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Extending Asphalt Pavement Life Using Thin Whitetopping

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16. Abstract (Limit: 200 words) <p>Due to budget constraints, many highway agencies are becoming interested in pavement preservation or rehabilitation rather than reconstruction to ensure pavement is in serviceable condition. Thin whitetopping (TWT) is the process of rehabilitating distressed asphalt concrete (AC) pavements by using a concrete overlay. This study was done to develop a design catalog for existing AC pavements to be overlaid with TWT. The finite element (FE) analysis was performed with SolidWorks, which is the 3-D FE software program used to develop this design catalog. The design considered different TWT thicknesses, existing AC layer thickness and modulus, the bonding condition between TWT and the existing AC layer, shoulder conditions and temperature differentials. Each model was built as a three-layer pavement system—composed of concrete (TWT), asphalt layer, and subgrade soil. The traffic load was modeled as a constant pressure with a rectangular area applied at the surface and with intensity equal to the tire inflation pressure of 100 psi. The expected lives of TWT overlays were estimated using fatigue equations developed by the Portland Cement Association (PCA).</p> <p>Results obtained from this study show that interface bonding condition is the most important factor affecting the behavior of TWT. With the increase of TWT thickness or existing AC thickness or AC modulus and the addition of a paved shoulder, concrete tensile stress decreases. Curling stress increases with the increase of TWT thickness and is not a function of AC properties. A design catalog was developed in terms of service life of the pavement. Unlike the unbonded TWT with an unpaved shoulder that results in catastrophic loss of rehabilitated pavement life, bonded TWT is expected to last 10 years, assumed in design. Thus, proper bonding must be ensured in order to have extended pavement life.</p>			
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Abstract

Due to budget constraints, many highway agencies are becoming interested in pavement preservation or rehabilitation rather than reconstruction to ensure pavement is in serviceable condition. Thin whitetopping (TWT) is the process of rehabilitating distressed asphalt concrete (AC) pavements by using a concrete overlay. This study was done to develop a design catalog for existing AC pavements to be overlaid with TWT. The finite element (FE) analysis was performed with SolidWorks, which is the 3-D FE software program used to develop this design catalog. The design considered different TWT thicknesses, existing AC layer thickness and modulus, the bonding condition between TWT and the existing AC layer, shoulder conditions and temperature differentials. Each model was built as a three-layer pavement system—composed of concrete (TWT), asphalt layer, and subgrade soil. The traffic load was modeled as a constant pressure with a rectangular area applied at the surface and with intensity equal to the tire inflation pressure of 100 psi. The expected lives of TWT overlays were estimated using fatigue equations developed by the Portland Cement Association (PCA).

Results obtained from this study show that interface bonding condition is the most important factor affecting the behavior of TWT. With the increase of TWT thickness or existing AC thickness or AC modulus and the addition of a paved shoulder, concrete tensile stress decreases. Curling stress increases with the increase of TWT thickness and is not a function of AC properties. A design catalog was developed in terms of service life of the pavement. Unlike the unbonded TWT with an unpaved shoulder that results in catastrophic loss of rehabilitated pavement life, bonded TWT is expected to last 10 years, assumed in design. Thus, proper bonding must be ensured in order to have extended pavement life.

CHAPTER 1 Introduction

1.1 Introduction

The extensive U.S. highway network, the economic backbone of the country, allows for almost 3 trillion miles of travel annually (TRIP 2010). But the condition of this network is deteriorating due to a limitation of funds to maintain it in good condition. As a result, 32% of the major roads are in poor or mediocre condition, which is responsible for \$67 billion in vehicle repair and operating costs. The American Reinvestment and Recovery Act of 2009 provided \$27 billion for highway projects. But about \$166 billion is needed annually to invest in highways and bridges to improve the condition and performance, while more than half of this amount is needed for pavement preservation (AASHTO 2010). Routine, preventive and corrective maintenance fall in the pavement preservation strategy. Due to budget constraints, pavements are left to deteriorate without timely maintenance treatment and, as such, become candidates for rehabilitation later on.

1.2 Asphalt Pavement Preservation Treatments

Both rigid and flexible pavement must withstand wheel loads, environmental effects, and temperature variations. Asphalt pavements deteriorate as a result of a combination of environmental and wheel load-related factors. Load-related stresses develop fatigue cracking and rutting, whereas environmental factors induce thermal cracking, block cracking, and weathering and raveling (Uzarowski and Bashir 2007). Due to these cracks, pavement loses load-carrying capacity and becomes more susceptible to adverse conditions leading to deterioration.

Asphalt pavement preservation works are divided into three major categories (Uzarowski and Bashir 2007):

- | | |
|---|---|
| <ul style="list-style-type: none"> 1. Routine Maintenance Treatment: <ul style="list-style-type: none"> i. Crack filling/ sealing ii. Patching | <ul style="list-style-type: none"> c. Micro surfacing d. Metro-mat e. Nova chip |
| <ul style="list-style-type: none"> 2. Preventive Maintenance Treatment: <ul style="list-style-type: none"> i. Fog seals ii. Surface rejuvenating iii. Micro-milling iv. Thin surfacing (a-f): <ul style="list-style-type: none"> a. Chip seal b. Slurry seal | <ul style="list-style-type: none"> f. Thin hot-mix asphalt overlays <ul style="list-style-type: none"> 3. Corrective Maintenance: <ul style="list-style-type: none"> i. Full-depth patching ii. Milling iii. Overlays |

Major rehabilitation treatments for asphalt pavements are (Uzarowski and Bashir 2007) (1) structural overlays; (2) recycling, either hot or cold in-place recycling (HIR or CIR) with emulsion or foamed asphalt, or full-depth reclamation (FDR) with foamed asphalt; and (3) reconstruction.

Rehabilitation with structural overlays contributes to the structural capacity of pavements. Structural overlays used for asphalt pavements are as follows: asphalt concrete, whitetopping and ultra-thin whitetopping.

Asphalt pavement can be overlaid with asphalt concrete (AC) or with a concrete layer. Whitetopping and ultra-thin whitetopping are the concrete overlays placed on distressed asphalt pavements.

Life-cycle cost (LCC) analysis of whitetopping and AC overlays were conducted by Lowey (2005). From the analysis, it was found that the initial cost of whitetopping is much higher than an AC overlay. When user cost with initial cost is considered however,

whitetopping is cheaper than AC overlay as whitetopping requires less maintenance during its service life. Moreover, concrete has a much higher albedo value, 0.35-0.4 for new concrete and 0.2-0.3 for most old concrete, than AC, which is 0.05-0.1 for new asphalt and 0.10-0.15 for aged asphalt (ACPA 2002). Albedo is defined as the ratio of reflected solar radiation to incoming solar radiation on the surface. Higher albedo values decrease absorbed heat preventing the formation of a heat island (HI). Heat islands are areas that are hotter than neighboring rural areas and influence energy demand, air and water quality, and warming in the city area. Thus, whitetopping may become a more cost-effective and greener rehabilitation alternative for AC overlay.

1.3 Problem Statement

There are a number of design procedures for whitetopping developed by various agencies: AASHTO (AASHTO 1993); the state of Colorado (Sheehan et al. 2004; Tarr et al. 1998); Portland Cement Association (PCA)/American Concrete Pavement Association (ACPA) (Wu et al. 1998); the state of New Jersey (Nenad et al. 1998); the modified ACPA (Riley et al. 2005); the state of Illinois (Roesler et al., 2008); and the state of Texas (Chul et al. 2008). Among these, only the AASHTO, Colorado and Texas procedures offer guidelines for thin and conventional whitetopping, whereas the other procedures are for ultra-thin whitetopping. All these procedures deal with slab thickness, support characteristics, pre-overlay preparation, and slab dimensions. No procedure provides guidelines for whitetopping responses as it concerns bonding conditions between whitetopping and the existing AC layer, existing AC layer modulus, AC thicknesses, and shoulder conditions.

1.4 Objectives of the Study

The objective of this study was to assess the behavior of 5 in., 6 in., and 7.5 in. thin whitetoppings on existing 5 in., 7 in., and 9 in. AC pavements for different bonding conditions with the AC layer, shoulder conditions, and existing AC moduli. Whitetopping responses for different temperature differentials were also assessed. Based on the behavior of TWT, service lives were calculated for different truck traffic, and a design catalog was developed for AC pavements overlaid with a thin whitetopping.

1.5 Organization of the Report

The report is divided into five chapters. Chapter 1 is the introduction to the study. Chapter 2 is the overview of thin whitetopping. Chapter 3 describes the methodology, and Chapter 4 contains the results of this study. Finally, conclusions of this study and recommendations for further study are presented in Chapter 5.

CHAPTER 2 Literature Review

2.1 Introduction

Budget constraints are forcing many state transportation agencies to look at pavement preservation or rehabilitation rather than reconstruction to ensure pavements are in serviceable condition. Figure 2.1 shows a pavement condition curve over time and the actions needed to retain the pavement in a desirable condition. Routine maintenance, preventive maintenance, and minor rehabilitation are preservation actions which extend the service life of pavement and defer the need for major rehabilitation. Major rehabilitation is undertaken when the pavement is in need of structural improvement.

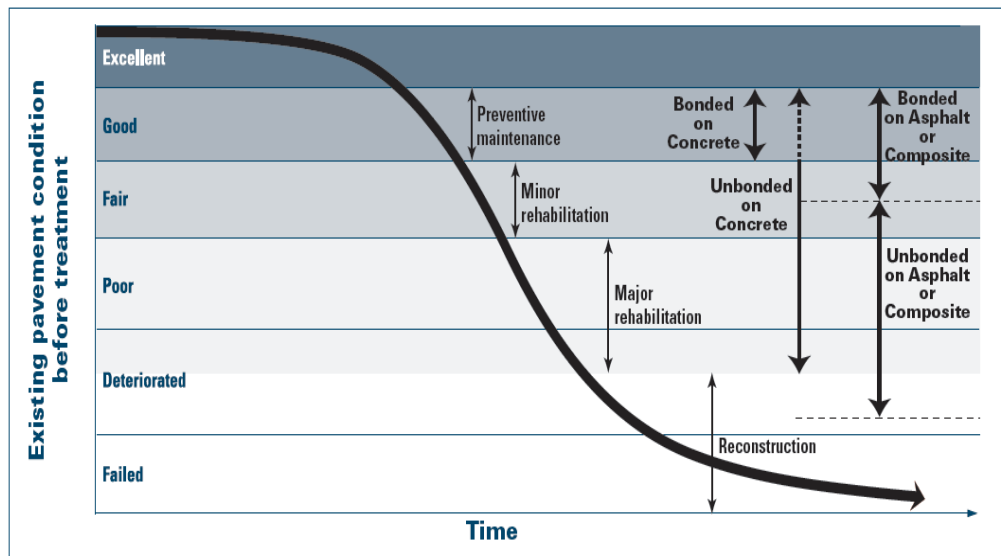


Figure 2.1 Typical Pavement Condition Curve over Time and Actions Needed

(Harrington 2008)

The process of rehabilitating asphalt concrete (AC) pavement with a concrete overlay is known as whitetopping (WT).

The first whitetopping project constructed in the United States was in 1918 in Terre Haute, Indiana (Hutchinson 1982). According to McGhee (1994), from 1918 to 1992 approximately 200 whitetopping projects were completed. Among them, 158 were jointed plain concrete pavement, 14 continuously reinforced concrete, 10 fiber-reinforced concrete, and 7 were jointed reinforced concrete pavement. Construction of whitetopping is not limited to the U.S. only. Other countries including Belgium, Sweden, Canada, Mexico, Brazil, the Republic of (South) Korea, Japan, France, Austria, and the Netherlands have undertaken recent projects with whitetopping (Rasmussen and Rozycki 2004). Agencies are widely expected to use whitetopping as it is a cost-competitive technique, can be constructed with minimal interruption of the traveling system, and a means of green construction as compared to asphalt concrete overlay.

2.1.1 Types of Whitetopping

Considering the thickness of this material, there are three types of whitetopping (WT) that can be utilized for restoration projects (Sheehan et al. 2004): conventional (thickness > 8 in); thin (thickness 4-8 in); and ultra-thin (thickness 2-4 in). A summary of whitetopping characteristics has been listed in Table 2.1. In this table, concrete overlays with a thickness greater than 4 in. are considered conventional whitetopping.

Table 2.1 Summary of Whitetopping Overlays (NHI)

	Conventional Whitetopping	Ultra-Thin Whitetopping
Typical Thicknesses	100 to 300 mm (4 to 12 in)	50 to 100 mm (2 to 4 in)
Condition of Existing Pavement	All deteriorated HMA pavements	Low-volume deteriorated HMA pavements (particularly in areas where rutting is a problem)
Bonding Condition	Designed as unbonded, but some partial bonding occurs (and may enhance pavement performance)	Strong bond required between existing HMA pavement and new PCC overlay
Preoverlay Repair	<ul style="list-style-type: none"> • Limited repair (failed areas only) • Possible milling to correct profile 	<ul style="list-style-type: none"> • Repair of areas unable to contribute to load-carrying capacity • Milling of the HMA surface
Minimum Thickness of HMA	50 mm (2 in) (after any milling)	75 to 150 mm (3 to 6 in) (after any milling)
Special Design and Construction Considerations	<ul style="list-style-type: none"> • Adequate support critical to performance • Adequate joint design (including joint spacing and load transfer) • Placement of a whitewash on HMA surface on hot days 	<ul style="list-style-type: none"> • Bonding with HMA pavement • PCC mix design is often high-strength and/or fiber modified • Extremely short joint spacings (typically between 0.6 and 1.8 m [2 to 6 ft])
PCC Overlay Types	<ul style="list-style-type: none"> • JPCP • JRCP • CRCP 	JPCP

A WT system can be classified into two types, depending on the bonding condition between the WT and AC layers:

1. Bonded: In bonded WT, a sound bond between the WT and the AC layer is maintained through proper construction techniques. Bonded WT is built to improve structural capacity and eliminate surface distresses of existing AC, which is in structurally good condition (Harrington 2008). Figure 2.2 shows the behavior of a bonded overlay under flexural loading.
2. Unbonded: In unbonded WT, a separation layer is provided at the interface of the WT and AC layer to prevent a bond between them. Unbonded WT is used to

rehabilitate AC pavement which is not in structurally good condition. Unbonded WT and its behavior under loading are shown in Figure 2.3.

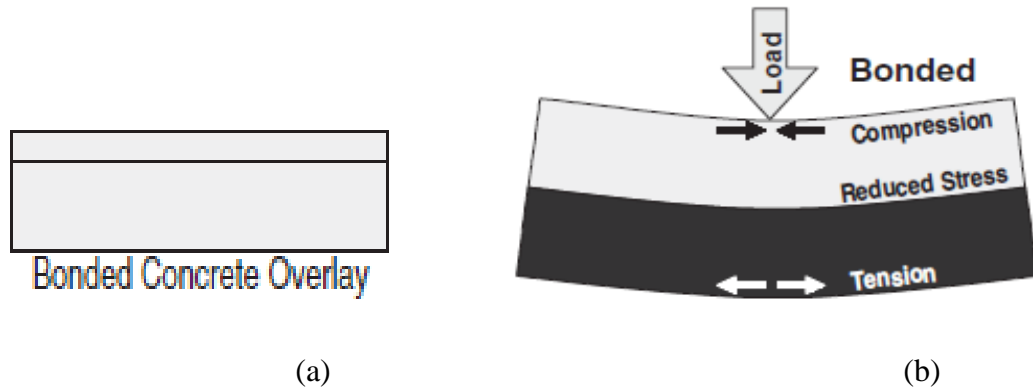


Figure 2.2 (a) Bonded WT and (b) Behavior of Bonded WT under Loading (Rasmussen and Rozycki 2004)

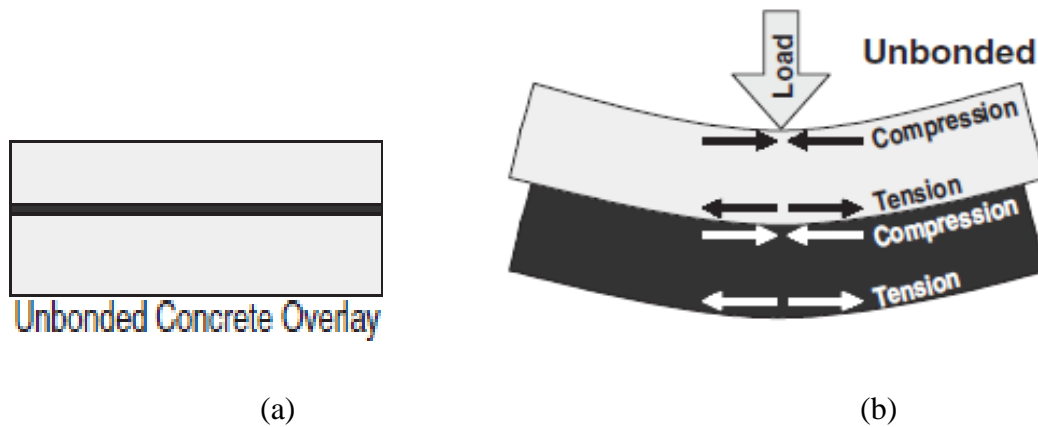


Figure 2.3 (a) Unbonded WT and (b) Behavior of Unbonded WT under Loading (Rasmussen and Rozycki 2004)

2.1.2 Benefits of Whitetopping

The formation of heat islands (HI) is a major concern in city areas by influencing energy demand, air and water quality and global warming. Heat islands are areas that are hotter than the neighboring rural area. Along with other factors, engineering materials such as asphalt concrete, Portland cement concrete, stone, and steel used in urban development are responsible for HI formation. Ting et al. (2001) conducted an evaluation of life-cycle costs (LCC) of reflective materials. They found that although asphalt concrete pavement (AC) has lower initial cost for rehabilitation work than that of whitetopping, the maintenance cost is much lower for whitetopping. Field measurements have shown that Portland cement concrete (PCC) has a much higher albedo value (0.35-0.4 for new concrete and 0.2-0.3 for most old concrete) than AC (0.05-0.1 for new asphalt and 0.10-0.15 for aged asphalt) (ACPA 2002). Albedo is defined as the ratio of reflected solar radiation to incoming solar radiation on the surface, and a higher albedo value provides better nighttime visibility. Thus, because of its high albedo rating, whitetopping may become a greener choice for rehabilitation of AC pavements.

Other benefits of using WT are as follows: it does not develop typical distresses as does asphalt overlay, repair is easy, it improves safety conditions, it is less affected by seasonal variations, it can be used as preventive maintenance or major rehabilitation, and it provides high level of serviceability (Harrington 2008; ACPA 1991). Moreover, construction of WT is quick and convenient: existing pavement can provide support, there is no need for extensive pre-overlay preparation, normal paving systems can be used, and overlaid pavement can be opened to traffic quickly (Harrington 2008; ACPA 1991).

2.1.3 Feasibility of Whitetopping

Whitetoppings can be applied to almost all existing pavements. In certain conditions, it is more cost effective than an AC overlay. That said, whitetoppings are generally not feasible under the following conditions (AASHTO 1993):

1. When the existing AC pavement is not highly deteriorated and other alternatives would be more cost effective.
2. When there is not adequate vertical clearance to accommodate the whitetopping.
3. When the existing AC pavement is prone to settlement and requires removal of the AC layer and stabilization of base layer.

Factors that should be considered in evaluating overall feasibility of whitetopping are listed in Table 2.2.

Table 2.2 Feasibility Guidelines for Whitetopping (NHI)

	CONVENTIONAL WHITETOPPING	ULTRA-THIN WHITETOPPING
CONSTRUCTIBILITY		
Vertical Clearance	Can be a problem.	Generally not a problem.
Traffic Control	Somewhat difficult to construct under traffic.	Somewhat difficult to construct under traffic.
Construction	Does not require special equipment.	Does not require special equipment.
PERFORMANCE PERIOD		
Existing Condition	Can be applied to very deteriorated HMA pavements. Minimum HMA thickness of 50 mm (2 in).	Can be applied to deteriorated HMA pavements. Minimum HMA thickness of 75 to 150 mm (3 to 6 in).
Extent of Repair	Very little repair is needed.	Milling of the HMA surface required to enhance bond.
Structural Adequacy	Substantial thickness may be required for structural adequacy.	Counting on bond between PCC overlay and HMA pavement to provide increased structural adequacy.
Future Traffic	Used under any traffic level.	Currently limited to lower volume roadways, but some experimental use on higher volume roadways.
Reliability	Very good.	Estimated to be fair to good (long-term performance data still lacking).
COST EFFECTIVENESS		
Initial Cost	Higher than conventional HMA overlay.	Slightly higher than conventional HMA overlay.
Life-Cycle Cost	Competitive.	Competitive.

