EXTENDING ASPHALT PAVEMENT LIFE WITH THIN WHITETOPPING


by


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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 rehabilitation of distressed asphalt concrete (AC) pavements 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, a 3-D FE software program to develop this design catalog. The design considered different TWT thicknesses, existing AC layer thicknesses and modulus, bonding conditions between TWT and existing AC layer, shoulder conditions and temperature differentials. Each model was built as a three-layer pavement system—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 addition of 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 unbonded TWT with 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.

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# CHAPTER 1 - INTRODUCTION

## 1.1 Introduction

The extensive U.S. highway network, the economic backbone of the United States, allows for almost 3 trillion miles of travel annually (TRIP, 2010). But the condition of this network is deteriorating due to limitation of funds to maintain it in good condition. As a result, 32 percent 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, when pavements are left to deteriorate without timely maintenance treatment, they 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):

1) Routine Maintenance Treatment:

   i. Crack filling/ sealing

   ii. Patching

2) Preventive Maintenance Treatment:

   i. Fog seals

   ii. Surface rejuvenating

   iii. Micro-milling

   iv. Thin surfacing:

      a. Chip seal

      b. Slurry seal

      c. Micro surfacing

      d. Metro-mat

      e. Nova chip

      f. Thin hot-mix asphalt overlays

3) Corrective Maintenance:

   i. Full-depth patching

   ii. Milling

   iii. Overlays

Major rehabilitation treatments for asphalt pavements are listed below (Uzarowski and Bashir, 2007):

1) Structural Overlays

2) Recycling:

    i. Hot in-place recycling (HIR)

    ii. Cold in-place recycling (CIR):

        a. with emulsion

        b. with foamed asphalt

    iii. Full-depth reclamation (FDR) with foamed asphalt

3) Reconstruction.

Rehabilitation with structural overlays contributes to structural capacity of pavements. Structural overlays used for asphalt pavements are as follows:

1) Asphalt concrete

2) Whitetopping and

3) 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 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 (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 formation of 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 of AC overlay.

## 1.4 Problem Statement

There are a number of design procedures for whitetopping:

1) AASHTO (AASHTO, 1993)

2) State of Colorado (Sheehan et al., 2004; Tarr et al., 1998)

3) Portland Cement Association (PCA)/American Concrete Pavement Association (ACPA) (Wu et al., 1998)

4) State of New Jersey (Nenad et al., 1998)

5) Modified ACPA (Riley et al., 2005)

6) State of Illinois (Roesler et al., 2008)

7) State of Texas (Chul et al., 2008)

Among these procedures, only AASHTO, Colorado, and Texas procedures offer guidelines for thin and conventional whitetopping, whereas 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 for 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 asses 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 Thesis

The thesis 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

In recent times, 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 actions needed to retain the pavement in 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 done when the pavement is in need of structural improvement.
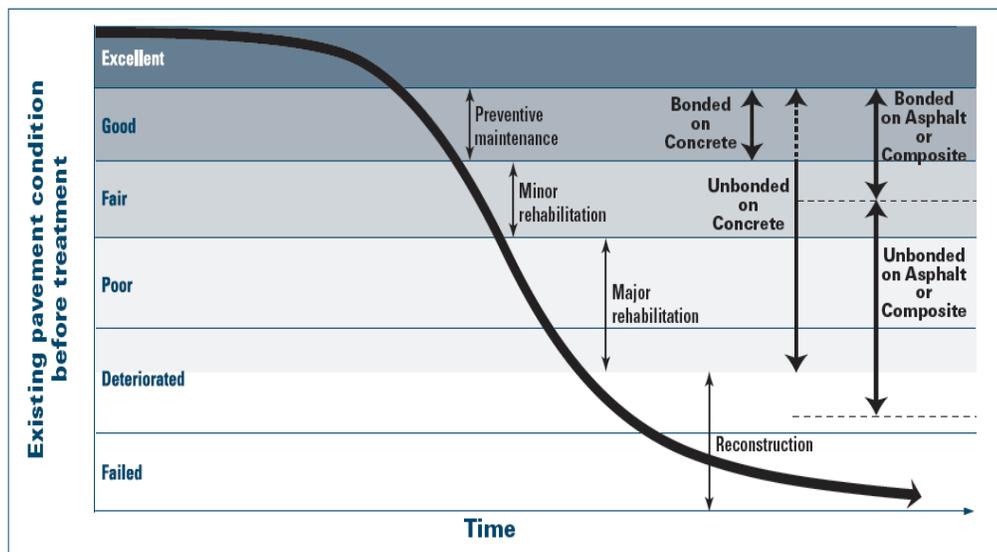


**Figure 2.1 Typical Pavement Condition Curve over Time and Actions Needed**

**(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 done. Among them, 158 were jointed plain concrete pavement, 14 continuously reinforced concrete, 10 fiber-reinforced concrete, and seven 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, as well as be a means of green construction compared to asphalt concrete overlay.

### 2.1.1 Types of Whitetopping

Considering the thickness, there are three types of whitetopping (WT) (Sheehan et al., 2004):

1) Conventional (thickness > 8in)

2) Thin (thickness 4-8 in)

3) Ultra-thin (thickness 2-4 in)

A summary of whitetopping characteristics has been listed in Table 2.1. In this table, concrete overlays with 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 | • Limited repair (failed areas only)<br>• Possible milling to correct profile | • Repair of areas unable to contribute to load-carrying capacity<br>• 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 | • Adequate support critical to performance<br>• Adequate joint design (including joint spacing and load transfer)<br>• Placement of a whitewash on HMA surface on hot days | • Bonding with HMA pavement<br>• PCC mix design is often high-strength and/or fiber modified<br>• Extremely short joint spacings (typically between 0.6 and 1.8 m [2 to 6 ft]) |
| PCC Overlay Types | • JPCP<br>• JRCP<br>• CRCP | JPCP |

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

1) Bonded: In bonded WT, a sound bond between the WT and the AC layer is maintained through proper construction technique. 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.

**(a)**                                     **(b)**
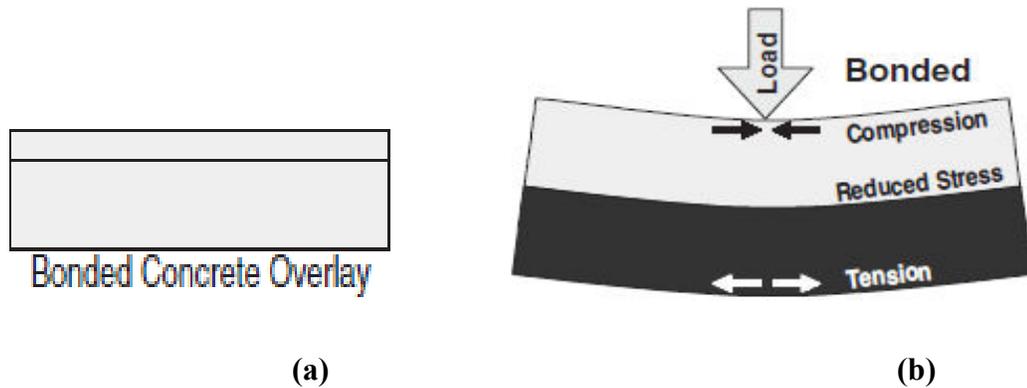
**Figure 2.2 (a) Bonded WT and (b) Behavior of Bonded WT under Loading**

**(Rasmussen and Rozycki, 2004)**

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.



