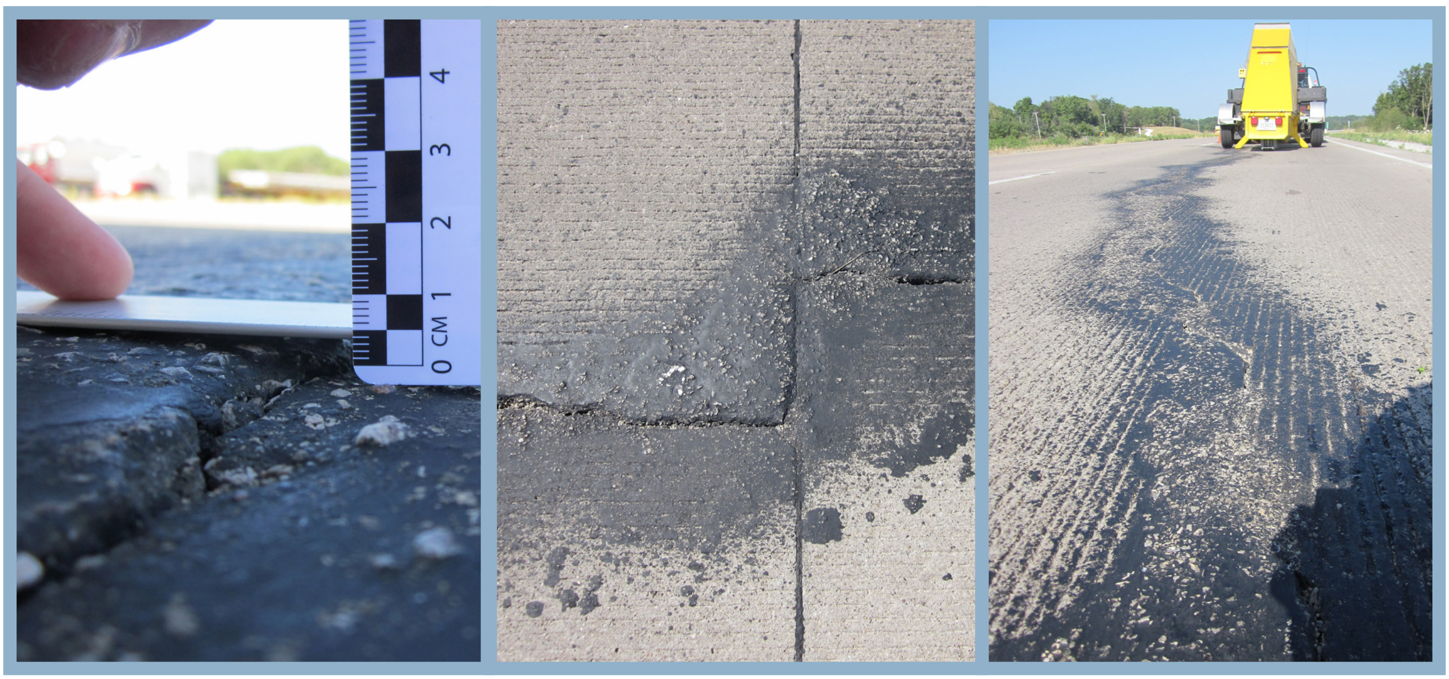
Improving the Foundation Layers for Concrete Pavements

TECHNICAL REPORT:

Field Assessment of Jointed Portland Cement Concrete Pavement with Premature Distresses – Iowa US 34 Field Study



February 2016

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Federal Highway Administration (DTFH 61-06-H-00011 (Work Plan #18))
FHWA TPF-5(183): California, Iowa (lead state), Michigan, Pennsylvania, Wisconsin

National Concrete Pavement Technology Center
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16. Abstract

This technical project report is one of the field project reports developed as part of the TPF-5(183) and FHWA DTFH 61-06-H-00011:WO18 studies.

The Iowa Department of Transportation (DOT) identified that a few sections of pavement on US Highway 34 near Mount Pleasant, Iowa showed early deterioration in ride quality. The Iowa State University research team conducted a visual survey of the cracked panels and the natural geography of the area and performed in situ falling weight deflectometer testing near center and joint of 140 panels over a span of about 830 m. Of the 140 panels, 25 panels showed distresses. The surface layer consisted of nominal 260 mm (10 in.) thick jointed portland cement concrete pavement (JPCP) placed over 150 to 260 mm (6 to 10 in.) thick subbase layer. Based on the project drawings, grading in the tested span required fills up to 10 m and cuts about to 3 m. Of the 830 m test span, about 480 m consisted of subgrade constructed with fill materials, about 350 m consisted of natural subgrade constructed in cut.

FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction (*k*) values between the cracked and uncracked panels, and cut and fill areas. Statistical t-test analysis was conducted to compare the measurement values obtained on panels with and without cracks and in cut and fill areas.

All cracked panels were located in the cut areas. Results showed statistically significant differences in the FWD test results between cracked and uncracked panels, with results on uncracked panels representing better support conditions than on cracked panels. Similarly, values in the fill areas showed support conditions that are better than in the cut areas. The *k* values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the *k* values were higher under cracked panels (38%) than under the uncracked panels (23%).

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Improving the Foundation Layers for Concrete Pavements: Field Assessment of Jointed Portland Cement Concrete Pavement with Premature Distresses – Iowa US 34 Field Study

Technical Report
February 2016

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ACKNOWLEDGMENTS

This research was conducted under Federal Highway Administration (FHWA) DTFH61-06-H-00011 Work Plan 18 and the FHWA Pooled Fund Study TPF-5(183), involving the following state departments of transportation:

- California
- Iowa (lead state)
- Michigan
- Pennsylvania
- Wisconsin

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LIST OF ACRONYMS AND SYMBOLS

BCI	Base curvature index
BDI	Base damage index
COV	Coefficient of variation
D_0	Deflection measured under the plate
D_0^*	Non-dimensional deflection coefficient
D_1 to D_7	Deflections measured away from the plate at various set distances
FWD	Falling weight deflectometer
I	Intercept
IRI	International roughness index
k	Modulus of subgrade reaction
$k_{\text{FWD-Dynamic}}$	Dynamic modulus of subgrade reaction from FWD test
$k_{\text{FWD-Static}}$	Static modulus of subgrade reaction from FWD test
$k_{\text{FWD-Static-Corr}}$	Static modulus of subgrade reaction from FWD test corrected for finite slab size
L	Relative stiffness
LTE	Load transfer efficiency
n	Number of measurements
P	Applied load
PCC	Portland cement concrete
r	Plate radius
SCI, BCI, BDI, AF	FWD deflection basin parameters
μ	Statistical mean or average
σ	Statistical standard deviation

EXECUTIVE SUMMARY

Quality foundation layers (the natural subgrade, subbase, and embankment) are essential to achieving excellent pavement performance. Unfortunately, many pavements in the United States still fail due to inadequate foundation layers. To address this problem, a research project, Improving the Foundation Layers for Pavements (FHWA DTFH 61-06-H-00011 WO #18; FHWA TPF-5(183)), was undertaken by Iowa State University (ISU) to identify, and provide guidance for implementing, best practices regarding foundation layer construction methods, material selection, in situ testing and evaluation, and performance-related designs and specifications. As part of the project, field studies were conducted in several in-service concrete pavements across the country that represented either premature failures or successful long-term pavements. A key aspect of each field study was to tie performance of the foundation layers to key engineering properties and pavement performance. In situ foundation layer performance data, as well as original construction data and maintenance/rehabilitation history data, were collected and geospatially and statistically analyzed to determine the effects of site-specific foundation layer construction methods, site evaluation, materials selection, design, treatments, and maintenance procedures on the performance of the foundation layers and of the related pavements. A technical report was prepared for each field study.

The Iowa Department of Transportation (DOT) identified that a few sections of pavement on US Highway 34 near Mount Pleasant, Iowa showed early deterioration in ride quality due to faulting, settlement, and longitudinal/transverse cracking. The identified sections were located between mile posts 208 and 200 and 194.5 to 193.7 on the west bound (WB) lane of US34. The section between mile posts 194.5 and 193.7 was selected for field testing in this study.

The Iowa State University (ISU) research team visited the site on June 25, 2012 and conducted a visual survey of the cracked panels and the natural geography of the area and performed in situ falling weight deflectometer testing near center and joint of 140 panels over a span of about 830 m. The start location of the testing was at Sta. 350+50. Of the 140 panels, 25 panels showed distresses ranging from longitudinal and transverse cracking, mid-panel cracking, corner cracking, and faulting. The ISU research team reviewed the as-built plans and cross-sections of the project site. The surface layer consisted of nominal 260 mm (10 in.) thick jointed portland cement concrete pavement (JPCP) placed over 150 to 260 mm (6 to 10 in.) thick subbase layer. Based on the project drawings, grading in the tested span required fills up to 10 m and cuts about to 3 m. Of the 830 m test span, about 300 m consisted of subgrade constructed with fill materials, about 530 m consisted of natural subgrade constructed in cut.

This report presents the field observations of the ISU research team and results and analysis of in situ falling weight deflectometer tests conducted on US34 WB between mile posts 194.5 and 196.7. FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction (k) values between the cracked and uncracked panels, and cut and fill areas. Statistical t-test analysis was conducted to compare the measurement values obtained on panels with and without cracks and in cut and fill areas. Pictures documenting the distresses observed on the pavement surface and cracks observed on embankment fill slopes are presented in this report.

Follwing are the key findings from this study:

- All of the cracked panels were located in the cut areas. Distresses observed on the pavement surface included longitudinal cracks, transverse cracks, mid-panel cracks, corner cracks, and faulting.
- Tension cracks were observed on the slope where about 10 m thick embankment fill was placed, which suggest possibility of slope movements.
- The D_0, $k_{FWD\text{-}Static\text{-}Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cracked and uncracked panels, with results on the uncracked panels representing better support conditions than on the cracked panels.
- The D_0, $k_{FWD\text{-}Static\text{-}Corr}$, SCI, BDI, and BCI values showed statistically significant differences between cut and fill areas, with results in the fill areas showing better support conditions than in the cut areas. (Note that all cracked panels were located in the cut area).
- The $k_{FWD\text{-}Static\text{-}Corr}$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the *k* values were higher under the cracked panels (38%) than under the uncracked panels (23%).
- The $k_{FWD\text{-}Static\text{-}Corr}$ values were on average about 1.1 times lower in cut areas than in fill areas. The COV of the *k* values were higher in the cut areas (31%) than in the fill areas (21%).
- There was no statistically significant difference in the I values between the cracked and uncracked panels and the cut and fill areas. The I values were all very low (≤ 1 μm). I > 5 μm is typically considered a trigger value suggesting void beneath the pavement.
- The joint LTE at all panels was relatively high (> 91%), and there was no statistically significant difference between the cracked and the uncracked panels and the cut and fill areas.

CHAPTER 1. INTRODUCTION

The Iowa Department of Transportation (DOT) identified that a few sections of pavement on US Highway 34 near Mount Pleasant, Iowa showed early deterioration in ride quality due to faulting, settlement, and longitudinal/transverse cracking. The identified sections were located between mile posts 208 and 200 and 194.5 to 193.7 on the west bound (WB) lane of US34. The section between mile posts 194.5 and 193.7 was selected for field testing in this study.