2.2 Overview of Thin Whitetopping

2.2.1 Design Parameters

The design process starts with the determination of structural performance requirements and expected design life. Generally, the design process of WT consists of the following steps:

1. Characterization of Existing Pavement: The condition of the pavement is an important factor in the selection of a design process. It can be subcategorized into the following steps (Harrington 2008):

- i. Surface condition: The existing surface condition is important for both bonded and unbonded concrete overlays. The remaining pavement thickness after milling should be investigated, because this thickness needs to provide structural support of the new overlay. Depending on the surface distresses, the decision is made whether the overlay will be bonded or unbonded.
 - ii. Structural condition: Investigation into what structural support the existing pavement can provide is needed. Joint condition and inhibitors for load transfer are also determined.
2. Traffic Characterization: Pavement truck loads are accurately determined. Additional detailed information is also necessary for exact prediction of traffic, such as axle-load spectra, seasonal distribution of traffic, growth, and day-night duration (Rasmussen and Rozycki 2004).
3. Concrete Materials: Concrete strength, coefficient of thermal expansion, aggregate properties, supplementary cementitious materials, and admixtures should be considered carefully.
4. Climatic Factor: Climatic conditions during both construction and service life affect overlay behavior. Material should be compatible with weather conditions and joints should be provided depending on seasonal changes in pavement temperatures (Harrington 2008).
5. Whitetopping thickness: Determination of whitetopping thickness reliability is the main factor. Reliability is determined depending on the importance of the roadway. After whitetopping thickness is selected, it is checked for overhead clearance, and

curb and gutter. Reduction of the existing AC thickness is also a factor at this stage (Harrington 2008).

6. Joint Spacing: In whitetopping, shorter joints are provided for reducing corner cracking and providing proper aggregate interlocking. The most common rule of thumb for whitetopping is the joint spacing (in inches) should be 12 to 18 times the slab thickness in inches (ACPA 1998). In many whitetopping projects, dowel and tie bars are provided to minimize movement along longitudinal joints, reducing the significance of joints.
7. Transition Area: During the design process, the transition area between the overlay and the adjacent AC layer should be given proper consideration. A thickened slab is recommended for these types of transitions (Rasmussen and Rozycki 2004).

2.2.2 Design Procedures

There are a number of procedures for design of whitetopping. Existing procedures include the AASHTO Guide (AASHTO 1993); the state of Colorado procedure (Sheehan et al. 2004; Tarr et al. 1998); the Portland Cement Association (PCA)/American Concrete Pavement Association (ACPA) method (Wu et al. 1998); the state of New Jersey method (Nenad 1998); the modified ACPA approach (Riley et al. 2005); the state of Illinois method (Roesler et al. 2008); and the state of Texas method (Chul et al. 2008). Among these, AASHTO, Colorado, and Texas provide design procedures which can be used for designing TWT, while the rest are for procedures for ultra-thin whitetopping (UTW). Recently, MnDOT (2010) has developed a design technique for thin whitetopping. A brief description of TWT design procedures is presented in the following section.

AASHTO

According to AASHTO (AASHTO 1993) the required design thickness is

$$D_{ol} = D_f \quad (2.1)$$

Where,

D_{ol} = required thickness of PCC overlays (WT), in.; and

D_f = slab thickness to carry future traffic, in.

The design thickness of WT depends on the structural capacity needed to carry future traffic and the condition of the existing AC layer. The design procedure involves the following steps: (1) the determination of material types and layer thicknesses of existing pavement; (2) prediction of future traffic for the design period, N_f ; and (3) a general survey of existing pavement distresses of heaves and swell, stripping of AC pavement, and large transverse cracks is required as these distresses may affect the service life of WT. Furthermore, as a fourth step, an adequate assessment of pavement condition by deflection testing with a heavy-load deflection device (generally of magnitude of 9,000 pounds) at a sufficient interval is recommended. The subgrade modulus (M_R) and effective pavement modulus (E_p) at each point are calculated according to the following equations.

$$M_R = \frac{0.24P}{d_r r} \quad (2.2)$$

Where,

M_R = backcalculated subgrade resilient modulus, psi;

P = applied load, pounds;

d_r = deflection at a distance r from the center of the load, in.; and

r = distance from center of load, in.

$$a_e = \sqrt{\left[a^2 + \left(D \sqrt[3]{\frac{E_p}{M_R}} \right)^2 \right]} \quad (2.3)$$

Where,

a_e = radius of stress bulb at the subgrade-pavement interface, in.;

a = NDT load plate radius, in.;

D = total thickness of pavement layers above the subgrade, in.; and

E_p = effective modulus of all pavement layers above the subgrade, psi.

Note that the effective dynamic k-value is determined from Figure 2.4 using M_R , E_p , and D .

After these four steps have been completed, the fifth recommended step, according to the AASHTO guidelines, is to complete coring and material testing if there is an unusual distress condition. Step six is to determine the required slab thickness using specific criteria in conjunction with the equations presented in step four, as indicated by the following instructions:

- i. The effective static k-value is determined from one of the following methods, along with the adjustment for seasonal effects:
 - a) The effective dynamic k-value obtained in step 4 is divided by 2 to obtain the effective static k-value. The obtained static k-value is adjusted for seasonal effects.
 - b) The effective static k-value is obtained from soils data and pavement layer types and thicknesses using Figure 2.4.
- ii. The design PSI (Present Serviceability Index) loss is determined by subtracting the PSI at time of next rehabilitation from the PSI immediately after overlay.
- iii. Joint load transfer, J , is determined from Table 2.3.

- iv. 28-day, 3rd point loading modulus of rupture is used as WT modulus of rupture.
- v. 28-day, elastic modulus is used as WT modulus.
- vi. Loss of support is determined from Table 2.4.
- vii. WT reliability, R (%) is selected from Table 2.5.
- viii. Overall standard deviation is selected to be between 0.30 and 0.40.
- ix. Drainage coefficient, C_d , is determined from Table 2.6.

Note that slab thickness D_f (thickness to carry future traffic) is determined from the nomograph shown in Figures 2.5 and 2.6.

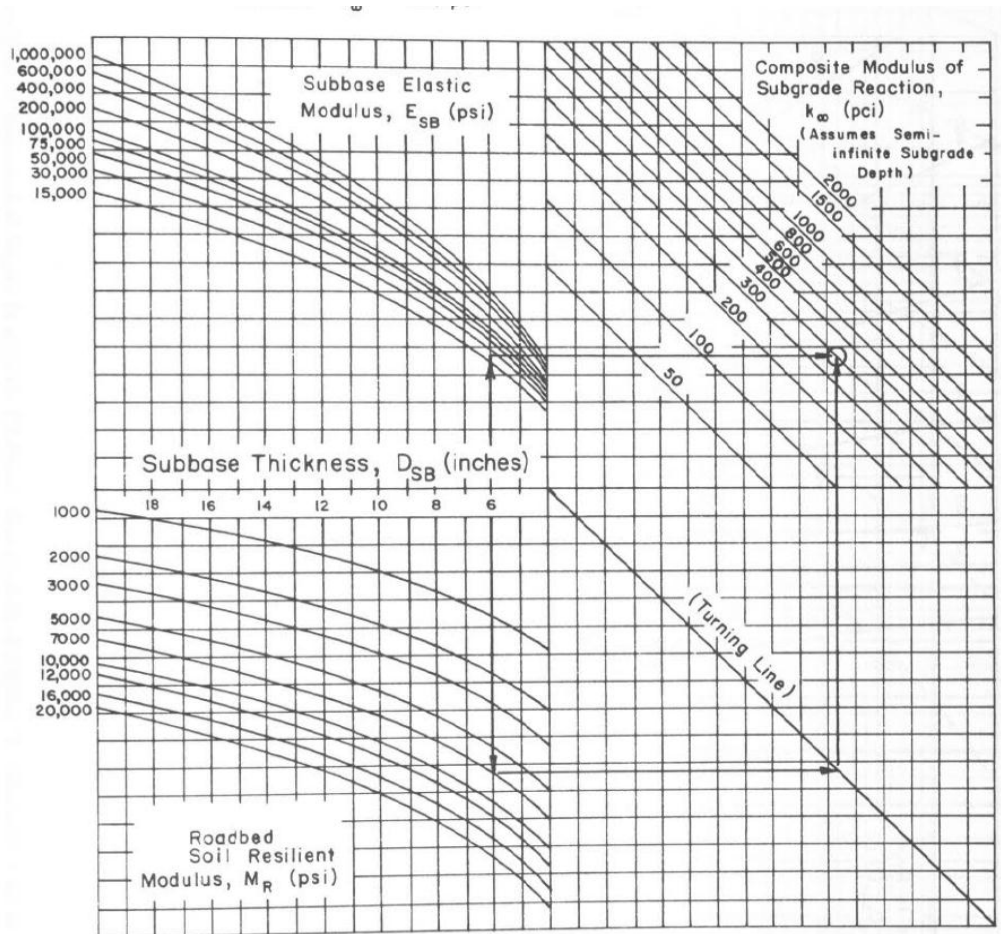


Figure 2.4 Chart for Estimating Composite Modulus of Subgrade Reaction

(AASHTO 1993)

Table 2.3 Recommended Load-Transfer Coefficients for Various Pavement Types and Design Conditions (AASHTO 1993)

Shoulder Load-Transfer Devices Pavement Type	Asphalt		Tied P.C.C.	
	Yes	No	Yes	No
1. Plain jointed and jointed reinforced	3.2	3.8-4.4	2.5-3.1	3.6-4.2
2. CRCP	2.9-3.2	N/A	2.3-2.9	N/A

Table 2.4 Typical Ranges of Loss of Support (LS) Factors for Various Types of Materials (AASHTO 1993)

Type of Material	Loss of Support (LS)
Cement-Treated Granular Base (E = 1,000,000 to 2,000,000 psi)	0.0 to 1.0
Cement Aggregate Mixtures (E = 500,000 to 1,000,000 psi)	0.0 to 1.0
Asphalt-Treated Base (E = 350,000 to 1,000,000 psi)	0.0 to 1.0
Bituminous Stabilized Mixture (E = 40,000 to 300,000 psi)	0.0 to 1.0
Lime Stabilized (E = 20,000 to 70,000 psi)	1.0 to 3.0
Unbound Granular Materials (E = 15,000 to 45,000 psi)	1.0 to 3.0
Fine-Grained or Natural Subgrade Materials (E = 3,000 to 40,000 psi)	2.0 to 3.0

Table 2.5 Suggested Level of Reliability for Various Functional Classifications (AASHTO 1993)

Functional Classification	Recommended Level of Reliability	
	Urban	Rural
Interstate and Other Freeways	85-99.9	80-99.9
Principal Arterials	80-99	75-95
Collectors	80-95	75-95
Local	50-80	50-80

Table 2.6 Recommended Values of Drainage Coefficient, C_d , for Rigid Pavement Design
(AASHTO 1993)

Quality of Drainage	Percent of Time Pavement Structure Is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1-5%	5-25%	Greater Than 25%
Excellent	1.25-1.2	1.20-1.15	1.15-1.00	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

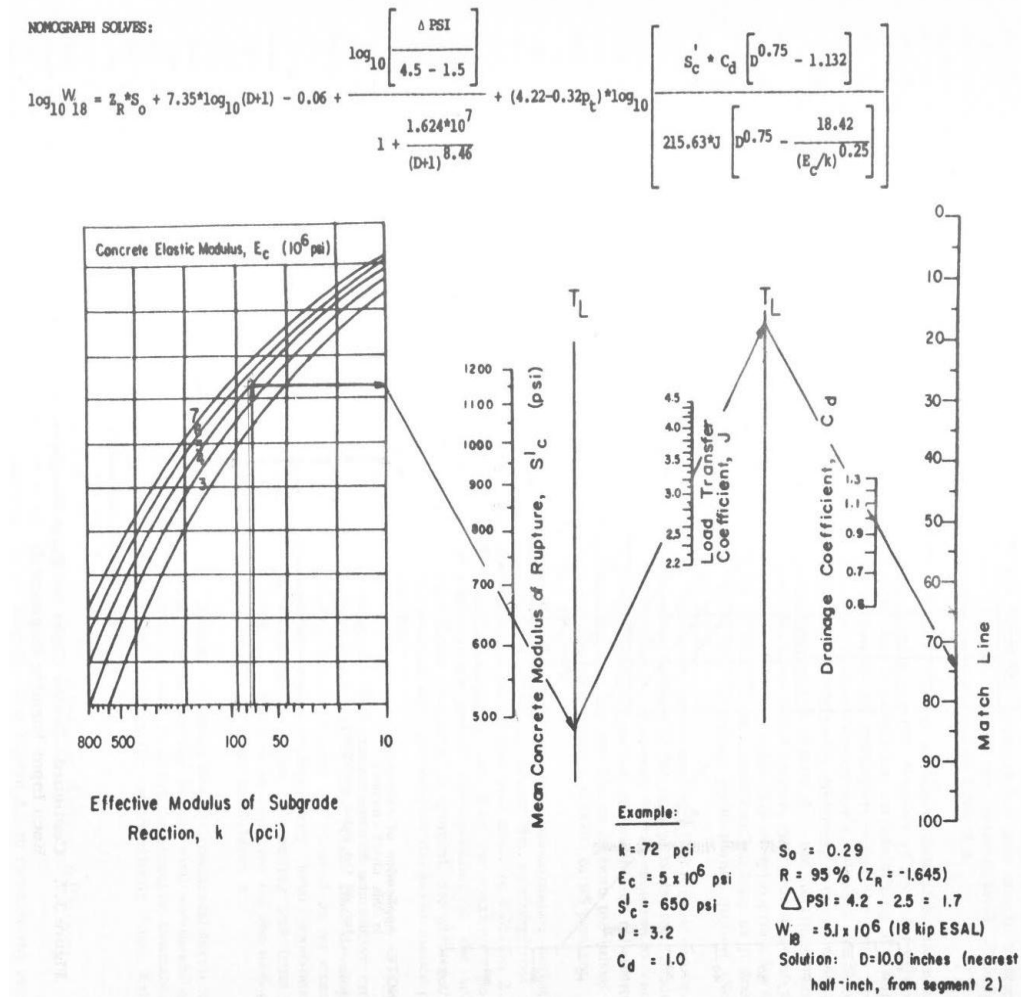


Figure 2.5 Design Chart for Rigid Pavement Based on Using Mean Values for Each Input Variable (Segment 1) (AASHTO 1993)

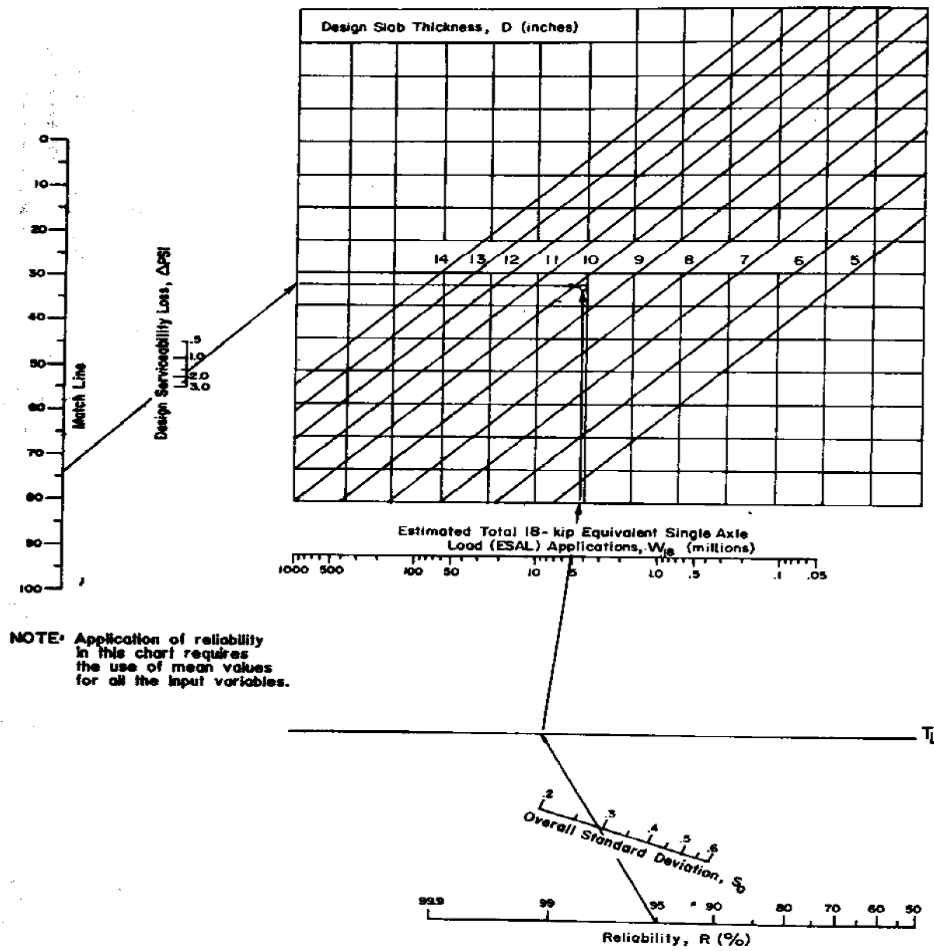


Figure 2.6 Design Chart for Rigid Pavement Based on Using Mean Values for Each Input Variable (Segment 2) (AASHTO 1993)

The seventh, and final step, is to determine the PCC overlay thickness using Eq. 2.1.

Colorado Procedure

In 1998, Tarr et al. developed the model for designing TWT, then in 2004 Sheehan et al. adjusted the model. Major findings of the Colorado design procedure (Sheehan et al. 2004; Tarr et al. 1998) can be summarized as follows:

1. For the design of TWT, critical load locations were identified and verified with field data. For a zero-temperature gradient (at early morning), the critical load-induced stresses were found.
2. A relationship was developed between the theoretical fully bonded TWT stress with partial-bonded stress from the analysis of experimental and theoretical concrete stress data. Correction factors for measured stress are shown in Eqs. 2.4 (a) and 2.4 (b).

The original model (Tarr et al. 1998):

$$\sigma_{ex} = 1.65 \times \sigma_{th} \quad (2.4 \text{ a})$$

The adjusted model (Sheehan et al. 2004):

$$\sigma_{ex} = 1.51 \times \sigma_{th} \quad (2.4 \text{ b})$$

Where,

σ_{ex} = measured experimental partially bonded stress, psi; and

σ_{th} = calculated fully bonded stress, psi.

A relationship between theoretical fully-bonded maximum asphalt flexural strains and partially bonded strains was also developed. Correction factors for measured asphalt flexural strains are shown in Eqs. 2.5 (a) and 2.5 (b).

The original model (Tarr et al. 1998):

$$\varepsilon_{ac} = 0.842 \times \varepsilon_{pcc} \quad (2.5 \text{ a})$$

The adjusted model (Sheehan et al. 2004):

$$\varepsilon_{ac} = 0.897 \times \varepsilon_{pcc} - 0.776 \quad (2.5 \text{ b})$$

Where,

ε_{ac} = measured asphalt surface strain, microstrain; and

ε_{pcc} = measured concrete bottom strain, microstrain.

3. The design procedure considers tied longitudinal joints. The relationship between the free-edge stress and tied-edge stress was also developed, which is shown by Eq. 2.6.

$$\sigma_{FE} = 1.87 \times \sigma_{TE} \quad (2.6)$$

Where,

σ_{FE} = longitudinal free-joint load-induced stress, psi, and

σ_{TE} = longitudinal tied-joint load-induced stress, psi.

