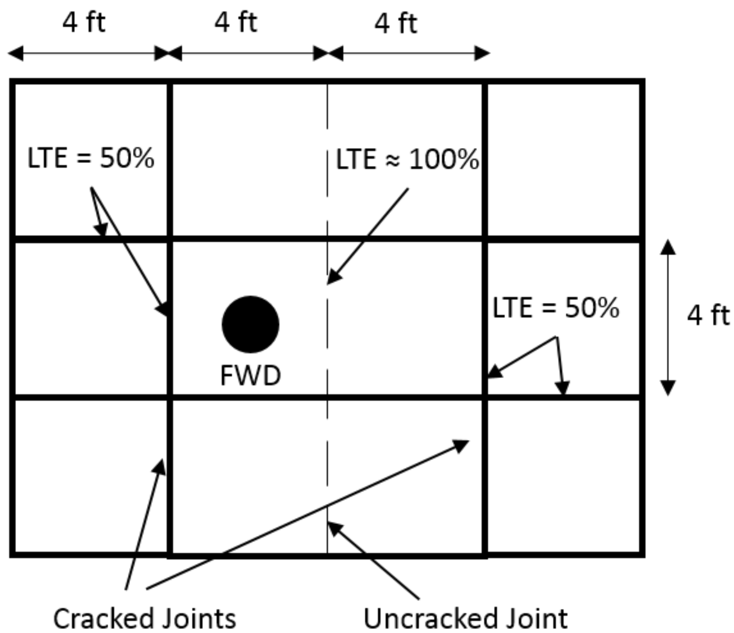

Figure 3.31. ILLI-SLAB analysis to simulate slab behavior with cracked and uncracked joints.

The $AREA_{24}$ deflection basins calculated from the analysis of Figure 3.31 analysis were significantly higher than those obtained from the standard case with all other inputs the same. Further analysis with 5 and 6 foot panel sizes confirmed the increase in $AREA_{24}$ when the transverse joint was continuous and uncracked. Experimentally, if an FWD load was applied on a slab with the preceding joint cracked but the subsequent joint uncracked, the pavement would produce a higher $AREA_{24}$ value than it would if all joints were exhibiting the same behavior. When the higher $AREA_{24}$ is used in the presented backcalculation process for BCOA, a higher effective thickness is estimated. Therefore, if h_{eff} spikes occur simultaneously with observed joints that are uncracked, the backcalculated h_{eff} is overestimated for those slabs.

Effective thickness results for intensive and periodic testing of Richland County Highway 9 are presented in Figures 3.28 and 3.29, respectively. Figure 3.28 also includes joint LTE data for the periodic testing section. The average backcalculated h_{eff} values for both sections were very high, which would be expected because few distresses and a smooth ride were noted in the survey, but the standard deviation was also very high (Table 3.6). As mentioned in Chapter 2, portions of the overlay were placed directly on cement-stabilized soil, so the variation in the data may be due to the underlying conditions and/or variation in concrete thickness. Additionally, as previously discussed, a spike in h_{eff} at Slab 2 appears to correspond to a drop in LTE at the joints between Slabs 1 and 2.

Figure 3.30 shows a comparison of the effective thickness results for the northbound and southbound lanes of Schank Avenue. Effective thickness was consistently higher in the southbound lane, but it was also more variable. As noted in the surveys, settlement was occurring in the embankment in the middle of the roadway section. The settlement may have been worse on one side of the pavement centerline, causing more structural deterioration and culminating in a lower h_{eff} value in the northbound lane. If settlement were uneven, it would also account for the variability in h_{eff} values in the southbound lane.

The missing values for effective thickness at certain slabs (e.g., at slab 8 in Figure 3.28) were due to a measurement error because the deflection measured 12 inches from the load plate (d_1) at these locations exceeded the deflection directly underneath the load plate (d_0). This deflection behavior cannot produce realistic backcalculation values. Any data points where this situation occurred were removed from the set.

Overall, the concrete–asphalt bond appeared to be at least partially intact across all of the projects that were tested because the average h_{eff} always exceeded the best-known concrete thickness (Table 3.6). The surveys support this finding, as debonding was only very rarely detected by sounding. There were some instances of localized dips in h_{eff} for certain projects, but it was impossible to determine whether debonding occurred at these locations or whether it was just variance in the underlying support stiffness and thickness.

CHAPTER 4 CASE STUDY: ILLINOIS ROUTE 53 IN WILL COUNTY

After the distress surveys and FWD data analysis detailed in Chapters 2 and 3 were completed in 2012-13, another UTW pavement, Illinois Route 53 in Will County was evaluated. Constructed in 2012, concern surrounded this project because of the development of rather severe premature distresses in certain parts of the project. In order to investigate the cause of these problems, distress surveys and FWD testing were carried out at Route 53 in summer 2014. This chapter will detail the findings from this project to determine what went wrong and how to prevent similar problems with future UTW projects.

4.1 Project Details and Distress Survey

The UTW section on Illinois Route 53 in Will County is a four lane divided highway with two lanes each in the northbound and southbound directions, running approximately four miles from Arsenal Road at the south end to Hoff Road at the north end. There was a significant amount of truck traffic on this section due to nearby intermodal facilities. As of 2013, single-unit trucks accounted for 12.3% of all traffic, while multi-unit trucks accounted for 19.4% of all traffic. Project details are provided in Table 4.1, and an overview of the project is pictured in Figure 4.1.

The existing pavement at Route 53 consisted of two asphalt overlays, completed in 1987 and 2000, of an older concrete pavement that dated to 1943. However, as revealed by pre-construction cores, the layer thicknesses varied at different points in the cross-section and in some areas the existing pavement was full-depth asphalt. A summary of the layer thicknesses measured from the pre-construction cores is shown in Table 4.2. The locations of the cores are organized in terms of the station and transverse offset from the centerline of the roadway. For the UTW construction, the existing asphalt surface, which as shown in Table 4.2 typically ranged from 7 to 10 inches, was milled down by approximately four inches and replaced with a concrete inlay.

Table 4.1. Route 53 Project Details

Completion Date	2012
Overlay Thickness	4 inches
Underlying Thickness/ Condition	Milled HMA over PCC (Table 4.2 for thicknesses)
Slab Size	4 by 4 feet
ADT	7,750
Fiber Reinforcement	4 lb/yd^3 synthetic

Table 4.2. Pre-Construction Core Data

Core Number	Station	Transverse Station Offset (feet)	Total Asphalt Thickness (inches)	Total PCC Thickness (inches)
Northbound				
3	30+00	7 Rt CL	9.75	5.75
23	40+41	48 Rt CL	11.00	—
6	51+50	21 Rt CL	7.75	12.25
24	70+89	47 Rt CL	2.25	9.00
29	94+00	17 Rt CL	16.50	—
9	94+00	43 Rt CL	9.75	10.25
11	125+50	28 Rt CL	7.00	11.75
14	164+00	63 Rt CL	8.50	10.50
15	186+00	50 Rt CL	7.50	11.25
18	226+00	44 Rt CL	8.50	10.50
19	245+50	36 Rt CL	7.00	12.00
25	264+80	16 Rt CL	8.00	11.00
Averages (Northbound)			8.63	10.43
Southbound				
1	18+00	17 Lt CL	9.00	—
2	26+50	6 Lt CL	9.50	7.50
4	37+00	29 Lt CL	12.00	8.00
5	41+00	23 Lt CL	9.25	7.25
28	44+86	13 Lt CL	14.50	—
26	51+50	26 Lt CL	11.50	—
7	62+00	35 Lt CL	9.50	7.75
27	75+00	10 Lt CL	2.50	9.75
8	77+00	20 Lt CL	8.25	9.25
22	97+75	47 Lt CL	16.00	—
10	107+00	43 Lt CL	9.50	7.50
12	132+00	33 Lt CL	11.25	6.25
13	152+00	66 Lt CL	11.50	7.25
16	194+00	35 Lt CL	10.00	—
17	215+00	44 Lt CL	9.00	15.25
20	254+00	36 Lt CL	6.00	9.50
Averages (Southbound)			9.95	8.66

Figure 4.1. Overview of Route 53 in Will County (June 2014).

Although the percentage of total slabs cracked in the three section that were surveyed was not high, as seen in Table 4.3, distresses were more prevalent in the northbound lane than in the southbound lane and much more serious. There were particularly severe distresses along the lane-shoulder joint at various points throughout the northbound lane (not necessarily included in the distress survey), as shown in Figure 4.2, and inner wheel path of the right (driving) lane, seen in Figure 4.3. A number of panels in these areas also featured asphalt or concrete patching, indicating the same problems had occurred there. An example of asphalt patching surrounded by other local distresses is shown in Figure 4.4.

Table 4.3. Route 53 Distress Survey

Section	1	2	3	Total	%
Direction	North	South	North		
Total # Slabs	810	750	750	2310	—
#Slabs Corner Breaks	8	2	7	17	0.7
# Slabs Longitudinal Cracks	1	0	1	2	0.1
# Slabs Transverse Cracks	3	0	7	10	0.4
# Slabs Diagonal Cracks	1	1	2	4	0.2
# Shattered Slabs	0	0	0	0	0.0
# Slabs Patched	2	0	0	0	0.0
# Slabs Replaced	0	0	0	0	0.0
Total # Slabs Cracked	11	3	15	29	1.3

Figure 4.2. Severe distresses at lane-shoulder joint in northbound lane (IDOT photo).

Figure 4.3. Distresses in the inner wheel path of the right lane in the northbound direction (June 2014).

Figure 4.4. Asphalt patching and distresses in the right lane in the northbound direction (June 2014).

From sounding tests, there was evidence of debonding throughout the project, but mostly in the northbound lane. In particular, the panels at the right edge of the northbound lane appeared to exhibit quite a bit of debonding. A few instances of faulting measuring almost one inch, as shown in Figure 4.5, were observed in both the transverse and longitudinal joints in the right lane in the northbound direction. There may also have been some drainage problems at various points along the right edge of the northbound lane, particularly in certain locations where the cross slope of the mainline pavement and shoulder did not match, leaving a potential area for water to accumulate. One such area is pictured in Figure 4.6.

Figure 4.5. Faulting in the longitudinal joints in the right lane in the northbound direction (June 2014).

Figure 4.6. Potentially problematic drainage area at joint with shoulder (June 2014).

There were some cracked panels in the southbound pavement, as well as isolated signs of debonding, but for the most part there were few distresses and the ride quality was smooth in the southbound lane. The ride quality and total level of distresses in the northbound lane were also largely fine over the entire project length, but the severity of the distresses, as well as the fact that the project was only two years old, made them a major concern.

4.2 FWD TESTING

To further analyze Route 53, intensive and periodic FWD testing was performed to calculate joint load transfer efficiency as well as backcalculate the effective k-value and thickness of the pavement. Intensive FWD testing allowed for characterization of smaller sections of the project in detail, while periodic FWD testing provided a gross understanding of the overall properties of the project).

4.2.1 Test Plans and Procedure

4.2.1.1 Intensive Testing

Intensive testing was conducted at four different sites along the project, which can be seen in Figure 4.7. Sites 1 and 2 are in the northbound lane, while Sites 3 and 4 are in the southbound lane. Testing at Sites 1 and 4 began at the entrance to Midewin National Tallgrass Prairie near the south end of the project. Testing at Site 1 proceeded northward in the northbound lane, while testing at Site 4 proceeded southward in the southbound lane. Testing at sites 2 and 3 began at an intersection with a road near a railroad crossing in the middle of the project just past the point where Route 53 curves toward the northeast. Testing at Site 3 proceeded northward in the northbound lane, while testing at Site 4 proceeded southward in the southbound lane.

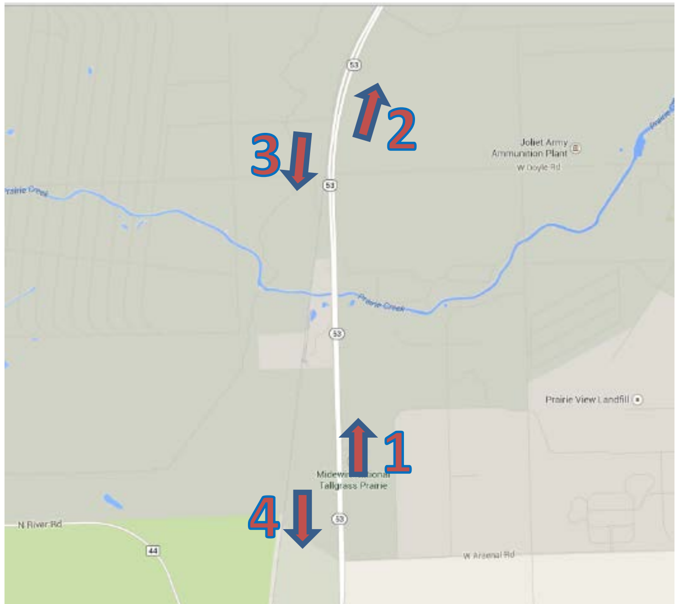

Figure 4.7. Intensive testing sites on Route 53.

At each site, intensive testing was performed in passes over a series of 15 consecutive slabs in three different rows (left lane center, right lane center, and right lane right edge), for a total of 45 slabs tested at each site, and each slab featured two to four drop locations. The drop pattern provided in Figure 4.8 shows the slabs that were tested and locations on each slab for the intensive testing at each site.

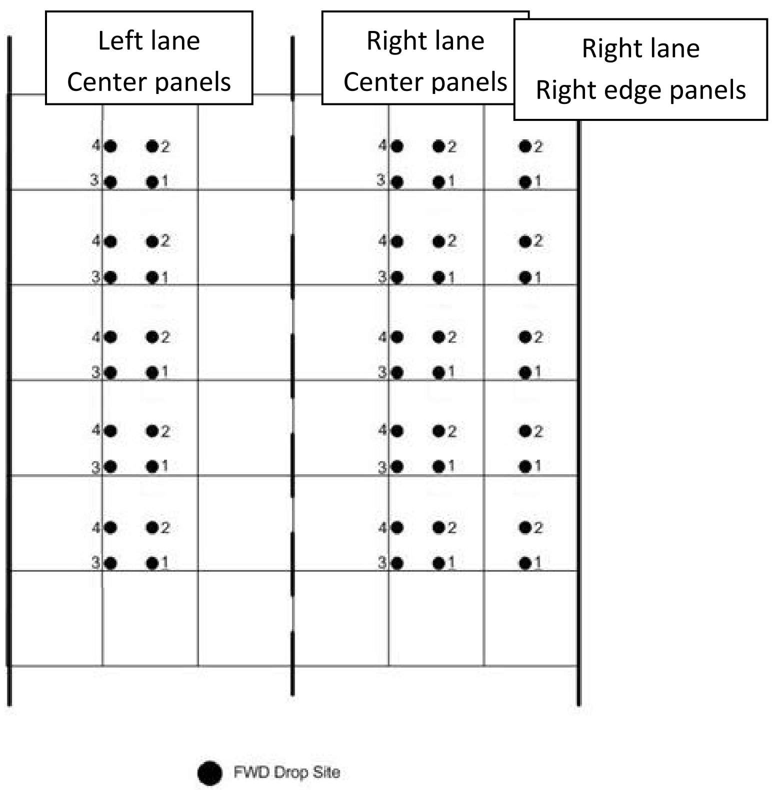

Figure 4.8. FWD drop pattern, Intensive Testing, Route 53.

4.2.1.2 Periodic Testing

In addition to intensive testing, periodic testing was performed at mile markers 1, 2, 3, and 4 in both the northbound and southbound directions. Periodic testing consisted only of the drops at locations 1 and 2 in Figure 4.2 in the right lane center panels every 100 feet for 10 slabs at each mile marker.

4.2.1.3 Test Procedure at Each Drop Location

The same test procedure at each drop location used on the UTW projects in Chapter 3 were used for intensive and periodic testing of Route 53. The deflections measured at each test location were used to characterize the UTW section by determining joint load transfer efficiency (LTE) and backcalculating the effective pavement thickness (h_{eff}) and modulus of subgrade reaction (k-value). The procedure for determining transverse and longitudinal joint LTE, as well as the backcalculation method used to determine h_{eff} and k, were outlined in Chapter 3.

4.2.2 FWD Results and Discussion

4.2.2.1 Load Transfer Efficiency

Transverse joint load transfer efficiencies were high throughout the project. Average transverse joint LTE ranged between 85% and 92% for each of the intensive testing sections. Average transverse joint LTE values obtained from periodic testing were in roughly the same range, between 82% and 90% for a given periodic section. Where it was measured, average longitudinal joint LTE ranged from 78% to 90%. Ride quality was good and little faulting was observed, even near the more distressed areas of the project, so these LTE findings make sense.

There were very few instances of local drops in transverse joint LTE, which have been found on a number of other UTW projects (Chapter 3) and may indicate dominant or cracked joints. Overall, this suggests that the support under the slab was continuous and likely thick. One of the few drops that was found with lower LTE in the project was during periodic testing of the southbound lane at Mile 3. The drop occurs at the joint just before periodic test slab 7 in the plot in Figure 4.9. Note that since these results are from periodic testing, the drop in Figure 4.9 does not represent a change in LTE between consecutive slabs, but rather between slabs 100 feet apart.

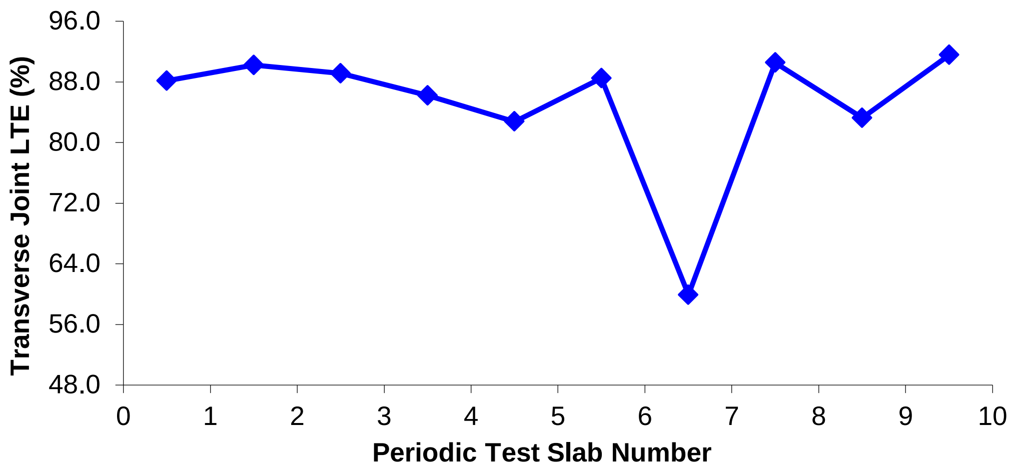

Figure 4.9. Variation in transverse joint LTE, Periodic Testing, Southbound, Mile 3.

4.2.2.2 Effective Thickness

A summary of the average backcalculated h_{eff} and k-values at each of the different test sections on Route 53 is provided in Table 4.4. Assumptions of layer characteristics to complete the backcalculations were the same as in Chapter 3 with one exception. The input modulus of elasticity (E) was increased from 5×10^6 psi to 5.87×10^6 psi based on measurements of shear wave velocity taken at Route 53. Shear wave velocity (v_s) was related to modulus of elasticity using the relationship shown in Equation 4.1, assuming a Poisson's ratio (v) of 0.15 and concrete density (ρ) of 140 lb/ft^3.

$$E = 2v_s(1 + \nu)\rho \tag{4.1}$$

The results in Table 4.4 are organized from test sites located at the south end of the project to the north end. For example, Sites 1 and 4, listed at the top of the table for intensive testing, are located at the south end of the project. Likewise, Mile 1 in the northbound lane and Mile 4 in the southbound lane, listed at the top of the table for periodic testing, are located at the south end of the project.

Table 4.4. Summary of Average Backcalculated Values

Intensive Testing							
Location		Left Lane, Center Panels		Right Lane, Center Panels		Right Lane, Right Panels	
Site	Direction	k (psi/in)	h_{eff} (in)	k (psi/in)	h_{eff} (in)	k (psi/in)	h_{eff} (in)
1	Northbound	370	5.62	443	6.03	285	5.66
4	Southbound	245	5.77	385	6.77	279	6.24
2	Northbound	402	6.90	495	5.86	250	5.99
3	Southbound	303	6.39	363	6.21	303	7.05

Periodic Testing (Right Lane, Center Panels)			
Mile	Direction	k (psi/in)	h_{eff} (in)
1	Northbound	528	5.41
4	Southbound	589	5.71
2	Northbound	492	5.52
3	Southbound	371	6.44
3	Northbound	381	5.39
2	Southbound	492	6.37
4	Northbound	284	5.22
1	Southbound	401	6.74

As seen in Table 4.4, effective thicknesses at sites tested in the southbound lane were, on average, higher than those in the northbound lane. This finding is supported most by the results of the periodic testing, which characterized the pavement project over a larger total area. The average h_{eff} at each southbound periodic test site were higher than any of the northbound sections, which seemed to agree with the pre-construction core data (Table 4.2). In addition, analyses of the FWD data suggest that most of these differences are statistically significant.

As seen from the intensive testing results in Table 4.4, the pavement structure (as characterized by h_{eff}) varied between different rows of panels within the same sites, and even between the adjacent center and right rows of panels in the right lane. Preliminary statistical analyses of the FWD results indicated that most of these differences were statistically significant, although the magnitude of this variation was not always very large.

There was also a large variation in the average subgrade k-value across different rows of panels within the same section. Most striking was the difference in k-value between the right lane, center panels and right lane, right panels at Sites 1 and 2 during intensive testing. Although the average h_{eff} did not change much between these rows (and the difference was not statistically significant), the average k-value in the panels at the right pavement edge dropped significantly as compared to the center panels in the right lane. Both of these sections were part of the northbound lane, which experienced significant distresses in some areas on the right pavement edge. These low k-values could be caused by many factors (soft or deteriorated subgrade, different underlying pavement or base layers or underneath the UTW, deteriorating support layers such as old concrete pavement), and appear to match up with the observed distresses on the right hand side of the northbound lane. At Sites 3 and 4 (southbound), there was variation in k-value across the different rows of panels, but the differences in magnitude were not as dramatic (similar to variation in h_{eff}). No edge distresses were observed in the southbound lane.

In addition to the pre-construction cores, cores of the UTW pavement in the right lane, center panels were also taken in summer 2014. Listed in Table 4.5 are the layer thicknesses measured from these UTW cores. Even though the nominal thickness of the concrete inlay was 4 inches for the entire project, as seen from the cores in Table 4.5, actual PCC thickness was about 3/4 of an inch greater on average in the southbound lane than in the northbound lane. Average thickness of the asphalt layer was also about 1 inch greater in the southbound lane, and the thickness differences in both layers were significant to a 95% confidence interval. These findings appeared to match up with the higher backcalculated h_{eff} values observed in the southbound lane.

Sample pictures of the UTW cores are provided in Figure 4.10. The core pictured in Figure 4.10(a) shows the PCC inlay bonded to the milled asphalt surface, but there appears to be debonding within the asphalt layer about two inches down, perhaps marking the interface between the separate overlays placed in 1987 and 2000. This debonding may have only resulted from the coring process itself, but it was observed in several other cores as well, and may signal a diminished contribution of the asphalt layer to the structural capacity of the UTW pavement.

In the picture of the core in Figure 4.10(b), the old PCC pavement that was underneath the asphalt layer appears to have completely crumbled, which was seen in several of the cores. This finding indicates that the old PCC layer may be very weak and deteriorated throughout the project, which could possibly lead to a lower backcalculated k-value.