(a)                                     (b)
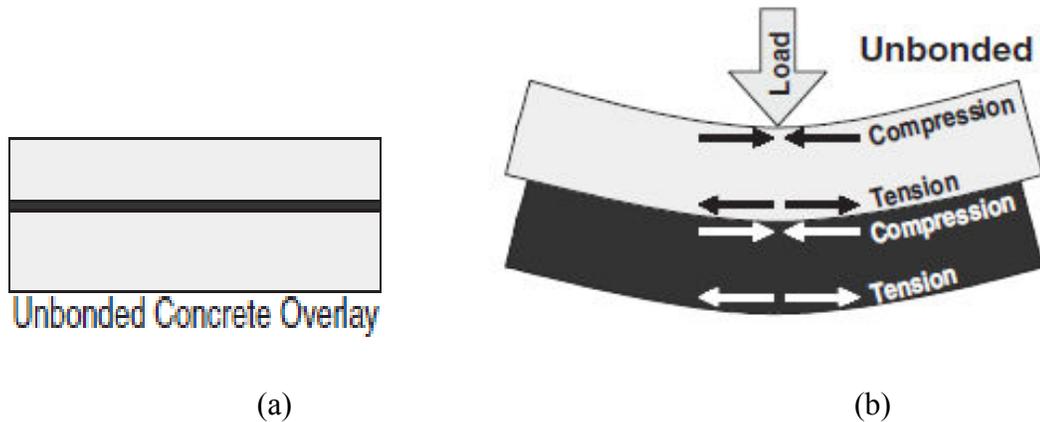
**Figure 2.3 (a) Unbonded WT and (b) Behavior of Unbonded WT under Loading**

**(Rasmussen and Rozycki, 2004)**

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## 2.1.2 Benefits of Whitetopping

The formation of heat islands (HI) is a major concern in city areas that influences 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. Thus, whitetopping may become a greener choice for rehabilitation of AC pavements. Higher albedo value also gives better nighttime visibility. Other benefits of using WT are as follows (Harrington, 2008; ACPA, 1991):

1) Does not develop typical distresses as does asphalt overlay.

2) Construction of WT is quick and convenient:

  i) Existing pavement can provide support;

  ii) No need for extensive pre-overlay preparation;

  iii) Normal paving systems can be used; and

  iv) Overlaid pavement can be opened to traffic quickly.

3) Repair is easy.

4) Improves safety conditions.

5) Is less affected by seasonal variations.

6) Can be used as preventive maintenance or major rehabilitation.

7) Provides high level of serviceability.

## 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. But, they 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 over-lay. | 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 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 would 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 here (Harrington, 2008).

6) Joints—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); state of Colorado procedure (Sheehan et al., 2004; Tarr et al., 1998); Portland Cement Association (PCA)/American Concrete Pavement Association (ACPA) method (Wu et al., 1998); state of New Jersey method (Nenad, 1998); modified ACPA approach (Riley et al., 2005); state of Illinois method (Roesler et al., 2008); and 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 ultra-thin whitetopping (UTW). Recently, MnDOT (2010) has developed a design technique for thin whitetopping. A brief description of TWT design procedures is presented here:

*AASHTO*

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

$$D_{ol} = D_f \qquad \text{(Eq. 2.1)}$$

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:

Step 1: Determination of material types and layer thicknesses of existing pavement.

Step 2: Prediction of future traffic for the design period, $N_f$.

Step 3: A general survey of existing pavement distresses of heaves and swell, stripping of AC pavement, and large transverse cracks is required. These distresses may affect the service life of WT.

Step 4: 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.

$$M_R = \frac{0.24P}{d_r r} \qquad \text{(Eq. 2.2)}$$

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.

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

(Eq. 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.

Effective dynamic k-value is determined from Figure 2.4 using $M_R$, $E_P$, and D.

Step 5: If there is any unusual distress condition, coring and material testing is recommended.

Step 6: Determination of required slab thickness:

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.
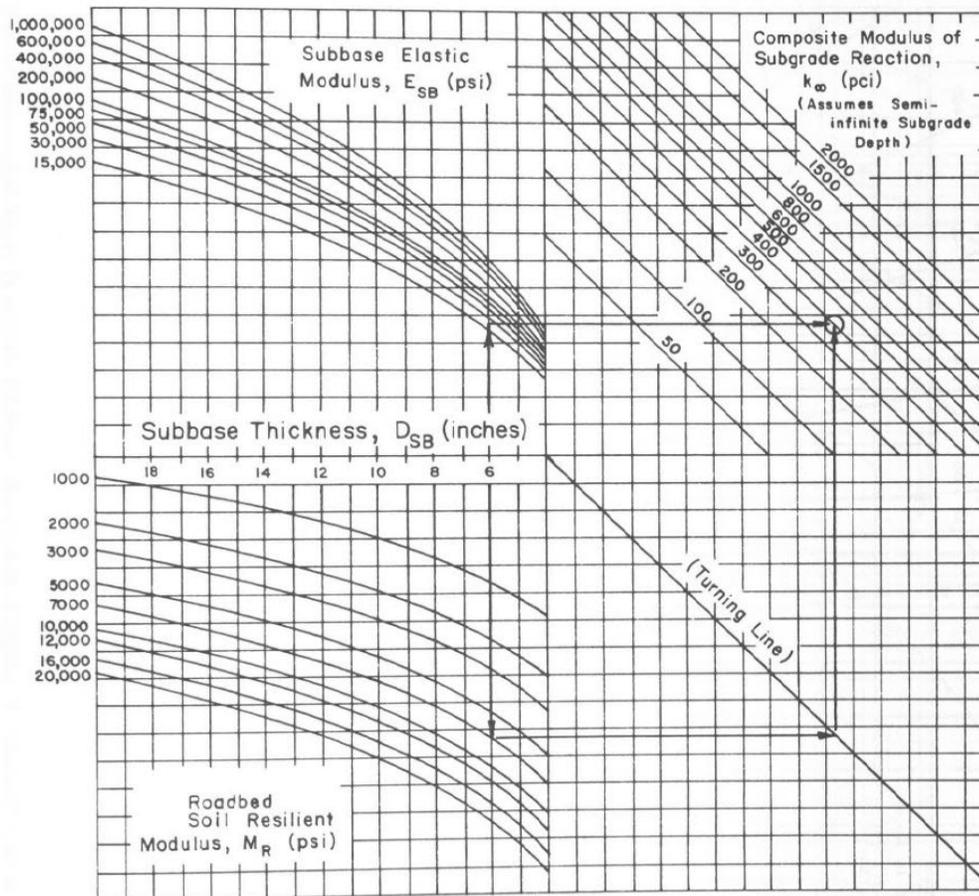
**Figure 2.4 Chart for Estimating Composite Modulus of Subgrade Reaction**

**(AASHTO, 1993)**

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, $3^{rd}$ 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.

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

Step 7: The PCC overlay thickness is then determined from Eq. 2.1.

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

| Shoulder | Asphalt | | Tied P.C.C. | |
|---|---|---|---|---|
| Load-Transfer Devices | | | | |
| Pavement Type | 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 |
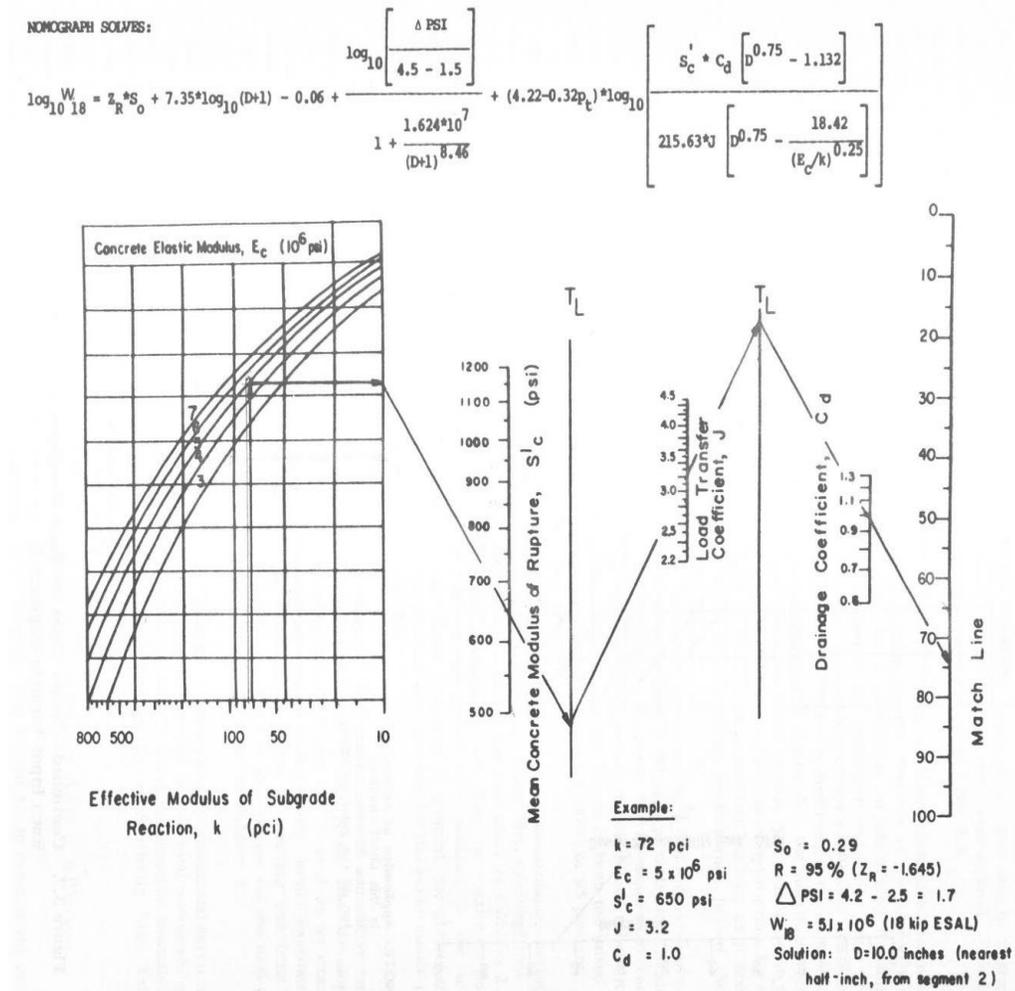


