EVALUATION OF PERFORMANCE OF WHITETOPPING PAVEMENT AS A REHABILITATION STRATEGY

By

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To the Faculty of Washington State University:

The members of the Committee appointed to examine the thesis of XIAOJUN LI find it satisfactory and recommend that it be accepted.

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EVALUATION OF PERFORMANCE OF WHITETOPPING

PAVEMENT AS A REHABILITATION STRATEGY

Abstract

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Whitetopping overlay is a relatively new rehabilitation technology for deteriorated asphalt pavement. It is defined as a Portland cement concrete (PCC) overlay on the deteriorated existing hot mixed asphalt (HMA) pavement to improve both the structural and functional capability. When the PCC overlay thickness is less than or equal to 4in., it is referred to as ultra-thin whitetopping (UTW).

The primary objectives of this thesis are to evaluate the performance of whitetopping pavement as a rehabilitation strategy and study the falling weight deflectometer (FWD) backcalculation method for whitetopping. Forensic investigation was conducted including FWD test, field distress survey, and shear strength test.

Traditional FWD backcalculation methods for whitetopping pavement layer properties, like Modcomp6 and equations based on “AREA” theory, were found not applicable to UTW. A new Critical Distance Method is proposed and its potential advantages are highlighted. The parameter of “relative slab stiffness” could be used to

estimate the backcalculation error and as a criterion to select suitable FWD backcalculation method for UTW.

Thermal stress has little effect on typical UTW overlay. However, for WT overlay with the joint spacing and slab thickness increased, thermal stress could become a major cause of fatigue.

3-D finite element modeling and fatigue life analysis indicated that whitetopping pavement is very sensitive to a load level higher than the 18-kip standard axle loads. Slightly increase in the axle load could significantly decrease the fatigue live of whitetopping pavements. Design of whitetopping should be based on a traffic spectrum or heavier loads than the 18-kip standard axle load.

Slab thickness, slab size, and overlay age were found to be statistically significant variables that affect the performance of whitetopping pavements. Slab thickness should be thicker than 4 in. and slab size should be smaller than 36 sq. ft. to get better whitetopping pavement performance.Coring and shear strength testing indicated that the bond was lost quickly in the field. The design method of whitetopping should fully consider the real bond condition, or based on unbond condition, to be conservative.

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Dedication

This thesis is dedicated to my wife, my lovely daughter, and my new born son, who provided great emotional support and gave me so much pleasure

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Traditionally, the most common rehabilitation method for existing hot mix asphalt (HMA) pavement is an asphalt overlay. However, the performance of HMA overlay is very sensitive to the conditions of the underlying HMA pavement. Wen et al. (2006) studied the performance of HMA overlay on existing HMA or Portland cement concrete (PCC) pavements in Wisconsin. For an overlay of HMA pavement, it was found that rutting in the underlying HMA pavement could recur in the asphalt overlay and that cracks in the existing pavement could be reflected in the HMA overlay.

Whitetopping overlay is a relatively new rehabilitation technology for deteriorated asphalt pavement. Whitetopping is defined as a PCC overlay on the prepared (for example, cold milled) existing HMA pavement to improve both the structural and functional capability. When the PCC overlay thickness is less than or equal to 4in., it is referred to as ultra-thin whitetopping (UTW) (Cole, L.W. 1997). Over the past two decades, whitetopping overlay has gained considerable interest and great acceptance as an alternative to HMA overlay (ACPA 2004). According ACPA (2004), whitetopping can provide many benefits, including long life, low maintenance, and low life-cycle cost. To be consistent with works done previously by others, in this study, the term “whitetopping” is used to refer to both WT and UTW in general. “WT” is used to refer to as concrete overlay thicker than 4 in. and “UTW” as overlay equal to/less than 4 in.

Many studies have been done focusing on the mechanical analysis, design and construction procedure, and performance of WT and UTW overlay. Lessons have been learned from these research projects to promote the development of WT and UTW overlays. The performance of whitetopping, especially UTW pavement has been found to be related to the special composite structure resulting from the bond at the PCC/HMA interface. The bond reduced the stresses in the PCC slabs by transferring more load to the underlying HMA layer (TRB 2004). A few major design and construction features affect the performance of whitetopping pavements, including the condition of the existing HMA, the pre-overlay treatment, concrete materials, joint spacing, and design method.

A number of WT or UTW projects have been built in Wisconsin, but to date, there has been no specific follow-up regarding their performance. Like projects in other states, individual projects in Wisconsin have shown mixed results in terms of performance. Causes for these large discrepancies need to be examined and understood so that they may be appropriately accounted for in design. Furthermore, estimates of the service life of WT and UTW projects need to be developed so that this rehabilitation technology can be appropriately incorporated into pavement life cycle cost analysis (LCCA). The performance assessment of whitetopping pavements will allow highway agencies to make informed decisions regarding the appropriate use of pavement improvement techniques.

1.2 RESEARCH OBJECTIVES

The primary objectives of this thesis are to evaluate the performance of whitetopping pavement as a rehabilitation strategy and study the falling weight deflectometer (FWD) backcalculation method for whitetopping.

Forensic investigation was conducted including FWD test, field distress survey, and shear strength test. FWD backcalculation methods were studied and a new Critical Distance Method was developed for UTW backcalculation. Performance of whitetopping pavement was evaluated by means of forensic investigation.

1.3 ORGANIZATION OF THE THESIS

This thesis describes the performance evaluation and service life estimate of whitetopping pavements in Wisconsin. Chapter 1 introduces the background and problem statement. Chapter 2 contains the literature review findings. Chapter 3 describes the evaluation methods on these whitetopping projects. Chapter 4 developed a new FWD backcalculation method for UTW. Chapter 5 describes the results of performance evaluation. Conclusions and recommendations are given in Chapter 6. References are provided in Chapter 7.

CHAPTER 2: LITERATURE REVIEW

2.1 INTRODUCTION

Whitetopping overlays provide the industry with an alternative to HMA overlays. A whitetopping overlay, which is defined as a Portland Cement Concrete (PCC) overlay over an existing hot mix asphalt (HMA) pavement, can be classified by thickness and by the bond type with the underlying HMA layer (Rasmussen and Rozycki 2004):

Conventional Whitetopping (WT)

Conventional WT thickness is typically more than 8 in. WT is designed and constructed without the need to consider the bond strength between the PCC and the underlying HMA layer.

Thin White-Topping (TWT)

TWT thickness is typically between 4 in. and 8 in. In general, the TWT is designed with consideration of establishing a reasonable bond between the PCC and the underlying HMA layer.

Ultra-Thin Whitetopping (UTW)

UTW thickness is typically between 2 in. and 4 in. The UTW requires a good bond with the underlying HMA layer to perform well as indicated by the literature (Cole 1997; Rasmussen et al. 2002; Lin and Wang 2005).

The type of bond between the PCC overlay and the underlying HMA layer is important, especially for UTW, because the bond reduces the stresses in the thin PCC layer by transferring some of the load to the underlying HMA layer. Figure 1 illustrates the difference between the stress behavior of bonded and unbonded overlays.

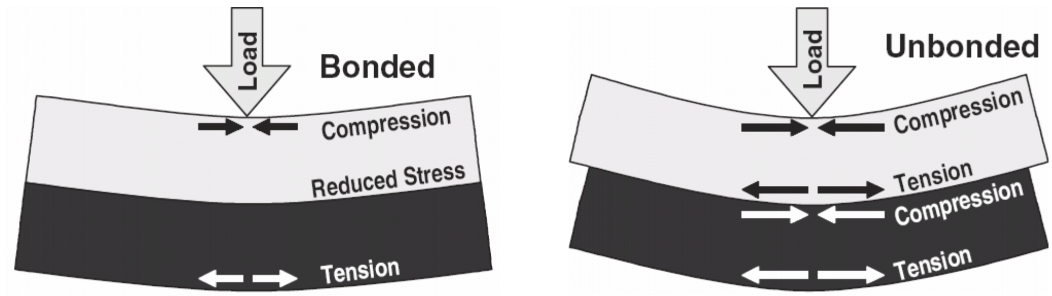

Figure 1: Bonded Vs. Unbonded behavior (Rasmussen et al. 2004)

As mentioned earlier, in this report, the term “whitetopping” is used to refer to any PCC overlay on existing HMA pavement, while WT and UTW refer to whitetopping with slab thickness of more than 4 in. and 4 in. or less, respectively. One of the earliest uses of whitetopping as a maintenance and rehabilitation method of pavements occurred in 1918 (Tarr et al. 2000). A comprehensive survey of UTW projects (Cole 1997) documented 189 concrete resurfacings of asphalt pavements on highways, airfields, streets, and county roads. These projects are located in 33 states, with thicknesses ranging from 4 in. for city streets to 18 in. for airfields.

Both UTW and WT are intended to correct structural and functional distress in an existing HMA pavement at a cost that is comparable to that of an HMA overlay, especially if a LCCA was used in the planning (Rasmussen and Rozycki 2004). The PCC surface has good durability and long term performance and that it decreases the maintenance time and life cycle cost of the pavement (Tarr et al. 2000). This is supported by a study of whitetopping projects in the state of Nebraska (Rea and Jensen 2005).

For example, an early experimental usage of UTW in Louisville, KY, with thicknesses of 2 in. and 3.5 in. and with a traffic loading of 400 to 600 trucks for 5.5 day per week, still performs well years after the initial construction (Cole 1997). This showed that UTW is applicable for low volume roads, residential streets, and parking lots (Lin and Wang 2005). However, the design of whitetopping needs to be done correctly. The literature indicates that insufficient thickness of whitetopping overlay, long joints, and weak underlying HMA pavement resulted in premature failure (WCPA 1999; Rasmussen et al. 2002; Lin and Wang 2005).

2.2 WHITETOPPING OVERLAY DESIGN

A general guideline for whitetopping construction was available as early as 1989 from the Portland Cement Association (PCA) and the American Concrete Pavement Association (PCA 1989; ACPA 1991; ACPA 1999). However, the design thickness methodology and guideline was not available until the development of the PCA UTW design procedure (Mack et al. 1997; ACPA 1997; Wu et al. 1998). This approach assumed a partial bond between the PCC overlay and the underlying HMA, instead of “fully bonded” or “completely unbonded” as in the previous design methods. This was followed by the state of Colorado and PCA investigation on WT pavements behavior under heavy traffic (Tarr et al. 1998). The state of Colorado and PCA study is similar to the earlier PCA study on UTW. The state of Colorado and PCA study found that there are performance differences between UTW and WT. Based on the findings, a procedure similar to PCA

PCC thickness design procedure (PCA 1984) was developed for thin whitetopping pavements.

Based on a review of the design guidelines, and the literature review, the design of a whitetopping overlay needs to consider and/or include the following factors in the design phase:

- the condition of the existing HMA
- the type of concrete materials used
- the slab thickness design
- the joint spacing design

2.2.1 Condition of the Existing HMA

The existing HMA pavement has deteriorated to some degree prior to the whitetopping overlay. Therefore, the condition of existing HMA affects the structural capacity of whitetopping pavement. Most agencies use a visual distress inspection method to assess the condition of existing asphalt pavements (NCHRP 2002). Although every state agency has different guidelines and methodology in doing the visual distress inspection, there are two standardized visual distress survey methods. This is an important point to mention since this study will compare the performance of whitetopping pavement in the state of Wisconsin with that in other published studies. The first one is the AASHTO (American Association of State Highway and Transportation Officials) Present Serviceability Index (PSI). To illustrate the use of this index, new pavement usually has a PSI value ranging from 4.0 to 4.5. Pavement is generally scheduled for resurfacing, rehabilitation, or replacement when the PSI approaches 2.5 (Rea and Jensen 2005). The second one is the PAVER SYSTEM Pavement Condition Index (PCI) (Shahin and Walther 1990). This index was used by Cole (1997) in surveying typical UTW performance. The PCI is calculated based on 19 different concrete pavement distresses using the American Society of Testing and Materials (ASTM) D5340 method. A newly built pavement typically has a PCI of 100, and a heavily deteriorated pavement has a PCI of 0. Rasmussen (2004) reported that falling weight deflectometer (FWD) testing or laboratory testing are more reliable methods (Rasmussen and Rozycki 2004) to determine the condition of existing HMA pavement. Examples of laboratory testing are wheel-track testing, and resilient or dynamic modulus measurement. Prior to 2008, the Wisconsin Department of Transportation (WisDOT) uses the pavement distress index (PDI) to quantify the conditions of pavements. Unlike PCI, a new pavement has a PDI of 0 and a PDI of 100 indicates the worst condition possible.

The thickness of the PCC overlay is heavily influenced by the condition of the existing HMA pavement. As shown in Figure 1, this is especially important for UTW pavement considering that the underlying/existing HMA pavement helps in reducing the stresses in the PCC overlay. The condition of the existing HMA layer can be improved by repairing existing distresses. Rasmussen (2002) reported that permanent deformation in the existing HMA layer may be a significant factor in the development of cracking on the PCC overlay layer. However, it may be costly to do the overlay repair. If the existing HMA layer is unable to provide good support to the WT layer, a thicker PCC overlay should be considered instead.

There are two common pre-overlay repair methods: milling, which is most common, and filling/patching. Besides creating a surface to provide a good bond between the existing HMA pavement and the PCC overlay, milling is able to remove any permanent deformation and smooth out any surface distortions. However, since milling reduces the thickness of the existing HMA layer, special attention needs to paid to the minimum thickness recommendation for the existing HMA. The ACPA guideline (1999) recommended a minimum of 3 in. of existing HMA. Another minimum thickness recommendation is 6 in. (Silfwerbrand 1997). Filling/patching is used to repair potholes and cracking in existing HMA pavement. Rasmussen (2004) reported that there are two types of distresses on existing HMA pavement that can indicate the existing HMA pavement may not be a good load carrying layer: extensive potholes and stripping. Extensive potholes may be an indication of weakened pavement structure. Stripping may be an indication of the excessive presence of moisture in the existing HMA pavement. The presence of moisture is hypothesized to reduce the bonding strength between the PCC overlay and the existing HMA layer. In both of these cases, a thicker PCC overlay should be considered.

In the American Concrete Pavement Association (ACPA) design guide (2002), the support by existing HMA pavement is converted into a k-value on the top of the HMA pavement which is then used to directly determine the thickness of WT slab. The k-value for the existing HMA pavement is determined by the k-value of the underlying subgrade, the thickness of the base layer, the type of the base layer, and the thickness of the existing HMA layer. Figure 2 is an example of the figure used in the ACPA design guide.

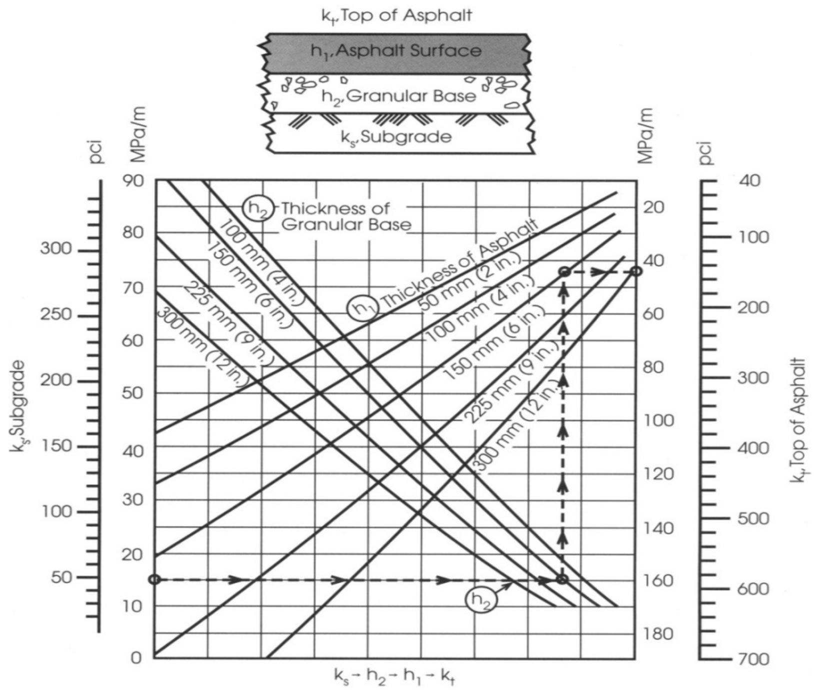

Figure 2: k-value on top of HMA pavement with granular base (ACPA 2002)

For the ACPA UTW pavement design, HMA thickness after milling and subgrade/sub-base k values are required to determine the slab thickness. When the HMA layer is too thin after milling (less than 3 in.), it is not a good candidate for UTW, as evidenced by the UTW study in Florida (Mia et al. 2002). With slabs of the same thickness, the support of existing asphalt pavements may vary significantly, largely due to the distresses and materials variation. Experimental tests of whitetopping pavements at the Federal Highway Administration (FHWA) accelerated loading facility (ALF) indicated that whitetopping pavement on a soft HMA layer was susceptible to slab cracking (Rasmussen et al. 2003).

2.2.2 Concrete Materials

The concrete mix for WT and UTW is not different than the concrete mix for standard PCC pavement. ACPA's WT design guide (2002) recommends that the concrete mix has a 28-day compressive strength of 4,000 psi, although concrete mixes with lesser compressive strength have been used with success. Rasmussen (2004) reported that aggregate thermal properties (coefficient of thermal expansion (CTE), thermal conductivity, and specific heat) and aggregate gradation needed to be considered in the concrete mix design. The CTE is of interest considering that the literature shows that there is a significant increase in the stresses in the WT layer due to the thermal gradients (Roessler 1998; Kumara et al. 2003; Lin and Wang 2005; and Wu et al. 2007).

Many whitetopping pavements feature fiber-reinforced concrete to reduce crack width, reduce surface spalling, and increase wear resistance (Rasmussen et al. 2004). This is due to relatively thin concrete slabs used in whitetopping pavements. This is especially important for UTW pavements. In the United States, most UTW pavements have used fibers in concrete (Rasmussen and Rozycki 2004). The types of fibers that have been used include fibrillated synthetic fibers, synthetic monofilament, and steel fibers. A common usage rate is about 1.8 kg/m3 (3 lbs/yd3) (Rasmussen and Rozycki 2004).

Many whitetopping pavements, especially UTW, including some in Wisconsin, featured fast-track construction using high early strength concrete to expedite the opening of pavements to traffic. Rasmussen (2004) recommended extra care in using these types of concrete mixes considering they have a greater potential for shrinkage, thus random cracking. How the fiber or high early strength concrete actually affects the performance of whitetopping pavements needs to be determined. Supplementary cementitious materials (SCM), such as fly ash and ground-granulated blast furnace slag, have been shown to work with TWT and UTW projects (Rasmussen and Rozycki 2004).

The ACPA WT guideline (2002) gave the following recommendations to insure that the WT layer concrete mix has sufficient durability.

1. In standard areas
    a. Water-cement plus pozzolan ratio < 0.53
    b. Cement + pozzolan content > 520 lb/cu. yd.
2. In areas with frequent freeze-thaw or high use of deicing agent
    a. Water-cement plus pozzolan ratio < 0.49
    b. Cement + pozzolan content > 560 lb/yd3

Table 1: Recommended total air content (ACPA 2002)

Nominal maximum size aggregate		Target percentage air content for exposure		
mm	(inch)	Severe	Moderate	Mild
37.5	1-1/2	5.5	4.5	2.5
25	1	6.0	4.5	3.0
19	3/4	6.0	5.0	3.5
12.5	1/2	7.0	5.5	4.0
9.5	3/8	7.5	6.0	4.5

Table 2: Exposure level (ACPA 2002)

Exposure	Freeze-Thaw	Deicers
Severe	Yes	Yes
Moderate	No long period	No
Mild	No	No

Total air content recommendations are summarized in Table 1. The level of exposure, which is summarized in Table 2, is determined by the amount of freeze-thaw and the presence of deicers.

2.2.3 Slab Thickness Design

For the design of WT pavements, the most commonly used design method is the ACPA guideline (2002). The AASHTO 1993 design method for whitetopping is similar to the ACPA method. The ACPA design method considers truck traffic, flexural strength of concrete, and the support k-value on top of the HMA pavement to select the WT slab thickness. The k-value on top of the HMA pavement is calculated based on the k-value of the subgrade, thickness of the base, and the thickness of HMA pavement (ACPA 2002). This was shown in Figure 2. The thickness of the HMA pavement used to calculate the support k-value on top of asphalt needs to be reduced if milling is planned and needed before the construction of the whitetopping. In the ACPA guideline, the flexural strength is determined from the compressive strength of the concrete material using the Equation (1).

$$f_r = C \times (f'_{cr})^{0.5} \tag{1}$$

where: f_r = flexural strength (modulus of rupture),

C = a constant (0.75 for metric unit and 0.90 for US units),

f'_{cr} = compressive strength.

For primary and interstate highways, the ACPA design guideline recommends a thickness ranging from 8 in. to 12 in. For secondary roads, the ACPA design guide recommends a thickness ranging from 5 in. to 7 in.

However, the condition of the asphalt layer is not taken into account in the ACPA approach. The Colorado DOT uses a mechanistic approach to design WT pavement. Three-dimensional finite element modeling (3-D FEM) was used to develop the design procedure, and then refined using field test results (Tarr et al. 1998; Tarr et al. 2000). Correction factors were used to take partial bonds between PCC and HMA into account, which cannot be realized in FEM analysis. The Colorado DOT design method requires many mechanistic inputs of material properties. The bottom of longitudinal joints are considered the critical location for cracking. A minimum whitetopping thickness of 5 in. is recommended.

For UTW pavements, the ACPA mechanistic design method is often used and was the basis of the Colorado design method of WT pavement. The ACPA design method for UTW uses corner cracking of PCC overlay and fatigue cracking of the underlying HMA pavement as controlling performance (Rasmussen and Rozycki 2004). Again, a 3-D FEM was the basis for the development of this design method. This was followed by an adjustment to field conditions, especially the consideration of the partial bond between the PCC and the HMA. According to the ACPA, UTW is essentially a maintenance strategy and is not to be designed for a life as long as a WT overlay or a conventional PCC pavement. In the ACPA guideline (2002), recommendations of maximum truck traffic are given for different combinations of UTW thickness, existing HMA thickness, joint spacing, design flexural strength, and sub-grade k-value.

At the transition areas (between UTW pavement and other types of pavement), there is a need for thicker slabs between the UTW applications and the asphalt roadways. This was recommended in the ACPA design guide (2002) and supported by field observations (Wu et al. 2007).

2.2.4 Joints Design

The performance of whitetopping pavement is sensitive to the slab size, which is relatively thin. When compared to conventional concrete pavement, whitetopping pavements generally have shorter joint spacing, especially UTW pavement. The purpose of this is to "have the cracks formed only on the joints" (Lin and Wang 2005). Otherwise, longitudinal cracks could occur in the middle of the slab, due to excessive tensile stress (Eacker 2004). The general rule for UTW and WT slab size is to select a joint spacing that is 12 to 18 times the slab thickness (Rasmussen and Rozycki 2004). The ACPA design guide (2002) provides recommendations for bar size, maximum spacing (distance to free edge or to nearest untied joint), and minimum bar length.

Designs using short joint spacing can significantly reduce tensile stresses due to load at the bottom of the slab. However, a smaller slab size will not always provide the best performance. A study of 3-in. thick whitetopping pavement at MnROAD indicated that 6 ft. (transverse) by 5 ft. (longitudinal) slabs performed better than 4 ft. by 4 ft. slabs (Burnham 2005). The longitudinal joints should be designed away from the wheelpath as the corners of the slabs are more prone to cracking. Dowel bars and tie bars are often not used for whitetopping pavement, especially for UTW which does not have enough thickness for dowel bars. Dowel bars and tie bars could become cost-prohibitive if the slab size is small. As the slab thickness increases, the joint spacing also increases . When this happens, dowel bars can and need to be used in whitetopping pavements

2.3 WHITETOPPING CONSTRUCTION PRACTICES

Construction of ultra-thin whitetopping consists of three fundamental steps (ACPA 2002; Lin and Wang 2005):

- Prepare the existing HMA pavement surface by milling and cleaning or by blasting with water or an abrasive material. This step removes rutting, restores the surface profile, and provides a roughened surface to enhance the bonding between the new PCC and the existing HMA pavement (ACPA 1999). This activity should be done 24 to 48 hours before concrete placement (Cole 1997).
- Place, finish, and cure the concrete overlay by using conventional techniques.
- Cut saw joints early at the prescribed spacing.
- Control the curing of concrete mix in the field.

Milling existing HMA pavement is the most common pre-overlay treatment before whitetopping overlay application. Milling helps create a good PCC-HMA bond, eliminates rutting and other irregularities, and provides uniform surface preparation. Milling is especially useful for whitetopping projects in which controlling the grade is important to match curb and gutter or to maintain structure clearance.

To create a good PCC-HMA bond, sufficiently cleaning the milled surface is very important. When the PCC overlay and asphalt layer are fully bonded, the pavement behaves as a composite pavement, reducing the tensile stress/strain at the bottom of the PCC overlay. This is supported by 3D-FEM studies (Nishizawa et al. 2003 and Kumara et al. 2003) and by field observations (Vandenbossche 2003; Lin and Wang 2005). The lack of a good bond has been reported to be responsible for premature failure of whitetopping pavement (McMullen et al. 1998; Rasmussen et al. 2002). In reality, the field instrumentation has demonstrated that in most cases, the PCC overlay and HMA are partially bonded (Tarr et al. 1998). It is also reported that a milled HMA surface has better bonding than an unmilled HMA surface and reduces the tensile strain at the bottom of PCC overlay by an average of 25 percent compared to PCC overlay on unmilled asphalt surface (Tarr et al. 2000). This finding supported Rasmussen's (2002) hypotheses that the presence of voids in the underlying asphalt pavement is one of the major causes of the different types of failures observed on UTW overlay surfaces during the ALF UTW study. The exact reason for this behavior is not clear and requires further investigation.

Iowa #406 tests on whitetopping pavement cores have been widely used to determine the shear strength of the bond (Iowa DOT 2000; Qi et al. 2004). A shear strength of 200 psi is reported to be sufficient to withstand the shearing force caused by vehicles (Tawfiq 2001). It is noted that in the Iowa shear test, no axial load is applied to the specimen to simulate the field conditions.

Other than milling, leveling course or direct placement are alternate methods prior to PCC overlay. Rasmussen (2004) reported that the new HMA material in the leveling course can further compact and shift under whitetopping surface deflections, which can result in premature cracking in the PCC overlay. When a whitetopping overlay is placed in hot weather, water fogging or whitewashing (lime slurry or curing compound) could be used to lower the temperature of the asphalt layer to prevent possible cracking in the PCC overlay. However, excessive water fogging or whitewashing could be detrimental to the bonding of PCC and HMA (Rasmussen et al. 2004).

The ACPA whitetopping guideline (2002) and the National Cooperative Highway Research Program (NCHRP) bulletin on whitetopping and ultra-thin whitetopping (Rasmussen and Rozycki 2004) summarized recommendations for the construction of whitetopping pavement. Curing compound should be applied at twice the normal rate (Mack et al. 1998; ACPA 1999 as quoted by Lin and Wang 2005). Joint sawing should be accomplished by lightweight saws as early as possible to control cracking (ACPA 2002).

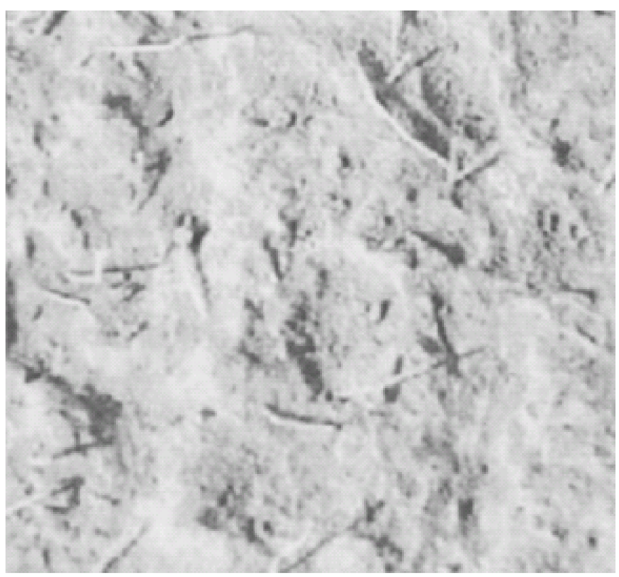

Figure 3: Separation of fiber on pavement surface (Lin and Wang 2005)

It is important to mention the weather conditions during the curing of concrete material. Lin (2005) reported that an air temperature higher than 90oF can result in the separation of fibers on the surface of the finished whitetopping, as shown in Figure 3. It is not known how this behavior influences the performance of whitetopping pavement.

2.4 WHITETOPPING DISTRESSES

The literature indicates that the primary types of distresses observed in whitetopping pavements are:

- Corner cracking
- Mid-slab cracking
- Joint faulting
- Joint spalling

2.4.1 Corner Cracking

In the literature review (Cole 1997; Rasmussen et al. 2002; Vandenbossche 2003; and Wu et al. 2007), corner cracking is reported to be the most commonly observed structural distress. Figure 4, a picture taken from the FHWA ALF UTW study (Rasmussen et al. 2002), is an example of the distress. It occurred when the concrete material fatigue limit, which is a function of the stress-to-strength ratio and the number of load applications, is exceeded. This distress is obviously influenced by the strength of the concrete material, which is influenced by the condition of the underlying HMA layer. One of the influencing conditions is the amount of rutting in the support layer. Rasmussen (2002, 2004) hypothesized that the rutting in the underlying layer created a void, which increased the stress levels in the UTW layer, as illustrated in Figure 5.

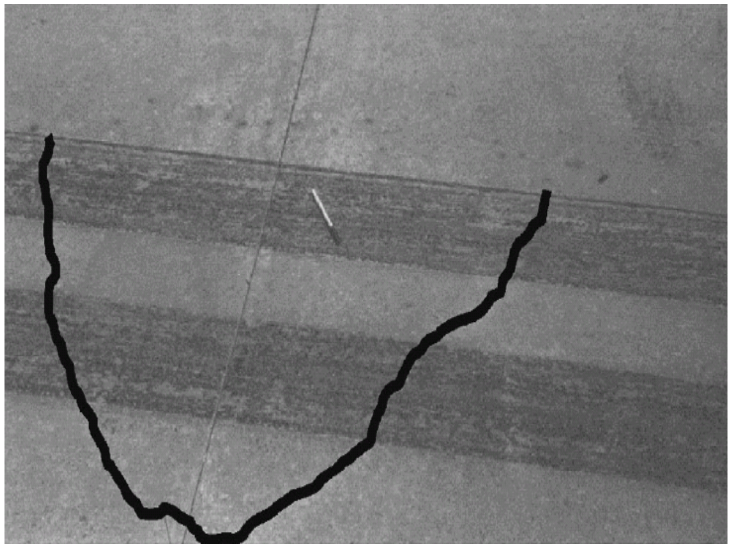

Figure 4: Corner Cracking (Rasmussen et al. 2002)

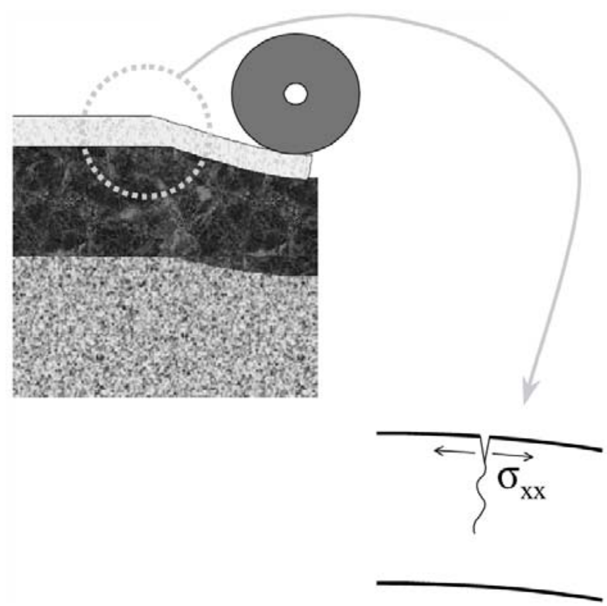

Figure 5: Corner cracking mechanism (Rasmussen et al. 2002)

Cole (1997) reported that corner cracking is common on UTW pavements especially at the transition between whitetopping pavement and conventional asphalt pavement. He hypothesized that this damage could be attributed to:

- Impact loading from vehicles moving across the junction of the asphalt roadway and concrete overlay, particularly when the junction is not smooth,
- vehicle loads rolling across the concrete overlay's free edge,
- de-bonding of the concrete overlay at the free edge,
- a combination of these factors.