(a) (b)

Figure 4.10. UTW Cores from Route 53 (IDOT photo).

Table 4.5. UTW Core Data

Core Number	Station	PCC Inlay Thickness (inches)	Total Asphalt Thickness (inches)	Old Underlying PCC Thickness (inches)
Northbound (Right Lane, Center Panels)				
2	45+00	3.75	4.50	4.75
4	71+50	3.75	5.75	8.50
6	98+00	3.75	5.25	10.00
8	124+50	3.75	4.00	10.00
10	151+00	4.75	3.75	11.00
12	177+00	4.00	2.50	11.00
14	203+50	3.25	4.25	10.50
16	230+00	4.00	4.25	11.25
18	256+50	3.75	2.00	10.50
Averages (Northbound)		3.86	4.03	9.72
Southbound (Right Lane, Center Panels)				
1	45+00	4.5	6	7.75
3	71+50	4.5	6	7.50
5	98+00	4.5	6.25	8.00
7	124+50	4	5.25	7.25
9	151+00	4	5.75	6.75
11	177+00	4.25	5	0.00
13	203+50	4	6	0.00
15	230+00	4.75	5.25	0.00
17	256+50	6.5	—	8.00
Averages (Southbound)		4.56	5.06	5.03

There is little evidence from the FWD testing or UTW coring that debonding between the concrete inlay and existing asphalt occurred in any of the sections that were tested (i.e. slabs where a dip in h_{eff} might indicate that only the PCC layer is contributing to the UTW structure, or cores where there was debonding at the concrete–asphalt interface). However, while performing the distress survey on Route 53, sounding of the pavement, especially the outer panels, did suggest that debonding was fairly prevalent on the project. It is possible that debonding was not as widespread as thought from sounding and/or contact friction may be present. It is also possible that the effective thickness backcalculation method is not as sensitive to debonding as assumed.

One phenomenon that has appeared in h_{eff} data in the past are large spikes that create significant variation between slabs. Two examples are provided in Figures 4.11 and 4.12. In Figure 4.11, there is one such spike at slab number 7. In Figure 4.12, these spikes occur at slabs 2, 6, and 10. As discussed previously (Chapter 3), the presence of an uncracked transverse joint can cause an over-estimation of h_{eff} at the next slab. Local drops in transverse joint LTE (as discussed in section 3.1) may indicate uncracked joints. In Figure 5, the spike in h_{eff} does appear to correspond to a drop in LTE at the preceding transverse joint. However, there are no such drops in LTE to explain the spikes in h_{eff} in Figure 6.

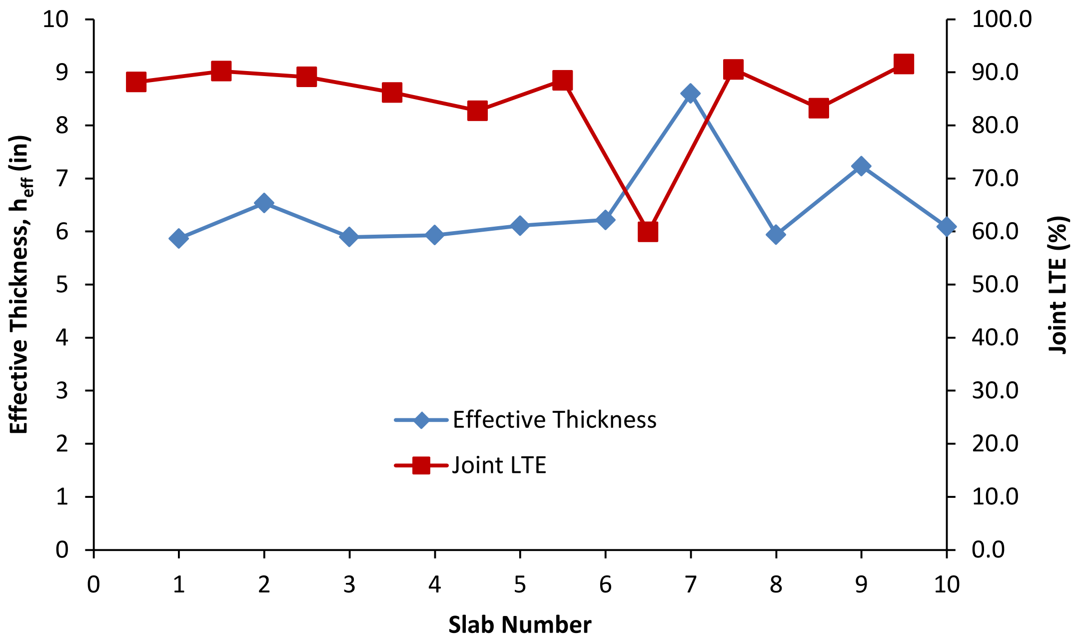

Figure 4.11. Comparison of local transverse joint LTE and effective slab thickness, periodic testing, Southbound, Mile 3.

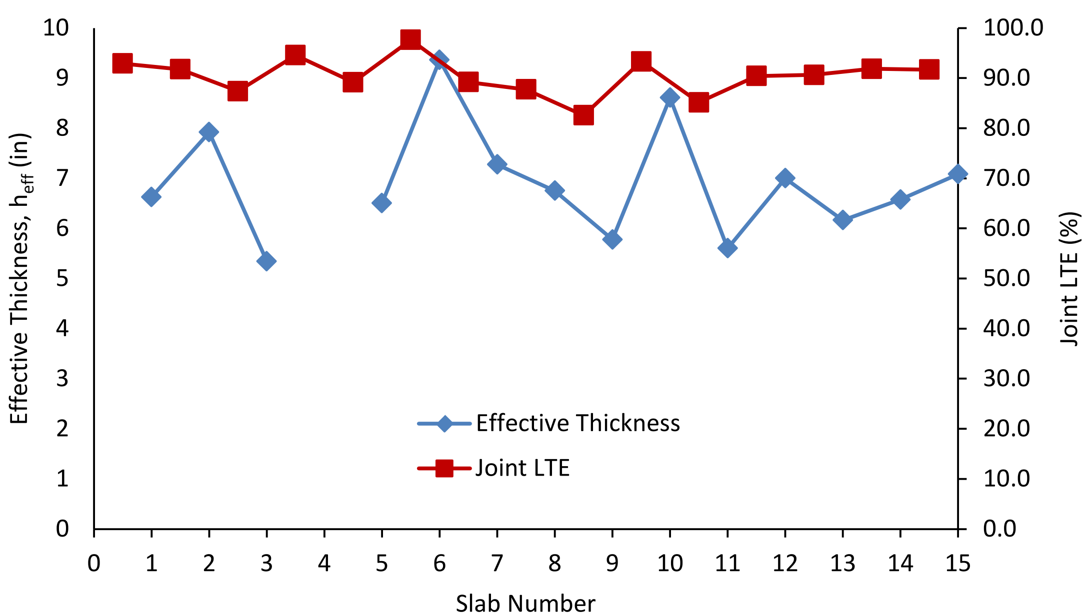

Figure 4.12. Comparison of local transverse joint LTE and effective slab thickness, intensive testing, Site 2, Left Lane, Center Panels

4.3 CONCLUSIONS AND LESSONS LEARNED

Overall, distresses were more common (Table 4.3) and significantly more severe in the northbound lane than in the southbound lane. These findings would suggest that some kind of change in the pavement structure, base or subbase layers, or traffic loads between the northbound and southbound lanes is causing the disparity in distress.

There is definitely a difference between the structures of the northbound and southbound pavements. Effective thickness backcalculation and coring of the UTW pavement confirmed that the cross-section of the southbound lane is thicker (about 3/4 inch in the concrete layer and 1 inch in the asphalt layer) than that of the northbound lane. The lower total thickness of the northbound lane may not be sufficient for the level of traffic at the project, especially given the high amount of trucks. Debonding within the asphalt layer, which would leave a diminished thickness of asphalt bonded to the concrete as a part of the UTW structure, may also be causing thickness-related distresses.

There may also be variation in the underlying pavement, base layers, or subgrade toward the right edge of the northbound lane, as suggested by the drop in backcalculated k-values in that area, which could be contributing to the distresses observed at the lane-shoulder joint. However, without cores taken in these locations along the northbound lane, it is not possible to confirm whether the cross-section differs in these areas, and whether it should have been accounted for during design. The low k-values may also be the sign of a deteriorating subgrade, which could have been caused by the drainage issues along the right pavement edge noted during the surveys.

While a too-thin pavement or weak subgrade may be major causes of the distresses on Route 53, an important factor that should not be overlooked is the role of traffic loads. With the rapid growth of the nearby intermodal facilities, it may not have been possible to anticipate the extent of the corresponding increase in truck traffic at the time the pavement was designed. It is also possible that there is a disparity in truck loads between the northbound and southbound lanes, as trucks heading northbound may tend to be full of cargo whereas trucks heading southbound may be more likely to be empty.

Ultimately all of the reasons listed above may be contributing to the early-age distresses at Route 53. The most significant takeaways from analysis of this project to prevent similar problems from occurring on other UTW projects in the future include:

- Characterize the existing pavement structure by coring or non-destructive testing to identify issues with the underlying pavement, base, or subgrade layers that may require further work or need to be accounted for during design. Specific things to look for include debonding between pavement layers, variation in layer thicknesses, and loss of subgrade support.
- Take care to have the best data possible available for designing concrete thickness on projects with heavy truck traffic. In these situations, small changes to or deviations from the design thickness appear to have a significant impact on UTW performance.
- Roadway cross slope and shoulder condition are important for promoting lateral surface drainage and avoiding significant amount of water infiltration in the joints.

CHAPTER 5 CONCLUSIONS AND RECOMMENDATIONS

The findings of the visual distress surveys and the FWD data analysis, as well as the case study at Route 53, were examined to help provide a greater understanding of factors that affect UTW performance. From this analysis, the following conclusions and recommendations are made regarding UTW pavement design and construction.

5.1 SLAB THICKNESS

While the structural design (thickness) of the concrete inlay or overlay for most of the UTW projects appeared to be sufficient, there is a strong likelihood that insufficient overlay thickness was a contributing factor to the severe distresses observed after just two years on Illinois Route 53 in Will County. However, an underestimation of the truck traffic may have been an additional reason the design experienced premature failures. With the lack of premature failures or distresses on other projects, there does not appear to be any immediate need to adjust the thickness design chart for UTW in Chapter 53 of the *Bureau of Design and Environment (BDE) Manual* at this time, but care should be taken to properly account for projected truck traffic when designing concrete thickness. A review of UTW structural design and accompanying design features by IDOT BMPR may reduce the risk of under designing a section.

5.2 PANEL SIZE

There were obvious distresses associated with short panel sizes, with joints falling in the wheel path, as found in the Decatur and Tuscola projects. When panels were too large, they were prone to longitudinal cracking, like on Marion Street, or faulting, which was seen in the 11 foot panel section at Bible Grove Road but not in the 5.5 foot panel section. Joint LTE values collected from FWD test data from Bible Grove Road supported these findings. Faulting was also observed to be more severe in the 11 foot panel section than in the 5.5 foot panel section of Piatt County Highway 4.

To avoid distresses associated with too-small or too-large panels, 5.5 to 6 foot slab sizes should be maintained on UTW roadways. Panel sizes of 4 to 6 feet appear to be working fine for UTW parking lots.

5.3 MACRO-FIBERS

On the basis of the data that were collected, macro-fibers proved very effective in providing extra structural capacity and maintaining joint load transfer efficiency in UTW pavements as assumed in the thickness design procedure. Several projects that used macro-fibers and had higher amounts of observed slab cracking, such as the Western Avenue bus pads and Marion Street, were still in good serviceable condition and provided smooth rides.

Chapter 53 of the *BDE Manual* does not currently require macro-fibers in UTW projects with a concrete thickness greater than 4 inches. On the basis of the survey and FWD observations, macro-fibers should be continued in all UTW pavements less than or equal to 4 inches. With the ability of macro-fibers to tie adjacent slab lanes together, provide additional slab capacity, and reduce the rate of crack deterioration, they should be considered for thicker UTW sections also up to 6 inches. Only a minimum dosage is necessary for parking lots with cars, but for roadways a design residual strength R_{150} > 20% is recommended to be continued. For projects with high distress severity in the underlying asphalt or heavy truck traffic, or where there are issues with the underlying support, higher synthetic macro-fiber dosages (up to 7.5 lb/yd^3) are recommended as a way to try to maintain continuity between adjacent slabs and prevent premature cracks from deteriorating rapidly.

Crimped steel fibers added to the thin, unbonded Marion Street overlay performed favorably despite the superficial fiber pop-out distresses. Overall, the findings of the UTW surveys did not produce data that would suggest discontinuing the use of synthetic macro-fibers. The only factor related to synthetic macro-fibers is making sure they are added at the batch plant and mixed with sufficient shearing of the mix. Projects where the macro-fibers were added directly in the ready-mixed concrete trucks without sufficient time to disperse tended to produce fiber balls.

5.4 SKEWED JOINTS

Skewed joints, especially when combined with large panel sizes (> 6 feet), such as at Sailor Springs Road and Piatt County Highway 4 with slab sizes of 11 by 11 feet or larger, leave the pavement susceptible to faulting and acute corner and longitudinal distress. The option to use skewed joints on all concrete pavement should be discontinued immediately.

5.5 SAW-CUTTING

Wide-cut joints in UTW pavement appeared to be susceptible to higher rates of joint deterioration through excessive moisture and debris intrusion. These issues were exacerbated when joints fell in the wheel path, as in Tuscola, or where there were foundation support issues, like at Schank Avenue. Thin, single-entry saw blades (< 3 mm) should be used to cut joints in UTW pavements.

5.6 DRAINAGE

Surface drainage of the roadway is especially important for UTW pavements. With the larger number of joints, the cross slope should be at least the minimum recommended for the functional class in order to keep surface water out of the joints which can deteriorate the asphalt–concrete bond under repeated loading.

5.7 FUTURE UTW FIELD EVALUATION

In addition to these recommendations for design and construction, there are also potential improvements that can be made in order to better evaluate UTW projects. There were a few instances of local reductions in h_{eff} in the data, which could mean a deterioration in the bond interface, but it is not possible without additional testing to confirm whether the deflection basin changes were because of interface debonding or just variation in the asphalt stiffness or concrete/asphalt thickness. Other non-destructive testing techniques could prove useful in supplementing effective thickness to analyze UTW bond conditions and variation of actual concrete and asphalt layer thicknesses, and joint cracking.

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APPENDIX A DISTRESS SURVEY SUPPLEMENTS

A.1 DISTRESS SURVEY GUIDELINES

Longitudinal Cracking: cracking primarily in the direction of traffic (pavement centerline)

Low Severity: crack widths < 1/8 inch

Medium Severity: crack widths between 1/8 to 1/2 inch and/or spalling < 3 inches

High Severity: crack widths > 1/2 inch and/or spalling > 3 inches

Transverse Cracking: cracking across the slab perpendicular to the direction of traffic

(Same severity guidelines as longitudinal)

Corner Breaking: cracking between adjacent edges of the slab with 1/2 of the length of the edges (if greater, classify as longitudinal or transverse cracking)

Low Severity: hairline crack with spalling < 10% of the total length of the crack

Medium Severity: spalling > 10% of the total length of the crack, corner piece is still intact, faulting < 1/2 inch

High Severity: spalling > 10% of the total length of the crack, corner piece broken into more than one section, faulting > 1/2 inch

Debonding:

Low Severity: audible detection of debonding within 6 inches of a single corner of the slab

Medium Severity: audible detection of debonding on a full edge or in the center of the slab

High Severity: total panel debonding

Joint Spalling:

Low Severity: < 3 inches wide

Medium Severity: between 3 and 6 inches wide

High Severity: > 3 inches wide

Slab Migration: shifting of transverse joints relative to joints in adjacent section:

Low Severity: < 1/2 inch

Medium Severity: between 1/2 inch and 1 1/2 inches

High Severity: > 1 1/2 inches

Faulting (measure)

Scaling (note)

Pop-Outs (note)

Patched Panels (note)

A.2 SELECTED CONCRETE MIX DESIGNS

Table A1. UTW Project Mix Designs (Roesler and Bordelon 2008)

Location		Schank Avenue	Cumberland County Highway 2	Piatt County Highway 4	Sailor Springs Road
Coarse Aggregate	lb/yd^3	1972	1836	1957	1814
	Type	020CAM11	022CMM11	022CAM07	022CMM11
Fine Aggregate	lb/yd^3	1001	1256	1220	1286
	Type	027FAM02	027FAM01	027FAM01	027FAM01
Cement (Type I)	lb/yd^3	515	575	534	534
Water	lb/yd^3	267	197	179	244
Fly Ash (Class C)	lb/yd^3	140	0	0	0
Synthetic Fibers	lb/yd^3	4	0	0	0
Air Entrainment	Type	Daravair 1400	Daravair 1400	Daravair 1400	Daravair 1400
Water Reducer	Type	WRDA 82	–	Daracem 65	WRDA 82
Retarder	Type	–	Daratard 17	–	–
w/c	Wt ratio	0.41	0.34	0.34	0.46
Coarse/fine	Wt ratio	1.97	1.46	1.60	1.41
% aggregate	Wt ratio	76.3	80.0	81.7	79.9

APPENDIX B DEFLECTION AND LOAD TRANSFER EFFICIENCY PLOTS

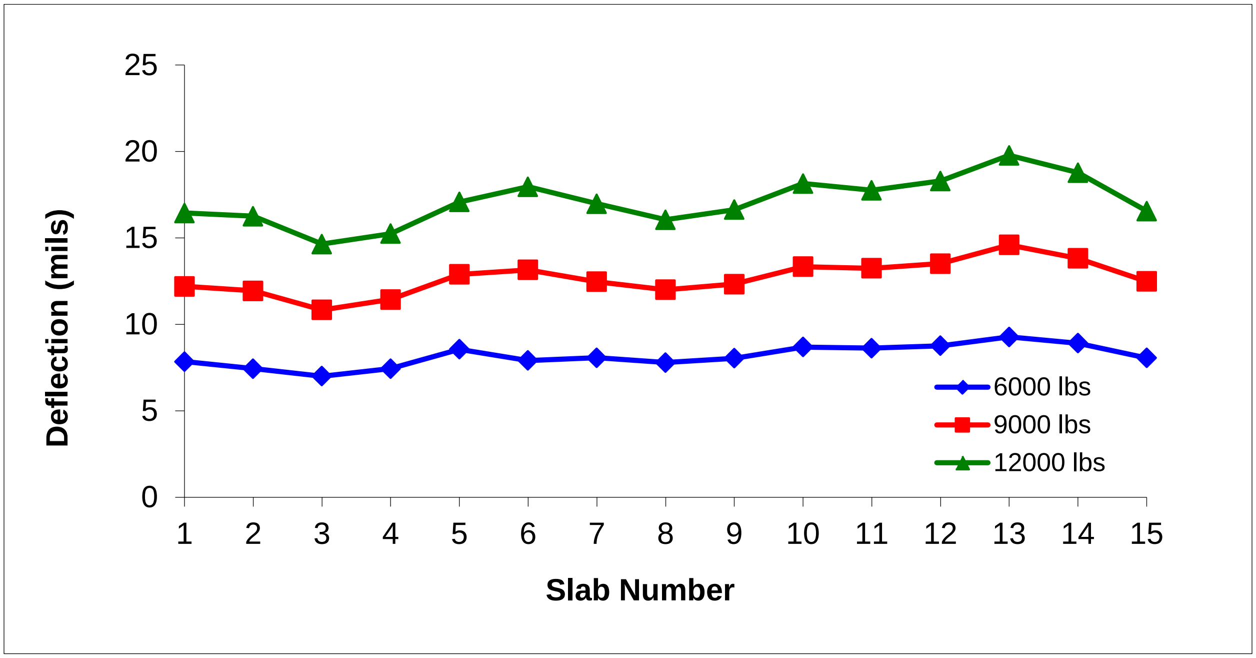

Figure B1. Normalized deflections (center slab), northeast section, McKinley lot, 2008.

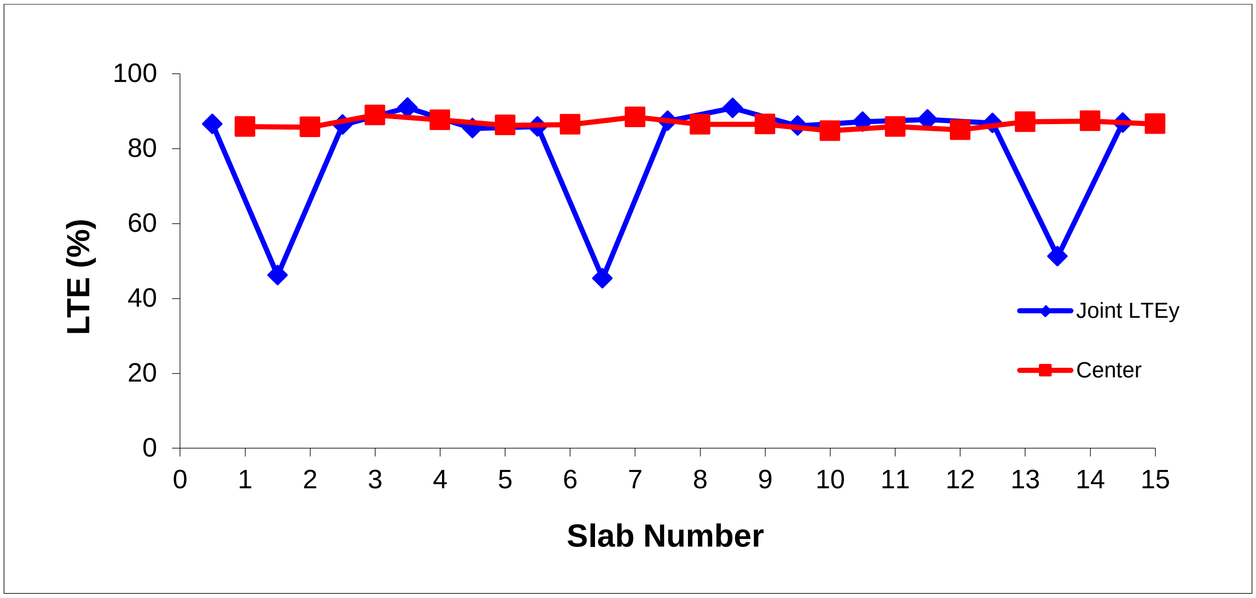

Figure B2. Transverse joint and center slab load transfer efficiency, northeast section, McKinley lot, 2008.