**Figure 2.5 Design Chart for Rigid Pavement Based on Using Mean Values for Each**

**Input Variable (Segment 1) (AASHTO, 1993)**

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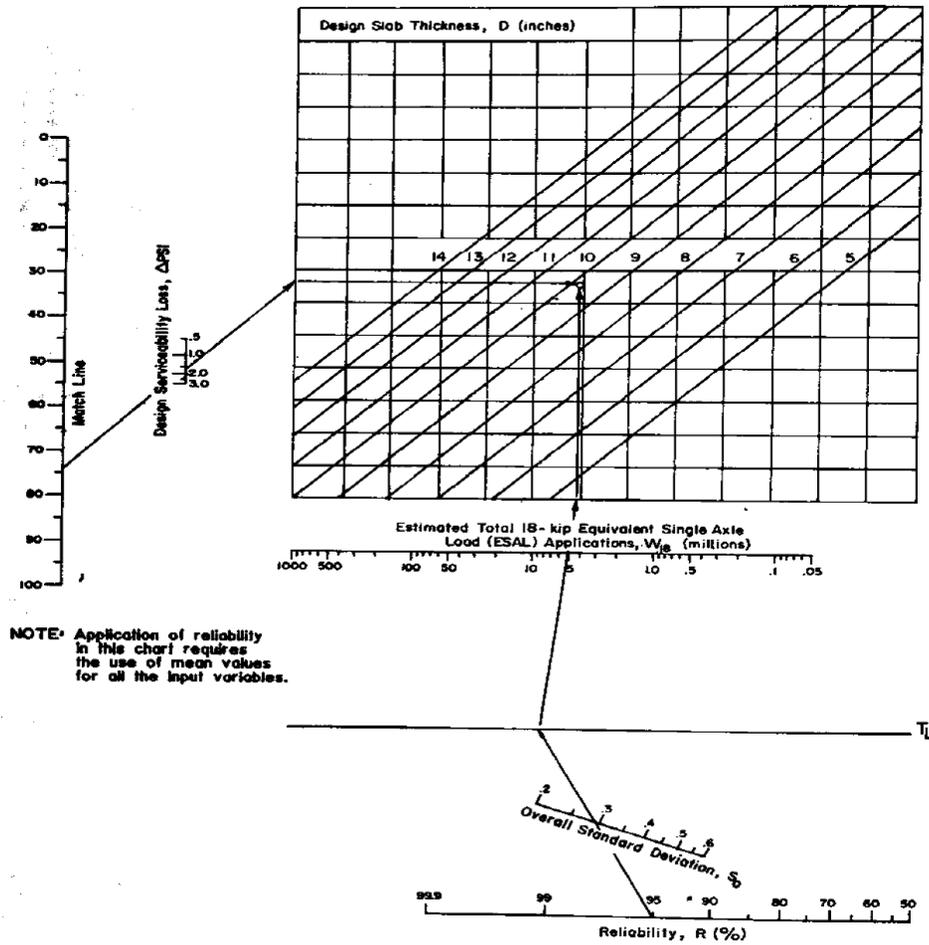
**Figure 2.6 Design Chart for Rigid Pavement Based on Using Mean Values for Each**

**Input Variable (Segment 2) (AASHTO, 1993)**

### *Colorado Procedure*

In 1998, Tarr et al. developed the model for designing TWT. Sheehan et al. (2004) 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 design of TWT, critical load locations were identified and verified with field data. For 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) is

    $$\sigma_{ex} = 1.65 \times \sigma_{th} \qquad\qquad \text{(Eq. 2.4 a)}$$

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

    $$\sigma_{ex} = 1.51 \times \sigma_{th} \qquad\qquad \text{(Eq. 2.4 b)}$$

    where

    $\sigma_{ex}$ = measured experimental partially bonded stress, psi; and

    $\sigma_{th}$ = 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 have been shown in Eqs. 2.5 (a) and 2.5 ( b).

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

    $$\varepsilon_{ac} = 0.842 \times \varepsilon_{pcc} \qquad\qquad \text{(Eq. 2.5 a)}$$

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

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

    where

    $\varepsilon_{ac}$ = measured asphalt surface strain, microstrain; and

    $\varepsilon_{pcc}$ = 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.

$$\sigma_{FE} = 1.87 \times \sigma_{TE} \qquad\qquad \text{(Eq. 2.6)}$$

where

$\sigma_{FE}$ = longitudinal free-joint load-induced stress, psi, and

$\sigma_{TE}$ = 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

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

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

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

$$\text{(Eq. 2.7 b)}$$

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

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

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

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

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

(Eq. 2.8 b)

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

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

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

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

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

(Eq. 2.9 b)

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

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

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

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

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

(Eq. 2.10 b)

where

$\sigma_{pcc}$ = maximum stress in concrete slab, psi;

$\varepsilon_{ac}$ = 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;

$t_{pcc}$ = thickness of asphalt layer, in;

$k$ = modulus of subgrade reaction, pci;

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

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

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

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

$\mu_{pcc}$ = poisons ratio for concrete, assumed 0.15;

$\mu_{ac}$ = poisons ratio for asphalt, assumed 0.35; and

$L$ = joint spacing, in.

5) Concrete fatigue criterion (PCA, 1984) which is based on the theory that fatigue resistance that 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

$$\log_{10}(N) = \frac{0.97187 - SR}{0.0828} \qquad \text{(Eq. 2.11)}$$

For $0.45 \leq SR \leq 0.55$,

$$N = \left( \frac{4.2577}{SR - 0.43248} \right)^{3.268} \qquad \text{(Eq. 2.12)}$$

For SR < 0.45,

N = Unlimited                                                                    (Eq. 2.13)

where

SR = flexural stress to strength (modulus of rapture 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:

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

where

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

$\varepsilon_{ac}$ = 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, AC layer, base, and subgrade were analyzed for

different geometry and loading conditions. The dimension of TWT is 18 ft × 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).