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The ISU research team reviewed the as-built plans and cross-sections of the project site. The surface layer consisted of nominal 260 mm (10 in.) thick jointed portland cement concrete pavement (JPCP) placed over 150 to 260 mm (6 to 10 in.) thick subbase layer. Based on the project drawings, grading in the tested span required fills up to 10 m and cuts about to 3 m. Of the 830 m test span, about 300 m consisted of subgrade constructed with fill materials and about 530 m consisted of natural subgrade constructed in cut.

This report presents the field observations of the ISU research team and results and analysis of in situ falling weight deflectometer tests. The FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction (k) values between the cracked and uncracked panels, and cut and fill areas.

Chapter 2 describes the FWD test procedure, the parameters calculated from the FWD data, and the statistical analysis procedures used in this study. Chapter 3 presents the results and analysis. Chapter 4 presents the key findings from this study.

CHAPTER 2. EXPERIMENTAL TESTING METHODS

Falling Weight Deflectometer

Falling weight deflectometer (FWD) tests were conducted using a Kuab FWD setup with a 300 mm (11.81 in) diameter loading plate by applying one seating drop and three loading drops. The applied loads varied from about 27 kN (6,000 lb) to 54 kN (12,000 lb) in the three loading drops. The actual applied loads were recorded using a load cell, and deflections were recorded using seismometers mounted on the device, per ASTM D4694-09 *Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device.* The FWD plate and deflection sensor setup and a typical deflection basin are shown in Figure 1. To compare deflection values from different test locations at the same applied contact stress, the values at each test location were normalized to a 40 kN (9,000 lb) applied force.

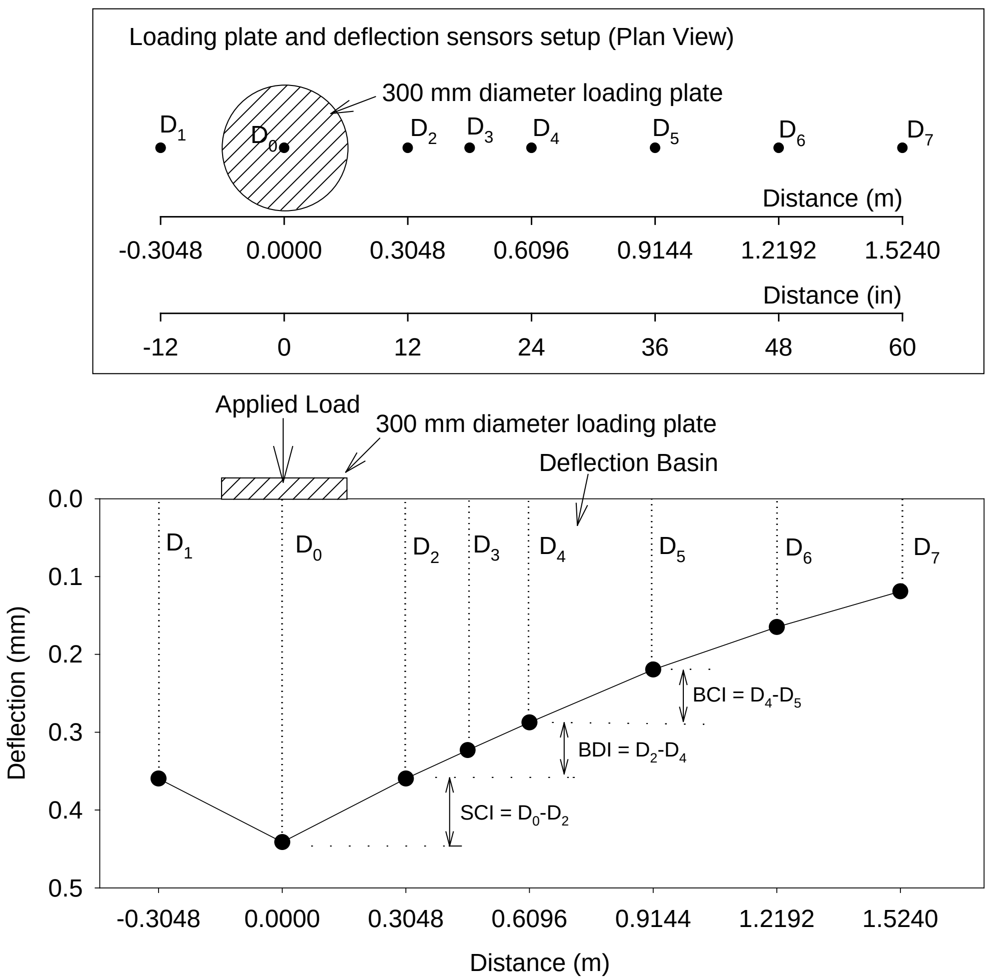

Figure 1. FWD deflection sensor setup used for this study and an example deflection basin

FWD tests were near mid panel and at joints. Tests conducted at the joints were used to determine joint load transfer efficiency (LTE) and voids beneath the pavement based on “zero” load intercept values. Tests conducted at the center of the slab panels were used to determine modulus of subgrade reaction (k) values and the intercept values. The procedures used to calculate these parameters are described below.

LTE was determined by obtaining deflections under the plate on the loaded slab (D_0) and deflections of the unloaded slab (D_1) using a sensor positioned about 305 mm (12 in.) away from the center of the plate (Figure 1). The LTE was calculated using Equation 4.

$$LTE(\%) = \frac{D_1}{D_0} \times 100 \qquad (4)$$

Voids underneath pavements can be detected by plotting the applied load measurements on the X-axis and the corresponding deflection measurements on the y-axis and plotting a best fit linear regression line, as illustrated in Figure 2, to determine the "zero" load intercept (I) values. AASHTO (1993) suggests I = 0.05 mm (2 mils) as a critical value for void detection. According to von Quintus and Simpson (2002), if I = -0.01 and +0.01 mm, then the response would be considered elastic. If I > 0.01 then the response would be considered deflection hardening, and if I < -0.01 then the response would be considered deflection softening.

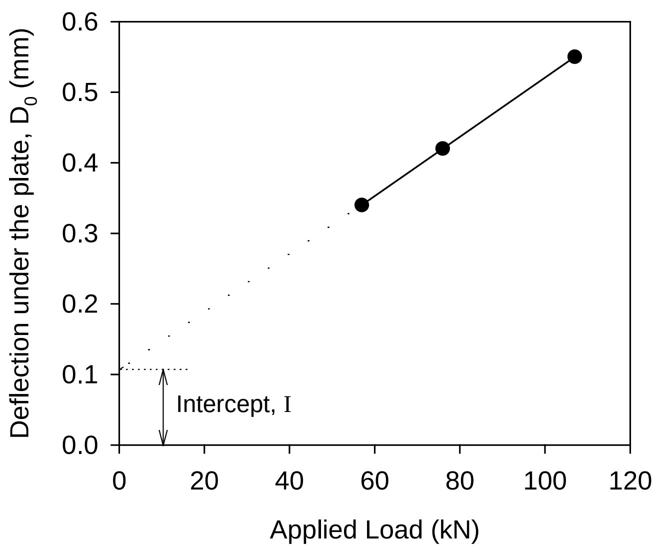

Figure 2. Void detection using load-deflection data from FWD test

Pavement layer temperatures at different depths were obtained during FWD testing, in accordance with the guidelines from Schmalzer (2006). The temperature measurements were used to determine equivalent linear temperature gradients (T_L) following the temperature-moment concept suggested by Jannsen and Snyder (2000). According to Vandenbossche (2005), I-values are sensitive to temperature induced curling and warping affects. Large positive temperature gradients (i.e., when the surface is warmer than the bottom) that cause the panel corners to curl down result in false negative I-values. Conversely, large negative gradients (i.e., when the surface is cooler than the bottom) that cause the panel corners to curl upward result in false positive I-values. Interpretation of I-values therefore should consider the temperature gradient. Concerning LTE measurements for doweled joints, the temperature gradient is reportedly not a critical factor (Vandenbossche 2005).

The SCI, BDI, BCI, and AF measurements are referred to as deflection basin parameters and are determined using the following equations:

$$\text{SCI (mm)} = D_0 - D_2 \tag{5}$$

$$\text{BDI (mm)} = D_2 - D_4 \tag{6}$$

$$\text{BCI (mm)} = D4 - D5 \tag{7}$$

$$\text{AF (mm)} = \frac{152.4 \times (D_0 + 2D_2 + 2D_4 + D_5)}{D_0} \tag{8}$$

where, D_0 = peak deflection measured directly beneath the plate, D_2 = peak deflection measured at 305 mm away from the plate center, D_4 = peak deflection measured at 510 mm away from the plate centre, and $D_5 =$ peak deflection measured at 914 mm away from the plate centre.

According to Horak (1987), the SCI parameter provides a measure of the strength/ stiffness of the upper portion (base layers) of the pavement foundation layers (Horak 1987). Similarly, BDI represents layers between 300 mm and 600 mm depth (base and subbase layers) and BCI represents layers between 600 mm and 900 mm depth (subgrade layers) from the surface (Kilareski and Anani 1982). The AF is primarily the normalized (with D_0) area under the deflection basin curve up to sensor D_5 (AASHTO 1993). AF has been used to characterize variations in the foundation layer material properties by some researchers (e.g., Stubstad 2002). Comparatively, lower SCI or BDI or BCI or AF values indicate better support conditions (Horak 1987).

The *k* values were determined using the $AREA_4$ method described in AASHTO (1993). Since the *k* value determined from FWD test represents a dynamic value, it is referred to here as $k_{\text{FWD-Dynamic}}$. Deflections obtained from four sensors (D_0, D_2, D_4, and D_5 shown in Figure 1 were used in the $AREA_4$ calculation. The AREA method was first proposed by Hoffman and Thompson (1981) for flexible pavements and has since been applied extensively for concrete pavements (Darter et al. 1995). $AREA_4$ is calculated using Equation 5 and has dimensions of length (in inches), as it is normalized with deflections under the center of the plate (D_0):

$$AREA_4 = 6 + 12 \times \left(\frac{D_2}{D_0}\right) + 12 \times \left(\frac{D_4}{D_0}\right) + 6 \times \left(\frac{D_5}{D_0}\right) \tag{5}$$

where D_0 = deflections measured directly under the plate (in.); D_2 = deflections measured at 305 mm (12 in.) away from the plate center (in.); D_4 = deflections measured at 610 mm (24 in.) away from the plate center (in.); and D_5 = deflections measured at 914 mm (36 in.) away from the plate center (in.). The $AREA_4$ method can also be calculated using different sensor configurations and setups, (i.e., using deflection data from 3, 5, or 7 sensors), and those methods are described in detail in the literature (Substad et al. 2006, Smith et al. 2007)

In early research conducted using the AREA method, the ILLI-SLAB finite element program was used to compute a matrix of maximum deflections at the plate center and the AREA values by varying the subgrade *k*, the modulus of the PCC layer, and the thickness of the slab (ERES Consultants, Inc. 1982). Measurements obtained from FWD tests were then compared with the ILLI-SLAB program results to determine the *k* values through back calculation. Barenberg and Petros (1991) and Ioannides (1990) proposed a forward solution procedure based on Westergaard's solution for loading on an infinite plate to replace the back calculation procedure. This forward solution presented a unique relationship between AREA value (for a given load and sensor arrangement) and the dense liquid radius of relative stiffness (L) in which subgrade is characterized by the *k* value. The radius of relative stiffness (L) is estimated using Equation 6:

$$L = \left[\frac{\ln\left(\frac{x_1 - AREA_4}{x_2} \right)}{x_3} \right]^{x_4} \tag{6}$$

where $x_1 = 36$, $x_2 = 1812.279$, $x_3 = -2.559$, $x_4 = 4.387$. It must be noted that the x_1 to x_4 values vary with the sensor arrangement and these values are only valid for the $AREA_4$ sensor setup. Once, the L value is known, the $k_{\text{FWD-Dynamic}}$ value can be estimated using Equation 7:

$$k_{FWD-Dynamic} \ (pci) = \frac{PD_0^*}{D_0 L^2} \tag{7}$$

where P = applied load (lbs), D_0 = deflection measured at plate center (inches), and D_0^* = non-dimensional deflection coefficient calculated using Equation 8:

$$D_0^* = a \cdot e^{-be^{-cL}} \tag{8}$$

where $a = 0.12450$, $b = 0.14707$, c = 0.07565. It must be noted that these equations and coefficients are valid for an FWD setup with an 11.81 in. diameter plate.