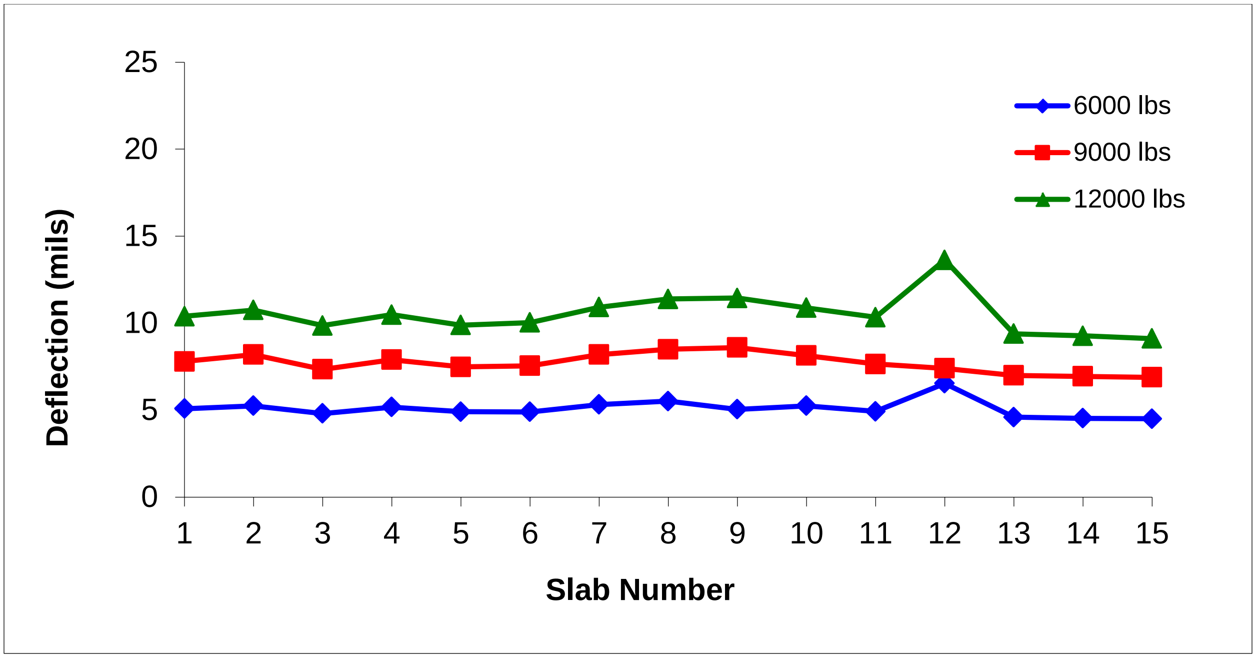

Figure B3. Normalized deflections (center slab), northwest section, McKinley lot, 2008.

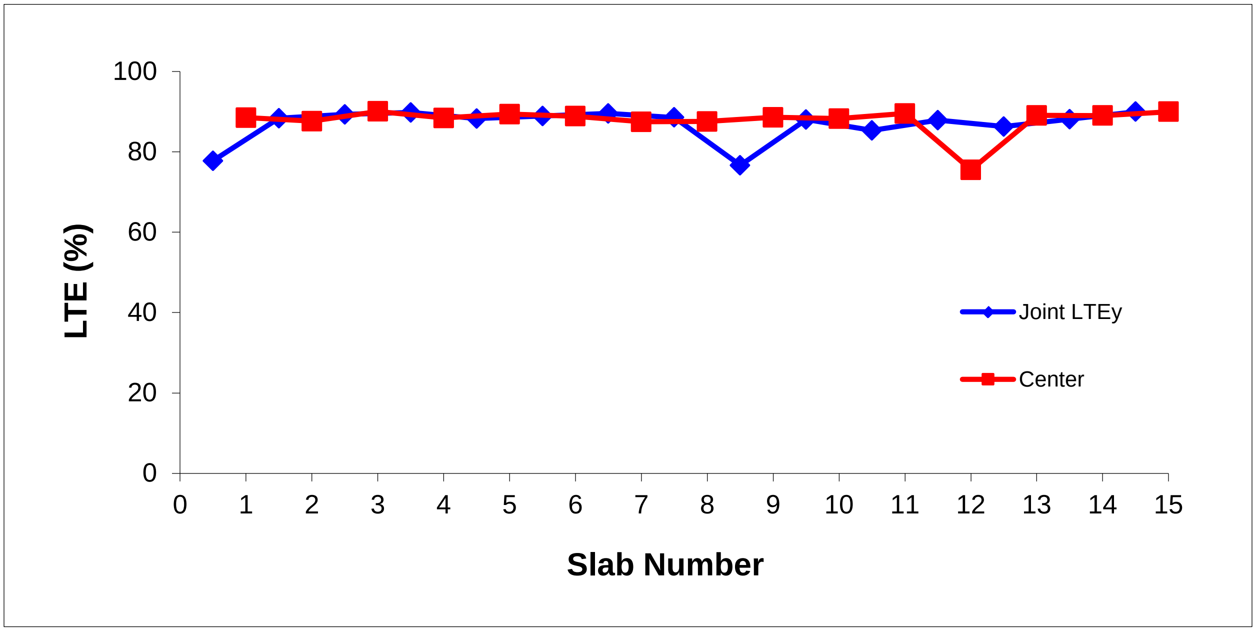

Figure B4. Transverse joint and center slab load transfer efficiency, northwest section, McKinley lot, 2008.

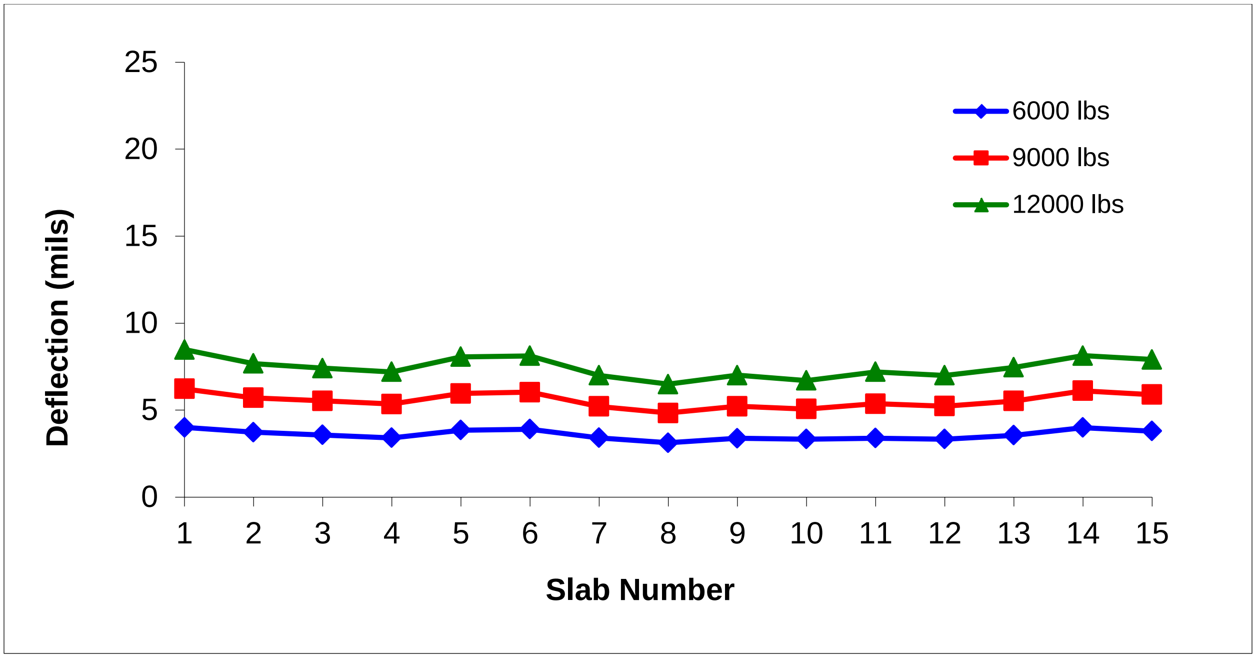

Figure B5. Normalized deflections (center slab), southeast section, McKinley lot, 2008.

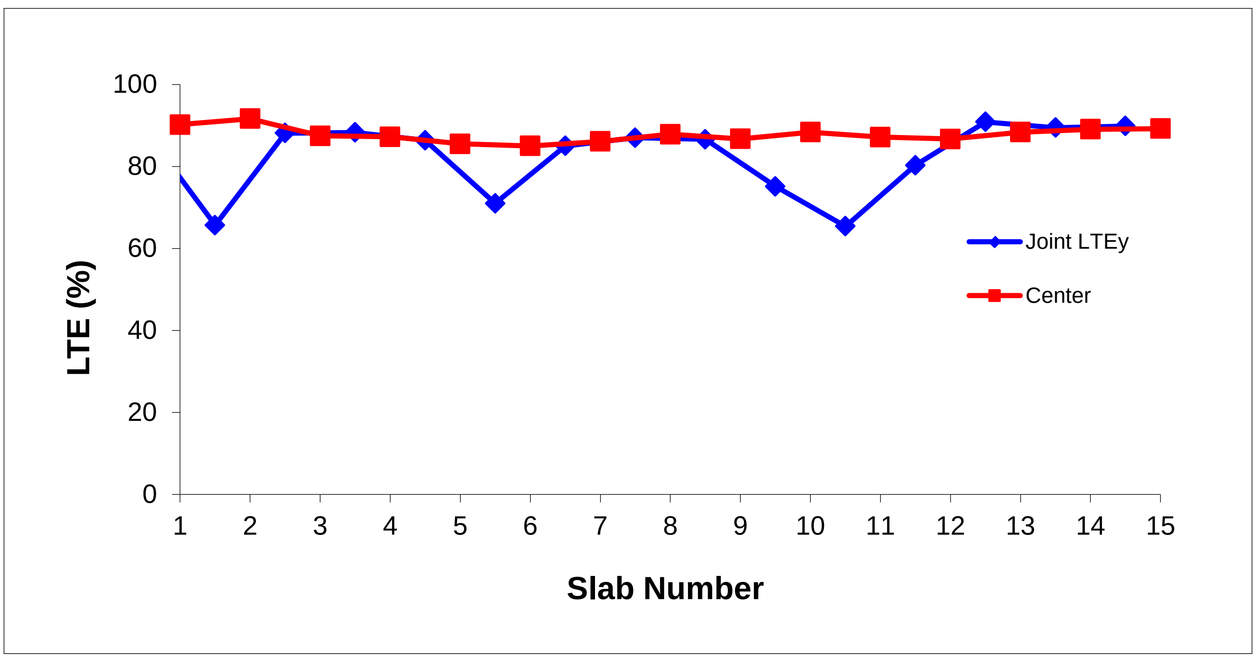

Figure B6. Transverse joint and center slab load transfer efficiency, southeast section, McKinley lot, 2008.

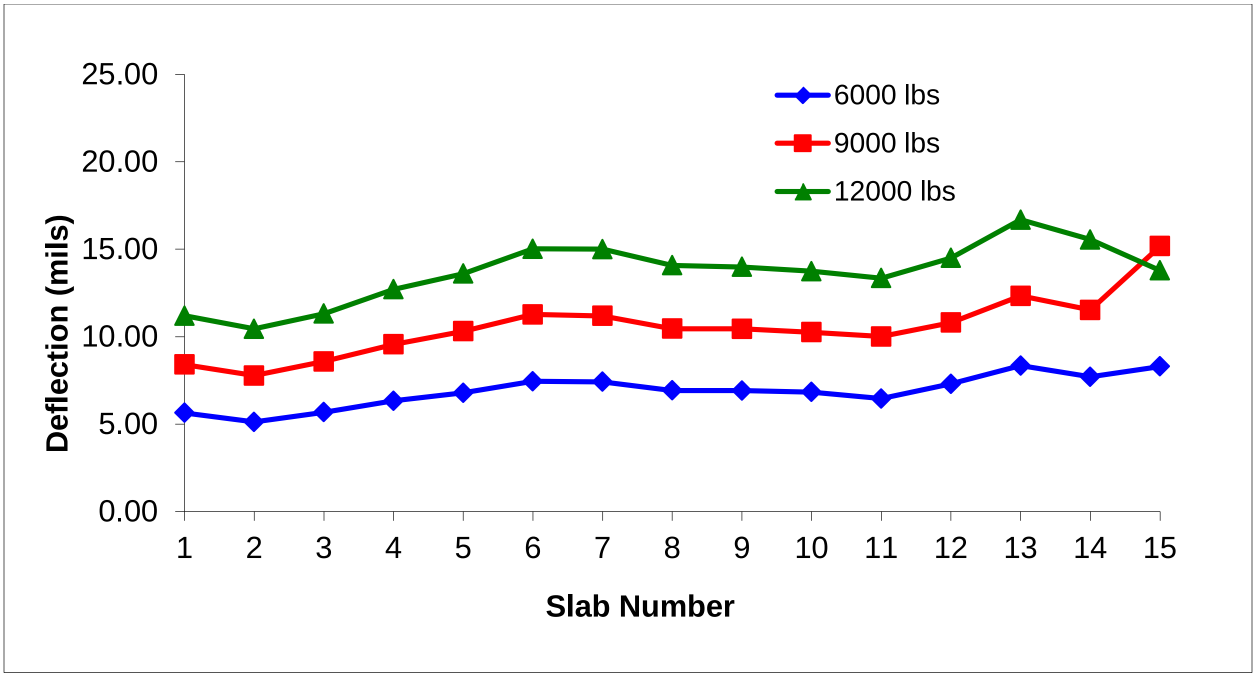

Figure B7. Normalized deflections (center slab), northeast section, McKinley lot, 2012.

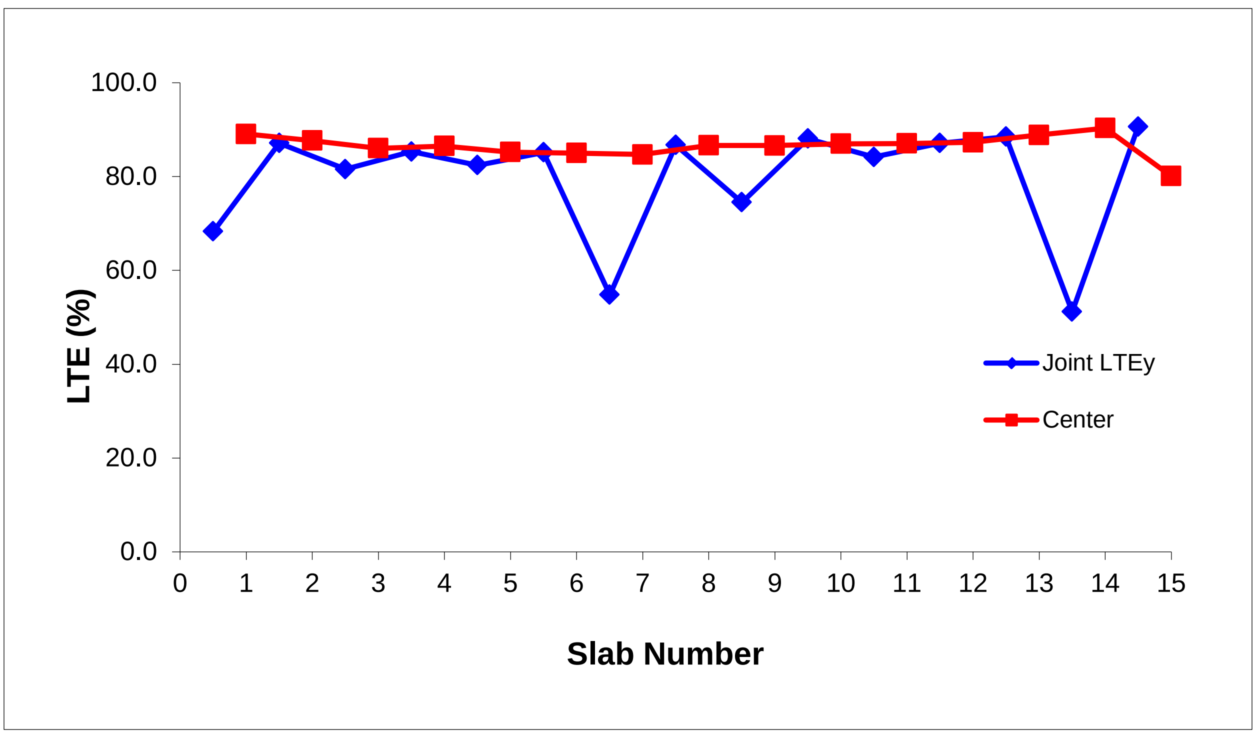

Figure B8. Transverse joint and center slab load transfer efficiency, northeast section, McKinley lot, 2012.

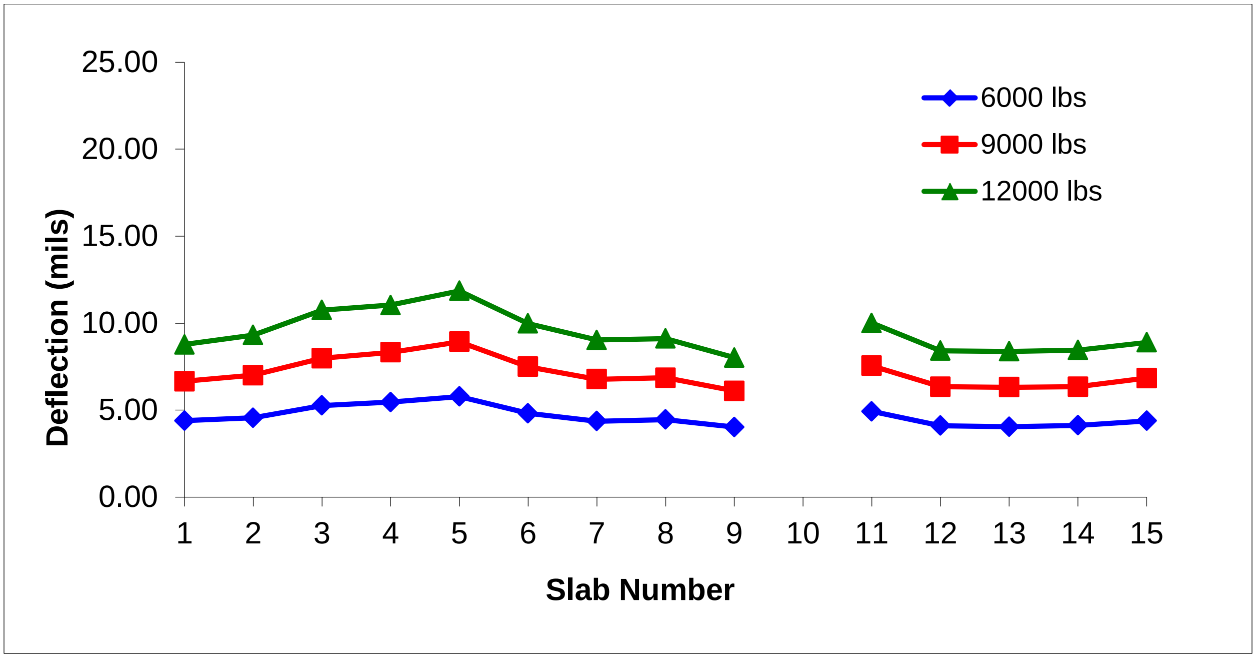

Figure B9. Normalized deflections (center slab), northwest section, McKinley lot, 2012 (no testing was done at Station 10 due to the presence of a manhole cover).

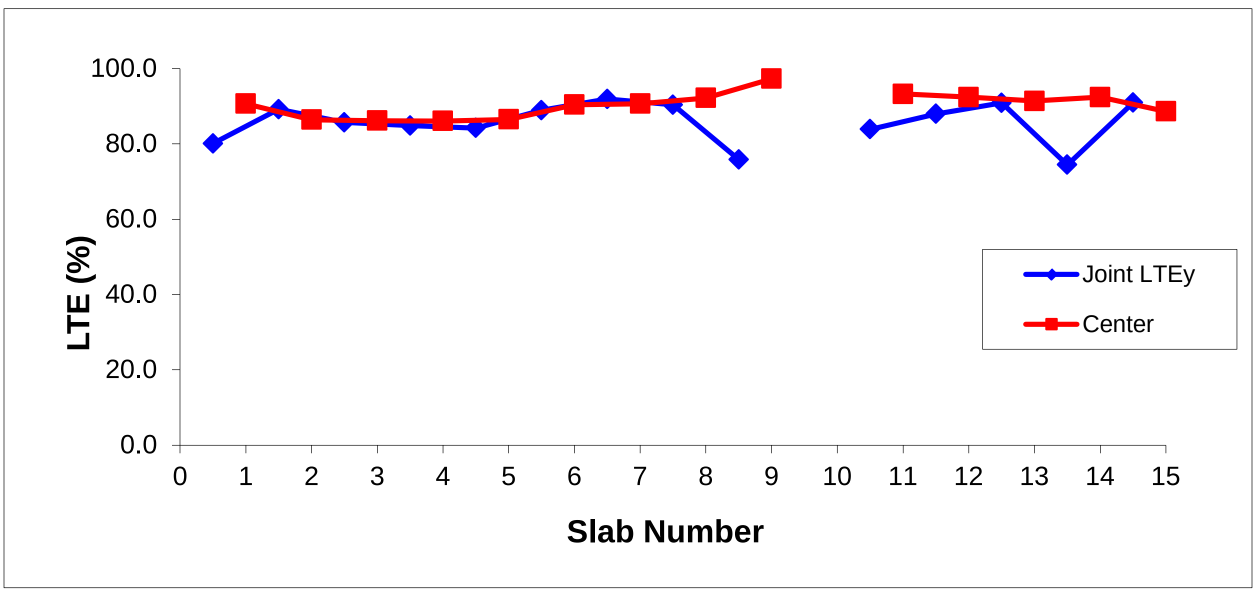

Figure B10. Transverse joint and center slab load transfer efficiency, northwest section, McKinley lot, 2012.

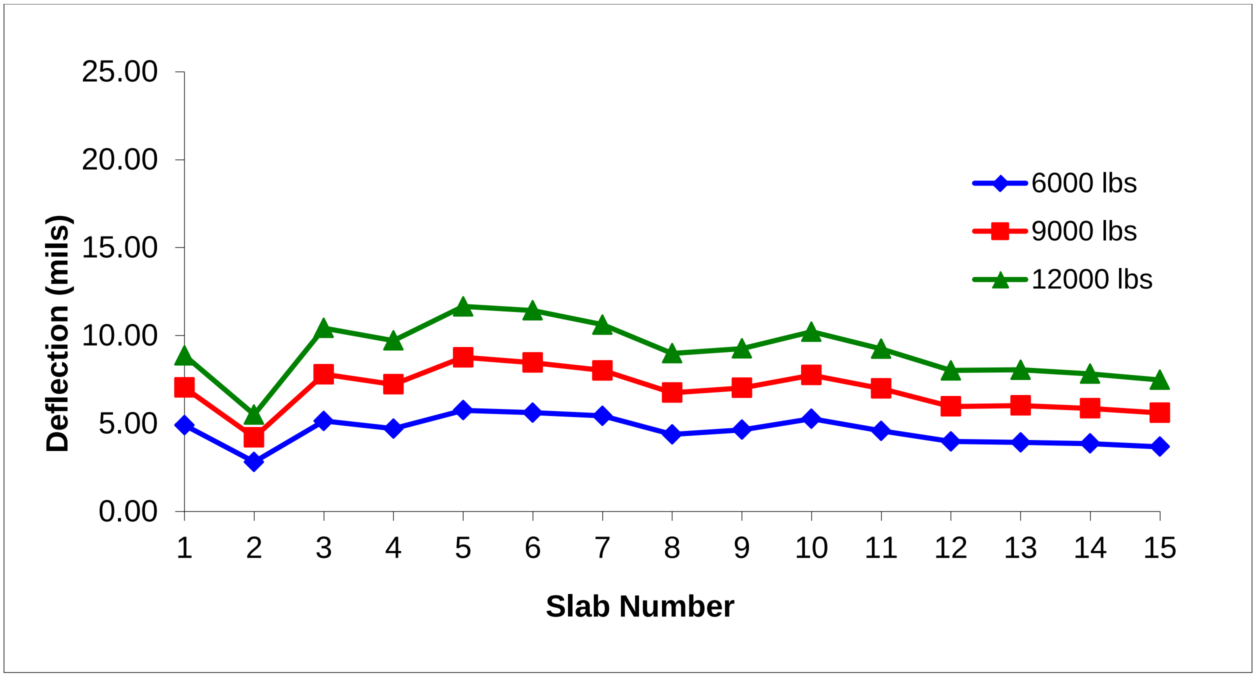

Figure B11. Normalized deflections (center slab), southeast section, McKinley lot, 2012.

