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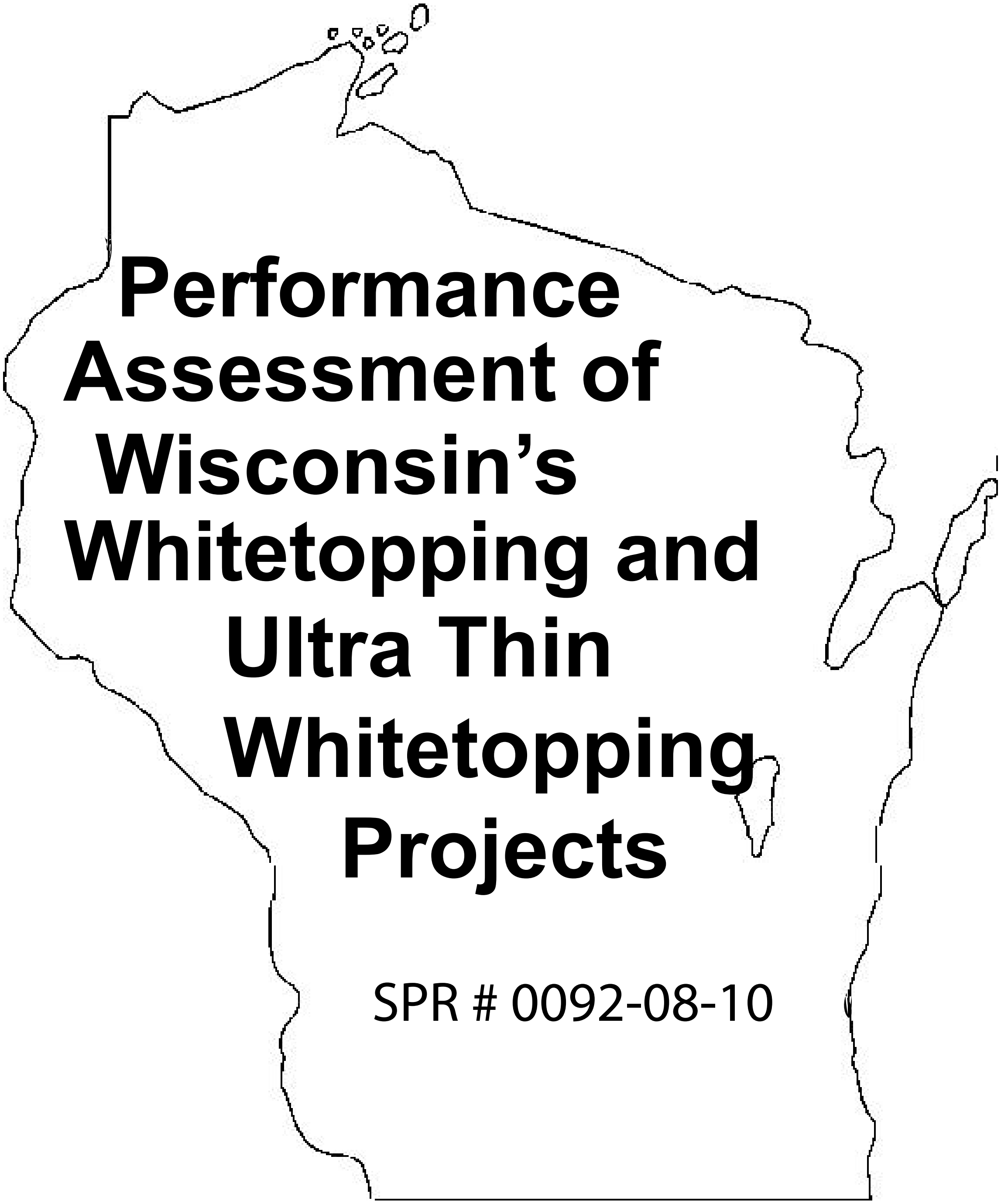

Performance Assessment of Wisconsin's Whitetopping and Ultra Thin Whitetopping Projects

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16. Abstract

Whitetopping overlay is a concrete overlay on the prepared existing hot mix asphalt (HMA) pavement to improve both the structural and functional capability. It's a relatively new rehabilitation technology for deteriorated HMA. If the slab thickness is less or equal to 4 in., it is referred to as ultra-thin whitetopping (UTW). "WT" is used to refer to concrete overlay thicker than 4 in. In this research, the term "whitetopping" is used to refer to both WT and UTW in general.

The primary objectives of this study are to catalog the whitetopping (WT) and UTW projects in Wisconsin, document pertinent design and construction elements, assess performance and estimate a service life of these projects. A comprehensive literature review was performed. A database of the WT and UTW projects was established covering 18 projects built from 1995 to 2007 in Wisconsin.

The performance of these WT and UWT projects were assessed, by mean of shear strength tests on field cores, falling weight deflectometer (FWD) tests on selected projects, and field distress survey on in-service projects. FWD backcalculation methods for WT and UTW pavements were studied and a Critical Distance Method was proposed and utilized for UTW pavement. Fatigue life was analyzed using 18kip, 22kip and 26kip single axle load level. Performance assessment was conducted using both Pavement Condition Index (PCI) and Pavement Distress Index (PDI). The performance of whitetopping projects in Wisconsin was found comparable to that in other states. Whitetopping overlay thickness, joint spacing and pavement age were found to have significant effects on pavement performance.

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EXECUTIVE SUMMARY

PROJECT SUMMARY

This research consists of cataloging the whitetopping (WT) and ultra-thin whitetopping (UTW) projects in Wisconsin, documenting pertinent design and construction elements, conducting forensic investigation, assessing performance of these projects, and estimating a service life for these WT and UTW projects for design and life cycle cost analysis (LCCA) in Wisconsin.

PROJECT BACKGROUND

In Wisconsin, a number of whitetopping projects have been built. However 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 service life need to be developed so that rehabilitation techniques can be appropriately incorporated in to pavement LCCA. Assessment of the performance to date and the estimate of the corresponding service life will allow highway agencies to make informed decisions regarding appropriate rehabilitation techniques.

The Wisconsin Department of Transportation, through the Wisconsin Highway Research Program, sponsored this study.

PROCESS

A comprehensive literature review was performed, to collect mechanical analysis, design and construction procedure, and performance of whitetopping overlay. Field evaluation was conducted, including shear strength tests, falling weight deflectometer (FWD) tests, and field distress surveys. FWD test backcalculation methods for whitetopping pavement were studied and fatigue life was analyzed. Performance assessment was conducted using both the Pavement Condition Index (PCI) and the Pavement Distress Index (PDI). The performance of whitetopping projects in Wisconsin was compared to the performance of whitetopping projects in other states. Factors affecting performance were statistically analyzed.

FINDINGS

After investigation and data analysis, a database of the whitetopping projects in Wisconsin was established. The performance of whitetopping pavements in Wisconsin was assessed and the service lives were analyzed using FWD backcalculated pavement properties and statistics. Specifically, the findings are as follows:

(1) Based on the literature review, whitetopping and ultra-thin whitetopping have gained popularity in the last twenty years. The condition of the existing asphalt pavement is important. A good bond between the PCC overlay and the existing HMA is recommended. Following proper whitetopping design

and construction practices is recommended to create whitetopping pavement that will perform according to the need of the agencies.

(2) As of 2008, there have been a total of 18 projects that could be defined as whitetopping in Wisconsin. The projects were built from 1995 to 2007. Slab thicknesses range from 4 in. to 9 in. and joint spacing range from 4 ft. by 4 ft. to 15 ft. by 15 ft. Eleven of the projects are UTW projects. The two most commonly used joint spacings are 4 ft. by 4 ft. and 6 ft. by 6 ft. Fiber was used in 13 projects and only 3 projects used dowel bars.

(3) 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 of whitetopping should be based on an unbonded condition, to be safe.

(4) Traditional backcalculation methods of concrete pavement layer properties, based on FWD testing, are not applicable to the UTW pavements. The new Critical Distance Method, developed by the team, shows potential to be used in UTW pavement FWD test backcalculation.

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

(6) Critical loading position depends on the pavement structure and slab layout. Thermal stress has little effect for typical UTW overlay due to the relatively short joint spacing and thin slab thickness. 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 major cause of fatigue.

(7) 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. Design of whitetopping should be based on heavier loads than the 18-kip standard axle load, or load spectrum.

(8) The performance of the whitetopping projects in Wisconsin is comparable to that in other states.

(9) Slab thickness, slab size, and pavement age of overlay were found to be statistically significant variables that affect the performance of whitetopping pavements.

(10) 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.

RECOMMENDATIONS

(1) It is recommended that a design method should be developed to reduce the variation of performance of whitetopping pavements in Wisconsin.

(2) The design method should be based on an unbonded condition to be conservative.

(3) The design method should not be based on the 18-kip standard axle loads. Instead, higher load levels or load spectrum should be used.

(4) It is recommended that the Mechanistic-Empirical Pavement Design Guide (MEPDG) could be calibrated, based on the performance of whitetopping pavements nationwide, and refined based on the performance of pavements in

Wisconsin. Alternatively, the current ACPA method can be modified as a simplified design approach.

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

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CHAPTER 1. INTRODUCTION

1.1. BACKGROUND

Traditionally, the most common rehabilitation method for existing hot mix asphalt (HMA) pavements is an asphalt overlay. However, the performance of HMA overlay is very sensitive to the conditions of the underlying HMA pavement. Wen et al. studied the performance of 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 HMA pavement could be reflected in the HMA overlay (Wen et al., 2006).

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). To be consistent with work done previously by others, in this research, the term “whitetopping” is used to refer to both WT and UTW in general. “WT” is used to refer to concrete overlay thicker than 4 in. and “UTW” to overlay equal to/less than 4 in. To be convenient, in the report IH, STH, USH, and CTH were used to refer to as Interstate Highway, State Highway, U.S. Highway, and County Highway respectively. Full road names were used for other local projects.

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 establishment of appropriate design procedures and the corresponding service life 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 study are to catalog the WT and UTW projects in Wisconsin, document pertinent design and construction elements, assess performance of these projects, statistically analyze factors affecting performance, and estimate a service life for WT and UTW.

1.3. ORGANIZATION OF REPORT

This report describes the performance assessment 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. A catalogue of whitetopping projects is provided in Chapter 4. Chapter 5 describes the results of performance assessment. Conclusions and recommendations are given in Chapter 6.

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 1997). 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 effects 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.

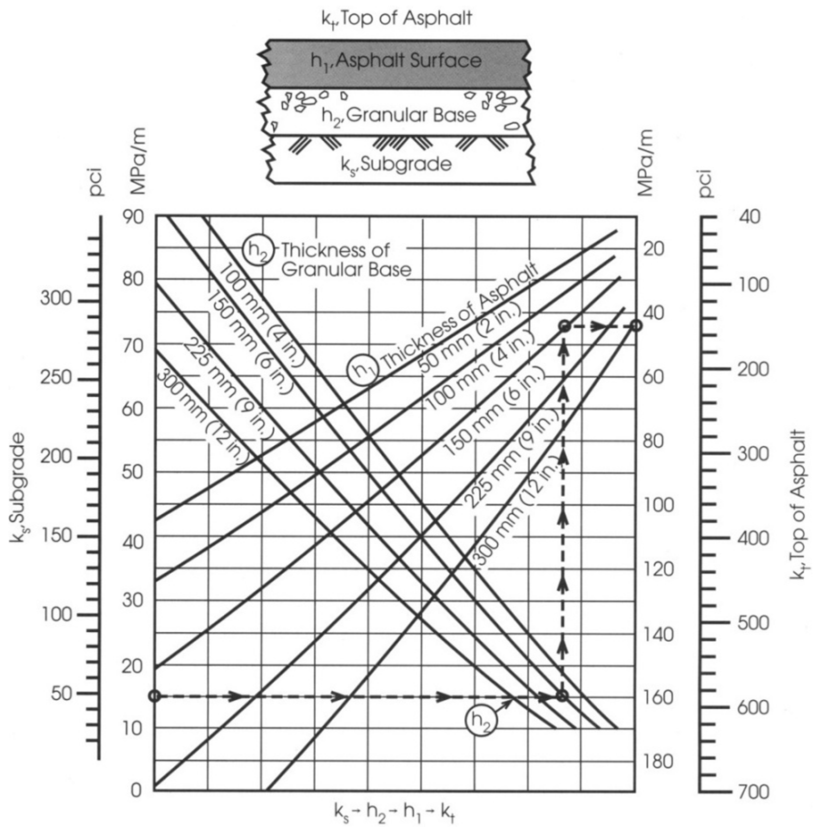

Figure 2. k-value on top of HMA pavement with granular base (ACPA 2002)

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/yd^3

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 following equation.

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

Where fr = flexural strength (modulus of rupture), C = a constant (0.75 for metric unit and 0.90 for US units), and 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 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). 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 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).

It is important to mention the weather conditions during the curing of concrete material. Lin (2005) reported that an air temperature higher than 90°F 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.

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

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.

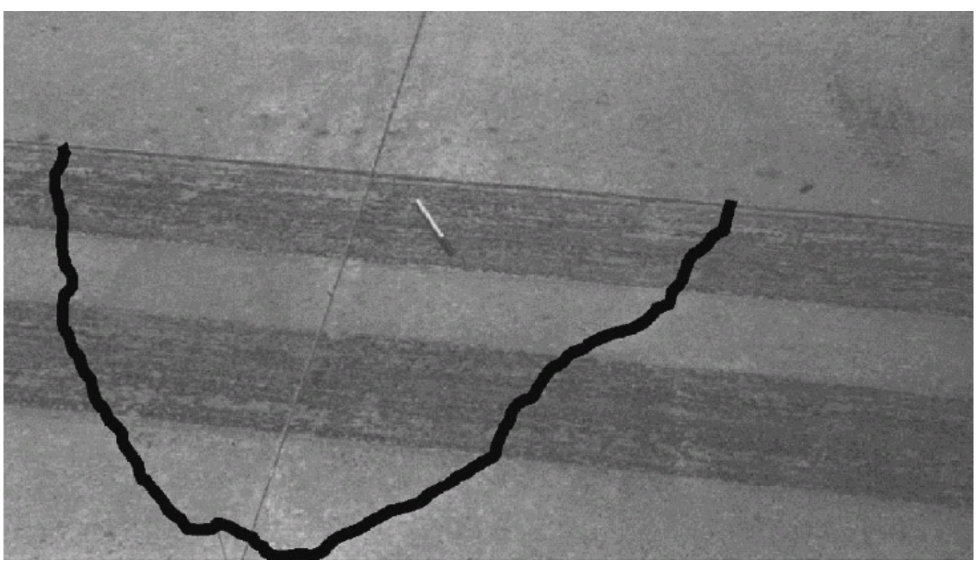

Figure 4. Corner Cracking (Rasmussen et al. 2002)

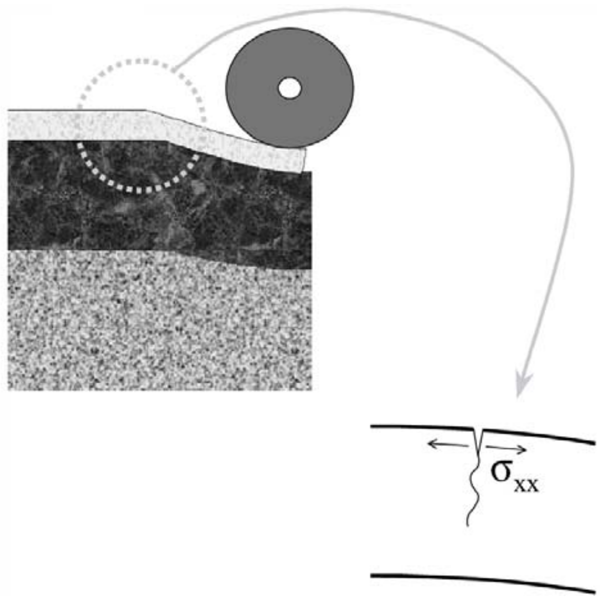

Figure 5. Corner cracking mechanism (Rasmussen et al. 2002)

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.