4. The finite element program ILSL2 (Khazanovich and Ionnides 1993) was used to determine concrete stress and asphalt strain. For prediction of concrete stress and asphalt strain, equations were derived from the least-square linear regression technique. The prediction relationships are shown in Eqs. 2.7 through Eq. 2.10. For 20-kip single-axle-load concrete stress:

The original model (Tarr et al., 1998) is

$$\sigma_{pcc} = 919 + \frac{18,492}{l_e} - 575.3 \times \log k + 0.000133 \times E_{ac} \quad (2.7 \text{ a})$$

The adjusted model (Sheehan et al., 2004) is

$$(\sigma_{pcc})^{\frac{1}{2}} = 18.879 + 2.918 \times \frac{t_{pcc}}{t_{ac}} + \frac{425.44}{l_e} - 6.955 \times 10^{-6} \times E_{ac} - 9.0366 \times \log k + 0.0133 \times L \quad (2.7 \text{ b})$$

For 20-kip single-axle-load asphalt strain:

The original model (Tarr et al., 1998) is

$$\frac{1}{\varepsilon_{ac}} = 8.51114 \times 10^{-9} \times E_{ac} + 0.008619 \times \frac{l_e}{L} \quad (2.8 \text{ a})$$

The adjusted model (Sheehan et al., 2004) is

$$(\varepsilon_{ac})^{\frac{1}{4}} = 8.224 - 0.2590 \times \frac{t_{pcc}}{t_{ac}} - 0.04419 \times l_e - 6.897 \times 10^{-7} \times E_{ac} + 1.1027 \times \log k \quad (2.8 \text{ b})$$

For 40-kip tandem-axle-load concrete stress:

The original model (Tarr et al., 1998) is

$$\sigma_{pcc} = 671.2 + \frac{1.582 \times 10^4}{l_e} - 437.1 \times \log k - 0.000099 \times E_{ac} \quad (2.9 \text{ a})$$

The adjusted model (Sheehan et al., 2004) is

$$(\sigma_{pcc})^{\frac{1}{2}} = 17.669 + 2.668 \times \frac{t_{pcc}}{t_{ac}} + \frac{408.52}{l_e} - 6.455 \times 10^{-6} \times E_{ac} - 8.3576 \times \log k + 0.00622 \times L \quad (2.9 \text{ b})$$

For 40-kip tandem-axle-load asphalt strain:

The original model (Tarr et al., 1998) is

$$\frac{1}{\varepsilon_{ac}} = 9.61792 \times 10^{-9} \times E_{ac} + 0.009776 \times \frac{l_e}{L} \quad (2.10 \text{ a})$$

The adjusted model (Sheehan et al., 2004) is

$$(\varepsilon_{ac})^{\frac{1}{4}} = 7.923 - 0.2503 \times \frac{t_{pcc}}{t_{ac}} - 0.0433 \times l_e - 6.746 \times 10^{-7} \times E_{ac} + 1.0451 \times \log k \quad (2.10 \text{ b})$$

Where,

σ_{pcc} = maximum stress in concrete slab, psi;

ε_{ac} = maximum strains at bottom of asphalt layer, microstrain;

E_{pcc} = concrete modulus of elasticity, assumed 4 million psi;

E_{pcc} = asphalt modulus of elasticity, psi;

t_{pcc} = thickness of concrete layer, in;

t_{ac} = thickness of asphalt layer, in;

k = modulus of subgrade reaction, pci;

l_e = effective radius of relative stiffness for fully bonded slabs, in;

Is equal to:

$$\left\{ E_{pcc} \times \frac{\left[\frac{t_{pcc}^3}{12} + t_{pcc} \times \left(NA - \frac{t_{pcc}}{2} \right)^2 \right]}{\left[k \times (1 - \mu_{pcc}^2) \right]} + E_{ac} \times \frac{\left[\frac{t_{ac}^3}{12} + t_{ac} \times \left(t_{pcc} - NA + \frac{t_{ac}}{2} \right)^2 \right]}{\left[k \times (1 - \mu_{ac}^2) \right]} \right\}^{0.25}$$

And,

NA = neutral axis from top of concrete slab, in.

$$= \frac{\left[E_{pcc} \times \frac{t_{pcc}^2}{2} + E_{ac} \times t_{ac} \times \left(t_{pcc} + \frac{t_{ac}}{2} \right) \right]}{\left[E_{pcc} \times t_{pcc} + E_{ac} \times t_{ac} \right]}$$

Where,

μ_{pcc} = poisons ratio for concrete, assumed 0.15;

μ_{ac} = poisons ratio for asphalt, assumed 0.35; and

L = joint spacing, in.

- Concrete fatigue criterion (PCA 1984) is based on the theory that fatigue resistance which remains after one load application is available for other load repetitions (Miner 1945). The PCA fatigue criterion is as follows.

For stress ratio (SR) > 0.55:

$$\log_{10}(N) = \frac{0.97187 - SR}{0.0828} \quad (2.11)$$

For $0.45 \leq SR \leq 0.55$:

$$N = \left(\frac{4.2577}{SR - 0.43248} \right)^{3.268} \quad (2.12)$$

For $SR < 0.45$:

$$N = \text{Unlimited} \quad (2.13)$$

Where,

SR = flexural stress to strength (modulus of rupture of concrete) ratio; and

N = number of allowable load repetitions.

The asphalt concrete fatigue equation (Asphalt Institute 1981) was also used in this study, as follows:

$$N = C \times 18.4 \times (4.32 \times 10^{-3}) \times \left(\frac{1}{\varepsilon_{ac}} \right)^{3.29} \times \left(\frac{1}{E_{ac}} \right)^{0.854} \quad (2.14)$$

Where,

N = number of load repetitions for 20 % or greater AC fatigue cracking;

ε_{ac} = maximum tensile strain in the asphalt layer;

E_{ac} = asphalt modulus of elasticity, psi;

C = correction factor = 10^M .

$$M = 4.84 \times \left[\frac{V_b}{V_v + V_b} - 0.69 \right]$$

V_b = volume of asphalt, percent; and

V_v = volume of air voids, percent.

Texas Procedure

In 2008, Chul et al. developed mechanistic procedures for designing TWT. A finite element program, ISLAB 2000, was used to model TWT behavior under load. The pavement structure consisting of TWT, an AC layer, base, and subgrade were analyzed for different geometry and loading conditions. The dimension of TWT is 18 ft \times 18 ft, and 6 ft joint spacing was provided. The loading conditions with a 20-kip single axle load and 34-kip tandem axle load have been shown in Figures 2.7 (a) and 2.7 (b).

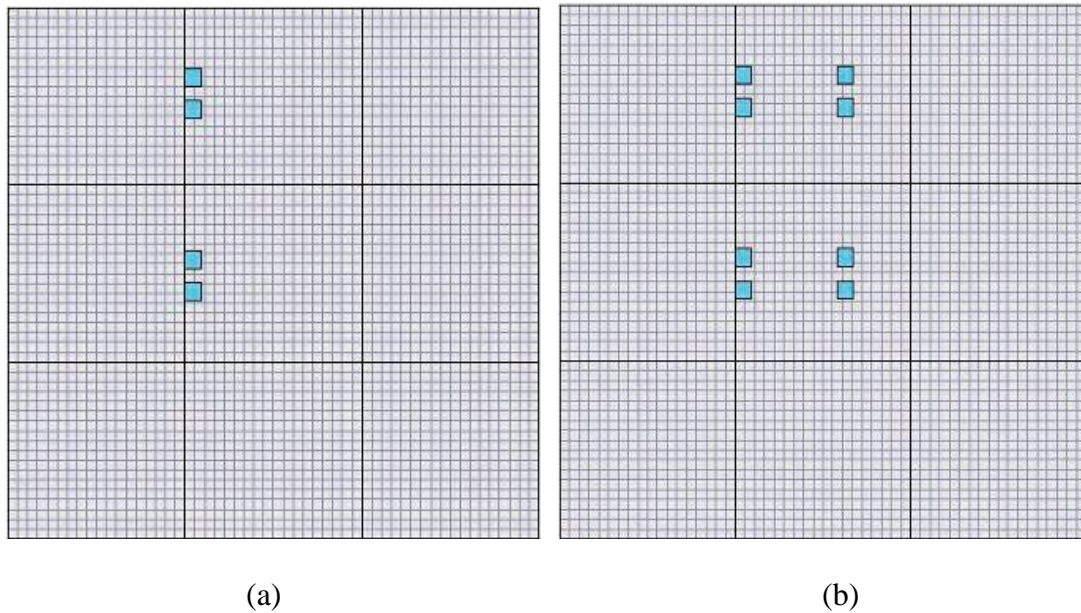


Figure 2.7 TWT Model Using ISLAB2000 (a) Single Axle Load and (b) Tandem Axle Load (Chul et al. 2008)

The Texas design procedure for TWT outlines four steps. Step one recommends that the structural condition and material properties be evaluated using a dynamic cone penetrometer (DCP) and a falling weight deflectometer (FWD). In step two, the subgrade resilient modulus is estimated using the following relationship:

$$M_R = \frac{438,000}{DCP^{1.12}} \quad (2.15)$$

Where,

M_R = resilient (elastic) modulus, psi; and

DCP = DCP index (mm/blow).

The ELSYM5 layered program is recommended for estimating the subgrade reaction modulus with the calculated modulus value from Eq. 2.15. The value of the static subgrade modulus is one half of the subgrade modulus.

In step three, the resilient modulus obtained in step two is used in a back-calculation program to estimate the resilient modulus of the subbase layer. Asphalt layer thickness that is more than 3 in. is treated as a separate layer in the back-calculation program—otherwise the AC layer is combined with the base layer. For step four, the required slab thickness is calculated using Eq. 2.16.

$$\begin{aligned} \log(t_{pcc}) = & 3.5615 + 0.1017 \times \log(ESALs) + 0.4982 \times \log(E_{pcc}) - 0.7232 \times \log(t_{AC}) - \\ & 0.3624 \times \log(E_{AC}) - 0.2695 \times \log(t_{BS}) - 0.0891 \times \log(E_{BS}) - 0.0287 \times \log(k) - \\ & 1.2250 \times \log(MR) \end{aligned} \quad (2.16)$$

Where,

t_{pcc} = required thickness of the TWT, in.;

ESALs=expected number of 18-kips ESALs;

E_{pcc} = concrete modulus of elasticity, psi;

t_{AC} = thickness of the asphalt layer, in.;

E_{AC} = asphalt modulus of elasticity, psi;

t_{BS} = thickness of the base layer, in.;

E_{BS} = base modulus of elasticity, psi;

k = modulus of subgrade reaction, pci; and

MR = modulus of rupture of concrete in TWT, psi.

2.2.3 Construction

The performance of TWT is highly dependent on the construction process. As the thickness of whitetopping is much thinner than the conventional overlay, there should not be much variability in the concrete layer. The construction process includes the following: pre-overlay works, overlay materials, placement and finishing, curing, and joint sawing.

2.2.3.1 Pre-Overlay Works

Pre-overlay works are very important to ensure long-term durability of TWT. Although existing AC distresses do not reflect through TWT as in asphalt overlay, there is still a need to repair the asphalt distresses to avoid different types of failure such as faulting, roughness, or shattered slab. Possible pre-overlay works on existing AC pavements for bonded TWT are tabulated in Table 2.7. Pre-overlay works for unbonded TWT are listed in Table 2.8.

Milling plays an important role by producing a sound bond between whitetopping and the AC layer. It levels out surface distortions, and removes soft loose asphalt and high spots which cause an inadequate bond. Milling also enhances the bond by roughening the surface, finishing the grade line before concrete placement, and establishing a cross slope for the new pavements (Rasmussen and Rozycki 2004).

For unbonded whitetopping, a separation layer is introduced between the TWT and AC layer so that no bond develops at the interface.

Table 2.7 Possible Pre-Overlay Works for Bonded TWT (Harrington 2008)

Existing Pavement Distress	Repair Work Performed
Rutting (<2inches)	None
Deep rutting (>2inches)	Milling
Shoving, slippage	Milling
Thermal cracking	Fill crack when opening is greater than maximum-size aggregate in the overlay
Fatigue cracking	Full-depth concrete patch
Pothole	Full-depth concrete patch

For bonded TWT, before concrete placement, the surface is properly cleaned by sweeping and using compressed air. No moisture is allowed at the asphalt surface prior to overlay placement.

Table 2.8 Possible Pre-Overlay Works for Unbonded TWT (Harrington 2008)

Existing Pavement Distress	Repair Work Performed
Area of subgrade/ subbase failure	Remove and replace with stable material; correct water problems
Severe distress that results in variation in strength of asphalt	Remove and replace with stable material; correct water problems
Potholes	Fill with asphalt
Shoving	Mill
Rutting (<2inches)	None or mill
Deep rutting (>2inches)	Mill
Crack width \geq 4 in.	Fill with asphalt
Crack width \leq 4 in.	None

2.2.3.2 Overlay Materials

During the concrete mixture proportioning, both faster construction and high concrete strength are considered. According to ACI 325.13R (ACI 2006) for whitetopping concrete mixtures, Type I and Type II cements are used. In some cases, Type III cements are used for the development of high early strength. Typical cement content varies between

500 and 700 lb/yd³. High-quality stones, river gravel and glacial deposits are used as aggregates for the overlay to ensure longevity. The maximum aggregate size is selected depending upon the design thickness of whitetopping. A maximum water-cement (w/c) ratio of 0.45 is recommended for moist environments and areas prone to the freeze-thaw cycle (PCA 2002). For bonded overlay, the w/c ratio is kept lower than the unbonded overlay to ensure proper bond strength, which can be affected by drying shrinkage, resulting from a high w/c (McGhee 1994). According to ACI 212.3R (ACI 2004), the following admixtures are recommended for sound concrete:

1. Air entrainment in the range of 4-6 % is recommended, which helps to increase workability and protects from segregation, bleeding and freeze thaw.
2. To increase the development rate of early strength, calcium chloride is used as an accelerator. For reinforced concrete, non-chloride accelerators are used;
3. Water reducers are added for lowering the w/c to maintain the desired slump; and
4. Supplementary cementitious materials, such as fly ash, slag, and silica fume may be used for improved workability, reduced permeability and alkali-silica reactivity, and increased long-term strength.

Typical values of 28 day compressive and flexural strengths for TWT mix are 4,000 psi and 650 psi, respectively (ACI 2004).

For bonded concrete overlay, generally the bond between the concrete and AC layer is achieved through cleaning the AC surface and removing surface contaminations. But, many agencies use cement grout of w/c ratio of 0.65 as a boning agent at the interface (ACPA 1990).

For unbonded TWT, materials used as a separator layer can include polyethylene sheeting, wax-based curing compounds, liquid asphalts, and hot-mixed asphalt (McGhee 1994).

Hot-poured rubberized materials, silicon materials, or preformed compression seals are used as joint-sealant materials (ACI 2006).

2.2.3.3 Concrete Placement and Finishing

During placement of concrete, the surface temperature of the asphalt should not be higher than 120⁰ F to avoid plastic shrinkage cracking (Harrington 2008). For placing the concrete either fixed-form or slip-form paving practices can be used (Rasmussen and Rozycki 2004). For fixed-form paving, the forms are supported in such a way that no movement can occur, and are designed to withstand lateral pressure from concrete, which is placed by a ready-mix truck, pump or other means. Figure 2.8 shows the use of a fixed form. With a slip-form paver, fresh concrete is placed by truck, pump, and other means in one operation. In this case, stringlines are needed for horizontal and vertical grade control. Figure 2.9 shows use of slip-form paving. Texturing of the finished concrete surface is performed by brooming, burlap dragging, turf dragging, or a tining comb, depending on the speed limit for the facility. Tining of the concrete surface is performed immediately after the moisture sheen evaporates (ACI 2006).



Figure 2.8 Use of Fixed Forms (Rasmussen and Rozycki 2004)



Figure 2.9 Use of Slip Forms (Rasmussen and Rozycki 2004)

2.2.3.4 Curing

Good curing practice is important to achieve the intended performance of TWT. As the TWT surface area-to-volume ratio is high, there is high potential for moisture loss. The curing compound should be applied at twice the normal rate used for conventional concrete pavement (Harrington 2008).

2.2.3.5 Joint Sawing

Joints should be properly sawed to avoid random cracking. It is done as soon as the concrete is hard enough for cutting without raveling or cracking. Transverse joints can be sawed by a conventional saw set to one-fourth of the slab thickness or by early-entry sawing set to not less than 1.25 in. Longitudinal joints are sawed to one-third of the slab thickness (Harrington 2008).

2.2.4 Performance of Thin Whitetopping Projects

A number of whitetopping projects have recently been constructed. In this section, a brief discussion is presented about TWT project performance in different states.

Colorado

In the state of Colorado, three whitetopping test pavements were constructed from 1996 to 1997 with different combinations of design and construction variables. In 2002 and 2005, two other projects were constructed. The first pavement (CDOT1) was on Santa Fe Drive, the second (CDOT2) on State Road 119 near Longmont, and the third (CDOT3) on US 287 near Lamar (Tarr et al. 1998). Other projects were constructed on SH 121 between Colorado Route C 470 and Park Hill Avenue in 2002 (Wu and Shehaan 2002), and at SH 83, Parker Road in 2005 (CPTP 2007).

CDOT1 had two sections, each 500 ft long and with a 60 in. joint spacing. The first 500 ft long section consisted of a 4 in. whitetopping placed on a 5 in. newly placed AC layer with no special surface preparation. The second section had 5 in. whitetopping on top of 4 in. AC pavement, and a portion of which had been milled before placing the concrete. On all sections, tie bars were used, longitudinal and transverse joints were sawcut, the soil underneath the pavement was A-7-6, and the modulus of subgrade reaction was approximately 150 pci.

CDOT2 was about one mile long and had two lanes of existing AC pavement. This project consisted of a number of sections. The east half of the section had a 1.5 in. new AC layer constructed on top of the existing AC pavement before placing a 5 in. whitetopping. On the other half, a 4.5 in. concrete layer was placed directly on top of the old pavement. In the other traffic lane, the existing AC layer was milled to 1.5 in. and a 6 in. whitetopping was placed on top of it. On all sections, old AC pavements were washed before placing the new concrete. Tie bars and dowel bars were used where the joint spacing was more than 12 ft. The modulus of subgrade reaction was found to be 340 pci.

The CDOT3 project had heavily trafficked three-mile long sections. A 6 in. concrete layer was placed on top of the milled and cleaned existing AC layer. The milled AC layer thickness and modulus of subgrade reaction were found to be 7 in. and 225 pci, respectively. Dowel bars were used on all sections except one, and tie bars were installed at all longitudinal joints.

At SH 121, the project was a four mile long section on a four-lane divided secondary arterial road. The 6 in. whitetopping was constructed on a 5-1/2 in. existing AC layer with 6 ft joint spacings. The TWT was constructed for a 10 year design period for

carrying approximately 1.3 million 18 kip-equivalent single-axle loads. The existing AC surface was milled to provide a desired bond at the interface. The SH 83 project was 1.9 miles long and on a six lane urban highway. The concrete overlay was 6 in. thick with 6 ft joint spacing. Performance of these two projects has not been reported to date.

In 2003, the first three projects were revisited and reviewed (Sheehan et al. 2004). Figures 2.10 to 2.15 show photographs from these projects.

The evaluation of these test pavements can be summarized as follows:

1. The overall condition of Santa Fe Drive was very good, except for some isolated longitudinal cracking with a few corner cracks and transverse shrinkage cracks over them. There were a number of shattered slabs at the stop sign approach on this pavement.
2. State Road 119 was also in a very good condition. Only one section containing 10-ft by 12-ft panels showed a number of mid-panel cracks. Considerable joint spalling, resulting from snow plow abrasion, was found. Overall ride quality of this test section was excellent.
3. U.S. Route 287 was also in very good condition. Minor transverse joint spalling was found frequently. Longitudinal cracks were also observed on 6 ft. by 6 ft panels. During initial construction, the northbound lane experienced significant cracking resulting from the placement of concrete on the hot asphalt layer, which initiated shrinkage cracking. Consequently, cooling of asphalt surface prior to concrete placement was recommended.



Figure 2.10 Typical Pavement Conditions Observed at Santa Fe Drive Site

(Sheehan et al. 2004)



Figure 2.11 Stop Sign Approach Distressed Area Observed at Santa Fe Drive Site

(Sheehan et al. 2004)



Figure 2.12 Typical Pavement Conditions Observed at the S.H. 119 Longmont Site
(Sheehan et al. 2004)



Figure 2.13 Typical Slab Cracking Observed in S.H. 119 Test Section No. 2
(Sheehan et al. 2004)



Figure 2.14 Typical Pavement Conditions Observed at the U.S. 287 Lamar Site
(Sheehan et al. 2004)



Figure 2.15 Typical Cracked Slab Conditions Observed at the U.S. 287 Lamar Site
(Sheehan et al. 2004)

Illinois

From 1998 to 2001, four experimental TWT projects were constructed at Clay county, Tuscola, Platt county, and Cumberland county in the state of Illinois (Winkelman 2002).