Lin and Wang's (2005) study on the Florida DOT experimental UTW pavement also hypothesized on the possible loss of the interface bond between the UTW pavement layer and the underlying AC layer due to crack growth within the interface layer. An important note in this Florida DOT study is the significant amount of truck traffic. As mentioned earlier (ACPA 2002), UTW is not typically designed for this type of traffic condition. Lin and Wang (2005) also hypothesized that the possible lack of quality control in the milling operation could be a possible cause in the less-than-desirable bond between the UTW and the underlying AC layers. This further emphasizes the need for good underlying HMA pavement as a support layer for the whitetopping pavement.

2.4.2 Mid-Slab Cracking

Figure 6 is an example of mid-slab cracking. Like corner cracking, mid-slab cracking occurs when the concrete loading exceeds the fatigue limit. Figure 7 illustrates the mid-slab cracking mechanisms. Rasmussen (2002) suggested two possible hypotheses depending on where the crack initiates.

Figure 6: Mid-Slab Cracking (Rasmussen et al. 2002)

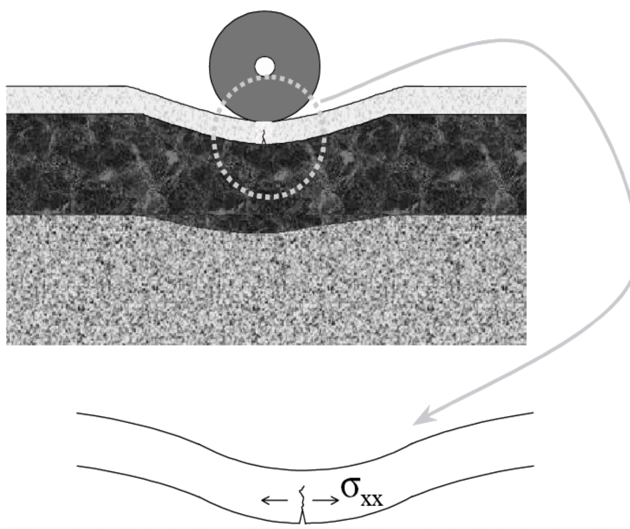

Figure 7: Mid-slab cracking mechanisms (Rasmussen et al. 2002)

- Mid-slab cracking initiates at the bottom of the slab

“Wheel load passes directly over the mid-slab, the stresses are highest directly beneath the load at the edge.” The presence of a void due to rutting in the underlying AC layer further increases the amount of stress.

- Mid-slab cracking initiates at the top of the slab

This is possibly induced by the tensile stresses at the top as the wheel load rolls onto the slabs in question. This hypothesis is supported by the strain gauges measurements in the slab as reported that there was a stress reversal in the top of the slab.

2.4.3 Joint Faulting

In the FHWA ALF study, joint faulting was observed along both the longitudinal and the transverse joints. Figure 8 is an example of a joint faulting along the longitudinal direction. Rasmussen (2002) hypothesized that this distress was caused by the "high vertical stresses introduced into the support layers" because of the ALF one-line loading. This mechanism is illustrated in Figure 9 for longitudinal joints and in Figure 10 for transverse joints.

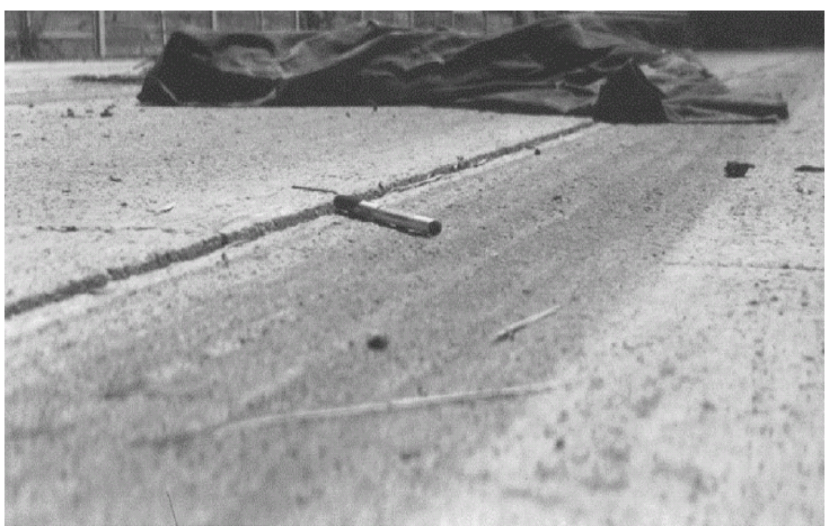

Figure 8: Longitudinal Faulting (Rasmussen et al. 2002)

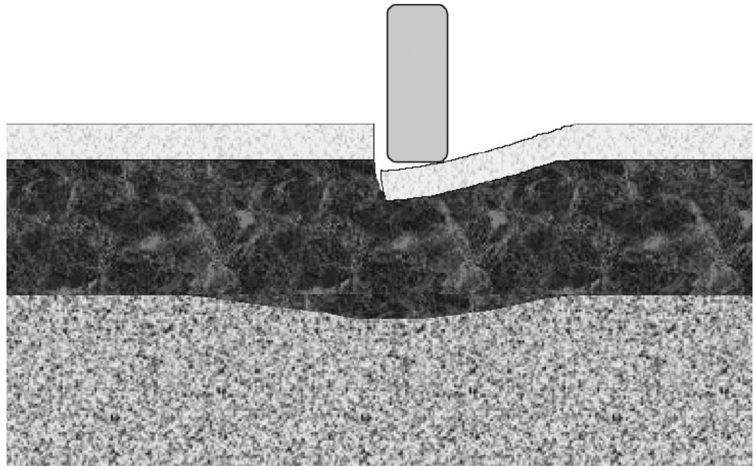

Figure 9: Longitudinal joint faulting mechanisms (Rasmussen et al. 2002)

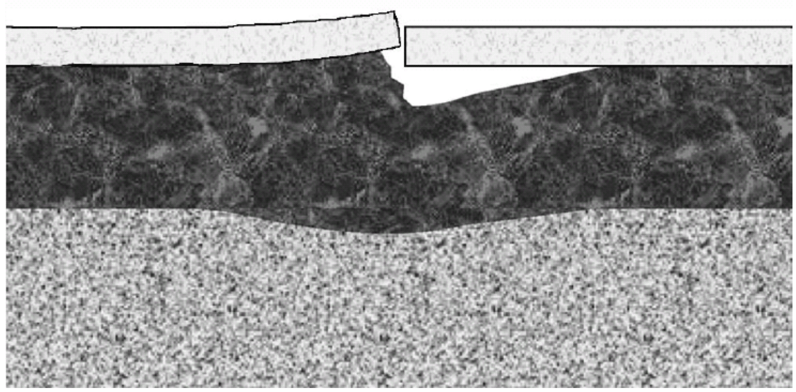

Figure 10: Transverse joint faulting mechanisms (Rasmussen et al. 2002)

2.4.4 Joint Spalling

Figure 11 shows an example of joint spalling. Rasmussen et al. (2002) indicated that there are two common types of joint spalling: delamination spalling and deflection spalling. Delamination spalling is a caused by horizontal micro-cracking introduced during the early-age concrete construction, and traffic loading. Deflection spalling, which is more commonly observed in airport pavements, is caused by a localized crushing of the material at the joints. Because of the typical thin thickness of the UTW layer, deflection spalling is hypothesized to be the cause of the joint spalling in the UTW ALF study. Figure 12 illustrates the joint spalling mechanism.

Figure 11: Spalling (Rasmussen et al. 2002)

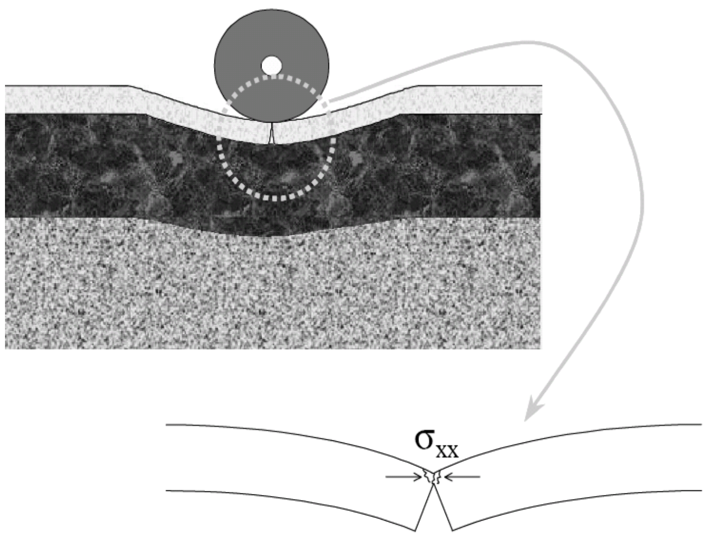

Figure 12: Joint Spalling Mechanisms (Rasmussen et al. 2002)

2.5 WHITETOPPING REPAIR METHODS

Yoon (2001) reported that removal and replacement of individual damaged panels in whitetopping pavement is an effective repair method. Damaged panels are identified and removed with the use of sawcut and jackhammer (Yoon et al. 2001). For multiple panel removal, milling may be used to remove the PCC overlay. The exposed underlying HMA pavement area is then cleaned thoroughly by air-blasting. This is followed by placing new concrete on the exposed area and then finished, textured, and sawed to match existing joints. Replaced panels were reported to perform well under FHWA ALF loading thus extending the service life of the overall whitetopping pavement. This can be considered another advantage of the use of whitetopping over conventional HMA overlay as this repair method can target specific slabs and reduce pavement maintenance cost. In a HMA overlay, whole pavement sections need to be resurfaced.

CHAPTER 3: FIELD EVALUATION OF WHITETOPPING PAVEMENTS

3.1 INTRODUCTION

The first step of this study was to develop a database of whitetopping projects in Wisconsin. Information was collected from the Wisconsin Concrete Pavement Association, the WisDOT, local governments, designers, and contractors. This information includes the as-built plan, special provisions, cost and design information, along with the first-hand information gathered by visiting the projects. There were a total of 18 projects that could be defined as whitetopping. These 18 projects were built from 1995 to 2007. Sixteen projects were still in-service and two of them out-of-service as of 2009. The slab thickness ranges from 4 in. to 9 in. and the joint spacing from 4 ft. by 4 ft. to 15 ft. by 15 ft.

The acronyms, IH, STH, USH, and CTH were used to as abbreviate the terms Interstate Highway, State Highway, U.S. Highway, and County Highway, respectively. Full road names were used for other local projects.

Table 3 lists the whitetopping projects and the information collected about them. Figure 13 shows the locations of the 18 whitetopping projects in Wisconsin. It is noted that most of projects were surveyed in the summer of 2008. A couple of projects were surveyed or re-visited in the summer of 2009. To evaluate the performance of these whitetopping pavements, distress surveys were conducted on 15 in-service pavements. Falling weight deflectometer (FWD) tests and coring were undertaken on selected projects to cover a range of performance.

Table 3: Catalogue of the WT and UTW projects in Wisconsin

No	Road or Project Name	Year	Type	County	In Service	Limits: Start	Limits: End	Surface Milling (inches)	After Whitetopping: PCC (inches)	After Whitetopping: HMA (inches)	After Whitetopping: Sub Base 1 (inches)	After Whitetopping: Sub Base 2 (inches)	Fiber (lbs/C.Y)
1	CTH A	2007		Dodge	Y	STH33	Hemlock Rd	2	7.5	2.5	4.5"pulverized HMA	14"CABC	
2	Duplainville Road	1999	Local Road	Waukesha	Y	CTH F	RR Crossing	1	7	6	7.6" PCC	6" CABC	3
3	Fond Du Lac Ave	2001	Local Road	Milwaukee	Y	Capitol Ave	52nd Street		4	1.5			3
4	Galena St	1995	Local Road	Milwaukee	Y	North 15th St	North 17th St		4	3	10" Gravel		3
5	Howard Ave	1999	Local Road	Milwaukee	Y				4				3
6	IH 94/STH 50 Ramp	1998	IH Off-ramp	Kenosha	Y	SB off ramp		4	4	4			3
7	Janesville and Rockwell Ave	1997	Intersection	Jefferson	N			4	4	2	6" CABC	9" Unkown	3
8	Lawndale Ave	1998	Local Road	Washington	Y			0	4	3.5	9" CABC	N/A	3
9	North 39th Avenue	1999	Local Road	Kenosha	Y				4	3.5			3
10	State Street	2000		Milwaukee	Y	Driveway to Central Ready Mix company			7				3
11	STH 50	2001	Highway	Kenosha	Y	Just west of IH94 and STH 50 intersection							
12	STH 54	2001	Highway	Portage	Y	Plover, WI	IH 39	0.5	7	6.5	7" HMA	17" CABC	
13	STH 82	2001	Highway	Adams	Y	STH 13	Adams, WI	0.5	5	1.5	HMA	CABC	3
14	STH 97	1999	Highway	Taylor	Y	Taylor CO Line	STH 64	0	4	3	10" CABC		1.5
15	STH33 and CTH "A"	2001	Intersection	Dodge	Y	Intersection		4	4				Y
16	STH33 and STH67	2001	Intersection	Dodge	N	Intersection			4				
17	USH 2/USH 53	2001		Douglas	Y	CH B	USH 2	<0.5	9	9	7"CABC		
18	Washington St. and 22nd St.	2001	Intersection	Kenosha	Y	Intersection		4	4				3

Table 3: Catalogue of the WT and UTW projects in Wisconsin (continued)

No	Road or Project Name	Design Traffic (ESAL)	Design Period (year)	Cummula-tive Traffic to Date (ESAL)	Project Length (feet)	Project Width (feet)	Slab size Length (feet)	Slab size Width (feet)	Slab size Thickness (inch)	FWD Test	Core thickness PCC (in.)	Core thickness HMA (in.)	Iowa shear test (psi)	Field Distress Survey
1	CTH A				22,420	24	15	15	7.5	Y	7.81	1.75	154.07	Y
2	Duplainville Road	4,494,979	20	2,247,490		22	5.5	11	7	Y				Y
3	Fond Du Lac Ave				375	60	4	4	4	Y	3.7	1.5		Y
4	Galena St	132,483	20	92,738	750	24	6.5	6	4					Y
5	Howard Ave	k					6	6	4					N
6	IH 94/STH 50 Ramp	1,230,361	10	676,528	200	36	4	4	4					Y
7	Janesville and Rockwell Ave	2,029,764	10	1,623,811			5.5	6	4					N
8	Lawndale Ave				750	32	4.75	6	4	Y	3.95	3.25	266.05	Y
9	North 39th Avenue	1,554,900	20	777,450	263	48	6	6	4		4.2	3.5	177.29	Y
10	State Street						5.5	6	6 --8					Y
11	STH 50						5	5						Y
12	STH 54	4,971,300	10	3,977,040	9,874	24	12	15	7					Y
13	STH 82	3,248,500	20	1,299,400	64,944	30	5	5	5	Y	6.13	1.5	124.59	Y
14	STH 97	819,717	20	409,900	7,920	22	5.5	6	4					Y
15	STH33 and CTH "A"				250	24	4	4	4					Y
16	STH33 and STH67				250	24	4	4	4					N
17	USH 2/USH 53	4,781,500	20	1,912,600	34,727	48	15	15	9					Y
18	Washington St. and 22nd St.				244	48	4	4	4					Y

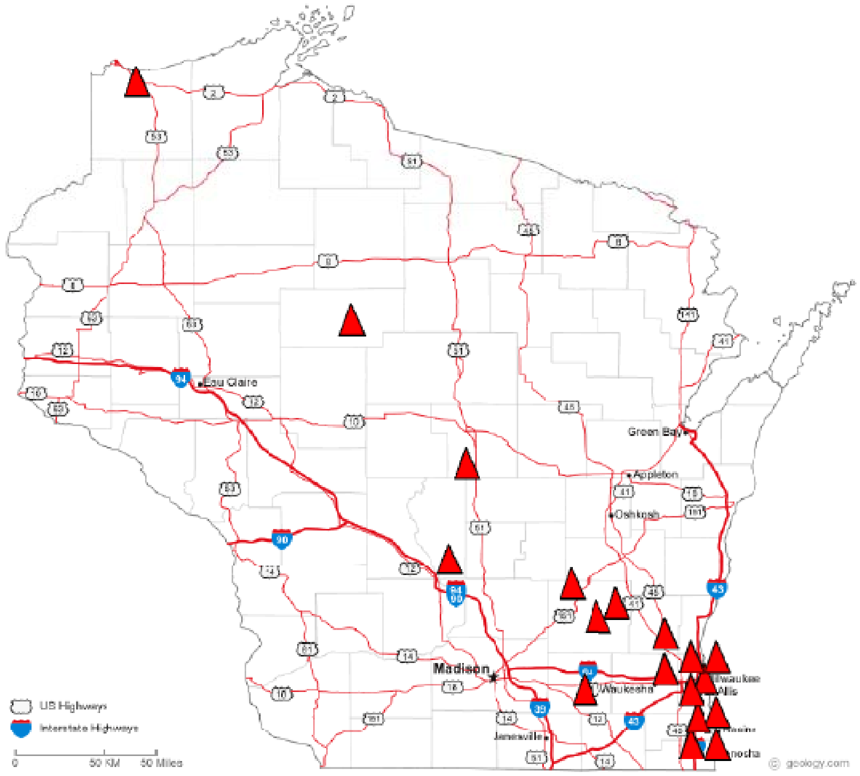

Figure 13: Locations of Whitetopping Projects in Wisconsin

3.2 FALLING WEIGHT DEFLECTOMETER TESTING

In order to get the in-situ properties of the whitetopping pavement, falling weight deflectometer (FWD) tests were conducted on five in-service projects. They were Fond Du Lac Avenue, Lawndale Avenue, Duplainvile Road, STH 82 and CTH “A”. FWD tests were performed from June 16 to 23, 2008, using three target load levels of 6,000, 9,000, and 12,000 lbs and three drops for each load level. The loading plate was placed in the wheel path and 7 sensors were used. The sensor spacing is as shown in Figure 14. FWD test data were used for backcalculating the pavement properties and estimating the fatigue lives of the pavements.

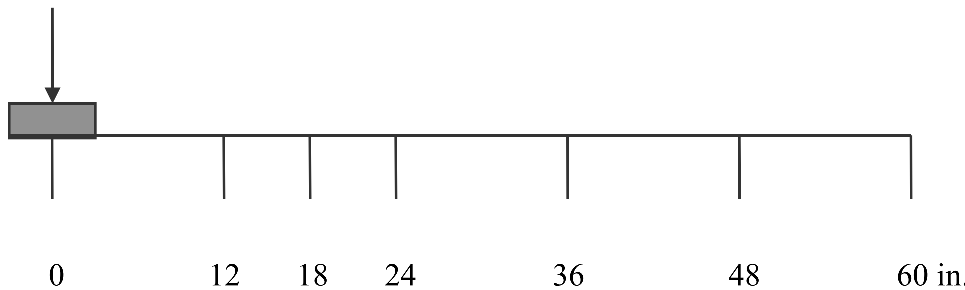

Figure 14: Deflection Sensor Spacing in FWD Test

3.3 PAVEMENT DISTRESS SURVEY

Distress surveys were conducted following two procedures. One procedure followed the guidelines of the WisDOT's "Pavement Surface Distresses Survey Manual" for Pavement Distress Index (PDI) which is a combination of many distresses, as well as individual distress severity and extent (Wisconsin DOT 1993). The other procedure followed the U.S. Army Corps of Engineers' MicroPAVER protocol for Pavement Condition Index (PCI) which is a symbol of the current condition of pavement (Micro PAVER, 2003). The distress surveys were performed to calculate both PDI and PCI.

Among the 18 whitetopping projects, 16 were still in service as of August 2009. Fifteen of them were included in the field distress survey. No survey was conducted on the Howard Avenue whitetopping project, because it is located in the Milwaukee County Water Plant and could not be accessed. In the distress survey for PDI calculation, 1 to 12 survey sections for each project were chosen according WisDOT's "Pavement Surface Distresses Survey Manual" based on the length of the projects. For some of the shorter ones, the whole project was surveyed. There were a total of 48 sections surveyed for the 15 projects. In the distress survey for PCI calculation, 3 to 18 sample units for each project were chosen randomly based on ASTM D6433-07. For some of the shorter ones, the whole project was surveyed but separated evenly into several sample units. There were a total of 129 sample units surveyed for the 15 projects. Most of the distress surveys were finished in May and June, 2008. Additional surveys were conducted in July 2009.

3.4 CORES FOR BOND STRENGTH TESTING

Based on the literature review, bond strength is essential to form a composite structure in WT and UTW pavement. Iowa shear strength tests (Iowa DOT 2000) were conducted to determine the bond strength between concrete slabs and existing HMA. The test's apparatus consists of a loading jig to accommodate a 4–in. nominal diameter. The jig is designed to provide a direct shearing force at the bonded interface. The specimen is placed in the testing jig in such a manner that the bonded interface is placed in the space between the main halves of the jig. A uniform tensile load is applied at the rate of 400 to 500 psi per minute, until the specimen fails. The shear bond strength of the specimen is calculated by dividing the maximum load carried by the specimen during the test by the cross-sectional area of the sample. A 4-in. diameter core barrel was used in the field. Coring was conducted on 5 pavements. The shear strength tests were conducted on the cores from 4 of them following the test protocol (Iowa 406-C). These 4 projects were Lawndale Avenue (Washington County), STH 82 (Adams County), North 39 Avenue (Kenosha County) and CTH "A" (Dodge County). Figure 15 shows the cores and the test equipment. There were no cores tested for Fond Du Lac Ave., because the PCC and HMA were separated during coring.

It should be noted that even for the 4 projects having unseparated cores, the concrete and asphalt were separated in most of the cores during coring and the shear strength could not be determined for the separated specimens.

Figure 15: Cores (left) and Equipment (right) Used in Bond Strength Test

CHAPTER 4: DEVELOPMENT OF A FWD BACKCALCULATION METHOD FOR UTW

4.1 TRADITIONAL FWD BACKCALCULATION METHODS

Backcalculation of the layer properties of concrete pavement is always a challenge. This is especially true for WT or UTW pavement with relatively thin slab thickness and short joint spacing (Cable et al, 2001). In this study, backcalculation programs, “Evercalc” (WS DOT 2005) and “Modcomp 6” (Irwin 2003), and equations based on “AREA” theory (Hall et al, 1991) were studied.

1) “Evercalc”

“Evercalc 5.0” is one of the three parts of the “Everseries” program which was developed by the Washington State Department of Transportation. It is based on the linear elastic model and is a useful FWD test backcalculation method for asphaltic pavement. However, when used in this study, it gave unreasonable HMA layer moduli and was not studied further.

2) “Modcomp 6”

“Modcomp 6” is a program developed by Cornell University. This program needs to establish a model of the layered pavement structure and match the calculated deflections with the measured surface deflections by iterative method that progressively adjusts the moduli of each layer (Irwin 2003). It can account for the linearity or nonlinearity of material properties and was recommended by the Federal Highway Administration for the long term pavement performance (LTPP) data analysis.

3) Equation based on "AREA" theory

Equations (2) to (4) (Hall et al, 1991) are used as a closed-form FWD backcalculation method for PCC pavement based on the "AREA" theory. It uses the deflection at 0, 12, 24, and 36 in. from loading center to obtain the "AREA" and uses the "AREA" to calculate the elastic solid radius of relative stiffness "l_e" which is related to the PCC modulus.

$$AREA = 6 * (1 + 2\left(\frac{d12}{d0}\right) + 2\left(\frac{d24}{d0}\right) + \left(\frac{d36}{d0}\right) \qquad (2)$$

$$l_e = \left[\frac{ln\left(\frac{36 - AREA}{4521.676}\right)}{-3.645}\right]^{\frac{1}{0.187}} \qquad (3)$$

$$l_e = \left[\frac{E_{PCC} * D_{PCC}^3(1 - v_s^2)}{6(1 - v_{PCC}^2)E_S}\right]^{1/3} \qquad (4)$$

where: d0, d12, d24, d36 = surface deflection at 0, 12, 24, 36 in. from loading center,

l_e = elastic solid radius of relative stiffness in in.,

E_{PCC} = PCC elastic modulus in psi,

D_{PCC} = PCC thickness in in.,

v_s = subgrade Poisson's ratio,

v_{PCC} = PCC Poisson's ratio,

E_s = subgrade elastic modulus in psi,

4.2 LIMITATION OF TRADITIONAL FWD BACKCALCULATION METHODS FOR UTW

Dowels or other load transfer devices are seldom used in UTW overlay due to the small slab thickness. Therefore the load transfer efficiency (LTE) was mainly affected by aggregate interlock between slabs and sub-structure support. In case of strong support and good PCC/HMA interface bond, the LTE could be as high as 80 to 90 percent (Roesler et al, 2008; Tia 2002). However, if the underlying support is poor and debonding occurred at PCC/HMA interface, the LTE could reduce significantly (Roesler et al, 2008). It means that the sub-structure support has more effect on LTE and the shape of deflection basin in UTW than that in conventional PCC pavement. The major differences between UTW and conventional PCC pavements are the slab thickness, the joint spacing, and the stiffness of the substructure. The slab thickness used in UTW is less or equal to 4 in. while in conventional PCC pavement it is normally more than 7 in., typically 12 in. The joint spacing used in UTW is generally less than 6 ft. by 6 ft. compared with about 12 ft. by 12 ft. in conventional PCC pavement. Because the UTW overlay is constructed on top of old HMA pavement, the stiffness of the underlying structure is higher than the base layer under conventional PCC pavement. This is the reason that although the aggregate interlock is reduced due to the thin slab thickness, the LTE is still high.

Therefore, the major difference of UTW from conventional PCC pavement could be characterized by the much lower relative stiffness of PCC slab due to the thinner slab thickness, shorter joint spacing, and stronger sub-structure support. The Modcomp6 is a program based on continuous elastic layer theory. The short joint spacing in UTW may result in discontinuity in the PCC layer which contradicts the continuity assumption, as shown in Figure 16. The equations based on "AREA" theory relate the "l_e" with the "AREA" by regression model from deflection data of conventional PCC pavement which may differ from that of UTW dramatically. For these reasons, concerns are raised when using traditional methods for FWD backcalculation of UTW pavement.

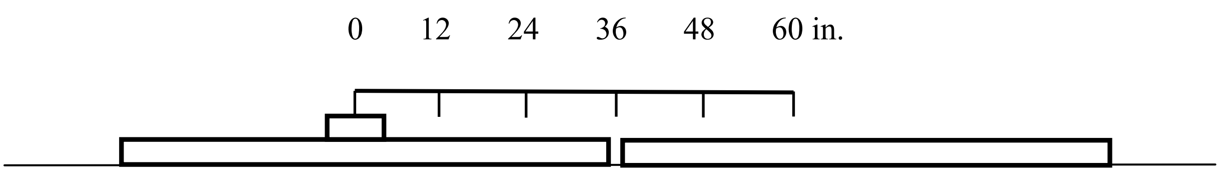

Figure 16: FWD Test Sensor Layout on UTW

4.3 DEVELOPMENT OF CRITICAL DISTANCE METHOD FOR UTW

In order to deal with the problems in FWD test backcalculation for UTW pavement, a new method for UTW pavement backcalculation, called Critical Distance Method, was developed in this study.

4.3.1 Methodology of the Development of Critical Distance Method

4.3.1.1 Approach based on St. Venant's principle

The Critical Distance Method was based on St. Venant's principle that "the difference between the stresses or strains caused by statically equivalent load systems is insignificant at distances greater than the largest dimension of the area over which the loads are acting." Theoretically, a critical distance can be identified beyond which the effect of the presence of the slab could be negligible. The deflections beyond the critical distance would be the same as those induced by placing a loading plate directly on the surface of the sub-structure without a UTW overlay. Figure 17 shows the application of St.

Venant's principle to UTW pavement. The deflection basin denoted by solid line was obtained by loading on slab surface, by dashed line was obtained by loading on surface of sub-structure. Beyond Critical Distance, these two deflection basin inclined to merge together. Therefore, the problem of UTW pavement with thin slabs becomes that of an equivalent asphalt pavement for the backcalculation of the modulus of the substructure. The properties of the substructure could be obtained, based on the deflections beyond the critical distance. Once the properties underneath the concrete slab are obtained, the modulus of PCC can be found by matching the deflection within the critical distance, based on iterations.

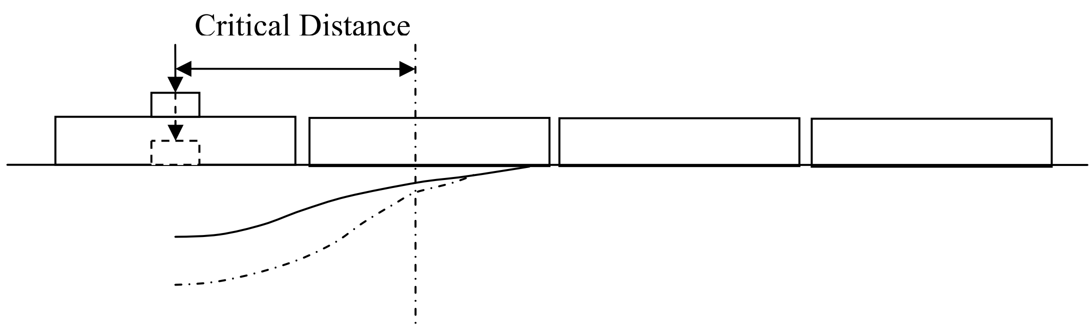

Figure 17: St. Venant's Principle Used in UTW Pavement

The key to this Critical Distance Method is to identify a consistent critical distance for backcalculation. This was accomplished using the numerous combinations of pavement simulations. Pavement structures with given material properties and layer thicknesses were modeled for both the UTW and equivalent substructure. The differences in deflections at various distances from the loading plate were used to identify the critical distance beyond which the differences in deflections between UTW and equivalent substructure are negligible. A 10% tolerance level was used in this study. The deflections were calculated using the "KENSLAB" program (Huang 2004), which is based on finite-element method. It was assumed that the UTW overlay was built on the old HMA pavement. An equivalent homogeneous semi-infinite substructure was assumed for the asphalt layer and underlying layers. The reason for this is that for many UTW pavements, the asphalt layer is relatively thin. Backcalculation of an asphalt pavement with a thin asphalt layer is a challenge itself. Another consideration is that for many existing UTW projects, the information about pavement underneath the concrete slab is often missing, unless coring and boring are conducted. For a semi-infinite space problem, a closed-form solution (Ahlvin, R. G. et al, 1962) could be used to backcalculate the composite modulus of the sub-structure. The PCC slab modulus could be calculated using an iteration method to match the deflections within Critical Distance. The backcalculated properties can be compared to the input properties to evaluate the effectiveness of this approach and traditional methods.

4.3.1.2 Outline of Development procedure

The procedure of development of the Critical Distance Method consisted of the following steps:

1) Modeling pavement structures to obtain deflection (both UTW and semi-infinite pavements),
2) Comparison of deflection difference,
3) Identification of critical distances,
4) Backcalculation of modulus of substructure of UTW pavement,
5) Determination of PCC modulus, and

6) Comparing the backcalculated moduli with the input moduli.

The flow chart of the research procedure is shown in Figure 18.

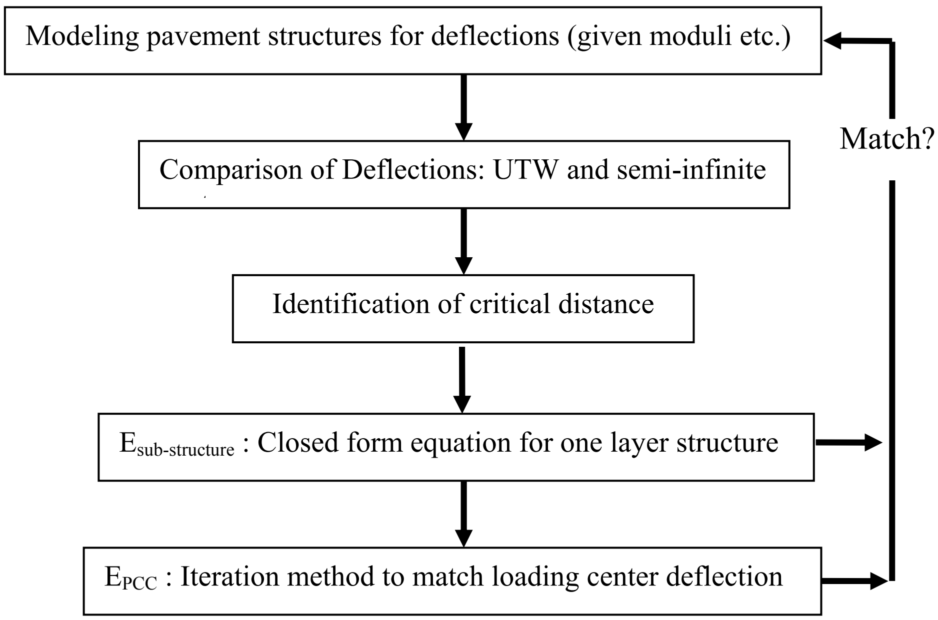

Figure 18: Flow Chart of the Research Procedure

4.3.2 Development of Critical Distance Method

4.3.2.1 Modeling pavement structure to obtain deflection

Pavement structures commonly used in UTW projects were selected in this study, as shown in Table 4. When FWD test is conducted on a UTW pavement, the sensors could be on different slabs for small slab size. Therefore, 4 slabs were simulated in "KENSLAB". For the aggregate interlock joint stiffness, a value of 40,000 psi which is typical for conventional PCC pavement (Khazanovich and Alex Gotlif 2003) was chosen. Considering the possible reduction of aggregate interlock joint stiffness due to the decrease of slab thickness in UTW, and the purpose of covering a large range of joint stiffness, an extreme value of 0 psi was included as well. Primary study showed that Poisson's ratio had small effect on the simulation results. Therefore, a Poisson's ratio of 0.15 for the PCC slab and 0.42 for equivalent sub-structure was assumed. Based on the information in Table 4, there are 108 combinations of pavement structures. All the combinations were simulated using KENSLAB. Some of the deflections are shown in Table 5. All other deflections are provided in Appendix A.