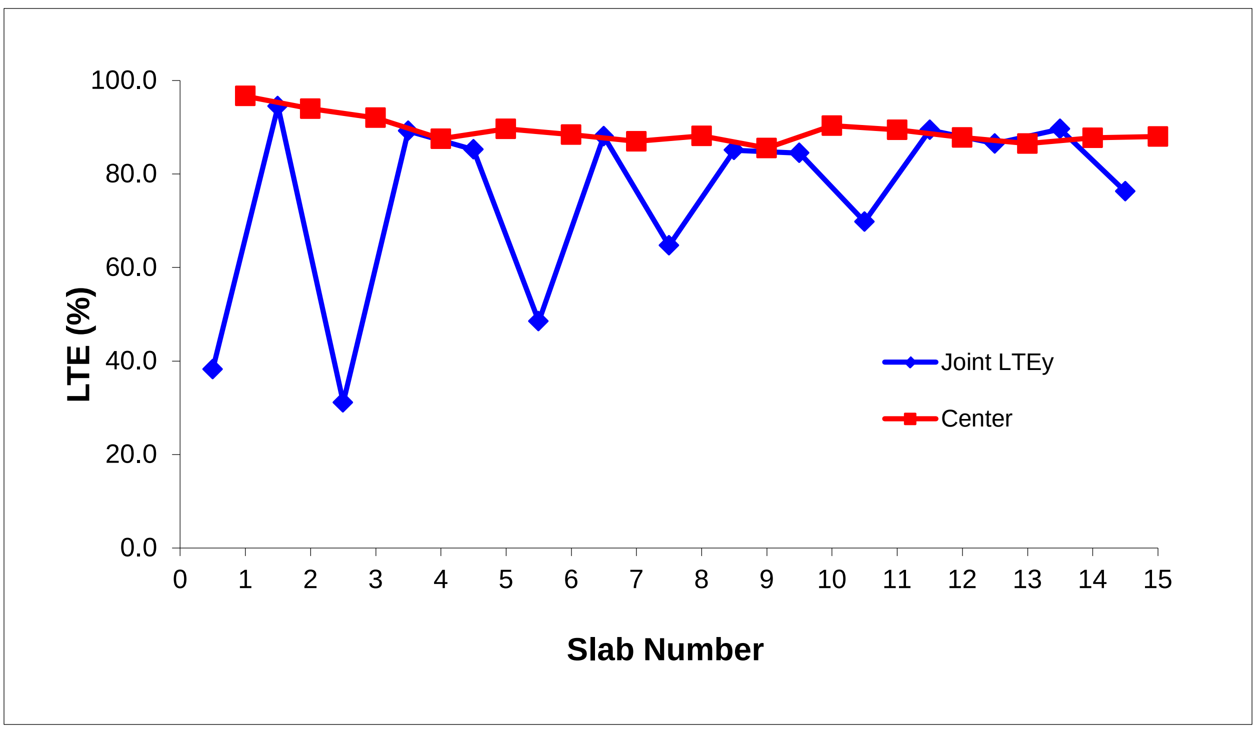

Figure B12. Transverse joint and center slab load transfer efficiency, southeast section, McKinley lot, 2012.

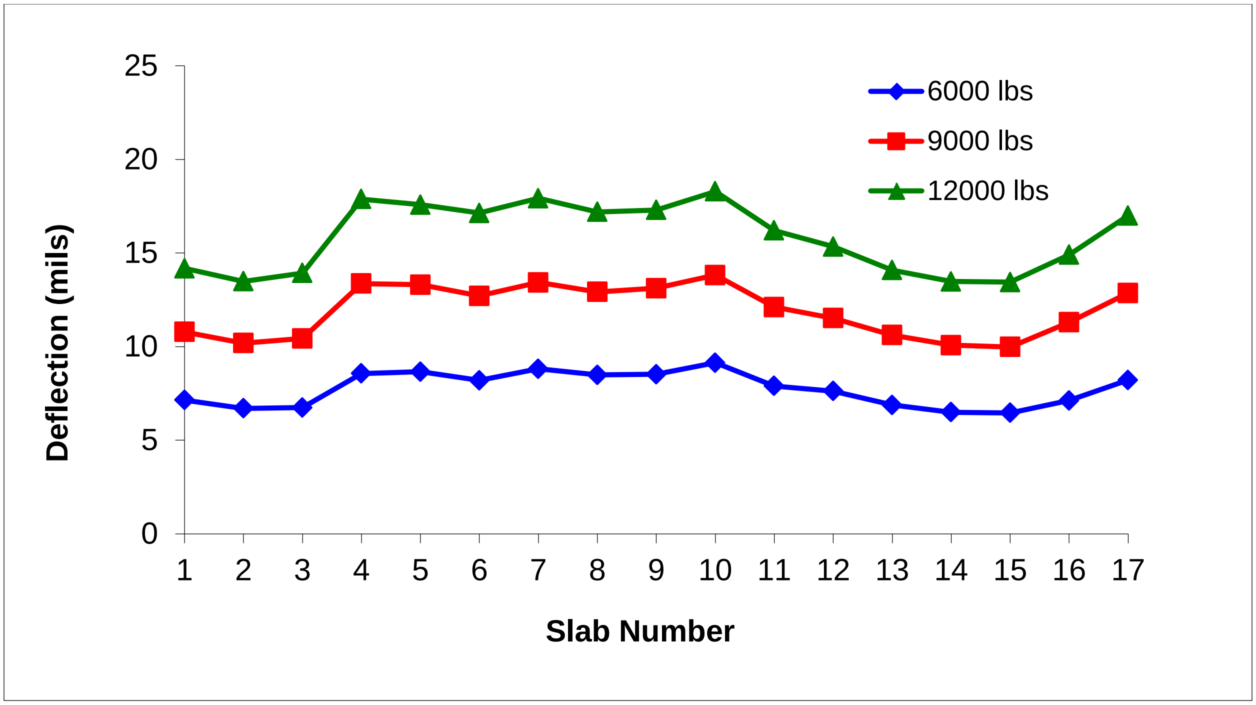

Figure B13. Normalized deflections (center slab), Bay 1, E-15 lot, 2008.

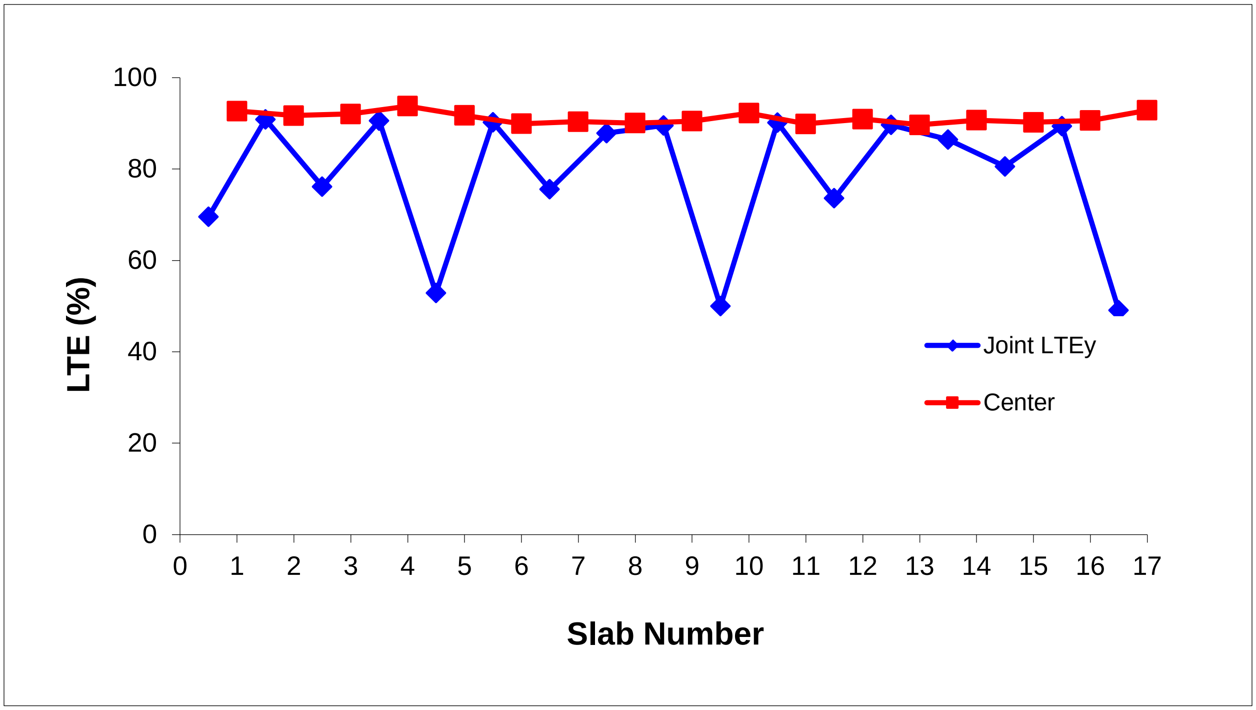

Figure B14. Transverse joint and center slab load transfer efficiency, Bay 1, E-15 lot, 2008.

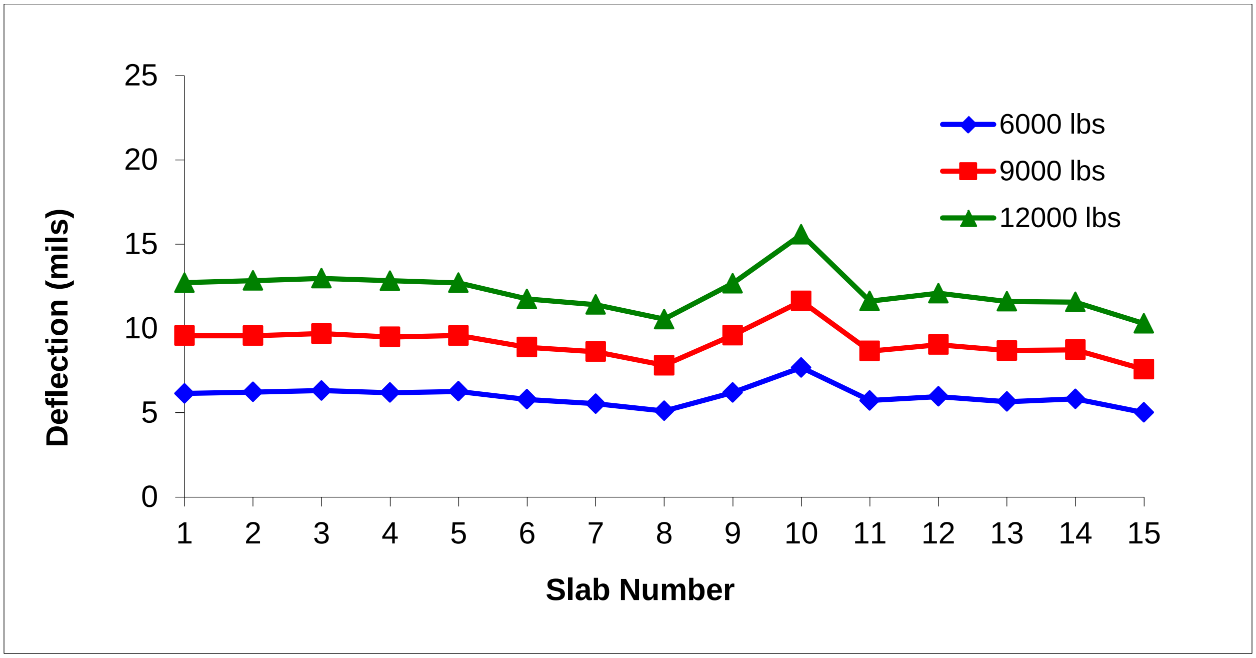

Figure B15. Normalized deflections (center slab), Bay 2, E-15 lot, 2008.

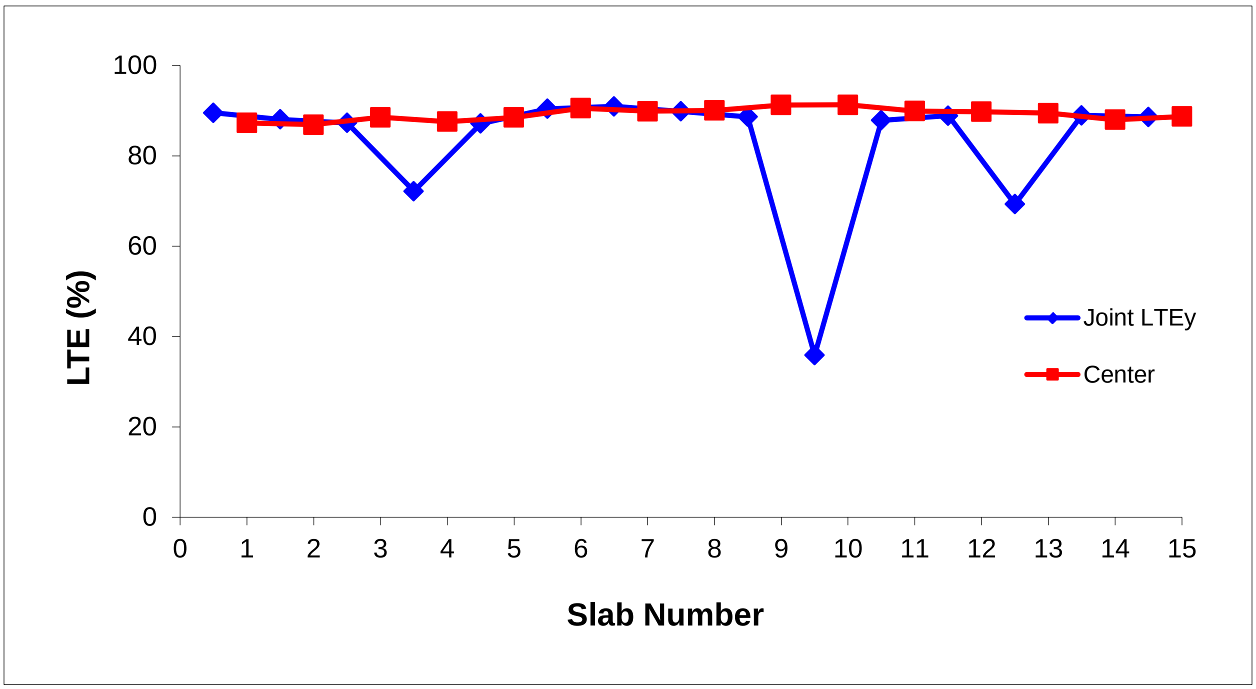

Figure B16. Transverse joint and center slab load transfer efficiency, Bay 2, E-15 lot, 2008.

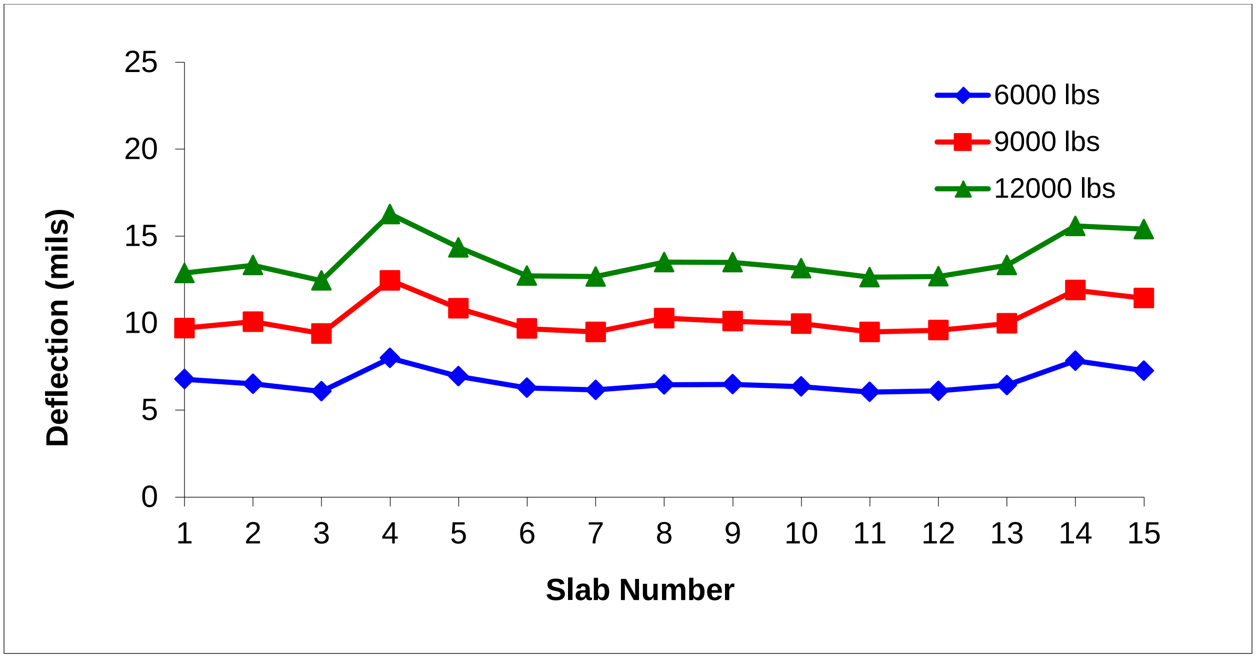

Figure B17. Normalized deflections (center slab), Bay 3, E-15 lot, 2008.

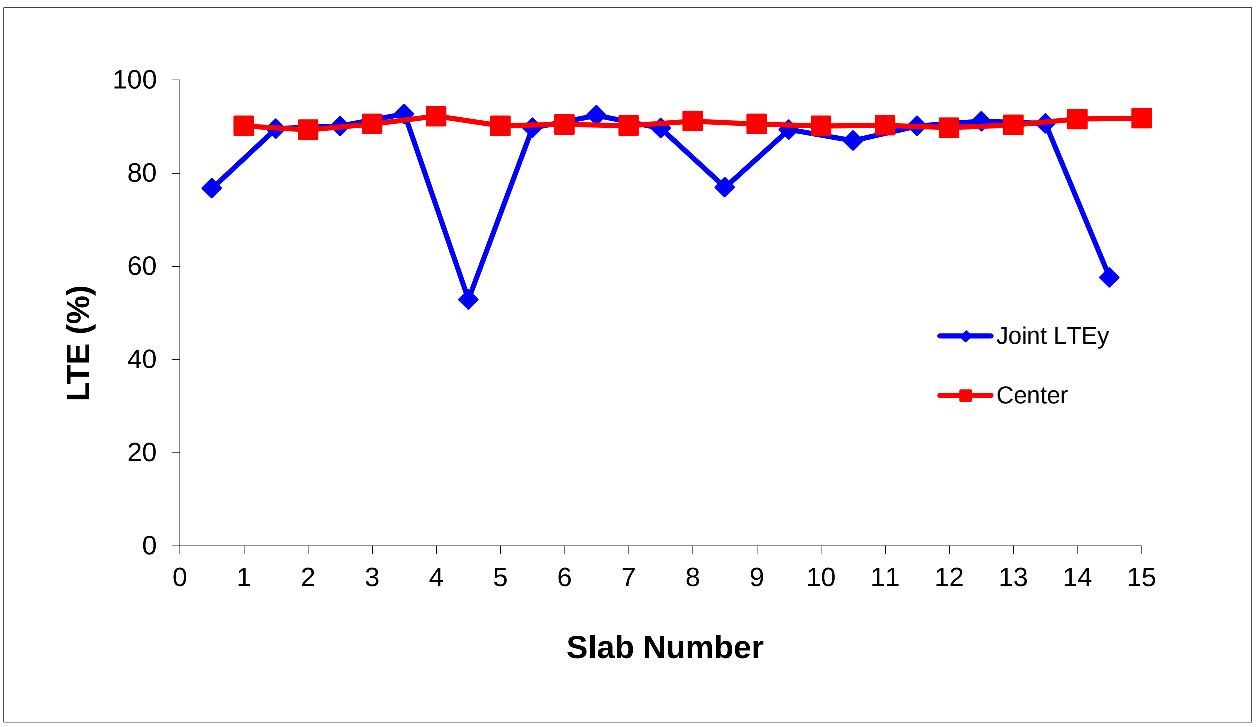

Figure B18. Transverse joint and center slab load transfer efficiency, Bay 3, E-15 lot, 2008.

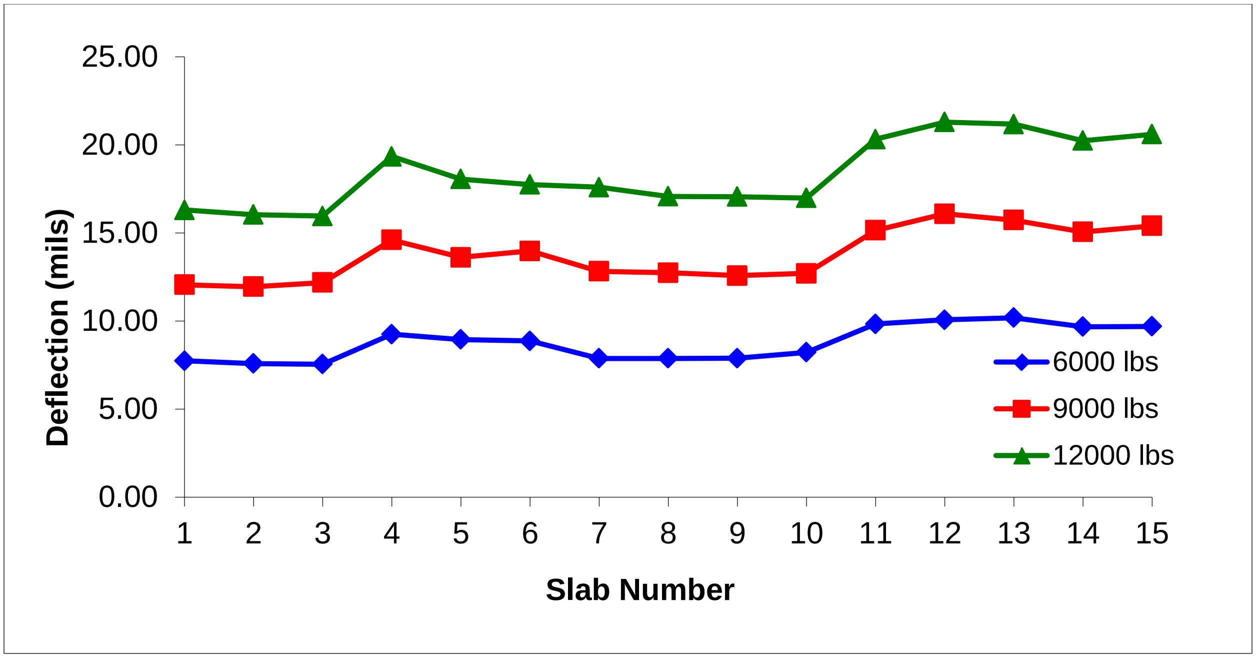

Figure B19. Normalized deflections (center slab), Bay 1, E-15 lot, 2012.