The advantages of the $AREA_4$ method are the ease of use without back calculations and the use of multiple sensor data. The disadvantages are that the process assumes that the slab and the subgrade are horizontally infinite. This assumption leads to underestimating the *k* values of jointed pavements. Crovetti (1993) developed the following slab size corrections for a square slab that is based on finite element analysis conducted using the ILLI-SLAB program and is for use in the $k_{\text{FWD-Dynamic}}$:

$$Adjusted\ D_0 = D_0\left(1 - 1.15085e^{-0.71878\left(\frac{L'}{L}\right)^{0.80151}}\right) \tag{9}$$

$$Adjusted\ L = L\left(1 - 0.89434e^{-0.61662\left(\frac{L'}{L}\right)^{1.04831}}\right) \tag{10}$$

where L′ = slab size (smaller dimension of a rectangular slab, length or width). This procedure also has limitations: (1) it considers only a single slab with no load transfer to adjacent slabs, and (2) it assumes a square slab. The square slab assumption is considered to produce sufficiently accurate results when the smaller dimension of a rectangular slab is assumed as L′ (Darter et al. 1995). Darter et al. (1995) suggested using $L' = \sqrt{Length \times Width}$ to further refine slab size corrections. However, no established procedures for correcting for load transfer to adjacent slabs have been reported, so accounting for load transfer remains as a limitation of this method.

AASHTO (1993) suggests dividing the $k_{\text{FWD-Dynamic}}$ value by a factor of 2 to determine the equivalent $k_{\text{FWD-Static}}$ value. The origin of this factor 2 dates back to Foxworthy's work in the 1980s. Foxworthy (1985) reported comparisons between the $k_{\text{FWD-Dynamic}}$ values obtained using Dynatest model 8000 FWD and the Static k values (Static k_{PLT}) obtained from 30 in. diameter plate load tests (the exact procedure followed to calculate the Static k_{PLT} is not reported in Foxworthy 1985). Foxworthy used the AREA based back calculation procedure using the ILLI-SLAB finite element program. Results obtained from Foxworthy's study (Figure 3) are based on 7 FWD tests conducted on PCC pavements with slab thicknesses varying from about 10 in. to 25.5 in. and plate load tests conducted on the foundation layer immediately beneath the pavement over a 4 ft x 5 ft test area. A few of these sections consisted of a 5 to 12 in. thick base course layer and some did not. The subgrade layer material consisted of CL soil from Sheppard Air Force Base in Texas, SM soil from Seymour-Johnson Air Force Base in North Carolina, and an unspecified soil type from McDill Air Force base in Florida. No slab size correction was performed on this dataset.

Data from Foxworthy (1985) yielded a logarithmic relationship between the dynamic and the static k values. On average, the $k_{\text{FWD-Dynamic}}$ values were about 2.4 times greater than the Static k_{PLT} values. Darter et al. (1995) indicated that the factor 2 is reasonable based on results from other test sites (Figure 3). Darter et al. (1995) also compared FWD test data from eight long-term pavement performance (LTPP) test sections with the Static k_{PLT} values and reported factors ranging from 1.78 to 2.16, with an average of about 1.91. The $k_{\text{FWD-Dynamic}}$ values used in that comparison were corrected for slab size. For the analysis conducted in this research project, the corrected $k_{\text{FWD-Dynamic}}$ values (for finite slab size) were divided by 2 and are reported as $k_{\text{FWD-Static-Corr}}$ values.

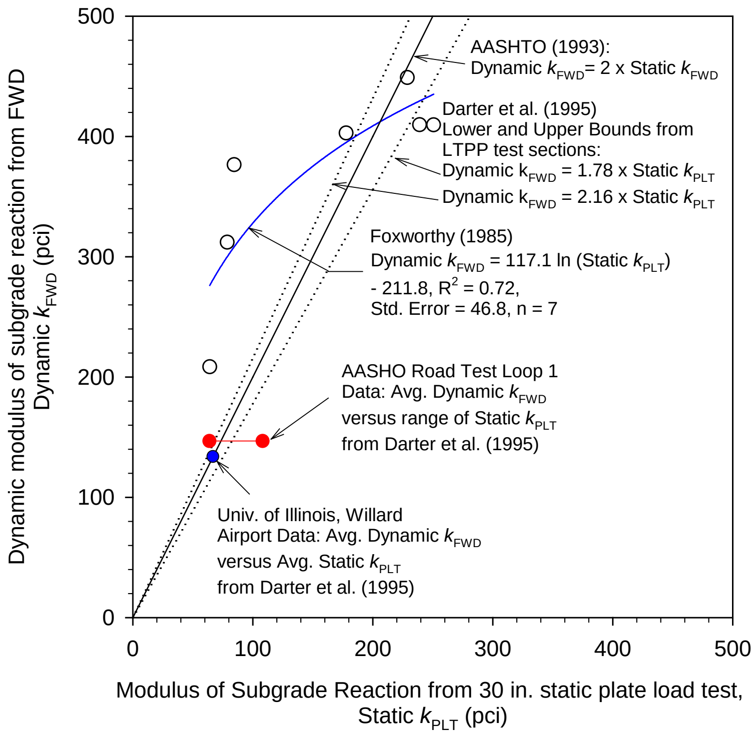

Figure 3. Static k_{PLT} values versus $k_{FWD\text{-}Dynamic}$ measurements reported in literature

Statistical Analysis

Student *t*-test analysis (Ott and Longnecker 2008) was conducted to assess differences between results obtained on cracked and uncracked panels, using the following equations:

$$t = \frac{\mu_0 - \mu_1}{s_p \sqrt{\frac{1}{n_0} + \frac{1}{n_1}}} \tag{6}$$

where,

$$S_p = \sqrt{\frac{(n_0 - 1) \times s_0^2 + (n_1 - 1) \times s_1^2}{n_0 + n_1 - 2}} \tag{7}$$

n_0 and n_1 = number of measurements obtained on cracked or uncracked section, respectively; S_p = pooled standard deviation; and s_0 and s_1 = standard deviation of measurements obtained on cracked or uncracked sections, respectively.

The observed *t*-values were compared with the minimum t-value for a one-tailed test with degree of freedom (df) = $n_0 + n_1 - 2$, for 95% confidence level (i.e., $\alpha = 0.05$). When comparing measurements from cracked or uncracked sections, if the *t*-values were greater than the minimum *t*-value, then it was concluded that there is sufficient evidence that the measurements were statistically different.

CHAPTER 3. FIELD TEST RESULTS, OBSERVATIONS, AND ANLAYSIS

Pictures in Figure 4 to Figure 12 show the various distresses observed on the pavement surface layer, such as longitudinal cracks, transverse cracks, mid-panel cracks, corner cracks, and faulting.

Figure 13 and Figure 14 show pictures of embankment slope on the north side of US34 near Sta. 347+00 that was built up with about 10 m thick embankment fill material. Tension cracks were observed on the slope as shown in Figure 14, which suggests the possibility of slope movements.

FWD test results measurements obtained near joints and mid-panel are presented in Figure 15 to Figure 22. The figures identify zones of cracked panels and cut or fill. All of the cracked panels are located in the cut areas.

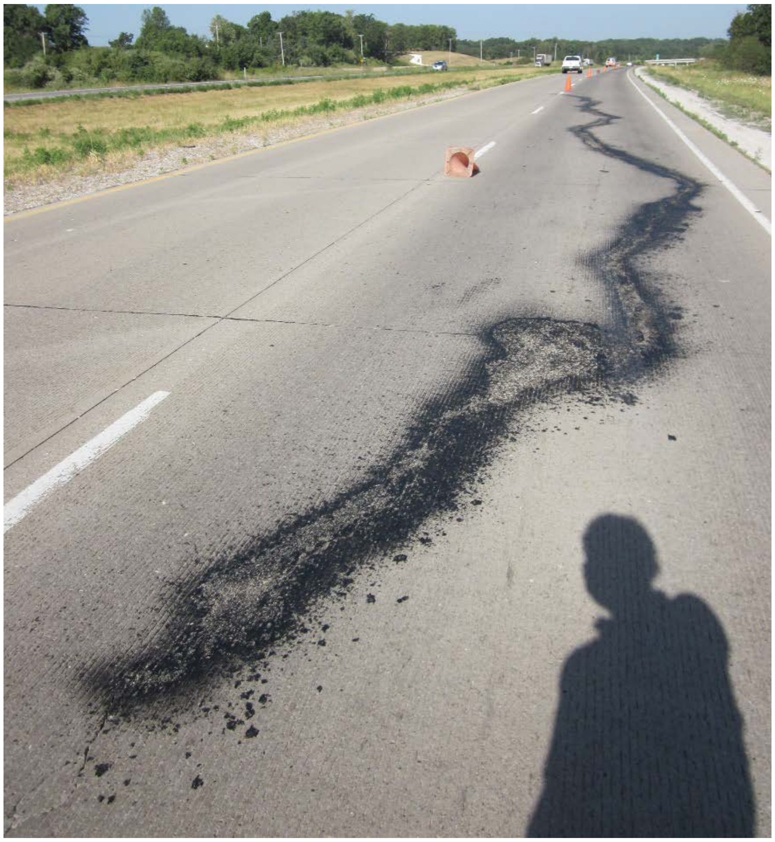

Figure 4. Longitudinal cracking near Sta. 350+00 (7/27/12)

Figure 5. Faulting measured along longitudinal crack near Sta. 350+00 (7/27/12)

Figure 6. Corner cracking observed near Sta. 350+25 (7/27/12)

Figure 7. Mid panel cracking observed near Sta. 349 (7/27/12)

Figure 8. Longitudinal cracking near Sta. 348+50 near mile post 194 (7/27/12)