In August 1998, there were 5 in. and 6 in. TWTs constructed on AC pavements which had minor edge cracking and rutting, and small potholes on County Highway 3 in Clay county. The whitetopping sections had partial-depth saw cuts. The concrete overlay was placed on the AC layer after milling for proper bond and construction platform, and for removing surface defects.

US Highway 36, east of Tuscola, had whitetopping constructed in May 1999. The existing AC pavement had moderate-to-high-severity transverse cracks and was overlaid with 4-7.5 in. TWT. Typical panel dimensions in feet were 0.7 to 1.3 times the overlay thickness in inches.

During September and October of 2000, the experimental TWT project was constructed in Platt county on County Highway 4. A 5 in. concrete overlay was placed prior to milling of the existing AC surface. The panels were either 5.5 ft by 5.5 ft or 11 ft by 11 ft.

In September 2001, a 3.5 mile section of County Highway 2 in Cumberland county was overlaid with a 5.75 in. concrete layer. Prior to concrete placement, the existing AC layer was milled to about three inches. The panels of TWT were 5.5 ft by 6.0 ft.

Early performance evaluation in 2001 showed good pavement performance which can be summarized as follows:

1. No crack or distress was found in the Clay county project.

2. In the Tuscola project, cracks observed were mid-panel cracking and corner breaks. All cracks were of low severity with no sign of debonding. Figure 2.16 shows the common corner breaks found on the Tuscola project.
3. Low-severity cracks were found on 0.5% of panels on the Platt county project. All of these cracks were corner breaks.
4. In the Cumberland county project, low-severity cracks were found on only four out of 1,440 panels during evaluation six months after construction.



Figure 2.16 Panel Corner Breaks in Tuscola Project (Winkelman 2002)

Florida

In May 2003, TWT was placed at Fernandina Beach Airport in Florida (Armaghani et al. 2005). The whitetoppings were 6 in. thick on the runway and 5 in. thick on taxiway C over severely rutted asphalt pavements. The panels were 5 ft. by 6.25 ft. for the runway and

4 ft. by 4 ft. for taxiway C. In 2004, a condition survey of this project was conducted. Findings from the survey can be summarized as follows:

1. No cracking of the whitetopping was found except some corner chipping, which might have resulted from inadequate timing of joint sawing. The shoulder was in good condition with no sign of erosion and skid resistance was very high according to FDOT's Runway Friction Tester results.
2. Stiffness of the pavement system resulting from whitetopping placement increased by an average of 300%, which indicated enhanced load-carrying capacity. Good bond between whitetopping and the AC layer was found from the falling weight deflectometer (FWD) testing.

Montana

In 1999, a TWT overlay was constructed at Great Falls in Montana with a TWT thickness of 4.5 in (MtDOT 2001). After two years, the overlaid pavement showed localized failure, and slab shattering was also observed. After inspection, an underlying storm drain was found to be the moisture source that led to such damage of the TWT. In 2000, another resurfacing was done with a 5 in. TWT overlay at East Idaho St. in the city of Kalispell (MtDOT 2001). Milling, vacuuming, and sweeping of the existing asphalt surface were completed for producing a sound bond at the interface. After concrete placement, one small section was found in poor condition as a result of improper vibration during construction. The MtDOT evaluation report (MtDOT 2008) showed that the TWT was found to be in good condition except for some minor hairline cracks (Figure 2.17) and spalling. No sign of debonding or faulting was found.



Figure 2.17 Hairline Crack in Kalispell Project Pavement (MtDOT 2008)

Minnesota

In 1996, a TWT project was constructed on Lor Ray Drive, in North Mankato with other ultra-thin whitetopping projects (Vandenbossche and Rettner 1999). A 6 in. concrete overlay with 10 ft by 12 ft panels was constructed on the southbound lanes south of U.S. 14 and another 4.5 in. TWT with 5 ft by 6 ft panels was constructed on the northbound lanes on top of an 11.5 in., full-depth AC layer. The AC pavement was severely rutted.

After 3.5 years and approximately 4.7 million ESALs, the pavement section was evaluated. No noticeable distresses or reflection cracks were found. Few corner cracks occurred and those were very tight. Ride quality of this section was excellent (Vandenbossche and Fagerness 2002).

2.3 Summary

Thin whitetopping (TWT) is a highly promising rehabilitation technique for AC pavements. Performance of TWT in different states is good and it can significantly increase the service life of existing AC pavements. AASHTO Colorado, and Texas procedures provide guidelines for designing thin whitetopping. AASHTO offers an empirical method, whereas Colorado and Texas provide mechanistic-empirical methods for design. TWT requires proper pre-overlay treatments to produce the intended interface condition. Placement, finishing, and joint sawing are done in the same way as conventional concrete pavement construction.

CHAPTER 3 Methodology

3.1 Introduction

Complex engineering problems like composite pavements cannot be analyzed through analytical methods. An advanced numerical procedure is required to solve the problem as it consists of modeling multiple layers—concrete slab, asphalt layer, subgrade, and the interaction between asphalt and the concrete layer. The first step of any numerical procedure is discretization, which is the process of dividing the analysis model into a number of subregions and nodes called finite elements. The finite element method (FEM) creates algebraic equations using integral formulations, and a complete solution is produced by assembling individual solutions for each element (Moaveni 1999). Well-defined material properties are applied to each element to represent the model.

Cheung and Zienkiewicz (1965) were the first to apply FEM to analyze pavement. There are two-dimensional (2-D) finite element programs available for composite pavement analysis such as ISLAB2000, J-SLAB, KENPAVE, and FECONS (Cable et al. 2005). These programs use classical theories based on a Winkler foundation. Two-dimensional software requires less memory and run time, but traffic loading needs to be modeled as linear. For more accurate results, three-dimensional (3-D) software such as ABAQUS, ADINA, ANSYS and SolidWorks is available. In this study, the 3-D software package SolidWorks (version 2009) was used to investigate the response of thin whitetopping.

3.2 Finite Element Model

3.2.1 Model Geometry

Each pavement with whitetopping was modeled as a three-layer pavement system: consisting of the TWT, the existing AC layer, and the subgrade layer (Figure 3.1). A thin interlayer was used between the subgrade and the existing AC layer so that a variable friction between these layers could be studied (Dumitru 2006). Only one half of the loaded area was modeled, as the pavement geometry and loading are symmetric. Each pavement modeled was 12 ft long and 3 ft wide. The subgrade depth was 30 in. (to limit the size of the mesh), and 6-ft joint spacing was considered in the TWT layer. The built models were for the following:

1. Three whitetopping thicknesses: 5 in., 6 in., and 7.5 in.;
2. Three existing AC pavement thicknesses: 5 in., 7 in., and 9 in.;
3. Two bonding conditions: fully bonded and unbonded;
4. Two shoulder conditions: unpaved and paved; and
5. Two temperature differentials: 1.5⁰F/in. and 3⁰F/in.

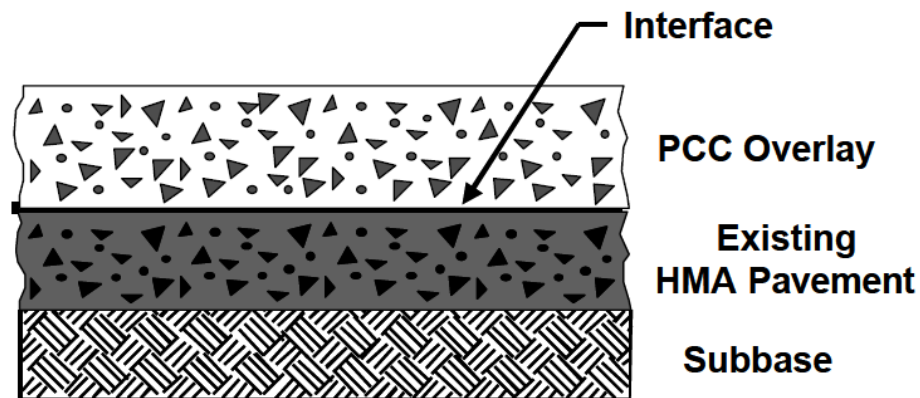


Figure 3.1 Three-Layer Pavement System (McGhee 1994)

3.2.2 Material Properties

All layer materials were considered isotropic and linear elastic except the interlayer between the subgrade and the AC layer. That interlayer was considered orthotropic having no stiffness in horizontal direction and the same properties of subgrade was provided in the vertical direction (Dumitru 2006). Properties of layer materials were as follows:

1. TWT:

Modulus of Elasticity = 4,000,000 psi

Poisson's Ratio = 0.15

Coefficient of Thermal Expansion = 5.5×10^{-6} (/°F)

2. AC layer:

Modulus of Elasticity = 250,000 and 350,000 psi

Poisson's Ratio = 0.4

Coefficient of Thermal Expansion = 13×10^{-6} (/°F)

3. Subgrade:

Modulus of Elasticity = 20,000 psi

Poisson's Ratio = 0.45

Coefficient of Thermal Expansion = 2×10^{-6} (/°F)

3.2.3 Model Meshing

Mesh size and mesh quality of the models affect the accuracy of the results. In general, a finer mesh gives more accurate results. A finer mesh results in a larger problem as it concerns size, which in turn results in higher computational effort and increased cost. To get better accuracy and to minimize project expenses, mesh size should be chosen in such a way that reasonably accurate results can be obtained at optimized computational

expenses. For this reason, convergence was checked to find out the optimum mesh size (Figure 3.2), and the TWT and AC layers were refined at their periphery (Figures 3.4a and 3.4b).

In this analysis, a high-quality mesh was used to get better results. Two types of meshing are available in SolidWorks (SolidWorks 2009): draft-quality mesh and high-quality mesh (Figure 3.3). The draft quality element has only four end nodes which provide a linear (1st order) displacement field. Thus, the strain field will be constant (0th order). As the stress is proportional to the strain, the stress field in a draft-quality mesh is also constant (0th order). The high-quality element has four corner nodes and six mid-edge nodes which can model a parabolic (2nd order) displacement field. Thus, the stress and strain fields are linear (1st order).

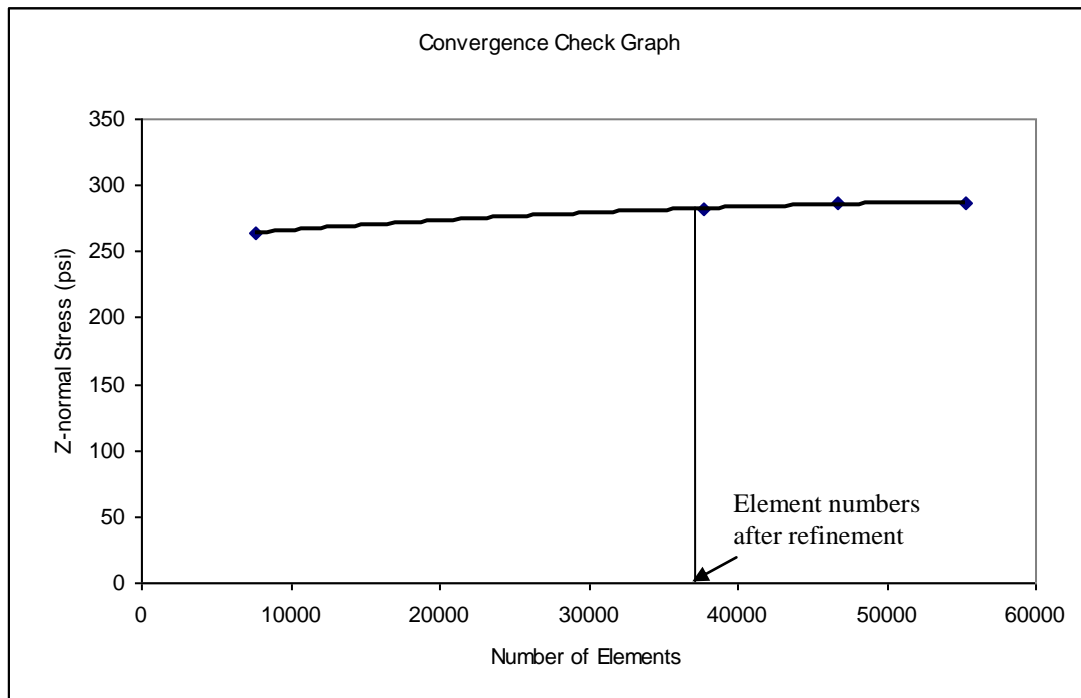


Figure 3.2 Convergence Check Using Z-Normal Stress as a Measure

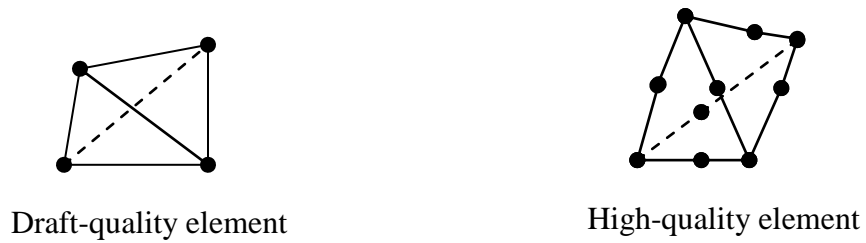


Figure 3.3 Draft-Quality and High-Quality Solid Element (after SolidWorks 2009)

3.2.4 Restraints

The bottom of the subgrade was fixed in all directions. Symmetry restraints were applied at all three directions of the model except on the left side (Figure 3.5a). That side was restrained in the direction of pavement width. All restraints of the model are shown in Figures 3.5a and 3.5b.

3.2.5 Model Loading

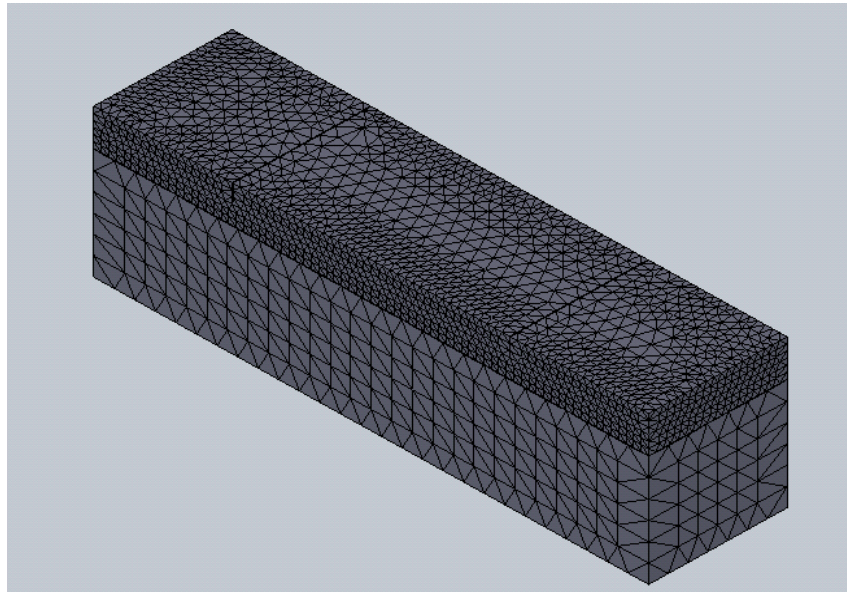
The typical model loaded area was assumed to be rectangular, constant over the applied surface and equal to the tire inflation pressure, as shown in Figure 3.6. The loading—20,000 lbs on a single axle with dual tires (legal load in Kansas)—was applied to the model as a pressure on the element faces (Dumitru 2006). Self-weight was considered for all layers.

Length of the contact rectangle L_x was computed from the tire width and inflation pressure.

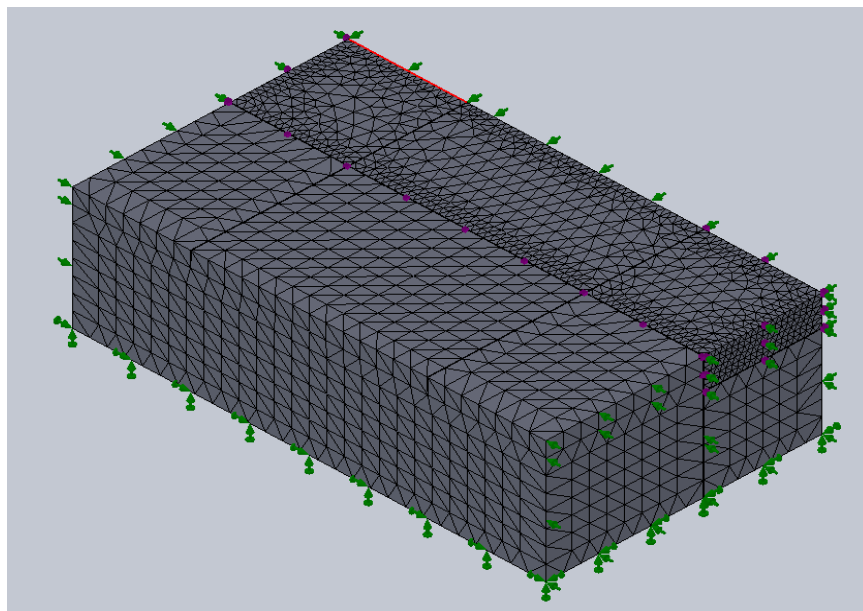
$$L_x = \frac{10,000}{2 \times 8 \times 100} = 6.25 \text{ in.}$$

The resulting SolidWorks model loading is shown in Figures 3.7a and 3.7b.

To observe the curling effect, temperature differentials of 1.5 °F/in and 3 °F/in were applied on the TWT surface. Figure 3.8 shows the application of the temperature differential. Temperature load was applied at the surface of TWT.