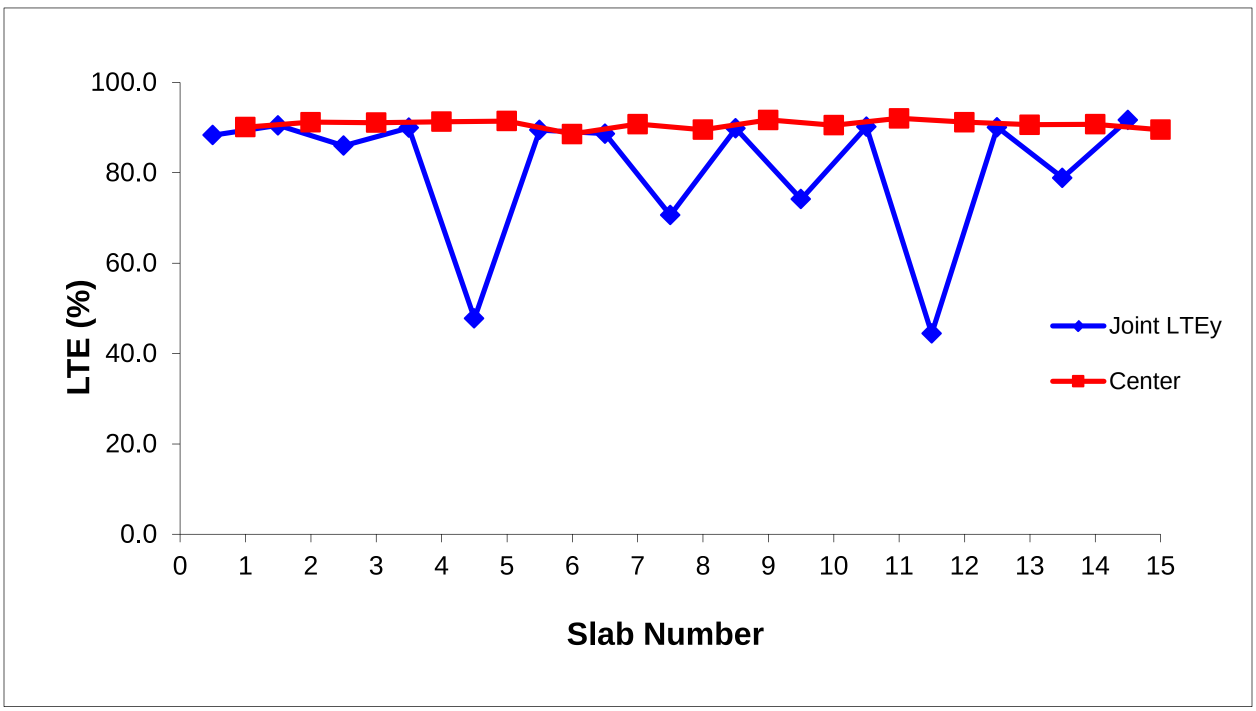

Figure B20. Transverse joint and center slab load transfer efficiency, Bay 1, E-15 lot, 2012.

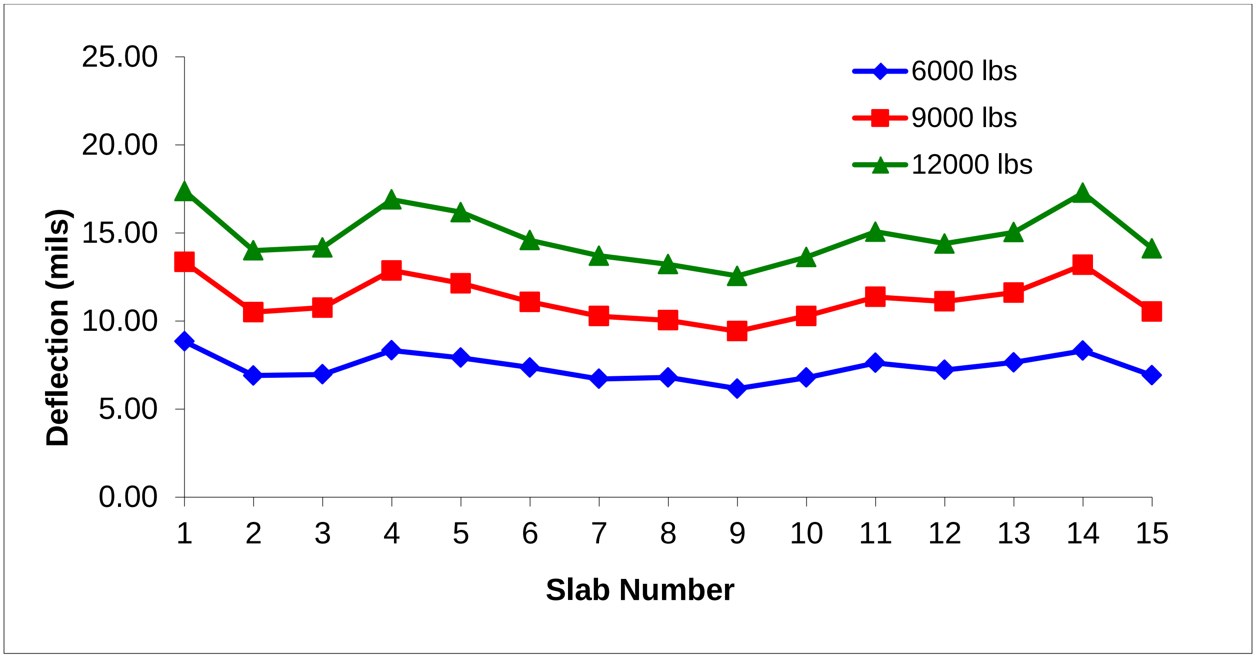

Figure B21. Normalized deflections (center slab), Bay 2, E-15 lot, 2012.

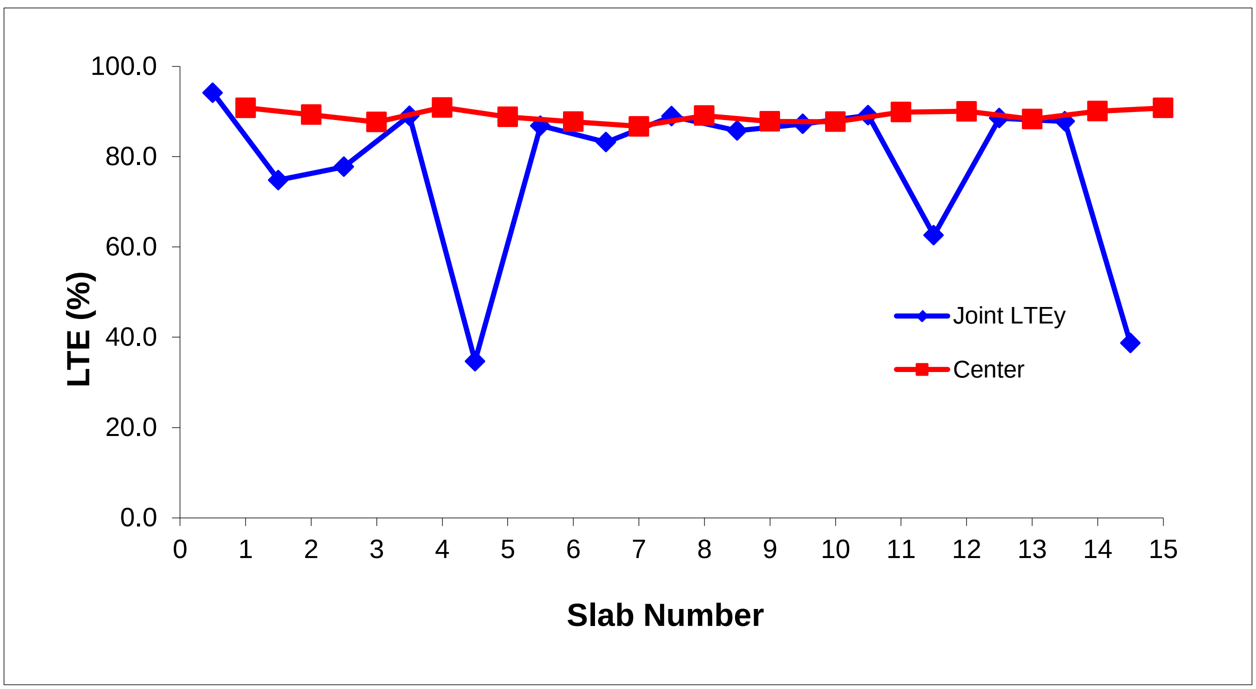

Figure B22. Transverse joint and center slab load transfer efficiency, Bay 2, E-15 lot, 2012.

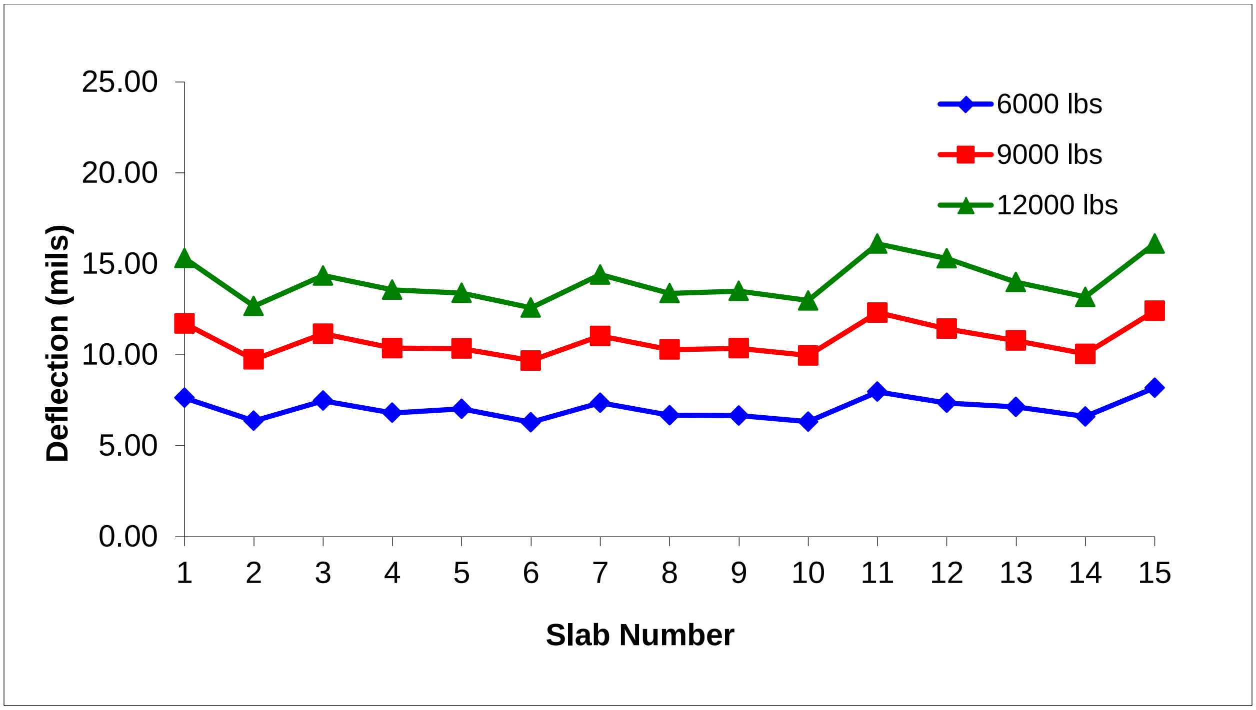

Figure B23. Normalized deflections (center slab), Bay 3, E-15 lot, 2012.

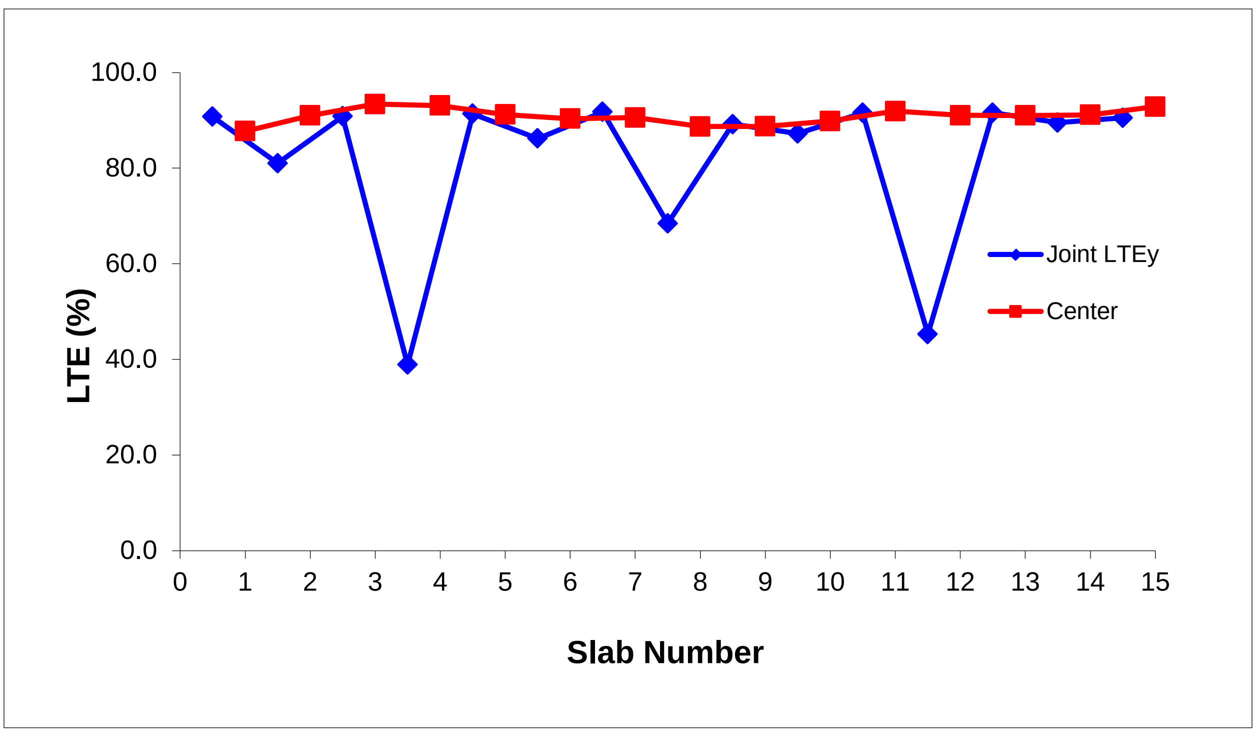

Figure B24. Transverse joint and center slab load transfer efficiency, Bay 3, E-15 lot, 2012.

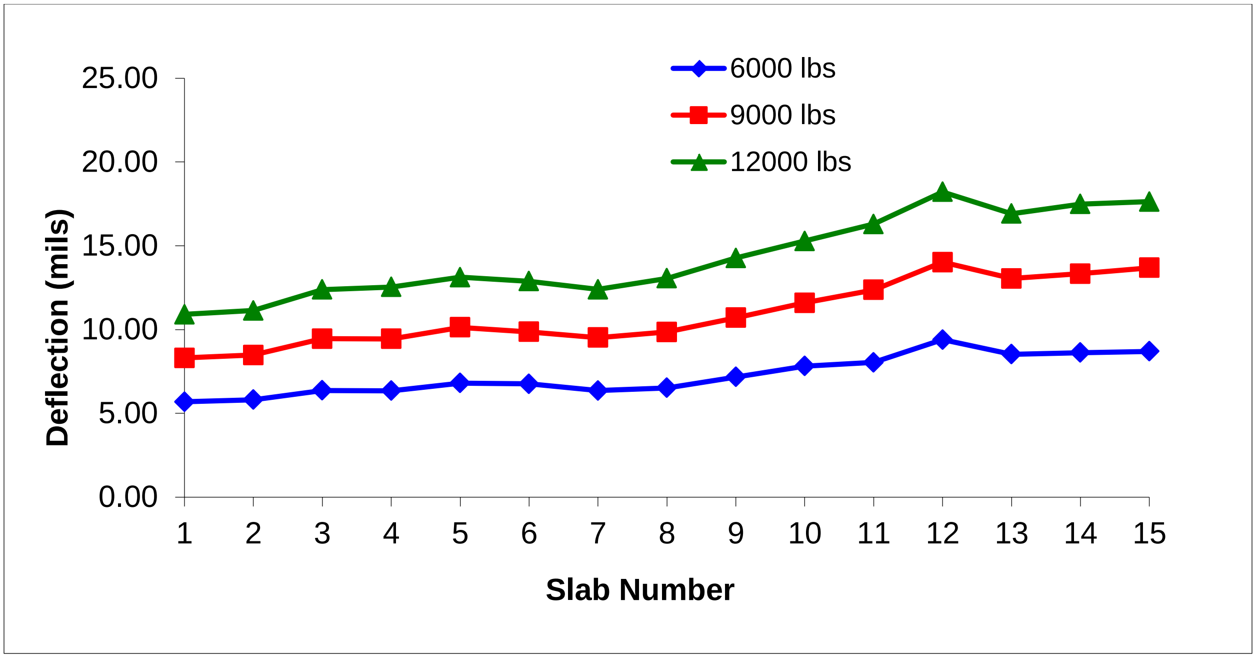

Figure B25. Normalized deflections (center slab), Bay 4, E-15 lot, 2012.

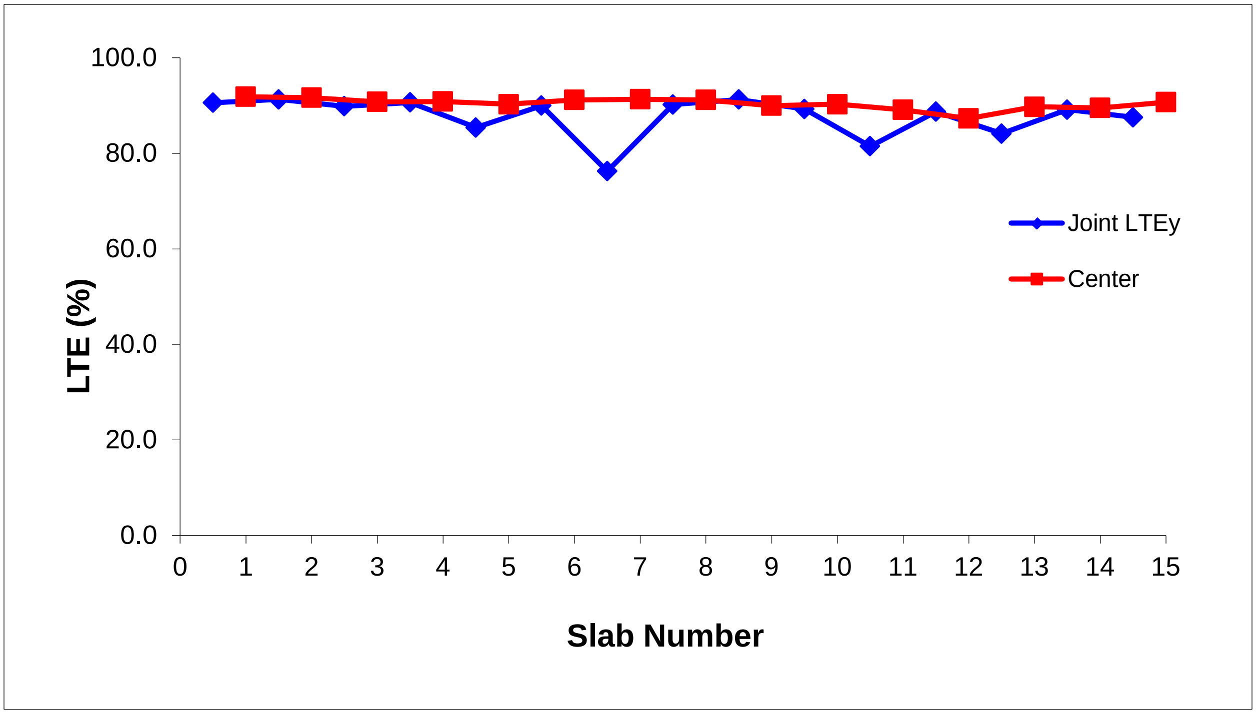

Figure B26. Transverse joint and center slab load transfer efficiency, Bay 4, E-15 lot, 2012.

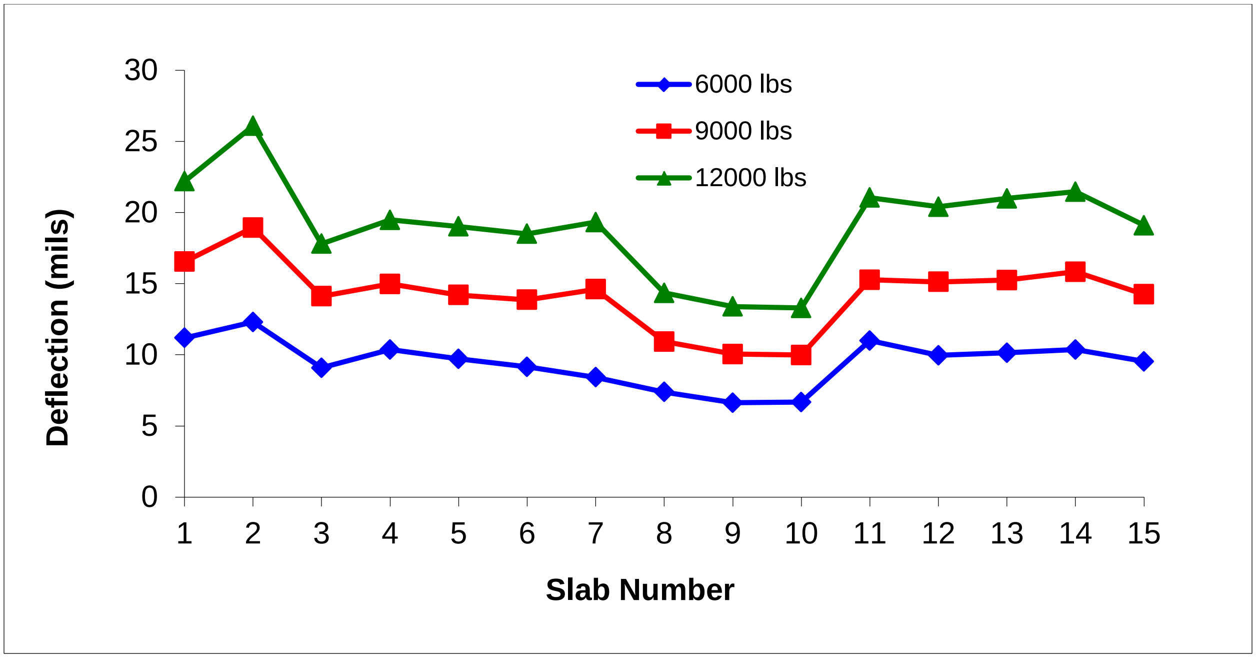

Figure B27. Normalized deflections (center slab), Talbot lot, 2008.