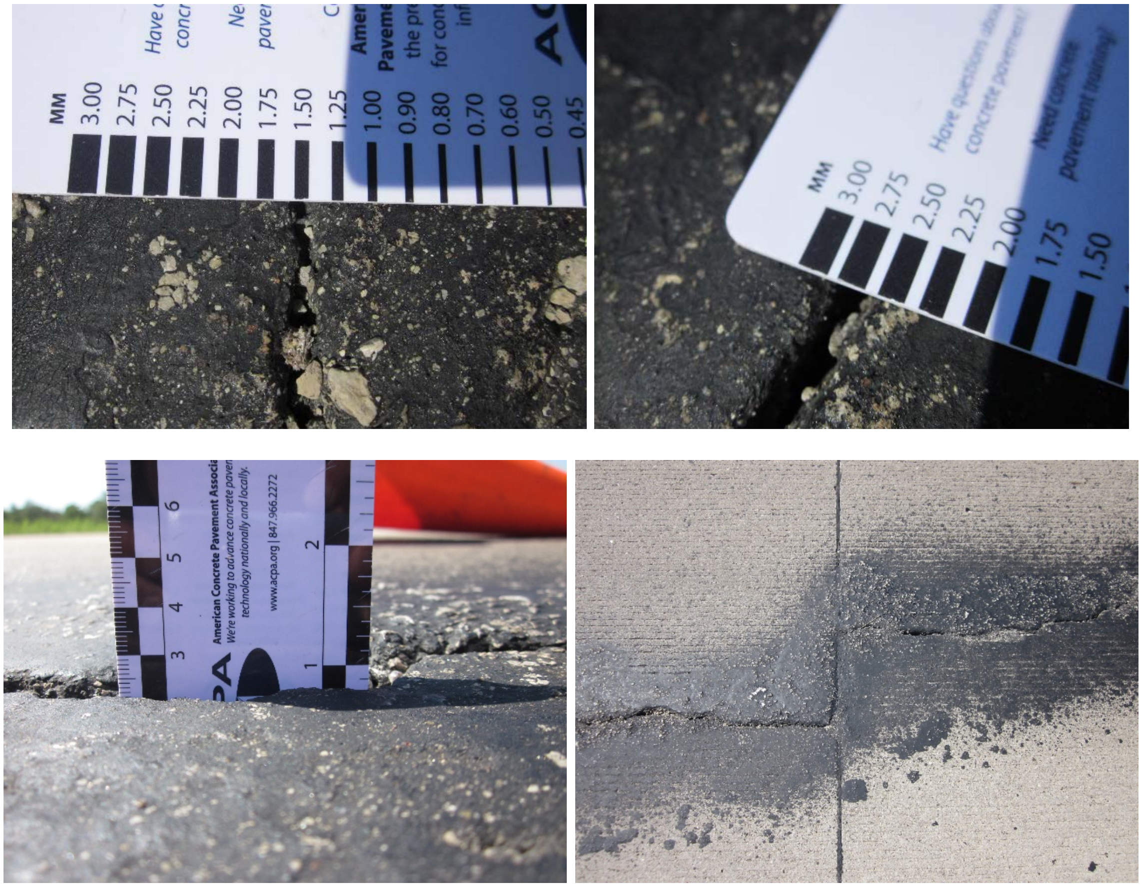

Figure 9. Close-up views of the cracks near mile post 194 (7/27/12)

Figure 10. Midpanel cracking near on panel 32 near mile post 194 (7/27/12)

Figure 11. Corner cracking on panel 84 (7/27/12)

Figure 12. Longitudinal and mid-panel cracking on panel 87 (7/27/12)

Figure 13. Looking down the creek valley near Sta. 347+00 (7/27/12).

Figure 14. Cracks observed on embankment fill slope near Sta. 347+00 (7/27/12)

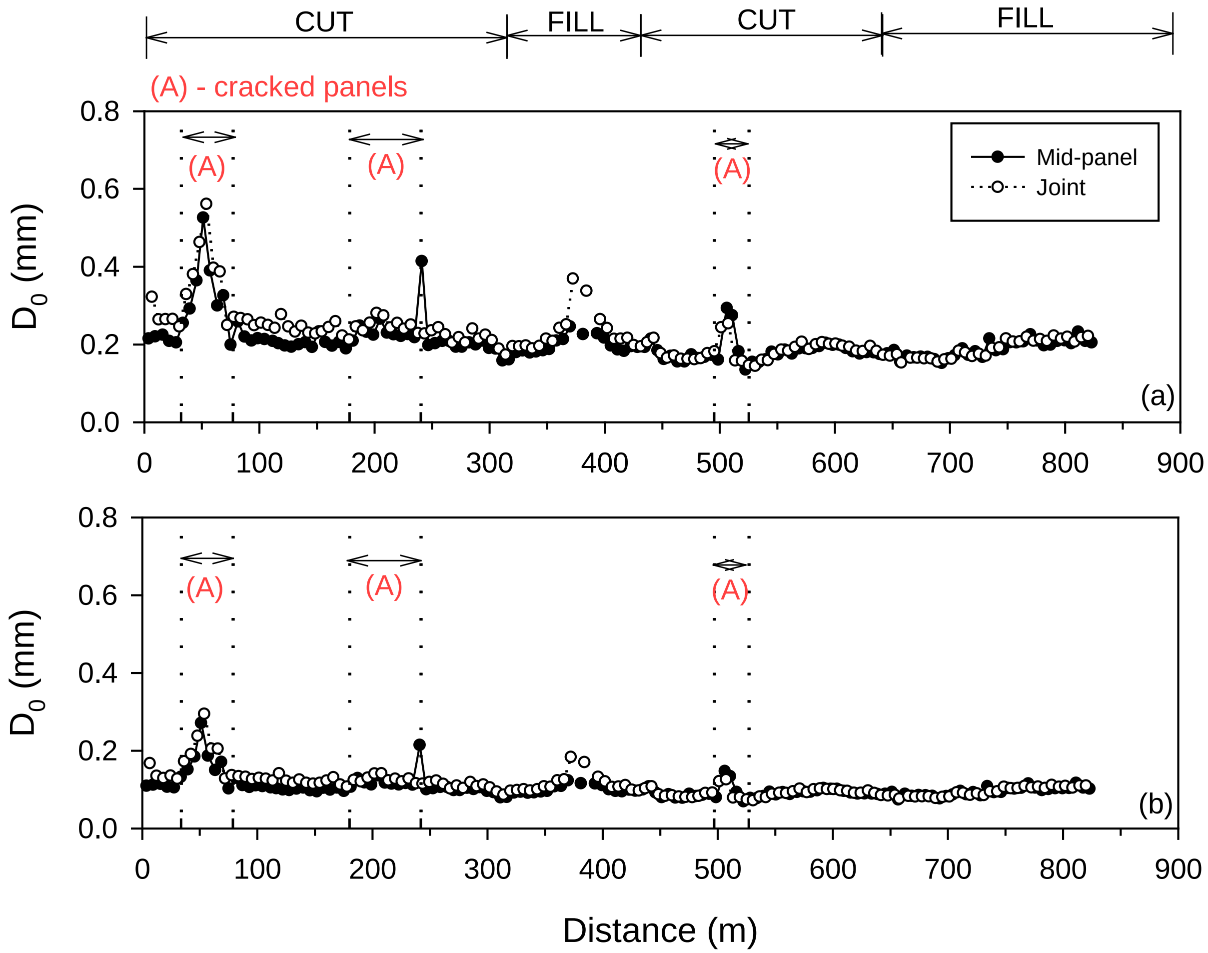

Figure 15. FWD D_0 versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

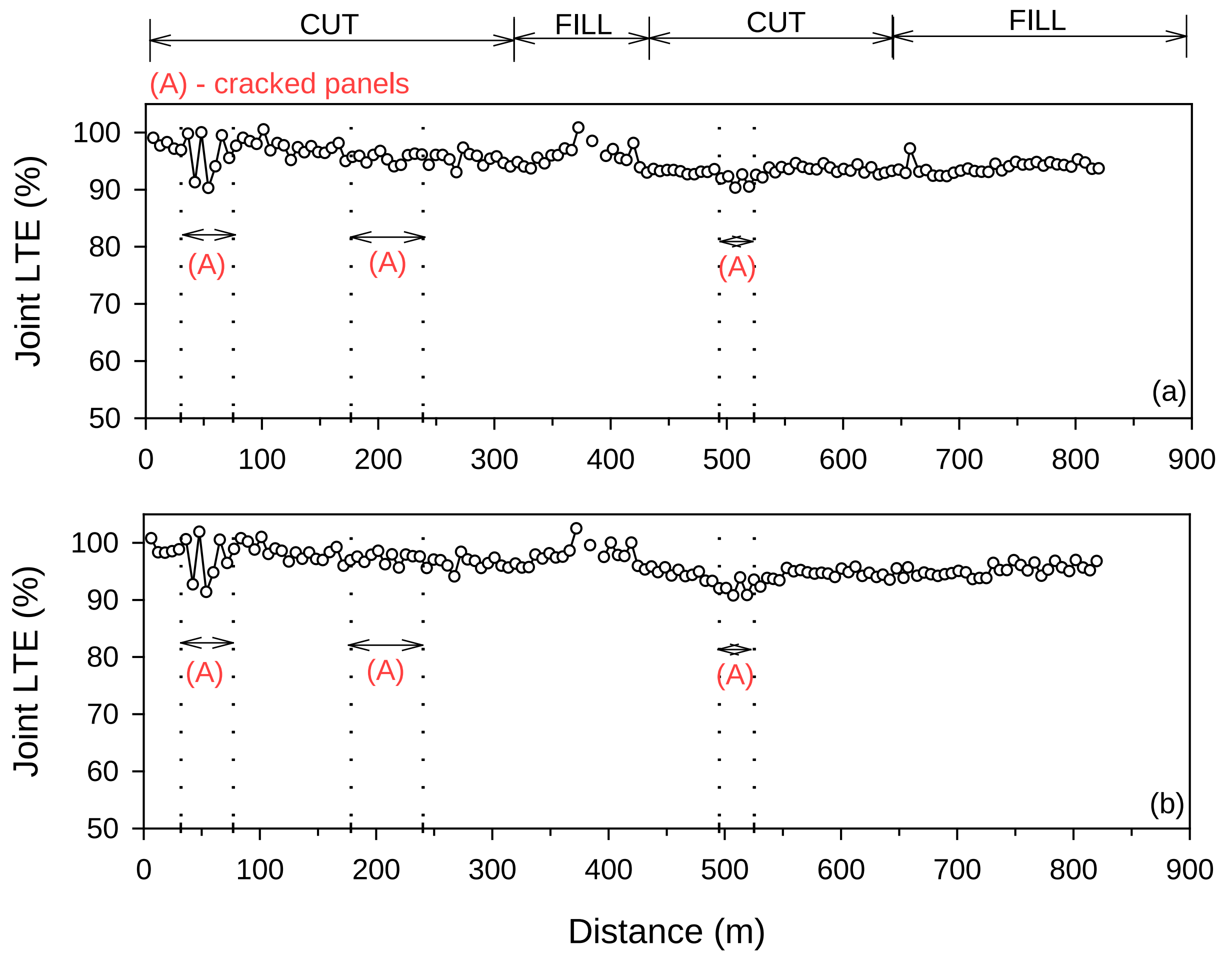

Figure 16. Joint LTE from FWD tests versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