(a)                                      (b)
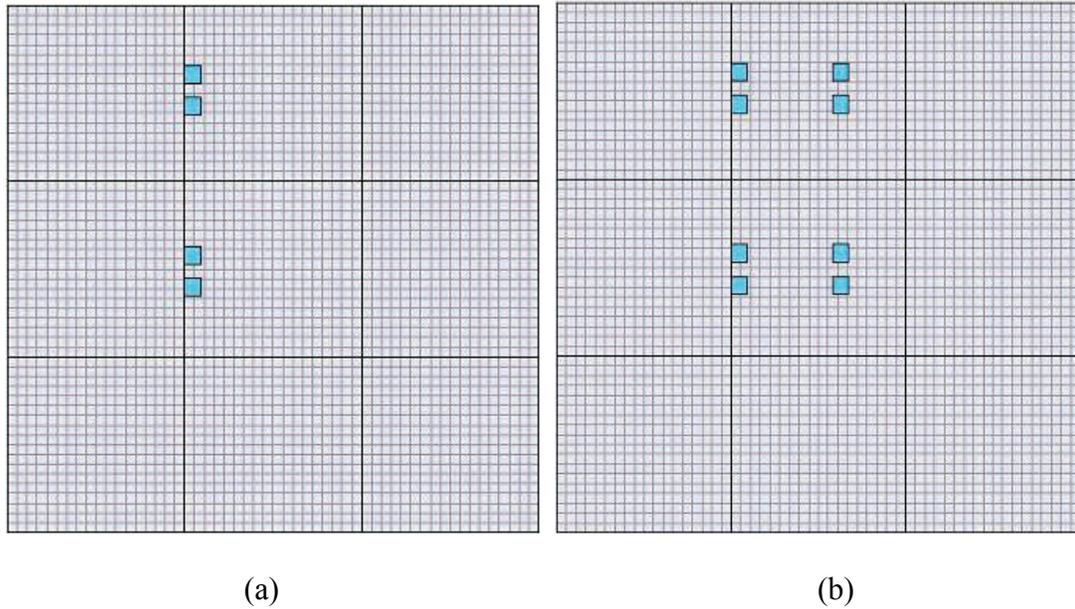
**Figure 2.7 TWT Model Using ISLAB2000 (a) Single Axle Load and (b) Tandem Axle**

**Load (Chul et al., 2008)**

Steps of the Texas design procedure for TWT are as follows:

Step 1: Structural condition and material properties are recommended to be

evaluated using a dynamic cone penetrometer (DCP), and a falling weight deflectometer

(FWD).

Step 2: The subgrade resilient modulus is estimated using the following

relationship:

$$M_R = \frac{438,000}{DCP^{1.12}} \qquad \text{(Eq. 2.15)}$$

where

$M_R$ = resilient (elastic) modulus, psi; and

26

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.

Step 3: The resilient modulus obtained in Step 2 is used in a back-calculation program to estimate the resilient modulus of the subbase layer. Asphalt layer thickness 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.

Step 4: The required slab thickness is calculated using Eq. 2.16.

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

(Eq. 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:

1) Pre-overlay works,

2) Overlay materials,

3) Placement and finishing,

4) Curing, and

5) 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 to produce a sound bond between whitetopping and the AC layer. It levels out surface distortions, and removes soft loose asphalt and high spots which cause 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 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 concrete mixtures for whitetopping, Type I and Type II cements are used. In some cases, Type III

cements are used for development of high early strength. Typical cement content varies between 500 and 700 lb/yd$^3$. High-quality stones to 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. 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 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 of 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, polyethylene sheeting, wax-based curing compounds, liquid asphalts, and hot-mixed asphalt materials are used as a separator layer (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, surface temperature of the asphalt should not be higher than $120^0$ F, which may cause 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 a way that no movement can occur. Concrete is placed by ready-mix truck, pump, or other means. The forms are also designed to withstand lateral pressure from the concrete. 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 tining comb, depending on the speed limit on the facility. Tining of the concrete surface is performed immediately after the moisture sheen evaporates from it (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 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, 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 4-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 represent 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 placement of concrete on the hot asphalt layer, which initiated shrinkage cracking. Thus, 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, 5-in. and 6-in. TWTs were 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 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; and

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; and

2) Stiffness of the pavement system resulting from whitetopping placement increased by an average of 300 percent, 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 (MtDOT, 2001). Thickness of TWT was 4.5 in. 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 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 done for producing 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, North Mankato with other ultra-thin whitetopping projects (Vandenbossche and Rettner, 1999). A 6-in. concrete overlay with 10 ft by 12 ft panels were constructed on the southbound lanes south of U.S. 14 and another 4.5-in. TWT with 5 ft by 6 ft panels were 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 preoverlay treatments to produce the intended interface condition. Placement, finishing, and joint sawing are done in the same way as in 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 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: TWT, existing AC layer, and 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 inches (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 9in.;

(3) Two bonding conditions: fully bonded and unbonded;

(4) Two shoulder conditions: unpaved and paved; and

(5) Two temperature differentials: $1.5^0$F/in. and $3^0$F/in.
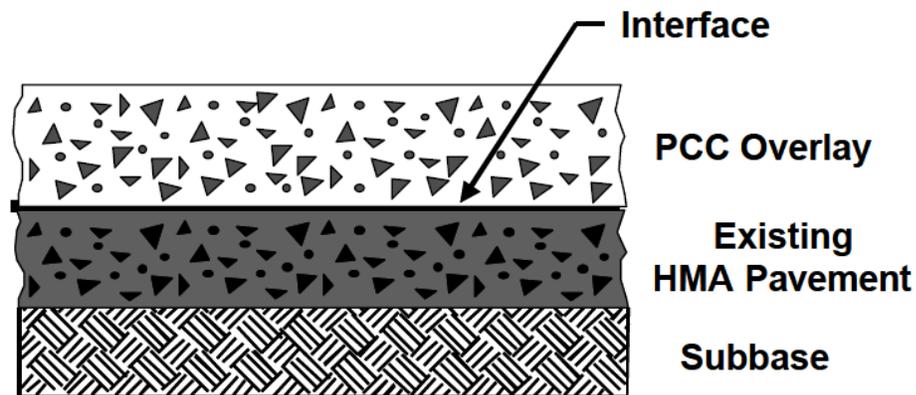


**Figure 3.1 Three-Layer Pavement System (McGhee, 1994)**

### 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 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 = $5.5 \times 10^{-6}$ (/$^o$F)

(2) AC layer:

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

Poisson's Ratio = 0.4

Coefficient of Thermal Expansion = $13 \times 10^{-6}$ (/$^o$F)

(3) Subgrade:

Modulus of Elasticity = 20,000 psi

Poisson's Ratio = 0.45

Coefficient of Thermal Expansion = $2 \times 10^{-6}$ (/$^o$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 size however, 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).
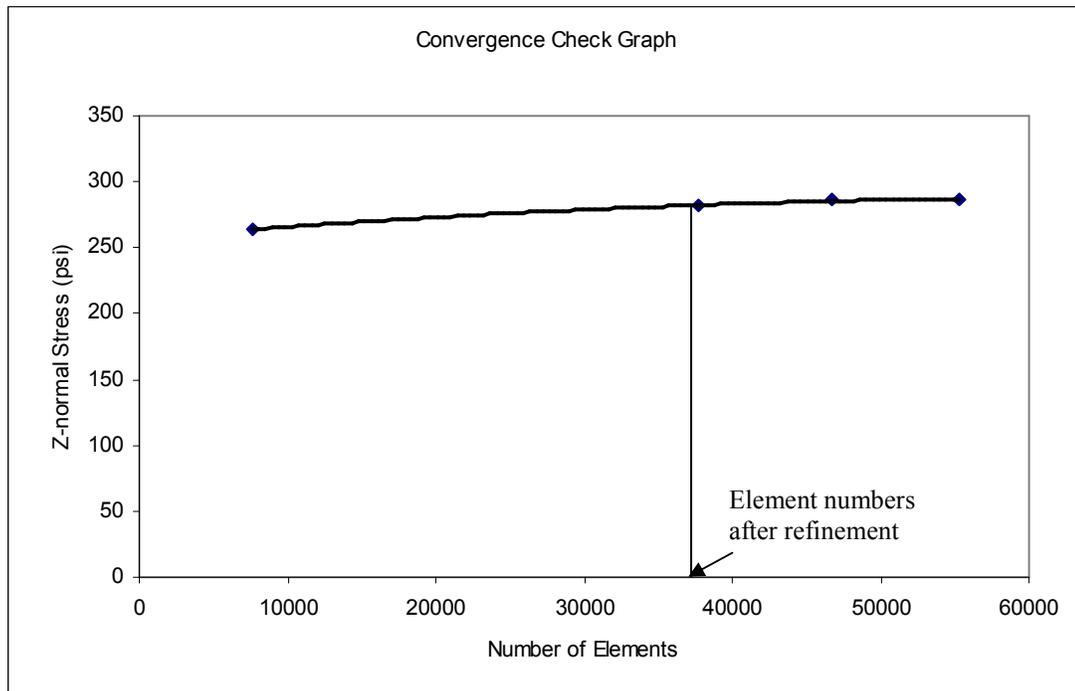


Figure 3.2 Convergence Check Using Z-Normal Stress as a Measure

<div align="center">

Draft-quality element          High-quality element

</div>

**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 5a). That side was restrained in the direction of pavement width. All restraints of the model are shown in Figures 3.5(a) and 3.5(b).