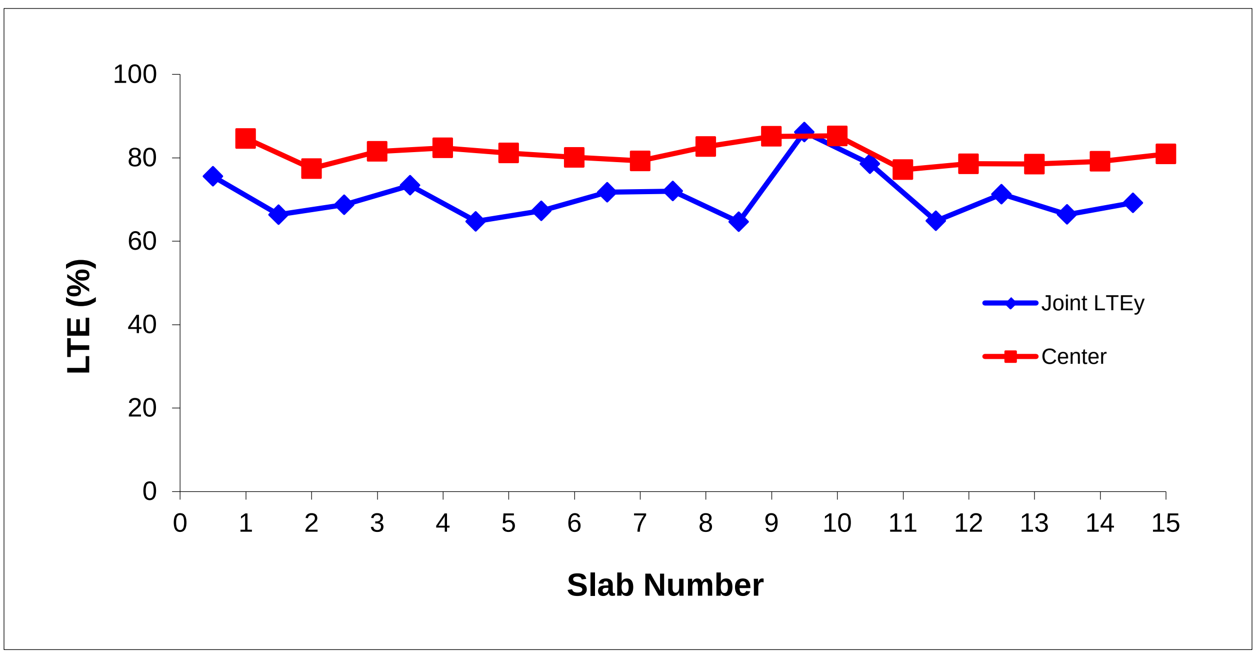

Figure B28. Transverse joint and center slab load transfer efficiency, Talbot lot, 2008.

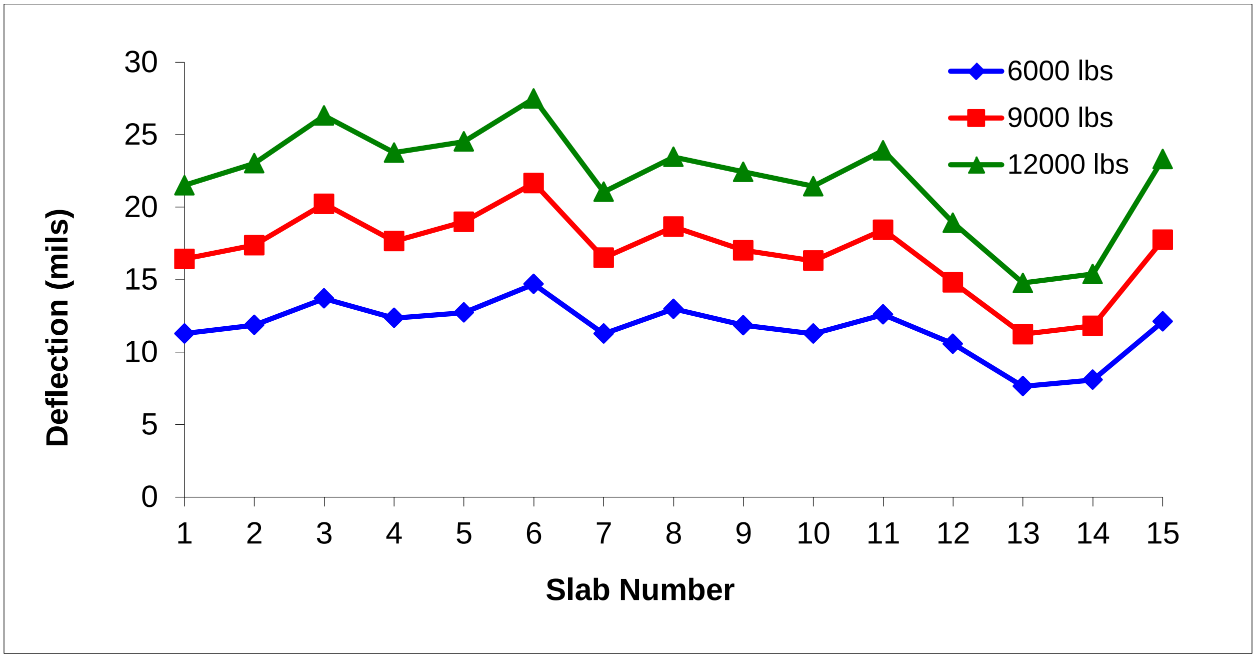

Figure B29. Normalized deflections (center slab), Talbot lot, 2012.

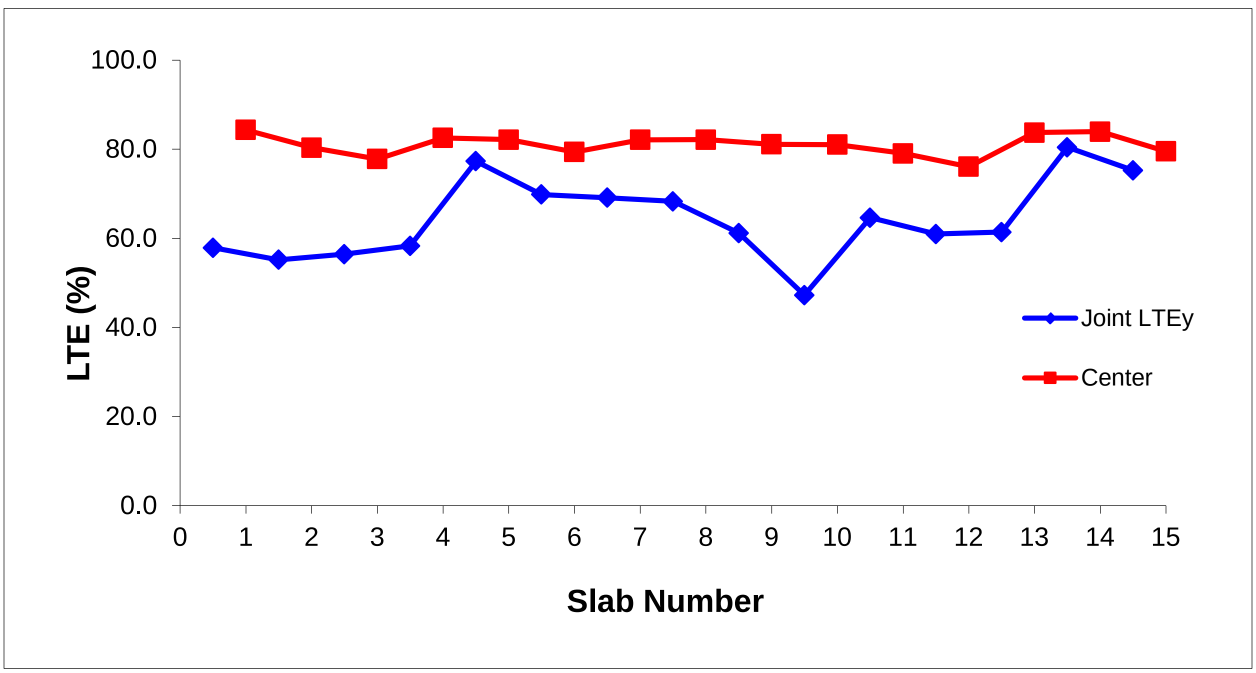

Figure B30. Transverse joint and center slab load transfer efficiency, Talbot lot, 2012.

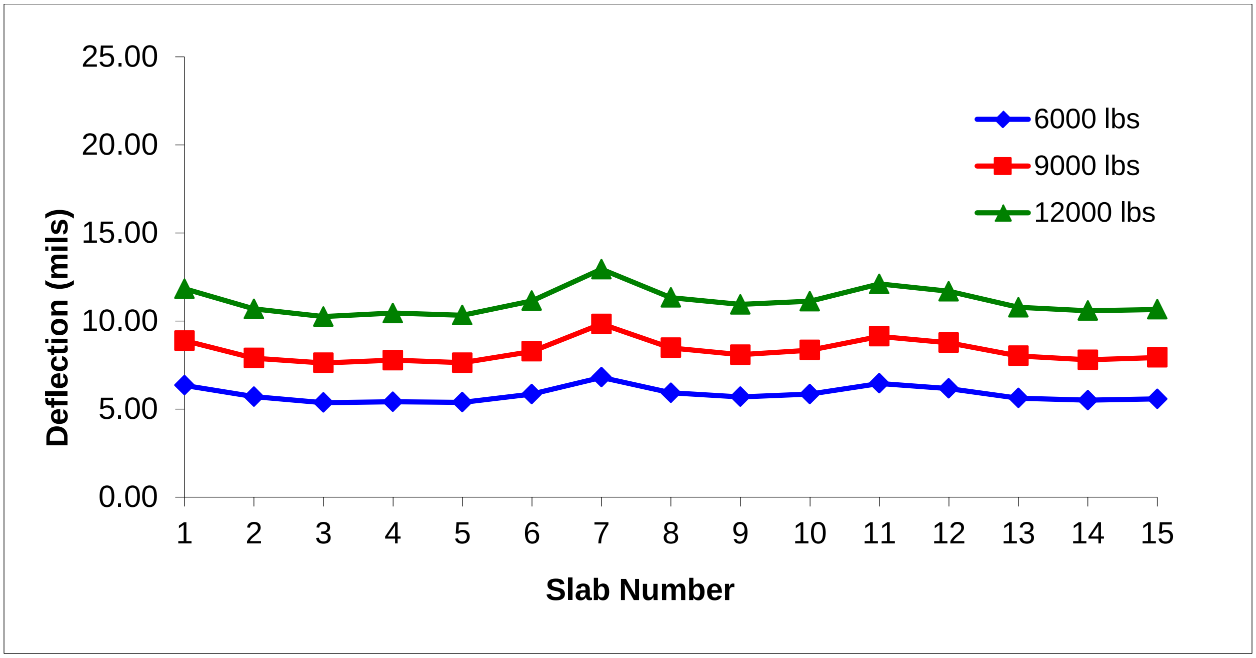

Figure B31. Normalized deflections (center slab), intensive testing on 5.5 foot panels, Bible Grove Road, 2012.

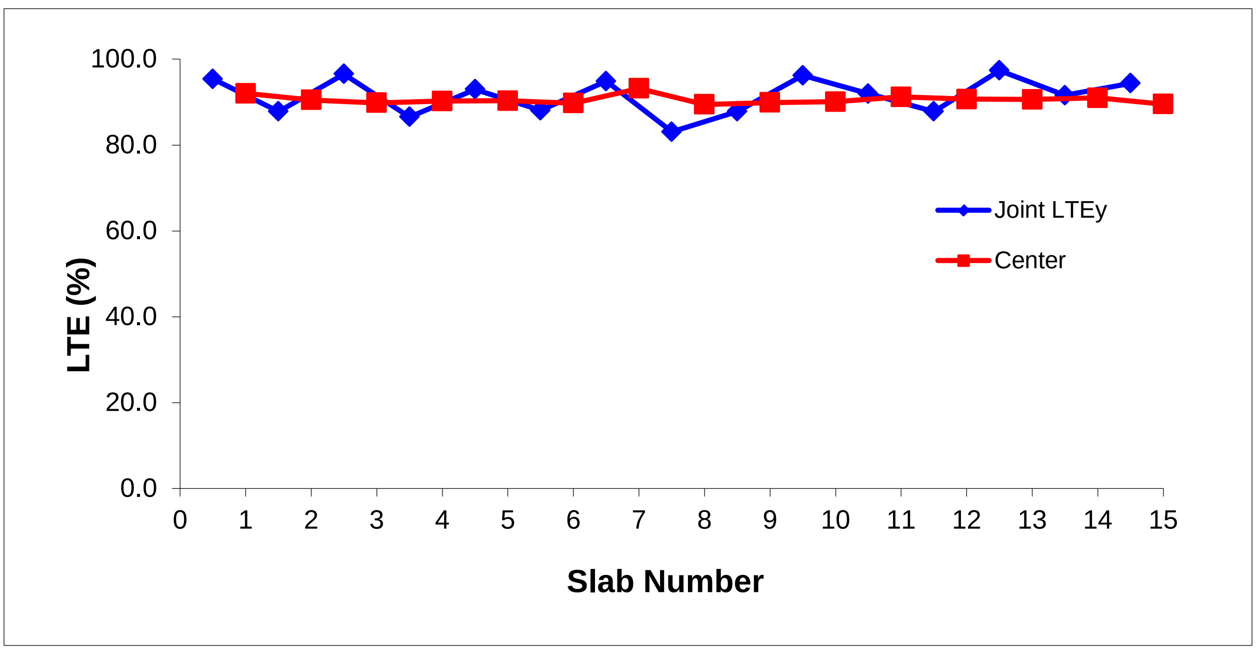

Figure B32. Transverse joint and center slab load transfer efficiency, intensive testing on 5.5 foot panels, Bible Grove Road, 2012.

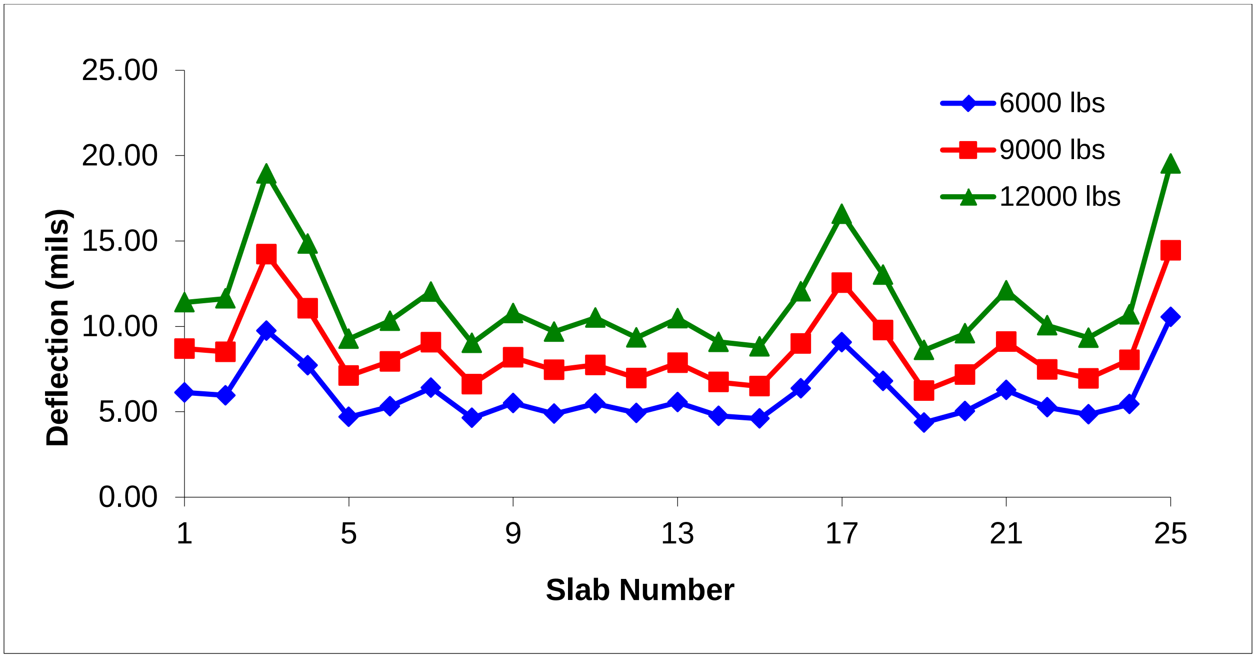

Figure B33. Normalized deflections (center slab), periodic testing on 5.5 foot panels, Bible Grove Road, 2012.

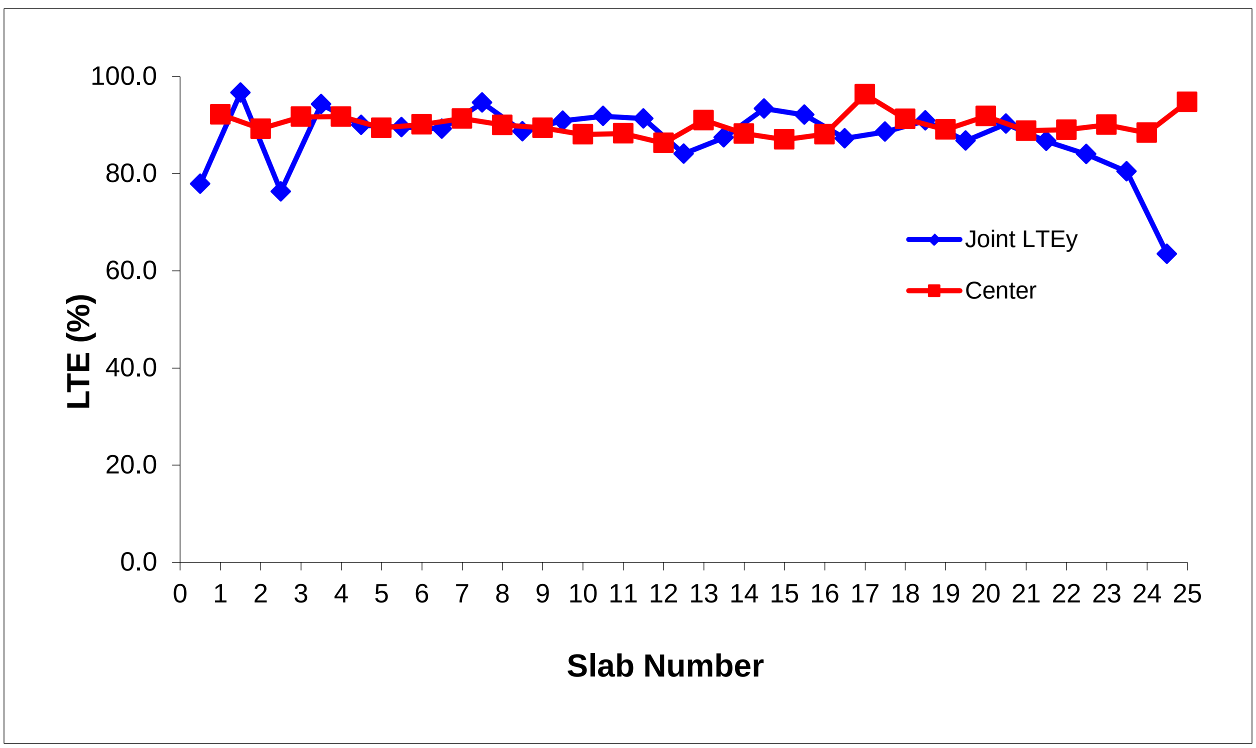

Figure B34. Transverse joint and center slab load transfer efficiency, periodic testing on 5.5 foot panels, Bible Grove Road, 2012.

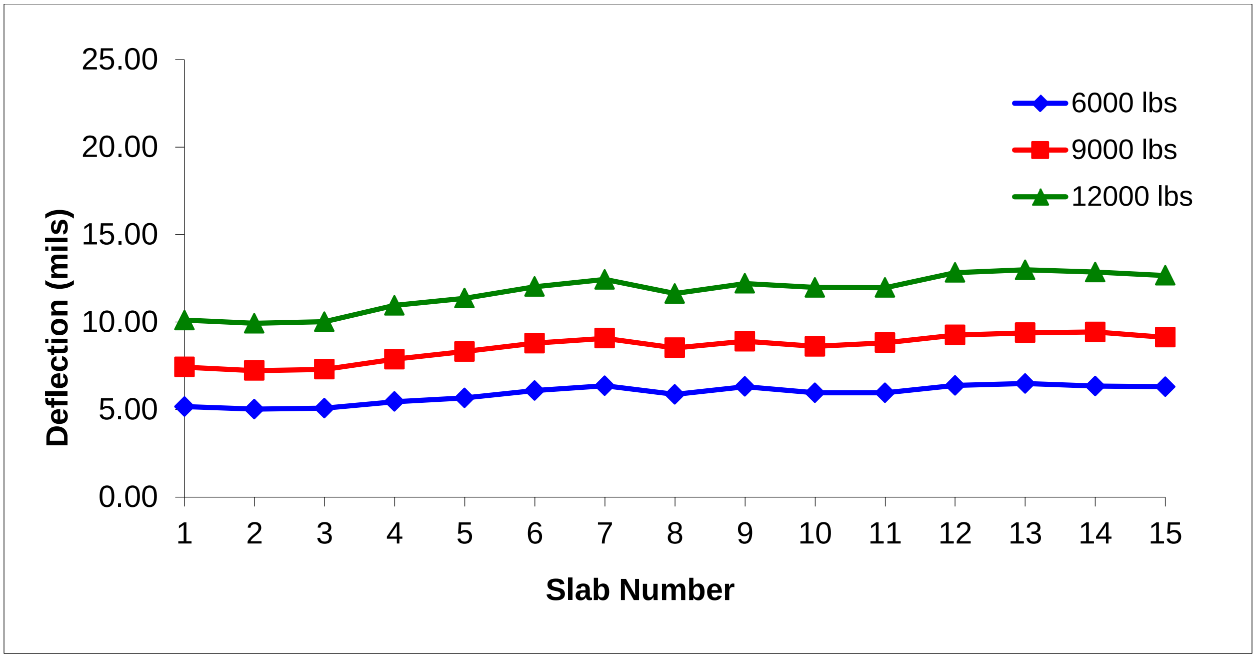

Figure B35. Normalized deflections (center slab), intensive testing on 11 foot panels, Bible Grove Road, 2012.

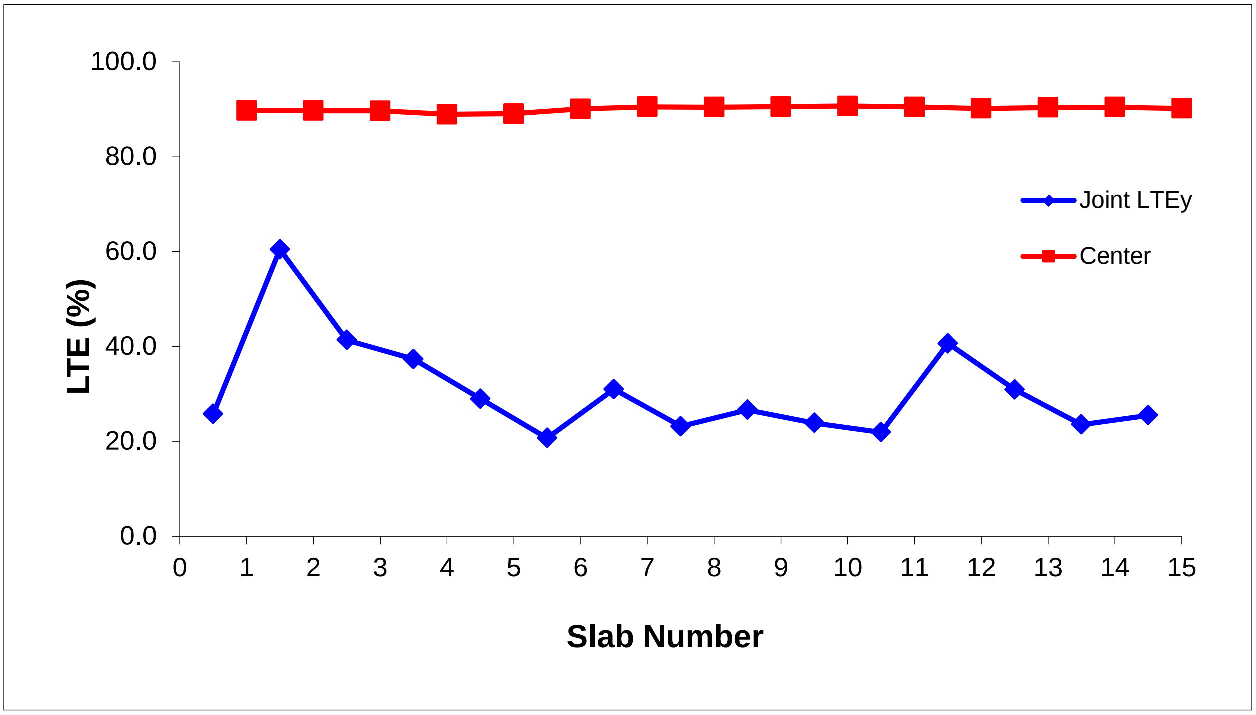

Figure B36. Transverse joint and center slab load transfer efficiency, intensive testing on 11 foot panels, Bible Grove Road, 2012.