- 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.

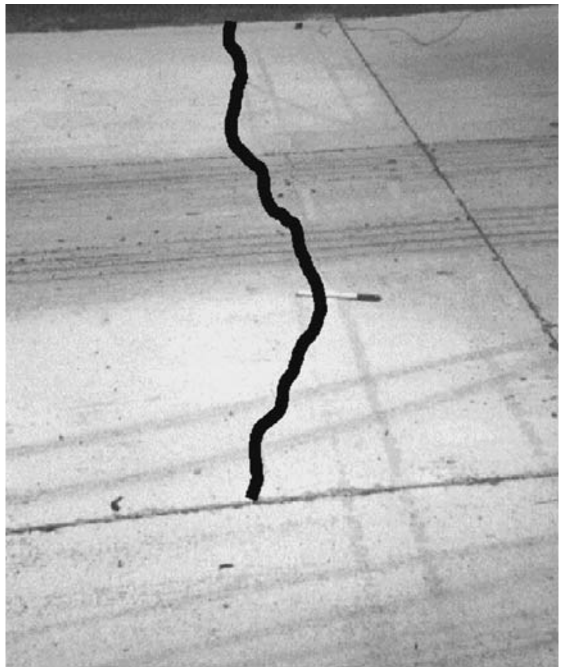

Figure 6. Mid-Slab Cracking (Rasmussen et al. 2002)

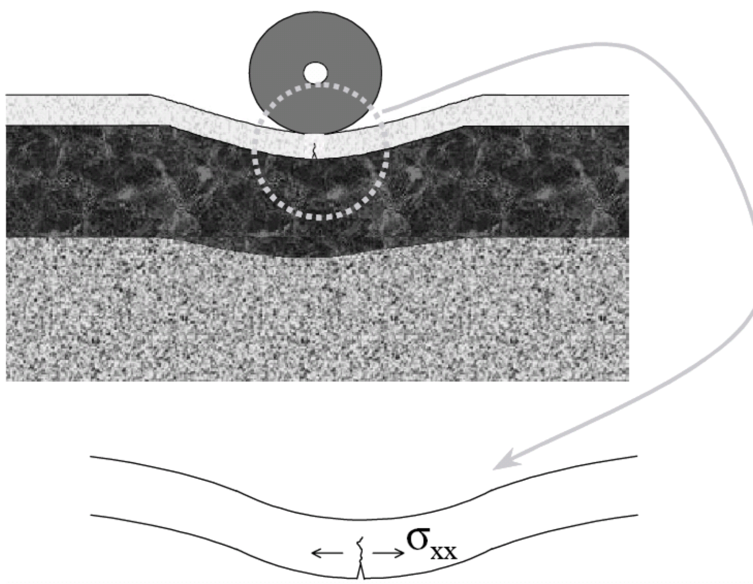

Figure 7. Mid-slab cracking mechanisms (Rasmussen et al. 2002)

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.

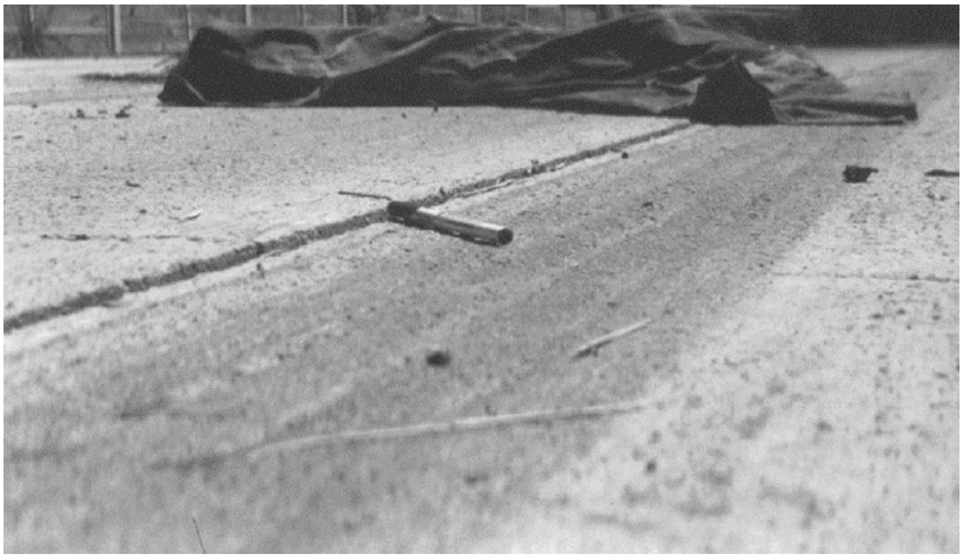

Figure 8. Longitudinal Faulting (Rasmussen et al. 2002)

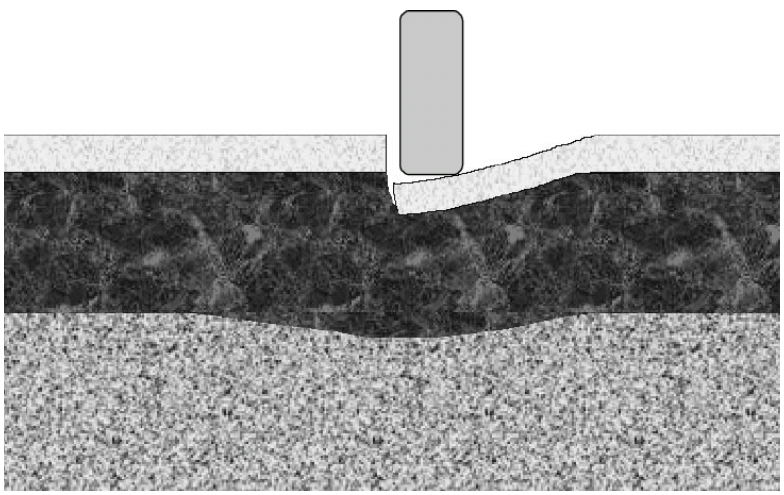

Figure 9. Longitudinal joint faulting mechanisms (Rasmussen et al. 2002)

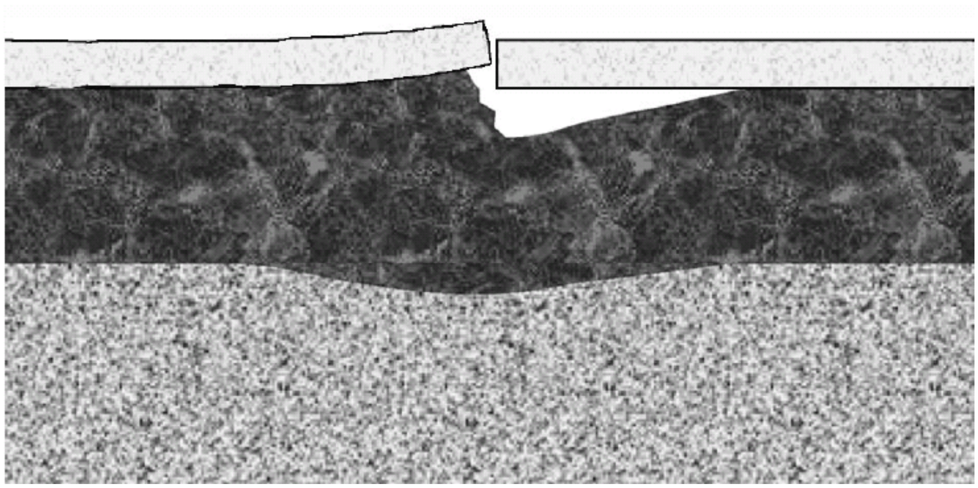

Figure 10. Transverse joint faulting mechanisms (Rasmussen et al. 2002)

2.4.4. Joint Spalling

Figure 11 shows an example of joint spalling. Rasmussen (2002) indicated that there are two common types of joint spalling: delamination spalling and deflection spalling. Delamination spalling is 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. However, there could be other reasons. Figure 12 illustrates the joint spalling mechanism by Rasmussen (2002).

Figure 11. Spalling (Rasmussen et al. 2002)

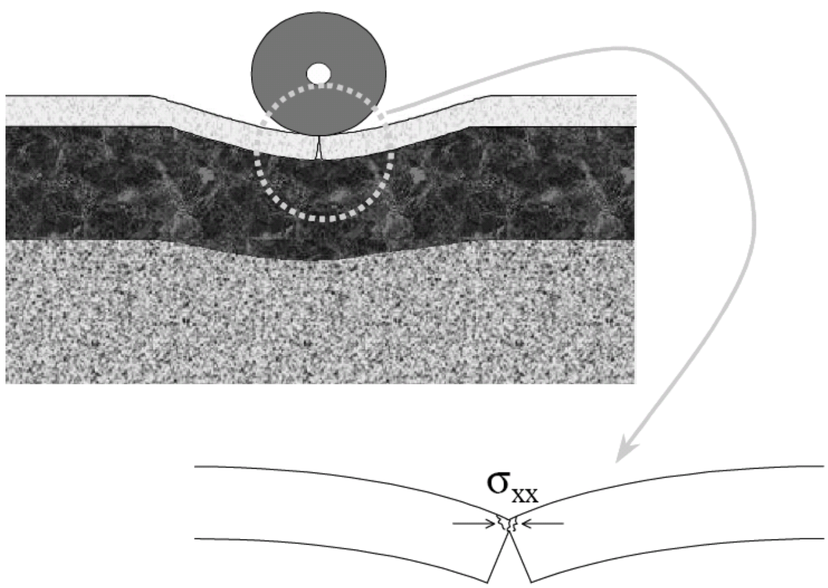

Figure 12. Joint Spalling Mechanisms (Rasmussen et al. 2002)

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 PROJECTS

To assess the performance of whitetopping pavements in Wisconsin, distress surveys were conducted on in-service pavements. Falling weight deflectometer (FWD) tests and coring were undertaken on selected projects to cover a range of performance.

3.1. 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 projects, 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 projects, 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.

3.2. 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. A 4-in. diameter core barrel was used in the field. The shear strength tests were conducted on the cores from 4 projects following the test protocol of the Iowa shear strength test (Iowa 406-C). These 4 projects are Lawndale Avenue (Washington County), STH 82 (Adams County), North 39 Avenue (Kenosha County) and CTH “A” (Dodge County). Figure 13 shows the cores and the test equipment. There were no cores tested for Fond Du Lac Ave., because the PCC and HMA were separated.

Figure 13. Cores (left) and Equipment (right) Used in Bond Strength Test

It is noted that the concrete and asphalt were separated in most of the cores and the shear strength could not be determined for the separated specimens.

3.3. 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 5,200, 9000, and 12500 lb, 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 shown in Figure 14. FWD test data were used for backcalculating the pavement properties and evaluating the performance of the projects.

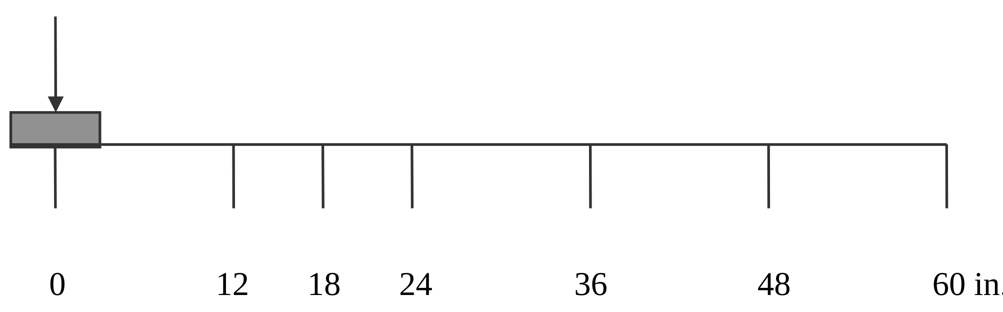

Figure 14: Deflection Sensor Spacing in FWD Test

CHAPTER 4. CATALOGUE OF THE WHITETOPPING PROJECTS IN WISCONSIN

4.1. INTRODUCTION

One of the purposes of this study was to develop a database of whitetopping projects in Wisconsin. The research group collected information from the Wisconsin Concrete Pavement Association, the Wisconsin Department of Transportation (WisDOT), local governments, designers, and contractors. The information collected 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 and 16 of them were in-service, two of them out-of-service as of 2009. The slab thickness ranges from 4 in. to 9 in. and the joint spacing ranges from 4 ft. by 4 ft. to 15 ft. by 15 ft. Although the research group tried to collect as much information as possible, some important information is still missing.

4.2. CATALOGUE OF THE WHITETOPPING PROJECTS IN WISCONSIN

Table 3 lists the whitetopping projects and the information collected about them. It should be noted that the last two projects in Table 3 are not considered as whitetopping. STH 13 is a concrete overlay of concrete pavement. For CTH “R”, the existing HMA was completely milled off. This section provides a detailed description of each project. Figure 15 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.

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

No	Road or Project Name	Year	Type	County	In Service	Project ID	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)	After Whitetopping Sub Grade	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	RD-00-04	CTH F	RR Crossing	1	7	6	7.6" PCC	6" CABC	Unknown	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		Unkown	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			Unknown	3
7	Janesville and Rockwell Ave	1997	Intersection	Jefferson	N	3991-02-50			4	4	2	6" CABC	9" Unkown	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	3994-07-70				4	3.5				3
10	State Street	2000		Milwaukee	Y		Driveway to Central Ready Mix company			7					3
11	STH33 and CTH "A"	2001	Intersection	Dodge	Y		Intersection		4	4					Y
12	STH33 and STH67	2001	Intersection	Dodge	N		Intersection			4					
13	STH 50	2001	Highway	Kenosha	Y		Just west of IH94 and STH 50 intersection								
14	STH 54	2001	Highway	Portage	Y	6414-01-70	Plover, WI	IH 39	0.5	7	6.5	7" HMA	17" CABC	Unknown	
15	STH 82	2001	Highway	Adams	Y	1430-01-72	STH 13	Adams, WI	0.5	5	1.5	HMA	CABC	Unknown	3
16	STH 97	1999	Highway	Taylor	Y	9536-01-73	Taylor CO Line	STH 64	0	4	3	10" CABC		Unknown	1.5
17	USH 2/USH 53	2001		Douglas	Y	1199-10-71	CH B	USH 2	<0.5	9	9	7"CABC			
18	Washington St. and 22nd St.	2001	Intersection	Kenosha	Y	#00-1014	Intersection		4	4					3
19	STH 13	1985	Highway	Adams	Y				0	2.5					
20	CTH R	2001-2002	Local Road	Manitowoc	Y				2	4					3

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

No	Road or Project Name	Design Traffic (mph)	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 Thick-ness (inch)	FWD Test	Core thickness PCC (in.)	Core thickness HMA (in.)	Iowa shear test (psi)	No. of lanes	Field Distress Survey	Notes
1	CTH A					22,420	24	15	15	7.5	Y	7.81	1.75	154.07	2	Y	
2	Duplainville Road		4,494,979	20	2,247,490		22	5.5	11	7	Y				4	Y	Dowel bar used
3	Fond Du Lac Ave					375	60	4	4	4	Y	3.7	1.5		3 (1 direction)	Y	
4	Galena St		132,483	20	92,738	750	24	6.5	6	4						Y	blowout and repaired in 1998
5	Howard Ave		k					6	6	4						N	Inside a water processing plant in Milwaukee
6	IH 94/STH 50 Ramp		1,230,361	10	676,528	200	36	4	4	4					3 (1 direction)	Y	
7	Janesville and Rockwell Ave	35	2,029,764	10	1,623,811			5.5	6	4						N	Re-surfaced in 2004
8	Lawndale Ave					750	32	4.75	6	4	Y	3.95	3.25	266.05	2	Y	Entire street
9	North 39th Avenue		1,554,900	20	777,450	263	48	6	6	4		4.2	3.5	177.29	4	Y	Entire street
10	State Street							5.5	6	6 --8						Y	Outbond lane 20 lb steel fiber. No traffic now
11	STH33 and CTH "A"					250	24	4	4	4					Int.	Y	
12	STH33 and STH67					250	24	4	4	4						N	Out of service
13	STH 50							5	5							Y	
14	STH 54	60	4,971,300	10	3,977,040	9,874	24	12	15	7					2	Y	Dowel bar used
15	STH 82	60	3,248,500	20	1,299,400	64,944	30	5	5	5	Y	6.13	1.5	124.59	2	Y	
16	STH 97		819,717	20	409,900	7,920	22	5.5	6	4					2	Y	
17	USH 2/USH 53	70	4,781,500	20	1,912,600	34,727	48	15	15	9						Y	Dowel bar used
18	Washington St. and 22nd St.					244	48	4	4	4					Int.	Y	
19	STH 13		2,160,800			17,989	24	12	12	2.5					2 (in one direction)	Y	Not considered as WT project
20	CTH R					6,400	30	5	6	4					4	Y	Not considered as WT project