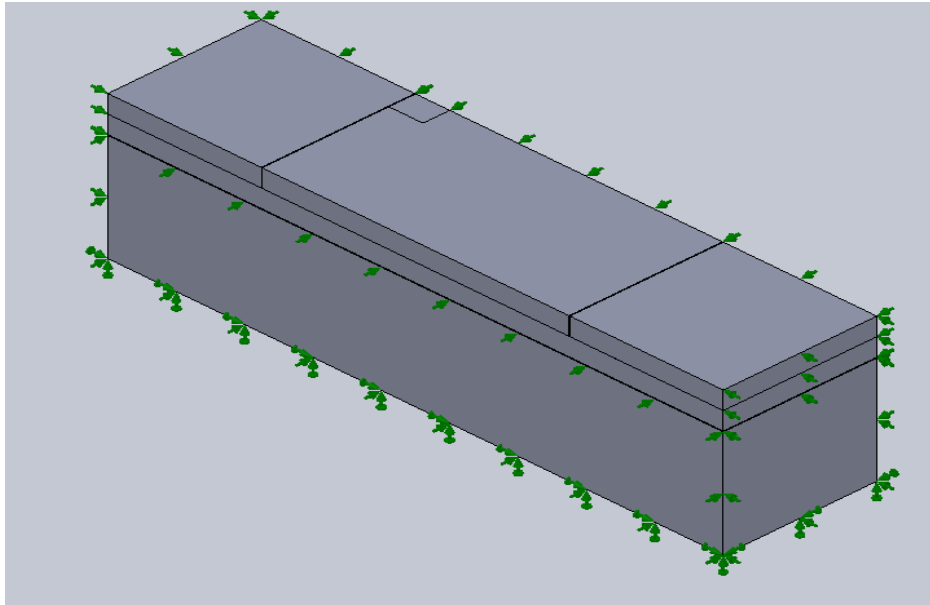


(a)

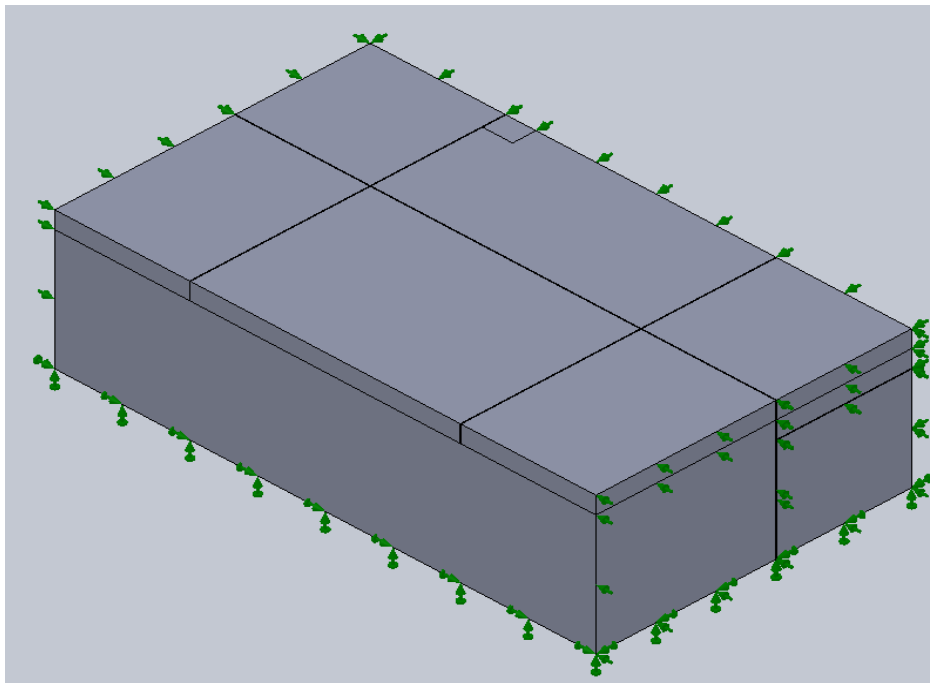


(b)

Figure 3.4 Model Mesh (a) No Paved Shoulder and (b) Paved Shoulder



(a)



(b)

Figure 3.5 Restraints of the Model (a) No Shoulder and (b) Shoulder

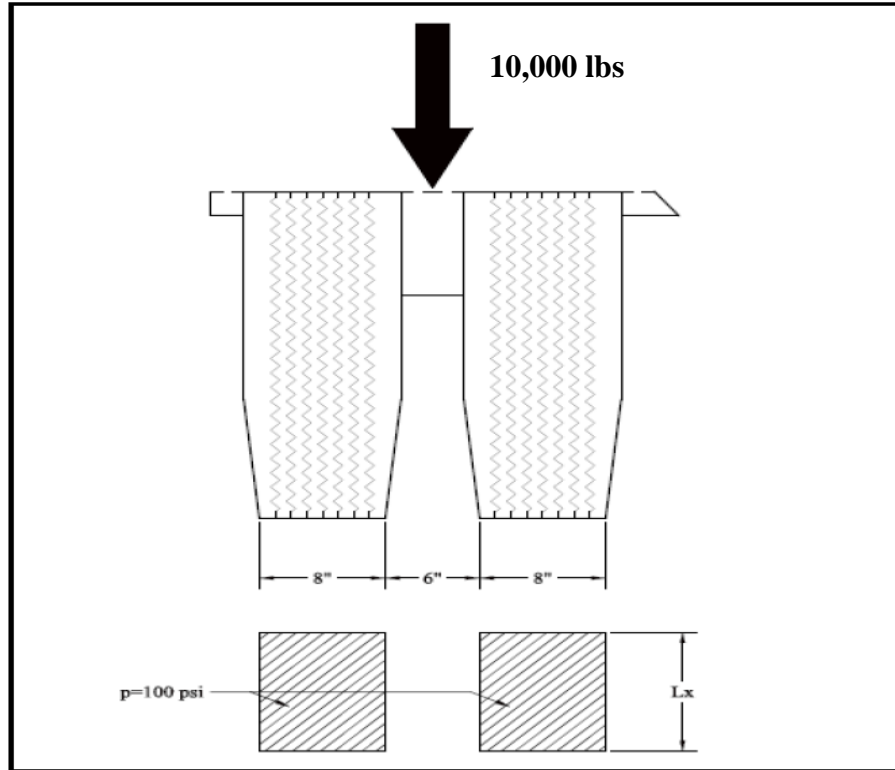
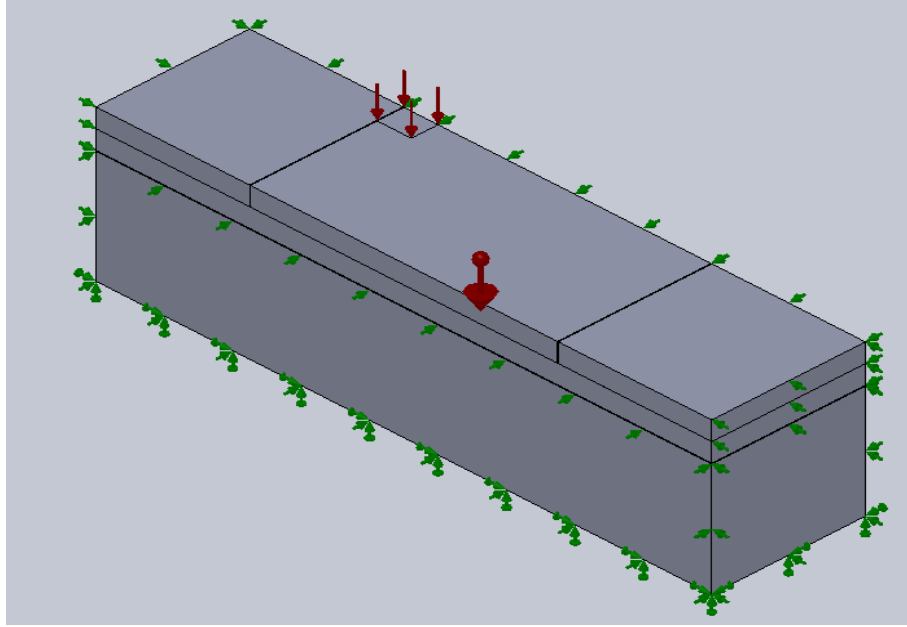
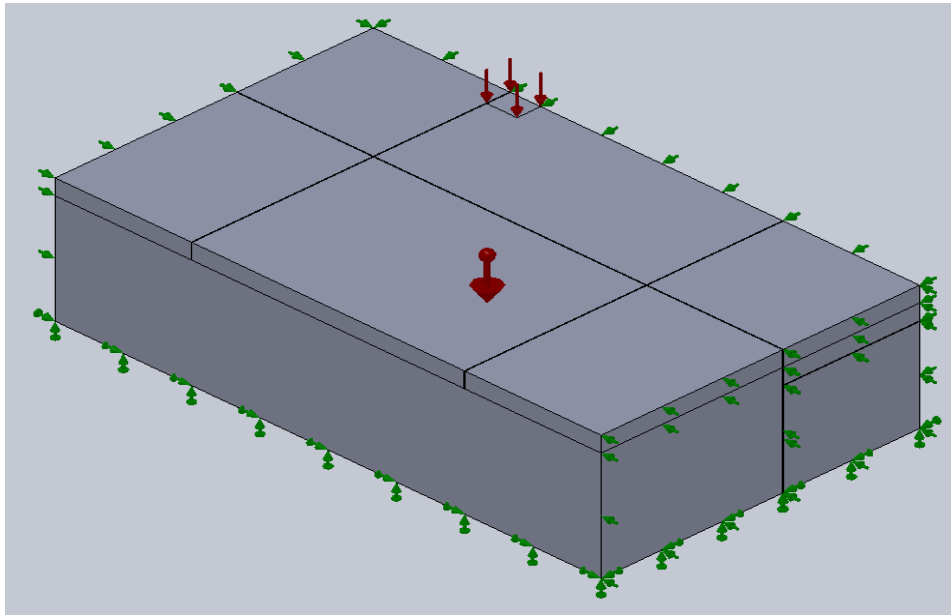


Figure 3.6 Load Model (after Dumitru 2006)



(a)



(b)

Figure 3.7 Loading Model in SW (a) No Shoulder and (b) Shoulder

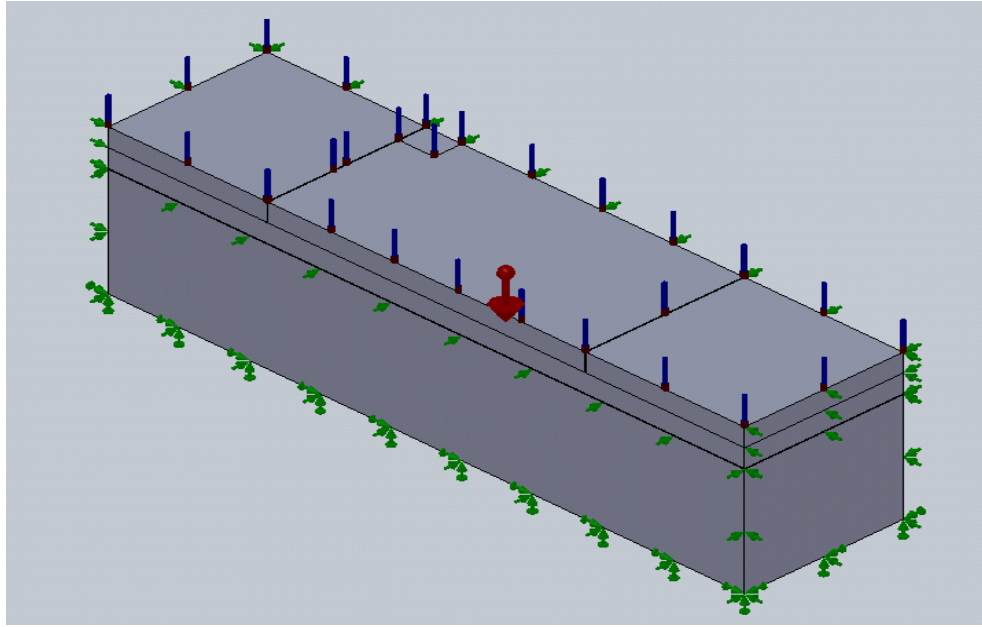


Figure 3.8 Loading Model for Temperature Differential

CHAPTER 4 Results and Discussions

4.1 Load-Induced Stresses

TWT has shorter joint spacings (4 ft–6 ft) compared to conventional concrete pavement that help in reducing concrete stresses. Thus, TWT behaves differently than conventional concrete pavement. The theoretical response (maximum transverse tensile stress) of TWT under the wheel load and varying temperature differentials was studied using the finite element model (FEM) in SolidWorks. The response was calculated at the bottom of the TWT layer. The stress calculation was done for different TWT thicknesses, existing AC thickness, AC modulus and for different bonding conditions between TWT and the existing AC layer. Maximum service lives of TWT were also estimated from the stress generated from the wheel loading. Contour plots of stress and deflection of TWT have been shown in Figures 4.1 and 4.2. Depending on the condition, tensile stress varied from 75 psi for bonded 7.5 in. TWT to as much as 442 psi for unbonded 5 in. TWT.

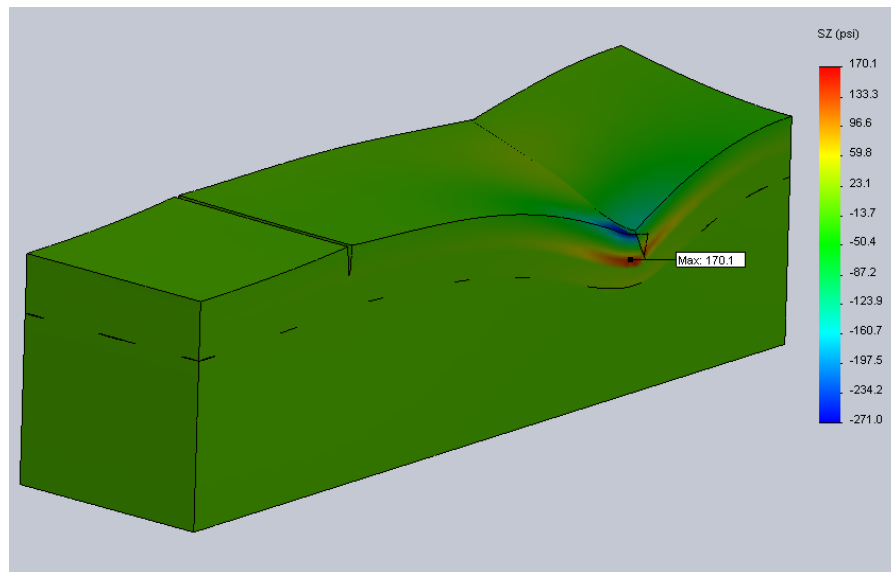


Figure 4.1 Transverse Stress Contour of TWT

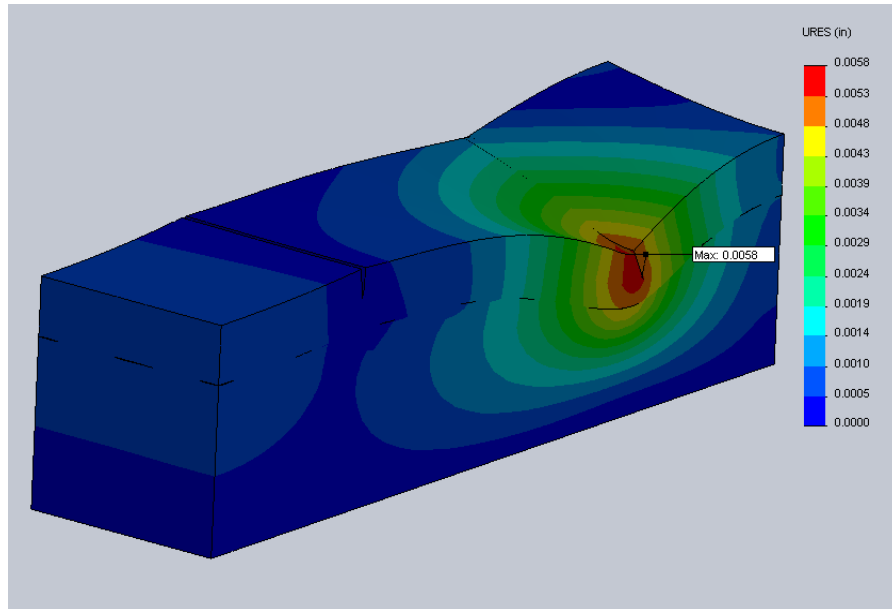
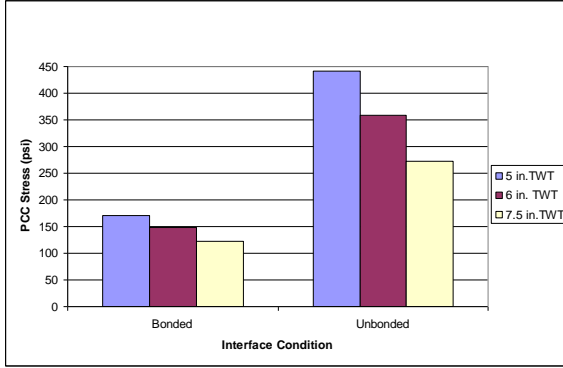


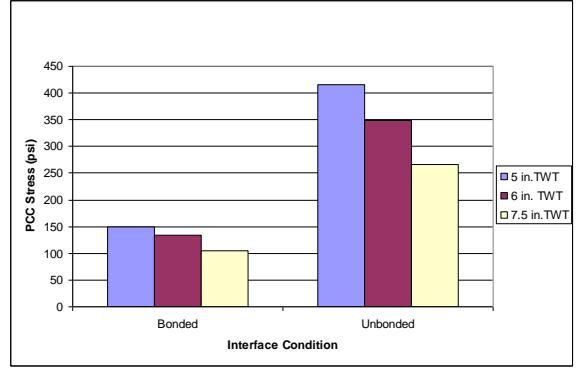
Figure 4.2 Deflection Contour of TWT

4.1.1 Effect of Interface Bonding Condition

The interface bonding condition between TWT and the existing AC layer is the most important factor affecting TWT behavior. For bonded TWT, the overlay and existing AC pavement act as a monolithic structure where the AC layer carries a significant portion of load. But unbonded TWT behaves as a new pavement, where the existing AC layer only provides a stable base. Load-induced stress for unbonded TWT is significantly higher than the bonded TWT. Figures 4.3 through 4.8 show the effect of bonding condition on PCC tensile stress. For 5 in. unpaved TWT on top of an AC layer of 5 in. thickness and 250 ksi modulus, transverse tensile stress for the bonded condition is 170 psi and for the unbonded condition is 442 psi (Figure 4.3a). Tensile stress increases about 160% for the change in interface condition from fully bonded to completely unbonded. TWT thickness has a more pronounced effect on an unbonded interface condition than a bonded condition.

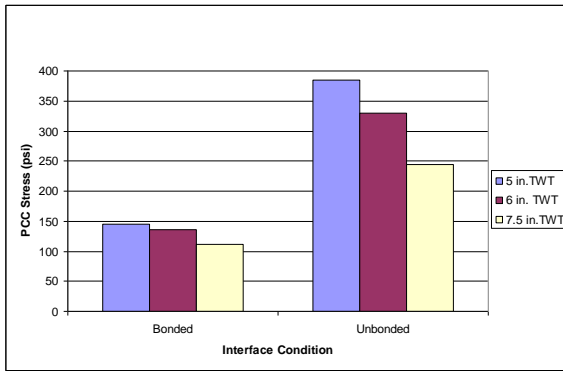


(a)

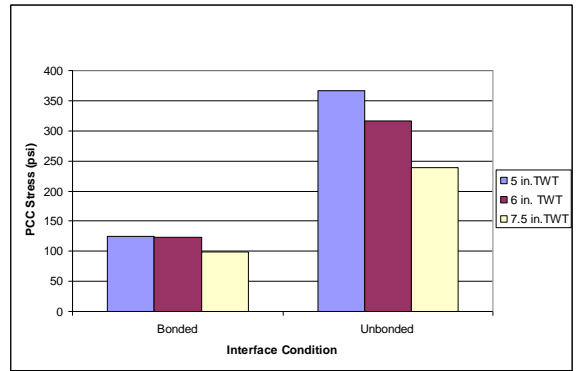


(b)

Figure 4.3 PCC Stress vs. Interface Bonding Condition Graph for 5 in. Existing AC Layer and Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

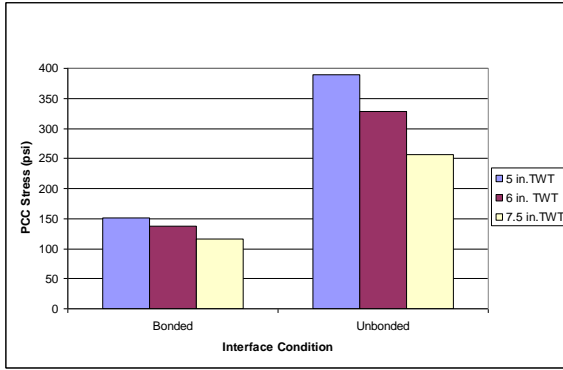


(a)

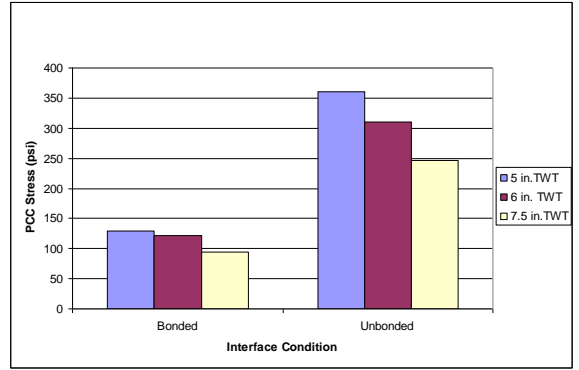


(b)

Figure 4.4 PCC Stress vs. Interface Bonding Condition Graph for 5 in. Existing AC Layer and Paved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

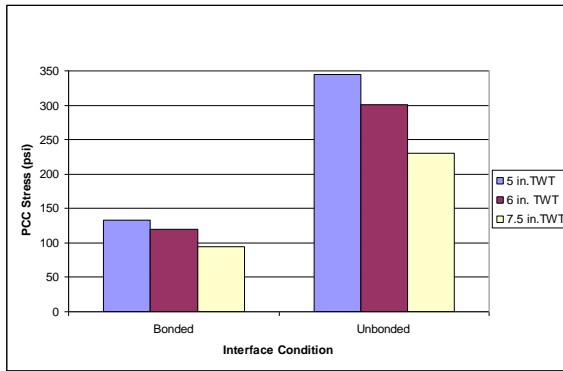


(a)

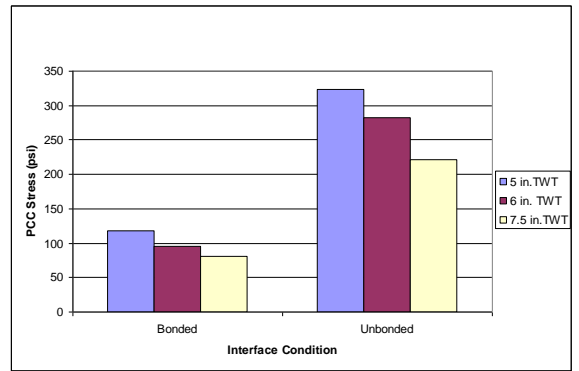


(b)