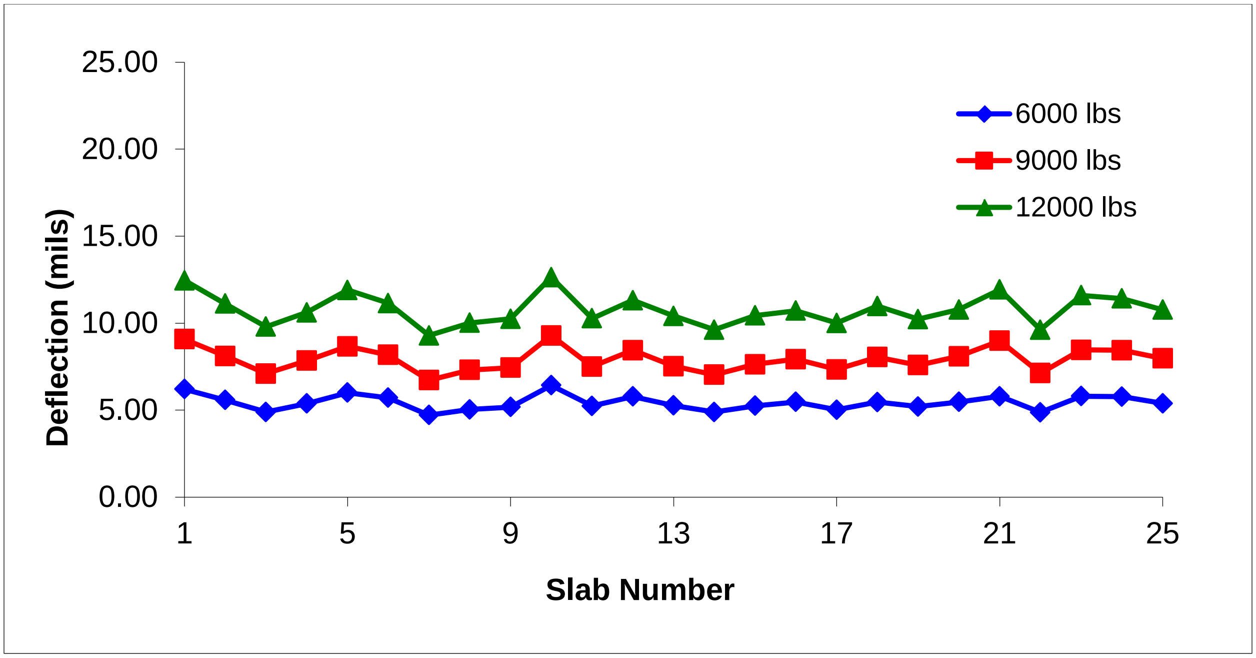

Figure B37. Normalized deflections (center slab), periodic testing on 11 foot panels, Bible Grove Road, 2012.

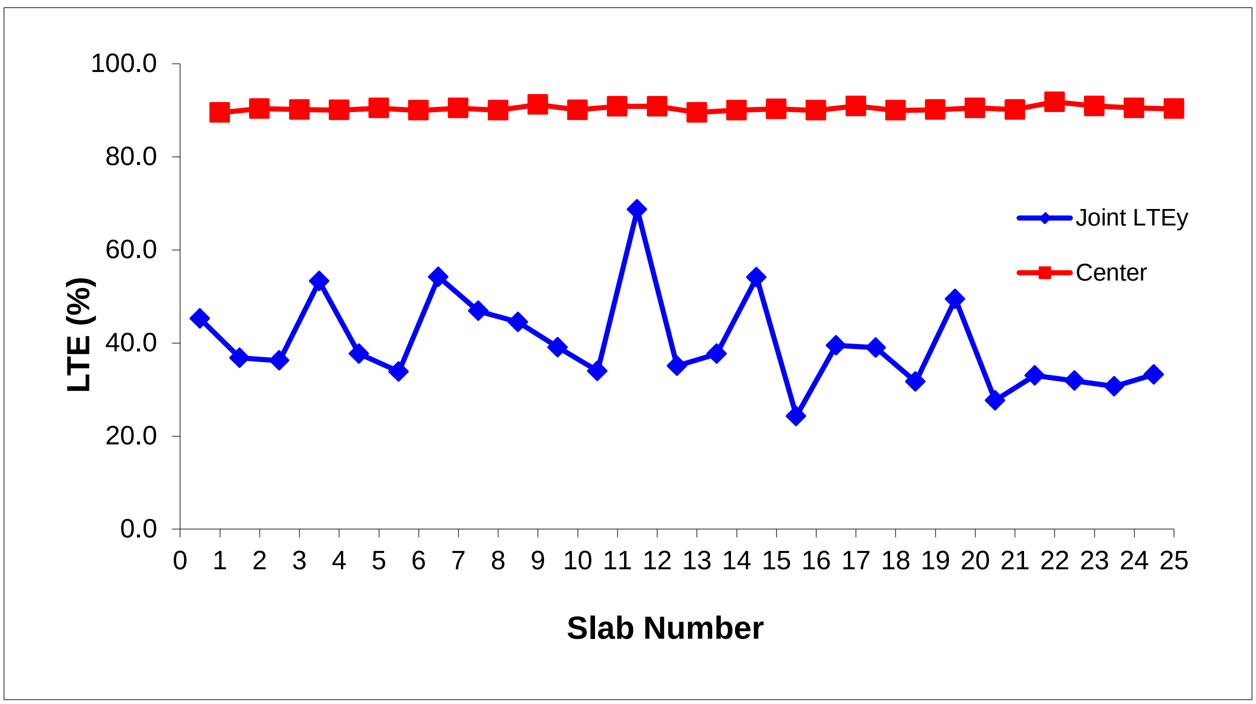

Figure B38. Transverse joint and center slab load transfer efficiency, periodic testing on 11 foot panels, Bible Grove Road, 2012.

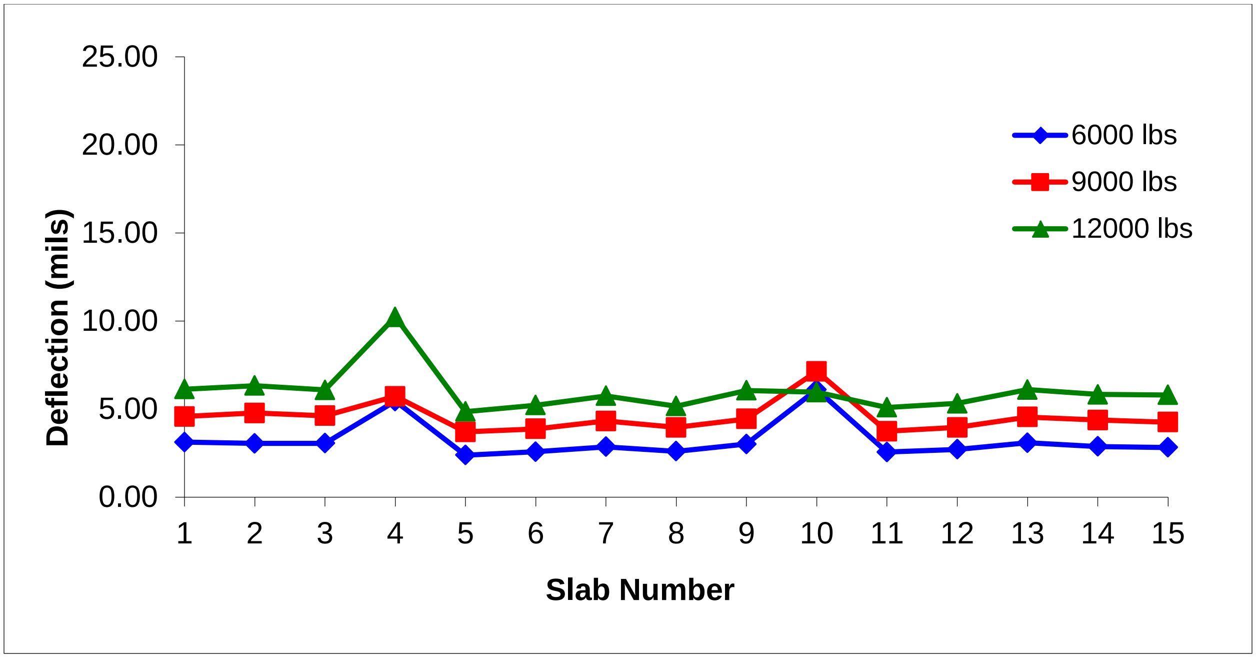

Figure B39. Normalized deflections (center slab), intensive testing, Richland County Hwy 9, 2012.

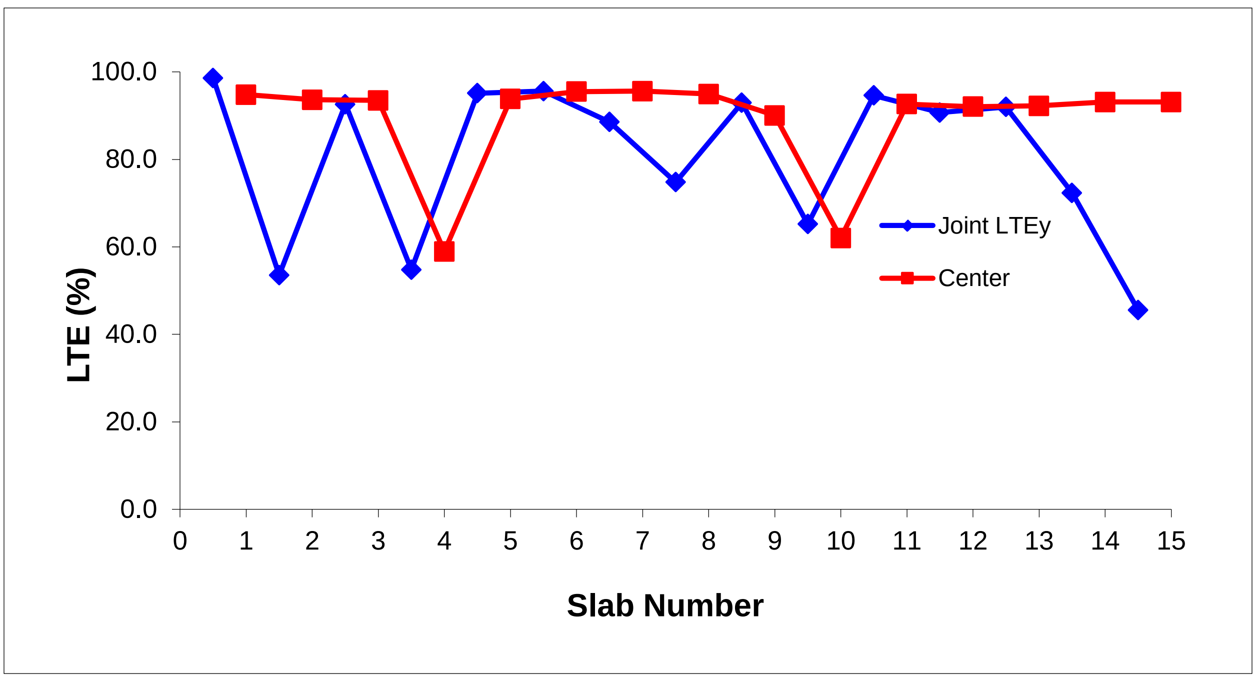

Figure B40. Transverse joint and center slab load transfer efficiency, intensive testing, Richland County Hwy 9, 2012.

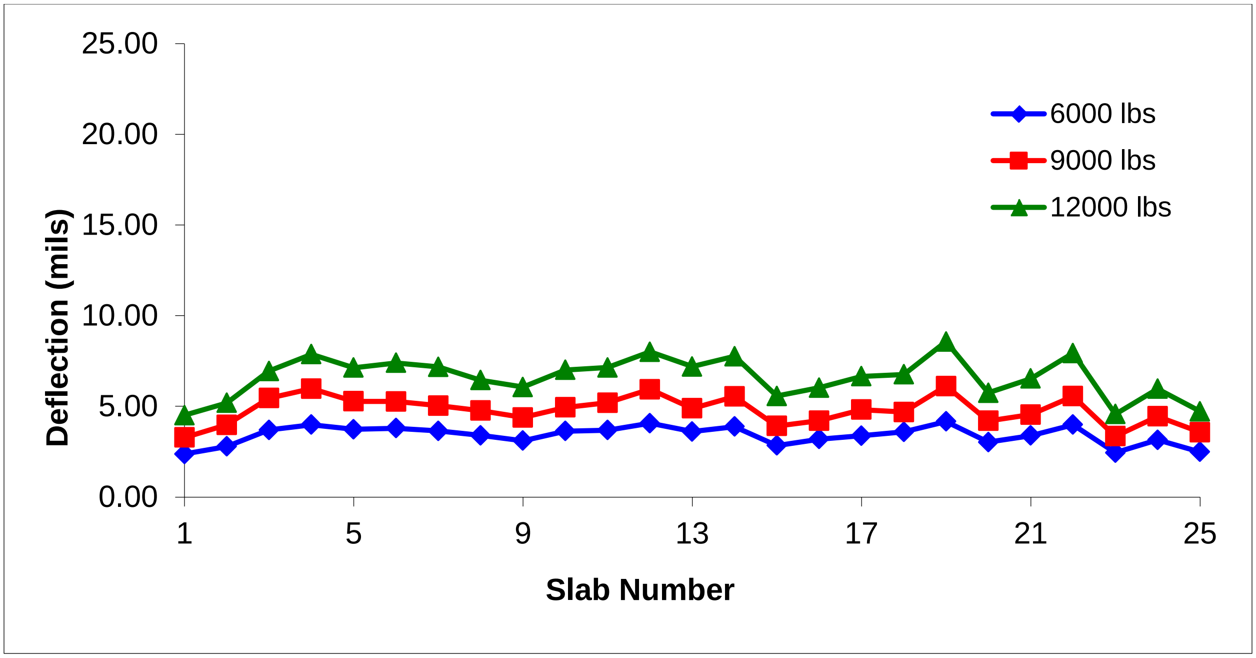

Figure B41. Normalized deflections (center slab), periodic testing, Richland County Hwy 9, 2013.

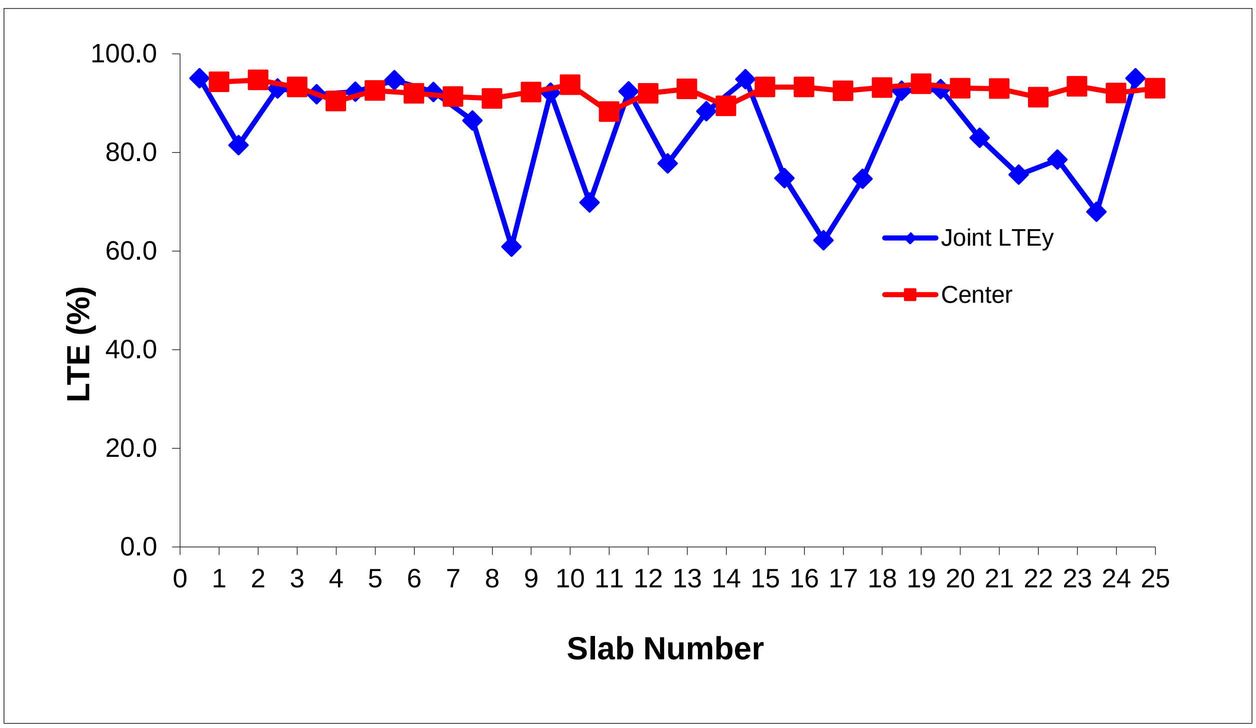

Figure B42. Transverse joint and center slab load transfer efficiency, periodic testing, Richland County Hwy 9, 2013.

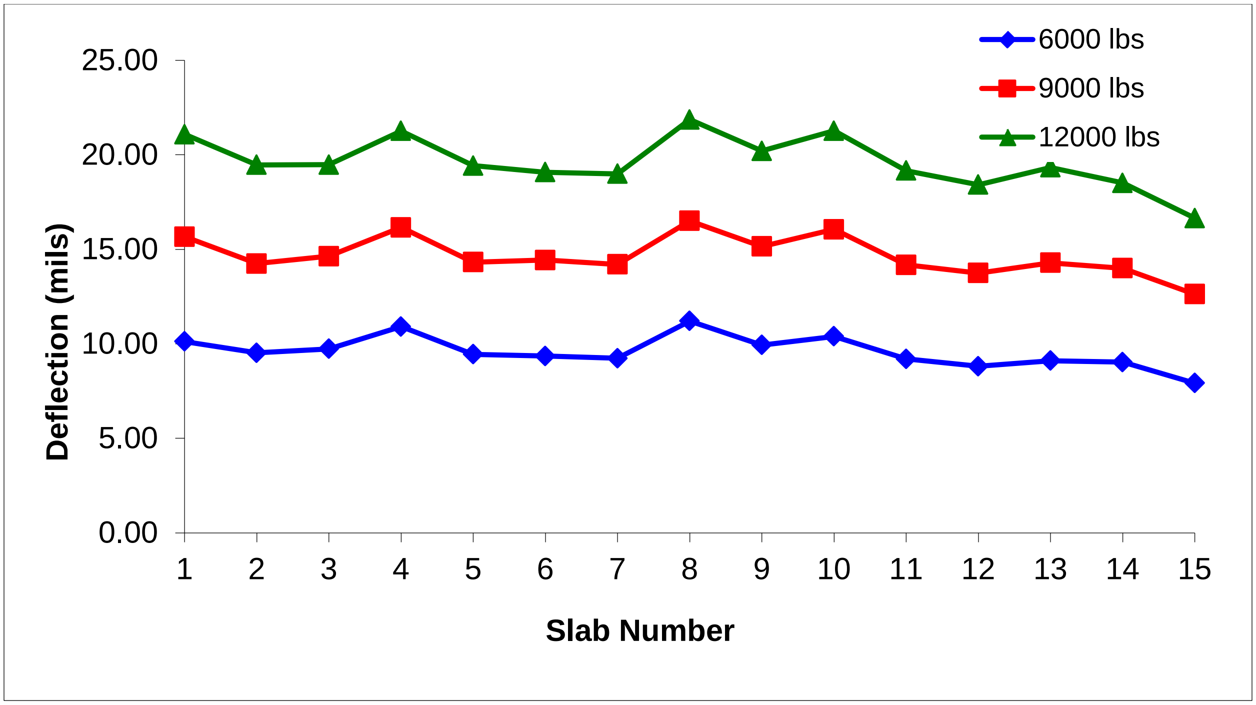

Figure B43. Normalized deflections (center slab), Section 1, North Lorang Road, 2013.

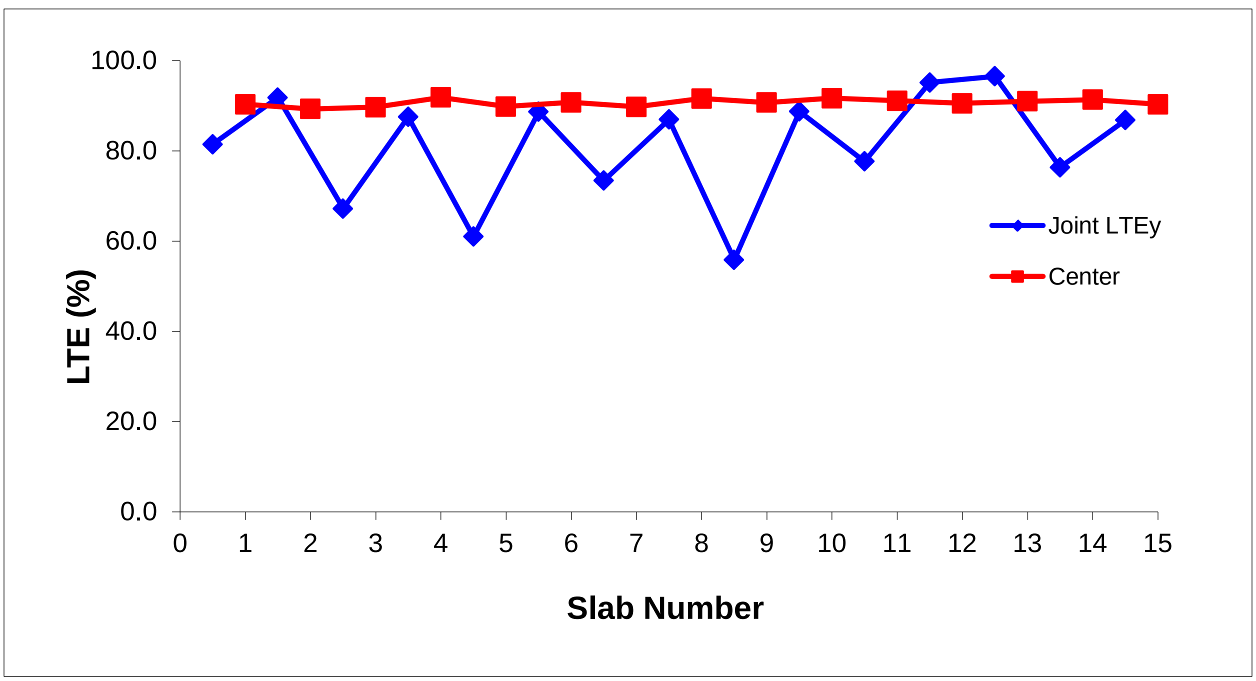

Figure B44. Transverse joint and center slab and load transfer efficiency, Section 1, North Lorang Road, 2013.