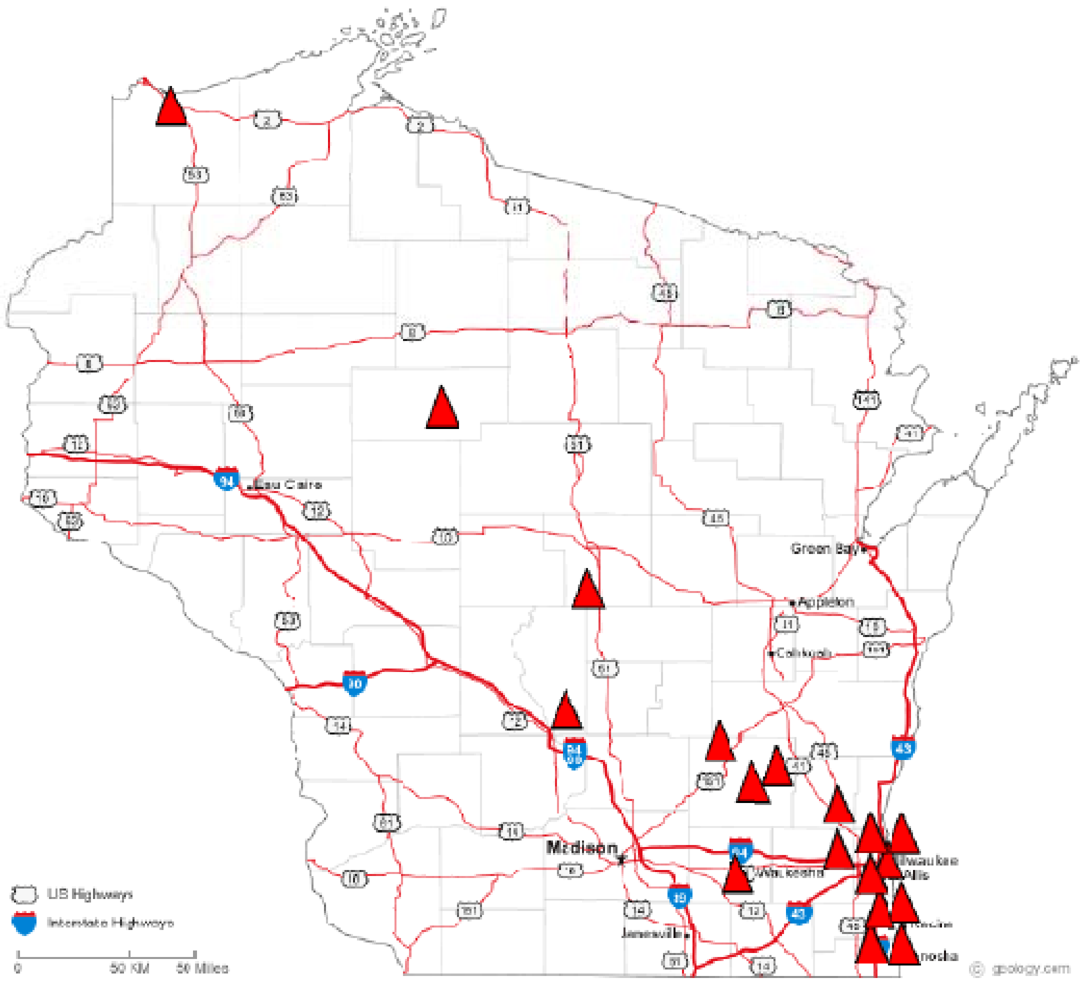

Figure 15. Locations of Whitetopping Projects in Wisconsin

4.2.1. 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 16 shows the condition of CTH “A” as of July 2008.

Figure 16. Condition of CTH “A” (2008)

4.2.2. Duplainville Road

Figure 17. Condition of Duplainville Road (2008)

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 17 shows the condition of Duplainville Road as of July 2008.

4.2.3. 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 18 shows the condition of this whitetopping project as of July 2009.

Figure 18. Condition of Fond Du Lac Avenue (2009)

4.2.4. 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 19 shows the condition of Galena Street as of July 2009.

Figure 19. Condition of Galena Street (2009)

4.2.5. 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.

4.2.6. 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 20 shows the condition of IH94/STH 50 as of July 2009.

Figure 20. Condition of IH 94/STH 50 Ramp (2009)

4.2.7. 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.

4.2.8. 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 21 shows the condition of Lawndale Avenue as of August 2008.

Figure 21. Condition of Lawndale Avenue (2008)

4.2.9. 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 22 shows the condition of North 39th Avenue as of July 2008.

Figure 22. Condition of North 39th Avenue (2008)

4.2.10. 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 23 shows the condition of State Street as of July 2009.

Figure 23. Condition of State Street (2009)

4.2.11. 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 24 shows the condition of the STH33 and CTH “A” intersection as of July 2008.

Figure 24. Condition of STH33 and CTH “A” (2008)

4.2.12. 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.

4.2.13. 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 25 (right).

Figure 25. Condition of STH 50 (2009)

4.2.14. 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 26 shows the condition of STH 54 as of July 2008.

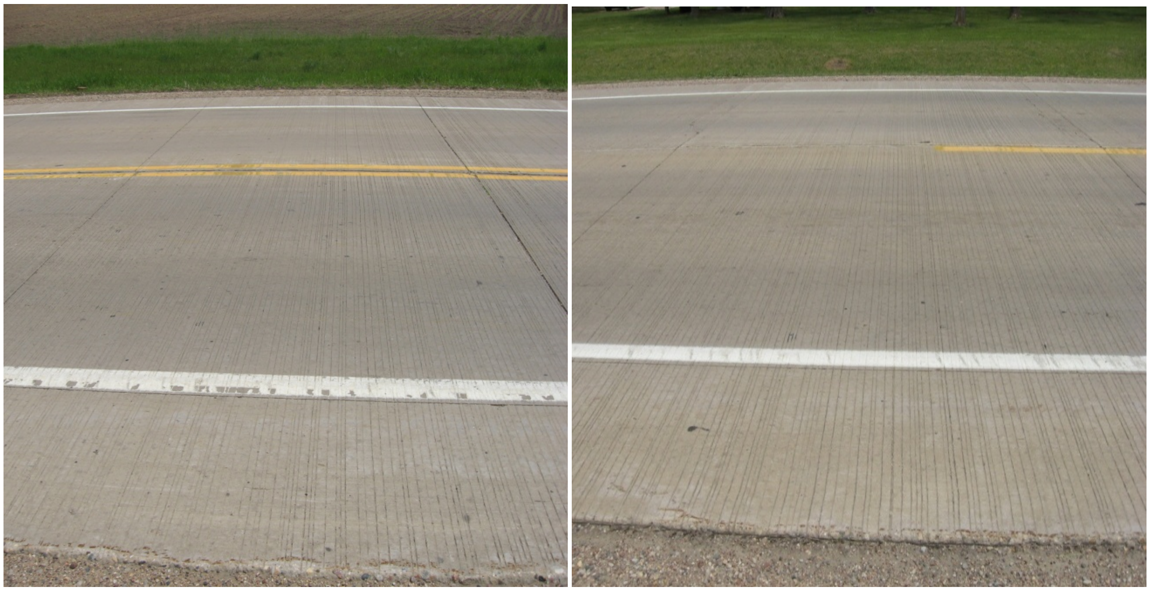

Figure 26. Condition of STH 54 (2008)

4.2.15. STH 82

Figure 27. Condition of STH 82 (2008)

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 27 shows the condition of STH 82 as of July 2008.

4.2.16. 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 28 shows the condition of STH 97 as of July 2008.

Figure 28. Condition of STH 97 (2008)

4.2.17. 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.

4.2.18. Washington Street and 22nd Street Intersection

Figure 29. Condition of Washington Street and 22nd Street Intersection (2008)

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 29 shows the condition of the Washington Street and 22^{nd} Street Intersection as of July 2008.

CHAPTER 5. ANALYSIS AND RESULTS

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

5.1. 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. So only test results of 4 projects are shown in Table 4.

Table 4. 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

From Table 4, 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. 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 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.

5.2. ANALYSIS BASED ON FWD TEST

5.2.1. Data Preparation

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 FWD equipment. Therefore, only CTH “A”, STH 82 and Lawndale Avenue were included in FWD test backcalculation in this study.

5.2.2. Traditional 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, J. K. et al, 2001). For small slab sizes, the sensors could be on separate slabs. In this research, backcalculation programs, including “Evercalc” (WS DOT 2005), “Modcomp 6” (Irwin, L.H. 2003), and equations based on “AREA” theory (Hall, K. T. et al, 1991) were tried. It is noted that all three approaches are based on the assumption of infinite slab size , which is false for UTW pavements, due to their small slab size.

5.2.2.1. “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 a useful FWD test backcalculation method for asphaltic pavement. When used in this research project, it gave unreasonable HMA layer moduli and was not used further.

5.2.2.2. “Modcomp 6”

“Modcomp 6” is a program developed by Cornell University. It accounts for the nonlinearity of material properties and was recommended by the Federal Highway Administration for the long term pavement performance (LTPP) data analysis.

5.2.2.3. Equation based on "AREA" theory

The equation based on the "AREA" theory is a closed-form backcalculation method for PCC pavement. It uses the deflection at 0, 12, 24, and 36 in. from loading center to get the "AREA" and uses the "AREA" to calculate the elastic solid radius of relative stiffness "l_e" which is related to the PCC modulus (Hall, K. T. et al, 1991).

5.2.3. Critical Distance Method

In order to deal with the problems in FWD test backcalculation for whitetopping projects, a new method for UTW pavement backcalculation, called Critical Distance Method, was developed in this research. For UTW pavement which has a slab thickness of 4 in. or less, the biggest challenge is the discontinuity of slab in the UTW overlay due to the relatively small slab size. The traditional FWD test backcalculation methods are based on the assumption of infinite slab dimensions (Roesler, J, A. et al, 2008). For UTW pavement, the slab size is typically 6 ft. or less. If the loading plate is placed at the center of the slab, some of the sensors would be placed on adjacent slabs.

This study introduced a new method to backcalculate the PCC and equivalent sub-structure properties for in-service UTW projects without the need for the assumption of continuity.

5.2.3.1. Development of UTW pavement's FWD test backcalculation method

The following sections describe the problem for backcalculation of properties of UTW pavement layers, the concept of a new backcalculation approach, its verification through simulations, and comparison of accuracy of this backcalculation method with traditional approaches.

1. Discontinuity in UTW pavement

Due to the relatively thin slab thickness, the size of whitetopping pavement is small. As a result, when conducting FWD tests, some of sensors will be placed on adjacent slab. Therefore, the assumption of continuous slab support is violated for traditional backcalculation method. A new method which accounts for the discontinuity has to be developed.

2. Development of Critical Distance approach

The approach used in this study 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 on which the loading plate was placed 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 30 shows the application of St. Venant's principle to UTW pavement. Therefore, the problem of UTW pavement with small slabs becomes that of an equivalent asphalt pavement for the backcalculation of the modulus of the substructure. The properties of the asphalt pavement 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.

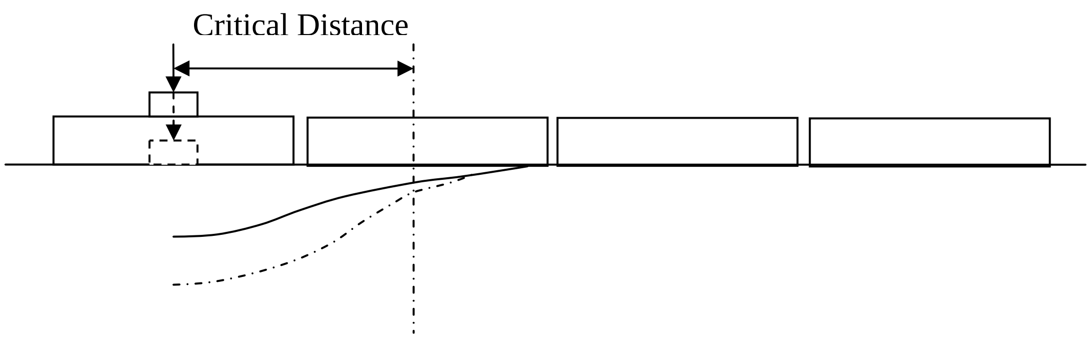

Figure 30. St. Venant's Principle Used in UTW Pavement

The key to this Critical Distance approach 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 semi-infinite substructure are negligible. A 5% tolerance level was used in this study. In this study, deflections were calculated using the "KENSLAB" program (Huang, Yang H. 2004). It was assumed that the UTW overlay was built on the old HMA pavement and the aggregate interlock between joints was small enough after years of traffic repetition. Therefore the stiffness of joint was negligible. 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. Also, for many existing UTW projects, information about the 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.

3. Verification of the Critical Distance Approach

The verification of the Critical Distance approach consisted of the following steps:

1) modeling pavement structures (both UTW and semi-infinite pavements) for simulations,

2) determination of deflections for both pavements,

3) identification of critical distances,

4) backcalculation of modulus of substructure of UTW pavements,

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 31.

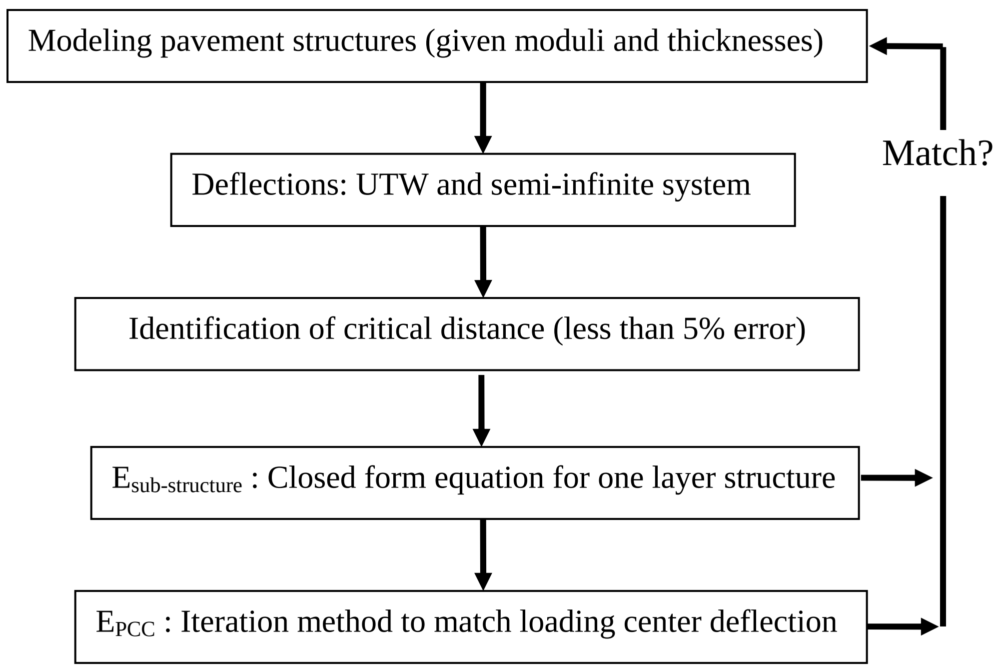

Figure 31. Flow Chart of the Research Procedure

Building Pavement Structures

Pavement structures commonly used in UTW projects were used in this study, as shown in Table 5. Based on the information in Table 5, there are 81 combinations of pavement structures. All the combinations were simulated.