Table 4: Pavement Structure Used in This Study

Joint stiffness of aggregate interlock (psi)	Equivalent moduli of sub-structure (ksi)	Slab thickness (in.)	PCC modulus (ksi)	Joint spacing (slab size) (ft.)
0 40,000	20 50	3 4 5	3000 5000 7000	4 by 4 5 by 5 6 by 6

4.3.2.2 Comparison of deflection difference

Deflections at distances of 0, 12, 24, 36, 48, and 60 in. from the loading center were calculated using the KENSLAB program. In order to be consistent with FWD testing, a 9000 lb load with a circular contact area of 5.91 in. radius was selected.

Deflections on the surface of different virtual UTW pavement structures were calculated based on different composite moduli of sub-structure, PCC moduli, slab thickness, joint stiffness, and joint spacing. Only some modeling results of "KENSLAB" are shown in Tables 5. All the modeling results are provided in Appendix A. Figures 19 through 30 show the simulation results of all the combinations, in terms of deflection differences between UTW and equivalent semi-infinite pavements at different distances from loading center.

Table 5: Deflections and Deflection Difference

(Joint Stiffness=0 psi, Slab thickness=4 in., E-sub=50 ksi)

Joint Stiffness=0 psi, Sub-structure under WT slab with equivalent modulus: E=50ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00612	-61.65	0.00601	-62.34	0.00574	-64.04
12	0.0041	0.00426	3.90	0.00419	2.20	0.00405	-1.22
24	0.002	Joint	Joint	0.00224	12.00	0.00229	14.50
36	0.00132	0.00138	4.55	0.00133	0.76	Joint	Joint
48	0.00098	0.00101	3.06	0.001	2.04	0.00101	3.06
60	0.00079	0.00079	0.00	0.00078	-1.27	0.0008	1.27
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00542	-66.04	0.00533	-66.60	0.00508	-68.17
12	0.0041	0.00404	-1.46	0.00397	-3.17	0.00383	-6.59
24	0.002	Joint	Joint	0.00231	15.50	0.00233	16.50
36	0.00132	0.00139	5.30	0.00135	2.27	Joint	Joint
48	0.00098	0.00103	5.10	0.00102	4.08	0.00102	4.08
60	0.00079	0.0008	1.27	0.0008	1.27	0.00081	2.53
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00499	-68.73	0.0049	-69.30	0.00468	-70.68
12	0.0041	0.00387	-5.61	0.0038	-7.32	0.00366	-10.73
24	0.002	Joint	Joint	0.00234	17.00	0.00234	17.00
36	0.00132	0.00139	5.30	0.00135	2.27	Joint	Joint
48	0.00098	0.00104	6.12	0.00103	5.10	0.00103	5.10
60	0.00079	0.0008	1.27	0.00081	2.53	0.00081	2.53

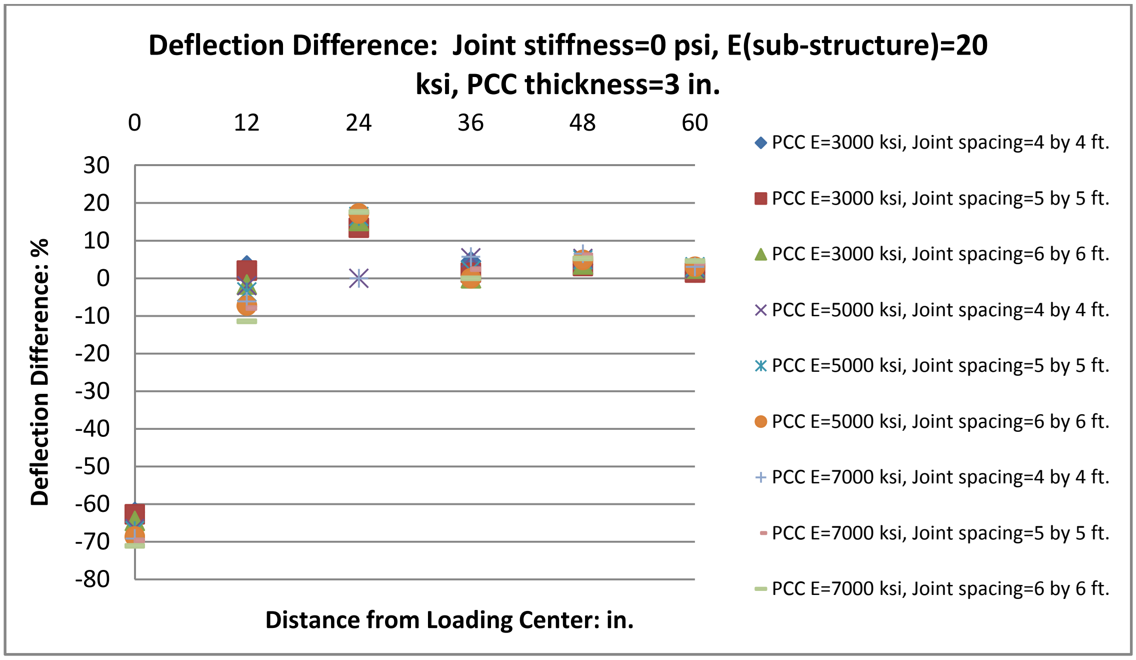

Figure 19: Plots of Deflection Differences for Joint Stiffness=0 psi, Sub-structure Modulus=20 ksi, PCC Thickness=3 in.

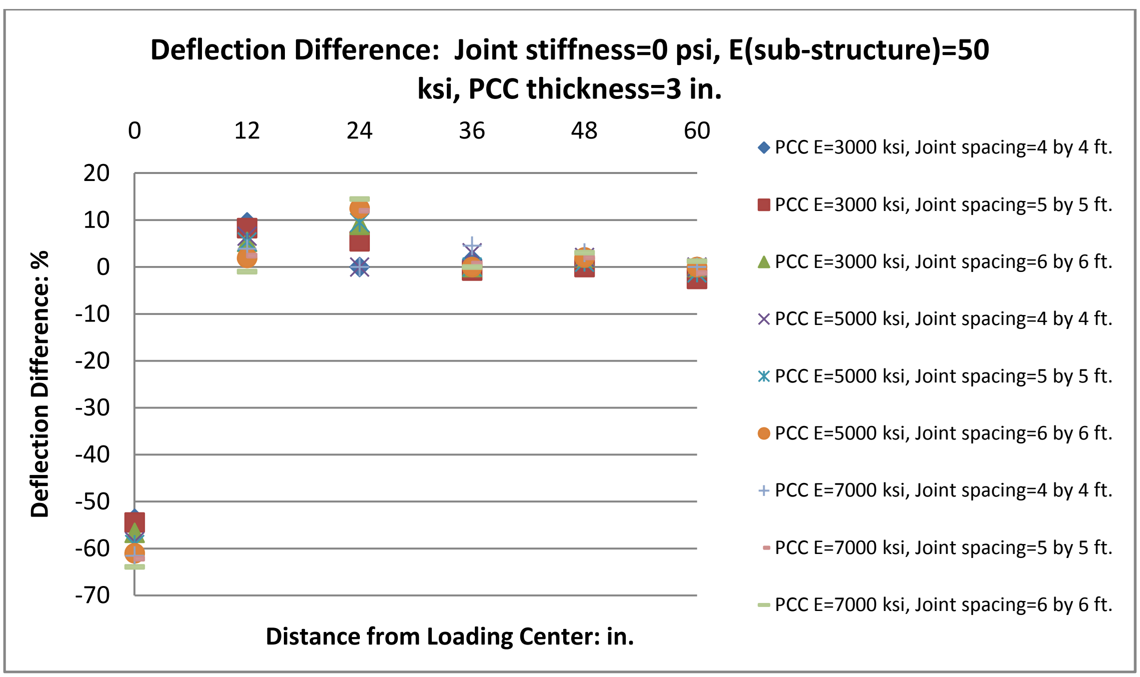

Figure 20: Plots of Deflection Differences for Joint Stiffness=0 psi, Sub-structure Modulus=50 ksi, PCC thickness=3 in.

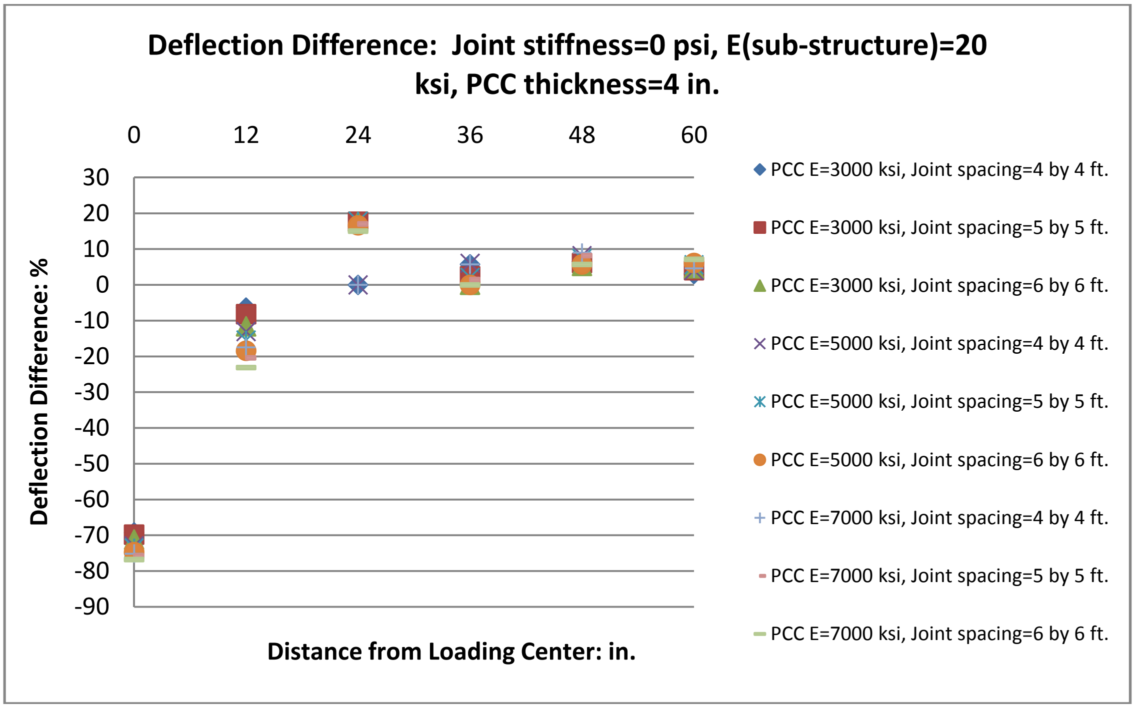

Figure 21: Plots of Deflection Differences for Joint Stiffness=0 psi, Sub-structure Modulus=20 ksi, PCC thickness=4 in.

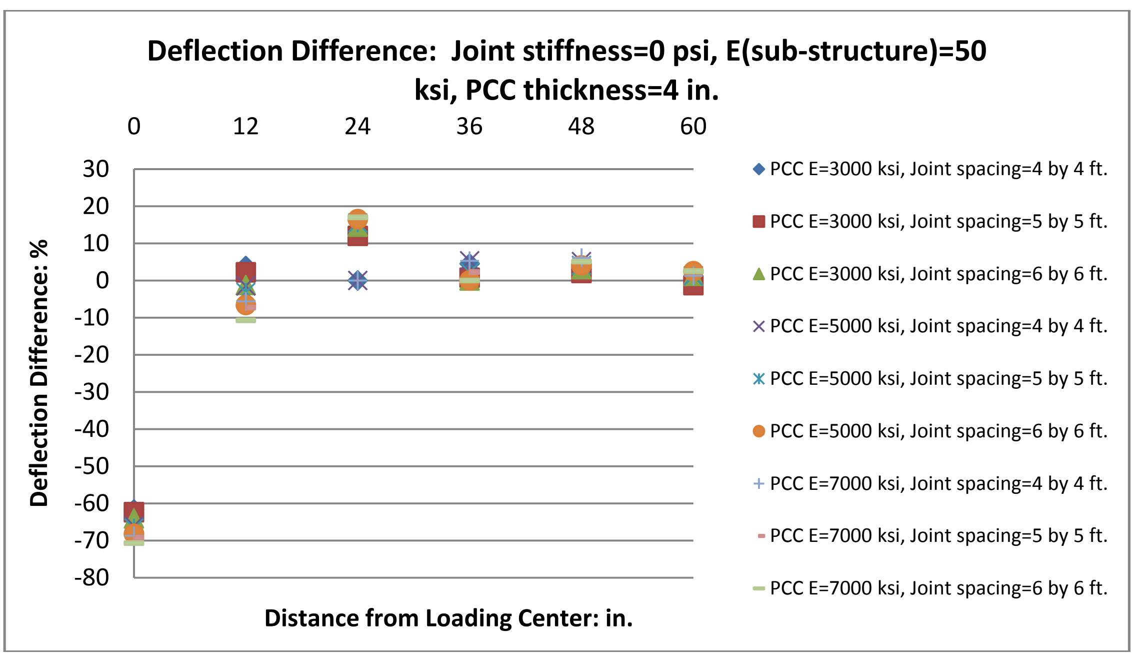

Figure 22: Plots of Deflection Differences for Joint Stiffness=0 psi, Sub-structure Modulus=50 ksi, PCC thickness=4 in.

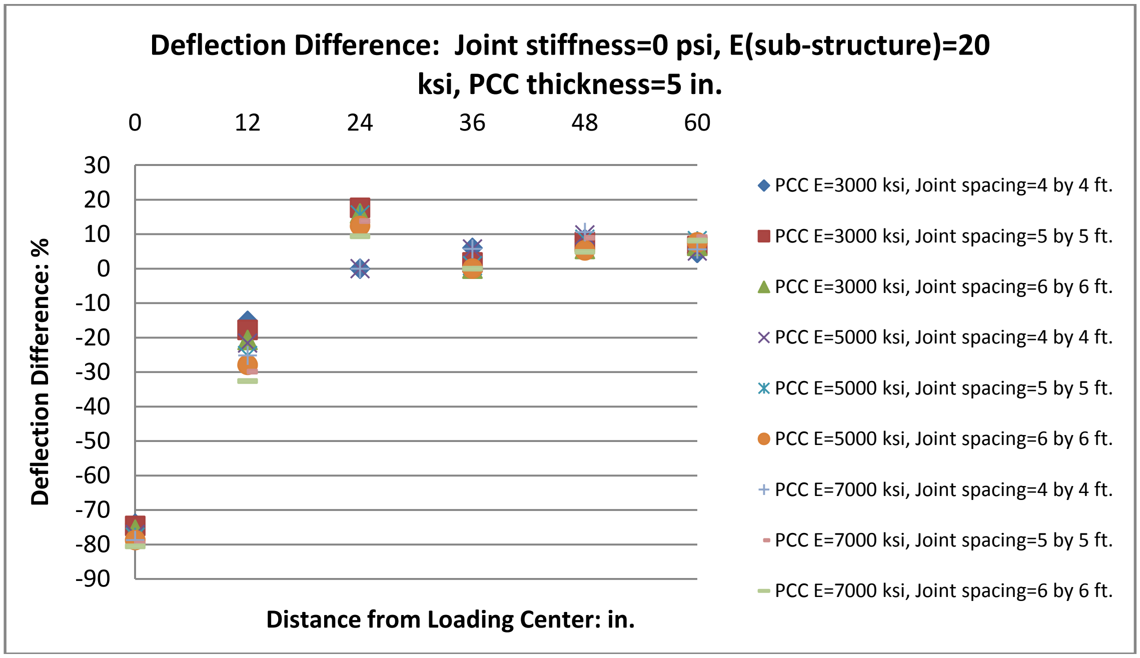

Figure 23: Plots of Deflection Differences for Joint Stiffness=0 psi, Sub-structure Modulus=20 ksi, PCC thickness=5 in.

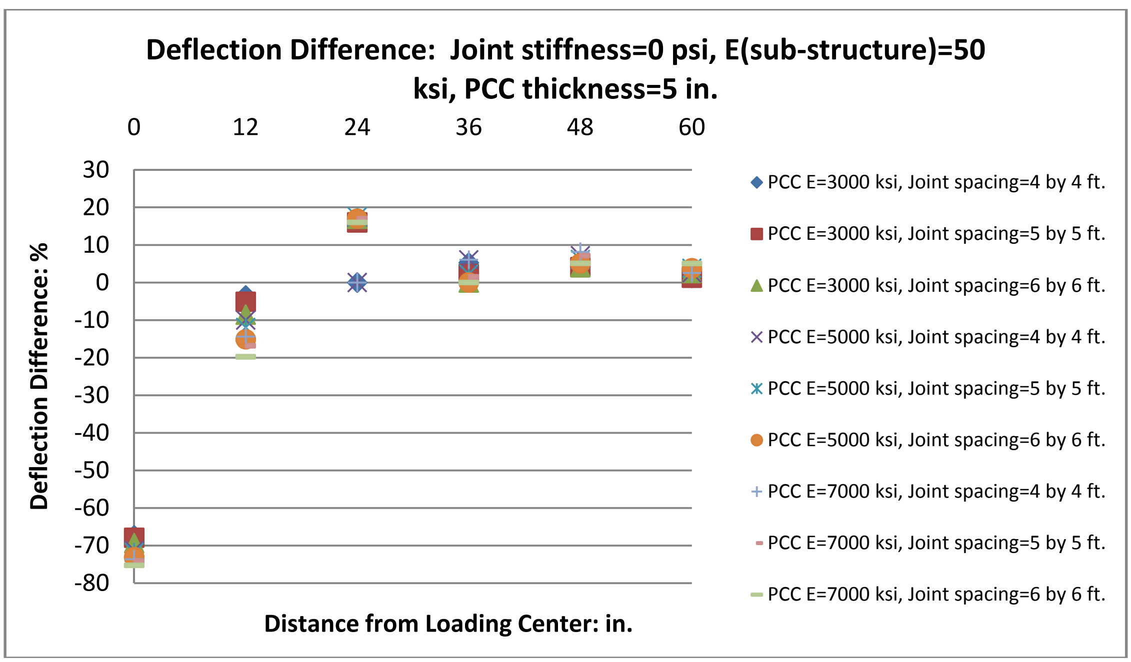

Figure 24: Plots of Deflection Differences for Joint Stiffness=0 psi, Sub-structure Modulus=50 ksi, PCC thickness=5 in.

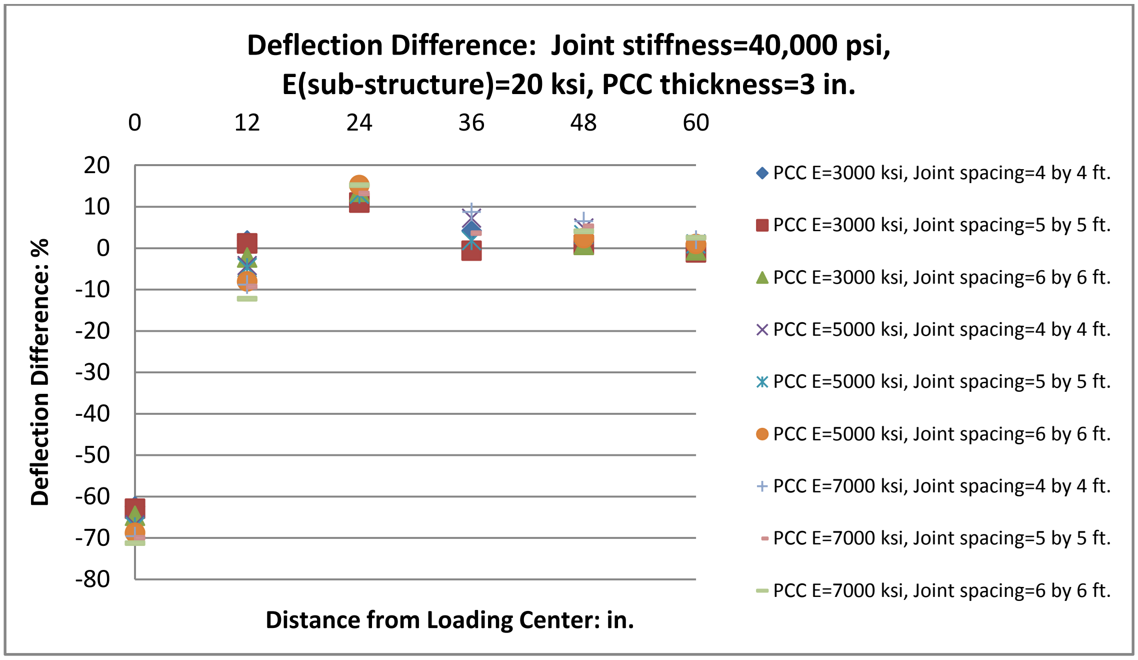

Figure 25: Plots of Deflection Differences for Joint Stiffness=40,000 psi, Sub-structure Modulus=20 ksi, PCC thickness=3 in.

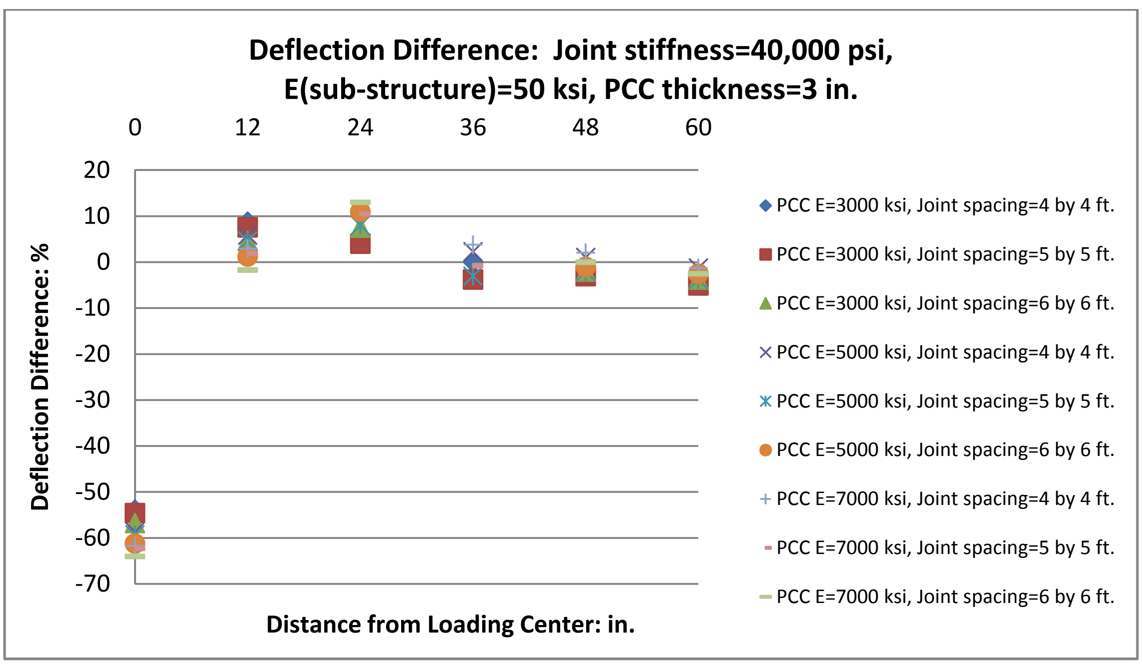

Figure 26: Plots of Deflection Differences for Joint Stiffness=40,000 psi, Sub-structure Modulus=50 ksi, PCC thickness=3 in.

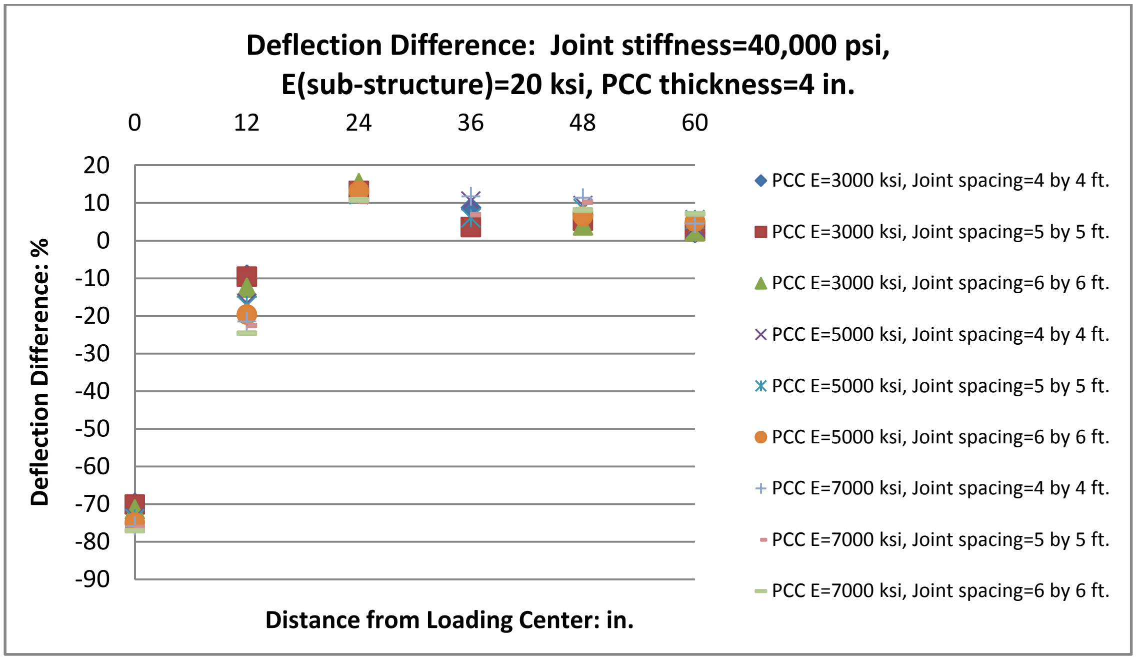

Figure 27: Plots of Deflection Differences for Joint Stiffness=40,000 psi, Sub-structure Modulus=20 ksi, PCC thickness=4 in.

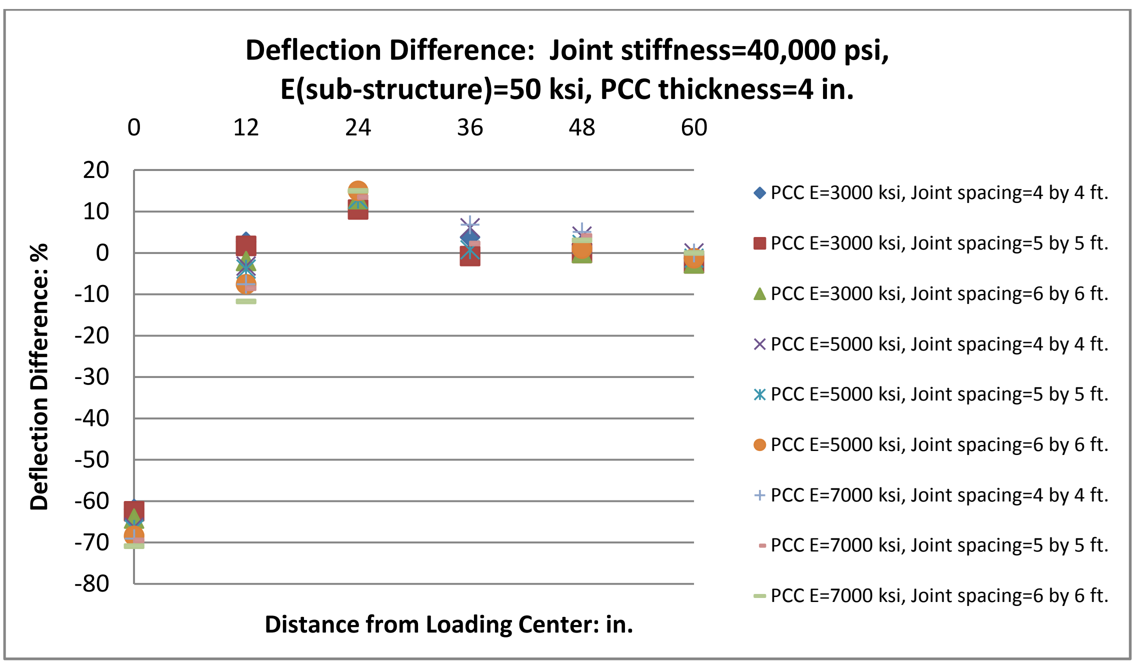

Figure 28: Plots of Deflection Differences for Joint Stiffness=40,000 psi, Sub-structure Modulus=50 ksi, PCC thickness=4 in.

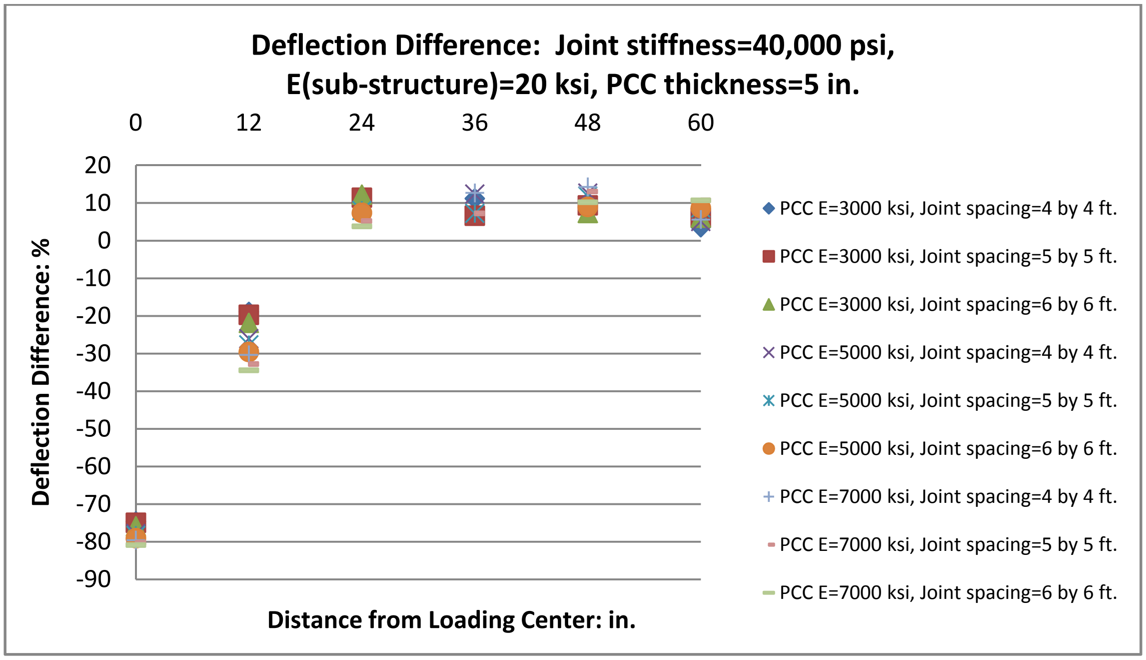

Figure 29: Plots of Deflection Differences for Joint Stiffness=40,000 psi, Sub-structure Modulus=20 ksi, PCC thickness=5 in.