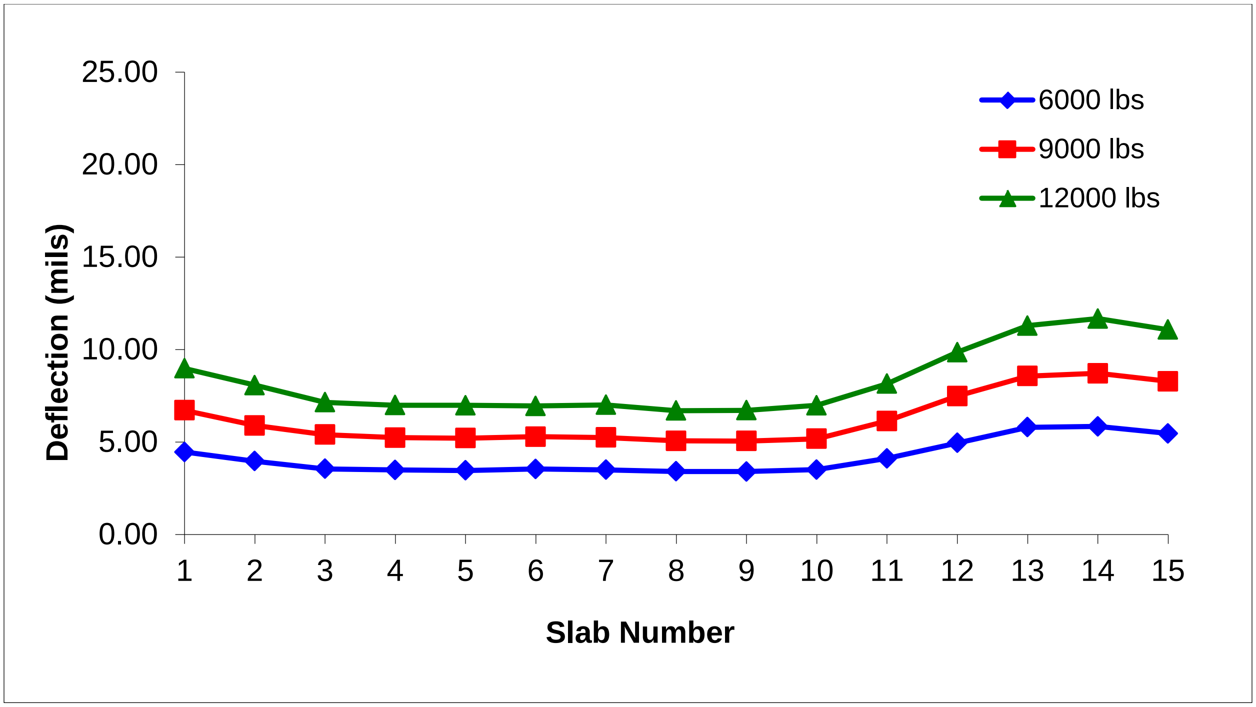

Figure B45. Normalized deflections (center slab), Section 2, North Lorang Road, 2013.

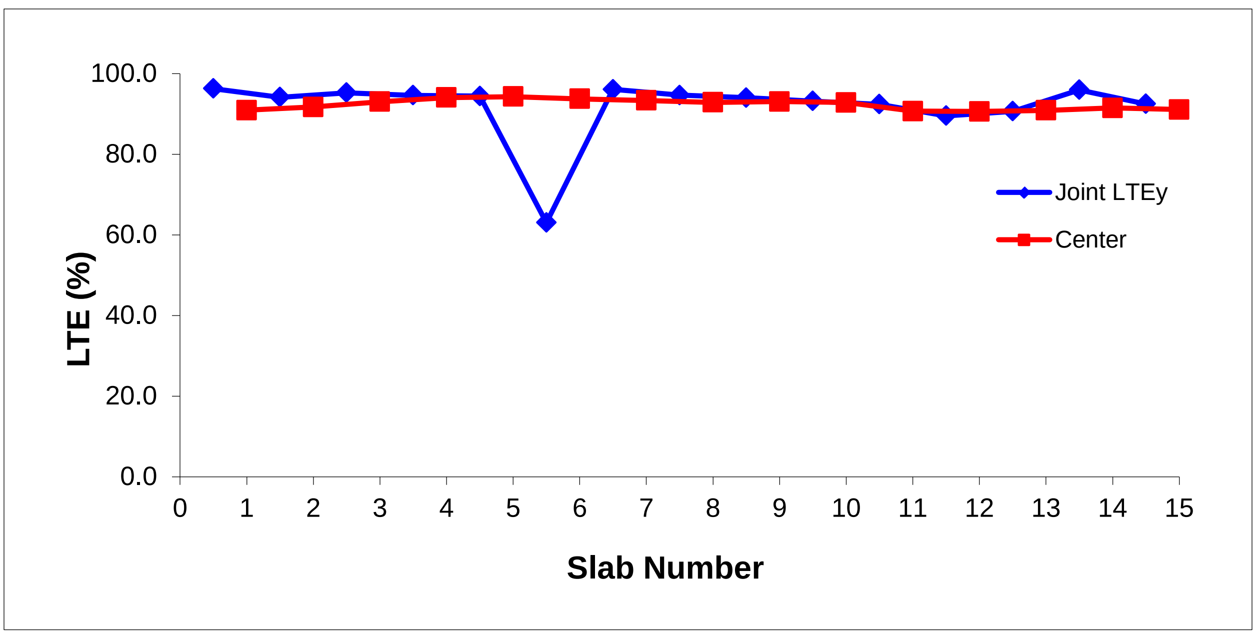

Figure B46. Transverse joint and center slab load transfer efficiency, Section 2, North Lorang Road, 2013.

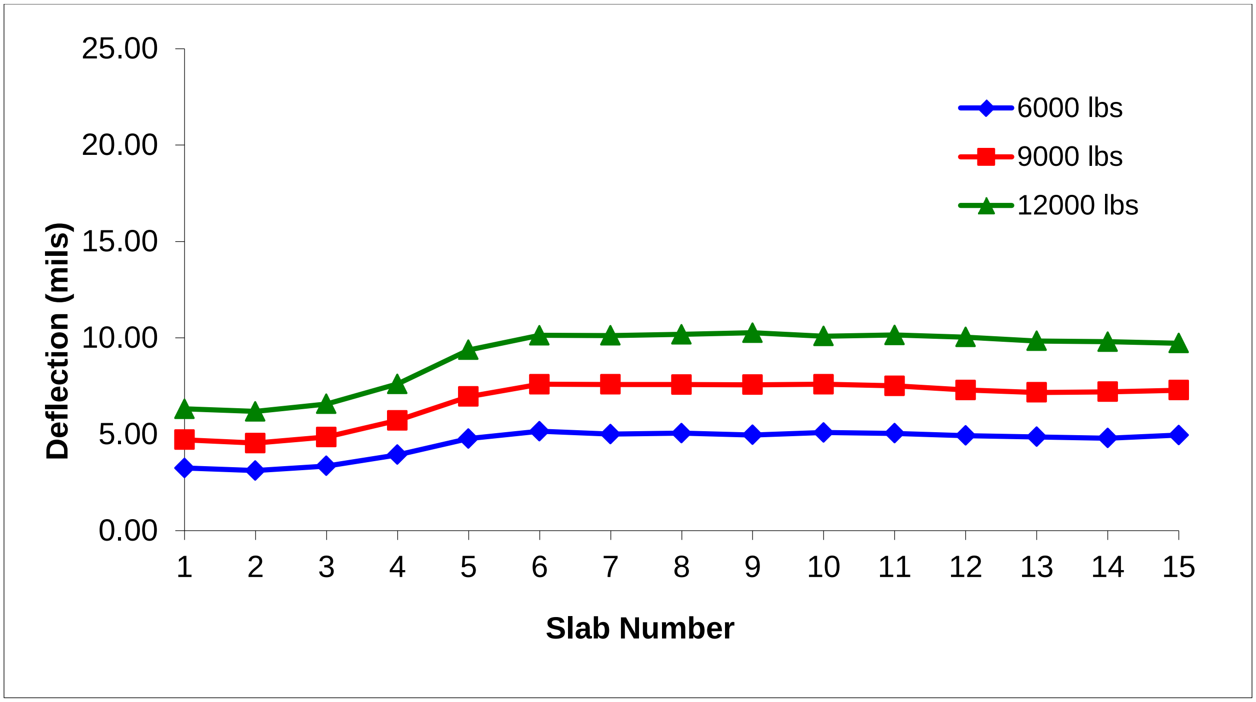

Figure B47. Normalized deflections (center slab), Section 3, North Lorang Road, 2013.

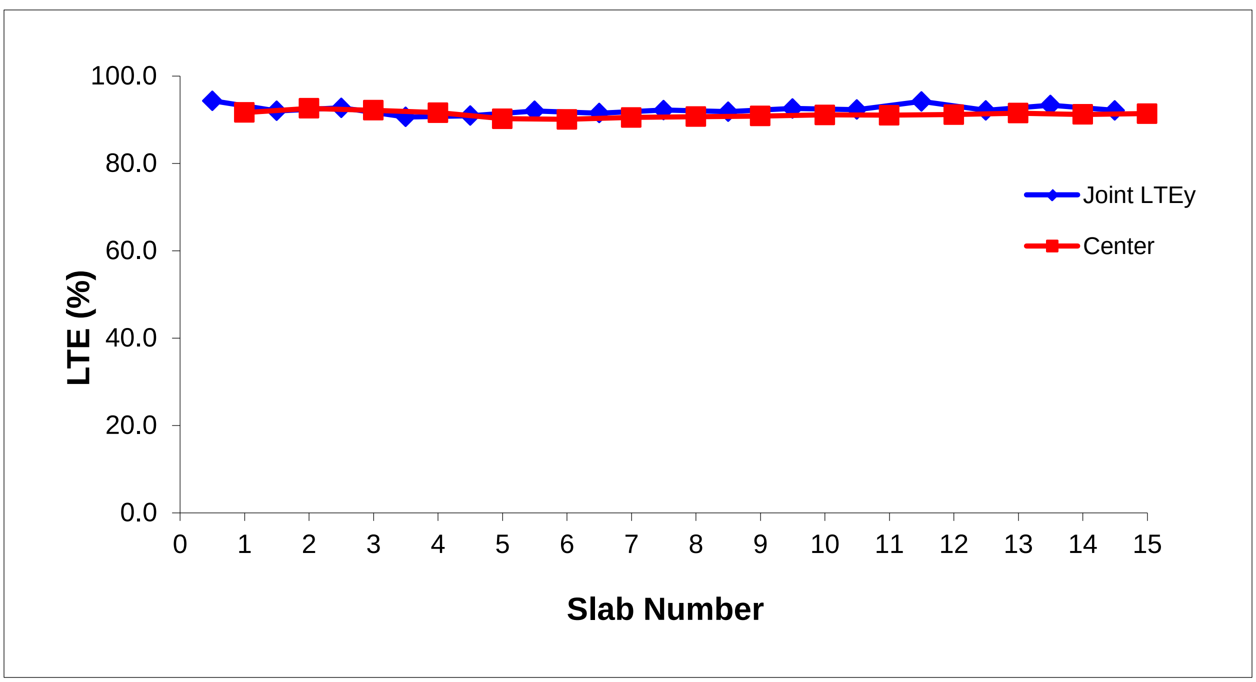

Figure B48. Transverse joint and center slab load transfer efficiency, Section 3, North Lorang Road, 2013.

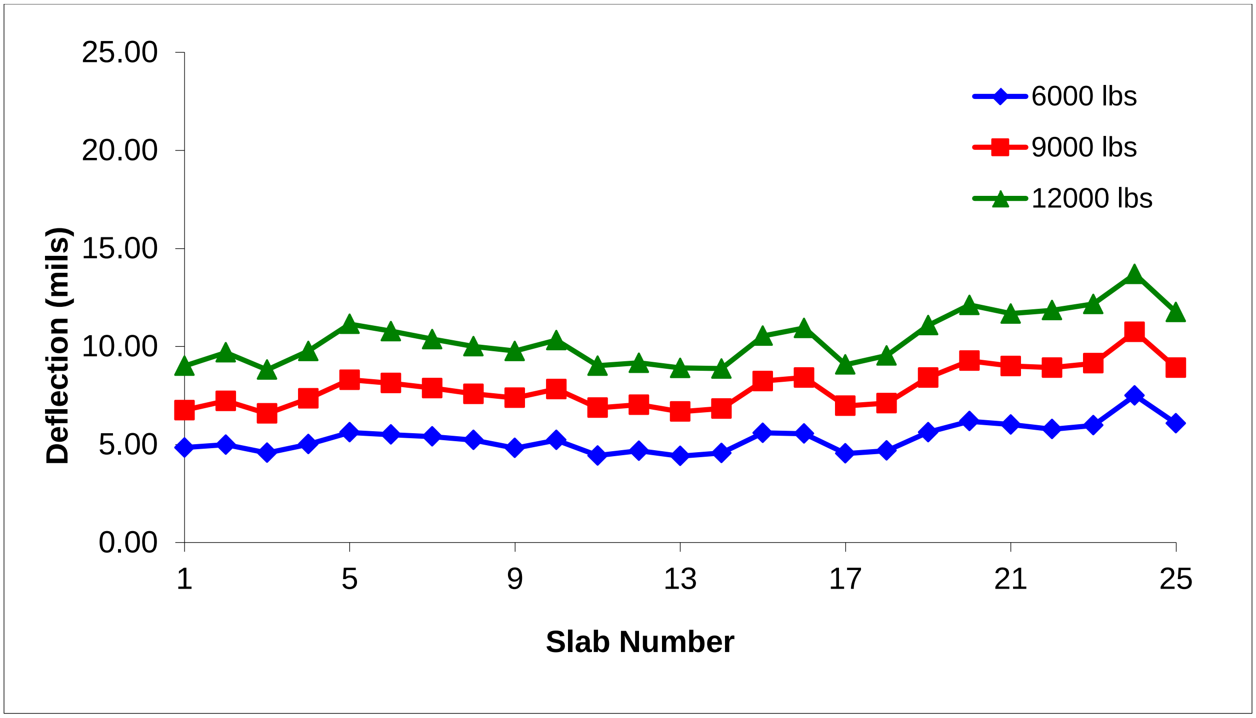

Figure B49. Normalized deflections (center slab), northbound Schank Avenue, 2013.

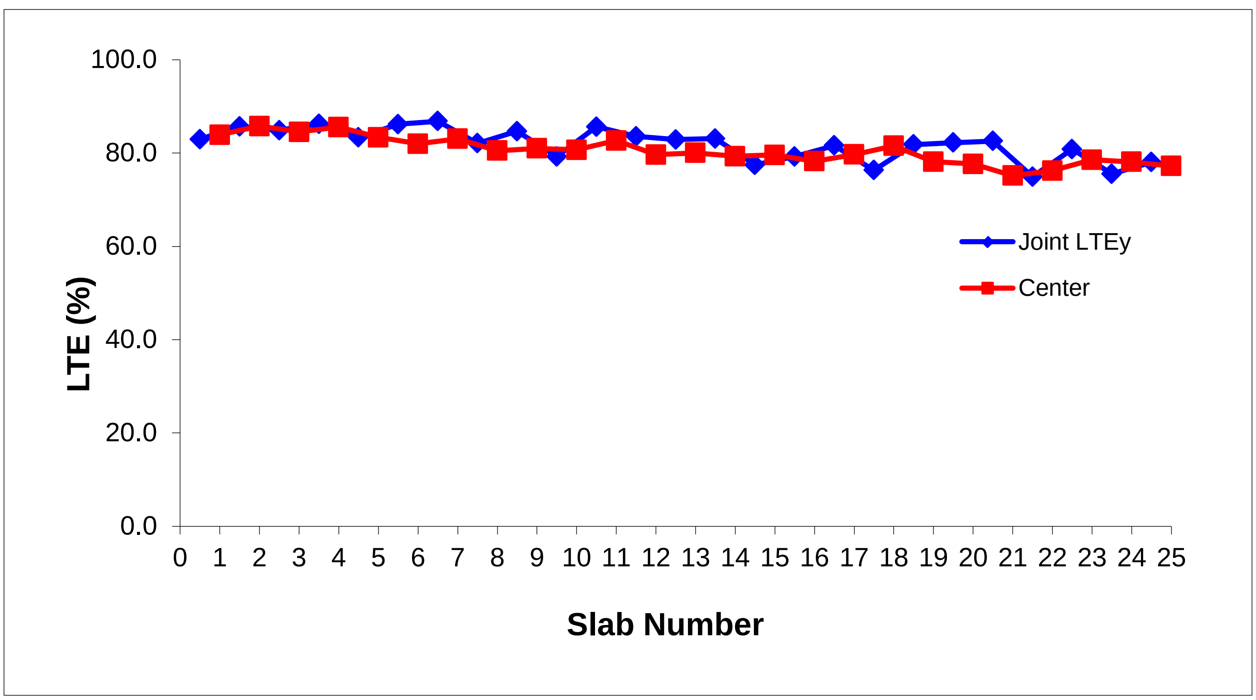

Figure B50. Transverse joint and center slab load transfer efficiency, northbound Schank Avenue, 2013.

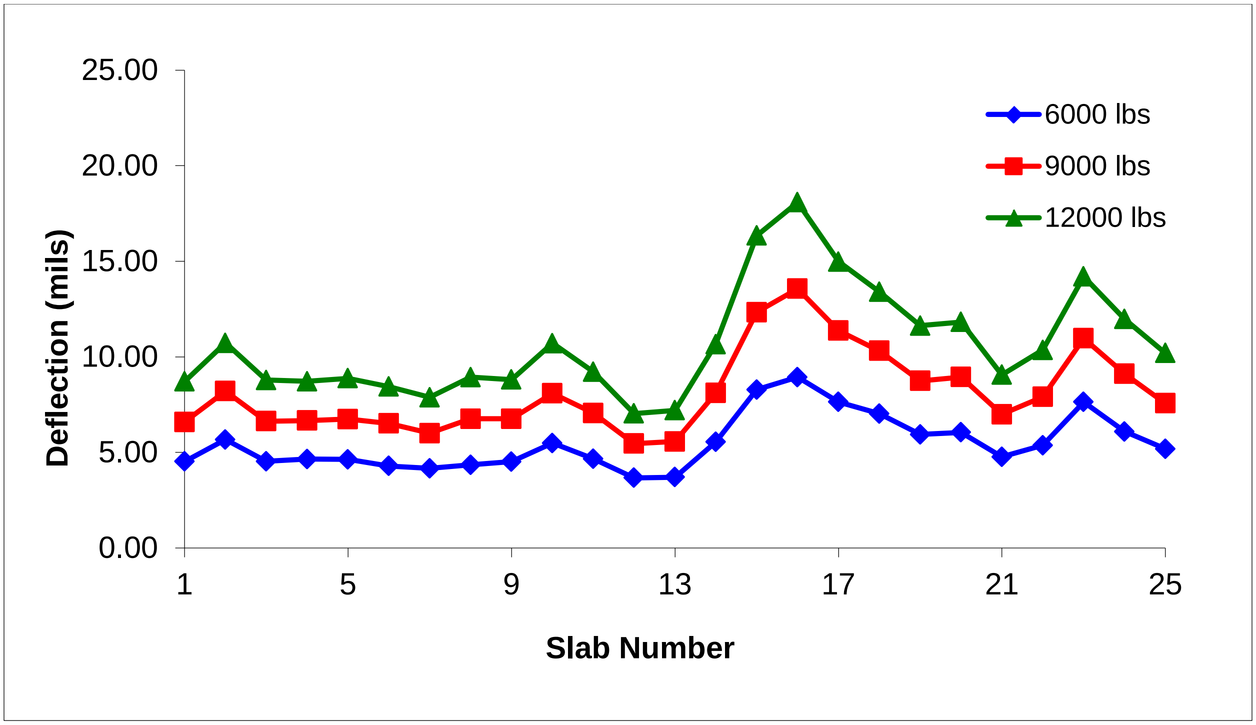

Figure B51. Normalized deflections (center slab), southbound Schank Avenue, 2013.

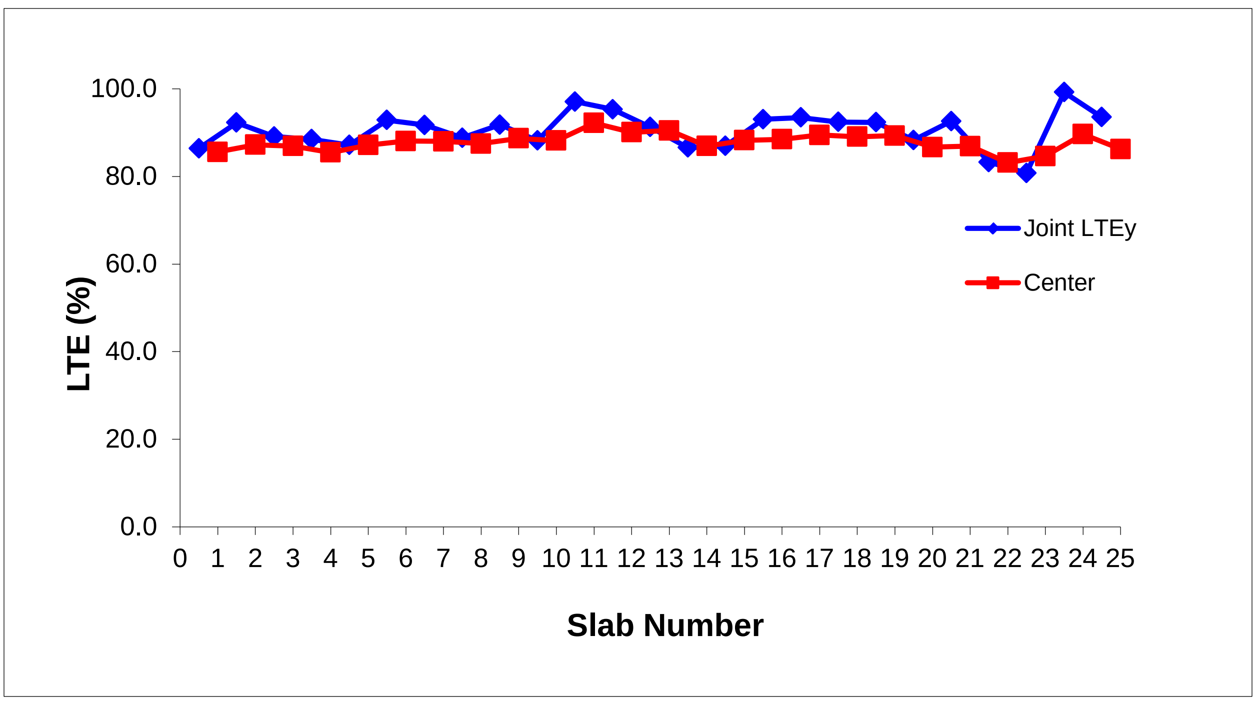

Figure B52. Transverse joint and center slab load transfer efficiency, southbound Schank Avenue, 2013.

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