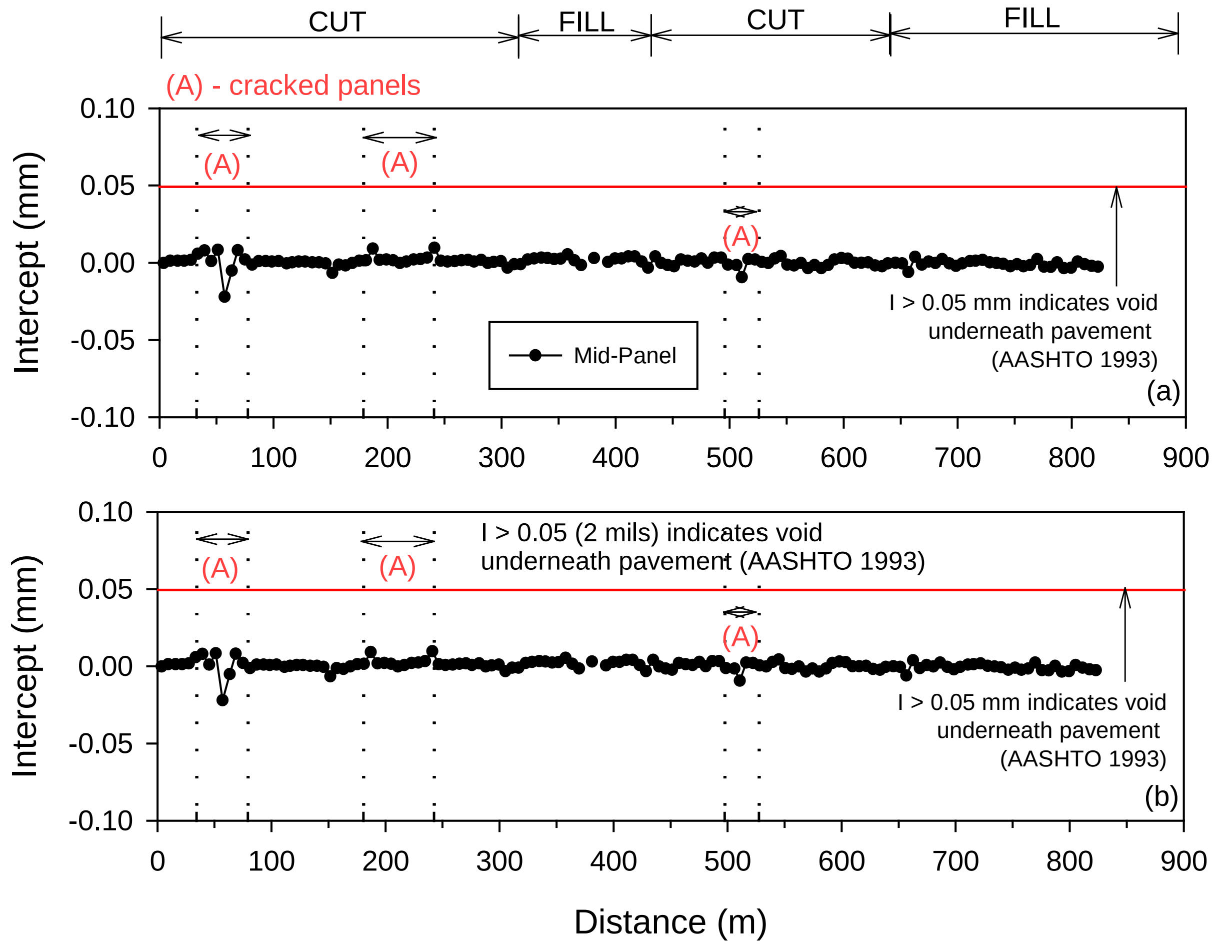

Figure 17. I-value versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

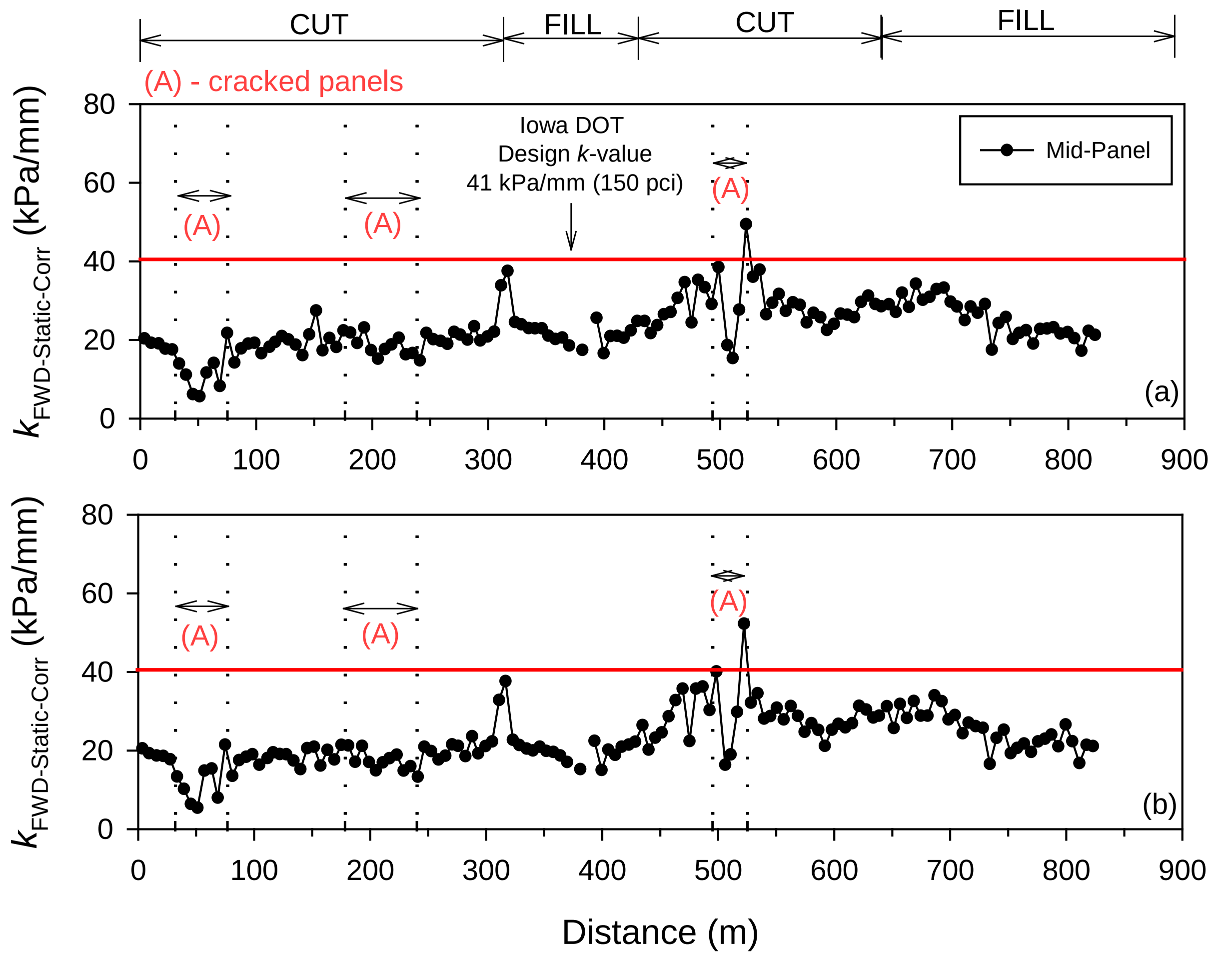

Figure 18. kFWD-Static-Corr versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