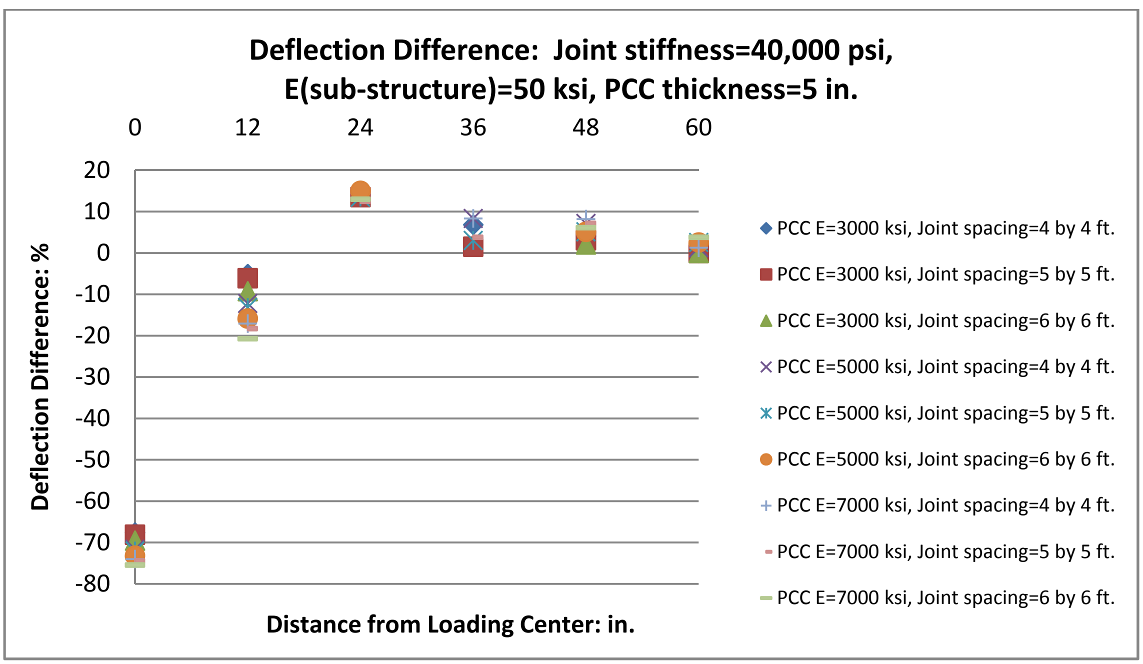

Figure 30: Plots of Deflection Differences for Joint Stiffness=40,000 psi, Sub-structure Modulus=50 ksi, PCC thickness=5 in.

4.3.2.3 Identification of Critical Distance

As seen in Figures 19 through 30, the deflection differences reduced quickly beyond 24 in. from the loading center. Most of the deflection differences were within 10% at 36 in. or farther from the loading center for the pavement structures used in this study. Therefore, 36 in. could be used as Critical Distance. However, Considering 24 in. and 36 in. from loading center are on the joint for 4 ft. by 4 ft. and 6 ft. by 6 ft joint spacing when loading at the slab center, in order to avoid using the deflections at the joint, 48 in. was selected as the Critical Distance in this study. It was also found that with the increase of the underlying support or the decrease of the slab thickness and slab strength, the deflection difference decreased.

4.3.2.4 Backcalculation of the equivalent sub-structure moduli

Once the critical distance is identified, the deflections of UTW pavements at critical distance or farther can be used to backcalculate the modulus of the substructure. Since the substructure is assumed to be a semi-infinite space, Ahlvin and Ulery's (1962) closed-form equation for single-layer elastic analysis, shown in Equation (5), could be used to backcalculate the sub-structure's composite moduli.

$$D_z = \frac{p(1+\mu)a}{E}\left[\frac{z}{a}A + (1-\mu)H\right] \qquad (5)$$

where: D_z = vertical deflection in in.,

p = pressure due to the load, psi,

a = equivalent load radius of the tire footprint in in.,

E = modulus of elasticity in psi, and

A and H = function values, could be found out from tables that depend on z/a and r/a, where:

z = depth of the point in question in in.,

r = radial distance in in. from the centerline of the point load to the point in question.

For each of the deflections at 48 and 60 in. from the loading center, one sub-structure equivalent modulus was backcalculated. Because which one was more accurate was uncertain, the average was used as final result. The backcalculated equivalent sub-structure moduli were then compared to the given moduli of the substructure to determine the accuracy of the backcalculation. The backcalculated moduli and their accuracy are shown in Tables 6 through 11. It can be seen that the Critical Distance Method is effective in determining the modulus of substructure, with errors within 4.43% from the input modulus for slab thickness of 3 in. The results for slab thickness of 4 in. and 5 in. showed that the maximum error increased up to 7.57% and 10.23% respectively, but within 5% in most cases.

4.3.2.5 Backcalculation of the PCC moduli

PCC moduli were backcalculated using iteration method to match the deflection on the surface of UTW pavement at distance of 0, 12, and 24 in. from loading center. However, it was found not practical to match the deflections at these three positions simultaneously. Because the deflection at loading center was maximum and least affected by the joint, it was selected as the single position at which the deflection would be matched. Again, KENSLABS was used to try several PCC moduli until the deflection at the loading center was matched. The results of the backcalculated PCC moduli are shown in Tables 6 through 11.

4.3.2.6 Comparison of Backcalculated moduli with input moduli

The backcalculated PCC moduli and their accuracy are shown in Tables 6 through 11. It can be seen that the error of backcalculated PCC moduli are within 15.33% for UTW with slab thickness of 3 in. The results for slab thickness of 4 in. and 5 in. indicated that the error could increase up to 22.43% and 35.71%, respectively. However, the error is within 20% in most cases which is considered to be acceptable for a concrete pavement backcalculation.

4.3.3 Effect of Aggregate Interlock Joint Stiffness on the Accuracy of Critical Distance Method

1) Analysis of the effect of aggregate interlock joint stiffness on the accuracy of Critical Distance Method

Only aggregate interlock joint stiffness of 0 psi and 40,000 psi were simulated in 4.3.2. The value of joint stiffness between UTW slabs should be within this range. Therefore, the effects of aggregate interlock joint stiffness on the accuracy of Critical Distance Method were analyzed in this section. Table 12 shows the pavement structure and the aggregate interlock joint stiffness used for this purpose.

Table 6: Backcalculation Accuracy of Three Methods (Joint Stiffness=0 psi, slab thickness=3 in.)

Pavement structure	Critical Distance Method (48, 60 in.)				MODCOMP 6				Hall's Equation (based on area theory)			
	Modulus of sub-structure: (ksi)	**Back Calculate Error (%)**	Modulus of PCC: (ksi)	**Back Calculate Error (%)**	Modulus of sub-structure: (ksi)	**Back Calculate Error (%)**	Modulus of PCC: (ksi)	**Back Calculate Error (%)**	Modulus of sub-structure: (ksi)	**Back Calculate Error (%)**	Modulus of PCC: (ksi)	**Back Calculate Error (%)**
Sub-E=20ksi, PCC E=3000ksi, 4 by 4 ft.	19.580	-2.10	3220	7.33	19.5	-2.50	3380	12.67	18.588	-7.06	5556.685	85.22
Sub-E=20ksi, PCC E=3000ksi, 5 by 5 ft.	19.618	-1.91	3190	6.33	19.5	-2.50	3570	19.00	18.582	-7.09	5812.013	93.73
Sub-E=20ksi, PCC E=3000ksi, 6 by 6 ft.	19.482	-2.59	3265	8.83	19.4	-3.00	4290	43.00	18.962	-5.19	6410.719	113.69
Sub-E=20ksi, PCC E=5000ksi, 4 by 4 ft.	19.331	-3.35	5560	11.20	19.5	-2.50	5170	3.40	18.837	-5.81	7619.600	52.39
Sub-E=20ksi, PCC E=5000ksi, 5 by 5 ft.	19.320	-3.40	5550	11.00	19.4	-3.00	5520	10.40	19.002	-4.99	7875.659	57.51
Sub-E=20ksi, PCC E=5000ksi, 6 by 6 ft.	19.320	-3.40	5560	11.20	19.5	-2.50	6410	28.20	19.282	-3.59	8789.814	75.80
Sub-E=20ksi, PCC E=7000ksi, 4 by 4 ft.	19.172	-4.14	7950	13.57	19.5	-2.50	6780	-3.14	18.883	-5.59	9608.448	37.26
Sub-E=20ksi, PCC E=7000ksi, 5 by 5 ft.	19.114	-4.43	7950	13.57	19.4	-3.00	7300	4.29	19.220	-3.90	9788.560	39.84
Sub-E=20ksi, PCC E=7000ksi, 6 by 6 ft.	19.141	-4.29	8000	14.29	19.5	-2.50	8470	21.00	19.401	-2.99	11015.791	57.37
Sub-E=50ksi, PCC E=3000ksi, 4 by 4 ft.	49.777	-0.45	3070	2.33	49.2	-1.60	4000	33.33	44.440	-11.12	8692.643	189.75
Sub-E=50ksi, PCC E=3000ksi, 5 by 5 ft.	50.675	1.35	2850	-5.00	50.5	1.00	3960	32.00	44.336	-11.33	9118.605	203.95
Sub-E=50ksi, PCC E=3000ksi, 6 by 6 ft.	49.526	-0.95	3120	4.00	49.1	-1.80	5120	70.67	45.228	-9.54	10070.119	235.67
Sub-E=50ksi, PCC E=5000ksi, 4 by 4 ft.	49.526	-0.95	5200	4.00	49.1	-1.80	6020	20.40	45.784	-8.43	11122.808	122.46
Sub-E=50ksi, PCC E=5000ksi, 5 by 5 ft.	50.094	0.19	4950	-1.00	49.7	-0.60	6160	23.20	45.657	-8.69	11650.646	133.01
Sub-E=50ksi, PCC E=5000ksi, 6 by 6 ft.	49.526	-0.95	5180	3.60	49.1	-1.80	7520	50.40	46.543	-6.91	12821.242	156.42
Sub-E=50ksi, PCC E=7000ksi, 4 by 4 ft.	49.281	-1.44	7350	5.00	49.1	-1.80	7870	12.43	46.446	-7.11	13336.108	90.52
Sub-E=50ksi, PCC E=7000ksi, 5 by 5 ft.	49.844	-0.31	7080	1.14	49.8	-0.40	8070	15.29	46.493	-7.01	13930.320	99.00
Sub-E=50ksi, PCC E=7000ksi, 6 by 6 ft.	48.972	-2.06	7500	7.14	48.6	-2.80	10100	44.29	47.291	-5.42	15419.712	120.28

Table 7: Backcalculation Accuracy of Three Methods (Joint Stiffness=0 psi, slab thickness=4 in.)

Pavement structure	Critical Distance Method				MODCOMP 6				Hall's Equation (based on area theory)			
	Modulus of sub-structure: (ksi)	**Back Calculate Error (%)**	Modulus of PCC: (ksi)	**Back Calculate Error (%)**	Modulus of sub-structure: (ksi)	**Back Calculate Error (%)**	Modulus of PCC: (ksi)	**Back Calculate Error (%)**	Modulus of sub-structure: (ksi)	**Back Calculate Error (%)**	Modulus of PCC: (ksi)	**Back Calculate Error (%)**
Sub-E=20ksi, PCC E=3000ksi, 4 by 4 ft.	19.172	-4.14	3410	13.67	19.6	-2.00	2920	-2.67	18.891	-5.54	4097.379	36.58
Sub-E=20ksi, PCC E=3000ksi, 5 by 5 ft.	19.114	-4.43	3420	14.00	19.5	-2.50	3150	5.00	19.243	-3.78	4169.361	38.98
Sub-E=20ksi, PCC E=3000ksi, 6 by 6 ft.	19.141	-4.29	3420	14.00	19.5	-2.50	3640	21.33	19.403	-2.99	4692.984	56.43
Sub-E=20ksi, PCC E=5000ksi, 4 by 4 ft.	18.934	-5.33	5880	17.60	19.8	-1.00	4360	-12.80	18.703	-6.49	6044.744	20.89
Sub-E=20ksi, PCC E=5000ksi, 5 by 5 ft.	18.866	-5.67	5880	17.60	19.6	-2.00	4760	-4.80	19.495	-2.52	5947.024	18.94
Sub-E=20ksi, PCC E=5000ksi, 6 by 6 ft.	18.966	-5.17	5820	16.40	19.7	-1.50	5420	8.40	19.392	-3.04	6816.466	36.33
Sub-E=20ksi, PCC E=7000ksi, 4 by 4 ft.	18.817	-5.91	8450	20.71	20	0.00	5590	-20.14	18.369	-8.16	7993.244	14.19
Sub-E=20ksi, PCC E=7000ksi, 5 by 5 ft.	18.660	-6.70	8450	20.71	19.7	-1.50	6270	-10.43	19.571	-2.14	7647.253	9.25
Sub-E=20ksi, PCC E=7000ksi, 6 by 6 ft.	18.877	-5.62	8220	17.43	19.9	-0.50	7010	0.14	19.243	-3.78	8911.359	27.31
Sub-E=50ksi, PCC E=3000ksi, 4 by 4 ft.	49.281	-1.44	3140	4.67	49.3	-1.40	3410	13.67	46.404	-7.19	5686.063	89.54
Sub-E=50ksi, PCC E=3000ksi, 5 by 5 ft.	49.844	-0.31	3050	1.67	49.9	-0.20	3520	17.33	46.573	-6.85	5940.387	98.01
Sub-E=50ksi, PCC E=3000ksi, 6 by 6 ft.	48.972	-2.06	3200	6.67	48.8	-2.40	4360	45.33	47.362	-5.28	6550.835	118.36
Sub-E=50ksi, PCC E=5000ksi, 4 by 4 ft.	48.495	-3.01	5500	10.00	49	-2.00	5250	5.00	47.158	-5.68	7752.476	55.05
Sub-E=50ksi, PCC E=5000ksi, 5 by 5 ft.	48.731	-2.54	5400	8.00	49	-2.00	5540	10.80	47.537	-4.93	8009.643	60.19
Sub-E=50ksi, PCC E=5000ksi, 6 by 6 ft.	48.430	-3.14	5500	10.00	48.7	-2.60	6600	32.00	48.209	-3.58	8942.017	78.84
Sub-E=50ksi, PCC E=7000ksi, 4 by 4 ft.	48.264	-3.47	7800	11.43	49.3	-1.40	6810	-2.71	47.329	-5.34	9732.433	39.03
Sub-E=50ksi, PCC E=7000ksi, 5 by 5 ft.	48.194	-3.61	7800	11.43	49.5	-1.00	7190	2.71	48.071	-3.86	9959.790	42.28
Sub-E=50ksi, PCC E=7000ksi, 6 by 6 ft.	48.194	-3.61	7800	11.43	49.2	-1.60	8460	20.86	48.596	-2.81	11128.945	58.98

Table 8: Backcalculation Accuracy of Three Methods (Joint Stiffness=0 psi, slab thickness=5 in.)

Pavement structure	Critical Distance Method				MODCOMP 6				Hall's Equation (based on area theory)			
	Modulus of sub-structure: (ksi)	**Back Calculate Error (%)**	**Modulus of PCC: (ksi)**	**Back Calculate Error (%)**	**Modulus of sub-structure: (ksi)**	**Back Calculate Error (%)**	**Modulus of PCC: (ksi)**	**Back Calculate Error (%)**	**Modulus of sub-structure: (ksi)**	**Back Calculate Error (%)**	**Modulus of PCC: (ksi)**	**Back Calculate Error (%)**
Sub-E=20ksi, PCC E=3000ksi, 4 by 4 ft.	18.852	-5.74	**3580**	19.33	**19.9**	-0.50	**2570**	-14.33	18.554	-7.23	3526.6	17.55
Sub-E=20ksi, PCC E=3000ksi, 5 by 5 ft.	18.740	-6.30	3590	19.67	19.7	-1.50	2840	-5.33	19.539	-2.30	3420.3	14.01
Sub-E=20ksi, PCC E=3000ksi, 6 by 6 ft.	18.921	-5.39	3520	17.33	19.8	-1.00	3190	6.33	19.334	-3.33	3959.8	31.99
Sub-E=20ksi, PCC E=5000ksi, 4 by 4 ft.	18.702	-6.49	6300	26.00	20.3	1.50	3670	-26.60	17.799	-11.00	5514.6	10.29
Sub-E=20ksi, PCC E=5000ksi, 5 by 5 ft.	18.537	-7.31	6180	23.60	19.9	-0.50	4230	-15.40	19.495	-2.53	5114.8	2.30
Sub-E=20ksi, PCC E=5000ksi, 6 by 6 ft.	18.870	-5.65	5870	17.40	20.2	1.00	4680	-6.40	19.022	-4.89	6041.8	20.84
Sub-E=20ksi, PCC E=7000ksi, 4 by 4 ft.	18.588	-7.06	9400	34.29	20.5	2.50	4550	-35.00	17.009	-14.96	7520.1	7.43
Sub-E=20ksi, PCC E=7000ksi, 5 by 5 ft.	18.416	-7.92	8950	27.86	20.1	0.50	5470	-21.86	19.257	-3.72	6801.8	-2.83
Sub-E=20ksi, PCC E=7000ksi, 6 by 6 ft.	18.863	-5.68	8250	17.86	20.5	2.50	6000	-14.29	18.599	-7.01	8205.2	17.22
Sub-E=50ksi, PCC E=3000ksi, 4 by 4 ft.	48.495	-3.01	3270	9.00	49.3	-1.40	3110	3.67	47.285	-5.43	4392.1	46.40
Sub-E=50ksi, PCC E=3000ksi, 5 by 5 ft.	48.731	-2.54	3250	8.33	49.4	-1.20	3300	10.00	47.863	-4.27	4537.3	51.24
Sub-E=50ksi, PCC E=3000ksi, 6 by 6 ft.	48.430	-3.14	3300	10.00	49.1	-1.80	3890	29.67	48.393	-3.21	5066.2	68.87
Sub-E=50ksi, PCC E=5000ksi, 4 by 4 ft.	47.736	-4.53	5750	15.00	49.5	-1.00	4690	-6.20	47.211	-5.58	6359.4	27.19
Sub-E=50ksi, PCC E=5000ksi, 5 by 5 ft.	47.668	-4.66	5720	14.40	49.1	-1.80	5080	1.60	48.567	-2.87	6383.7	27.67
Sub-E=50ksi, PCC E=5000ksi, 6 by 6 ft.	47.899	-4.20	5650	13.00	49.5	-1.00	5780	15.60	48.607	-2.79	7266.5	45.33
Sub-E=50ksi, PCC E=7000ksi, 4 by 4 ft.	47.513	-4.97	8200	17.14	50.1	0.20	6020	-14.00	46.718	-6.56	8329.7	19.00
Sub-E=50ksi, PCC E=7000ksi, 5 by 5 ft.	47.154	-5.69	8200	17.14	49.3	-1.40	6680	-4.57	48.988	-2.02	8103.3	15.76
Sub-E=50ksi, PCC E=7000ksi, 6 by 6 ft.	47.612	-4.78	8000	14.29	49.7	-0.60	7500	7.14	48.510	-2.98	9332.1	33.32

Table 9: Backcalculation Accuracy of Three Methods (Joint Stiffness=40000 psi, slab thickness=3 in.)

Pavement structure	Critical Distance Method （48，60 in.）				MODCOMP 6				Hall's Equation (based on area theory)			
	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)
Sub-E=20ksi, PCC E=3000ksi, 4 by 4 ft.	19.938	-0.31	3020	0.67	19.9	-0.50	3300	10.00	19.069	-4.66	5420.2	80.67
Sub-E=20ksi, PCC E=3000ksi, 5 by 5 ft.	20.110	0.55	2950	-1.67	20	0.00	3420	14.00	18.830	-5.85	5762.4	92.08
Sub-E=20ksi, PCC E=3000ksi, 6 by 6 ft.	20.058	0.29	2990	-0.33	19.9	-0.50	4060	35.33	19.156	-4.22	6369.6	112.32
Sub-E=20ksi, PCC E=5000ksi, 4 by 4 ft.	19.516	-2.42	5370	7.40	19.7	-1.50	5170	3.40	19.571	-2.14	7322.4	46.45
Sub-E=20ksi, PCC E=5000ksi, 5 by 5 ft.	19.668	-1.66	5260	5.20	19.8	-1.00	5400	8.00	19.336	-3.32	7778.5	55.57
Sub-E=20ksi, PCC E=5000ksi, 6 by 6 ft.	19.746	-1.27	5200	4.00	19.9	-0.50	6240	24.80	19.599	-2.01	8647.7	72.95
Sub-E=20ksi, PCC E=7000ksi, 4 by 4 ft.	19.269	-3.66	7760	10.86	19.7	-1.50	6930	-1.00	19.830	-0.85	9131.6	30.45
Sub-E=20ksi, PCC E=7000ksi, 5 by 5 ft.	19.331	-3.35	7720	10.29	19.7	-1.50	7280	4.00	19.661	-1.69	9634.0	37.63
Sub-E=20ksi, PCC E=7000ksi, 6 by 6 ft.	19.444	-2.78	7580	8.29	19.8	-1.00	8280	18.29	19.823	-0.88	10742.0	53.46
Sub-E=50ksi, PCC E=3000ksi, 4 by 4 ft.	50.936	1.87	2800	-6.67	50.5	1.00	3770	25.67	44.841	-10.32	8656.5	188.55
Sub-E=50ksi, PCC E=3000ksi, 5 by 5 ft.	52.150	4.30	2540	-15.33	51.8	3.60	3710	23.67	44.723	-10.55	9052.7	201.76
Sub-E=50ksi, PCC E=3000ksi, 6 by 6 ft.	51.536	3.07	2650	-11.67	51.1	2.20	4660	55.33	45.620	-8.76	10001.2	233.37
Sub-E=50ksi, PCC E=5000ksi, 4 by 4 ft.	50.094	0.19	4950	-1.00	49.7	-0.60	5890	17.80	46.354	-7.29	10978.3	119.57
Sub-E=50ksi, PCC E=5000ksi, 5 by 5 ft.	51.536	3.07	4500	-10.00	51	2.00	5850	17.00	46.117	-7.77	11595.1	131.90
Sub-E=50ksi, PCC E=5000ksi, 6 by 6 ft.	50.936	1.87	4700	-6.00	50.4	0.80	7170	43.40	47.004	-5.99	12770.7	155.41
Sub-E=50ksi, PCC E=7000ksi, 4 by 4 ft.	49.844	-0.31	7080	1.14	49.7	-0.60	7760	10.86	47.263	-5.47	13109.3	87.28
Sub-E=50ksi, PCC E=7000ksi, 5 by 5 ft.	50.675	1.35	6650	-5.00	50.4	0.80	7910	13.00	46.867	-6.27	13861.3	98.02
Sub-E=50ksi, PCC E=7000ksi, 6 by 6 ft.	50.675	1.35	6700	-4.29	50.5	1.00	9320	33.14	47.819	-4.36	15264.1	118.06

Table 10: Backcalculation Accuracy of Three Methods (Joint Stiffness=40000 psi, slab thickness=4 in.)

Pavement structure	Critical Distance Method （48，60 in.）				MODCOMP 6				Hall's Equation (based on area theory)			
	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)
Sub-E=20ksi, PCC E=3000ksi, 4 by 4 ft.	19.233	-3.84	3350	11.67	19.8	-1.00	2990	-0.33	19.859	-0.70	3887.7	29.59
Sub-E=20ksi, PCC E=3000ksi, 5 by 5 ft.	19.331	-3.35	3300	10.00	19.8	-1.00	3140	4.67	19.686	-1.57	4103.2	36.77
Sub-E=20ksi, PCC E=3000ksi, 6 by 6 ft.	19.444	-2.78	3250	8.33	19.9	-0.50	3570	19.00	19.837	-0.82	4582.6	52.75
Sub-E=20ksi, PCC E=5000ksi, 4 by 4 ft.	18.877	-5.62	5850	17.00	19.9	-0.50	4580	-8.40	20.093	0.47	5601.8	12.04
Sub-E=20ksi, PCC E=5000ksi, 5 by 5 ft.	18.760	-6.20	5910	18.20	19.7	-1.50	4920	-1.60	20.095	0.47	5820.5	16.41
Sub-E=20ksi, PCC E=5000ksi, 6 by 6 ft.	18.984	-5.08	5780	15.60	19.9	-0.50	5460	9.20	20.130	0.65	6515.3	30.31
Sub-E=20ksi, PCC E=7000ksi, 4 by 4 ft.	18.613	-6.94	8540	22.00	20.1	0.50	6000	-14.29	20.025	0.12	7296.9	4.24
Sub-E=20ksi, PCC E=7000ksi, 5 by 5 ft.	18.487	-7.57	8570	22.43	19.8	-1.00	6560	-6.29	20.336	1.68	7460.9	6.58
Sub-E=20ksi, PCC E=7000ksi, 6 by 6 ft.	18.660	-6.70	8450	20.71	20	0.00	7240	3.43	20.263	1.32	8354.4	19.35
Sub-E=50ksi, PCC E=3000ksi, 4 by 4 ft.	49.844	-0.31	3050	1.67	49.9	-0.20	3380	12.67	47.293	-5.41	5599.8	86.66
Sub-E=50ksi, PCC E=3000ksi, 5 by 5 ft.	50.675	1.35	2860	-4.67	50.6	1.20	3450	15.00	46.949	-6.10	5910.8	97.03
Sub-E=50ksi, PCC E=3000ksi, 6 by 6 ft.	50.675	1.35	2870	-4.33	50.6	1.20	4060	35.33	47.898	-4.20	6515.1	117.17
Sub-E=50ksi, PCC E=5000ksi, 4 by 4 ft.	49.041	-1.92	5300	6.00	49.6	-0.80	5230	4.60	48.480	-3.04	7529.2	50.58
Sub-E=50ksi, PCC E=5000ksi, 5 by 5 ft.	49.844	-0.31	5060	1.20	50.2	0.40	5340	6.80	48.186	-3.63	7939.9	58.80
Sub-E=50ksi, PCC E=5000ksi, 6 by 6 ft.	50.094	0.19	5010	0.20	50.4	0.80	6210	24.20	48.941	-2.12	8845.9	76.92
Sub-E=50ksi, PCC E=7000ksi, 4 by 4 ft.	48.805	-2.39	7500	7.14	50	0.00	6840	-2.29	49.080	-1.84	9366.6	33.81
Sub-E=50ksi, PCC E=7000ksi, 5 by 5 ft.	49.041	-1.92	7350	5.00	50	0.00	7140	2.00	49.045	-1.91	9766.7	39.52
Sub-E=50ksi, PCC E=7000ksi, 6 by 6 ft.	49.281	-1.44	7300	4.29	50.3	0.60	8210	17.29	49.541	-0.92	10936.6	56.24

Table 11: Backcalculation Accuracy of Three Methods (Joint Stiffness=40000 psi, slab thickness=5 in.)

Pavement structure	Critical Distance Method （48，60 in.）				MODCOMP 6				Hall's Equation (based on area theory)			
	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)	Modulus of sub-structure: (ksi)	Back Calculate Error (%)	Modulus of PCC: (ksi)	Back Calculate Error (%)
Sub-E=20ksi, PCC E=3000ksi, 4 by 4 ft.	18.808	-5.96	3550	18.33	20.1	0.50	2690	-10.33	20.108	0.54	3238.8	7.96
Sub-E=20ksi, PCC E=3000ksi, 5 by 5 ft.	18.645	-6.78	3600	20.00	19.8	-1.00	2920	-2.67	20.227	1.14	3334.5	11.15
Sub-E=20ksi, PCC E=3000ksi, 6 by 6 ft.	18.821	-5.89	3550	18.33	20	0.00	3250	8.33	20.180	0.90	3748.7	24.96
Sub-E=20ksi, PCC E=5000ksi, 4 by 4 ft.	18.469	-7.65	6420	28.40	20.4	2.00	4010	-19.80	19.806	-0.97	4927.7	-1.45
Sub-E=20ksi, PCC E=5000ksi, 5 by 5 ft.	18.222	-8.89	6350	27.00	20	0.00	4510	-9.80	20.486	2.43	4936.0	-1.28
Sub-E=20ksi, PCC E=5000ksi, 6 by 6 ft.	18.459	-7.71	6200	24.00	20.1	0.50	4950	-1.00	20.350	1.75	5545.0	10.90
Sub-E=20ksi, PCC E=7000ksi, 4 by 4 ft.	18.294	-8.53	9500	35.71	20.6	3.00	5060	-27.71	19.424	-2.88	6520.0	-6.86
Sub-E=20ksi, PCC E=7000ksi, 5 by 5 ft.	17.955	-10.23	9400	34.29	20	0.00	6010	-14.14	20.488	2.44	6513.4	-6.95
Sub-E=20ksi, PCC E=7000ksi, 6 by 6 ft.	18.187	-9.06	8900	27.14	20.2	1.00	6560	-6.29	20.300	1.50	7294.0	4.20
Sub-E=50ksi, PCC E=3000ksi, 4 by 4 ft.	49.041	-1.92	3200	6.67	49.9	-0.20	3120	4.00	48.782	-2.44	4263.5	42.12
Sub-E=50ksi, PCC E=3000ksi, 5 by 5 ft.	49.281	-1.44	3150	5.00	50	0.00	3280	9.33	48.583	-2.83	4488.9	49.63
Sub-E=50ksi, PCC E=3000ksi, 6 by 6 ft.	49.526	-0.95	3100	3.33	50.3	0.60	3770	25.67	49.201	-1.60	5000.6	66.69
Sub-E=50ksi, PCC E=5000ksi, 4 by 4 ft.	48.037	-3.93	5560	11.20	50.1	0.20	4760	-4.80	49.369	-1.26	6074.0	21.48
Sub-E=50ksi, PCC E=5000ksi, 5 by 5 ft.	48.194	-3.61	5550	11.00	49.7	-0.60	5100	2.00	49.672	-0.66	6279.4	25.59
Sub-E=50ksi, PCC E=5000ksi, 6 by 6 ft.	48.194	-3.61	5500	10.00	50.1	0.20	5730	14.60	49.647	-0.71	7076.6	41.53
Sub-E=50ksi, PCC E=7000ksi, 4 by 4 ft.	47.815	-4.37	7950	13.57	50.8	1.60	6130	-12.43	49.459	-1.08	7791.8	11.31
Sub-E=50ksi, PCC E=7000ksi, 5 by 5 ft.	47.441	-5.12	8000	14.29	49.9	-0.20	6720	-4.00	50.333	0.67	7917.9	13.11
Sub-E=50ksi, PCC E=7000ksi, 6 by 6 ft.	47.668	-4.66	8000	14.29	50.3	0.60	7500	7.14	50.053	0.11	8987.3	28.39

Table 12: Pavement structures used to analyze the effect of joint stiffness on the accuracy of Critical Distance Method

<table>
<tr><th>Slab Thickness (in.)</th><th>E_{PCC} (ksi)</th><th>E_{sub} (ksi)</th><th>Joint spacing (ft)</th><th>Joint Stiffness (psi)</th></tr>
<tr><td rowspan="10">3
4</td><td rowspan="10">5000</td><td rowspan="10">20</td><td rowspan="10">4by4
5by5
6by6</td><td>0</td></tr>
<tr><td>200</td></tr>
<tr><td>500</td></tr>
<tr><td>1000</td></tr>
<tr><td>2000</td></tr>
<tr><td>5000</td></tr>
<tr><td>10000</td></tr>
<tr><td>20000</td></tr>
<tr><td>30000</td></tr>
<tr><td>40000</td></tr>
</table>

"KENSLAB" was used to obtain the deflections for each combination in Table 12. The Critical Distance Method was used to backcalculate sub-structure equivalent modulus and PCC modulus. The backcalculated PCC modulus was compared with the assumed PCC modulus to get the backcalculation error. The results are shown in Table 13 and Figure 31. It can be seen that the fluctuation of the backcalculation error is within 4.03%. If joint stiffness of aggregate interlock was below 10,000 psi, its effect was further reduced to within 1.13%.