Figure 4.5 PCC Stress vs. Interface Bonding Condition Graph for 7 in. Existing AC Layer and Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

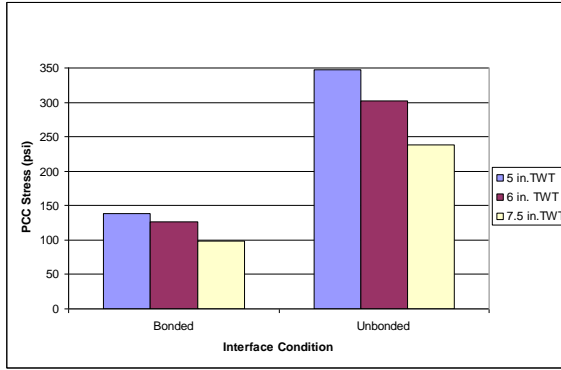


(a)

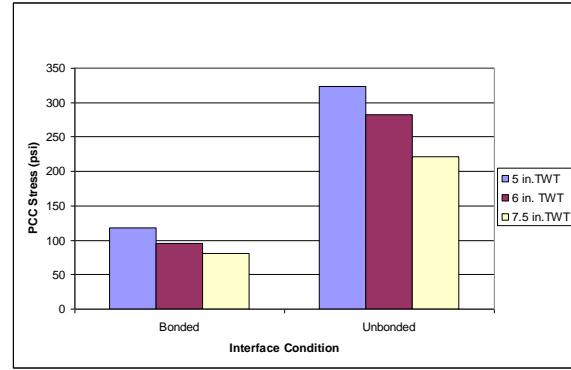


(b)

Figure 4.6 PCC Stress vs. Interface Bonding Condition Graph for 7 in. Existing AC Layer and Paved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

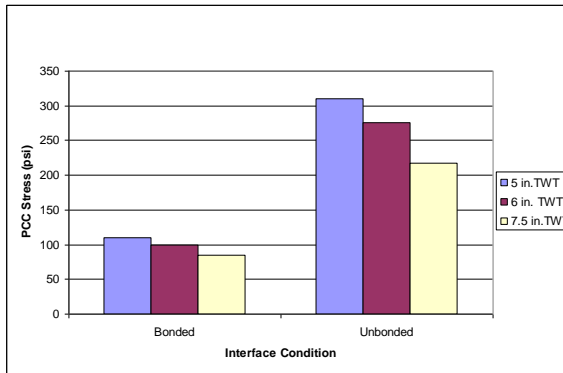


(a)

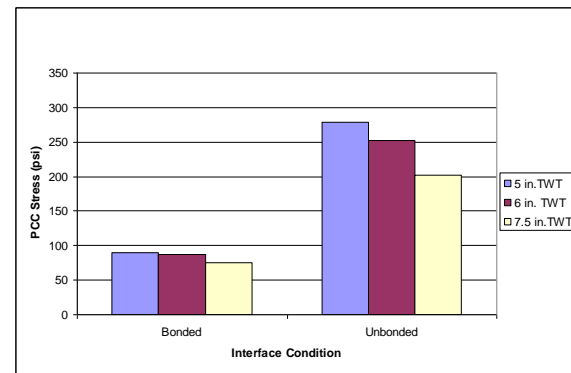


(b)

Figure 4.7 PCC Stress vs. Interface Bonding Condition Graph for 9 in. Existing AC Layer and Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi



(a)



(b)

Figure 4.8 PCC Stress vs. Interface Bonding Condition Graph for 9 in. Existing AC Layer and Paved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

Unbonded TWT induces much higher stress, but there is no mathematical relationship between bonded and unbonded concrete stresses. Colorado DOT (Sheehan et al. 2004) provided a relationship (Eq. 2.4 b) for bonded and partial-bonded whitetopping.

As interface bonding condition plays a major role in the behavior of TWT, partial bonding between the interfaces was also considered in this study. For this purpose, frictional coefficients other than 1 (for bonded condition) were assumed at the interface. Friction is a resistive force that occurs at the contact surface. The SolidWorks software calculates static friction forces by multiplying the normal forces generated at the contacting locations by the specified coefficient of friction. The direction of the friction force at that location is opposite to the direction of motion. Frictional coefficient value is specified for use with no penetration (unbonded) contact for static and nonlinear studies (SolidWorks 2009). The addition of frictional coefficient resulted in a considerable drop in transverse tensile stress for an unpaved shoulder condition, but did not show any significant effect for the paved shoulder condition. Figures 4.9 and 4.10 show the PCC stress variation for different frictional coefficients used at the interface for unpaved and paved TWT. For frictional coefficients 1 (fully bonded), 0.75, 0.5, 0.25 and 0 (unbonded), tensile stresses for 5 in. unpaved TWT on a 5 in. AC layer and modulus of 250 ksi, were found as 170, 393, 401, 422, and 442 psi, respectively. For frictional coefficients 1 (fully bonded), 0.75, 0.5, 0.25 and 0 (unbonded), tensile stresses for a 5 in. paved TWT on 5 in. AC layer and modulus of 250 ksi were found as 145, 371, 373, 375, and 385 psi, respectively.

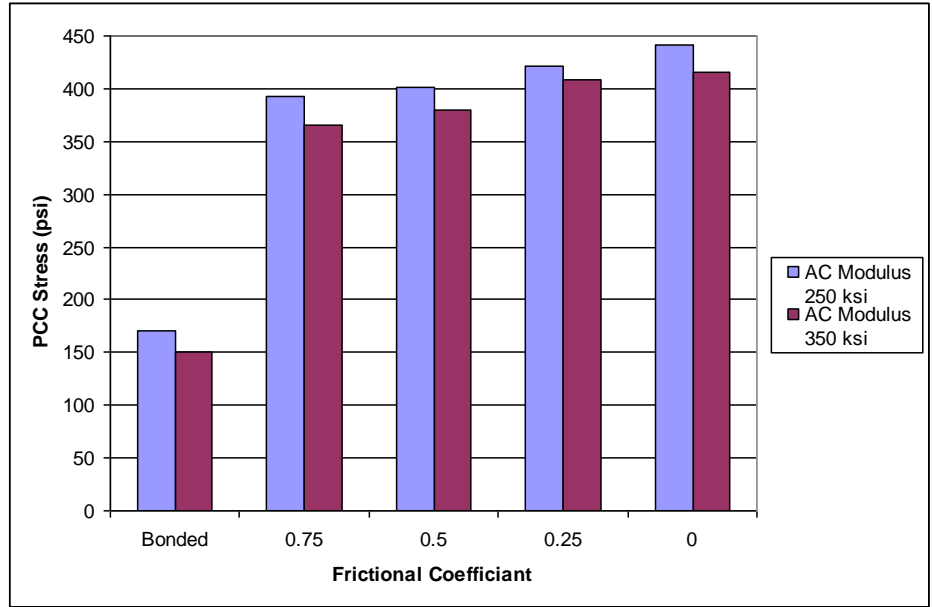


Figure 4.9 PCC Stress vs. Frictional Coefficient Graph for Unpaved TWT

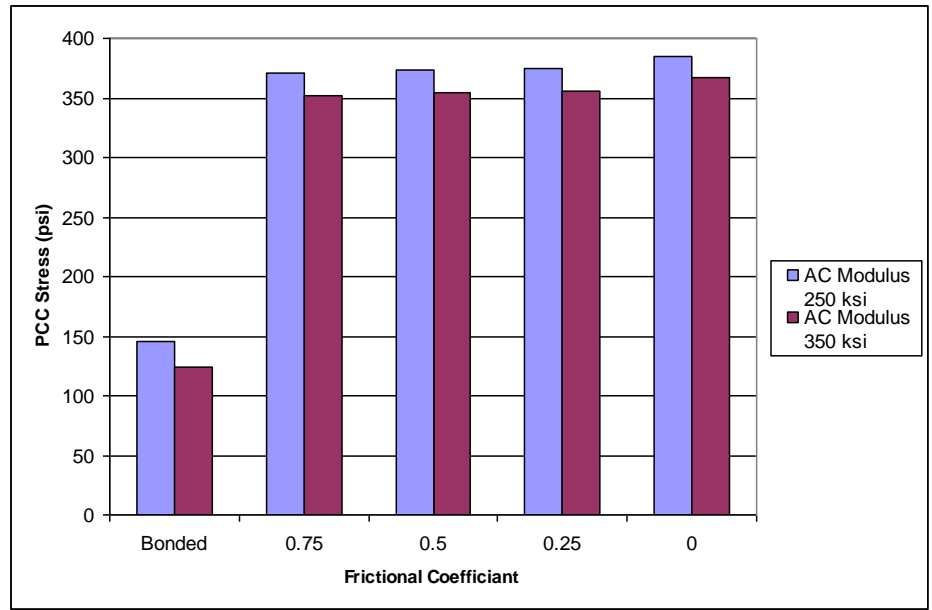


Figure 4.10 PCC Stress vs. Frictional Coefficient Graph for Paved TWT

4.1.2 Effect of Thin Whitetopping Thickness

Figures 4.11 through 4.14 show the variation of PCC tensile stress with TWT thickness. Tensile stress significantly decreases with an increase in TWT thickness. For unpaved and unbonded 5 in., 6 in., and 7.5 in. TWT (Figure 4.12a) on top of an AC layer of 5 in. and modulus of 250 ksi, tensile stresses were found as 442, 359 and 272 psi, respectively. Changes in stress were found to be from 23% to 32% for unbonded TWT and 14% to 20% for bonded TWT. For unbonded TWT and an unpaved shoulder, the effect of TWT thickness is more pronounced than other conditions.

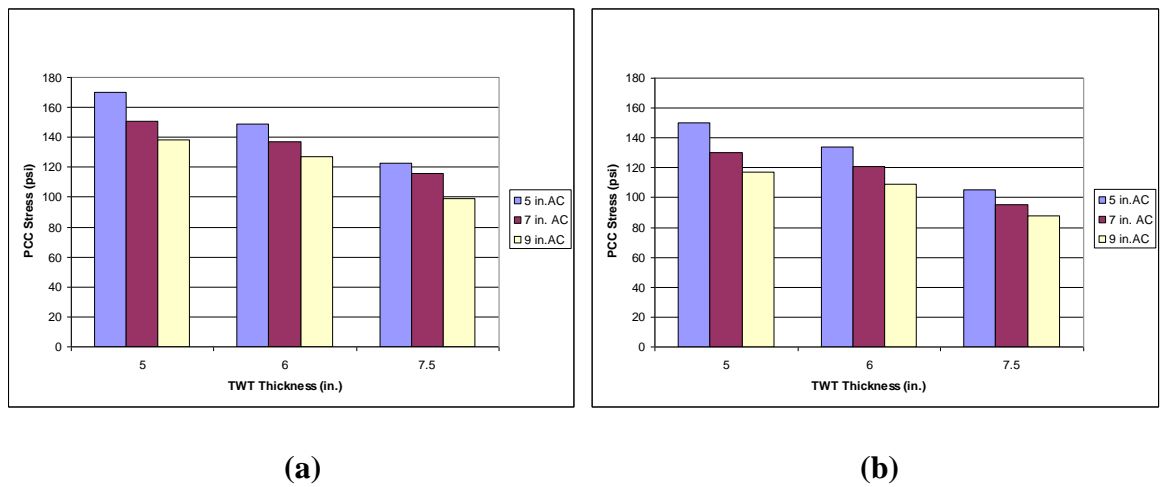
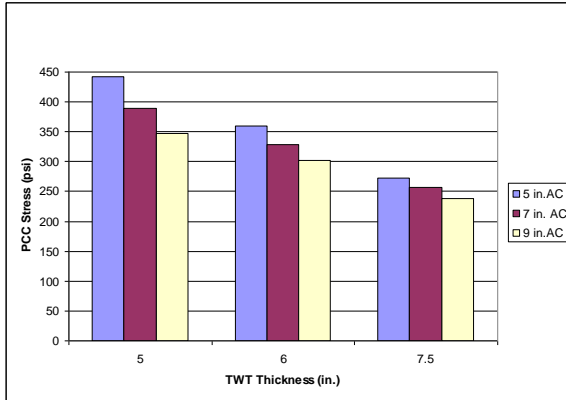
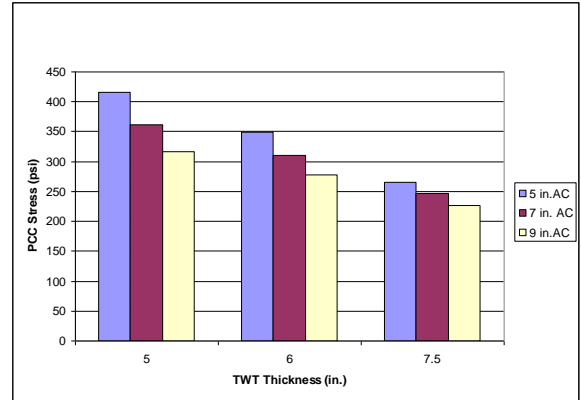


Figure 4.11 PCC Stress vs. TWT Thickness Graph for Bonded Condition and Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

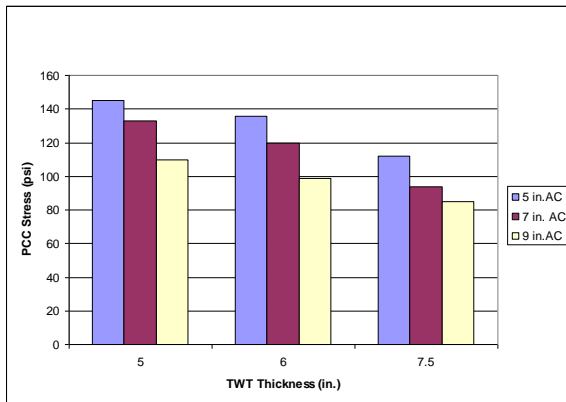


(a)

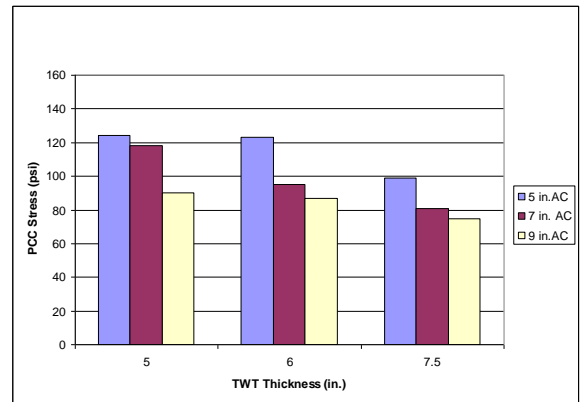


(b)

Figure 4.12 PCC Stress vs. TWT Thickness Graph for Unbonded Condition and Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi



(a)



(b)

Figure 4.13 PCC Stress vs. TWT Thickness Graph for Bonded Condition and Paved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

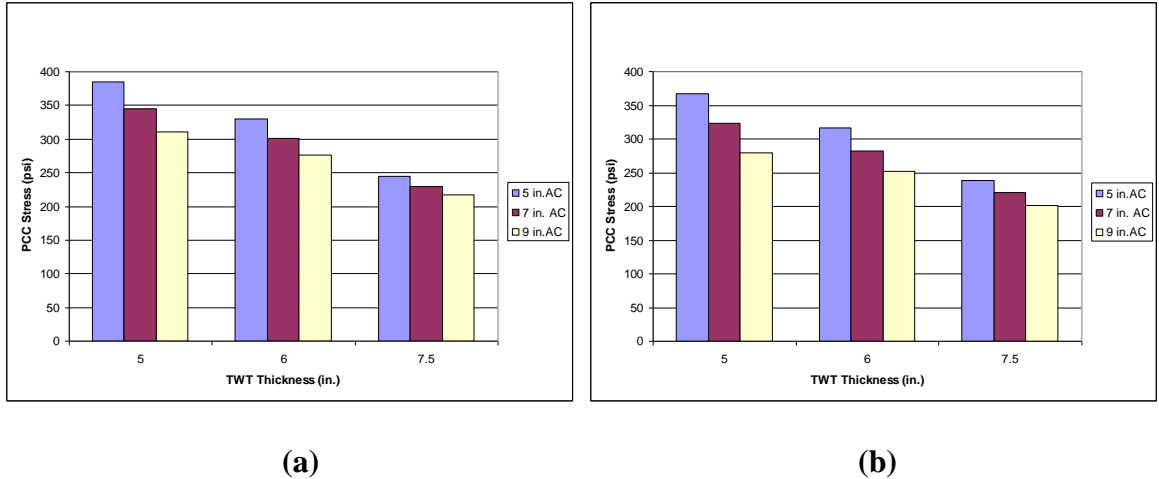
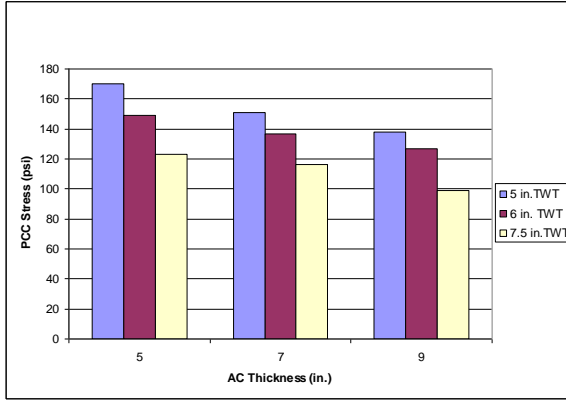


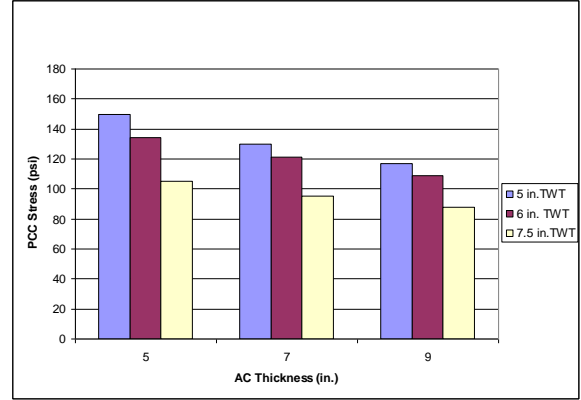
Figure 4.14 PCC Stress vs. TWT Thickness Graph for Unbonded Condition and Paved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

4.1.3 Effect of Existing AC Thickness

Figures 4.15 through 4.18 show the variation of PCC stress with the existing AC thickness. PCC stress decreases with the increase of AC thickness, as higher AC thickness enhances the underlying support to TWT. For 5 in. unbonded TWT and an unpaved shoulder condition, the tensile stresses for 5 in., 7 in. and 9 in. AC layers with 250 ksi modulus were observed as 442, 389, and 348 psi, respectively (Figure 4.16a). Tensile stresses decrease about 12% to 14% for a 2 in. increase in AC thickness. The existing AC thickness has a more pronounced effect on the stresses for 5 in. TWT than 6 in. and 7.5 in. TWTs.