Table 5. Pavement Structure Used in This Study

Equivalent moduli of sub-structure (ksi)	Slab thickness (in.)	PCC modulus (ksi)	Joint spacing (slab size) (ft.)
20	3	3000	4 by 4
50	4	5000	5 by 5
80	5	7000	6 by 6

Calculation of the surface deflection

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, and joint spacing. All the modeling results are provided in Appendix A. Figures 32 through 37 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.

Deflection data analysis

As seen in Figures 32 through 37, beyond 24 in. from the loading center, the deflection differences reduced quickly. Most of the deflection differences were within 5% at 36 in. or farther from the loading center for the pavement structures used in this study. Therefore, 36 in. could be used as the Critical Distance. 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.

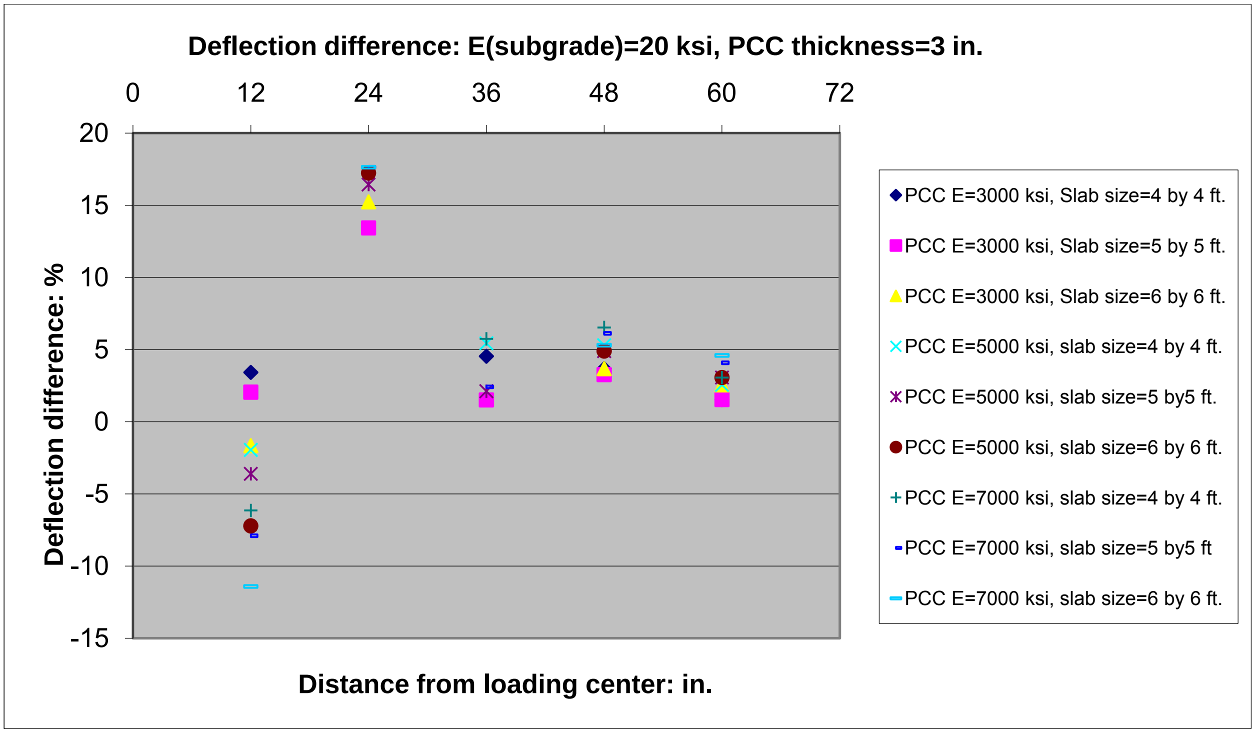

Figure 32. Plots of Deflection Differences for Sub-structure Modulus=20 ksi, PCC Thickness=3 in.

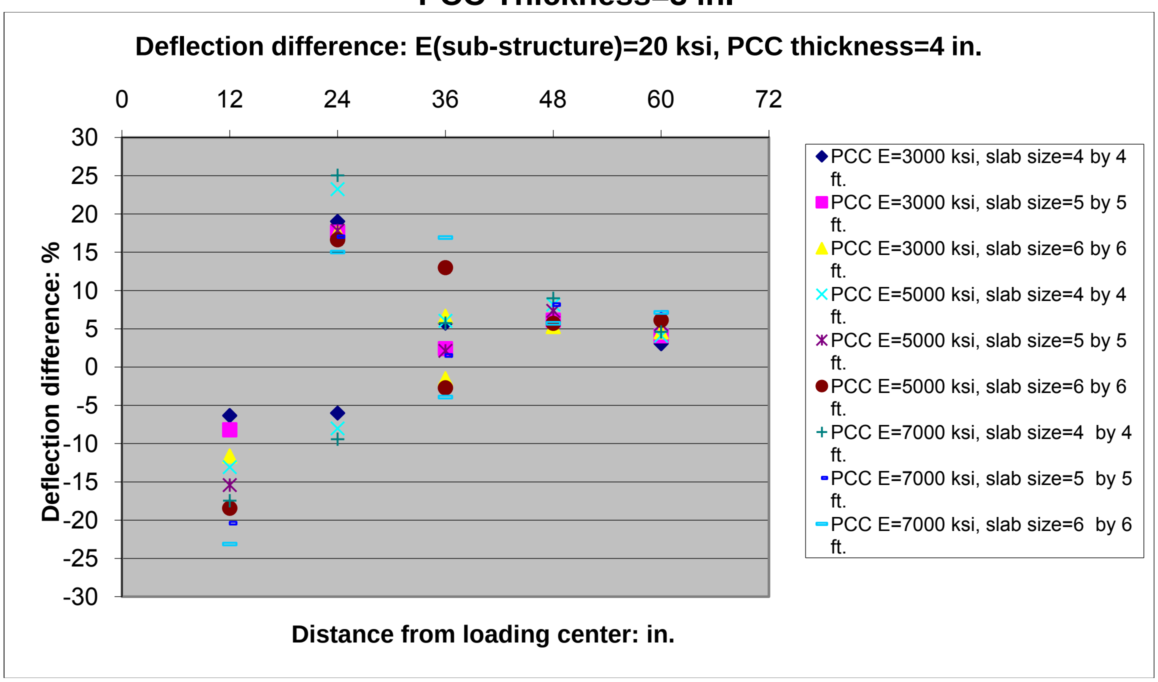

Figure 33. Plots of Deflection Differences for Sub-structure Modulus=20 ksi, PCC thickness=4 in.

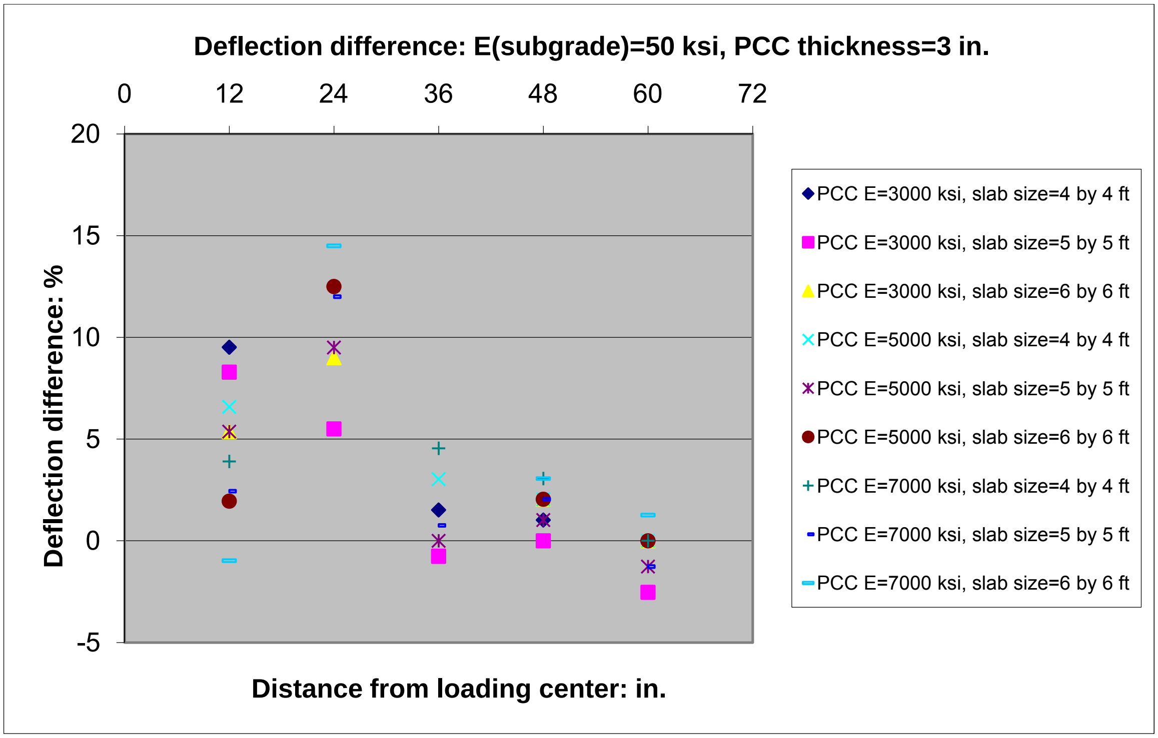

Figure 34. Plots of Deflection Differences for Sub-structure Modulus=50 ksi, PCC thickness=3 in.

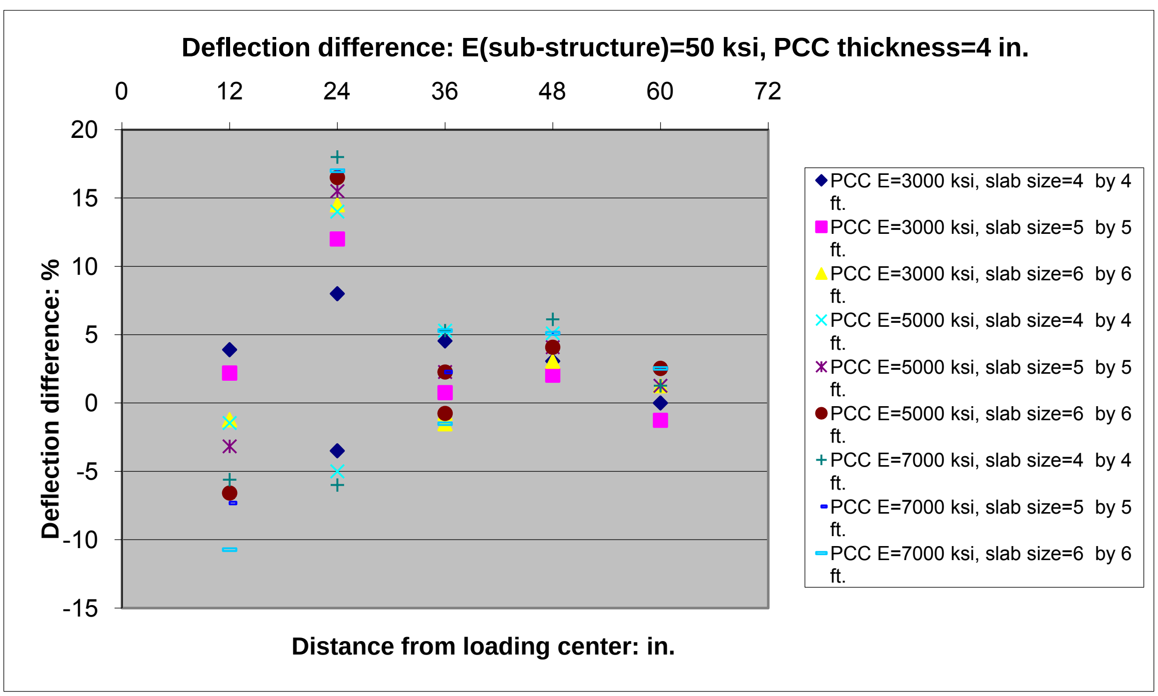

Figure 35. Plots of Deflection Differences for Sub-structure Modulus=50 ksi, PCC thickness=4 in.

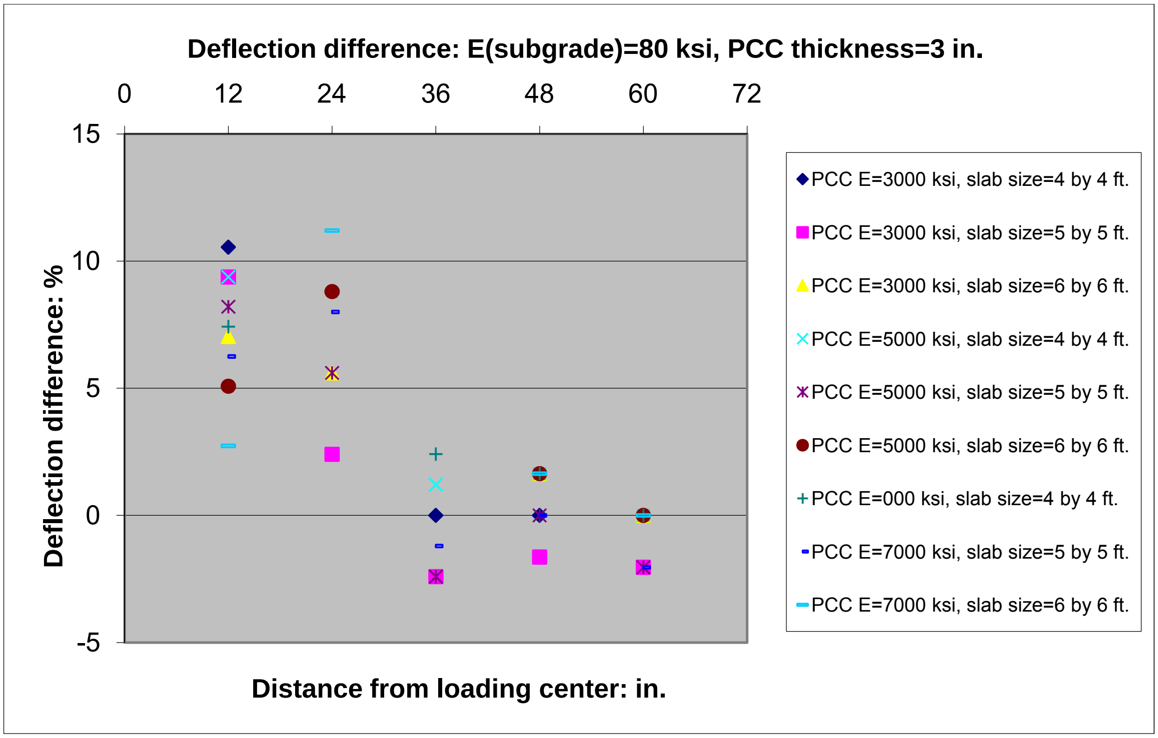

Figure 36. Plots of Deflection Differences for Sub-structure Modulus=80 ksi, PCC thickness=3 in.