Table 13: The accuracy of Critical Distance Method with Different Joint Stiffness

Joint Stiffness (psi)	Average Backcalculation Error (%)
0	14.17
200	14.30
500	14.37
1000	14.67
2000	14.70
5000	15.30
10000	15.13
20000	14.33
30000	13.20
40000	11.27

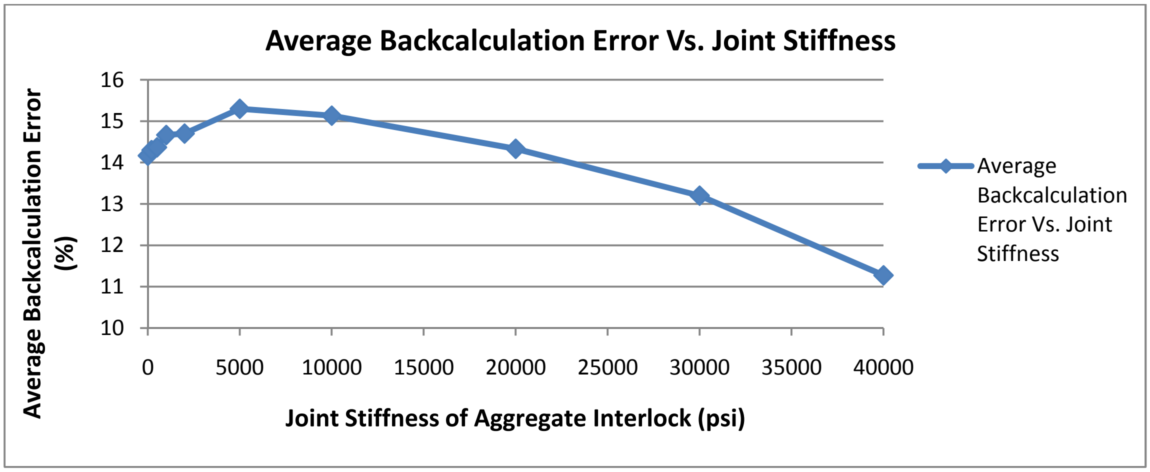

Figure 31: Statistical Comparison of the Effect of Different Joint Stiffness

The deflection differences at different distance from loading center with aggregate interlock joint stiffness of 200 psi and 40,000 psi were plotted in Figure 32. The variation between the deflection differences at distance 48 in. and 60 in. from loading center resulted in the difference in the backcalculation error.

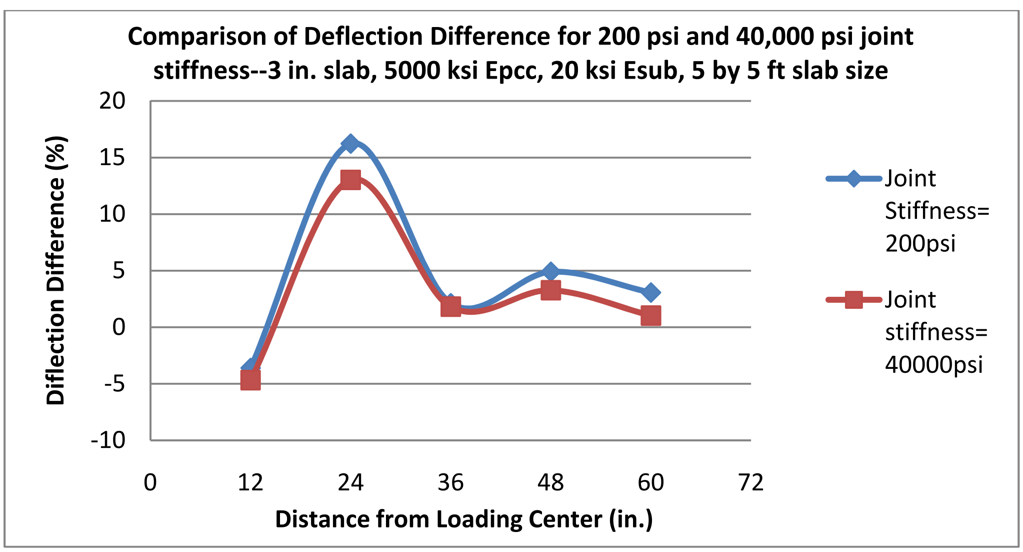

Figure 32: Comparison of Deflection Difference at Different Location for 200 psi and 40,000 psi Joint Stiffness

2) Estimate aggregate interlock joint stiffness in UTW using MEPDG

Although in UTW pavement the LTE can be as high as 80 to 90 percent (Roesler et al, 2008; Tia 2002), the joint stiffness due to aggregate interlock could be low due to the thin slab thickness. This was proved by using the models in Mechanistic-Empirical Pavement Design Guide (MEPDG). The MEPDG program was used to obtain joint width first. Joint stiffness of aggregate interlock was calculated using models in MEPDG as shown from Equations (6) to (9).

$$S_0 = 0.05 h_{PCC} e^{-0.032jw} \tag{6}$$

where: S_0= dimensionless aggregate joint shear capacity,

h_{PCC}= PCC slab thickness, in.,

jw= joint opening, mils.

$$\log(J_{AGG}) = -3.19626 + 16.09737 \times e^{-e^{-\left(\frac{S-a}{f}\right)}} \tag{7}$$

where:J_{AGG}=dimentionless joint stiffness on the transverse joint for the current increment, I,

a=0.35,

f=0.38,

S= joint shear capacity equal to S_0 at the first time increment.

$$AGG = J_{AGG}\, k\, l \tag{8}$$

where: AGG= joint stiffness, psi,

k= modulus of subgrade reaction, pci,

l = radius of relative stiffness, in., which can be calculated using Equation (9)

$$l = \left[\frac{E{h_{PCC}}^3}{12(1 - v^2)k}\right]^{0.25} \tag{9}$$

where: E= PCC modulus, psi,

h_{PCC}= slab thickness, in.,

v= Poisson's ratio,

k=modulus of subgrade reaction, pci.

The minimum slab thickness for PCC overlay on HMA pavement (whitetopping) in the MEPDG program is 6 in. A case study was conducted based on climatic data of Madison, Wisconsin, a pavement structure consisted of 6-in. slab, 6-in. HMA, and 12-in. granular aggregate base. The joint width, k-value, and PCC modulus were determined by MEPDG program. The joint stiffness was found to be 608.5 psi for the first year and 455.8 psi for the 20th year.

Because UTW pavement has equal or less than 4 in. slab thickness which cannot be simulated using MEPDG program, a manual procedure was carried out. For a 4-in. thick UTW slab, joint width was calculated using models in MEPDG, as shown in Equations (10) to (17).

$$jw = \mathrm{Max}\left(12000 * L * \beta * \left(\alpha_{PCC} * (T_{constr} - T_{mean}) + \varepsilon_{sh,m}\right), 0\right) \tag{10}$$

where: jw=joint opening, mils (0.001in.)

L=joint spacing, ft,

β=friction coefficient between the base and the PCC; assume equal 0.65 for a stabilized base and 0.85 for a granular base,

α_{PCC} =PCC coefficient of thermal expansion, in/in/ ^{o}F,

T_{mean}=mean monthly nighttime mid depth temperature, ^{o}F,

T_{constr}=PCC temperature at set, ^{o}F,

$\varepsilon_{sh,m}$=PCC slab mean shrinkage strain, which can be calculated from Equation (11)

$$\varepsilon_{sh,m} = \varepsilon_{sh,b} + (\varepsilon_{sh,t} - \varepsilon_{sh,b})\frac{h_d}{h_{PCC}} \tag{11}$$

where: $\varepsilon_{sh,m}$=PCC slab mean shrinkage strain,

$\varepsilon_{sh,b}$=shrinkage strain at the bottom surface of the PCC slab, which can be calculated from Equation (12),

$\varepsilon_{sh,t}$= shrinkage strain at the top surface of the PCC slab, which can be calculated from Equation (13),

h_d=depth of a drier portion of the PCC slab set equal to 2 in.,

h_{PCC}=PCC slab thickness, in.

$$\varepsilon_{sh,b} = \varepsilon_{su} S_t S_{hbot} \tag{12}$$

where: ε_{su}=ultimate shrinkage strain, x10^{-6}, which can be calculated from Equation (14),

S_t=time factor for moisture-related slab warping, which can be calculated from Equation (17),

S_{hbot}=relative humidity factor at the bottom of the PCC slab, assumed to be equal to 90 percent,

$$\varepsilon_{sh,t} = \varepsilon_{su} S_t (S_{h\,max} - \Phi S_{ht}) \quad (13)$$

where: ε_{su}=ultimate shrinkage strain, $x10^{-6}$, which can be calculated from Equation (14),

S_{hi}=relative humidity factor for month i, which can be calculated from Equation (16),

$S_{h\,max}$=maximum average relative humidity factor, maximum of S_{hi},

S_t=time factor for moisture-related slab warping, which can be calculated from Equation (17).

$$\varepsilon_{su} = C_1 C_{2t} (26 w^{2.1} (f_c')^{-0.28} + 270) \quad (14)$$

where: ε_{su}=ultimate shrinkage strain, $x10^{-6}$,

C_1=cement type factor: 1.0 for type I cement, 0.85 for type II cement, 1.10 for type III cement,

C_2=type of curing factor: 0.75 if steam cured, 1.0 if cured in water or 100% relative humidity,

w=water content, lb/ft^3 for the PCC mix under consideration,

f_c'=28-day PCC compressive strength, psi (determined from AASHTO T22). Here can use PCC modulus to predict using Equation (15), in order to using same value as provided by MEPDG program.

$$f_c' = \left(\frac{E}{5700}\right)^2 \quad (15)$$

where: $f_c^{'}$=28-day PCC compressive strength, psi,

E=PCC modulus of elasticity, psi.

$$S_{hi} = 1.1RH_a \qquad \text{for } RH_a < 30\%$$
$$S_{hi} = 1.4 - 0.01RH_a \qquad \text{for } 30\% < RH_a < 80\% \tag{16}$$
$$S_{hi} = 3.0 - 0.03RH_a \qquad \text{for } RH_a \geq 80\%$$

Where: S_{hi}=relative humidity factor for month I,

RH_a=ambient average relative humidity, percent.

$$S_t = \frac{Age}{n + Age} \tag{17}$$

where: S_t=time factor for moisture-related slab warping,

Age=PCC age, days since placement,

n=time to develop 50% of ultimate shrinkage strain, days. Use 35 (the ACI Committee 209 recommended value), unless more accurate information is available.

After joint width was calculated, the joint stiffness of aggregate interlock could be calculated using Equations (6) to (9) (from MEPDG). A manual procedure was carried out for a 4-in. thick slab. The assumed values are shown in Table 14. The joint stiffness due to aggregate interlock was 1459.9 psi for the first month of service. It should be noted that there could be some difference in the input, such as the environmental data, between manual procedure and the MEPDG program. It's obvious that this difference could have effect on the resulted aggregate interlock joint stiffness. However it's still conservative to assume the joint stiffness of aggregate interlock is below 2000 psi for UTW pavement. The MEPDG indicated a LTE of 40% based on 30% "base LTE" for a 6-in. whitetopping pavement. This is significantly different from the in-situ test data which was as high as 80 to 90 percent (Roesler, J, A., 2008; Tia 2002). Therefore a constant value of 30% LTE for base may not be appropriate for whitetopping pavement.

Table 14: Input Values Used in Manual Procedure to Calculate Joint Stiffness

v	**E**	**K**	**H_{PCC}**	**L, ft**	**β**	**α_{PCC}**	**T_{mean}, ^{O}F**	**T_{constr}, ^{O}F**
0.15	4862500	175.25	4	6	0.65	0.000005	70	75
h_d	**Φ**	**RH_a,**	**S_{hbot}**	**Age, days**	**n**	**C_1**	**C_2**	**w, lb/ft^3**
2	0.5	0.4	90%	30	35	0.85	1.2	12

4.4 COMPARISON WITH TRADITIONAL METHODS

Using the deflections obtained from pavement simulations, backcalculations were performed using Equations (2) to (4) which was based on "AREA" theory (Hall et al. 1991).The Modcomp 6 program (Irwin 2003) was used as another approach for backcalculation. Results from the "AREA" theory and Modcomp 6 were compared with those of Critical Distance Method, as shown in Table 6 through 11 for slab thickness of 3 in., 4 in. and 5 in. with aggregate interlock joint stiffness of 0 psi and 40,000 psi. The comparisons were plotted in Figures 33 through 38 as well.

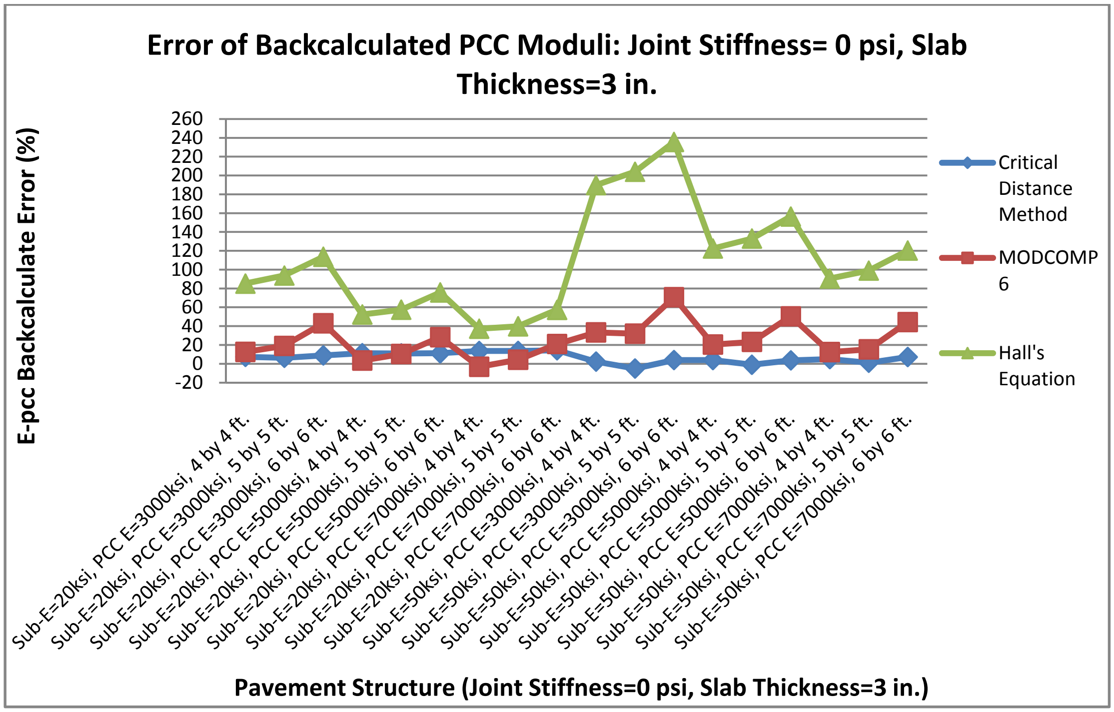

Figure 33: Comparison of Three Methods: Joint Stiffness=0 psi, Slab Thickness=3 in.

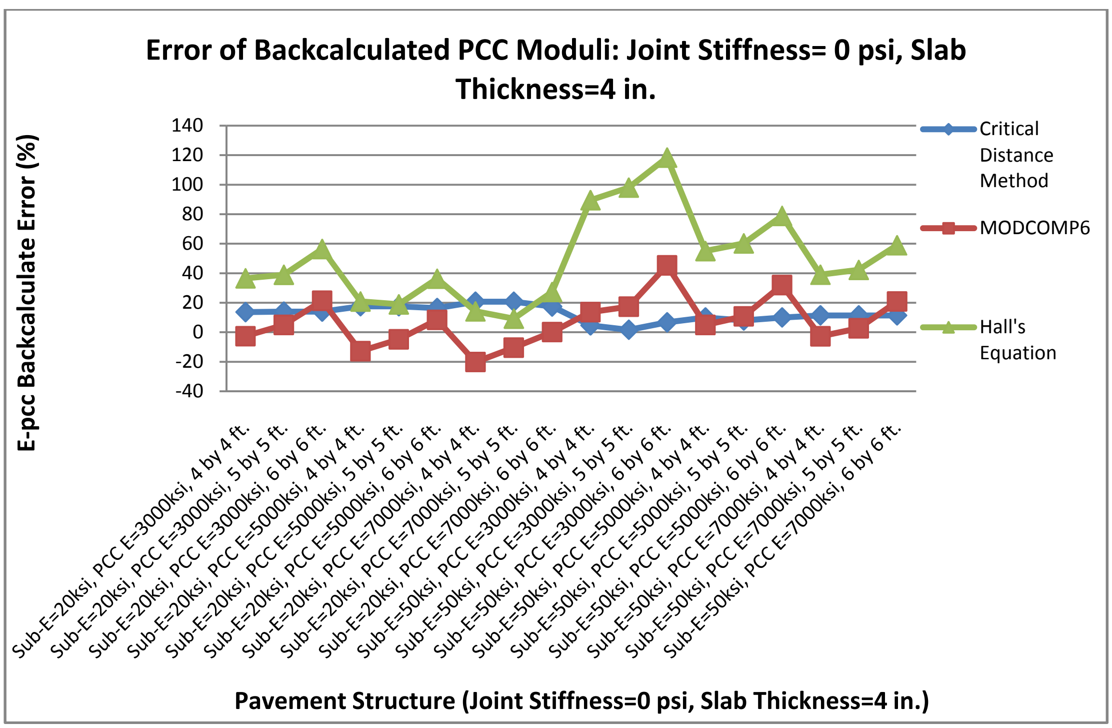

Figure 34: Comparison of Three Methods: Joint Stiffness=0 psi, Slab Thickness=4 in.

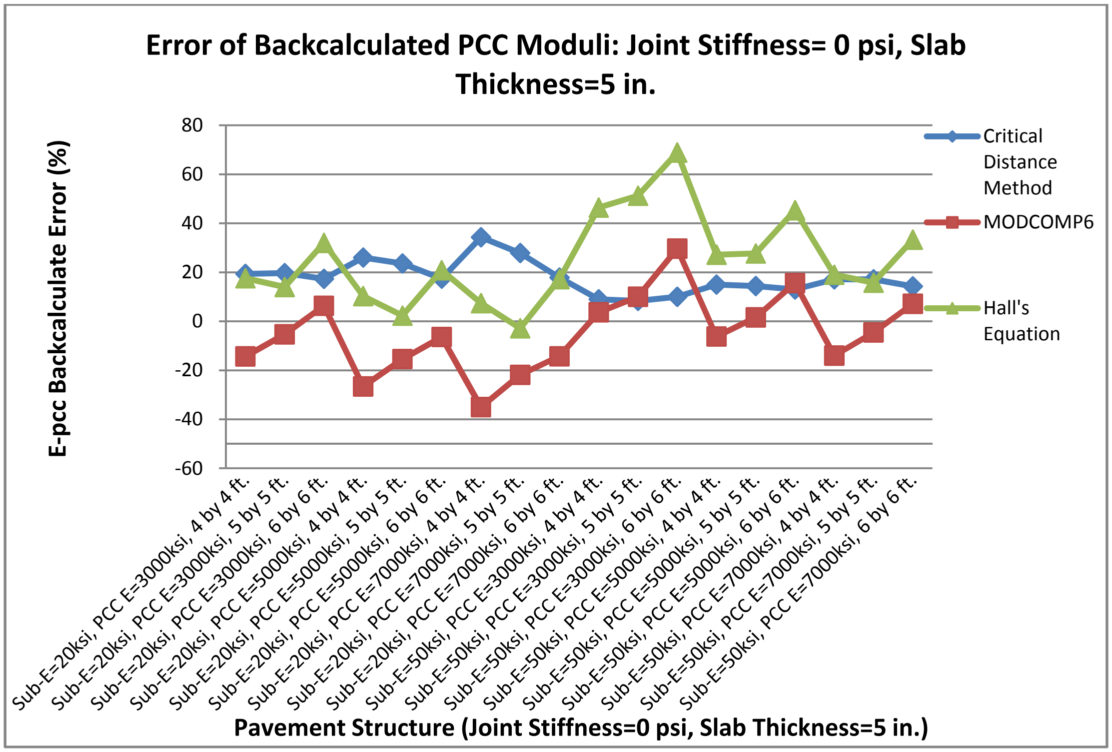

Figure 35: Comparison of Three Methods: Joint Stiffness=0 psi, Slab Thickness=5 in.

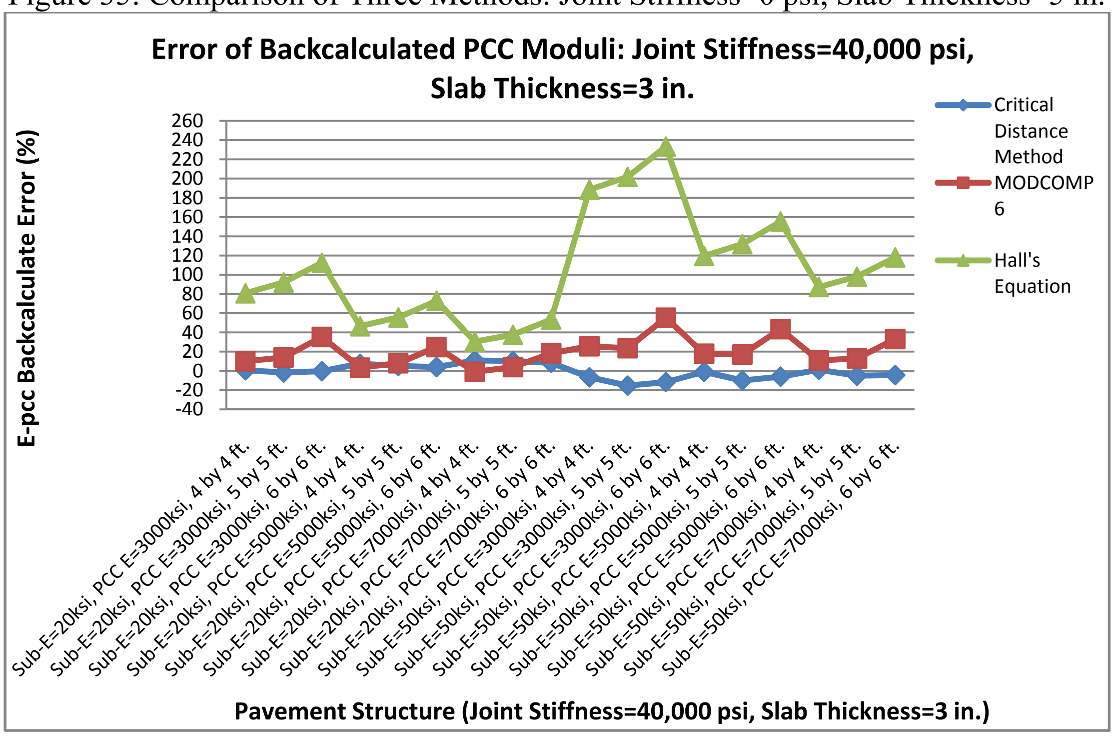

Figure 36: Comparison of Three Methods: Joint Stiffness=40,000psi, Slab Thickness=3 in.

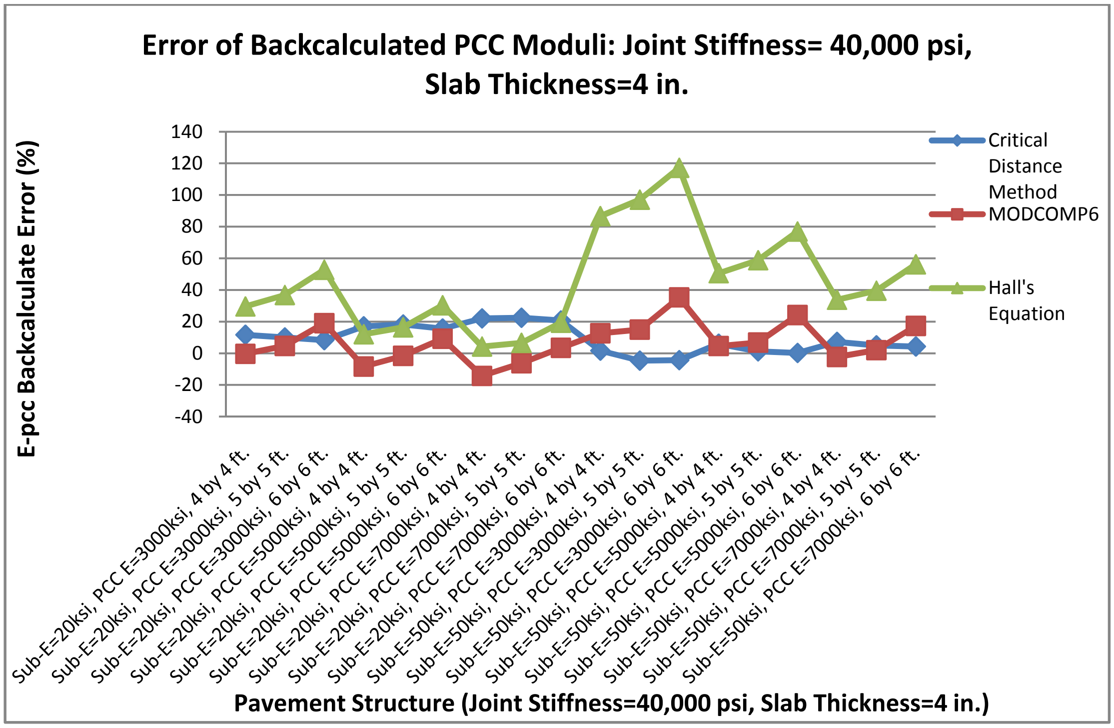

Figure 37: Comparison of Three Methods: Joint Stiffness=40,000psi, Slab Thickness=4 in.

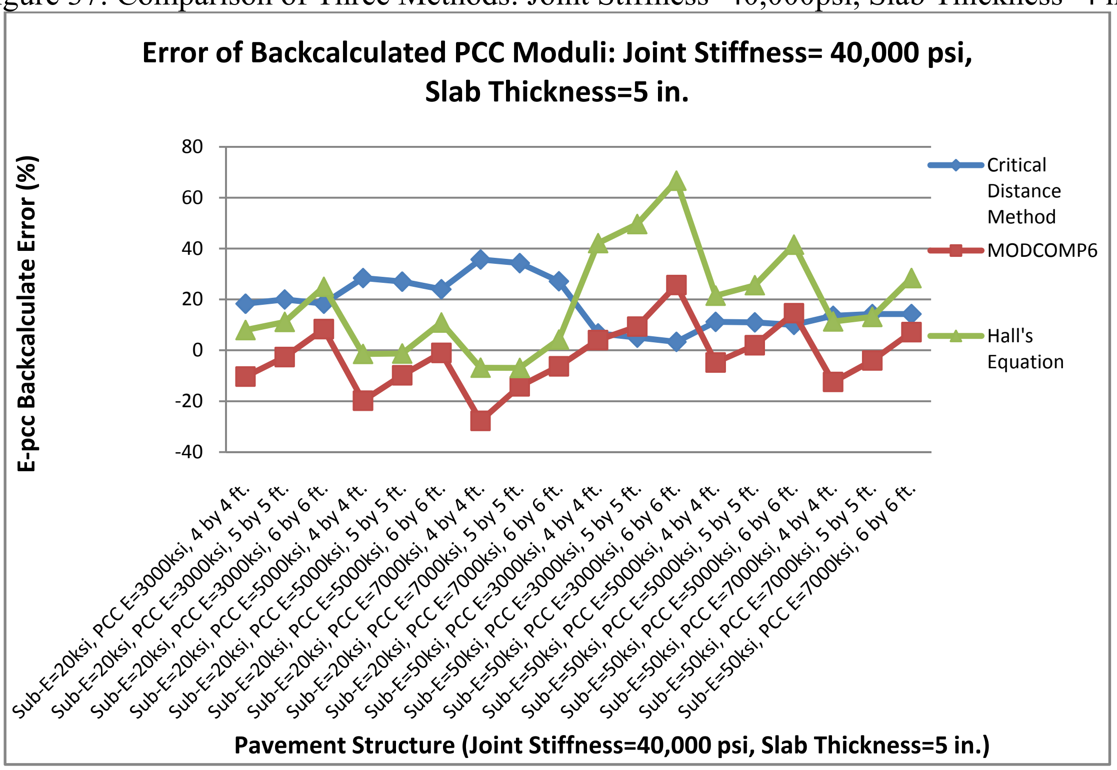

Figure 38: Comparison of Three Methods: Joint Stiffness=40,000psi, Slab Thickness=5 in.

Based on this study, the accuracy of Critical Distance Method for sub-structure moduli is within 4.43% for 3 in. slab thickness, 7.57% for 4 in. slab thickness and 10.23% for 5 in, slab thickness respectively. For PCC moduli, the accuracy is within 15.33% for 3 in. slab thickness, 22.43% for 4 in. slab thickness and 35.71% for 5 in. slab thickness, respectively. Further analysis indicated that the accuracy increased with the decrease of slab thickness and slab modulus or with the increase of underlying support. The maximum error of the backcalculated sub-structure and PCC moduli using these three methods for different slab thickness are shown in Table 15. From Table 15, it can be seen that, from the maximum error standpoint, Critical Distance Method are more accurate for 3 in. and 4 in. slab thickness and has almost same accuracy with Modcomp6 for 5 in. slab.

Table 15: maximum error of the three methods for different slab thickness

Slab Thckness (in.)	Critical Distance Method		MODCOMP 6		Hall's Equation ("AREA" theory)	
	Max-Error of E-sub (%)	Max-Error of E-PCC (%)	Max-Error of E-sub (%)	Max-Error of E-PCC (%)	Max-Error of E-sub (%)	Max-Error of E-PCC (%)
3	4.43	15.33	3.60	70.67	11.33	235.67
4	7.57	22.43	2.60	45.33	8.16	118.36
5	10.23	35.71	3.00	35.00	14.96	68.87

Statistical test results were shown in Figure 39 to compare Critical Distance method with MODCOMP6 and Hall's equation (based on "AREA" theory). It can be seen from Figure 39 that Critical Distance Method and MODCOMP6 are better than Hall's equation for all the three slab thicknesses used in this study. When slab thickness is 4 in., Critical Distance method has almost same accuracy with MODCOMP6. When slab thickness is 3 in. (less than 4 in.), Critical Distance Method is better than MODCOMP6. Therefore, for UTW pavement which the slab thickness is typically less or equal to 4 in., Critical Distance Method is more accurate than the other two methods.

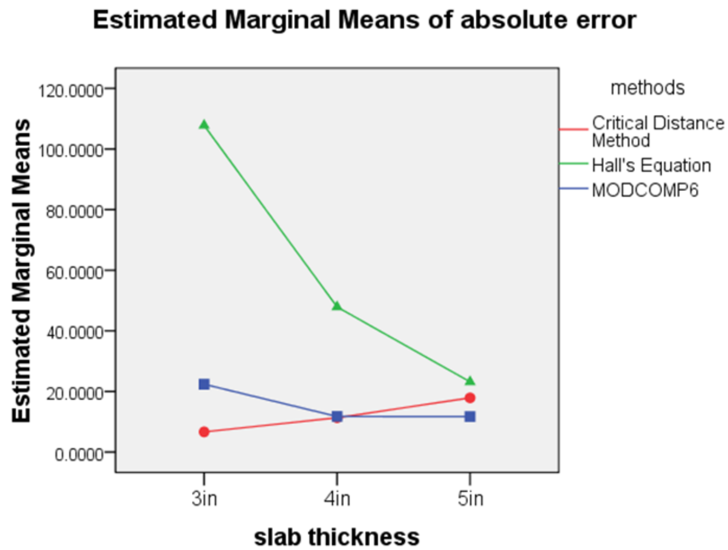

Figure 39: Statistical Comparison of Backcalculation Error of Three Methods

Further study showed that the "relative slab stiffness" which is defined by Equation (18) has high correlation with the backcalculation accuracy as shown in Figures 40 through 42.

$$\text{Relative slab stiffness} = \frac{h_{PCC}^2 E_{PCC}}{E_{sub}} \quad (18)$$

where: h_{pcc}= slab thickness, in.,

E_{pcc}= slab modulus, psi,

E_{sub}= equivalent sub-structure modulus, psi.