### 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 $^{0}$F/in and 3 $^{0}$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.
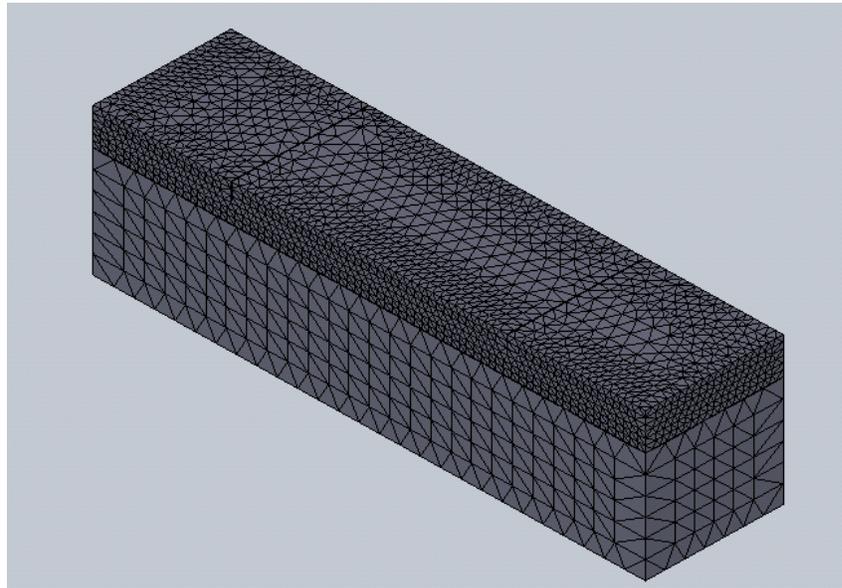


(a)
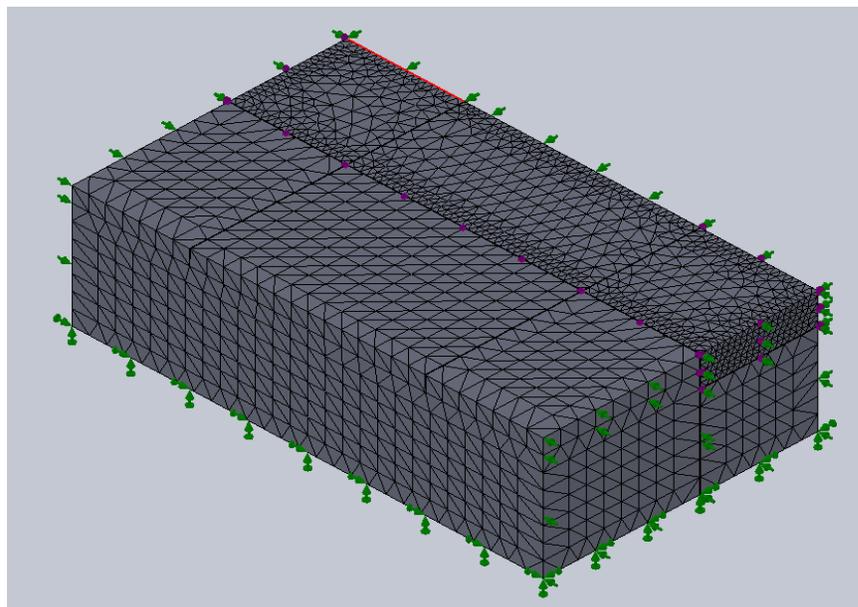


(b)

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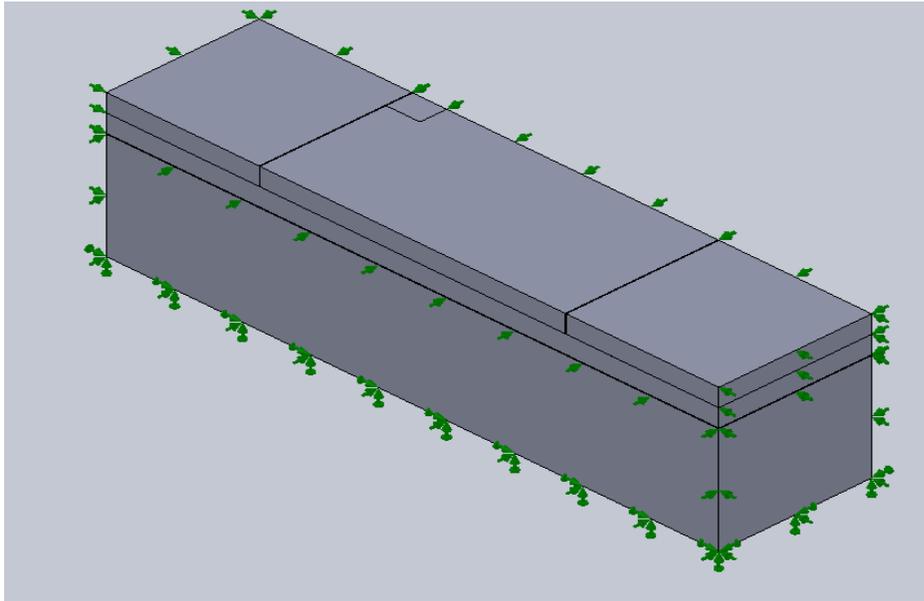
**(a)**
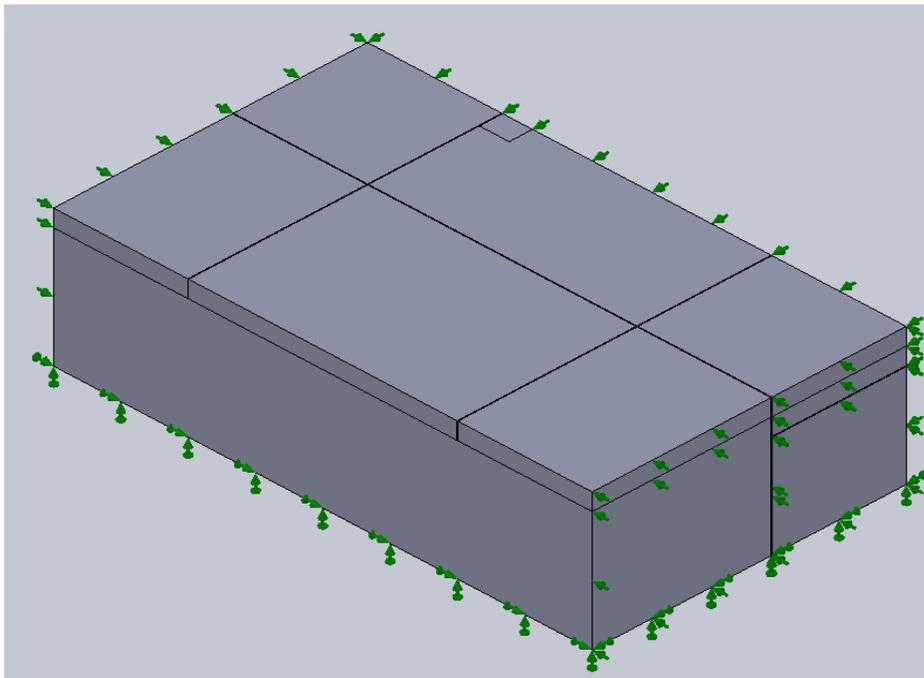