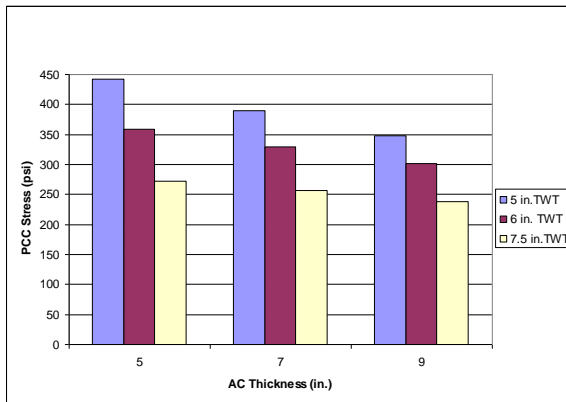


(a)

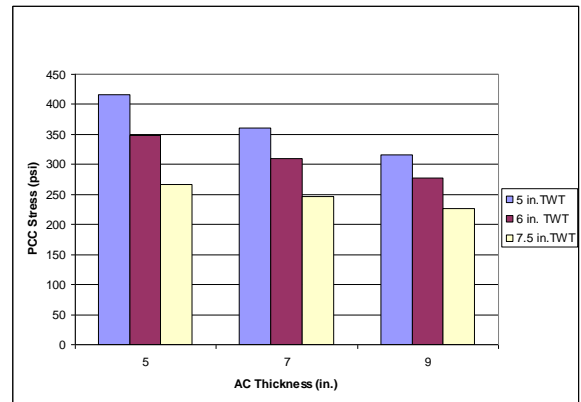


(b)

Figure 4.15 PCC Stress vs. Existing AC Thickness Graph for Bonded Condition and Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

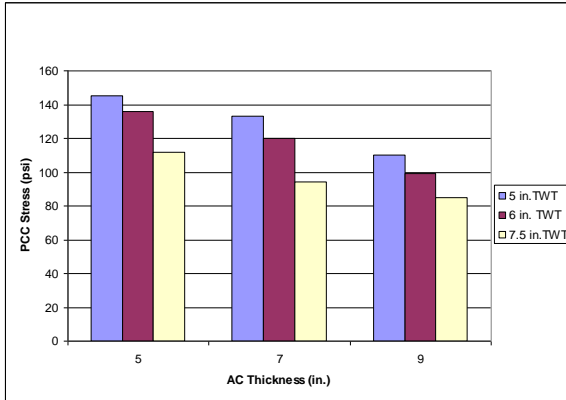


(a)

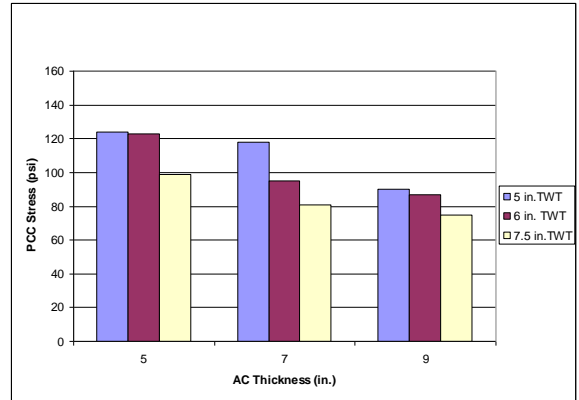


(b)

Figure 4.16 PCC Stress vs. Existing AC Thickness Graph for Unbonded Condition and Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

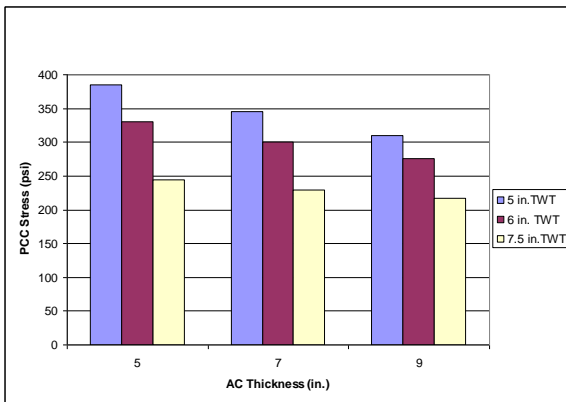


(a)

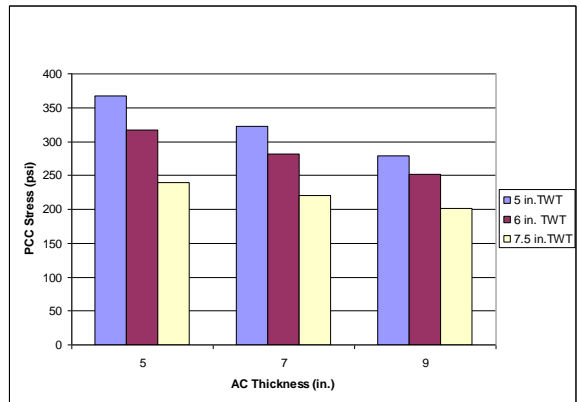


(b)

Figure 4.17 PCC Stress vs. Existing AC Thickness Graph for Bonded Condition and Paved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi



(a)



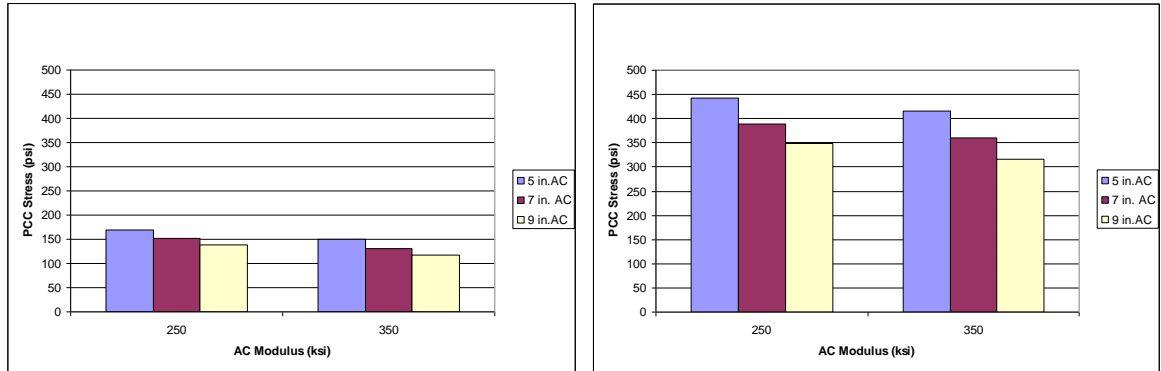
(b)

Figure 4.18 PCC Stress vs. Existing AC Thickness Graph for Unbonded Condition and Paved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

4.1.4 Effect of AC Modulus

The existing AC modulus is representative of the existing AC pavement condition. PCC stress versus AC modulus plots for different parameters have been shown in Figures

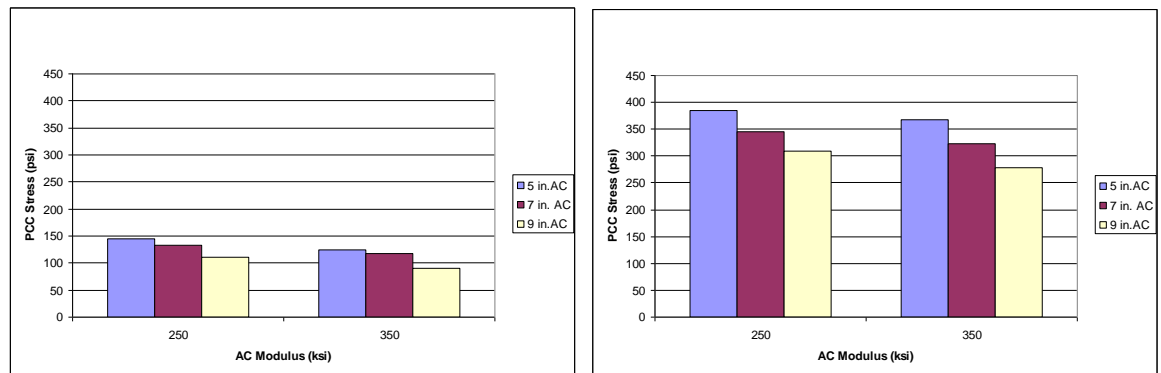
4.19 through 4.24. With the increase in AC modulus, PCC stress decreases. The AC modulus has less effect on unbonded PCC stress than on the bonded. For 250 ksi and 350 ksi AC moduli, tensile stresses were observed to be 442 psi and 416 psi for 5 in. unbonded TWT, 5 in. AC layer, and the unpaved shoulder condition (Figure 4.19b). The stress change due to AC modulus was about 6% for unbonded overlay and 14% for bonded overlay.



(a)

(b)

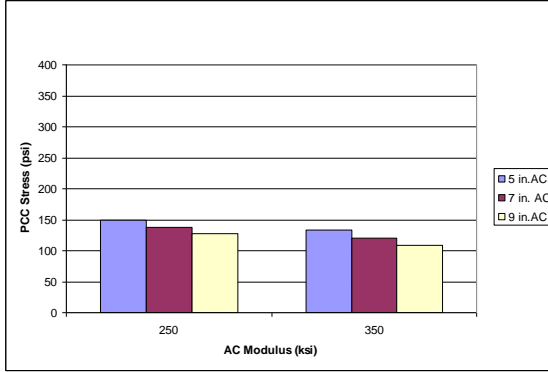
Figure 4.19 PCC Stress vs. Existing AC Modulus Graph for 5 in. TWT and Unpaved Shoulder (a) Bonded and (b) Unbonded



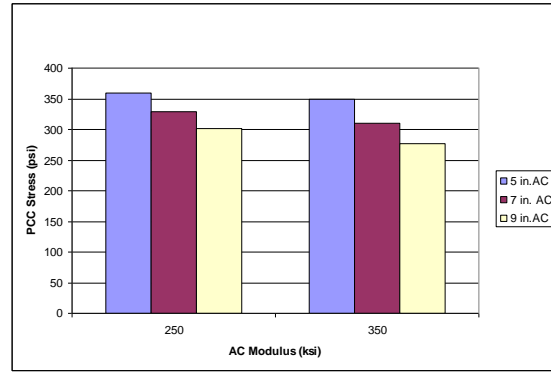
(a)

(b)

Figure 4.20 PCC Stress vs. Existing AC Modulus Graph for 5 in. TWT and Paved Shoulder (a) Bonded and (b) Unbonded

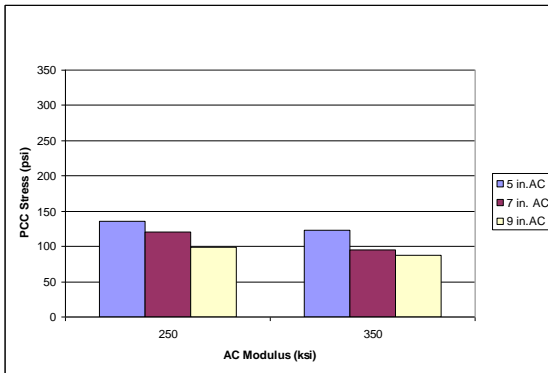


(a)

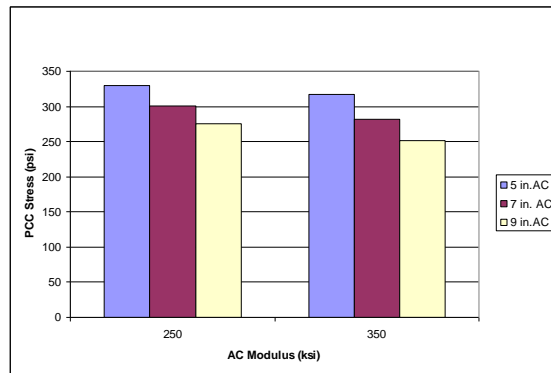


(b)

Figure 4.21 PCC Stress vs. Existing AC Modulus Graph for 6 in. TWT and Unpaved Shoulder (a) Bonded and (b) Unbonded

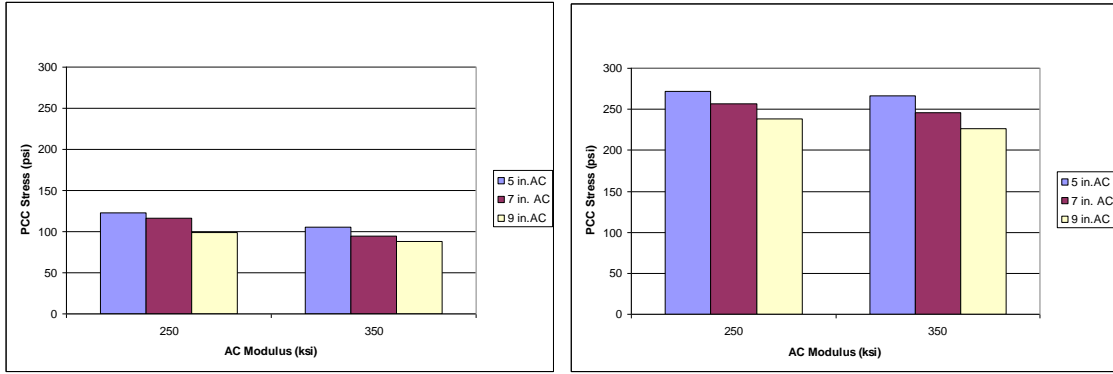


(a)



(b)

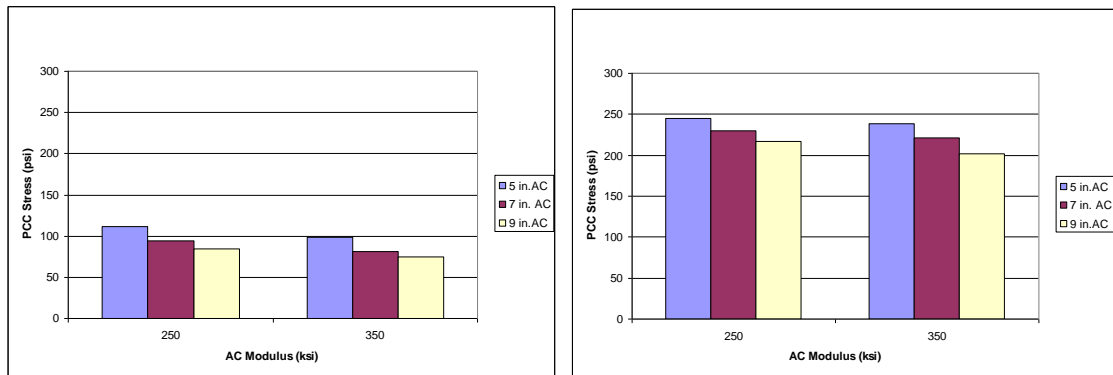
Figure 4.22 PCC Stress vs. Existing AC Modulus Graph for 6 in. TWT and Paved Shoulder (a) Bonded and (b) Unbonded



(a)

(b)

Figure 4.23 PCC Stress vs. Existing AC Modulus Graph for 7.5 in. TWT and Unpaved Shoulder (a) Bonded and (b) Unbonded



(a)

(b)

Figure 4.24 PCC Stress vs. Existing AC Modulus Graph for 7.5 in. TWT and Paved Shoulder (a) Bonded and (b) Unbonded

4.1.5 Effect of Shoulder Condition

A paved shoulder provides lateral support to pavement; thus, the addition of a paved shoulder decreases the transverse tensile stress of TWT. This phenomenon has been shown with Figures 4.25 to 4.30. For unbonded 5 in. TWT, the 5 in. AC layer, and an AC

modulus of 250 ksi, tensile stresses were found as 442 and 385 psi, respectively, for the unpaved and paved shoulders (Figure 4.26a). The difference of about 15% was found for the change in shoulder condition.

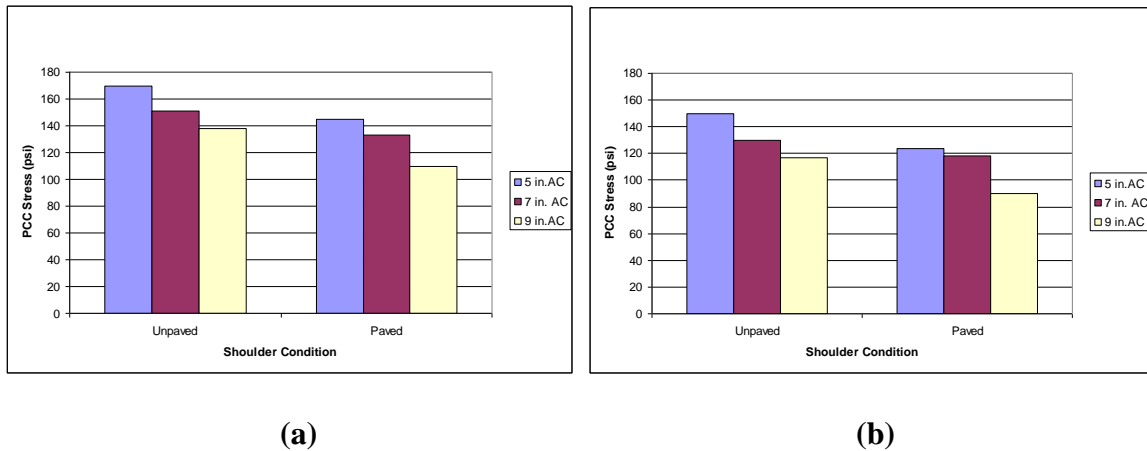


Figure 4.25 PCC Stress vs. Shoulder Condition Graph for 5 in. Bonded TWT (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

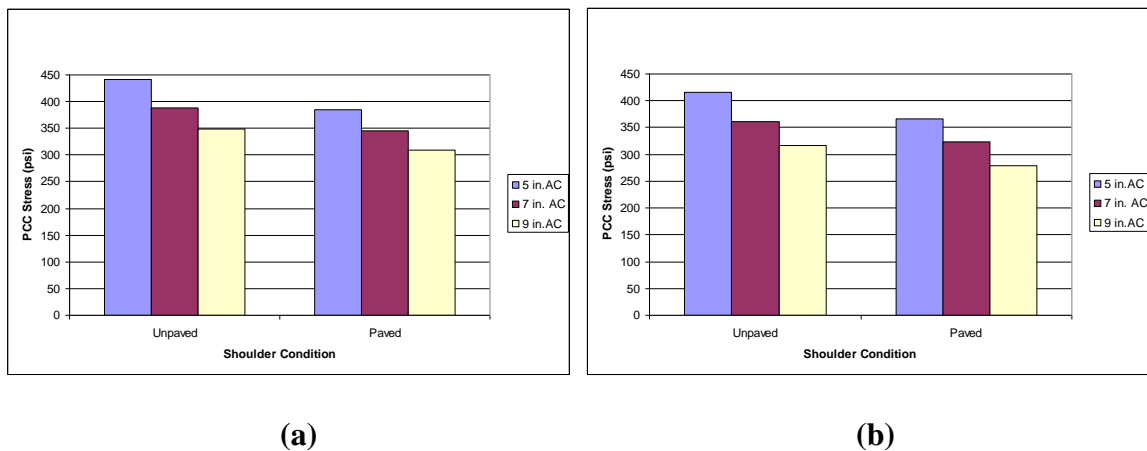
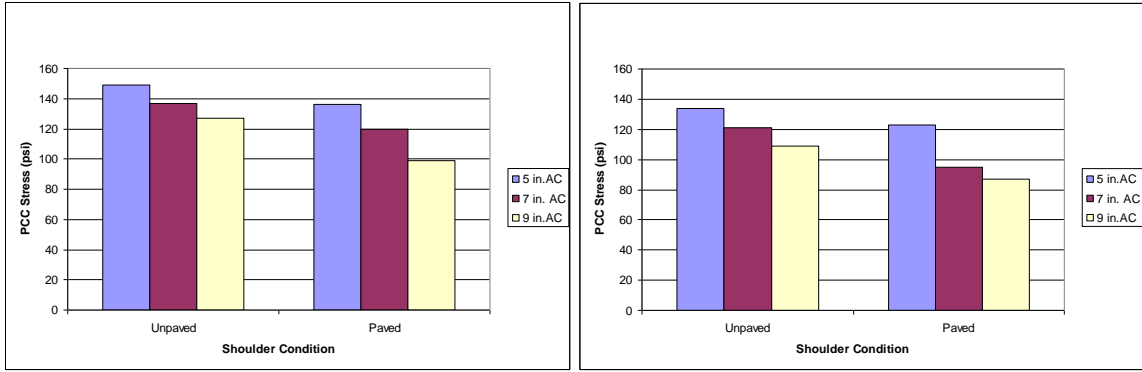


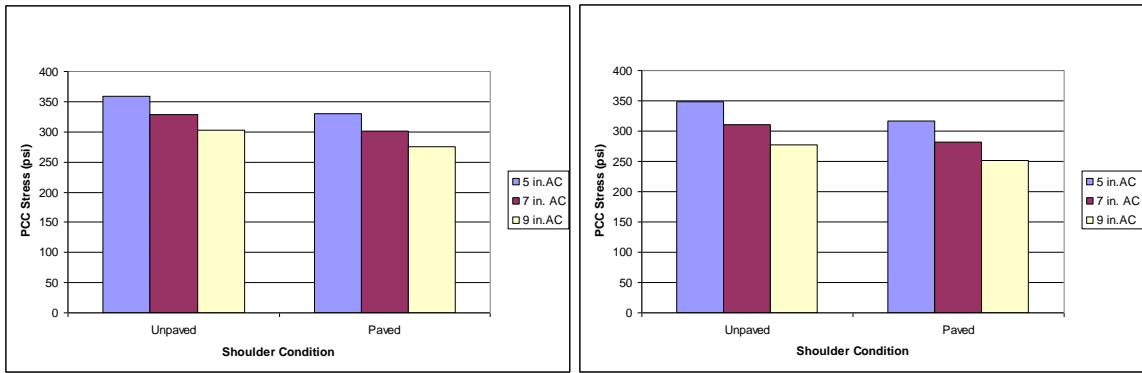
Figure 4.26 PCC Stress vs. Shoulder Condition Graph for 5 in. Unbonded TWT (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi