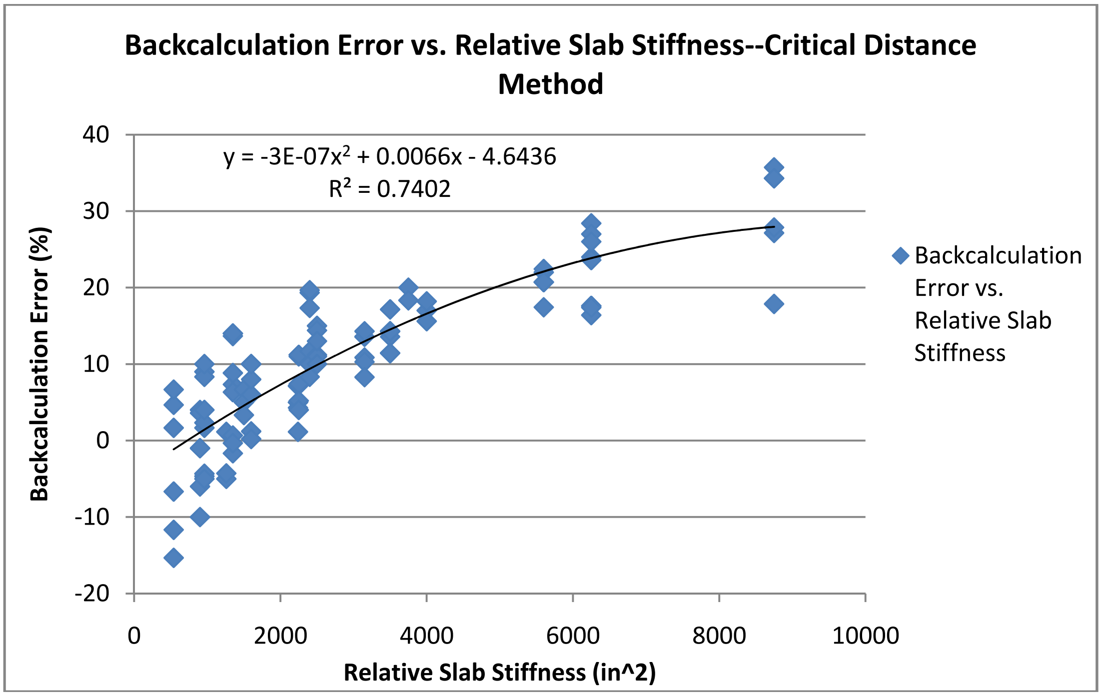

Figure 40: Relation Between Backcalculation Error and "Relative Slab Stiffness"—Critical Distance Method

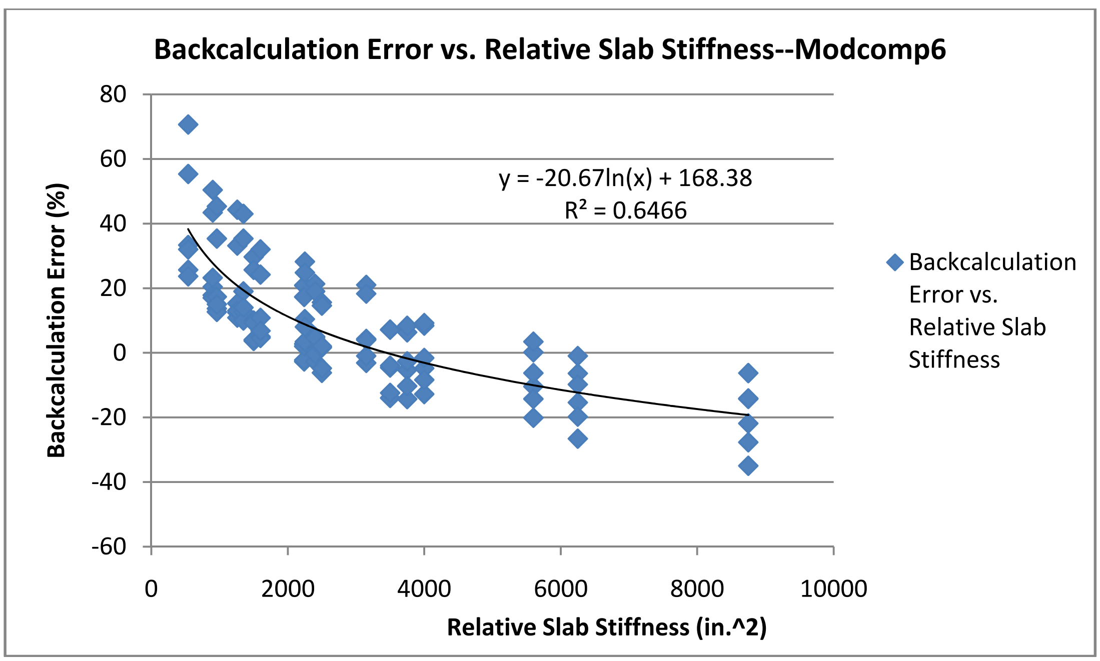

Figure 41: Relation Between Backcalculation Error and "Relative Slab Stiffness"—Modcomp6 Program

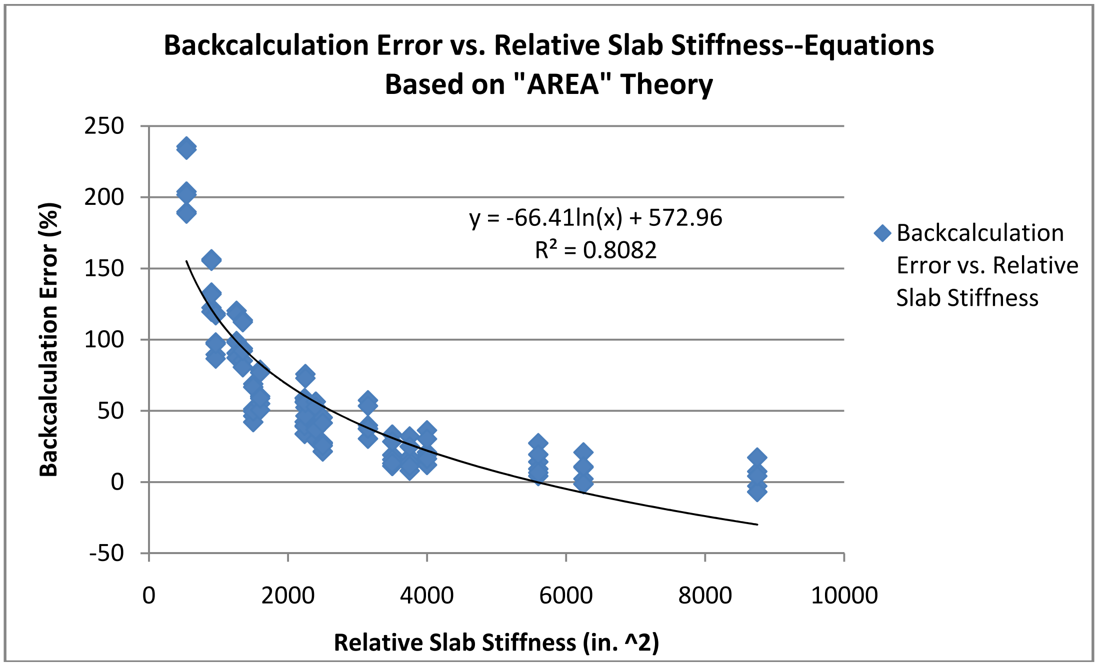

Figure 42: Relation Between Backcalculation Error and "Relative Slab Stiffness"—Hall's Equations Based on "AREA" Theory

From Figures 40 through 42, it can be seen that with the decrease of the "relative slab stiffness", the backcalculation accuracy increased for Critical Distance Method, but decreased for Modcomp6 and Hall's equation. When "relative slab stiffness" is below 5000 in^2, Critical Distance Method has the error within 20%. Even when "relative slab stiffness" increased to 10,000 in^2, the error is still within about 30%. Modcomp6 and Hall's equation are suitable when "relative slab stiffness" is more than 2000 in^2 or 4000 in^2 respectively. However, when "relative slab stiffness" is below 2000 in^2, the error tends to increase steeply. It is noted when using Modcomp6, when "relative slab stiffness" is more than 8000 in^2, the error based on Modcomp6 starts increasing again in a reversed direction. Based on the analysis, Equations (19) to (21) could be used to estimate error of the backcalculated PCC moduli for Critical Distance Method, Modcomp6, and Hall's equation respectively:

$$y = -0.0000003(x)^2 + 0.0066(x) - 4.6436 \qquad (19)$$

$$y = -20.67\ln(x) + 168.38 \qquad (20)$$

$$y = -66.41\ln(x) + 572.96 \qquad (21)$$

where: y= backcalculation error, %,

x=relative slab stiffness, in^2.

When "relative slab stiffness" is low, which is the case for UTW pavement, Modcomp6 and Hall's equation resulted in large error. This proved the hypothesis that the traditional method of Modcomp6 and equations based on "AREA" theory were not suitable to FWD backcalculation of UTW.

4.5 CONCLUSIONS OF CRITICAL DISTANCE METHOD

The traditional backcalculation method of pavement layer properties, based on FWD testing, is not applicable to UTW pavements. The new Critical Distance Method based on St. Venant's principle can be used for modulus backcalculation of UTW pavement. A Critical Distance of 36 in. or 48 in. from the center of the loading plate is recommended. The determination of Critical Distance needs to consider the location of the joint. The accuracy of backcalculated moduli is within 7.57% for equivalent sub-structure and 22.43% for PCC slab when slab thickness is equal or less than 4 in. When "relative slab stiffness" is below 5000 in^2, Critical Distance Method has the backcalculation accuracy within 20%. Traditional backcalculation methods, such as the “AREA” theory and Modcomp 6, are fairly accurate in backcalculating the modulus of substructure. However, the error for PCC modulus is excessive when the “relative slab stiffness” is low. It is demonstrated that the Critical Distance Method is more accurate for UTW pavement evaluation when compared to Modcomp6 and the models based on “AREA” theory.

CHAPTER 5: EVALUATION OF WHITETOPPING PAVEMENT

This chapter provides an evaluation of the performance of selected whitetopping pavements. The FWD test results were used to backcalculate the layer properties of whitetopping pavements and then estimate the fatigue lives. Statistical analysis was conducted to develop relationship between design/construction variables and pavement performance from field distress survey. The bond strength between PCC and HMA was analyzed.

5.1 ANALYSIS BASED ON FWD TEST

5.1.1 Data Preparation

Before processing the FWD test data, a quality check was conducted to remove abnormal test data, such as higher deflection at farther distance. The data for Fond Du Lac Ave was abnormal. This is probably attributable to the severe slab breakup. Unreasonable data was also found for Duplainville Road. This was likely due to the malfunction of FWD equipment. Therefore, only Lawndale Avenue, CTH “A” and STH 82 were included in FWD test backcalculation in this study.

5.1.2 FWD Backcalculation Results

For the three projects, Lawndale Avenue has a slab thickness of 4 in., which means using the Critical Distance Method is appropriate. As a comparison, backcalculated moduli using Modcomp 6 for this project was also provided. The Critical Distance Method and Modcomp6 were both used for STH 82 which has a slab thickness of 5 in. to demonstrate the procedure to use Equations (19) and (20) to estimate the backcalculation error of PCC moduli. CTH "A" has a slab thickness of 7.5 in. for which Modcomp 6 is appropriate.

A Poisson's ratio of 0.15 for the PCC slab and 0.42 for sub-structure was assumed in this study. Due to the significant variation of the pavement conditions for each project, the backcalculation was performed station by station, as shown in Tables 16 through 18.

Table 16: Backcalculation Results of Lawndale Avenue

Projects	Station	MODCOMP6 Program				Critical Distance Method			
		$E_{sub-structure}$ (ksi)	E_{PCC} (ksi)	Relative Slab Stiffness (in^2)	Estimated Error (%)	$E_{sub-structure}$ (ksi)	E_{PCC} (ksi)	Relative Slab Stiffness (in^2)	Estimated Error (%)
Lawndale Avenue	0	28.7	533	297	50.68	23.749	483	325	-2.53
	7	25.6	3770	2356	7.88	18.894	8350	7071	27.03
	13	25.6	2750	1719	14.40	21.738	3780	2782	11.40
	16	33.6	1130	538	38.41	26.248	1570	957	1.40
	43	26.6	1690	1017	25.26	26.696	1130	677	-0.31
	49	38.1	697	293	50.99	31.38	650	331	-2.49
	56	34.8	756	348	47.44	30.412	565	297	-2.71
	62	35.2	645	293	50.96	30.033	492	262	-2.93

Table 17: Backcalculation Results of CTH "A"

Projects	MODCOMP6 Program					
	Stations	$E_{sub-structure}$ (ksi)	E_{PCC} (ksi)	Stations	$E_{sub-structure}$ (ksi)	E_{PCC} (ksi)
CTH A	85	17.2	11700	9899	18.7	14200
	102	20.3	12700	9912	27.7	9360
	118	26.3	8360	9928	28.6	8240
	135	19.0	10500	15093	40.5	6720
	148	14.4	16100	15102	47.5	5260
	5023	21.2	5110	15119	49.0	4120
	5043	16.7	11500	15132	54.8	5650
	5056	23.6	9400	15148	38.8	3800
	5076	20.8	13100	20008	58.9	3800
	5092	31.8	13200	20024	32.2	4600
	9873	34.0	4380	20037	53.8	3700
	9882	26.8	6970	20053	47.9	4960

Table 18: Backcalculation Results of STH 82

Projects	Station	MODCOMP6 Program				Critical Distance Method			
		$E_{sub\text{-}structure}$ (ksi)	E_{PCC} (ksi)	Relative Slab Stiffness (in^2)	Estimated Error (%)	$E_{sub\text{-}structure}$ (ksi)	E_{PCC} (ksi)	Relative Slab Stiffness (in^2)	Estimated Error (%)
STH 82	495	20	6080	4864	-7.10	19.124	8950	7488	27.96
	502	19.5	1160	952	26.62	17.769	32000	28814	-63.55
	509	23.7	7570	5111	-8.12	21.571	13300	9865	31.27
	515	23.1	6140	4253	-4.32	21.863	9000	6586	25.81
	6569	16.3	7900	7755	-16.74	14.445	20500	22707	-9.46
	6575	21	2550	1943	11.87	19.914	3180	2555	10.26
	6582	19.4	4170	3439	0.06	16.746	7630	7290	27.53
	6588	22.2	4520	3258	1.19	21.088	6200	4704	19.76
	6595	19.8	3740	3022	2.74	16.464	7350	7143	27.19
	9876	22.9	5900	4122	-3.68	22.887	7300	5103	21.23
	9882	23.6	4050	2746	4.72	21.534	5950	4421	18.67
	9889	23.5	5500	3745	-1.69	21.79	8300	6095	24.44
	9892	22.1	3070	2223	9.09	20.777	4020	3096	12.91
	9899	21.1	3880	2942	3.29	19.664	5500	4475	18.88
	15296	21	3490	2659	5.38	17.491	6570	6010	24.19
	15306	21.3	3900	2930	3.38	20.461	5050	3949	16.74
	15319	24.1	3290	2184	9.45	22.856	4170	2919	12.07
	15329	22.8	3880	2723	4.89	22.913	4350	3038	12.64
	15342	23.7	1670	1127	23.12	21.026	2280	1735	5.90
	21176	14.6	2550	2795	4.35	13.166	3900	4739	19.90
	21182	5.3	2200	6642	-13.54	4.006	12800	51123	-451.31
	21192	19.6	3130	2555	6.21	19.381	3620	2988	12.40
	21199	9.6	4710	7850	-16.99	7.929	19150	38643	-197.58
	21202	18.6	7330	6305	-12.47	16.588	15600	15047	26.74
	26425	26.3	5290	3218	1.44	25.748	6600	4101	17.38
	26435	30	5130	2736	4.79	28.638	6660	3721	15.76
	26442	29.2	2910	1595	15.95	24.313	5050	3323	13.98
	26451	30.2	6460	3423	0.16	28.645	8900	4971	20.75
	26458	33.5	3960	1891	12.42	29.184	6200	3399	14.32

From Table 16, it can be seen that the sub-structure moduli obtained from Modcomp 6 were consistent with the corresponding moduli from the Critical Distance Method. However, the PCC moduli showed large discrepancies between these two methods. The estimated error shown in Table 16 was calculated from Equations (19) and (20). It indicated that the pavement properties backcalculated from Critical Distance

Method were more accurate. In addition, the PCC moduli show large variations between the stations. The moduli at different stations range from 483ksi to 8,350ksi. The variation might be resulted from some broken slabs on which the FWD tests were conducted. Photos of Lawndale Avenue are shown in Figure 43, indicating that some slabs were broken.

Figure 43: Photos of Lawndale Avenue

The average backcalculated modulus for Lawndale Ave was 1,811.94ksi. Based on the normal range of PCC moduli, it is reasonable to consider a slab as completely broken if the backcalculated PCC modulus is below 1,000ksi (Bush, A. J. et al, 1989). For Lawndale Avenue, slabs at 4 out of 8 stations, were considered as completely broken, based on the backcalculated PCC moduli. However, even if the backcalculated PCC modulus is above 1,000ksi, it does not necessarily mean the slab is undamaged. Micro-cracks in the slab may reduce its modulus, but not to a level of 1,000ksi or lower. The threshold to discriminate the structural conditions of slabs using FWD backcalculated modulus needs future study.

The estimated errors in Table 18 were calculated using Equations (19) and (20) as well. From Table 18, it can be seen that the estimated error of Modcomp6 is smaller than that of Critical Distance Method. Therefore the pavement properties backcalculated from Modcomp6 was used as results for further study. For STH 82, the average PCC modulus was 4,349.31ksi with a standard deviation of 1,739.16ksi. STH 82 was in good condition with minimal distresses. Of these three projects, CTH “A” had the highest PCC modulus, 8,226.25ksi with a standard deviation of 3,850.24ksi. CTH “A” has been in service for only one year and is in excellent condition. It can be seen that the backcalculated PCC modulus correlates with the field performance, which will be addressed in detail in a later section of this thesis.

The average sub-structure equivalent moduli for STH 82, Lawndale Ave, and CTH “A” were 21.7, 26.1, and 32.1ksi, respectively. The difference among the backcalculated sub-structure moduli of the three projects could be explained by the pavement structure that was underlying the whitetopping overlay. CTH “A” has a relatively strong sub-structure with 7-in. HMA and 14-in. CABC. Lawndale Avenue has a substructure of 3.5-in. HMA and 9-in. CABC. STH 82 has an HMA thickness of 1.5 in. and an unknown thickness of CABC. The backcalculated equivalent substructure modulus correlates with the thickness of the HMA.

In summary, the backcalculated layer properties of whitetopping pavements correlated with the field conditions and can be used as an indicator for pavement performance and structural capacity. The backcalculated layer properties were used to predict the remaining fatigue lives of these whitetopping pavements, based on finite element modeling of pavement.

5.1.3 3-D Finite Element Modeling Based on FWD Backcalculation

The backcalculated layer properties were input into a finite element program to obtain the critical stresses for predicting the remaining fatigue lives of whitetopping pavements. Due to the large variations of the backcalculated PCC moduli, this process was carried out on the basis of stations.

To determine the critical stresses using the 3-D finite element (3-D FE) modeling, the critical loading position and the loading level have to be considered.

1) The Critical Loading Position.

In order to simulate real traffic loading condition, a traffic load should be applied at the wheel path according to the slab layout on the road. Considering the traffic wandering and the relatively small slab size for Lawndale Ave. and STH 82, critical loading position was analyzed first. Real pavement structures were used along with the assumption of 2 levels of PCC moduli, 2,000 ksi and 4,000 ksi, and 2 levels of composite k-values, 300 pci and 500 pci. An 18 kip single axle was applied on the middle of edge or corner of the slab. The maximum tensile stresses, either on the surface or bottom of the slab, were determined. Table 19 shows different combinations of pavement properties and the modeling results using EverFE (WS DOT, 2005), a 3-D FE program.

From Table 19, it can be seen that for Lawndale Avenue and STH 82, the critical stress is at the bottom of the slab when loaded at the middle of slab edge. For CTH “A”, the loading position was selected at the middle of the slab edge, due to the large slab size of 15 by 15 ft. (See Figure 44).

Table 19: Comparison of the Maximum Tensile Stress - Loading at Corner and Edge

Lawndale Avenue					
Loading Position	E_{PCC} (ksi)	2000		4000	
	k-value (pci)	300	500	300	500
Corner	σ-corner max (psi)	412	359	488	430
Edge	σ-edge max (psi)	**485**	**416**	**575**	**510**
STH 82					
Loading Position	E_{PCC} (ksi)	2000		4000	
	k-value (pci)	300	500	300	500
Corner	σ-corner max (psi)	340	306	376	350
Edge	σ-edge max (psi)	**357**	**326**	**388**	**366**

Figure 44: Slab Layout on CTH A

2) Loading Level.

The AASHTO pavement design guide (1993) uses the Equivalent Single Axle Load (ESAL) as design input and converts other load groups into ESALs using the Equivalent Axle Load Factor which was based on the AASHTO test road. Preliminary analysis indicated that for whitetopping pavement with strong base support, the standard 18-kip axle load resulted in an indefinite number of loads in most of the cases. Therefore, it seems that loads heavier than 18 kips cause the damage to the concrete slab. Similar to the study in Florida on whitetopping pavements (Wu et al 1998), a range of axle loads, 18, 22, 26 kip (single axle, dual tire), were used in the modeling.

The pavement ages of STH 82 and Lawndale Avenue were more than 7 years as of 2008 when the FWD test was conducted. Based on the discussion in section 4.2.3.3, in KENSLAB modeling, it was assumed that the load transfer between slabs mainly provided by the sub-structure support, instead of aggregate interlock. For CTH "A", due to the 15 ft. by 15 ft. slab size, only one slab was modeled. The KENSLAB modeling results are shown in Tables 20 through 25.

5.1.4 Remaining Fatigue Life Analysis

Using KENSLAB modeling results, remaining fatigue life analysis was performed using the fatigue equations recommended by the Portland Cement Association (Packard and Tayabji, 1985), as below:

$$\text{For } \frac{\sigma}{S_C} \geq 0.55: \ \log N_f = 11.737 - 12.077\left(\frac{\sigma}{S_C}\right) \quad (22)$$

$$\text{For } 0.45 < \frac{\sigma}{S_C} < 0.55: \ N_f = \left(\frac{4.2577}{\sigma/S_C - 0.4325}\right)^{3.268} \quad (23)$$

$$\text{For } \frac{\sigma}{S_C} \leq 0.45: \ N_f = \text{unlimited} \quad (24)$$

where: N_f is the allowable number of traffic repetitions,

σ is the flexural stress in slab in psi,

S_C is the modulus of rupture of concrete in psi, which can be calculated from:

$$S_C = \frac{43.5E_C}{10^6} + 488.5 \qquad (25)$$

where: E_C is the concrete modulus of elasticity in psi, which is the backcalculated PCC modulus in this study.

The fatigue life analysis results are shown in Tables 20 through 22. For Lawndale Ave., the backcalculated PCC moduli for some stations were low (as shown in Table 20), indicating broken slabs, as discussed previously. When the slabs are broken, tensile stresses at the bottom of broken slabs are low, because most of the loads are carried by the underlying layers. The low tensile stress, however, could result in unlimited number of loads to carry, which is not reasonable. Therefore, care should be exercised in analyzing the fatigue life for pavement with broken slabs.

From Table 21, it can be seen that for CTH “A”, the loads used resulted in unlimited fatigue life at all stations. One of the explanations relies on the relatively strong pavement structure of this project. It has a 7.5 in. slab, 7 in. of HMA, and 14 in. of CABC. Higher load might have to be used.

It can be seen from Tables 20 through 22 that the whitetopping pavements are very sensitive to the heavy loads. Increasing the load level from 18 kips to 26 kips significantly reduced the fatigue lives, especially for Lawndale Ave. and STH 82. Therefore, for whitetopping pavements, the design should be based on real traffic spectra or loads heavier than the standard 18-kip axle loads.

The thermal stresses were obtained using KENSLAB, assuming 3 ^{o}F/in. temperature gradient in PCC slab. for each project are shown in Tables 23 through 25. As expected, thin slab thickness and short joint spacing greatly reduced the thermal stresses.

CTH “A” with thickness of 7 in. and joint spacing of 15 ft. by 15 ft. has the highest thermal stress among these three projects while Lawdale has lowest thermal stress due to its thin slab thickness. Thermal stress has little effect for typical UTW overlay due to the relatively thin slab thickness and short joint spacing. However, if the joint spacing increased, like in CTH “A”, using 15 ft. by 15 ft., thermal stress could have a significant effect and could become a major cause of fatigue.

Table 20: KENSLAB Modeling and Remaining Fatigue Life Analysis Results

(Lawndale Ave. 18, 22, 26 kip load)

Load Level	**Stations**	**H_{pcc}**	**Es (ksi)**	**Epcc (ksi)**	**Stress (18kip)**	**PCC Strength**	**Stress ratio R**	**R>0.55**	**0.45<R<0.5 5**	**R<0.45**
	0	4.00	23.749	483	110.6	n/a	n/a			broken
	7	4.00	18.894	8350	515.6	851.7	0.605	26,673		
	13	4.00	21.738	3780	363.4	652.9	0.557	103,592		
18 kips	16	4.00	26.248	1570	217.6	556.8	0.391			unlimited
	43	4.00	26.696	1130	178.5	537.7	0.332			unlimited
	49	4.00	31.380	650	112.0	n/a	n/a			broken
	56	4.00	30.412	565	103.7	n/a	n/a			broken
	62	4.00	30.033	492	94.6	n/a	n/a			broken
Load Level	**Stations**	**Hpc c**	**Es (ksi)**	**Epcc (ksi)**	**Stress (22kip)**	**PCC Strength**	**Stress ratio R**	**R>0.55**	**0.45<R<0.5 5**	**R<0.45**
	0	4.00	23.749	483	135.2	509.5	0.265			broken
	7	4.00	18.894	8350	630.1	851.7	0.740	635		
	13	4.00	21.738	3780	444.2	652.9	0.680	3,317		
22 kips	16	4.00	26.248	1570	265.9	556.8	0.478		2,855,784	
	43	4.00	26.696	1130	218.2	537.7	0.406			unlimited
	49	4.00	31.380	650	136.9	516.8	0.265			broken
	56	4.00	30.412	565	126.7	513.1	0.247			broken
	62	4.00	30.033	492	115.6	509.9	0.227			broken
Load Level	**Stations**	**Hpc c**	**Es (ksi)**	**Epcc (ksi)**	**Stress (26kip)**	**PCC Strength**	**Stress ratio R**	**R>0.55**	**0.45<R<0.5 5**	**R<0.45**
	0	4.00	23.749	483	159.7	509.5	0.313			broken
	7	4.00	18.894	8350	744.7	851.7	0.874	15		
	13	4.00	21.738	3780	525.0	652.9	0.804	106		
26 kips	16	4.00	26.248	1570	314.3	556.8	0.564	83,132		
	43	4.00	26.696	1130	257.8	537.7	0.479		2,489,094	
	49	4.00	31.380	650	161.8	516.8	0.313			broken
	56	4.00	30.412	565	149.7	513.1	0.292			broken
	62	4.00	30.033	492	136.6	509.9	0.268			broken

Table 21: KENSLAB Modeling and Remaining Fatigue Life Analysis Results

(CTH "A". 18, 26 kip load)

Load Levels	**Stations**	**Hpcc**	**Es (ksi)**	**Epcc (ksi)**	**Stress under 18kip load**	**PCC Strength**	**Stress ratio R**	**R>0.55**	**0.45<R<0.55**	**R<0.45**
18 kips	85	7.50	17.200	11700	237.9	997.5	0.239			unlimited
	102	7.50	20.300	12700	234.3	1041.0	0.225			unlimited
	118	7.50	26.300	8360	204.8	852.2	0.240			unlimited
	135	7.50	19.000	10500	228.9	945.3	0.242			unlimited
	148	7.50	14.400	16100	259.7	1188.9	0.218			unlimited
	5023	7.50	21.200	5110	192.6	710.8	0.271			unlimited
	5043	7.50	16.700	11500	238.5	988.8	0.241			unlimited
	5056	7.50	23.600	9400	214.7	897.4	0.239			unlimited
	5076	7.50	20.800	13100	234.6	1058.4	0.222			unlimited
	5092	7.50	31.800	13200	216.5	1062.7	0.204			unlimited
	9873	7.50	34.000	4380	164.0	679.0	0.242			unlimited
	9882	7.50	26.800	6970	196.0	791.7	0.248			unlimited
	9899	7.50	18.700	14200	242.7	1106.2	0.219			unlimited
	9912	7.50	27.700	9360	207.5	895.7	0.232			unlimited
	9928	7.50	28.600	8240	200.5	846.9	0.237			unlimited
	15093	7.50	40.500	6720	175.7	780.8	0.225			unlimited
	15102	7.50	47.500	5260	156.8	717.3	0.219			unlimited
	15119	7.50	49.000	4120	143.6	667.7	0.215			unlimited
	15132	7.50	54.800	5650	153.4	734.3	0.209			unlimited
	15148	7.50	38.800	3800	150.9	653.8	0.231			unlimited
	20008	7.50	58.900	3800	130.7	653.8	0.200			unlimited
	20024	7.50	32.200	4600	168.8	688.6	0.245			unlimited
	20037	7.50	53.800	3700	133.8	649.5	0.206			unlimited
	20053	7.50	47.900	4960	153.6	704.3	0.218			unlimited

Table 21: KENSLAB Modeling and Remaining Fatigue Life Analysis Results

(CTH "A". 18, 26 kip load) (Continued)

Load Levels	Stations	H_{PCC}	Es (ksi)	Epcc (ksi)	Stress under 26kip load	PCC Strength	Stress ratio R	R>0.55	0.45<R<0.55	R<0.45
26 kips	85	7.50	17.200	11700	343.7	997.5	0.345			unlimited
	102	7.50	20.300	12700	338.4	1041.0	0.325			unlimited
	118	7.50	26.300	8360	295.9	852.2	0.347			unlimited
	135	7.50	19.000	10500	330.6	945.3	0.350			unlimited
	148	7.50	14.400	16100	375.2	1188.9	0.316			unlimited
	5023	7.50	21.200	5110	278.2	710.8	0.391			unlimited
	5043	7.50	16.700	11500	344.5	988.8	0.348			unlimited
	5056	7.50	23.600	9400	310.1	897.4	0.346			unlimited
	5076	7.50	20.800	13100	338.8	1058.4	0.320			unlimited
	5092	7.50	31.800	13200	312.7	1062.7	0.294			unlimited
	9873	7.50	34.000	4380	236.8	679.0	0.349			unlimited
	9882	7.50	26.800	6970	283.1	791.7	0.358			unlimited
	9899	7.50	18.700	14200	350.6	1106.2	0.317			unlimited
	9912	7.50	27.700	9360	299.7	895.7	0.335			unlimited
	9928	7.50	28.600	8240	289.6	846.9	0.342			unlimited
	15093	7.50	40.500	6720	253.8	780.8	0.325			unlimited
	15102	7.50	47.500	5260	226.5	717.3	0.316			unlimited
	15119	7.50	49.000	4120	207.4	667.7	0.311			unlimited
	15132	7.50	54.800	5650	221.6	734.3	0.302			unlimited
	15148	7.50	38.800	3800	218.0	653.8	0.333			unlimited
	20008	7.50	58.900	3800	188.7	653.8	0.289			unlimited
	20024	7.50	32.200	4600	243.8	688.6	0.354			unlimited
	20037	7.50	53.800	3700	193.2	649.5	0.297			unlimited
	20053	7.50	47.900	4960	221.9	704.3	0.315			unlimited

Table 22: KENSLAB Modeling and Remaining Fatigue Life Analysis Results

(STH 82. 18, 22, 26 kip load)

Load levels	Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 18kip load	PCC Strength	Stress ratio R	R>0.55	0.45<R<0.55	R<0.45
18 kips	495	5.00	20.00	6080	332.6	753.0	0.442			unlimited
	502	5.00	19.50	1160	172.8	539.0	0.321			unlimited
	509	5.00	23.70	7570	337.2	817.8	0.412			unlimited
	515	5.00	23.10	6140	319.7	755.6	0.423			unlimited
	6569	5.00	16.30	7900	374.2	832.2	0.450			unlimited
	6575	5.00	21.00	2550	241.1	599.4	0.402			unlimited
	6582	5.00	19.40	4170	298.7	669.9	0.446			unlimited
	6588	5.00	22.20	4520	293.3	685.1	0.428			unlimited
	6595	5.00	19.80	3740	285.7	651.2	0.439			unlimited
	9876	5.00	22.90	5900	316.6	745.2	0.425			unlimited
	9882	5.00	23.60	4050	276.0	664.7	0.415			unlimited
	9889	5.00	23.50	5500	307.2	727.8	0.422			unlimited
	9892	5.00	22.10	3070	254.6	622.0	0.409			unlimited
	9899	5.00	21.10	3880	283.0	657.3	0.431			unlimited
	15296	5.00	21.00	3490	272.8	640.3	0.426			unlimited
	15306	5.00	21.30	3900	282.6	658.2	0.429			unlimited
	15319	5.00	24.10	3290	252.9	631.6	0.400			unlimited
	15329	5.00	22.80	3880	275.2	657.3	0.419			unlimited
	15342	5.00	23.70	1670	188.2	561.1	0.335			unlimited
	21176	5.00	14.60	2550	277.8	599.4	0.463		9,748,417	
	21182	5.00	5.30	2200	361.0	584.2	0.618	18,800		
	21192	5.00	19.60	3130	268.7	624.7	0.430			unlimited
	21199	5.00	9.60	4710	375.2	693.4	0.541		161,018	
	21202	5.00	18.60	7330	356.4	807.4	0.441			unlimited
	26425	5.00	26.30	5290	292.0	718.6	0.406			unlimited
	26435	5.00	30.00	5130	275.7	711.7	0.387			unlimited
	26442	5.00	29.20	2910	221.4	615.1	0.360			unlimited
	26451	5.00	30.20	6460	298.2	769.5	0.388			unlimited
	26458	5.00	33.50	3960	238.4	660.8	0.361			unlimited