**(b)**

**Figure 3.5 Restraints of the Model (a) No Shoulder, and (b) Shoulder**
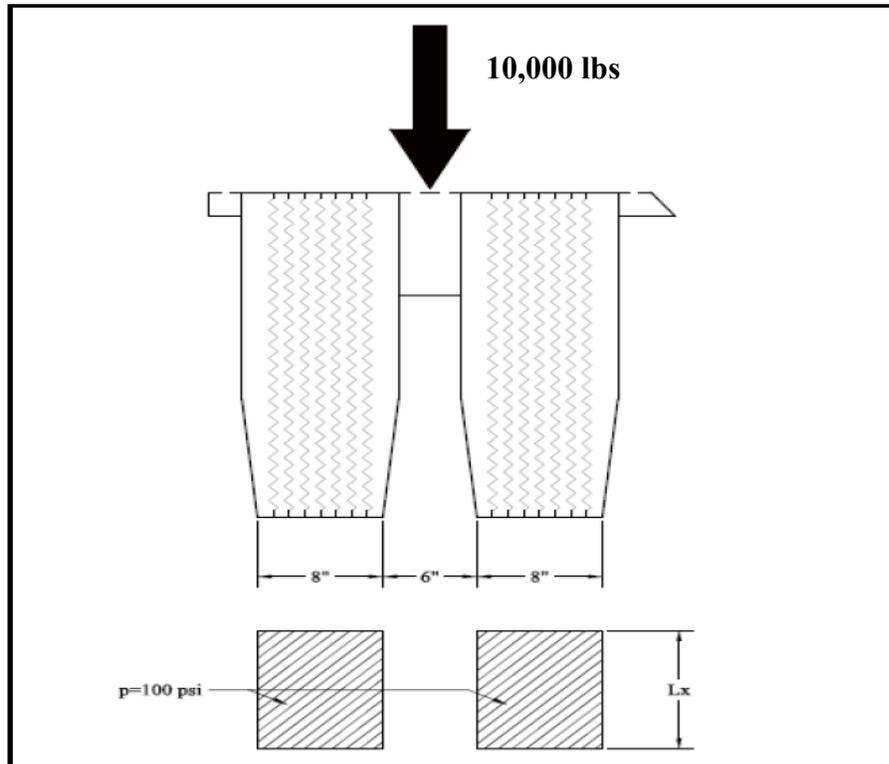
**Figure 3.6 Load Model (after Dumitru, 2006)**
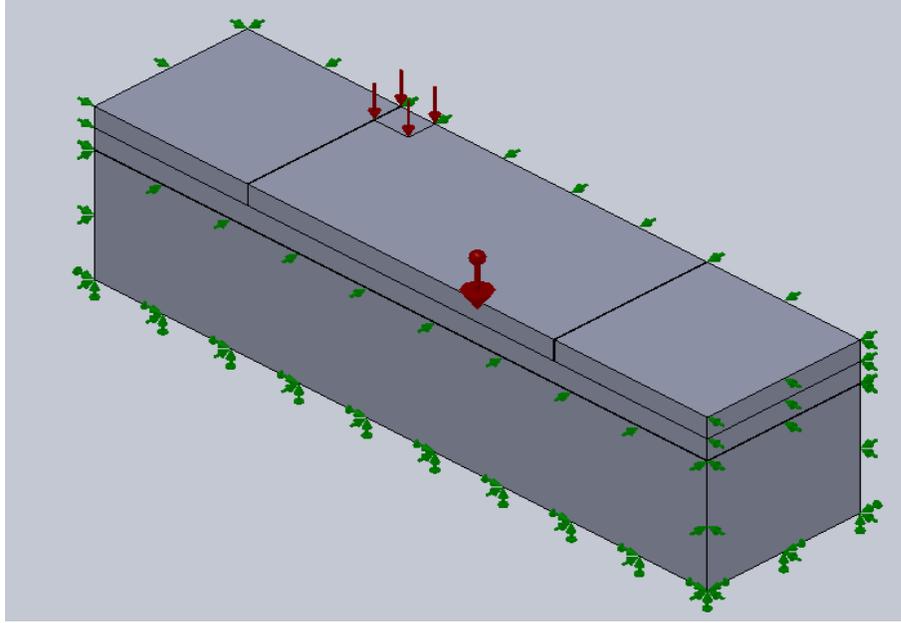
**(a)**
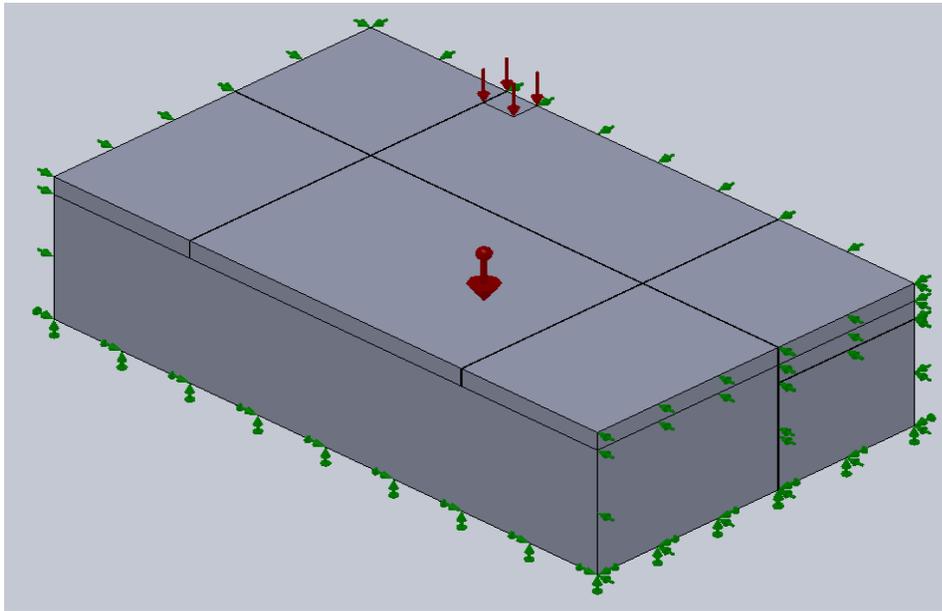


**(b)**

**Figure 3.7 Loading Model in SW (a) No Shoulder, and (b) Shoulder**
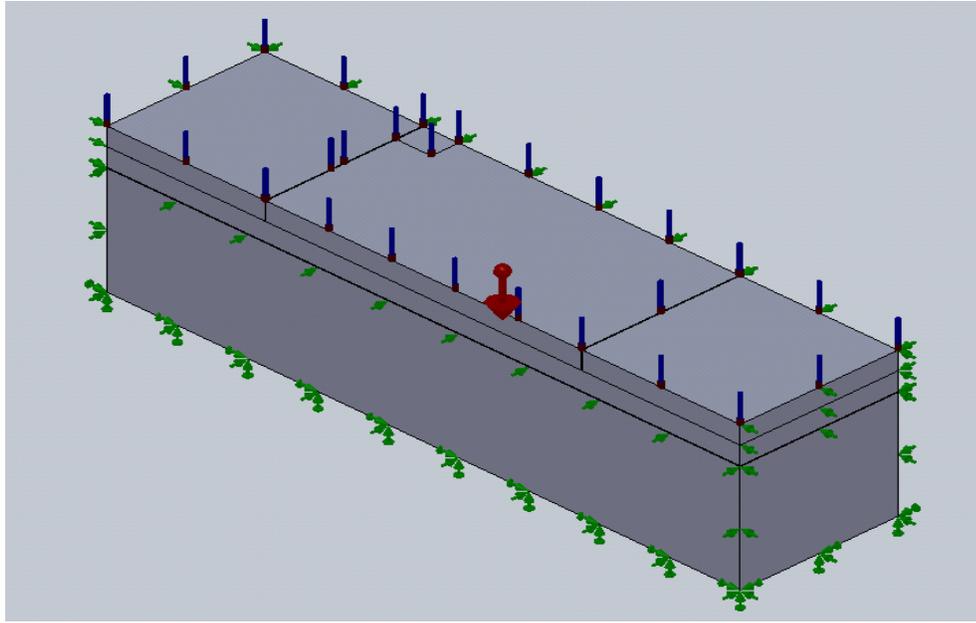
**Figure 3.8 Loading Model for Temperature Differential**