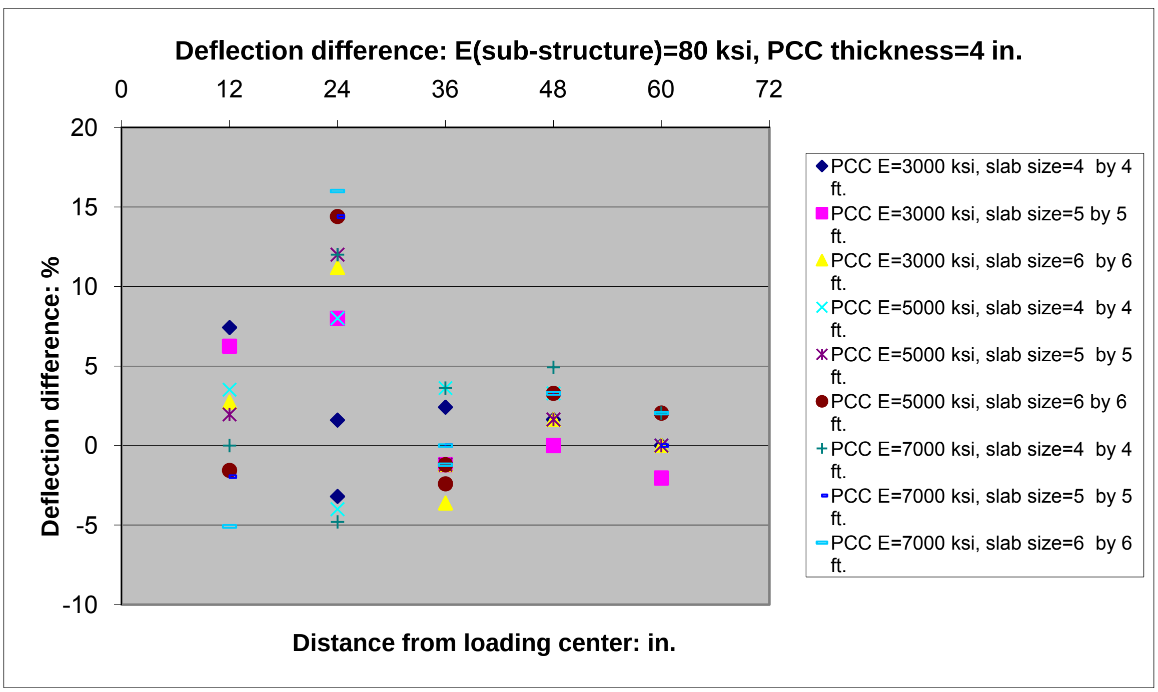

Figure 37. Plots of Deflection Differences for Sub-structure Modulus=80 ksi, PCC thickness=4 in.

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 (Ahlvin, R. G. et al 1962) for single-layer elastic analysis, shown in Equation (2), could be used to backcalculate the sub-structure's composite moduli.

$$D_z = \frac{p(1+\mu)a}{E}\left[\frac{z}{a}A + (1-\mu)H\right] \tag{2}$$

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.

From each of the deflections at 36, 48 and 60 in. from the loading center, one sub-structure equivalent modulus was backcalculated. The average was used as final result. The backcalculated equivalent sub-structure moduli was then compared to the given modulus of the substructure to determine the accuracy of this backcalculation. The backcalculated moduli and their accuracy are shown in Table 6. It can be seen that the Critical Distance Method is effective in determining the modulus of substructure, with an error rate within 5% in most cases.

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, or the discontinuity, 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. Some of the backcalculated PCC moduli and their accuracy are shown in Table 6. The error of backcalculated PCC modulus is within 20% in most cases, which is considered to be acceptable for a concrete pavement backcalculation.

5.2.3.2. Compare with other methods

Based on the deflections obtained from pavement simulations, backcalculations were performed using equations based on the "AREA" theory (Hall, K. T. et. al. 1991), as follows:

$$AREA = 6 * (1 + 2\left(\frac{d12}{d0}\right) + 2\left(\frac{d24}{d0}\right) + \left(\frac{d36}{d0}\right) \quad (3)$$

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

$$l_e = \left[\frac{ln\left(\frac{36 - AREA}{4521.676}\right)}{-3.645} \right]^{\frac{1}{0.187}} \tag{5}$$

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

le = 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,

The Modcomp 6 program (Irwin, L.H. 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 for slab thickness of 3 in. and 4 in..

From Table 6, it can be seen that the accuracy of the Critical Distance Method is within 5% for sub-structure moduli and within 20% for PCC moduli. The accuracy increased with the decrease of slab thickness and slab modulus or with the increase of underlying support. Using the "AREA" theory, relatively accurate (within 10%) sub-structure moduli could be obtained. However, for PCC moduli, the error could be up to 343%. For backcalculation by Modcomp 6, it was noted that the error for the backcalculated moduli of sub-structure is within 15%

whereas the error for backcalculated moduli of PCC could be up to 91.67%. This indicates that both the “AREA” theory and Modcomp 6 are only applicable to traditional concrete pavements and the Critical Distance Method is more accurate for UTW pavements.

Limited simulations were also conducted on slab thickness of 5 in. The simulation results are shown in Table 7. It indicates the critical distance method is applicable to 5 in. whitetopping pavement, too.

Conclusions based on the 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 in UTW pavement FWD test backcalculation. A critical distance of 36 in. from the center of the loading plate is typical for the pavement structures analyzed in this study. The accuracy of backcalculated moduli is within 5% for equivalent sub-structure and 20% for PCC slab from the modeling data. Traditional backcalculation methods, such as the “AREA” theory and Modcomp 6, are fairly accurate in backcalculating the modulus of substructure, but the error for PCC modulus is excessive. Because the small slab thickness of UTW pavement violates the assumptions of traditional approaches, it is demonstrated that the Critical Distance Method is more accurate for UTW pavement evaluation.

Table 6. Back Calculated Sub-structure Moduli, PCC moduli and the Accuracy Using Different Method

Pavement structure	Critical Distance Method				Hall's Equation (based on area theory)				MODCOMP 6			
	Modulus of sub-structure: (ksi)	Difference from real value (%)	**Modulus of PCC: (ksi)**	Difference from real value (%)	**Modulus of sub-structure: (ksi)**	Difference from real value (%)	**Modulus of PCC: (ksi)**	Difference from real value (%)	**Modulus of sub-structure: (ksi)**	Difference from real value (%)	**Modulus of PCC: (ksi)**	Difference from real value (%)
Sub-E=20ksi, slab thickness=3in., PCC E=3000ksi, slab size= 4 by 4	19.375	***-3.13***	3320	***10.67***	18.588	***-7.06***	5556.685	***85.22***	19.5	***-2.50***	3380	***12.67***
Sub-E=20ksi, slab thickness=3in., PCC E=3000ksi, slab size= 6 by 6	19.729	***-1.36***	3130	***4.33***	18.962	***-5.19***	6410.719	***113.69***	19.4	***-3.00***	4290	***43.00***
Sub-E=20ksi, slab thickness=3in., PCC E=7000ksi, slab size= 4 by 4	19.031	***-4.85***	8100	***15.71***	18.883	***-5.59***	9608.448	***37.26***	19.6	***-2.00***	6760	***-3.43***
Sub-E=20ksi, slab thickness=3in., PCC E=7000ksi, slab size= 6 by 6	19.248	***-3.76***	7850	***12.14***	19.401	***-2.99***	11015.791	***57.37***	19.5	***-2.50***	8450	***20.71***
Sub-E=80ksi, slab thickness=3in., PCC E=3000ksi, slab size= 4 by 4	80.034	***0.04***	2980	***-0.67***	68.891	***-13.89***	11442.929	***281.43***	79.5	***13.57***	4330	***44.33***
Sub-E=80ksi, slab thickness=3in., PCC E=3000ksi, slab size= 6 by 6	80.446	***0.56***	2950	***-1.67***	70.565	***-11.79***	13293.557	***343.12***	79.3	***13.29***	5750	***91.67***
Sub-E=80ksi, slab thickness=3in., PCC E=7000ksi, slab size= 4 by 4	78.977	***-1.28***	7280	***4.00***	72.848	***-8.94***	16597.022	***137.10***	79.2	***13.14***	8490	***21.29***
Sub-E=80ksi, slab thickness=3in., PCC E=7000ksi, slab size= 6 by 6	80.446	***0.56***	6850	***-2.14***	74.126	***-7.34***	19122.188	***173.17***	79.2	***13.14***	10700	***52.86***

Table 6. Back Calculated Sub-structure Moduli, PCC moduli and the Accuracy Using Different Method (cont.)

Pavement structure	Critical Distance Method				Hall's Equation (based on area theory)				MODCOMP 6			
	Modulus of sub-structure: (ksi)	Difference from real value (%)	**Modulus of PCC: (ksi)**	Difference from real value (%)	**Modulus of sub-structure: (ksi)**	Difference from real value (%)	**Modulus of PCC: (ksi)**	Difference from real value (%)	**Modulus of sub-structure: (ksi)**	Difference from real value (%)	**Modulus of PCC: (ksi)**	Difference from real value (%)
Sub-E=20ksi, slab thickness=4in., PCC E=3000ksi, slab size= 4 by 4	19.031	***-4.85***	3480	***16.00***	18.891	***-5.54***	4097.379	***36.58***	19.6	***-2.00***	2920	***-2.67***
Sub-E=20ksi, slab thickness=4in., PCC E=3000ksi, slab size= 6 by 6	19.25	***-3.75***	3350	***11.67***	19.403	***-2.99***	4692.984	***56.43***	19.5	***-2.50***	3640	***21.33***
Sub-E=20ksi, slab thickness=4in., PCC E=7000ksi, slab size= 4 by 4	18.795	***-6.02***	8500	***21.43***	18.369	***-8.16***	7993.244	***14.19***	20	***0.00***	5560	***-20.57***
Sub-E=20ksi, slab thickness=4in., PCC E=7000ksi, slab size= 6 by 6	18.836	***-5.82***	8280	***18.29***	19.243	***-3.78***	8911.359	***27.31***	19.9	***-0.50***	7010	***0.14***
Sub-E=80ksi, slab thickness=4in., PCC E=3000ksi, slab size= 4 by 4	78.977	***-1.28***	3120	***4.00***	72.889	***-8.89***	7039.956	***134.67***	79.5	***13.57***	3690	***23.00***
Sub-E=80ksi, slab thickness=4in., PCC E=3000ksi, slab size= 6 by 6	80.446	***0.56***	2950	***-1.67***	74.189	***-7.26***	8166.743	***172.22***	79.4	***13.43***	4670	***55.67***
Sub-E=80ksi, slab thickness=4in., PCC E=7000ksi, slab size= 4 by 4	77.313	***-3.36***	7800	***11.43***	75.444	***-5.70***	11319.296	***61.70***	78.2	***11.71***	7560	***8.00***
Sub-E=80ksi, slab thickness=4in., PCC E=7000ksi, slab size= 6 by 6	78.988	***-1.27***	7250	***3.57***	76.955	***-3.81***	13105.741	***87.22***	78.6	***12.29***	9300	***32.86***

Table 7. Back Calculated Sub-structure Moduli, PCC Moduli and the Accuracy Using Critical Distance Method and Modcomp6 for 5 in. Slab Thickness, 5 by 5 ft. Joint Spacing

Pavement structure	Critical Distance Method				MODCOMP 6			
	Modulus of sub-structure: (ksi)	Difference from real value (%)	Modulus of PCC: (ksi)	Difference from real value (%)	Modulus of sub-structure: (ksi)	Difference from real value (%)	Modulus of PCC: (ksi)	Difference from real value (%)
Sub-E=20ksi, slab thickness=5in., PCC E=3000ksi, slab size= 5 by 5	18.984	-5.08	3450	15.00	19.7	-1.50	2840	-5.33
Sub-E=20ksi, slab thickness=5in., PCC E=7000ksi, slab size= 5 by 5	18.886	-5.57	8260	18.00	20.1	0.50	5470	-21.86
Sub-E=50ksi, slab thickness=5in., PCC E=3000ksi, slab size= 5 by 5	48.690	-2.62	3250	8.33	49.3	-1.40	3320	10.67
Sub-E=50ksi, slab thickness=5in., PCC E=7000ksi, slab size= 5 by 5	47.760	-4.48	7950	13.57	49.3	-1.40	6680	-4.57
Sub-E=80ksi, slab thickness=5in., PCC E=3000ksi, slab size= 5 by 5	79.597	-0.50	3050	1.67	79.9	-0.12	3560	18.67
Sub-E=80ksi, slab thickness=5in., PCC E=7000ksi, slab size= 5 by 5	78.233	-2.21	7600	8.57	79.9	-0.12	7100	1.43

5.2.4. FWD Backcalculation Results

As discussed in section 5.2.1, only CTH “A”, STH 82, and Lawndale Avenue were included in the FWD test backcalculation in this research. Among these 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 was also used for STH 82 which has a slab thickness of 5 in. 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 8 through 10.

Table 8. Backcalculation Results of Lawndale Avenue

Projects	Stations	MODCOMP6 Program		Critical Distance Method	
		$E_{sub-structure}$ (ksi)	E_{PCC} (ksi)	$E_{sub-structure}$ (ksi)	E_{PCC} (ksi)
Lawndale Avenue	0	28.700	533.000	23.749	483.000
	7	25.600	3770.000	18.894	8350.000
	13	25.600	2750.000	21.738	3780.000
	16	33.600	1130.000	26.248	1570.000
	43	26.600	1690.000	26.696	1130.000
	49	38.100	697.000	31.380	650.000
	56	34.800	756.000	30.412	565.000
	62	35.200	645.000	30.033	492.000

Table 9. Backcalculation Results of CTH “A”

Projects	MODCOMP6 Program

	Stations	$E_{sub\text{-}structure}$ (ksi)	E_{PCC} (ksi)	Stations	$E_{sub\text{-}structure}$ (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 10. Backcalculation Results of STH 82

Projects	Stations	MODCOMP6 Program		Critical Distance Method	
		$E_{sub\text{-}structure}$ (ksi)	E_{PCC} (ksi)	$E_{sub\text{-}structure}$ (ksi)	E_{PCC} (ksi)
STH 82	495	20.0	6080	19.124	8950
	502	19.5	1160	17.769	32000
	509	23.7	7570	21.571	13300
	515	23.1	6140	21.863	9000
	6569	16.3	7900	14.445	20500
	6575	21.0	2550	19.914	3180
	6582	19.4	4170	16.746	7630
	6588	22.2	4520	21.088	6200
	6595	19.8	3740	16.464	7350
	9876	22.9	5900	22.887	7300
	9882	23.6	4050	21.534	5950
	9889	23.5	5500	21.79	8300
	9892	22.1	3070	20.777	4020
	9899	21.1	3880	19.664	5500
	15296	21.0	3490	17.491	6570
	15306	21.3	3900	20.461	5050
	15319	24.1	3290	22.856	4170
	15329	22.8	3880	22.913	4350

	15342	23.7	1670	21.026	2280
	21176	14.6	2550	13.166	3900
	21182	5.3	2200	4.006	12800
	21192	19.6	3130	19.381	3620
	21199	9.6	4710	7.929	19150
	21202	18.6	7330	16.588	15600
	26425	26.3	5290	25.748	6600
	26435	30.0	5130	28.638	6660
	26442	29.2	2910	24.313	5050
	26451	30.2	6460	28.645	8900
	26458	33.5	3960	29.184	6200

From Table 8, 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. 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 result from broken slabs on which some of FWD tests were conducted. Photos of Lawndale Avenue are shown in Figure 38, indicating that some slabs were broken.

Figure 38. Photos of Lawndale Avenue

Figure 38. Photos of Lawndale Avenue (continued)

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 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. Of these three projects, CTH “A” had the highest PCC modulus, 8,226.25 ksi with a standard deviation of 3,850.24ksi. CTH “A” has been in service for only one year and is in excellent condition. For STH 82, the average PCC modulus was 5,944.78ksi with a standard deviation of 1,937.86ksi (excluded outliers). STH 82 was in good condition with minimal distresses. 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 report.

The average sub-structure moduli for STH 82, Lawndale Ave, and CTH “A” were 20.97 (excluding outliers), 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. 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 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.

5.2.5. 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 in the 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 conditions, 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 edge or corner of the slab. The maximum tensile stresses, either on the surface or bottom of the slab, were determined. Table 11 shows different combinations of pavement properties and the modeling results using EverFE (WS DOT, 2005), a 3-D Finite Element (FE) program.

Table 11. 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	δ-corner max (psi)	412	359	488	430
Edge	δ-edge max (psi)	**485**	**416**	**575**	**510**
STH 82					
Loading Position	E_{PCC} (ksi)	2000		4000	
	k-value (pci)	300	500	300	500
Corner	δ-corner max (psi)	340	306	376	350
Edge	δ-edge max (psi)	**357**	**326**	**388**	**366**

From Table 11, 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 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 39).