Table 22: KENSLAB Modeling and Remaining Fatigue Life Analysis Results

(STH 82. 18, 22, 26 kip load) (Continued)

Load levels	Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 22kip load	PCC Strength	Stress ratio R	R>0.55	0.45<R<0.55	R<0.45
22 kips	495	5.00	20.00	6080	406.5	753.0	0.540		167,270	
	502	5.00	19.50	1160	211.2	539.0	0.392			unlimited
	509	5.00	23.70	7570	412.2	817.8	0.504		630,249	
	515	5.00	23.10	6140	390.7	755.6	0.517		364,623	
	6569	5.00	16.30	7900	457.3	832.2	0.550	125,951		
	6575	5.00	21.00	2550	294.6	599.4	0.491		1,184,944	
	6582	5.00	19.40	4170	365.1	669.9	0.545		143,497	
	6588	5.00	22.20	4520	358.4	685.1	0.523		291,028	
	6595	5.00	19.80	3740	349.2	651.2	0.536		187,029	
	9876	5.00	22.90	5900	387.0	745.2	0.519		334,276	
	9882	5.00	23.60	4050	337.4	664.7	0.508		537,330	
	9889	5.00	23.50	5500	375.4	727.8	0.516		382,697	
	9892	5.00	22.10	3070	311.2	622.0	0.500		751,609	
	9899	5.00	21.10	3880	345.9	657.3	0.526		260,372	
	15296	5.00	21.00	3490	333.4	640.3	0.521		318,167	
	15306	5.00	21.30	3900	345.4	658.2	0.525		274,030	
	15319	5.00	24.10	3290	309.0	631.6	0.489		1,345,495	
	15329	5.00	22.80	3880	336.3	657.3	0.512		452,828	
	15342	5.00	23.70	1670	230.1	561.1	0.410			unlimited
	21176	5.00	14.60	2550	339.5	599.4	0.566	78,863		
	21182	5.00	5.30	2200	441.2	584.2	0.755	413		
	21192	5.00	19.60	3130	328.4	624.7	0.526		265,236	
	21199	5.00	9.60	4710	458.6	693.4	0.661	5,615		
	21202	5.00	18.60	7330	435.6	807.4	0.540		168,886	
	26425	5.00	26.30	5290	356.9	718.6	0.497		899,973	
	26435	5.00	30.00	5130	336.9	711.7	0.473		3,916,601	
	26442	5.00	29.20	2910	270.6	615.1	0.440			unlimited
	26451	5.00	30.20	6460	364.5	769.5	0.474		3,831,918	
	26458	5.00	33.50	3960	291.3	660.8	0.441			unlimited

Table 22: KENSLAB Modeling and Remaining Fatigue Life Analysis Results

(STH 82. 18, 22, 26 kip load) (Continued)

Load Levels	Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 26kip load	PCC Strength	Stress ratio R	R>0.55	0.45<R<0.55	R<0.45
26 kips	495	5.00	20.00	6080	480.4	753.0	0.638	10,762		
	502	5.00	19.50	1160	249.6	539.0	0.463		10,096,027	
	509	5.00	23.70	7570	487.1	817.8	0.596	34,964		
	515	5.00	23.10	6140	461.7	755.6	0.611	22,772		
	6569	5.00	16.30	7900	540.5	832.2	0.650	7,811		
	6575	5.00	21.00	2550	348.2	599.4	0.581	52,673		
	6582	5.00	19.40	4170	431.4	669.9	0.644	9,112		
	6588	5.00	22.20	4520	423.6	685.1	0.618	18,619		
	6595	5.00	19.80	3740	412.7	651.2	0.634	12,107		
	9876	5.00	22.90	5900	457.3	745.2	0.614	21,151		
	9882	5.00	23.60	4050	398.7	664.7	0.600	31,096		
	9889	5.00	23.50	5500	443.7	727.8	0.610	23,649		
	9892	5.00	22.10	3070	367.8	622.0	0.591	39,461		
	9899	5.00	21.10	3880	408.8	657.3	0.622	16,812		
	15296	5.00	21.00	3490	394.0	640.3	0.615	20,219		
	15306	5.00	21.30	3900	408.2	658.2	0.620	17,643		
	15319	5.00	24.10	3290	365.2	631.6	0.578	56,764		
	15329	5.00	22.80	3880	397.5	657.3	0.605	27,118		
	15342	5.00	23.70	1670	271.9	561.1	0.485		1,782,451	
	21176	5.00	14.60	2550	401.2	599.4	0.669	4,506		
	21182	5.00	5.30	2200	521.4	584.2	0.893	9		
	21192	5.00	19.60	3130	388.2	624.7	0.621	17,045		
	21199	5.00	9.60	4710	541.9	693.4	0.782	199		
	21202	5.00	18.60	7330	514.8	807.4	0.638	10,870		
	26425	5.00	26.30	5290	421.8	718.6	0.587	44,489		
	26435	5.00	30.00	5130	398.2	711.7	0.560	95,373		
	26442	5.00	29.20	2910	319.8	615.1	0.520		327,211	
	26451	5.00	30.20	6460	430.8	769.5	0.560	94,591		
	26458	5.00	33.50	3960	344.3	660.8	0.521		313,664	

Table 23: KENSLAB Modeling of Maximum Thermal Tensile Stress (Lawndale Ave)

Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 3 ^{0}F/in. temp. gradient
0	4.00	23.749	483	17.7
7	4.00	18.894	8350	235.6
13	4.00	21.738	3780	130.7
16	4.00	26.248	1570	57.5
43	4.00	26.696	1130	41.2
49	4.00	31.380	650	23.8
56	4.00	30.412	565	21.0
62	4.00	30.033	492	18.6

Table 24: KENSLAB Modeling of Maximum Thermal Tensile Stress (CTH "A")

Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 3 ^{0}F/in. temp. gradient
85	7.50	17.200	11700	691.1
102	7.50	20.300	12700	764.2
118	7.50	26.300	8360	556.8
135	7.50	19.000	10500	647.7
148	7.50	14.400	16100	825.5
5023	7.50	21.200	5110	347.2
5043	7.50	16.700	11500	677.3
5056	7.50	23.600	9400	610.7
5076	7.50	20.800	13100	787.1
5092	7.50	31.800	13200	852.8
9873	7.50	34.000	4380	301.3
9882	7.50	26.800	6970	471.5
9899	7.50	18.700	14200	816.8
9912	7.50	27.700	9360	619.7
9928	7.50	28.600	8240	553.4
15093	7.50	40.500	6720	462.1
15102	7.50	47.500	5260	361.3
15119	7.50	49.000	4120	281.3
15132	7.50	54.800	5650	387.6
15148	7.50	38.800	3800	260.4
20008	7.50	58.900	3800	258.0
20024	7.50	32.200	4600	316.6
20037	7.50	53.800	3700	251.5
20053	7.50	47.900	4960	340.3

Table 25: KENSLAB Modeling of Maximum Thermal Tensile Stress (STH 82)

Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 3 ^{0}F/in. temp. gradient
495	5.00	20	6080	155.0
502	5.00	19.5	1160	49.5
509	5.00	23.7	7570	188.4
515	5.00	23.1	6140	166.6
6569	5.00	16.3	7900	156.0
6575	5.00	21	2550	92.7
6582	5.00	19.4	4170	123.5
6588	5.00	22.2	4520	136.6
6595	5.00	19.8	3740	116.2
9876	5.00	22.9	5900	162.3
9882	5.00	23.6	4050	130.7
9889	5.00	23.5	5500	157.5
9892	5.00	22.1	3070	106.9
9899	5.00	21.1	3880	121.9
15296	5.00	21	3490	114.0
15306	5.00	21.3	3900	122.7
15319	5.00	24.1	3290	115.3
15329	5.00	22.8	3880	125.6
15342	5.00	23.7	1670	69.2
21176	5.00	14.6	2550	81.7
21182	5.00	5.3	2200	47.6
21192	5.00	19.6	3130	103.8
21199	5.00	9.6	4710	92.3
21202	5.00	18.6	7330	163.3
26425	5.00	26.3	5290	160.6
26435	5.00	30	5130	165.8
26442	5.00	29.2	2910	111.7
26451	5.00	30.2	6460	191.6
26458	5.00	33.5	3960	145.1

5.2 ANALYSIS BASED ON PAVEMENT DISTRESS SURVEY

5.2.1 Performance Evaluation and Analysis

5.2.1.1 Performance evaluation

Whitetopping pavement performance was analyzed based on PCI and PDI in this study. The field distress survey data was processed using MicroPAVER 5.2 for PCI and the method provided in the "Pavement Surface Distresses Survey Manual" (Wisconsin DOT 1993) for PDI. The PCI and PDI of in-service whitetopping pavements are shown in Table 26, except for Howard Avenue which could not be accessed.

Table 26: Pavement Performance—ASTM PCI and WisDOT PDI

No	Project				ASTM PCI	WisDOT PDI
	County	Road Name	Year	Age		
13	Dodge	CTH A	2007	1	89	4.65
12	Waukesha	Duplainville Rd	1999	9	85	6.7
2	Milwaukee	Fond Du Lac Ave	2001	7	58	64.4
1	Milwaukee	Galena ST	1995	13	55	65.76
6	Kenosha	IH94/STH 50 Ramp	1998	10	72	41.73
8	Washington	Lawndale Ave	1998	10	76	32.11
9	Kenosha	North 39th Avenue	1999	9	78	13.1
15	Milwaukee	State Street	2000	8	94	7.76
4	Dodge	STH 33 and CTH "A"	2001	7	69	34.1
5	Kenosha	STH 50	2001	7	71	27.57
7	Portage	STH 54	2001	7	74	26.63
14	Oxford	STH 82	2001	7	91	7.37
10	Taylor	STH 97	1999	9	81	6.73
11	Douglas	USH 2/ USH 53	2001	7	82	32.4
3	Kenosha	Washington and 22nd	2001	7	64	25.66

Galena Street and Fond Du Lac Avenue appear to be in the worst condition. Duplainville Road, CTH “A”, STH 82, and State Street show good performance. This agrees with the fatigue life analysis for CTH “A” and STH 82.

A good correlation exists between PCI and PDI, as expected. Figure 45 shows the relationship between PCI and PDI. The regression equation is as follows:

$$(\mathrm{PDI}) = -1.496(\mathrm{PCI}) + 140.04 \qquad (26)$$

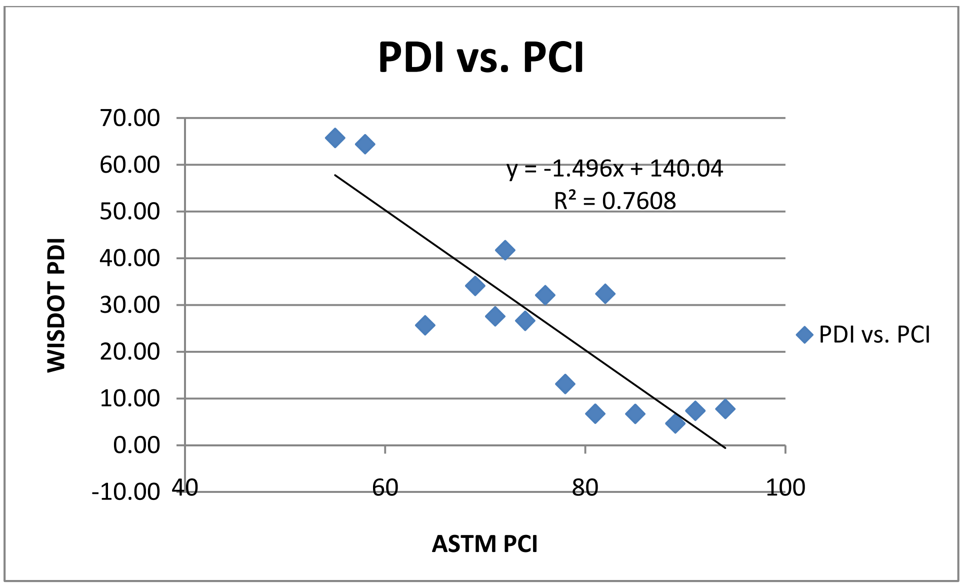

Figure 45: Linear Relationship between ASTM PCI and WisDOT PDI

It should be noted that many whitetopping pavements are short or are located at intersections. It was found that the transition areas or ends of whitetopping pavements are typically in severely deteriorated condition, likely due to the impact by vehicles, as compared to the rest of pavement. This was also reported in other studies (Wu 2007). Therefore, when determining the PCI or PDI, the short whitetopping pavements were at a disadvantage when compared to longer projects. Figure 46 shows the transition areas of the IH94/STH50 Ramp. It is suggested that thicker slabs be used in these areas.

Figure 46: Localized Severe Distress at the Entrance (left) and Exit (right) End of IH94/STH50 Ramp

5.2.1.2 Relation between performance and cumulative ESALs

For projects having cumulative ESALs, Figure 47 shows each project's PCI/PDI and the ESALs experienced. Because each project had different design ESALs, the PCI/PDI appears no correlation with cumulative ESALs. However, if we define the "relative age" as the cumulative ESALs divided by design ESALs (cumulative ESALs/ Design ESALs), an increase trend of PCI with the decrease of "relative age" can be seen in Figure 48. It means a pavement having lower "relative age" tends to have better performance. A reasonable correlation between PCI and "relative age" can be seen in Figure 49 with R^2 =0.5052. It indicates that the development of pavement's deterioration reflects the "relative age" reasonably. When "relative age" reaches about 80%, PCI could be reduced to 60 to 65, which is a bad performance condition. It shows the rate of development of deterioration is a little bit more quickly than the design expected. In other words, these whitetopping pavements were somewhat under designed.

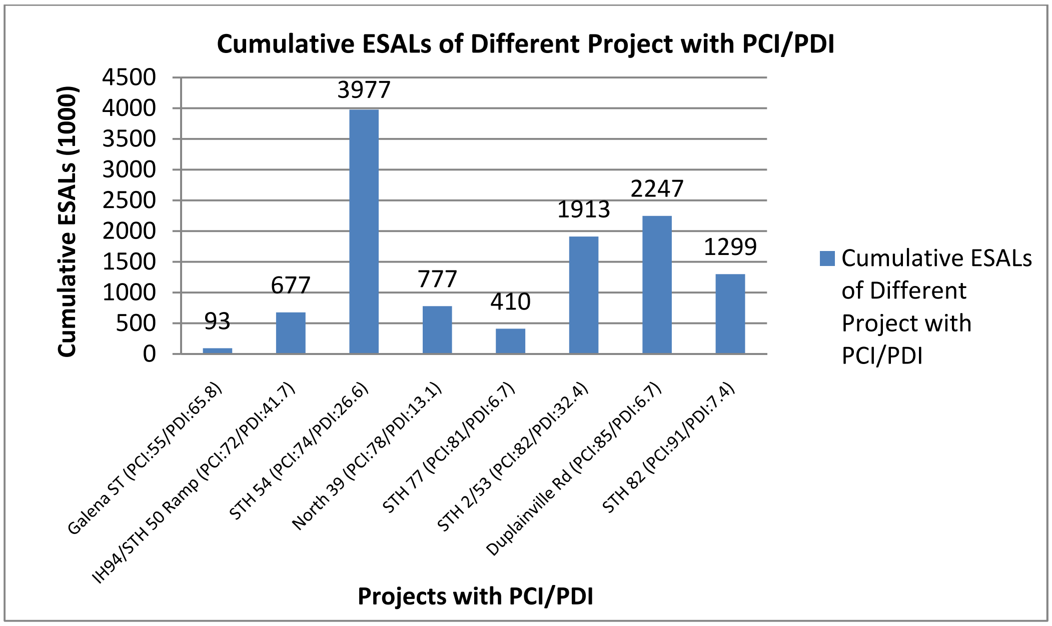

Figure 47: Cumulative ESALs of Different Projects

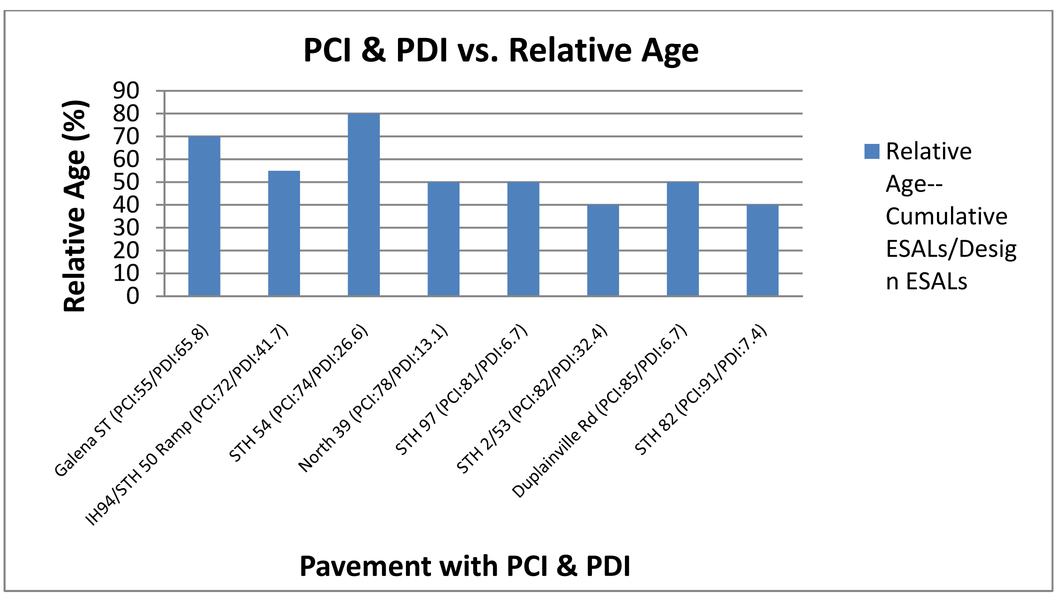

Figure 48: Increase Trend of PCI with Decrease of Relative Age

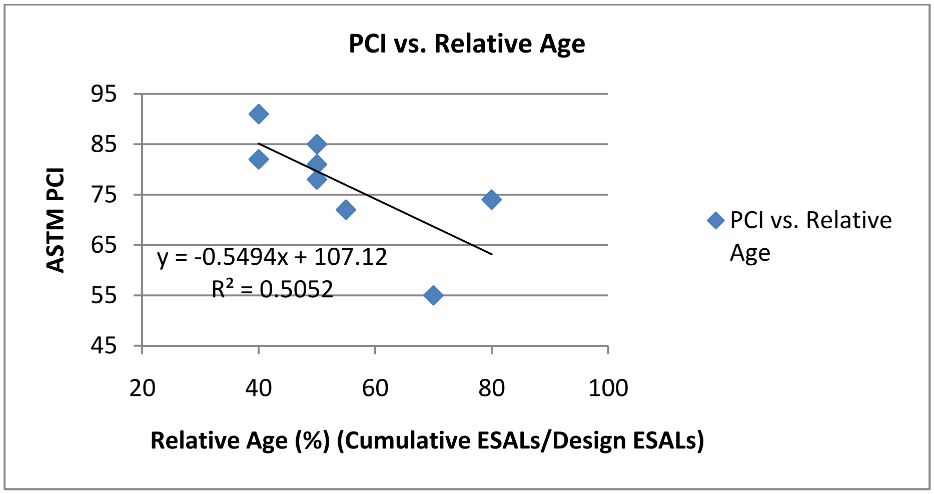

Figure 49: Relative between PCI and Pavement Age

5.2.1.3 Relation between performance and PCC modulus

The relationship between the pavement performance and FWD backcalculated layer properties was also explored. In this study, only 3 projects had backcalculated moduli. Figure 50 shows that the PDI decreases with the increase of the PCC modulus. The backcalculated PCC modulus could be an indicator of the pavement performance. However, there are too few data to make a conclusion.

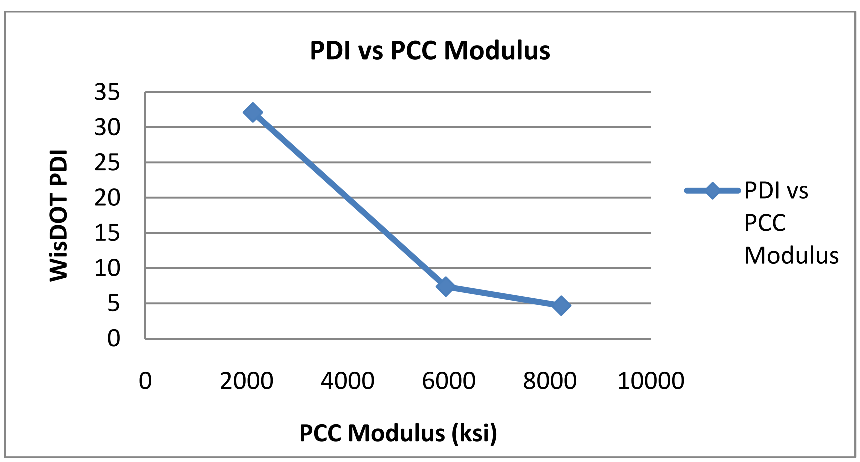

Figure 50: Relationship between WisDOT PDI and PCC Modulus

5.2.1.4 Development of whitetopping pavement performance

In order to study the development of whitetopping pavement performance, the pre-overlay and post-overlay performances were collected. The pre-overlay condition and historic performances are obtained in Pavement Information Files (PIF), if these pavements are located in the STH or IH system. The historic performance evaluation is recorded in the format of PDI. Unfortunately most of the whitetopping projects are local roads or too short which are not included in the PIF database. The historic performance information is available only for STH 54 and STH 82, as shown in Table 27. Prior to the whitetopping, STH 82 was resurfaced with HMA in 1988. STH 54 was repaired in 1993 which can be shown by the reduction of PDI. Both STH 54 and STH 82 were overlaid with concrete overlay in 2001. It can be seen from Table 27 that the pavement condition of STH 82 (PDI = 51.58), was better than that of STH 54 (PDI =75.50) before whitetopping. Because STH 82 and STH 54 whitetopping overlay has 20 and 10 years design life respectively, if a same pavement condition was expected at the end of the pavement's life, the design PDI progression rate (change of PDI divided by design life) of STH 54 would be 2 times of that of STH 82. However, the PDI progression rate of STH 54, 3.8 per year, is 3.6 times of that of STH 82, 1.05 per year. This proved that the pre-overlay HMA condition had effects on the performance of whitetopping, based on the assumption that both pavements were correctly designed. It is interesting to note that the HMA overlay on STH 82, prior to the whitetopping, lasted 12 years before the rehabilitation was needed. After eight years in service, STH 82 whitetopping pavement is still in excellent condition with a PDI of only 7.37. The life of the whitetopping pavement at STH 82 is expected to be fairly long. However, this observation needs to be verified with the design information for both the HMA overlay and whitetopping pavement.

Table 27: Historic Pavement Performance (PDI) of STH 82 and STH 54

STH 82	**Year**	**1988**	**1990**	**1992**	**1994**	**1996**	**1998**	**2000**	**2001**	**2002**	**2004**	**2008***
	PDI	0.00	9.50	15.58	15.33	27.50	41.67	51.58	0.00	1.75	3.83	7.37
STH 54	**Year**	**1989**	**1991**	**1993**	**1995**	**1997**	**1999**	**2000**	**2001**	**2002**	**2004**	**2008***
	PDI	83.00	70.00	24.50	40.50	80.00	70.00	75.50	0.00	3.00	6.00	26.63

*Field Survey by The Team

5.2.2 Statistical Analysis and Results

Based on the data collected, statistical analysis was conducted to identify the design and construction factors that affect the performance of these whitetopping pavements in Wisconsin. The performance of whitetopping pavements, PDI or PCI, was used as the dependent variable. Independent variables included slab thickness, slab size, ESAL, HMA thickness, age, and use of fibers. However, none of these variables was found to be statistically significant.

It was decided to categorize the pavements, based on slab thickness and slab size, two essential parameters of whitetopping pavements. The pavements were categorized as slab thickness either ≤ 4 in. or > 4 in., and slab size either ≤36 sq. ft. or >36 sq. ft.

It was found that when the PCI was used as a dependent variable, the slab thickness and the slab size were statistically significant variables. Pavement age has a significance level of 0.051 which is very close to being considered statistically significant. However, when the PDI was used as the dependent variable, only slab thickness was statistically significant. Tables 28 and 29 show the results of the statistical analysis, based on the PCI and the PDI, respectively.

Table 28: Statistical Test Results of the effects on PCI

Tests of Between-Subjects Effects

Dependent Variable: Performance PCI

Source		Type III Sum of Squares	df	Mean Square	F	Sig.	Partial Eta Squared
Intercept	Hypothesis	41152.368	1	41152.368	563.060	.000	.991
	Error	356.329	4.875	73.087[a]			
PCCthick	Hypothesis	989.681	1	989.681	42.425	**.000**	.858
	Error	163.295	7	23.328[b]			
Age	Hypothesis	379.505	4	94.876	4.067	**.051**	.699
	Error	163.295	7	23.328[b]			
Slabsize	Hypothesis	281.104	1	281.104	12.050	**.010**	.633
	Error	163.295	7	23.328[b]			

a. .695 MS(inserv) + .305 MS(Error)

b. MS(Error)

Table 29: Statistical Test Results of the effects on PDI

Tests of Between-Subjects Effects

Dependent Variable: Performance PDI

Source		Type III Sum of Squares	df	Mean Square	F	Sig.	Partial Eta Squared
Intercept	Hypothesis	4065.174	1	4065.174	12.119	.013	.667
	Error	2024.986	6.037	335.425[a]			
PCCthick	Hypothesis	1488.740	1	1488.740	6.751	**.036**	.491
	Error	1543.596	7	220.514[b]			
Age	Hypothesis	1542.973	4	385.743	1.749	.243	.500
	Error	1543.596	7	220.514[b]			
Slabsize	Hypothesis	522.562	1	522.562	2.370	.168	.253
	Error	1543.596	7	220.514[b]			

a. .695 MS(inserv) + .305 MS(Error)

b. MS(Error)

5.3 ANALYSIS BASED ON BOND STRENGTH

Five cores were obtained for each of 5 projects. Most of the cores had separated PCC and HMA and could not be tested for bond strength. All of cores were separated for Fond Du Lac Ave, probably due to the severely deteriorated slabs. Iowa shear strength tests were performed on cores in which PCC and HMA were not separated. The test results of 4 projects are shown in Table 30 and Figure 51.

Table 30: Iowa Shear Strength Test Results

No.	**Project**	**Pavement Age (year)**	**Pre-overlay Preparation**	**Specimen No.**	**Iowa Shear Strength (psi)**	**Average Shear Strength (psi)**
1	Lawndale Ave	10	cleaning	1-3	266.0	266.0
2	North 39 Ave	9	n/a	1-5	177.3	177.3
3	Country Highway A	1	2” milling	4-3	123.3	154.1
				3-3	174.8	
				5-3	164.1	
4	STH 82	7	0.5" milling	1-2	124.6	124.6
	Average					171.7

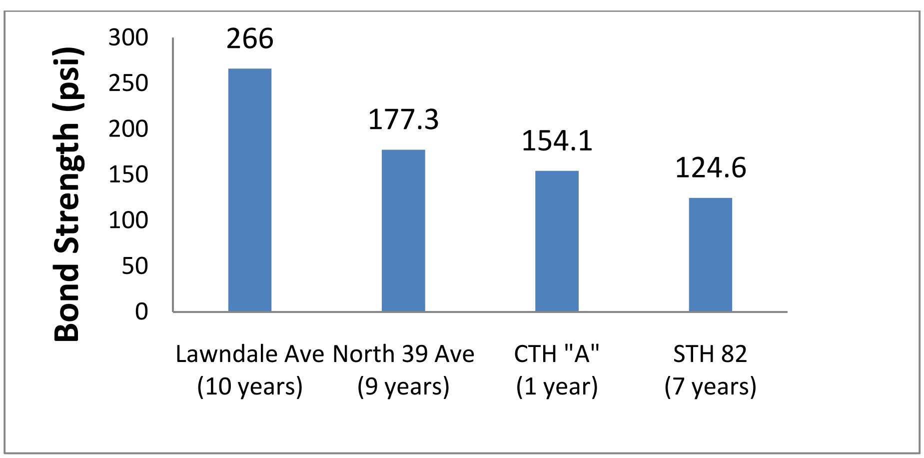

Figure 51: Bond Strength of Four Whitetopping Pavements

From Table 30 and Figure 51, it can be seen that the shear strength ranges from 124 psi to 266 psi. A shear strength of 200 psi was reported to be sufficient to withstand the shearing force by vehicles (Tawfiq 2001). As discussed previously, concrete and HMA were separated in most of cores during coring. Only the cores that did not separate were tested in the laboratory. Because the bond strength of a separated core should be lower than the integrated one, the average bond strength should be lower than the test results. At the time of the distress surveys, CTH "A" had the best performance, followed by STH 82, North 39th Ave and Lawdale Ave (refer to Table 26). It seems that there is no correlation between the performance and the bond strength, based on the limited data. However, for CTH "A", three sound cores could be obtained, while the other three projects had only one core that was un-separated. This is probably due to the fact that CTH "A" was only in service for one year and the bond has not been broken yet in most cases.

It seems that most of the whitetopping pavements lost the bond between PCC and HMA. There does not seem to be a correlation between pre-overlay treatment and PCC/HMA bond strength. The result can't prove milling has more advantage. The data is limited to make a conclusive finding. However, it is suggested that the design of whitetopping pavements should be based on an unbonded condition, to be safe.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

Based on the literature review, field assessment, and analysis of the performance of the whitetopping pavements, the following conclusions and recommendations can be made.

6.1 CONCLUSIONS

(1) Traditional backcalculation methods of concrete pavement layer properties, based on FWD testing, are not applicable to the UTW pavements. The new Critical Distance Method shows potential to be used in UTW pavement FWD test backcalculation. The parameter of "relative slab stiffness" could be used to estimate the backcalculation error of PCC modulus and as a criterion to choose suitable backcalculation method.

(2) The backcalculated PCC modulus correlates with the pavement performance reasonably well, and the backcalculated substructure modulus reflects the structural capacity of the substructure.

(3) Critical loading position depends on the pavement structure and slab layout. Thermal stress has little effect on typical UTW overlay due to the relatively short joint spacing and thin slab thickness. However, if the joint spacing increased, such as the case of CTH "A", using 15 ft. by 15 ft., thermal stress could have a significant effect and could become a major cause of fatigue.

(4) Whitetopping pavement is very sensitive to a load level higher than the 18-kip standard axle loads. Slightly increasing the axle load could significantly decrease the

fatigue lives of whitetopping pavements. The design method should not be based on the 18-kip standard axle loads. Instead, higher load levels or a load spectrum should be used.

(5) Slab thickness, slab size, and pavement age were found to be statistically significant variables that affect the performance of whitetopping pavements. Slab thickness should be thicker than 4 in. and slab size should be smaller than 36 sq. ft. to get better whitetopping pavement performance.

(6) For most of the whitetopping pavement cores, the concrete and HMA were separated. This indicates that the bond was lost quickly in the field. The design method of whitetopping should fully consider the real bond condition, or based on unbond condition, to be conservative.

(7) The whitetopping pavements show great potential to be a viable rehabilitation method. However, they also show mixed performance. The design method needs to be improved.

6.2 RECOMMENDATIONS

(1) The FWD backcalculation method for whitetopping pavements needs to be further developed and validated.

(2) In this study, the models in MEPDG were used to estimate the aggregate interlock joint stiffness for UTW. The applicability of this models when used for UTW need further study.

(3) It is recommended that the MEPDG could be calibrated to incorporate the analysis of UTW (when slab thickness is below 6 in.)

(4) In MEPDG, the LTE due to base/subbase for PCC overlay on HMA pavement (whitetopping) is set as a constant of 30%. Using this value, the total LTE is very low when slab thickness is 6 in. This is different significantly from the in-situ test value of about 80% to 90% even the slab thickness is below 6 in. The LTE due to base/subbase for whitetopping especially UTW needs to be calibrated by field test.

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APPENDIX A: THE DEFLECTION AND THE DEFLECTION DIFFERENCE ON SURFACE OF HMA AND PCC

Table A.1: Deflections and Deflection Difference

Joint stiffness=0 psi, E (substructure)=20ksi, PCC thickness=3 in.