(a)

(b)

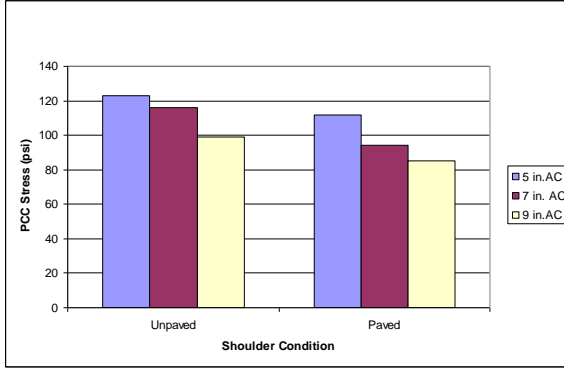
Figure 4.27 PCC Stress vs. Shoulder Condition Graph for 6 in. Bonded TWT (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi



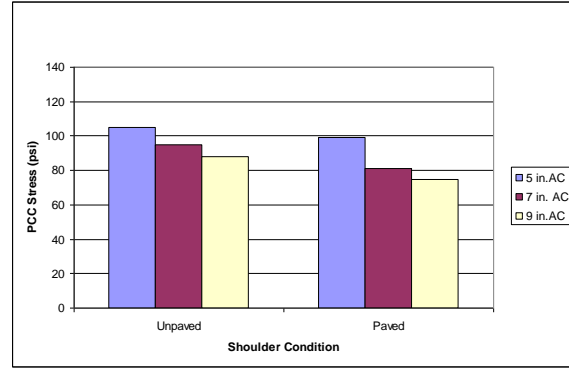
(a)

(b)

Figure 4.28 PCC Stress vs. Shoulder Condition Graph for 6 in. Unbonded TWT (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

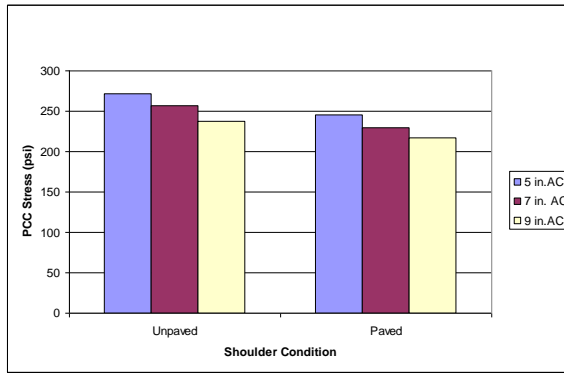


(a)



(b)

Figure 4.29 PCC Stress vs. Shoulder Condition Graph for 7.5 in. Bonded TWT (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi



(a)



(b)

Figure 4.30 PCC Stress vs. Shoulder Condition Graph for 7.5 in. Unbonded TWT (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi

4.2 Curling Effect

The curling effect for temperature differentials of 1.5⁰F/in and 3⁰F/in on TWT were also examined. The contour plot of curling stress has been shown in Figure 4.11. Curling stresses for different TWT thicknesses have been shown in Figures 4.12 and 4.13. The

stresses do not change greatly with the change of existing AC thicknesses. From the figures, it appears with the increase of TWT thickness, the curling stress increases and the bonding condition and AC modulus have no effect on curling stress. For 5 in., 6 in. and 7.5 in. TWT, curling stresses for 1.5⁰F/in temperature differential were found to be 83, 94, and 120 psi, respectively. The change in curling stress due to TWT thickness change is about 14% to 30%.

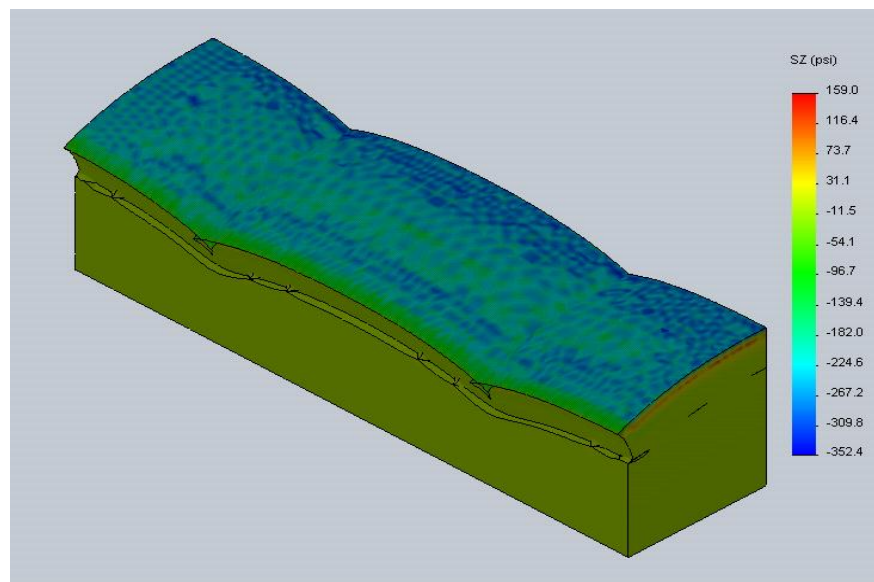
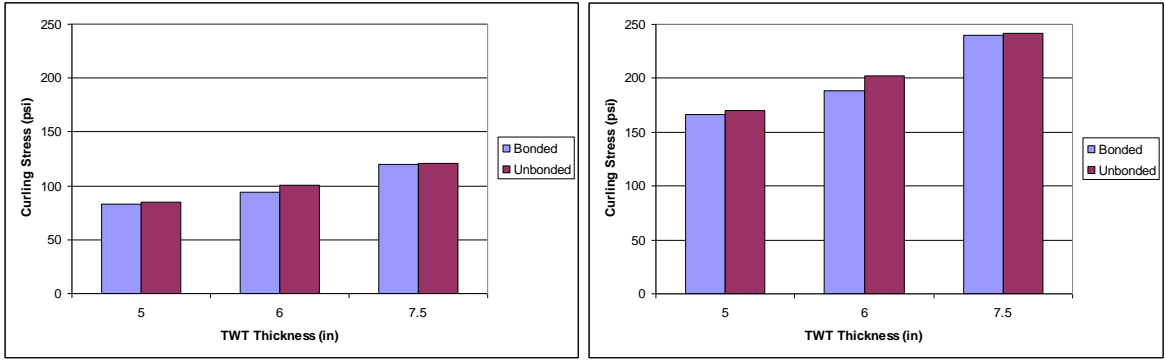


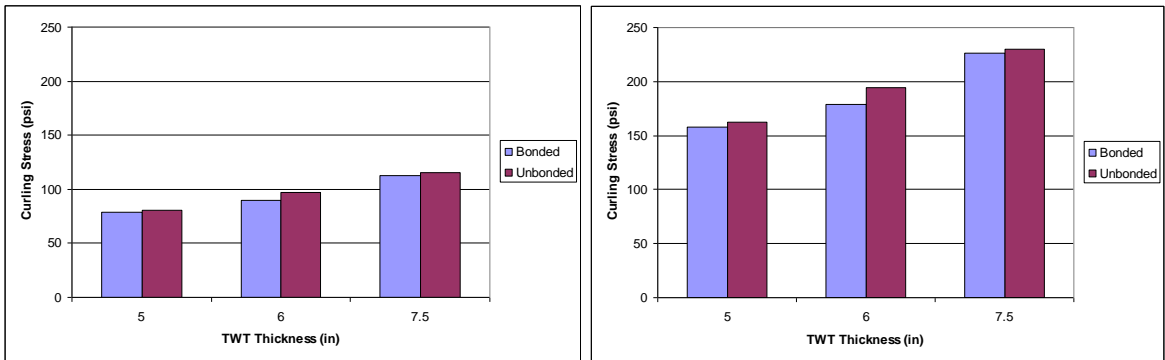
Figure 4.31 Contour Plot of Curling Stress



(a)

(b)

Figure 4.32 PCC Stress vs. TWT Thickness for AC Modulus 250 ksi (a) Temp. Diff. of 1.5⁰F/in and (b) Temp. Diff. 3⁰F/in



(a)

(b)

Figure 4.33 PCC Stress vs. TWT Thickness for AC Modulus 350 ksi (a) Temp. Diff. of 1.5⁰F/in and (b) Temp. Diff. 3⁰F/in

4.3 Verification of the FE Model

As this study is based on numerical work, the finite element model needs to be independently verified. For this purpose, stresses obtained from SolidWorks were compared with those from ANSYS and KENSLABS. KENSLABS is a two-dimensional finite element software which permits the analysis of rigid pavement response under static

load (Huang 2004). It is a part of a computer package called KENPAVE. The package has an input program SLABSINP and graphic programs SGRAPH and CONTOUR.

Table 4.4 shows the comparison of stresses obtained from SolidWorks and ANSYS. For different TWT thicknesses, 14 to 19% differences in stresses were found. The employed version of ANSYS limits the user's creation to a mesh with up to 32,000 nodes; however, in SolidWorks more than 50,000 nodes were used to create mesh. The use of higher nodes in SolidWorks resulted in differences in calculated PCC stresses from ANSYS.

Table 4.5 shows the comparison of stresses obtained from SolidWorks and KENSLABS. The results indicated 20 to 25% differences for different TWT thicknesses. For the two layers model, KENSLABS ignores the properties of the bottom/foundation layer. In the current study, KENSLABS cannot take into account the load-carrying capacity of the existing AC layer; thus, it shows higher PCC stresses than that obtained in SolidWorks.

Table 4.1 Comparison of Stresses Obtained from SolidWorks and ANSYS

TWT (in.)	PCC Stress (psi)		% Difference
	SolidWorks	ANSYS	
5	170	147	14
6	149	121	19

Table 4.2 Comparison of Stresses Obtained from SolidWorks and KENSLABS

TWT (in.)	PCC Stress (psi)		% Difference
	SolidWorks	KENSLABS	
5	170	204	20
6	149	187	25

4.4 Service Lives and Design Catalog

According to the PCA fatigue criterion (PCA 1984), allowable load repetitions (N) were calculated based on the stress ratio of the concrete. The stress ratio (SR) is the ratio of the PCC tensile stress to the concrete modulus of rupture.

For $SR > 0.55$

$$\log_{10}(N) = \frac{0.97187 - SR}{0.0828}$$

For $0.45 \leq SR \leq 0.55$

$$N = \left(\frac{4.2577}{SR - 0.43248} \right)^{3.268}$$

For $SR < 0.45$

N = Unlimited

Where,

SR = flexural stress to strength (modulus of rupture of concrete) ratio, and

N = the number of allowable load repetitions

From the allowable load repetitions, the design life can be estimated. The maximum service life of a whitetopping was taken as 10 years, under the consideration of durability. For different parameters considered in this study, Tables 4.6, 4.7 and 4.8 represent the design catalogs of TWT for 0, 3% and 6% traffic growth, respectively. The results show that for fully bonded conditions of all TWT and AC layer thicknesses, TWT is expected to last through the 10 years assumed in design. But unbonded TWT with unpaved shoulder results in a catastrophic loss of pavement life. In the design catalogs, design parameter combinations in the shaded cells are not recommended because their calculated service lives were less than 10 years.

Table 4.3 Design Catalog for Thin Whitetopping for No Traffic Growth

AC Thickness (in)	AC Modulus (ksi)	Bonding Condition	Shoulder Condition	TWT Thickness* (in)	Service Life (Years)			
					Daily Truck Traffic** per Lane			
					≤200	300	400	500
5	250	Bonded	Unpaved	5	10	10	10	10
7				10	10	10	10	
9				10	10	10	10	
5	250	Unbonded	Unpaved	5	0.06	0.04	0.03	0.02
				6	2.13	1.42	1.07	0.85
				7.5	10	10	10	10
5	350	Unbonded	Unpaved	5	0.19	0.12	0.09	0.07
				6	3.34	2.23	1.67	1.34
				7.5	10	10	10	10
5	250	Unbonded	Paved	5	0.7	0.47	0.35	0.28
				6	9.8	6.52	4.89	3.91
				7.5	10	10	10	10
5	350	Unbonded	Paved	5	1.5	1.01	0.76	0.61
				6	10	10	10	10
				7.5	10	10	10	10
7	250	Unbonded	Unpaved	5	0.6	0.4	0.3	0.2
				6	10	7	5.2	4.2
				7.5	10	10	10	10
7	350	Unbonded	Unpaved	5	2	1.3	1	0.8
				6	10	10	10	10
				7.5	10	10	10	10
7	250	Unbonded	Paved	5	4.1	2.7	2	1.6
				6	10	10	10	10
				7.5	10	10	10	10
7	350	Unbonded	Paved	5	10	10	8.1	6.5
				6	10	10	10	10
				7.5	10	10	10	10
9	250	Unbonded	Unpaved	5	3.5	2.3	1.8	1.4
				6	10	10	10	10
				7.5	10	10	10	10
9	350	Unbonded	Unpaved	5	10	10	10	10
				6	10	10	10	10
				7.5	10	10	10	10
9	250	Unbonded	Paved	5	10	10	10	10
				6	10	10	10	10
				7.5	10	10	10	10
9	350	Unbonded	Paved	5	10	10	10	10
				6	10	10	10	10
				7.5	10	10	10	10

* Concrete Modulus of Rupture = 650 psi ** Truck Factor = 1.5

Table 4.4 Design Catalog for Thin Whitetopping for 3% Traffic Growth

AC Thickness (in)	AC Modulus (ksi)	Bonding Condition	Shoulder Condition	TWT Thickness* (in)	Service Life (Years)			
					Daily Truck Traffic** per Lane			
					≤200	300	400	500
5	250	Bonded	Unpaved	5	10	10	10	10
7				10	10	10	10	
9				10	10	10	10	
5	350	Unbonded	Paved	5	10	10	10	10
				6	10	10	10	10
				7.5	10	10	10	10
5	250	Unbonded	Unpaved	5	0.01	0.00	0.00	0.00
				6	0.19	0.12	0.09	0.07
				7.5	10	10	10	10
5	350	Unbonded	Unpaved	5	0.02	0.01	0.01	0.01
				6	0.29	0.19	0.15	0.12
				7.5	10	10	10	10
5	250	Unbonded	Paved	5	0.1	0.04	0.03	0.02
				6	0.9	0.57	0.43	0.34
				7.5	10	10	10	10
5	350	Unbonded	Paved	5	0.1	0.09	0.07	0.05
				6	2.3	1.56	1.17	0.94
				7.5	10	10	10	10
7	250	Unbonded	Unpaved	5	0.1	0.00	0.00	0.00
				6	0.9	0.6	0.5	0.4
				7.5	10	10	10	10
7	350	Unbonded	Unpaved	5	0.2	0.1	0.1	0.1
				6	4.8	3.2	2.4	1.9
				7.5	10	10	10	10
7	250	Unbonded	Paved	5	0.4	0.2	0.2	0.1
				6	10	10	8.1	6.4
				7.5	10	10	10	10
7	350	Unbonded	Paved	5	1.4	0.9	0.7	0.6
				6	10	10	10	10
				7.5	10	10	10	10
9	250	Unbonded	Unpaved	5	0.3	0.2	0.2	0.1
				6	10	9.2	6.9	5.5
				7.5	10	10	10	10
9	350	Unbonded	Unpaved	5	2.6	1.7	1.3	1.0
				6	10	10	10	10
				7.5	10	10	10	10
9	250	Unbonded	Paved	5	4.76	3.17	2.38	1.9
				6	10	10	10	10
				7.5	10	10	10	10
9	350	Unbonded	Paved	5	10	10	10	10
				6	10	10	10	10
				7.5	10	10	10	10

* Concrete Modulus of Rupture = 650 psi ** Truck Factor = 1.5

Table 4.5 Design Catalog for Thin Whitetopping for 6% Traffic Growth

AC Thickness (in)	AC Modulus (ksi)	Bonding Condition	Shoulder Condition	TWT Thickness* (in)	Service Life (Years)			
					Daily Truck Traffic** per Lane			
					≤200	300	400	500
5	250	Bonded	Unpaved	5	10	10	10	10
7				10	10	10	10	
9				10	10	10	10	
5	350	Unbonded	Paved	5	10	10	10	10
				6	10	10	10	10
				7.5	10	10	10	10
5	250	Unbonded	Unpaved	5	0.00	0.00	0.00	0.00
				6	0.16	0.11	0.08	0.06
				7.5	10	10	10	10
5	350	Unbonded	Unpaved	5	0.01	0.01	0.01	0.01
				6	0.25	0.17	0.13	0.10
				7.5	10	10	10	10
5	250	Unbonded	Paved	5	0.1	0.04	0.03	0.02
				6	0.7	0.49	0.37	0.30
				7.5	10	10	10	10
5	350	Unbonded	Paved	5	0.1	0.08	0.06	0.05
				6	2.0	1.36	1.02	0.81
				7.5	10	10	10	10
7	250	Unbonded	Unpaved	5	0.00	0.00	0.00	0.00
				6	0.8	0.5	0.4	0.3
				7.5	10	10	10	10
7	350	Unbonded	Unpaved	5	0.1	0.1	0.1	0.1
				6	4.1	2.8	2.1	1.7
				7.5	10	10	10	10
7	250	Unbonded	Paved	5	0.3	0.2	0.2	0.1
				6	10	9.3	7.0	5.6
				7.5	10	10	10	10
7	350	Unbonded	Paved	5	1.2	0.8	0.6	0.5
				6	10	10	10	10
				7.5	10	10	10	10
9	250	Unbonded	Unpaved	5	0.3	0.2	0.1	0.1
				6	10	8.0	6.0	4.8
				7.5	10	10	10	10
9	350	Unbonded	Unpaved	5	2.2	1.5	1.1	0.9
				6	10	10	10	10
				7.5	10	10	10	10
9	250	Unbonded	Paved	5	4.14	2.76	2.07	1.66
				6	10	10	10	10
				7.5	10	10	10	10
9	350	Unbonded	Paved	5	10	10	10	10
				6	10	10	10	10
				7.5	10	10	10	10

* Concrete Modulus of Rupture = 650 psi ** Truck Factor = 1.5

CHAPTER 5 Conclusions and Recommendations

5.1 Conclusions

The objective of this study was to assess the behavior of thin whitetoppings on existing AC pavements for different bonding conditions with the AC layer, AC thicknesses, shoulder conditions, existing AC modulus and temperature differentials. Based on this study, a design catalog was developed for thin whitetopping. From this study, the following conclusions can be drawn:

- 1) Interface bonding conditions between TWT and the existing AC layer are the most important factors that affect TWT behavior. Transverse tensile stress increases about 160% for the change in interface condition from bonded to unbonded. TWT thickness has a more pronounced effect on unbonded interface conditions than on fully bonded conditions.
- 2) The addition of a frictional coefficient resulted in a considerable drop in tensile stress for unpaved shoulder conditions, but did not show any significant effect for paved shoulder conditions.
- 3) For different TWT thicknesses, changes in stresses were found to vary from 23% to 32% for unbonded TWT and 14% to 20% for bonded TWT. The existing AC thickness has a more pronounced effect on concrete stresses for 5 in. TWT than 6 in. and 7.5 in. TWT.
- 4) Concrete tensile stresses decrease about 12% to 14% for a 2 in. increase in AC thickness. The corresponding change due to the AC modulus was about 6% for unbonded overlay and 14% for bonded overlay. The difference is about 15% for changes of shoulder conditions. The change of curling stress due to TWT thickness change was about 14% to 30%.

- 5) For fully bonded conditions of all TWT and AC layer thicknesses, TWT is expected to last through the 10 years assumed in design. Unbonded TWT with unpaved shoulders results in very early pavement failures.

5.2 Recommendations

The following recommendations are made from this study:

- 1) Further studies should be conducted with field experiments to determine the actual behavior of thin whitetopping.
- 2) The effect of environment, subgrade soil types and different joint spacing may be investigated.
- 3) Pavement response under moving loads would give a better approximation of the actual scenario.
- 4) In the field, it is very difficult to achieve full bond between the concrete and the AC layer; therefore, partial bonding at the interface should be investigated in detail.

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