Figure 39. 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 were used, 18, 22, 26 kip (single axle, dual tire), 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 5.2.3.1, in KENSLAB modeling, it was assumed that the load transfer between slabs was 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 12 through 17.

5.2.6. Remaining Fatigue Life Analysis Based on 3-D Finite Modeling

Remaining fatigue life analysis was performed using the fatigue equations recommended by the Portland Cement Association (Packard and Tayabji, 1985), as below:

$$\text{For } \frac{\delta}{S_C} \geq 0.55\text{: } \log N_f = 11.737 - 12.077\left(\frac{\delta}{S_C}\right) \tag{6}$$

$$\text{For } 0.45 < \frac{\delta}{S_C} < 0.55\text{: } N_f = \left(\frac{4.2577}{\delta/S_C - 0.4325}\right)^{3.268} \tag{7}$$

$$\text{For } \frac{\delta}{S_C} \leq 0.45\text{: } N_f = \text{unlimited} \tag{8}$$

Where: N_f is the allowable number of traffic repetitions,

δ is the flexural stress in slab in psi,

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

$$S_C = \frac{43.5E_C}{10^6} + 488.5 \qquad (9)$$

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 also shown in Tables 12 through 14. For Lawndale Ave., the backcalculated PCC moduli for some stations were low, 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, results in unlimited number of loads to carry, which is not reasonable. Therefore, care should be exercised in using the fatigue life for pavement with broken slabs.

From Table 13, 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 crushed aggregate base course (CABC). Higher load might have to be used.

It can be seen 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 loads that are heavier than the standard 18-kip axle loads.

The thermal stresses for each project are shown in Tables 15 through 17. 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.

Table 12. KENSLAB Modeling and Remaining Fatigue Life Analysis Results (Lawndale Ave. 18, 22, 26 kip load)

Load Level	Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress (18kip)	PCC Strength	Stress ratio R	R>0.55	0.45<R<0.55	R<0.45
18 kips	0	3.95	23.749	483.000	110.6	n/a	n/a			broken
	7	3.95	18.894	8350.000	515.6	851.7	0.605	26,673		
	13	3.95	21.738	3780.000	363.4	652.9	0.557	103,592		
	16	3.95	26.248	1570.000	217.6	556.8	0.391			unlimited
	43	3.95	26.696	1130.000	178.5	537.7	0.332			unlimited
	49	3.95	31.380	650.000	112.0	n/a	n/a			broken
	56	3.95	30.412	565.000	103.7	n/a	n/a			broken
	62	3.95	30.033	492.000	94.6	n/a	n/a			broken
	Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress (22kip)	PCC Strength	Stress ratio R	R>0.55	0.45<R<0.55	R<0.45
22 kips	0	3.95	23.749	483.000	135.2	509.5	0.265			broken
	7	3.95	18.894	8350.000	630.1	851.7	0.740	635		
	13	3.95	21.738	3780.000	444.2	652.9	0.680	3,317		
	16	3.95	26.248	1570.000	265.9	556.8	0.478		2,855,784	
	43	3.95	26.696	1130.000	218.2	537.7	0.406			unlimited
	49	3.95	31.380	650.000	136.9	516.8	0.265			broken
	56	3.95	30.412	565.000	126.7	513.1	0.247			broken
	62	3.95	30.033	492.000	115.6	509.9	0.227			broken
	Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress (26kip)	PCC Strength	Stress ratio R	R>0.55	0.45<R<0.55	R<0.45
26 kips	0	3.95	23.749	483.000	159.7	509.5	0.313			broken
	7	3.95	18.894	8350.000	744.7	851.7	0.874	15		
	13	3.95	21.738	3780.000	525.0	652.9	0.804	106		
	16	3.95	26.248	1570.000	314.3	556.8	0.564	83,132		
	43	3.95	26.696	1130.000	257.8	537.7	0.479		2,489,094	
	49	3.95	31.380	650.000	161.8	516.8	0.313			broken
	56	3.95	30.412	565.000	149.7	513.1	0.292			broken
	62	3.95	30.033	492.000	136.6	509.9	0.268			broken

Table 13. 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	11700.000	237.9	997.5	0.239			unlimited
	102	7.50	20.300	12700.000	234.3	1041.0	0.225			unlimited

	118	7.50	26.300	8360.000	204.8	852.2	0.240			unlimited
	135	7.50	19.000	10500.000	228.9	945.3	0.242			unlimited
	148	7.50	14.400	16100.000	259.7	1188.9	0.218			unlimited
	5023	7.50	21.200	5110.000	192.6	710.8	0.271			unlimited
	5043	7.50	16.700	11500.000	238.5	988.8	0.241			unlimited
	5056	7.50	23.600	9400.000	214.7	897.4	0.239			unlimited
	5076	7.50	20.800	13100.000	234.6	1058.4	0.222			unlimited
	5092	7.50	31.800	13200.000	216.5	1062.7	0.204			unlimited
	9873	7.50	34.000	4380.000	164.0	679.0	0.242			unlimited
	9882	7.50	26.800	6970.000	196.0	791.7	0.248			unlimited
	9899	7.50	18.700	14200.000	242.7	1106.2	0.219			unlimited
	9912	7.50	27.700	9360.000	207.5	895.7	0.232			unlimited
	9928	7.50	28.600	8240.000	200.5	846.9	0.237			unlimited
	15093	7.50	40.500	6720.000	175.7	780.8	0.225			unlimited
	15102	7.50	47.500	5260.000	156.8	717.3	0.219			unlimited
	15119	7.50	49.000	4120.000	143.6	667.7	0.215			unlimited
	15132	7.50	54.800	5650.000	153.4	734.3	0.209			unlimited
	15148	7.50	38.800	3800.000	150.9	653.8	0.231			unlimited
	20008	7.50	58.900	3800.000	130.7	653.8	0.200			unlimited
	20024	7.50	32.200	4600.000	168.8	688.6	0.245			unlimited
	20037	7.50	53.800	3700.000	133.8	649.5	0.206			unlimited
	20053	7.50	47.900	4960.000	153.6	704.3	0.218			unlimited

Table 13. KENSLAB Modeling and Remaining Fatigue Life Analysis Results (CTH "A". 18, 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	85	7.50	17.200	11700.000	343.7	997.5	0.345			unlimited
	102	7.50	20.300	12700.000	338.4	1041.0	0.325			unlimited
	118	7.50	26.300	8360.000	295.9	852.2	0.347			unlimited
	135	7.50	19.000	10500.000	330.6	945.3	0.350			unlimited
	148	7.50	14.400	16100.000	375.2	1188.9	0.316			unlimited
	5023	7.50	21.200	5110.000	278.2	710.8	0.391			unlimited
	5043	7.50	16.700	11500.000	344.5	988.8	0.348			unlimited
	5056	7.50	23.600	9400.000	310.1	897.4	0.346			unlimited
	5076	7.50	20.800	13100.000	338.8	1058.4	0.320			unlimited
	5092	7.50	31.800	13200.000	312.7	1062.7	0.294			unlimited
	9873	7.50	34.000	4380.000	236.8	679.0	0.349			unlimited
	9882	7.50	26.800	6970.000	283.1	791.7	0.358			unlimited
	9899	7.50	18.700	14200.000	350.6	1106.2	0.317			unlimited
	9912	7.50	27.700	9360.000	299.7	895.7	0.335			unlimited
	9928	7.50	28.600	8240.000	289.6	846.9	0.342			unlimited
	15093	7.50	40.500	6720.000	253.8	780.8	0.325			unlimited
	15102	7.50	47.500	5260.000	226.5	717.3	0.316			unlimited
	15119	7.50	49.000	4120.000	207.4	667.7	0.311			unlimited
	15132	7.50	54.800	5650.000	221.6	734.3	0.302			unlimited
	15148	7.50	38.800	3800.000	218.0	653.8	0.333			unlimited
	20008	7.50	58.900	3800.000	188.7	653.8	0.289			unlimited
	20024	7.50	32.200	4600.000	243.8	688.6	0.354			unlimited
	20037	7.50	53.800	3700.000	193.2	649.5	0.297			unlimited
	20053	7.50	47.900	4960.000	221.9	704.3	0.315			unlimited

Table 14. KENPAVE 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
18kips	495	5.0	19.124	8950.000	371.3	877.8	0.423			unlimited
	502	5.0	17.769	32000.000	453.8	1880.5	0.241			unlimited
	509	5.0	21.571	13300.000	393.2	1067.1	0.368			unlimited
	515	5.0	21.863	9000.000	360.3	880.0	0.409			unlimited
	6569	5.0	14.445	20500.000	443.6	1380.3	0.321			unlimited
	6575	5.0	19.914	3180.000	268.7	626.8	0.429			unlimited
	6582	5.0	16.746	7630.000	369.0	820.4	0.450			unlimited
	6588	5.0	21.088	6200.000	329.4	758.2	0.434			unlimited
	6595	5.0	16.464	7350.000	367.3	808.2	0.454		29,933,960	
	9876	5.0	22.887	7300.000	337.1	806.1	0.418			unlimited
	9882	5.0	21.534	5950.000	323.4	747.3	0.433			unlimited
	9889	5.0	21.790	8300.000	353.4	849.6	0.416			unlimited
	9892	5.0	20.777	4020.000	288.1	663.4	0.434			unlimited
	9899	5.0	19.664	5500.000	324.6	727.8	0.446			unlimited
	15296	5.0	17.491	6570.000	352.1	774.3	0.455		28,704,325	
	15306	5.0	20.461	5050.000	312.4	708.2	0.441			unlimited
	15319	5.0	22.856	4170.000	282.2	669.9	0.421			unlimited
	15329	5.0	22.913	4350.000	286.2	677.7	0.422			unlimited
	15342	5.0	21.026	2280.000	229.8	587.7	0.391			unlimited
	21176	5.0	13.166	3900.000	330.1	658.2	0.502		707,302	
	21182	5.0	4.006	12800.000	471.8	1045.3	0.451			unlimited
	21192	5.0	19.381	3620.000	284.6	646.0	0.441			unlimited
	21199	5.0	7.929	19150.000	464.1	1321.5	0.351			unlimited
	21202	5.0	16.588	15600.000	421.7	1167.1	0.361			unlimited
	26425	5.0	25.748	6600.000	316.1	775.6	0.408			unlimited
	26435	5.0	28.638	6660.000	306.5	778.2	0.394			unlimited
	26442	5.0	24.313	5050.000	295.3	708.2	0.417			unlimited
	26451	5.0	28.645	8900.000	334.6	875.7	0.382			unlimited
	26458	5.0	29.184	6200.000	297.5	758.2	0.392			unlimited

Table 14. 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
22kips	495	5.0	19.124	8950.000	453.8	877.8	0.517		366,316	
	502	5.0	17.769	32000.000	554.6	1880.5	0.295			unlimited
	509	5.0	21.571	13300.000	480.6	1067.1	0.450			unlimited
	515	5.0	21.863	9000.000	440.3	880.0	0.500		749,600	
	6569	5.0	14.445	20500.000	542.2	1380.3	0.393			unlimited
	6575	5.0	19.914	3180.000	328.4	626.8	0.524		282,930	
	6582	5.0	16.746	7630.000	451.0	820.4	0.550	125,294		
	6588	5.0	21.088	6200.000	402.6	758.2	0.531		221,653	
	6595	5.0	16.464	7350.000	448.9	808.2	0.555	106,969		
	9876	5.0	22.887	7300.000	412.0	806.1	0.511		462,679	
	9882	5.0	21.534	5950.000	395.3	747.3	0.529		237,354	
	9889	5.0	21.790	8300.000	431.9	849.6	0.508		519,710	
	9892	5.0	20.777	4020.000	352.2	663.4	0.531		222,162	
	9899	5.0	19.664	5500.000	396.7	727.8	0.545		143,106	
	15296	5.0	17.491	6570.000	430.4	774.3	0.556	105,651		
	15306	5.0	20.461	5050.000	381.8	708.2	0.539		171,003	
	15319	5.0	22.856	4170.000	344.9	669.9	0.515		397,776	
	15329	5.0	22.913	4350.000	349.8	677.7	0.516		378,198	
	15342	5.0	21.026	2280.000	280.8	587.7	0.478		2,803,296	
	21176	5.0	13.166	3900.000	403.5	658.2	0.613	21,518		
	21182	5.0	4.006	12800.000	576.7	1045.3	0.552	118,584		
	21192	5.0	19.381	3620.000	347.8	646.0	0.538		174,816	
	21199	5.0	7.929	19150.000	567.2	1321.5	0.429			
	21202	5.0	16.588	15600.000	515.5	1167.1	0.442			
	26425	5.0	25.748	6600.000	386.4	775.6	0.498		832,625	
	26435	5.0	28.638	6660.000	374.7	778.2	0.481		2,172,128	
	26442	5.0	24.313	5050.000	360.9	708.2	0.510		493,044	
	26451	5.0	28.645	8900.000	409.0	875.7	0.467		6,779,513	
	26458	5.0	29.184	6200.000	363.7	758.2	0.480		2,454,937	

Table 14. 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
26kips	495	5.0	19.124	8950.000	536.3	877.8	0.611	22,838		
	502	5.0	17.769	32000.000	655.4	1880.5	0.349			unlimited
	509	5.0	21.571	13300.000	568.0	1067.1	0.532		212,257	
	515	5.0	21.863	9000.000	520.4	880.0	0.591	39,364		
	6569	5.0	14.445	20500.000	640.8	1380.3	0.464		8,950,193	
	6575	5.0	19.914	3180.000	388.2	626.8	0.619	18,098		
	6582	5.0	16.746	7630.000	533.0	820.4	0.650	7,777		
	6588	5.0	21.088	6200.000	475.8	758.2	0.628	14,395		
	6595	5.0	16.464	7350.000	530.5	808.2	0.656	6,456		
	9876	5.0	22.887	7300.000	486.9	806.1	0.604	27,657		
	9882	5.0	21.534	5950.000	467.2	747.3	0.625	15,378		
	9889	5.0	21.790	8300.000	510.5	849.6	0.601	30,189		
	9892	5.0	20.777	4020.000	416.2	663.4	0.627	14,450		
	9899	5.0	19.664	5500.000	468.9	727.8	0.644	9,028		
	15296	5.0	17.491	6570.000	508.6	774.3	0.657	6,370		
	15306	5.0	20.461	5050.000	451.3	708.2	0.637	10,982		
	15319	5.0	22.856	4170.000	407.6	669.9	0.608	24,474		
	15329	5.0	22.913	4350.000	413.4	677.7	0.610	23,456		
	15342	5.0	21.026	2280.000	331.9	587.7	0.565	82,481		
	21176	5.0	13.166	3900.000	476.8	658.2	0.724	972		
	21182	5.0	4.006	12800.000	681.5	1045.3	0.652	7,298		
	21192	5.0	19.381	3620.000	411.1	646.0	0.636	11,249		
	21199	5.0	7.929	19150.000	670.3	1321.5	0.507		546,775	
	21202	5.0	16.588	15600.000	609.2	1167.1	0.522		303,349	
	26425	5.0	25.748	6600.000	456.6	775.6	0.589	42,384		
	26435	5.0	28.638	6660.000	442.8	778.2	0.569	73,318		
	26442	5.0	24.313	5050.000	426.5	708.2	0.602	29,080		
	26451	5.0	28.645	8900.000	483.4	875.7	0.552	117,470		
	26458	5.0	29.184	6200.000	429.8	758.2	0.567	77,790		

Table 15. KENSLAB Modeling of Maximum Thermal Tensile Stress (Lawndale Ave)

Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 3 ^{0}F/in. temp. gradient
0	3.95	23.749	483.000	17.7
7	3.95	18.894	8350.000	235.6
13	3.95	21.738	3780.000	130.7
16	3.95	26.248	1570.000	57.5
43	3.95	26.696	1130.000	41.2
49	3.95	31.380	650.000	23.8
56	3.95	30.412	565.000	21.0
62	3.95	30.033	492.000	18.6