Sub-structure under WT slab with equivalent modulus: E=20ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01512	-62.11475821	0.01488	-62.7161113	0.01418	-64.4700576
12	0.01025	0.0106	3.414634146	0.01046	2.048780488	0.01008	-1.65853659
24	0.00499	Joint		0.00566	13.42685371	0.00575	15.23046092
36	0.00331	0.00346	4.531722054	0.00336	1.510574018	Joint	
48	0.00245	0.00254	3.673469388	0.00253	3.265306122	0.00254	3.673469388
60	0.00196	0.00199	1.530612245	0.00199	1.530612245	0.00201	2.551020408
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01339	-66.4495114	0.01316	-67.0258081	0.01254	-68.5793034
12	0.01025	0.01005	-1.951219512	0.00988	-3.6097561	0.00951	-7.2195122
24	0.00499	Joint		0.00581	16.43286573	0.00585	17.23446894
36	0.00331	0.00349	5.438066465	0.00338	2.114803625	Joint	
48	0.00245	0.00258	5.306122449	0.00257	4.897959184	0.00257	4.897959184
60	0.00196	0.00201	2.551020408	0.00202	3.06122449	0.00202	3.06122449
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01232	-69.13054372	0.0121	-69.681784	0.01154	-71.0849411
12	0.01025	0.00962	-6.146341463	0.00944	-7.90243902	0.00908	-11.4146341
24	0.00499	Joint		0.00587	17.63527054	0.00587	17.63527054
36	0.00331	0.0035	5.740181269	0.00339	2.416918429	Joint	
48	0.00245	0.00261	6.530612245	0.0026	6.12244898	0.00258	5.306122449
60	0.00196	0.00202	3.06122449	0.00204	4.081632653	0.00205	4.591836735

Table A.2: Deflections and Deflection Difference

Joint stiffness=0 psi, E (substructure)=20ksi, PCC thickness=4 in.

Equivalent sub-structure modulus: E=20ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01227	-69.25582561	0.01205	-69.8070659	0.0115	-71.1851666
12	0.01025	0.0096	-6.341463415	0.00941	-8.19512195	0.00906	-11.6097561
24	0.00499	Joint		0.00587	17.63527054	0.00587	17.63527054
36	0.00331	0.0035	5.740181269	0.00339	2.416918429	Joint	
48	0.00245	0.00261	6.530612245	0.0026	6.12244898	0.00258	5.306122449
60	0.00196	0.00202	3.06122449	0.00204	4.081632653	0.00205	4.591836735
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01079	-72.96416938	0.01056	-73.540466	0.01011	-74.668003
12	0.01025	0.00891	-13.07317073	0.00867	-15.4146341	0.00836	-18.4390244
24	0.00499	Joint		0.00588	17.83567134	0.00582	16.63326653
36	0.00331	0.00351	6.042296073	0.00338	2.114803625	Joint	
48	0.00245	0.00265	8.163265306	0.00263	7.346938776	0.00259	5.714285714
60	0.00196	0.00204	4.081632653	0.00207	5.612244898	0.00208	6.12244898
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.00992	-75.14407417	0.00966	-75.79554	0.00927	-76.7727387
12	0.01025	0.00846	-17.46341463	0.00816	-20.3902439	0.00788	-23.1219512
24	0.00499	Joint		0.00584	17.03406814	0.00574	15.03006012
36	0.00331	0.0035	5.740181269	0.00336	1.510574018	Joint	
48	0.00245	0.00267	8.979591837	0.00265	8.163265306	0.00259	5.714285714
60	0.00196	0.00205	4.591836735	0.0021	7.142857143	0.0021	7.142857143

Table A.3: Deflections and Deflection Difference

Joint stiffness=0 psi, E (substructure)=20ksi, PCC thickness=5 in.

Sub-structure under WT slab with equivalent modulus: E=20ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01037	-74.01653721	0.01013	-74.6178903	0.0097	-75.6953145
12	0.01025	0.0087	-15.12195122	0.00843	-17.7560976	0.00813	-20.6829268
24	0.00499	Joint		0.00587	17.63527054	0.00579	16.03206413
36	0.00331	0.00351	6.042296073	0.00337	1.812688822	Joint	
48	0.00245	0.00266	8.571428571	0.00264	7.755102041	0.00259	5.714285714
60	0.00196	0.00205	4.591836735	0.00209	6.632653061	0.00209	6.632653061
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.00915	-77.07341518	0.00884	-77.8501629	0.00849	-78.7271361
12	0.01025	0.00804	-21.56097561	0.00767	-25.1707317	0.00739	-27.902439
24	0.00499	Joint		0.00577	15.63126253	0.00561	12.4248497
36	0.00331	0.0035	5.740181269	0.00334	0.906344411	Joint	
48	0.00245	0.00269	9.795918367	0.00266	8.571428571	0.00258	5.306122449
60	0.00196	0.00206	5.102040816	0.00212	8.163265306	0.00211	7.653061224
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.00849	-78.72713606	0.00809	-79.7293911	0.00777	-80.5311952
12	0.01025	0.00767	-25.17073171	0.00719	-29.8536585	0.00691	-32.5853659
24	0.00499	Joint		0.00568	13.82765531	0.00546	9.418837675
36	0.00331	0.0035	5.740181269	0.00331	0	Joint	
48	0.00245	0.00271	10.6122449	0.00267	8.979591837	0.00257	4.897959184
60	0.00196	0.00207	5.612244898	0.00214	9.183673469	0.00212	8.163265306

Table A.4: Deflections and Deflection Difference

Joint stiffness=0 psi, E (substructure)=50ksi, PCC thickness=3 in.

Sub-structure under WT slab with equivalent modulus: E=50ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00739	-53.69674185	0.00727	-54.4486216	0.00692	-56.641604
12	0.0041	0.00449	9.512195122	0.00444	8.292682927	0.00432	5.365853659
24	0.002	Joint		0.00211	5.5	0.00218	9
36	0.00132	0.00134	1.515151515	0.00131	-0.75757576	Joint	
48	0.00098	0.00099	1.020408163	0.00098	0	0.001	2.040816327
60	0.00079	0.00079	0	0.00077	-2.53164557	0.00079	0
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00662	-58.52130326	0.00652	-59.1478697	0.00622	-61.0275689
12	0.0041	0.00437	6.585365854	0.00432	5.365853659	0.00418	1.951219512
24	0.002	Joint		0.00219	9.5	0.00225	12.5
36	0.00132	0.00136	3.03030303	0.00132	0	Joint	
48	0.00098	0.001	2.040816327	0.00099	1.020408163	0.001	2.040816327
60	0.00079	0.00079	0	0.00078	-1.26582278	0.00079	0
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00614	-61.52882206	0.00604	-62.1553885	0.00576	-63.9097744
12	0.0041	0.00426	3.902439024	0.0042	2.43902439	0.00406	-0.97560976
24	0.002	Joint		0.00224	12	0.00229	14.5
36	0.00132	0.00138	4.545454545	0.00133	0.757575758	Joint	
48	0.00098	0.00101	3.06122449	0.001	2.040816327	0.00101	3.06122449
60	0.00079	0.00079	0	0.00078	-1.26582278	0.0008	1.265822785

Table A.5: Deflections and Deflection Difference

Joint stiffness=0 psi, E (substructure)=50ksi, PCC thickness=4 in.

Equivalent sub-structure modulus: E=50ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00612	-61.65	0.00601	-62.34	0.00574	-64.04
12	0.0041	0.00426	3.90	0.00419	2.20	0.00405	-1.22
24	0.002	Joint		0.00224	12.00	0.00229	14.50
36	0.00132	0.00138	4.55	0.00133	0.76	Joint	
48	0.00098	0.00101	3.06	0.001	2.04	0.00101	3.06
60	0.00079	0.00079	0.00	0.00078	-1.27	0.0008	1.27
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00542	-66.04	0.00533	-66.60	0.00508	-68.17
12	0.0041	0.00404	-1.46	0.00397	-3.17	0.00383	-6.59
24	0.002	Joint		0.00231	15.50	0.00233	16.50
36	0.00132	0.00139	5.30	0.00135	2.27	Joint	
48	0.00098	0.00103	5.10	0.00102	4.08	0.00102	4.08
60	0.00079	0.0008	1.27	0.0008	1.27	0.00081	2.53
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00499	-68.73	0.0049	-69.30	0.00468	-70.68
12	0.0041	0.00387	-5.61	0.0038	-7.32	0.00366	-10.73
24	0.002	Joint		0.00234	17.00	0.00234	17.00
36	0.00132	0.00139	5.30	0.00135	2.27	Joint	
48	0.00098	0.00104	6.12	0.00103	5.10	0.00103	5.10
60	0.00079	0.0008	1.27	0.00081	2.53	0.00081	2.53

Table A.6: Deflections and Deflection Difference

Joint stiffness=0 psi, E(substructure)=50ksi, PCC thickness=5 in.

Sub-structure under WT slab with equivalent modulus: E=50ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00522	-67.29323308	0.00512	-67.9197995	0.00489	-69.3609023
12	0.0041	0.00396	-3.414634146	0.00389	-5.12195122	0.00375	-8.53658537
24	0.002	Joint		0.00232	16	0.00234	17
36	0.00132	0.00139	5.303030303	0.00135	2.272727273	Joint	
48	0.00098	0.00103	5.102040816	0.00102	4.081632653	0.00102	4.081632653
60	0.00079	0.0008	1.265822785	0.0008	1.265822785	0.00081	2.53164557
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00459	-71.2406015	0.0045	-71.8045113	0.0043	-73.0576441
12	0.0041	0.00369	-10	0.00361	-11.9512195	0.00348	-15.1219512
24	0.002	Joint		0.00235	17.5	0.00234	17
36	0.00132	0.0014	6.060606061	0.00135	2.272727273	Joint	
48	0.00098	0.00105	7.142857143	0.00104	6.12244898	0.00103	5.102040816
60	0.00079	0.00081	2.53164557	0.00082	3.797468354	0.00082	3.797468354
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00421	-73.62155388	0.00412	-74.1854637	0.00395	-75.2506266
12	0.0041	0.00351	-14.3902439	0.00341	-16.8292683	0.00329	-19.7560976
24	0.002	Joint		0.00234	17	0.00232	16
36	0.00132	0.0014	6.060606061	0.00134	1.515151515	Joint	
48	0.00098	0.00106	8.163265306	0.00105	7.142857143	0.00103	5.102040816
60	0.00079	0.00081	2.53164557	0.00083	5.063291139	0.00083	5.063291139

Table A.7: Deflections and Deflection Difference

Joint stiffness=40000 psi, E (substructure)=20ksi, PCC thickness=3 in.

Sub-structure under WT slab with equivalent modulus: E=20ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01501	-62.390378	0.0148	-62.916562	0.01412	-64.620396
12	0.01025	0.01044	1.8536585	0.01037	1.1707317	0.01002	-2.2439024
24	0.00499			0.00554	11.022044	0.00567	13.627255
36	0.00331	0.00345	4.2296073	0.00329	-0.6042296		
48	0.00245	0.0025	2.0408163	0.00247	0.8163265	0.00247	0.8163265
60	0.00196	0.00195	-0.5102041	0.00194	-1.0204082	0.00195	-0.5102041

Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01325	-66.800301	0.01307	-67.251315	0.01248	-68.729642
12	0.01025	0.00982	-4.195122	0.00977	-4.6829268	0.00943	-8
24	0.00499			0.00564	13.026052	0.00575	15.230461
36	0.00331	0.00355	7.2507553	0.00337	1.8126888		
48	0.00245	0.00257	4.8979592	0.00253	3.2653061	0.00251	2.4489796
60	0.00196	0.00198	1.0204082	0.00198	1.0204082	0.00198	1.0204082

Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01215	-69.556502	0.01199	-69.957404	0.01148	-71.235279
12	0.01025	0.00935	-8.7804878	0.0093	-9.2682927	0.009	-12.195122
24	0.00499			0.00565	13.226453	0.00575	15.230461
36	0.00331	0.0036	8.7613293	0.00343	3.6253776		
48	0.00245	0.00261	6.5306122	0.00258	5.3061224	0.00255	4.0816327
60	0.00196	0.002	2.0408163	0.00201	2.5510204	0.00201	2.5510204

Table A.8: Deflections and Deflection Difference

Joint stiffness=40000 psi, E (substructure)=20ksi, PCC thickness=4 in.

Sub-structure under WT slab with equivalent modulus: E=20ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.0121	-69.681784	0.01194	-70.082686	0.01143	-71.360561
12	0.01025	0.00932	-9.0731707	0.00927	-9.5609756	0.00897	-12.487805
24	0.00499			0.00565	13.226453	0.00575	15.230461
36	0.00331	0.0036	8.7613293	0.00343	3.6253776		
48	0.00245	0.00262	6.9387755	0.00258	5.3061224	0.00255	4.0816327
60	0.00196	0.002	2.0408163	0.00201	2.5510204	0.00201	2.5510204
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01057	-73.51541	0.01043	-73.866199	0.01002	-74.89351
12	0.01025	0.00855	-16.585366	0.00849	-17.170732	0.00824	-19.609756
24	0.00499			0.0056	12.224449	0.00565	13.226453
36	0.00331	0.00366	10.574018	0.00351	6.0422961		
48	0.00245	0.00268	9.3877551	0.00266	8.5714286	0.00261	6.5306122
60	0.00196	0.00203	3.5714286	0.00207	5.6122449	0.00206	5.1020408
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.00967	-75.770484	0.0095	-76.196442	0.00916	-77.048359
12	0.01025	0.00805	-21.463415	0.00794	-22.536585	0.00773	-24.585366
24	0.00499			0.00551	10.420842	0.00553	10.821643
36	0.00331	0.0037	11.782477	0.00354	6.9486405		
48	0.00245	0.00273	11.428571	0.0027	10.204082	0.00265	8.1632653
60	0.00196	0.00205	4.5918367	0.0021	7.1428571	0.0021	7.1428571

Table A.9: Deflections and Deflection Difference

Joint stiffness=40000 psi, E (substructure)=20ksi, PCC thickness=5 in.

Sub-structure under WT slab with equivalent modulus: E=20ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.01013	-74.61789	0.00999	-74.96868	0.00961	-75.920822
12	0.01025	0.00831	-18.926829	0.00823	-19.707317	0.00801	-21.853659
24	0.00499			0.00556	11.422846	0.0056	12.224449
36	0.00331	0.00368	11.178248	0.00353	6.6465257		
48	0.00245	0.0027	10.204082	0.00268	9.3877551	0.00263	7.3469388
60	0.00196	0.00203	3.5714286	0.00208	6.122449	0.00208	6.122449
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.00886	-77.80005	0.00866	-78.301178	0.00836	-79.052869
12	0.01025	0.00758	-26.04878	0.00741	-27.707317	0.00722	-29.560976
24	0.00499			0.00539	8.0160321	0.00536	7.4148297
36	0.00331	0.00372	12.386707	0.00355	7.2507553		
48	0.00245	0.00276	12.653061	0.00274	11.836735	0.00267	8.9795918
60	0.00196	0.00206	5.1020408	0.00213	8.6734694	0.00213	8.6734694
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.03991	0.00816	-79.553996	0.00788	-80.255575	0.00763	-80.881984
12	0.01025	0.00714	-30.341463	0.00689	-32.780488	0.00672	-34.439024
24	0.00499			0.00525	5.2104208	0.00518	3.8076152
36	0.00331	0.00373	12.688822	0.00355	7.2507553		
48	0.00245	0.0028	14.285714	0.00277	13.061224	0.0027	10.204082
60	0.00196	0.00207	5.6122449	0.00217	10.714286	0.00217	10.714286

Table A.10: Deflections and Deflection Difference

Joint stiffness=40000 psi, E (substructure)=50ksi, PCC thickness=3 in.

Sub-structure under WT slab with equivalent modulus: E=50ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00736	-53.884712	0.00725	-54.573935	0.0069	-56.766917
12	0.0041	0.00446	8.7804878	0.00441	7.5609756	0.00429	4.6341463
24	0.002			0.00208	4	0.00215	7.5
36	0.00132	0.00132	0	0.00127	-3.7878788		
48	0.00098	0.00097	-1.0204082	0.00095	-3.0612245	0.00096	-2.0408163
60	0.00079	0.00077	-2.5316456	0.00075	-5.0632911	0.00076	-3.7974684
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.0066	-58.646617	0.00649	-59.33584	0.00619	-61.215539
12	0.0041	0.00434	5.8536585	0.00429	4.6341463	0.00415	1.2195122
24	0.002			0.00216	8	0.00222	11
36	0.00132	0.00135	2.2727273	0.00128	-3.030303		
48	0.00098	0.00099	1.0204082	0.00096	-2.0408163	0.00097	-1.0204082
60	0.00079	0.00078	-1.2658228	0.00076	-3.7974684	0.00077	-2.5316456
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=3 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00611	-61.716792	0.00602	-62.280702	0.00574	-64.035088
12	0.0041	0.00422	2.9268293	0.00418	1.9512195	0.00403	-1.7073171
24	0.002			0.00221	10.5	0.00226	13
36	0.00132	0.00137	3.7878788	0.00131	-0.7575758		
48	0.00098	0.001	2.0408163	0.00098	0	0.00098	0
60	0.00079	0.00078	-1.2658228	0.00077	-2.5316456	0.00077	-2.5316456

Table A.11: Deflections and Deflection Difference

Joint stiffness=40000 psi, E (substructure)=50ksi, PCC thickness=4 in.

Sub-structure under WT slab with equivalent modulus: E=50ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00608	-61.904762	0.00599	-62.468672	0.00571	-64.223058
12	0.0041	0.00421	2.6829268	0.00417	1.7073171	0.00402	-1.9512195
24	0.002			0.00221	10.5	0.00226	13
36	0.00132	0.00137	3.7878788	0.00131	-0.7575758		
48	0.00098	0.001	2.0408163	0.00098	0	0.00098	0
60	0.00079	0.00078	-1.2658228	0.00077	-2.5316456	0.00077	-2.5316456
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00538	-66.290727	0.0053	-66.79198	0.00505	-68.358396
12	0.0041	0.00397	-3.1707317	0.00394	-3.902439	0.00379	-7.5609756
24	0.002			0.00226	13	0.0023	15
36	0.00132	0.0014	6.0606061	0.00133	0.7575758		
48	0.00098	0.00102	4.0816327	0.001	2.0408163	0.00099	1.0204082
60	0.00079	0.00079	0	0.00078	-1.2658228	0.00078	-1.2658228
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=4 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00494	-69.047619	0.00487	-69.486216	0.00465	-70.864662
12	0.0041	0.00379	-7.5609756	0.00375	-8.5365854	0.00362	-11.707317
24	0.002			0.00227	13.5	0.0023	15
36	0.00132	0.00141	6.8181818	0.00135	2.2727273		
48	0.00098	0.00103	5.1020408	0.00102	4.0816327	0.00101	3.0612245
60	0.00079	0.00079	0	0.00079	0	0.00079	0

Table A.12: Deflections and Deflection Difference

Joint stiffness=40000 psi, E (substructure)=50ksi, PCC thickness=5 in.

Sub-structure under WT slab with equivalent modulus: E=50ksi							
Load=82psi							
Distance from loading center (in.)	Load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=3000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00517	-67.606516	0.00509	-68.107769	0.00486	-69.548872
12	0.0041	0.00389	-5.1219512	0.00385	-6.097561	0.00372	-9.2682927
24	0.002			0.00227	13.5	0.0023	15
36	0.00132	0.00141	6.8181818	0.00134	1.5151515		
48	0.00098	0.00102	4.0816327	0.00101	3.0612245	0.001	2.0408163
60	0.00079	0.00079	0	0.00079	0	0.00079	0
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=5000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00453	-71.616541	0.00446	-72.055138	0.00428	-73.182957
12	0.0041	0.0036	-12.195122	0.00356	-13.170732	0.00345	-15.853659
24	0.002			0.00227	13.5	0.0023	15
36	0.00132	0.00143	8.3333333	0.00136	3.030303		
48	0.00098	0.00105	7.1428571	0.00103	5.1020408	0.00103	5.1020408
60	0.00079	0.0008	1.2658228	0.00081	2.5316456	0.00081	2.5316456
Distance from loading center (in.)	load on AC	Load on PCC					
		PCC thickness=5 in.					
	Deflection (in.)	PCC E=7000 ksi					
		Slab size: 4 by 4 ft. Deflection (in.)	Deflection difference%	Slab size: 5 by 5 ft. Deflection (in.)	Deflection difference%	Slab size: 6 by 6 ft. Deflection (in.)	Deflection difference%
0	0.01596	0.00415	-73.997494	0.00408	-74.43609	0.00392	-75.438596
12	0.0041	0.0034	-17.073171	0.00335	-18.292683	0.00325	-20.731707
24	0.002			0.00225	12.5	0.00226	13
36	0.00132	0.00143	8.3333333	0.00137	3.7878788		
48	0.00098	0.00106	8.1632653	0.00105	7.1428571	0.00104	6.122449
60	0.00079	0.0008	1.2658228	0.00082	3.7974684	0.00082	3.7974684

APPENDIX B: INTRODUCTION OF THE WIHTETOPPING PAVEMENTS IN THE STUDY

Introduction of Whitetopping Projects

1. CTH "A"

CTH "A" was finished in 2007 in Dodge County. The project is 4.2 miles long with slab thickness of 7.5 in. and slab size of 15 ft. by 15 ft. The existing HMA had 2 in. milled off before whitetopping. The field distress survey indicates a PCI of 89 and PDI of 4.65. No other distress found except several minor Distressed Joints/Cracks and Patching. Figure B1 shows the condition of CTH "A" as of July 2008.

Figure B1: Condition of CTH "A" (2008)

2. Duplainville Road

The Duplainville Road whitetopping project is a local road located in Waukesha County. It was still in service as of July 2008. This project was built in 1999 with slab thickness of 7 in. and joint spacing of 5.5 ft. by 11 ft. Three pounds of fiber per cubic yard of concrete was used in mix design. The existing HMA had 1 in. milled off before the whitetopping overlay. The field distress survey indicates a PCI of 85 and a PDI of 6.70 showing a good condition. There are several Distressed Joints/Cracks found in this project. Figure B2 shows the condition of Duplainville Road as of July 2008.

Figure B2: Condition of Duplainville Road (2008)

3. Fond Du Lac Ave

The Fond Du Lac Avenue whitetopping project is a local road located in Milwaukee County. It was still in service as of July 2009. This UTW project was built in 2001, was 375 ft. long, with a slab thickness of 4 in. and joint spacing of 4 ft. by 4 ft. Three pounds of fiber per cubic yard of concrete was used in the mix design. The cored thickness was 3.7 in. for the PCC slab and 1.5 in. for the HMA. The field distress survey indicates a PCI of 58 and PDI of 64.4. The major types of distress are Slab Breakup, Distressed Joints/Cracks, and Patching. Figure B3 shows the condition of this whitetopping project as of July 2009.

Figure B3: Condition of Fond Du Lac Avenue (2009)

Figure B4: Condition of Galena Street (2009)

4. Galena Street

Galena Street was built in 1995 in Milwaukee County. It was the first whitetopping project in Wisconsin. This project is 750 ft. long with a slab thickness of 4 in. and slab size of 6.5 ft. by 6 ft. Three pounds per cubic yard fiber was used in this project. Cold milling was used as pre-overlay preparation. It was reported that a severe blow-up appeared at the intersection and permanent repair was performed in 1998. The field distress survey resulted in a PCI of 55 and PDI of 65.76. The major types of distress are Slab Breakup, Distressed Joints/Cracks, and Patching. Many slabs have been replaced by full-depth patching. Figure B4 shows the condition of Galena Street as of July 2009.

5. Howard Avenue

This whitetopping project is located inside a water processing plant in Milwaukee County. It was still in service as of July 2009. This UTW projects was built in 1999 with a slab thickness of 4 in. and joint spacing varying from 4 ft. to 6 ft. Three pounds per cubic yard of polypropylene fiber was used in the mix design. No distress survey was conducted for this project, because it can not be accessed due to security restrictions.

6. IH 94/ STH 50 Ramp

The IH 94/STH 50 Ramp is located on the off-ramp of IH 94 in Kenosha County. It is 200 ft. long. This project was finished in 1998. The existing 7.5 in. HMA had 4 in. milled off before the whitetopping overlay. The slab thickness is 4 in. and the slab size is 4 ft. by 4 ft. Three pounds of fiber per cubic yard of concrete was used in the mix design. The field distress survey indicates a PCI of 72 and PDI of 41.73. The major types of distress are Slab Breakup and Distressed Joints/Cracks. There are localized severely broken slabs at the transition areas. Figure B5 shows the condition of IH94/STH 50 as of July 2009.

Figure B5: Condition of IH 94/STH 50 Ramp (2009)

7. Janesville Avenue and Rockwell Avenue Intersection

The Janesville Avenue and Rockwell Avenue Intersection whitetopping project in Jefferson County was finished in 1997. It has been out of service since 2004. The project had a slab thickness of 4 in. and slab size of 5.5 ft. by 6 ft. A 4-in. cold milling was performed before whitetopping and 3 pounds of fiber per cubic yard of concrete was used in this project.

8. Lawndale Avenue

The Lawndale Avenue whitetopping project is a local road located in Slinger village, Washington County. It was still in service as of July 2008. This UTW project was built in 2001, was 750 ft. long, with a slab thickness of 4 in. and joint spacing of 4 ft. by 4 ft. The cored thickness is 3.95 in. for the slab and 3.25 in. for the HMA. Three pounds of fiber per cubic yard of concrete was used in the mix design. The field distress survey indicats a PCI of 76 and PDI of 32.11. The major type of distress is Slab Breakup. Figure B6 shows the condition of Lawndale Avenue as of August 2008.

Figure B6: Condition of Lawndale Avenue (2008)

Figure B7: Condition of North 39th Avenue (2008)

9. North 39th Avenue

The North 39th Avenue whitetopping project was built in 1999 in Kenosha County. It has a slab thickness of 4 in. and a slab size of 6 ft. by 6 ft. The cores indicated a slab thickness of 4.2 in. and HMA of 3.5 in. Three pounds of fiber per cubic yard of concrete was used in the mix design. The field distress survey indicates a PCI of 78 and PDI of 13.10. There are some Slab Breakups and Distressed Joints/Cracks found in this project. Figure B7 shows the condition of North 39th Avenue as of July 2008.

10. State Street

State Street is located in Milwaukee County. This road was built in 2000 for Central Ready Mix company which has been closed. The slab thickness varies from 6 in. to 8 in. The slab size is 5.5 ft. by 6 ft. 3 pounds per cubic yard polypropylene fiber was used in this project except for the outbound lane which used 20 pounds steel fiber per cubic yard of concrete. The field distress survey indicates a PCI of 94 and PDI of 7.76. Several Slab Breakups and Distressed Joints/Cracks are found. Figure B8 shows the condition of State Street as of July 2009.

Figure B8: Condition of State Street (2009)

11. STH 33 and CTH "A" Intersection

The STH33 and CTH "A" intersection is located in Dodge County and was built in 2001. Four inches of the existing HMA was milled off and a 4-in. thickness of whitetopping was placed with joint spacing of 4 ft. by 4 ft. The field distress survey indicates a PCI of 69 and PDI of 34.10. The major type of distress is Slab Breakup (corner cracking). Figure B9 shows the condition of the STH33 and CTH "A" intersection as of July 2008.

Figure B9: Condition of STH33 and CTH "A" (2008)

12. STH 33 and STH67 Intersection

The STH33 and STH67 intersection is located in Dodge County and was built in 2001. It has been out of service since 2008 prior to the survey. It had a slab thickness of 4 in. and slab size of 4 ft. by 4 ft.

13. STH 50

The STH 50 whitetopping project is located close to the IH 94/STH 50 Ramp project in Kenosha County. It was finished in 2001. There is no other information available except that the slab size is 5 ft. by 5 ft.. The field distress survey indicates a PCI of 71 and PDI of 27.57. Figure 25 shows the condition of the STH 50 whitetopping project as of July 2009. The transition areas exhibit severe slab breakup, as shown in Figure B10 (right).

Figure B10: Condition of STH 50 (2009)

14. STH 54

The STH 54 whitetopping project was built in 2001 in Portage County with a slab thickness of 7 in. and slab size of 12 ft. by 15 ft. Dowel bars were used in this project. The pavement structure consists of a 7-in. PCC slab over 13.5 in. HMA 17 in. crushed aggregate base course (CABC). The existing HMA had 0.5 in. milled off as pre-overlay preparation. The field distress survey indicates a PCI of 74 and PDI of 26.63. There are several Slab Breakups and Distressed Joints/Cracks found in this project. Figure B11 shows the condition of STH 54 as of July 2008.

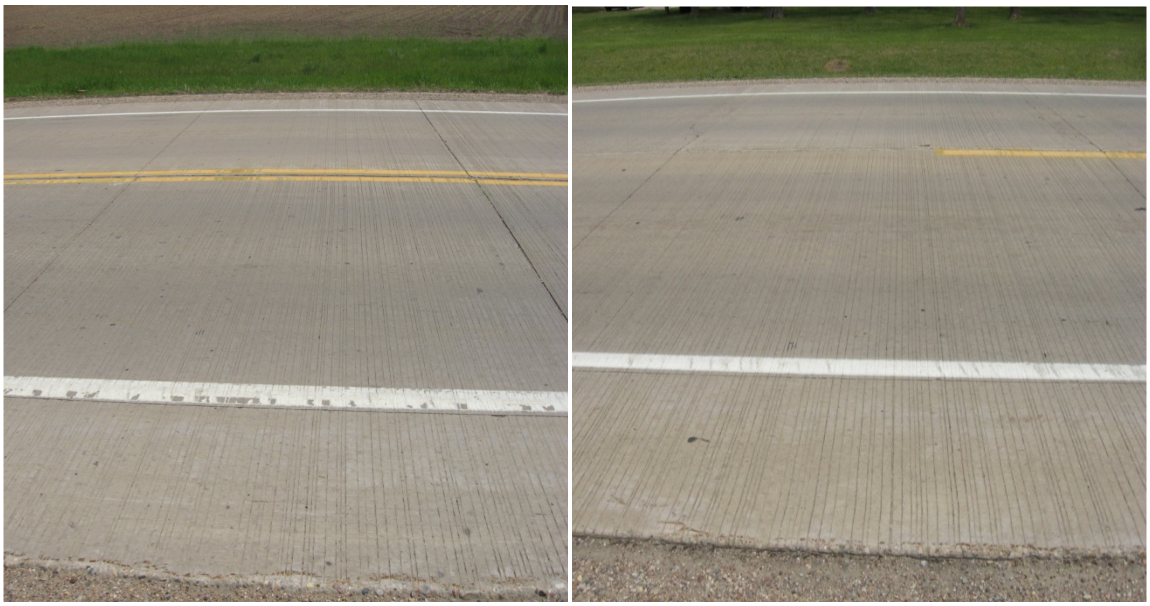

Figure B11: Condition of STH 54 (2008)

15. STH 82

The STH 82 whitetopping project is located in Adams County. It is currently the longest whitetopping project in Wisconsin at 12.3 miles, and was still in service as of July 2008. This project was built in 2001 with a slab thickness of 5 in. and joint spacing of 5 ft. by 5 ft. Three pounds of fiber per cubic yard of concrete was used in the mix design. Less than 0.5 in. of HMA was milled off as pre-overlay surface preparation. The field distress survey indicates a PCI of 91 and PDI of 7.37. There are several Distressed Joints/Cracks and Patching found in this project. Figure B12 shows the condition of STH 82 as of July 2008.

Figure B12 Condition of STH 82 (2008)

Figure B13: Condition of STH 97 (2008)

16. STH 97

STH 97 in Taylor County was finished in 1999. The project is 1.5 miles long with a slab thickness of 4 in. and slab size of 5.5 ft. by 6 ft. No milling was conducted before the overlay and only 1.5 pounds of fiber per cubic yard of concrete was used. The field distress survey indicates a PCI of 81 and PDI of 6.73. Several corner breaks were found in this project. Figure B13 shows the condition of STH 97 as of July 2008.

17. USH 2/USH 53

USH 2/USH53 is located in Douglas County. It was built in 2001. In this part of the road, USH 2 and USH 53 merged together. The project is 6.6 miles long with a slab thickness of 9 in. and slab size of 15 ft. by 15 ft. Dowel bars were used in this project. Less than 0.5 in. of the existing HMA was milled off during the surface preparation before whitetopping. The field distress survey indicates a PCI of 82 and PDI of 32.40. Some Slab Breakup and Distressed Joints/Cracks are found.

18. Washington Street and 22nd Street Intersection

The Washington Street and 22nd Street intersection is located in Kenosha County and was built in 2001. The existing HMA had 4 in. milled off. The slab thickness is 4 in. and the slab size is 4 ft. by 4 ft. Three pounds of fiber per cubic yard of concrete was used in the mix design. The field distress survey indicates a PCI of 64 and PDI of 25.66. The major types of distress are Slab Breakup and Patching. Figure B14 shows the condition of the Washington Street and 22nd Street Intersection as of July 2008.

Figure B14: Condition of Washington Street and 22nd Street Intersection (2008)