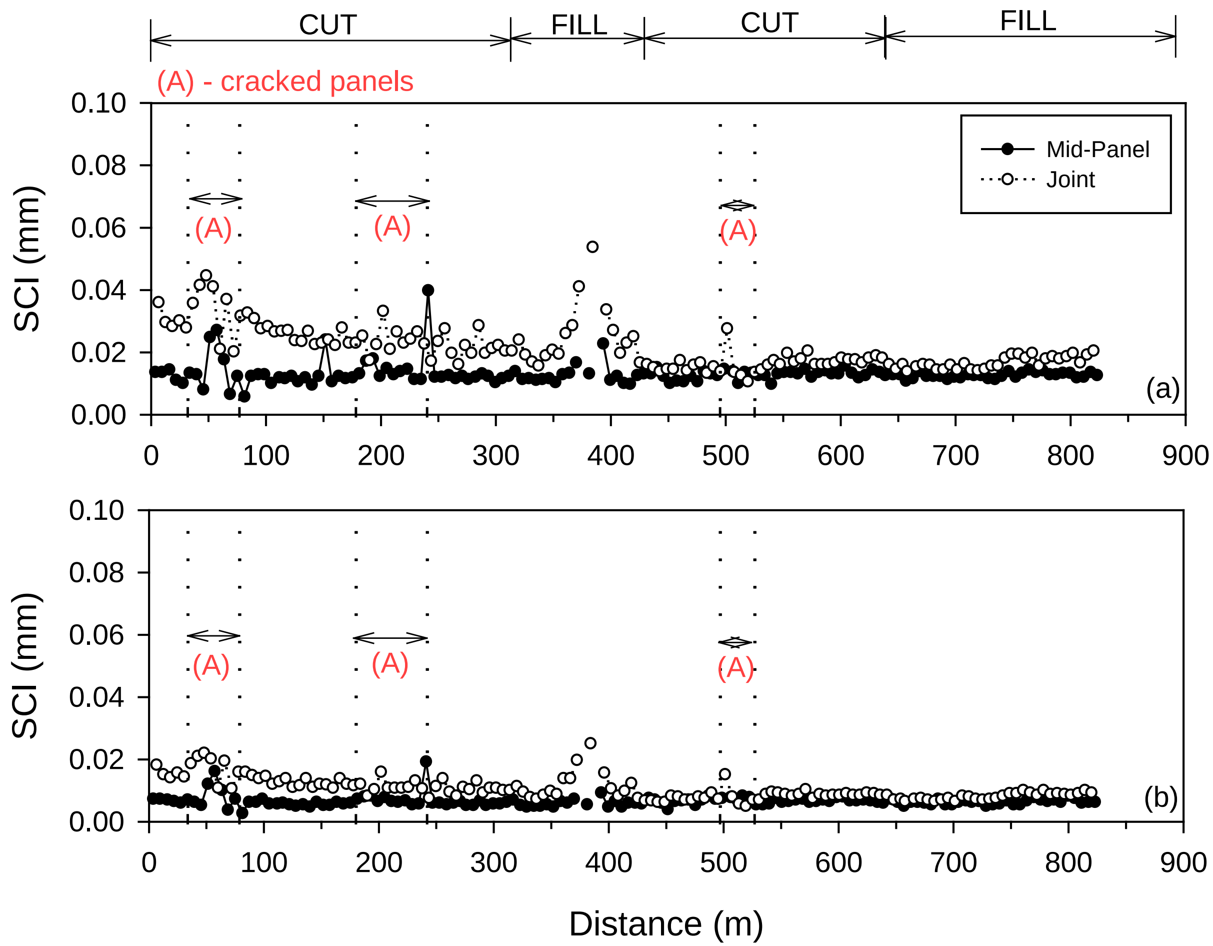

Figure 19. SCI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

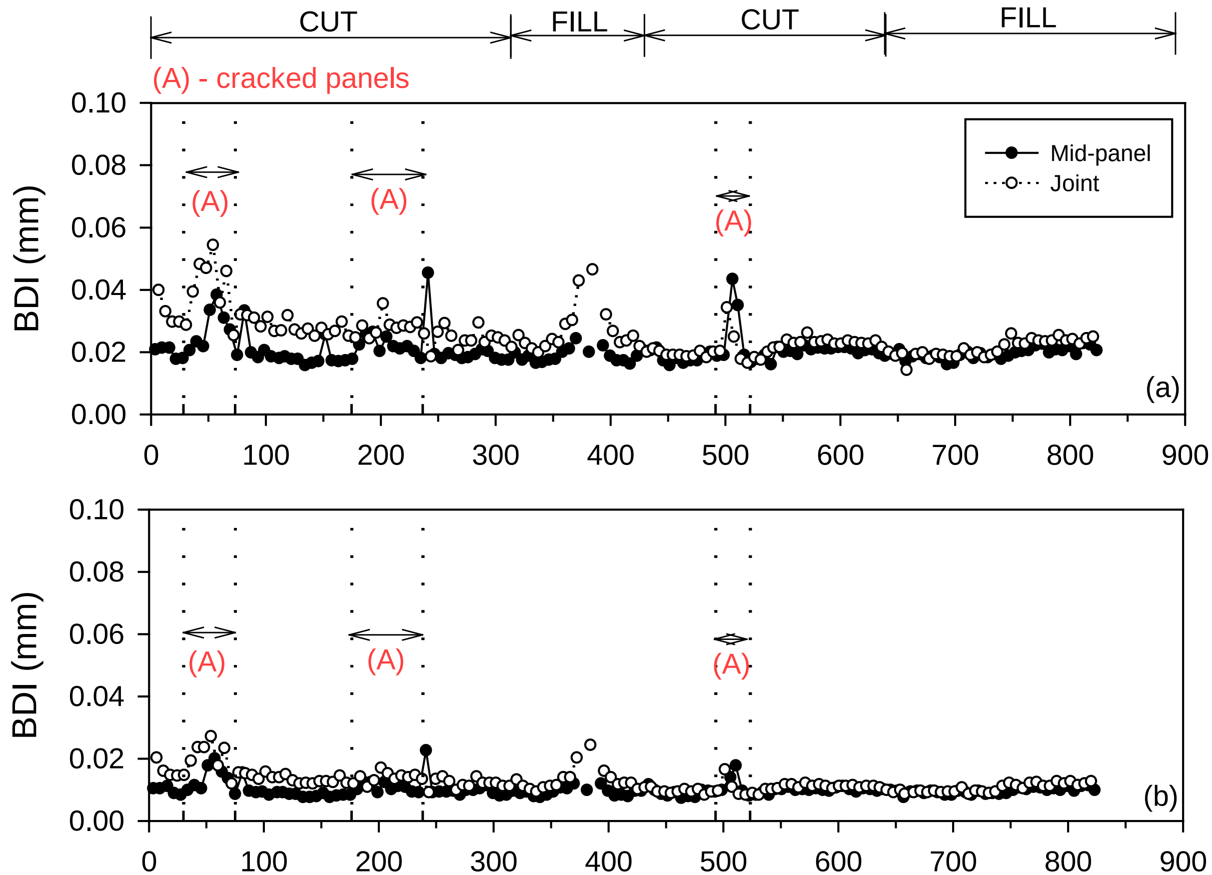

Figure 20. BDI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

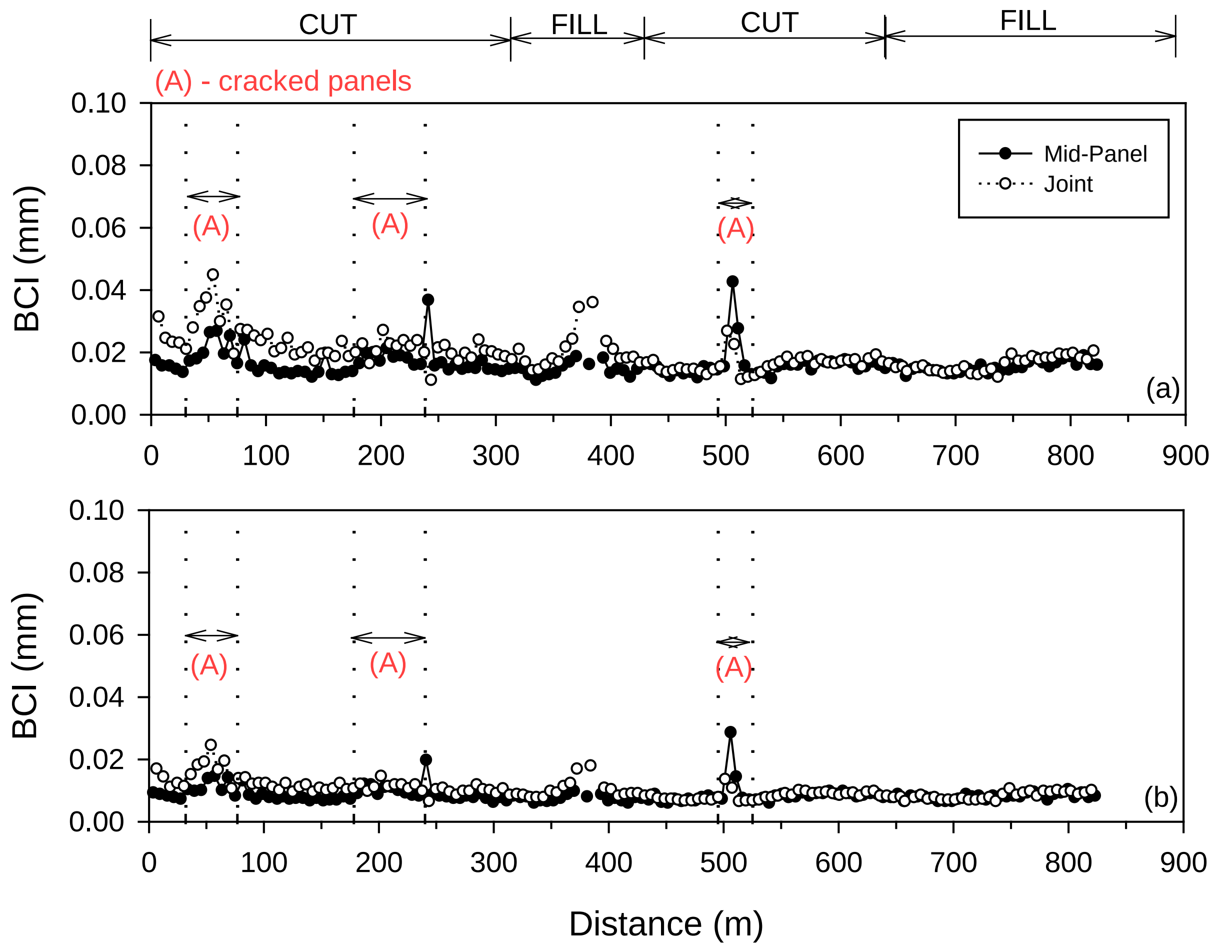

Figure 21. BCI versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

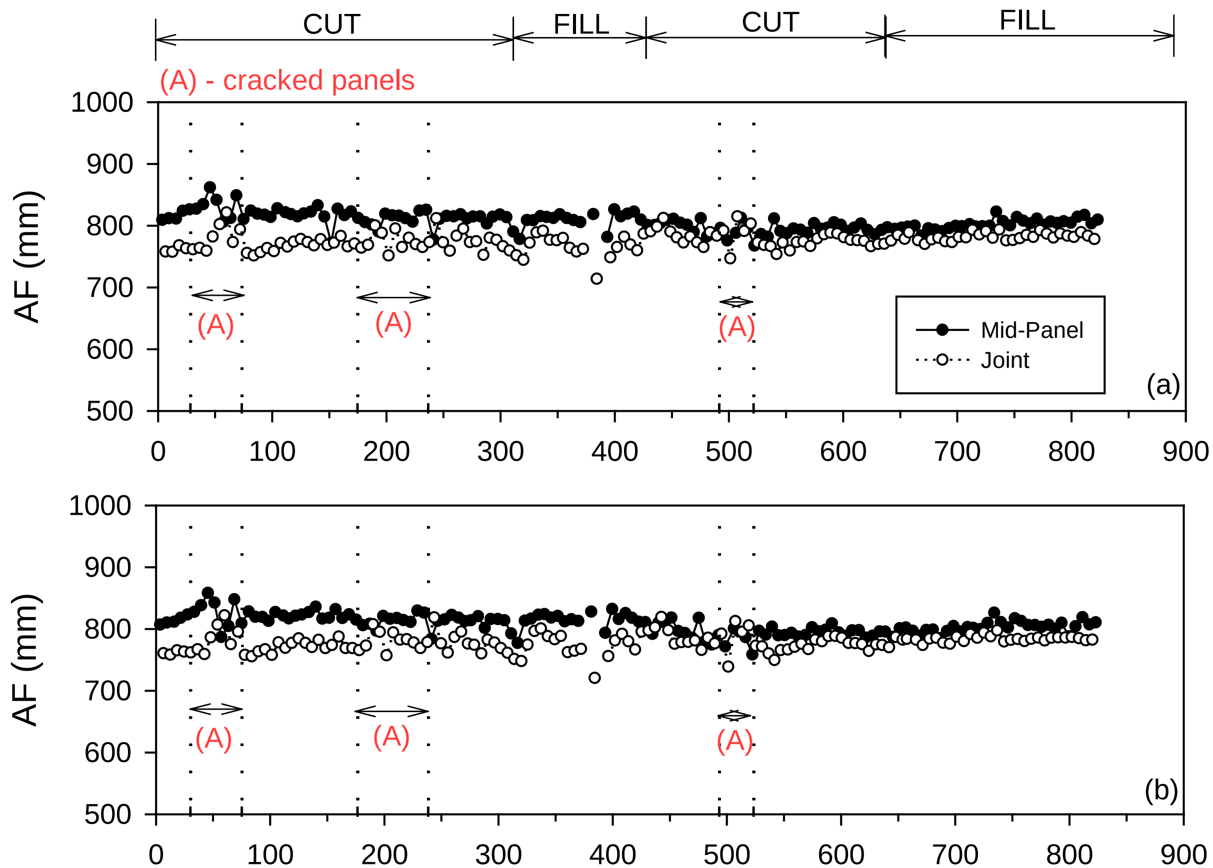

Figure 22. AF versus distance with applied loads of (a) 80 kN (18,000 lb) and (b) 40 kN (9,000 lb) [Distance 0 = Sta 350+50]

Box plots showing FWD test measurements obtained on panels with and without cracks are shown along with number of measurements (n), mean, and standard deviation statistics in Figure 23 and Figure 24 for tests conducted near mid-panel and joint, respectively. Summaries of t-test analysis results that compare measurement values obtained on panels with and without cracks are provided in Table 1 for tests conducted near mid-panel and Table 2 for tests near joint.

Following are the key findings from the statistical analysis test results:

- The D_0, $k_{\text{FWD-Static-Corr}}$, SCI, BDI, and BCI values showed statistically significant differences between cracked and uncracked panels, with results on uncracked panels representing better support conditions than on cracked panels.

- The $k_{\text{FWD-Static-Corr}}$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the *k* values were higher under cracked panels (38%) than under the uncracked panels (23%).

- There was no statistically significant difference in the I values between the cracked and uncracked panels. The I values were all very low ($\leq$ 1 μm). I > 5 μm is typically considered a trigger value suggesting void beneath the pavement.

- The joint LTE at all panels was relatively high (> 91%) and there was no statistically significant difference between the cracked and the uncracked panels.