Table 16. KENPAVE 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	11700.000	691.1
102	7.50	20.300	12700.000	764.2
118	7.50	26.300	8360.000	556.8
135	7.50	19.000	10500.000	647.7
148	7.50	14.400	16100.000	825.5
5023	7.50	21.200	5110.000	347.2
5043	7.50	16.700	11500.000	677.3
5056	7.50	23.600	9400.000	610.7
5076	7.50	20.800	13100.000	787.1
5092	7.50	31.800	13200.000	852.8
9873	7.50	34.000	4380.000	301.3
9882	7.50	26.800	6970.000	471.5
9899	7.50	18.700	14200.000	816.8
9912	7.50	27.700	9360.000	619.7
9928	7.50	28.600	8240.000	553.4
15093	7.50	40.500	6720.000	462.1
15102	7.50	47.500	5260.000	361.3
15119	7.50	49.000	4120.000	281.3
15132	7.50	54.800	5650.000	387.6
15148	7.50	38.800	3800.000	260.4
20008	7.50	58.900	3800.000	258.0
20024	7.50	32.200	4600.000	316.6
20037	7.50	53.800	3700.000	251.5
20053	7.50	47.900	4960.000	340.3

Table 17. KENPAVE Modeling of Maximum Thermal Tensile Stress (STH 82)

Stations	Hpcc	Es (ksi)	Epcc (ksi)	Stress under 3 ^{0}F/in. temp. gradient
495	5.0	19.124	8950.000	180.5
502	5.0	17.769	32000.000	235.8
509	5.0	21.571	13300.000	225.1
515	5.0	21.863	9000.000	195.7
6569	5.0	14.445	20500.000	184.7
6575	5.0	19.914	3180.000	105.5
6582	5.0	16.746	7630.000	156.4
6588	5.0	21.088	6200.000	160.6
6595	5.0	16.464	7350.000	152.5
9876	5.0	22.887	7300.000	181.8
9882	5.0	21.534	5950.000	158.6
9889	5.0	21.790	8300.000	188.4
9892	5.0	20.777	4020.000	123.7
9899	5.0	19.664	5500.000	145.8
15296	5.0	17.491	6570.000	150.3
15306	5.0	20.461	5050.000	141.5
15319	5.0	22.856	4170.000	131.4
15329	5.0	22.913	4350.000	134.9
15342	5.0	21.026	2280.000	85.8
21176	5.0	13.166	3900.000	100.7
21182	5.0	4.006	12800.000	56.6
21192	5.0	19.381	3620.000	113.0
21199	5.0	7.929	19150.000	109.1
21202	5.0	16.588	15600.000	195.1
26425	5.0	25.748	6600.000	181.9
26435	5.0	28.638	6660.000	191.2
26442	5.0	24.313	5050.000	151.5
26451	5.0	28.645	8900.000	224.5
26458	5.0	29.184	6200.000	184.4

5.3. ANALYSIS BASED ON PAVEMENT DISTRESS SURVEY

5.3.1. Performance Assessment and Analysis

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” for PDI. The PCI and PDI of in-service whitetopping pavements are shown in Table 18, except for Howard Avenue which could not be accessed.

Table 18. Pavement Performance—ASTM PCI and WisDOT PDI

No	Project				ASTM PCI	WISDOT PDI
	County	Road Name	Year	Age		
1	Milwaukee	Galena ST	1995	13	55	65.76
2	Milwaukee	Fond Du Lac Ave	2001	7	58	64.40
3	Kenosha	Washington and 22nd	2001	7	64	25.66
4	Dodge	STH 33 and CTH “A”	2001	7	69	34.10
5	Kenosha	STH 50	2001	7	71	27.57
6	Kenosha	IH94/STH 50 Ramp	1998	10	72	41.73
7	Portage	STH 54	2001	7	74	26.63
8	Washington	Lawndale Ave	1998	10	76	32.11
9	Kenosha	North 39th Avenue	1999	9	78	13.10
10	Taylor	STH 97	1999	9	81	6.73
11	Douglas	USH 2/USH 53	2001	7	82	32.40
12	Waukesha	Duplainville Rd	1999	9	85	6.70
13	Dodge	CTH A	2007	1	89	4.65
14	Adams	STH 82	2001	7	91	7.37
15	Milwaukee	State Street	2000	8	94	7.76

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 40 shows the relationship between PCI and PDI. The regression equation is as follows:

$$(\mathrm{PDI}) = -1.496(\mathrm{PCI}) + 140.04 \qquad (10)$$

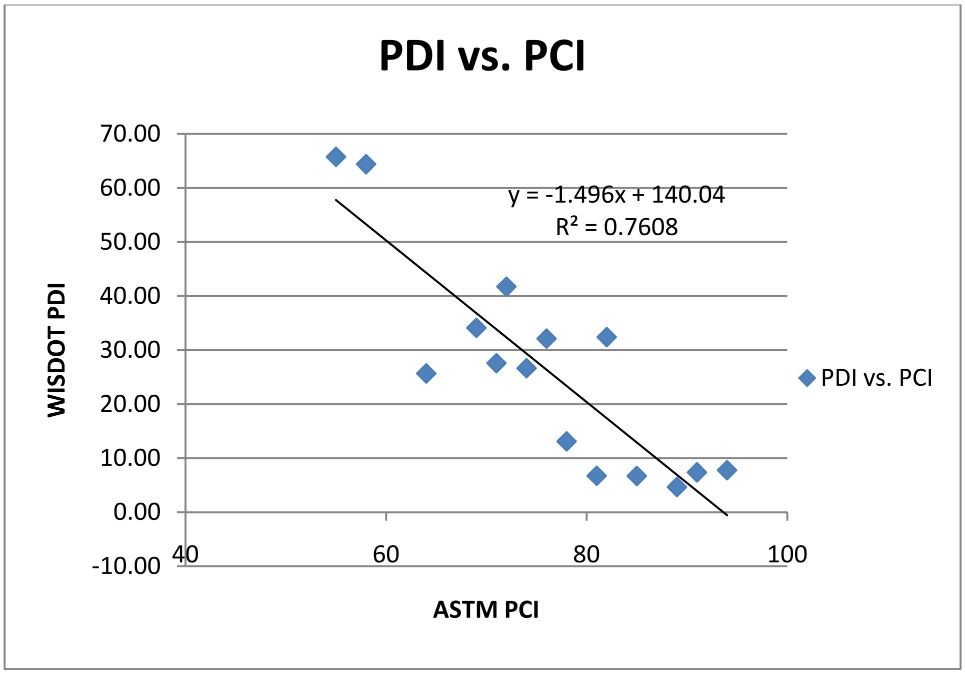

Figure 40. Linear Relationship between ASTM PCI and WisDOT PDI

For projects having cumulative ESALs, Figure 41 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. The average

PCI and PDI of these 15 whitetopping pavements was 75.9 and 26.4 respectively and the average pavement age was 7.9 years.

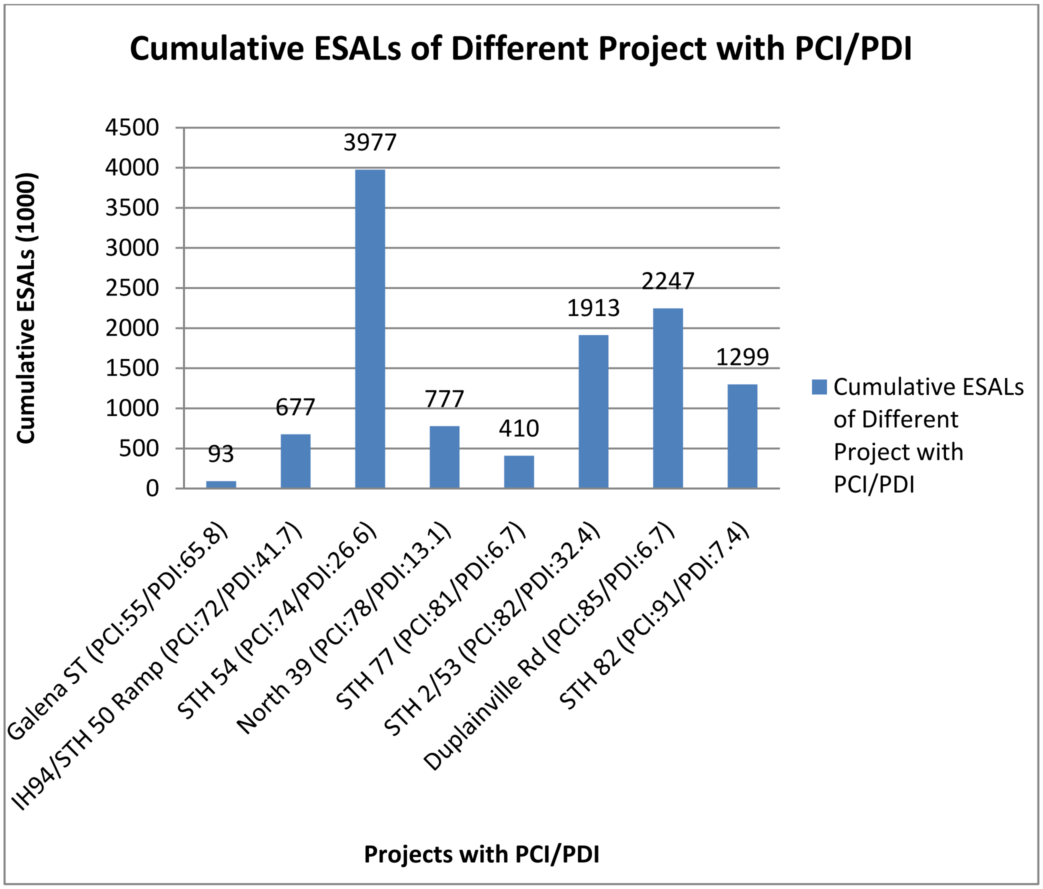

Figure 41. Cumulative ESALs of Different Projects

The relationship between the pavement performance and FWD backcalculated layer properties was also explored. In this study, only 3 projects had backcalculated moduli. Figure 42 and 43 show that the PDI decreases with the increase of the PCC modulus or the decrease of the FWD test center deflections. It should be noted that the average modulus of STH 82 excluded some outliers.

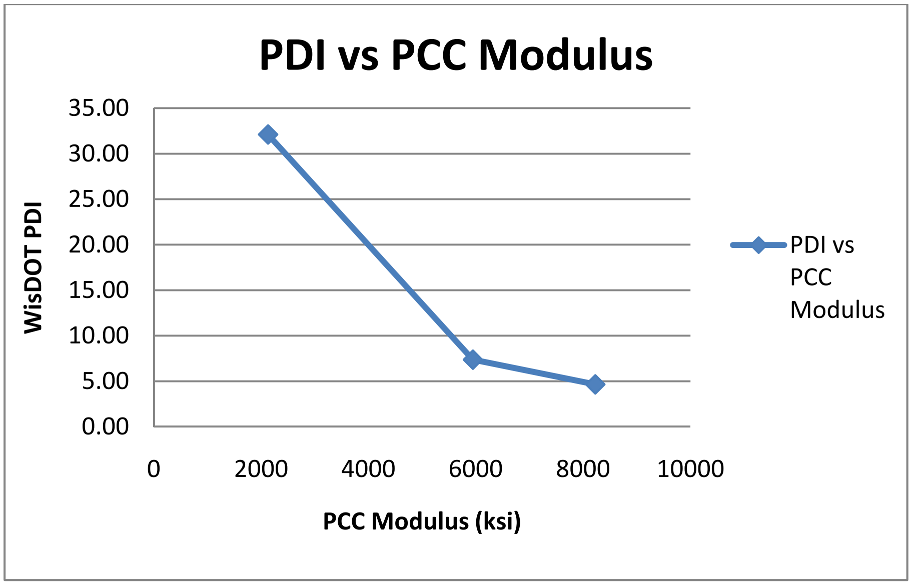

Figure 42. Relationship between WisDOT PDI and PCC Modulus

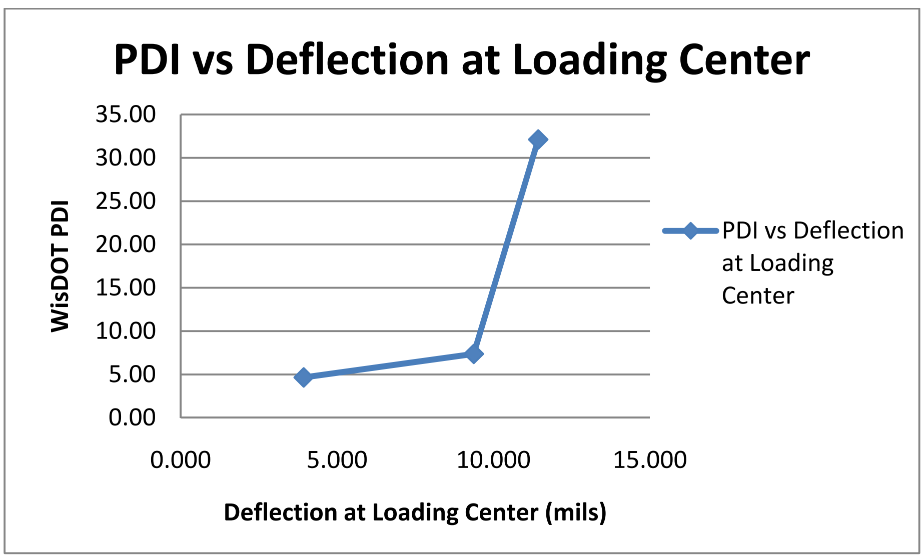

Figure 43. Relationship between WisDOT PDI and FWD Deflection at Loading Center

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 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 19. Prior to the whitetopping, STH 82 was resurfaced with HMA in 1988. STH 54 was repaired between 1992 to 1994, as evidenced by a reduction of the PDI in 1993. Both STH 54 and STH 82 were overlaid with concrete overlay in 2001. It can be seen from Table 19 that the pavement condition of STH 82 (PDI = 51.58), was better than that of STH 54 (PDI =75.50) before whitetopping. The PDI progression rate of STH 54, 3.8 per year, is higher than that of STH 82, 1.05 per year. This proved that the pre-overlay pavement 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 asphalt 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 asphalt overlay and whitetopping pavement.

Table 19. 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

It is to 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 44 shows the transition areas of the IH94/USH50 Ramp. It is suggested that thicker slabs be used in these areas.

Figure 44. Localized Severe Distress at the Entrance (left) and Exit (right) End of IH94/USH50 Ramp

In summary, the whitetopping pavements in Wisconsin show great potential to be a viable rehabilitation method for asphalt pavements. However, it is also noted that these pavements show mixed performance. This could be attributed to the design method used, such as bond condition and load levels as discussed previously. It is recommended that WisDOT develop a whitetopping pavement design method applicable for the State of Wisconsin.

5.3.2. Comparison of Performance with Other States

The performance of whitetopping projects in Wisconsin were compared to those in other states from the literature review. PCI was used as the pavement performance indicator in the literature review.

The performance of whitetopping pavement in other states was obtained from two publications, "Ultrathin Whitetopping in California and Nevada: A 13-Year Performance Perspective of Performance Based on Joint Spacing, Thickness, and Traffic Loading" (Akers D.J. and Warren R. 2005) which focused on the whitetopping projects in California and Nevada, and "Performance of Ultrathin Whitetopping Roadways." (Cole, L.W. 1999) which focused on projects in Tennessee and Georgia. Table 20 shows the PCI and pavement age of the WT or UTW projects in other states.

Figure 45 shows the number of whitetopping projects at different pavement ages for these three research projects. It can be seen that the whitetopping pavements in Wisconsin have service lives comparable to those in Tennessee and Georgia. It is noted that most of the projects included in California and Nevada were parking lots or at schools or churches which would not experience much traffic.