# 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.
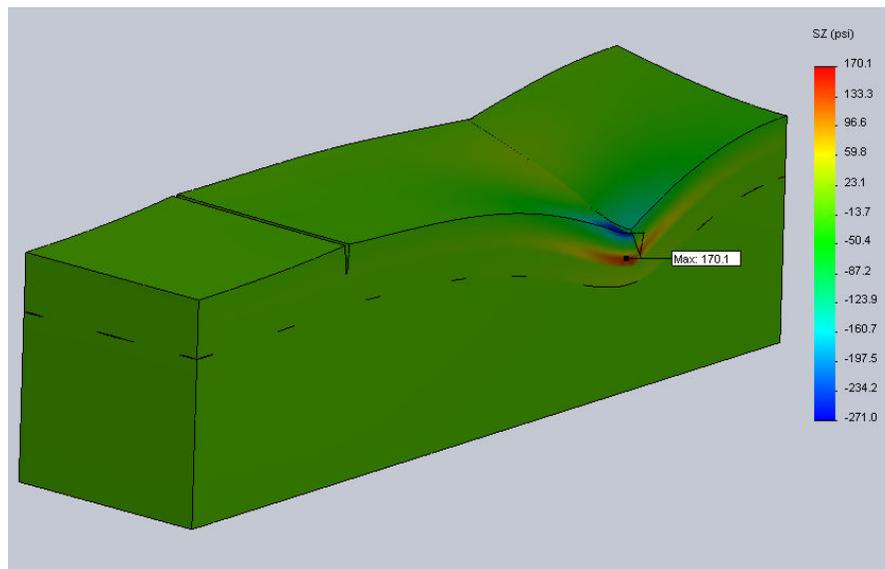


**Figure 4.1 Transverse Stress Contour of TWT**
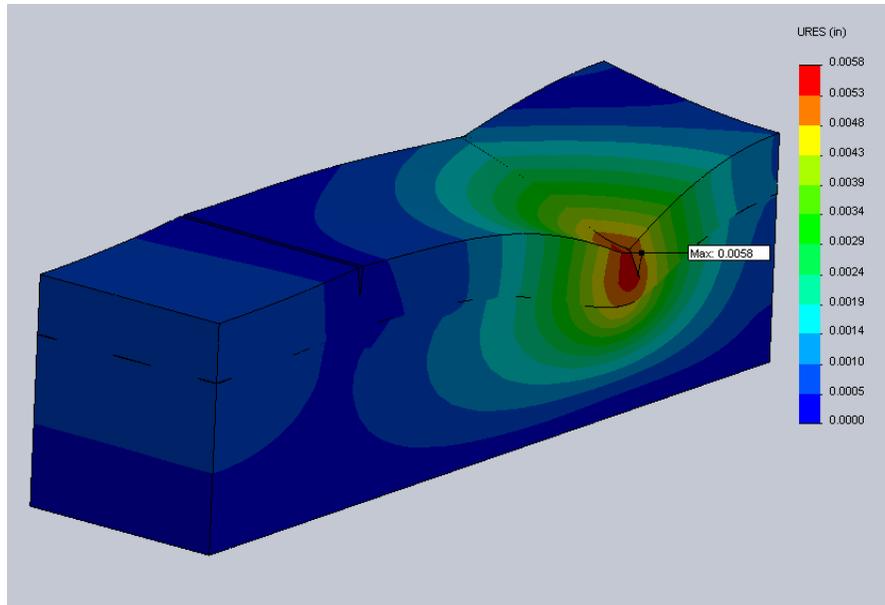
**Figure 4.2 Deflection Contour of TWT**

### *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.
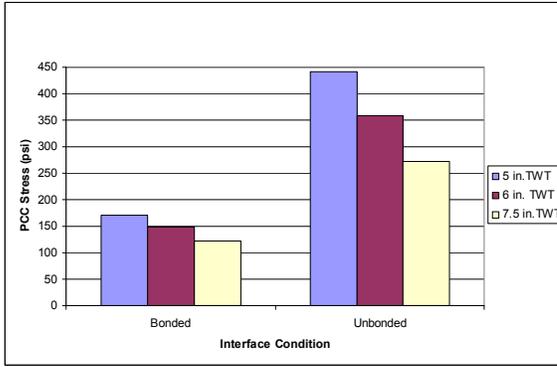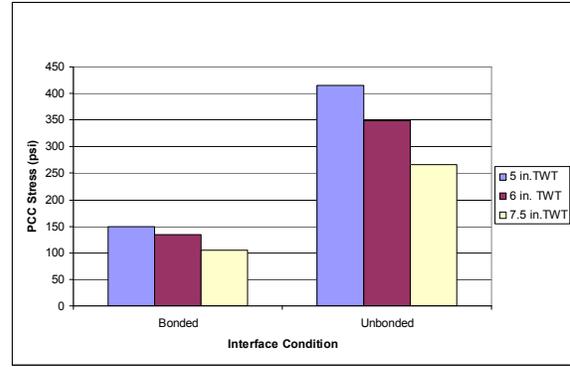
**(a)**                                                    **(b)**

**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**
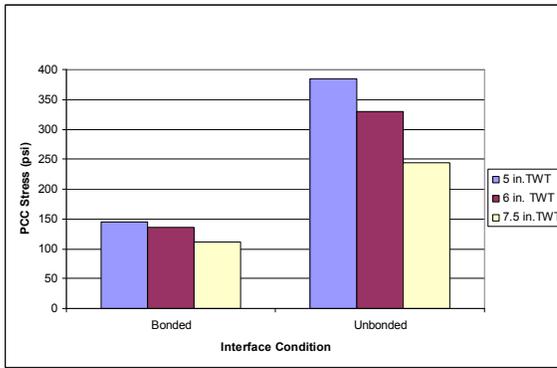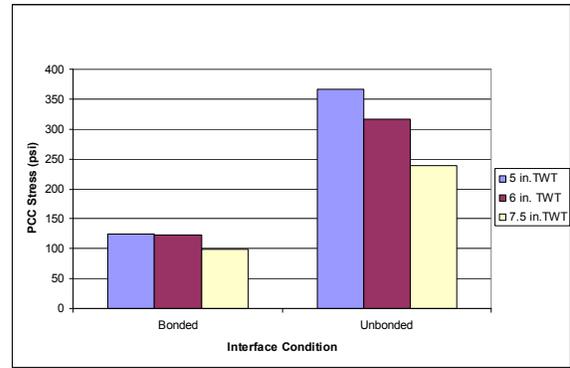


**(a)**                                                    **(b)**

**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**
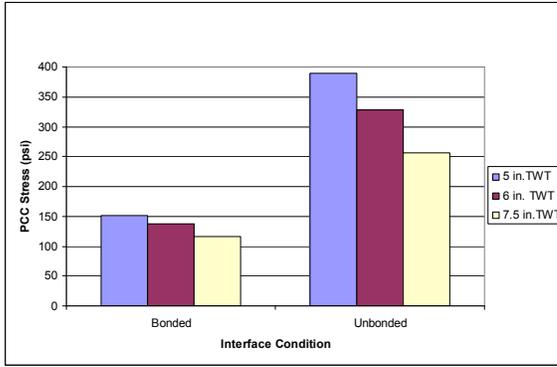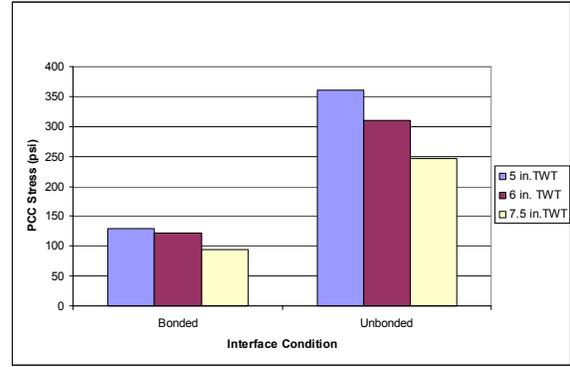
(a)                                                    (b)