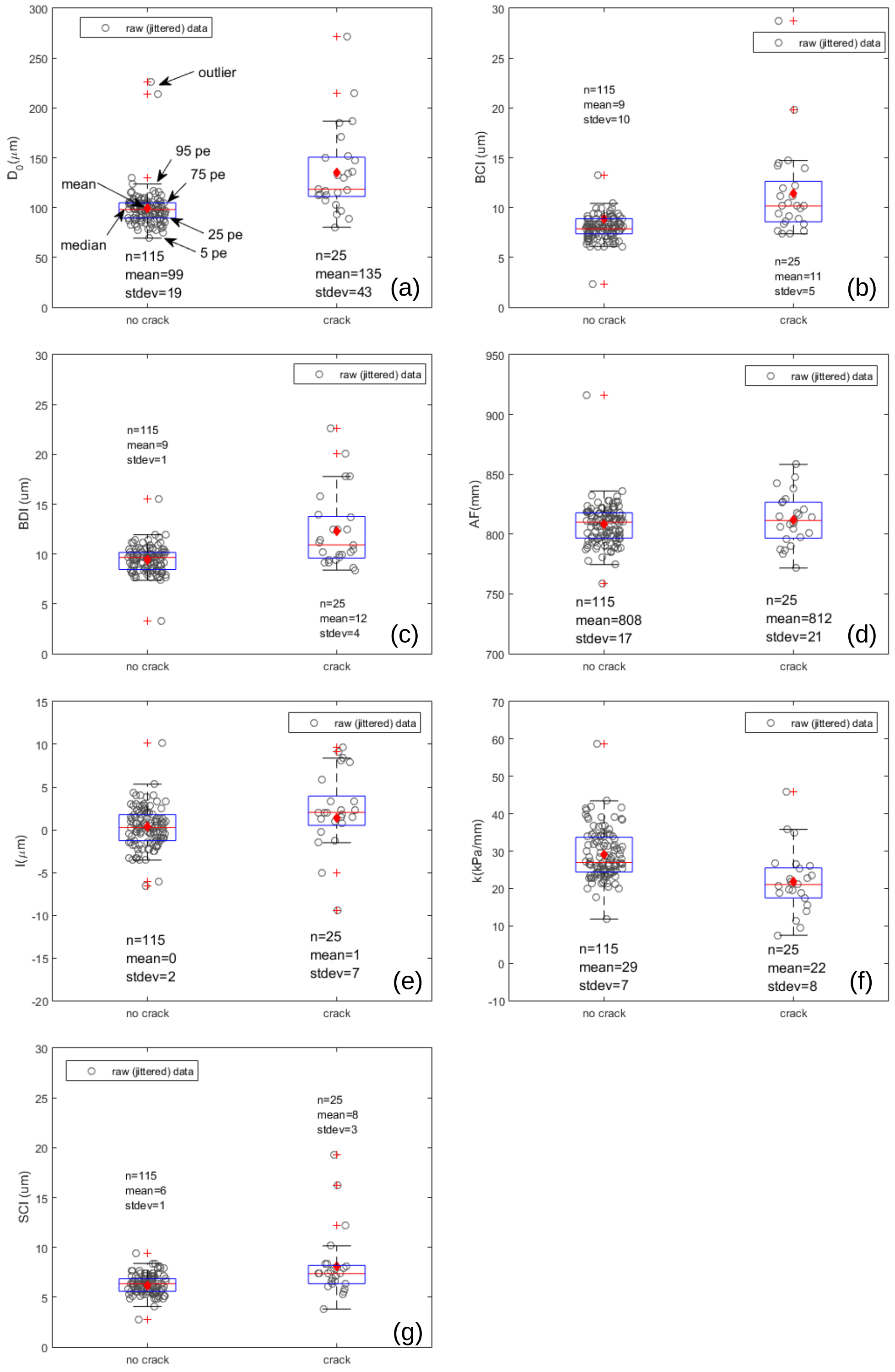

Figure 23. Box plots of FWD deflection basin parameters near mid-panel comparing panels with and without cracks: (a) D_0, (b) BCI, (c) BDI, (d) AF, (e) I-value, (f) $k_{FWD\text{-}Static\text{-}Corr}$, and (g) SCI

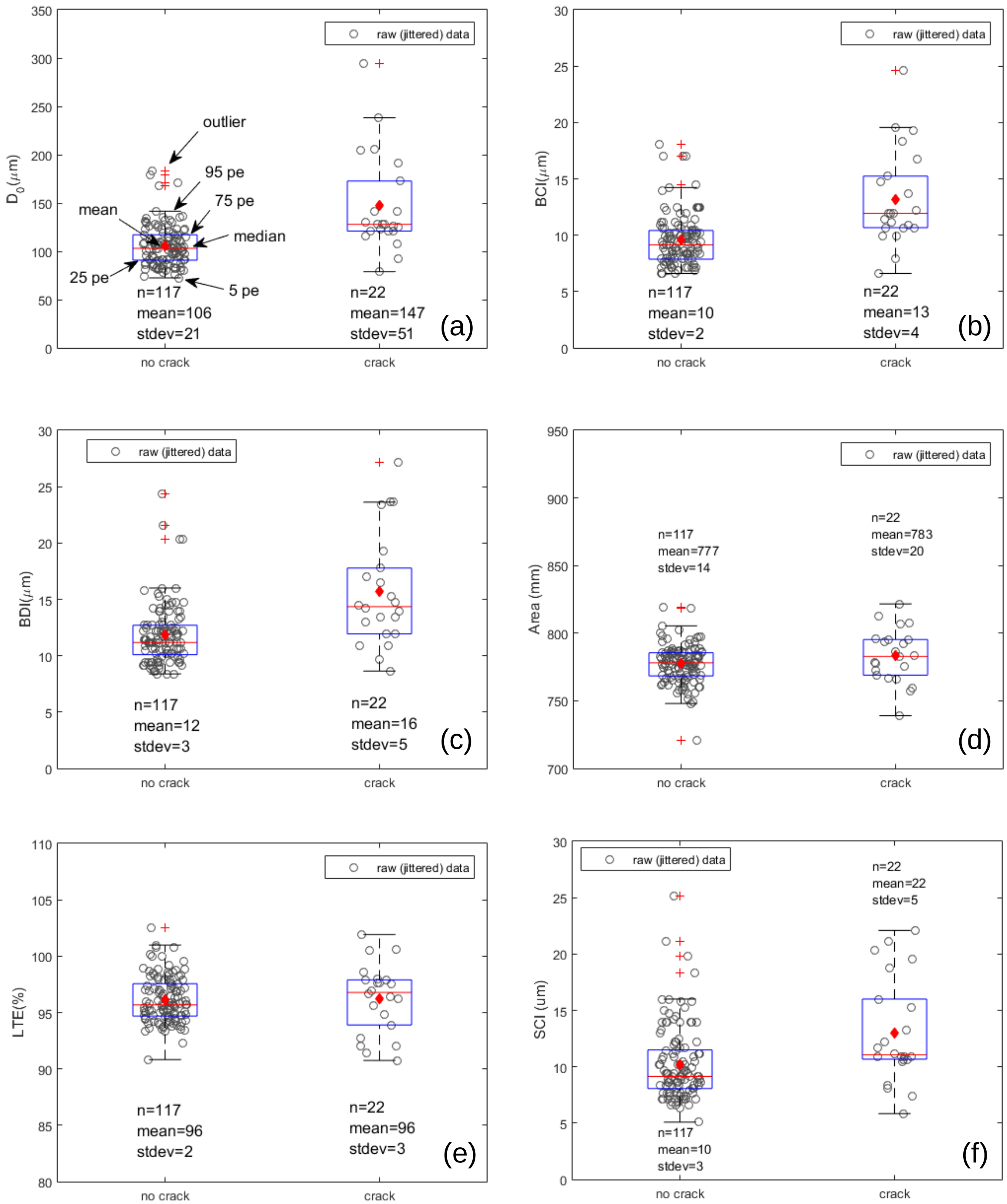

Figure 24. Box plots of FWD deflection basin parameters near joint comparing panels with and without cracks: (a) D_0, (b) BCI, (c) BDI, (d) AF, (e) joint LTE, and (f) SCI

Table 1. Summary of *t* test analysis results on FWD deflection basin parameters near mid-panel on cracked versus uncracked panels

Parameter	No crack or crack	Mean	COV (%)	t-value	P_r
D_0 (µm)	No Crack	99	20	-4.07	< 0.001
	Crack	135	32		
I (µm)	No Crack	< 1	760	-8.1	0.212
	Crack	1	473		
$k_{FWD\text{-}Static\text{-}Corr}$ (kPa/mm)	No Crack	29	23	4.06	< 0.001
	Crack	22	38		
SCI (µm)	No Crack	6	23	-2.82	0.005
	Crack	8	41		
BDI (µm)	No Crack	9	14	-3.71	< 0.001
	Crack	12	31		
BCI (µm)	No Crack	9	109	-1.99	0.025
	Crack	11	41		
AF (mm)	No Crack	808	2	-0.93	0.18
	Crack	812	3		

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cracked and the uncracked panels

Table 2. Summary of *t* test analysis results on FWD deflection basin parameters near joint on uncracked versus cracked panels

Parameter	No crack or crack	Mean	COV (%)	t-value	P_r
D_0 (µm)	No Crack	106	20	-3.73	< 0.001
	Crack	147	35		
LTE (%)	No Crack	96	2	-0.11	0.46
	Crack	96	3		
SCI (µm)	No Crack	10	32	-2.73	0.006
	Crack	13	36		
BDI (µm)	No Crack	12	22	-3.54	< 0.001
	Crack	16	32		
BCI (µm)	No Crack	10	24	-3.88	< 0.001
	Crack	13	32		
AF (mm)	No Crack	777	2	-1.35	0.094
	Crack	783	3		

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cracked and the uncracked panels

Box plots showing FWD test measurements obtained in cut and fill areas are shown along with number of measurements (n), mean, and standard deviation statistics in Figure 25 and Figure 26 for tests conducted near mid-panel and joint, respectively. Summary of t-test analysis results comparing measuremet values obtained on panels in cut and fill areas as provided in Table 3 and Table 4, respectively.

Following are the key findings from the statistical analysis test results:

- The D_0, $k_{\text{FWD-Static-Corr}}$, SCI, BDI, and BCI values showed statistically significant differences between cut and fill areas, with results in fill areas showing better support conditions than in cut areas. As indicated earlier, all cracked panels were located in the cut areas.

- The $k_{\text{FWD-Static-Corr}}$ values were on average about 1.1 times lower in cut areas than in fill areas. The COV of the *k* values were higher in the cut areas (31%) than in the fill areas (21%).

- There was no statistically significant difference in the I values cut and fill areas. The I values were all very low (≤ 1 μm).

- The joint LTE at all panels was relatively high (> 91%) and there was no statistically significant difference between tests conducted in cut and fill areas.