Table 20. PCI of Whitetopping Projects in Different Research

State	No	Project				ASTM PCI
		County	Road Name	Age	Cumulative ESALs	
Projects in Wisconsin	1	Dodge	CTH A	1		89
	2	Waukesha	Duplainville Rd	9	2,247,490	85
	3	Milwaukee	Fond Du Lac Ave	7		58
	4	Milwaukee	Galena ST	13	92,738	55
	5	Kenosha	IH94/STH 50 Ramp	10	676,528	72
	6	Washington	Lawndale Ave	10		76
	7	Kenosha	North 39th Avenue	9	777,450	78
	8	Milwaukee	State Street	8		94
	9	Dodge	STH 33 and CTH "A"	7		69
	10	Kenosha	STH 50	7		71
	11	Portage	STH 54	7	3,977,040	74
	12	Adams	STH 82	7	1,299,400	91
	13	Taylor	STH 97	9	409,900	81
	14	Douglas	USH 2/USH 53	7	1,912,600	82
	15	Kenosha	Washington and 22nd	7		64
Projects in California and Nevada	1	Palm Springs Pavillion		5	5,430	98
	2	Alpine Community Church		10	5,989	98
	3	Bobby Duke Middle School		10	6,177	94
	4	Palm Springs Library		5	7,959	96
	5	Mizel Senior Center		8	8,584	96
	6	Valley View Elementary		10	10,272	97
	7	Peter Pendleton School		6	11,633	97
	8	John Kelly School		7	12,495	99
	9	LA County Fairground, South Road		5	18,945	96
	10	Fifth Avenue at Marine Street		7	20,027	98
	11	Poway Mortuary		13	25,900	80
	12	Indio Heights Center		11	31,728	91
	13	Alamo Truch Stop		2	52,603	99
	14	LA County Fairground, Main Gate		3	58,002	97
	15	Charleston Bus Stop, WB		1	132,708	97
	16	Charleston Bus Stop, EB		1	132,708	94
	17	Spring Mountain and Valley View		2	1,597,575	94
	18	Sunset at Boulder Highway		4	1,680,001	96
projects in Tennessee and Georgia	1	Belvoir Ave.		5	101,000	80
	2	Green St.		5	311,000	89
	3	STH 56		4	381,000	83
	4	Concorde St.		6	132,000	95
	5	Cruisick St. (outside)		5	302,000	89
	6	Cruisick St. (inside)		5	201,000	66
	7	I-85 Weigh Station (approach)		5	650,000	97
	8	I-85 Weigh Station (leave)		5	650,000	74
	9	Wesley Chapel Rd.		5	347,000	n/a
	10	Marbut Rd.		5	92,000	88

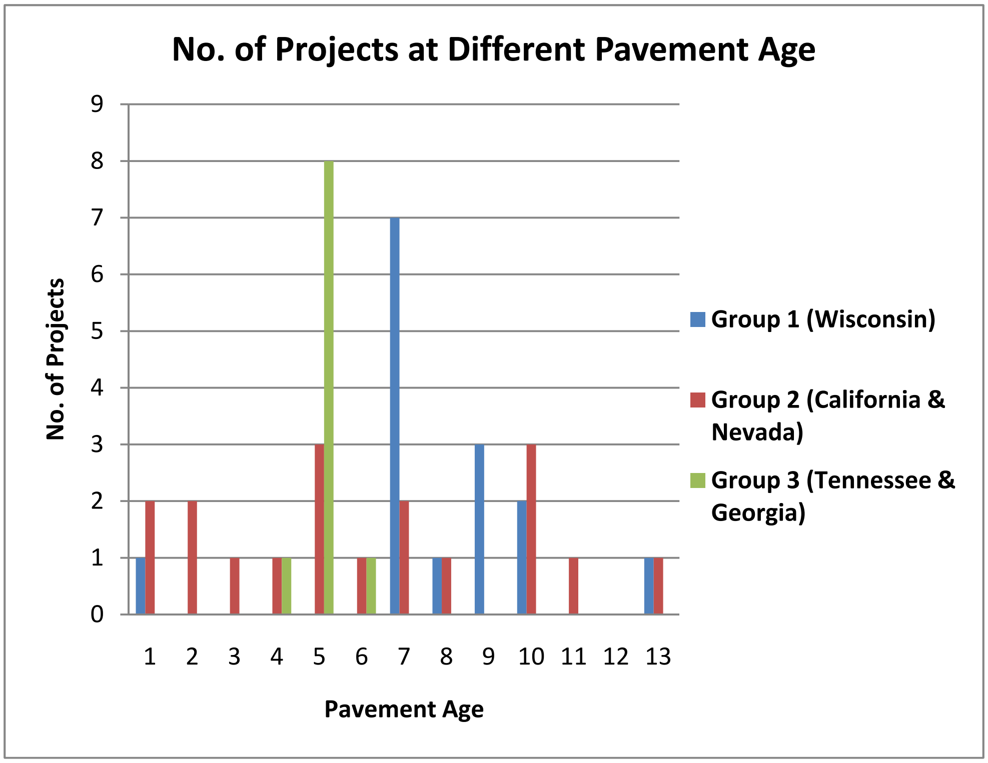

Figure 45. Number of Projects at Different Pavement Age for 3 Groups

Illinois, Minnesota, and Michigan DOTs, which have similar climatic conditions to Wisconsin, also published the performance of whitetopping pavements (Winkelman, 2005,Burnham, 2005,Eacker, 2004) which have similar climatic conditions to Wisconsin.

Three UTW pavements were built on an interstate highway in 1997 in Minnesota. In 2004 the pavements deteriorated significantly after approximately 6,000,000 ESALs in the driving lane. No PCI was provided in the research.

In Michigan, 4 WT or UTW test sections were built in 1999. After three years, less than 5% of the panels experienced some kinds of distress. Most of the distresses were cracking.

In Illinois, 9 whitetopping projects were evaluated. The ESALs and percentage of cracked panels was summarized in Table 21.

Table 21. ESALs and Percentage of Cracked Panels of Whitetopping Pavement in Illinois

Projects	Estimated ESALs	Percentage of Cracked Panels (%)
U.S. Highway 36 and Oakland Avenue	1,000,000	18.8
U.S. Highway 36 and Country Club Road	1,360,000	71.2
U.S. Highway 51 and Pleasant Hill Road	290,000	13.0
U.S. Highway 36 (Tuscola)	720,000	6.1
Clay County Highway 3	60,000	0.0
Piatt County Highway 4	80,000	1.0
Cumberland County Highway 2	220,000	0.3
U.S. Highway 45	660,000	2.9
Illinois Route 13	530,000	9.6

Because PCI values were not provided in these three studies, a direct comparison could not be performed.

5.4. STATISTICAL ANALYSIS AND RESULTS

5.4.1. Survival Analysis

Survival analysis was performed using SPSS. However, at this point, only two whitetopping projects are out of service. Therefore, the survival analysis did not obtain the survival lives of whitetopping pavements.

5.4.2. Factorial Analysis

Based on the data collected, statistical analysis was conducted to identify the design and construction factors that affect the performance of 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 for 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 an 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 22 and 23 show the results of the statistical analysis, based on the PCI and the PDI, respectively.

Table 22. 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 22. Statistical Test Results of the effects on PCI (Continued)

Estimated Marginal Means

1. pcc low / high <=4

Dependent Variable:Performance PCI

pcc low / high <=4	Mean	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
1.00	64.992	2.349	59.438	70.547
2.00	92.813	3.265	85.092	100.534

2. joint spacing <= 6 by 6 or not

Dependent Variable:Performance PCI

joint spacing <= 6 by 6 or not	Mean	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
1.00	86.787	3.265	79.066	94.508
2.00	71.018	2.591	64.891	77.145

3. Period in service

Dependent Variable:Performance PCI

Period in service	Mean	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
2.00	82.974	5.130	70.843	95.105
8.00	70.705	2.188	65.532	75.878
10.00	82.013	2.565	75.947	88.078
11.00	82.026	5.130	69.895	94.157
13.00	76.795	6.307	61.881	91.708

Table 23. 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)

Estimated Marginal Means

1. pcc low / high <=4

Estimates

Dependent Variable:Performance PDI

pcc low / high <=4	Mean	Std. Error	95% Confidence Interval	
			Lower Bound	Upper Bound
1.00	41.860	7.222	24.782	58.937
2.00	7.738	10.039	-16.001	31.477

From the statistical analysis result, based on a 0.05 significance level, the PCI is significantly higher for whitetopping overlay with a thickness of more than 4 in. (group 2) than less or equal to 4 in. (group 1) . The PCI is significantly lower for whitetopping overlay with slab sizes more than 36 sq. ft. (group 2) than less or equal to 36 sq. ft. (group 1). However pavement age has a P-value of 0.51. The PDI appears to be significantly affected by only whitetopping overlay thickness. The pavement performance is significantly affected by whitetopping overlay thickness and joint spacing. Performance is better for whitetopping pavement that has a slab thickness “more than 4 in.” than “less or equal to 4 in.”. Performance is better for whitetopping pavement that has a slab size “less or equal to 36 sq. ft.” than “more than 36 sq. ft.”. The performance is almost (0.51>0.5) significantly affected by pavement age.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

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

6.1. CONCLUSIONS

(1) Based on the literature review, whitetopping and ultra-thin whitetopping have gained popularity in the last twenty years. The condition of the existing asphalt pavement is important. A good bond between the PCC overlay and the existing HMA is recommended. Following proper whitetopping design and construction practices is recommended to create a whitetopping pavement that will perform according to the need of the agencies. The condition of existing HMA is not considered in ACPA design procedure.

(2) As of 2008, there have been a total of 18 projects that could be defined as whitetopping in Wisconsin. The projects were built from 1995 to 2007. Slab thicknesses range from 4 in. to 9 in. and joint spacing range from 4 ft. by 4 ft. to 15 ft. by 15 ft. Eleven of the projects are UTW projects. The two most commonly used joint spacings are 4 ft. by 4 ft. and 6 ft. by 6 ft. Fiber was used in 13 projects and only 3 projects used dowel bars.

(3) 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 of whitetopping should be based on an unbonded condition, to be safe.

(4) 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.

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

(6) Critical loading position depends on the pavement structure and slab layout. Thermal stress has little effect for typical UTW overlay due to the relatively short joint spacing and thin slab thickness. 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.

(7) 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. Design of whitetopping should be based on heavier loads than the 18-kip standard axle load.

(8) The performance of the whitetopping projects in Wisconsin is comparable to that in other states.

(9) 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” and slab size should be smaller than 36 sq. ft.

(10) 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) It is recommended that a design method should be developed to reduce the variation of performance of whitetopping pavements in Wisconsin.

(2) The design method should be based on an unbonded condition to be conservative.

(3) 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.

(4) It is recommended that the Mechanistic-Empirical Pavement Design Guide (MEPDG) could be calibrated based on the performance of whitetopping pavements in Wisconsin. Alternatively, the current ACPA design method can be modified as a simplified design approach, after accounting for bond condition, load level and condition of HMA.

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

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APPENDIX: A

Table A.1. Deflections and Deflection Difference for 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 for 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 for 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 for 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 for 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 for 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 for E(substructure)=80ksi, PCC thickness=3 in.

Sub-structure under WT slab with equivalent modulus: E=80ksi							
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	0.00998	0.00507	-49.19839679	0.00497	-50.2004008	0.00472	-52.7054108
12	0.00256	0.00283	10.546875	0.0028	9.375	0.00274	7.03125
24	0.00125	Joint		0.00128	2.4	0.00132	5.6
36	0.00083	0.00083	0	0.00081	-2.40963855	Joint	
48	0.00061	0.00061	0	0.0006	-1.63934426	0.00062	1.639344262
60	0.00049	0.00049	0	0.00048	-2.04081633	0.00049	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	0.00998	0.00458	-54.10821643	0.0045	-54.9098196	0.00429	-57.0140281
12	0.00256	0.0028	9.375	0.00277	8.203125	0.00269	5.078125
24	0.00125	Joint		0.00132	5.6	0.00136	8.8
36	0.00083	0.00084	1.204819277	0.00081	-2.40963855	Joint	
48	0.00061	0.00062	1.639344262	0.00061	0	0.00062	1.639344262
60	0.00049	0.00049	0	0.00048	-2.04081633	0.00049	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	0.00998	0.00426	-57.31462926	0.00419	-58.0160321	0.004	-59.9198397
12	0.00256	0.00275	7.421875	0.00272	6.25	0.00263	2.734375
24	0.00125	Joint		0.00135	8	0.00139	11.2
36	0.00083	0.00085	2.409638554	0.00082	-1.20481928	Joint	
48	0.00061	0.00062	1.639344262	0.00061	0	0.00062	1.639344262
60	0.00049	0.00049	0	0.00048	-2.04081633	0.00049	0

Table A.8. Deflections and Deflection Difference for E(substructure)=80ksi, PCC thickness=4 in.

Equivalent sub-structure modulus: E=80ksi							
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	0.00998	0.00425	-57.41	0.00418	-58.12	0.00398	-60.12
12	0.00256	0.00275	7.42	0.00272	6.25	0.00263	2.73
24	0.00125	Joint		0.00135	8.00	0.00139	11.20
36	0.00083	0.00085	2.41	0.00082	-1.20	Joint	
48	0.00061	0.00062	1.64	0.00061	0.00	0.00062	1.64
60	0.00049	0.00049	0.00	0.00048	-2.04	0.00049	0.00
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	0.00998	0.00378	-62.12	0.00372	-62.73	0.00355	-64.43
12	0.00256	0.00265	3.52	0.00261	1.95	0.00252	-1.56
24	0.00125	Joint		0.0014	12.00	0.00143	14.40
36	0.00083	0.00086	3.61	0.00082	-1.20	Joint	
48	0.00061	0.00063	3.28	0.00062	1.64	0.00063	3.28
60	0.00049	0.00049	0.00	0.00049	0.00	0.0005	2.04
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	0.00998	0.0035	-64.93	0.00343	-65.63	0.00328	-67.13
12	0.00256	0.00256	0.00	0.00251	-1.95	0.00243	-5.08
24	0.00125	Joint		0.00143	14.40	0.00145	16.00
36	0.00083	0.00086	3.61	0.00083	0.00	Joint	
48	0.00061	0.00064	4.92	0.00063	3.28	0.00063	3.28
60	0.00049	0.0005	2.04	0.00049	0.00	0.0005	2.04

Table A.9. Deflections and Deflection Difference for E(substructure)=80ksi, PCC thickness=5 in.

Sub-structure under WT slab with equivalent modulus: E=80ksi							
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	0.00998	0.00365	-63.42685371	0.00358	-64.1282565	0.00342	-65.7314629
12	0.00256	0.00261	1.953125	0.00257	0.390625	0.00248	-3.125
24	0.00125	Joint		0.00141	12.8	0.00144	15.2
36	0.00083	0.00086	3.614457831	0.00083	0	Joint	
48	0.00061	0.00063	3.278688525	0.00062	1.639344262	0.00063	3.278688525
60	0.00049	0.00049	0	0.00049	0	0.0005	2.040816327
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	0.00998	0.00322	-67.73547094	0.00316	-68.3366733	0.00302	-69.739479
12	0.00256	0.00246	-3.90625	0.00241	-5.859375	0.00233	-8.984375
24	0.00125	Joint		0.00144	15.2	0.00146	16.8
36	0.00083	0.00087	4.819277108	0.00083	0	Joint	
48	0.00061	0.00064	4.918032787	0.00063	3.278688525	0.00064	4.918032787
60	0.00049	0.0005	2.040816327	0.0005	2.040816327	0.0005	2.040816327
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	0.00998	0.00296	-70.34068136	0.0029	-70.9418838	0.00278	-72.1442886
12	0.00256	0.00235	-8.203125	0.00229	-10.546875	0.00222	-13.28125
24	0.00125	Joint		0.00146	16.8	0.00146	16.8
36	0.00083	0.00087	4.819277108	0.00083	0	Joint	
48	0.00061	0.00065	6.557377049	0.00064	4.918032787	0.00064	4.918032787
60	0.00049	0.0005	2.040816327	0.0005	2.040816327	0.00051	4.081632653

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