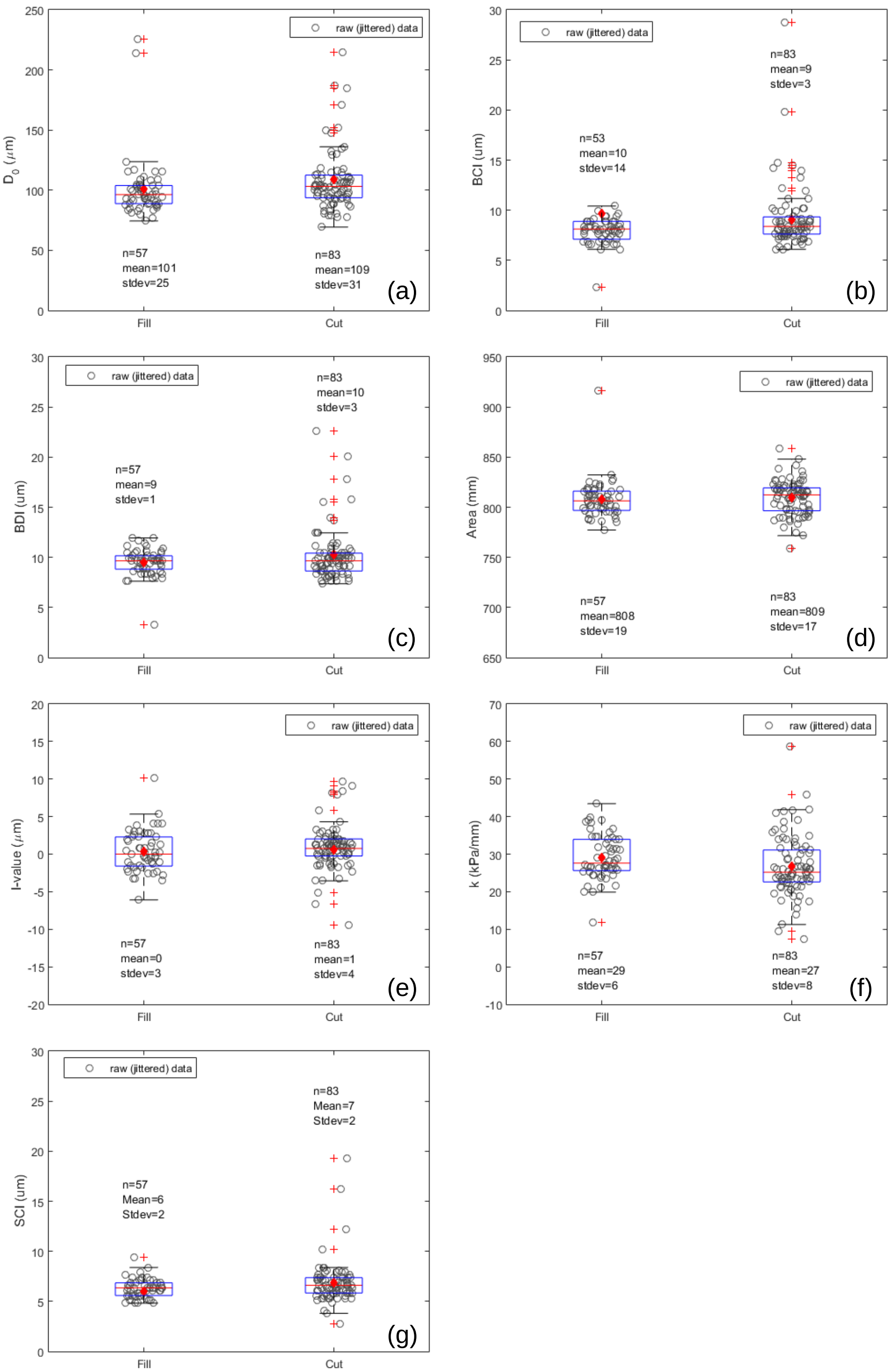

Figure 25. Box plots of FWD deflection basin parameters near mid-panel comparing panels located in fill and cut areas: (a) D_0, (b) BCI, (c) BDI, (d) AF, (e) I-value, (f) $k_{FWD\text{-}Static\text{-}Corr}$, and (g) SCI

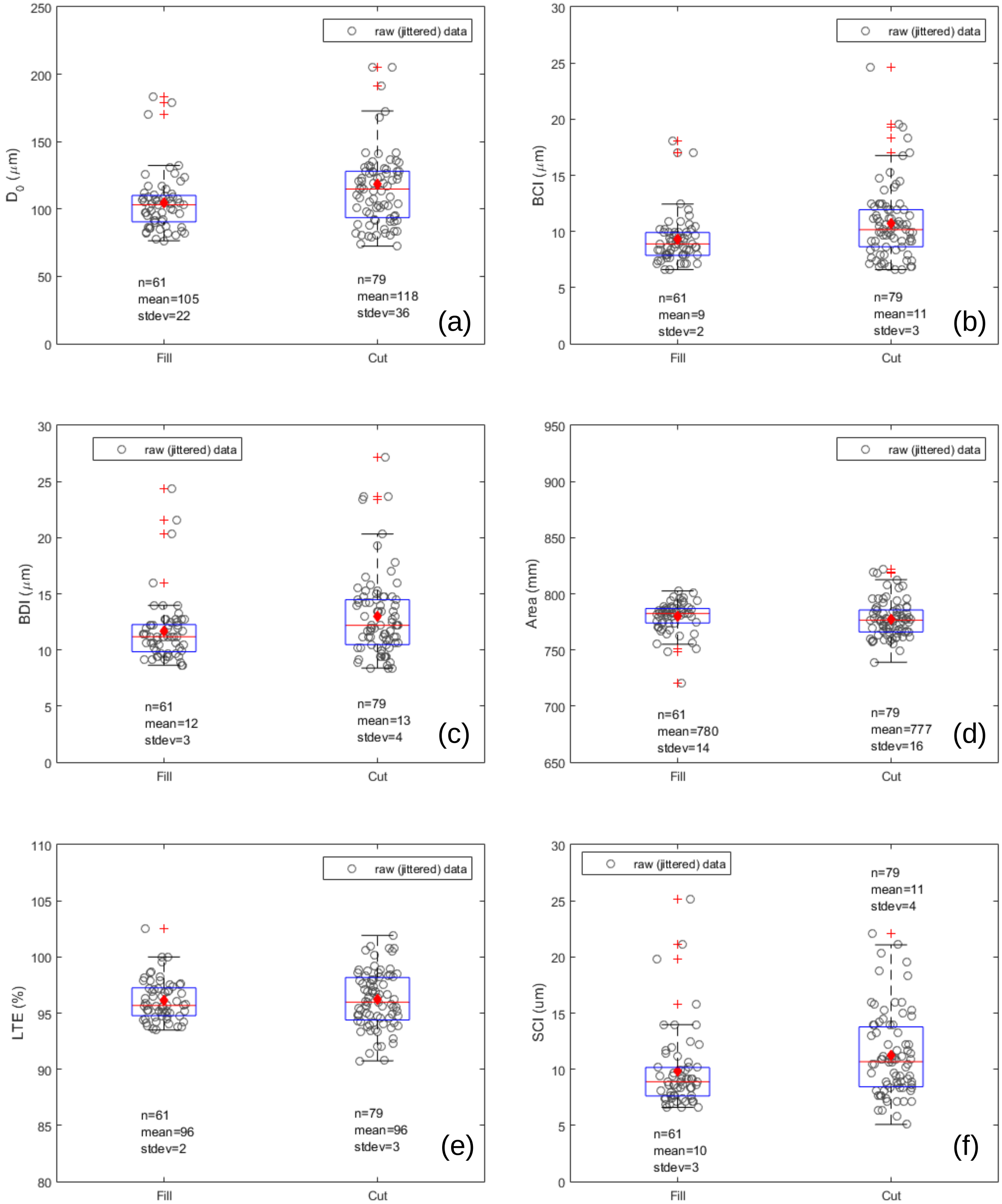

Figure 26. Box plots of FWD deflection basin parameters near joint comparing panels located in cut and fill areas: (a) D_0, (b) BCI, (c) BDI, (d) AF, (e) joint LTE, and (f) SCI

Table 3. Summary of *t* test analysis results on FWD deflection basin parameters near mid-panel in cut versus fill areas

Parameter	Fill or Cut	Mean	COV (%)	t-value	P_r
D_0 (µm)	Fill	101	25	-1.77	0.039
	Cut	109	28		
I (µm)	Fill	0	791	-0.47	0.319
	Cut	1	644		
$k_{FWD\text{-}Static\text{-}Corr}$ (kPa/mm)	Fill	29	21	2.04	0.022
	Cut	27	31		
SCI (µm)	Fill	6	30	-2.46	0.008
	Cut	7	32		
BDI (µm)	Fill	9	14	-2.15	0.017
	Cut	10	26		
BCI (µm)	Fill	10	142	0.342	0.367
	Cut	9	34		
AF (mm)	Fill	808	2	-0.42	0.337
	Cut	809	2		

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cut and fill areas

Table 4. Summary of *t* test analysis results on FWD deflection basin parameters near joint in cut versus fill areas

Parameter	Cut or Fill	Mean	COV (%)	t-value	P_r
D_0 (µm)	Fill	105	21	-2.78	0.003
	Cut	118	31		
LTE (%)	Fill	96	2	-0.08	0.467
	Cut	96	3		
SCI (µm)	Fill	10	36	-2.35	0.010
	Cut	11	33		
BDI (µm)	Fill	12	25	-2.42	0.008
	Cut	13	28		
BCI (µm)	Fill	9	24	-3.07	0.001
	Cut	11	31		
AF (mm)	Fill	780	2	0.92	0.180
	Cut	777	2		

Note: Highlighted cell indicates statistically significant difference at 95% confidence level between the cut and fill areas

CHAPTER 4. SUMMARY AND CONCLUSIONS

This report presented the field observations of the ISU research team and results and analysis of in situ falling weight deflectometer tests conducted on US34 WB between mile posts 194.5 and 196.7. FWD tests were conducted to evaluate differences in the deflection basin parameters and the modulus of subgrade reaction (k) values between the cracked and uncracked panels, and cut and fill areas. Statistical t-test analysis was conducted to compare the measurement values obtained on panels with and without cracks and in cut and fill areas. Pictures documenting the distresses observed on the pavement surface and cracks observed on embankment fill slopes are presented in this report.

Follwing are the key findings from this study:

- All of the cracked panels were located in the cut areas. Distresses observed on the pavement surface included longitudinal cracks, transverse cracks, mid-panel cracks, corner cracks, and faulting.
- Tension cracks were observed on the slope where about 10 m thick embankment fill was placed, which suggest possibility of slope movements.
- The D_0, $k_{\text{FWD-Static-Corr}}$, SCI, BDI, and BCI values showed statistically significant differences between cracked and uncracked panels, with results on the uncracked panels representing better support conditions than on the cracked panels.
- The D_0, $k_{\text{FWD-Static-Corr}}$, SCI, BDI, and BCI values showed statistically significant differences between cut and fill areas, with results in the fill areas showing better support conditions than in the cut areas. (Note that all cracked panels were located in the cut area).
- The $k_{\text{FWD-Static-Corr}}$ values were on average about 1.3 times lower under cracked panels than under uncracked panels. The COV of the k values were higher under the cracked panels (38%) than under the uncracked panels (23%).
- The $k_{\text{FWD-Static-Corr}}$ values were on average about 1.1 times lower in cut areas than in fill areas. The COV of the k values were higher in the cut areas (31%) than in the fill areas (21%).
- There was no statistically significant difference in the I values between the cracked and uncracked panels and the cut and fill areas. The I values were all very low (≤ 1 μm). I > 5 μm is typically considered a trigger value suggesting void beneath the pavement.
- The joint LTE at all panels was relatively high (> 91%) and there was no statistically significant difference between the cracked and the uncracked panels and the cut and fill areas.

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