**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**
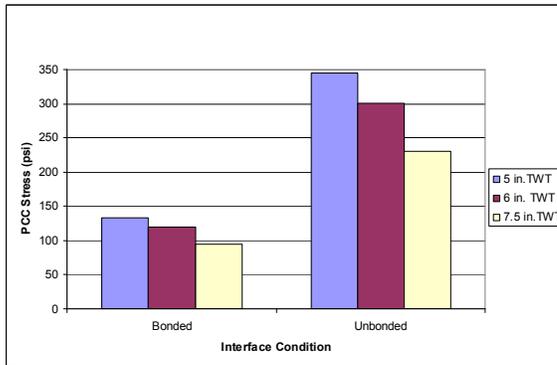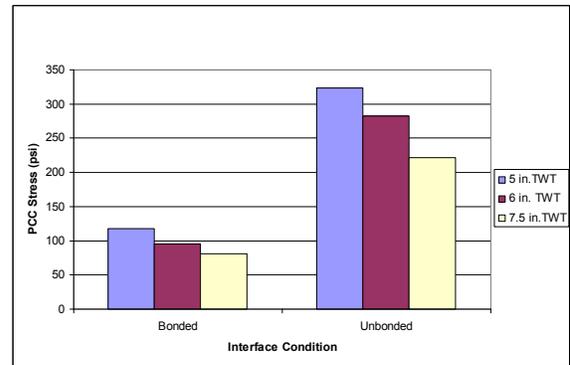


(a)                                                    (b)

**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**
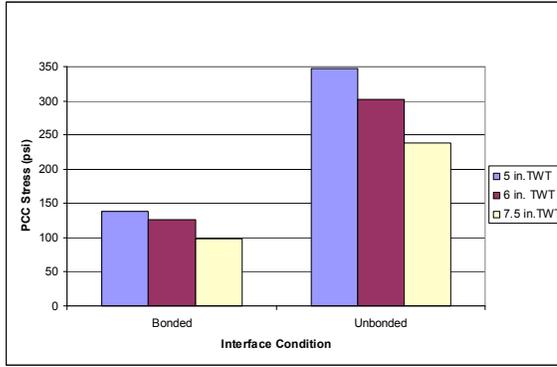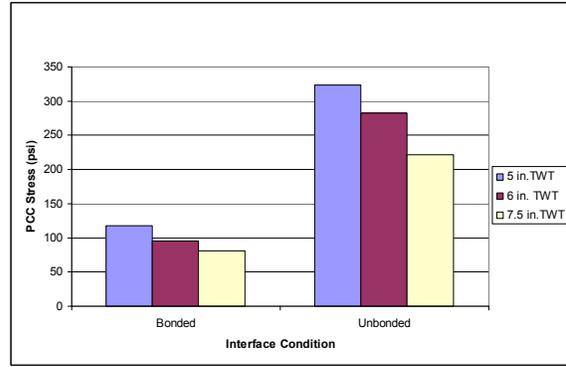
(a)                                                                (b)

**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**
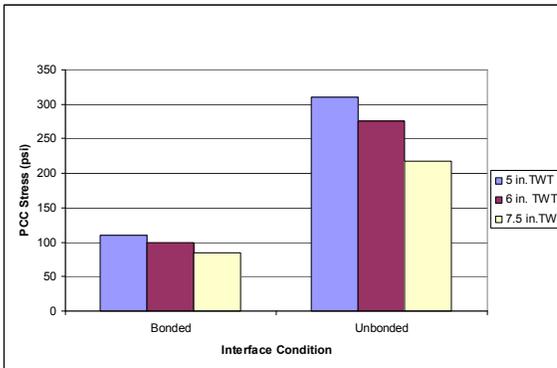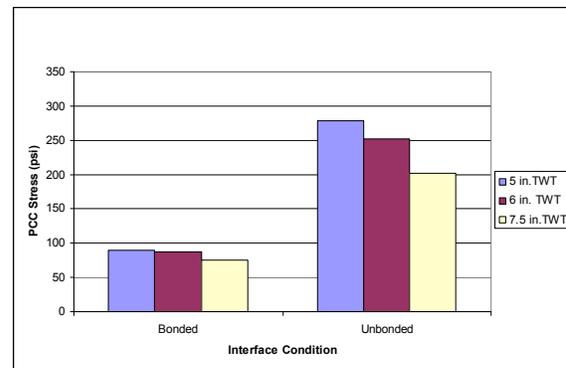


(a)                                                                (b)

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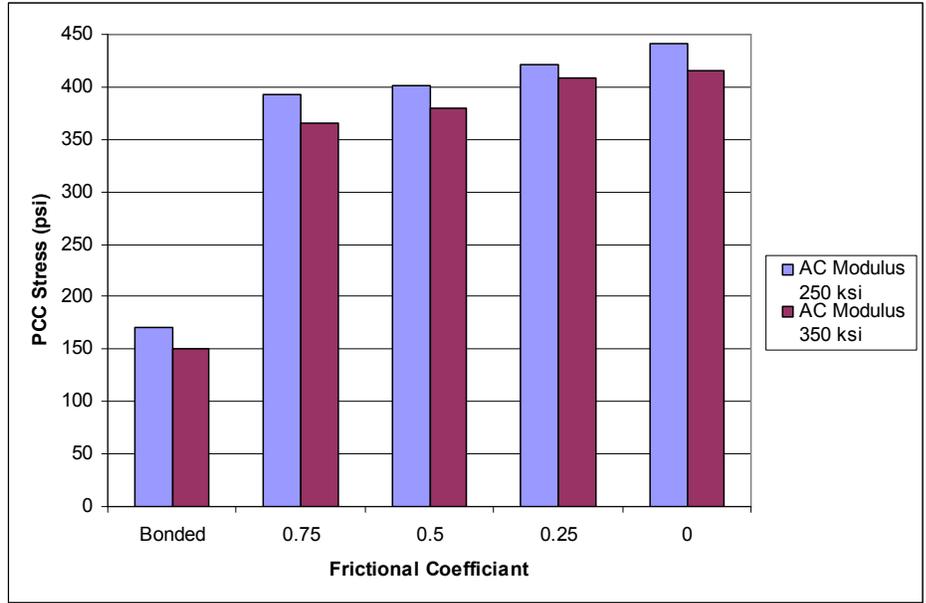
**Figure 4.9 PCC Stress vs. Frictional Coefficient Graph for Unpaved TWT**
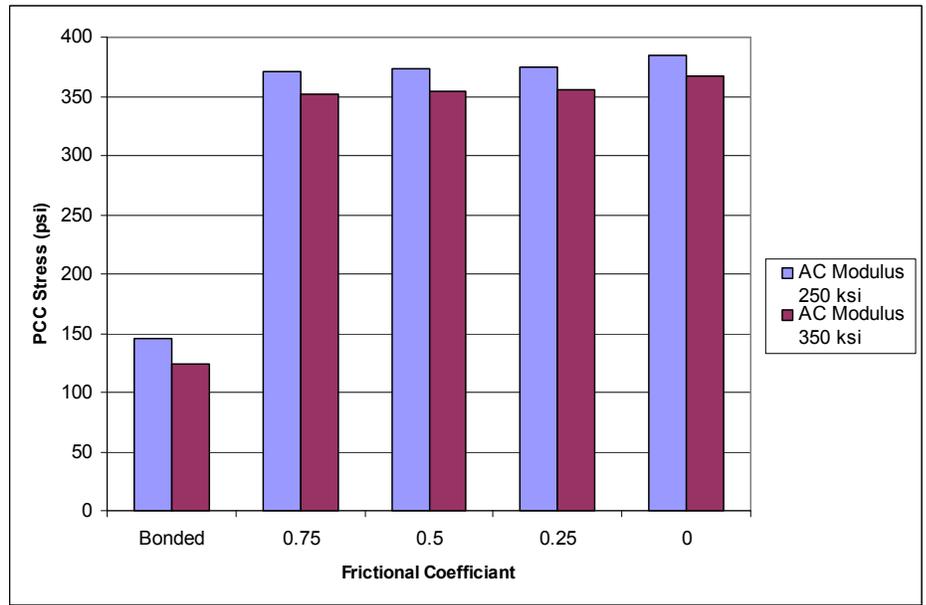


**Figure 4.10 PCC Stress vs. Frictional Coefficient Graph for Paved TWT**

## 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.



(a)                                                         (b)
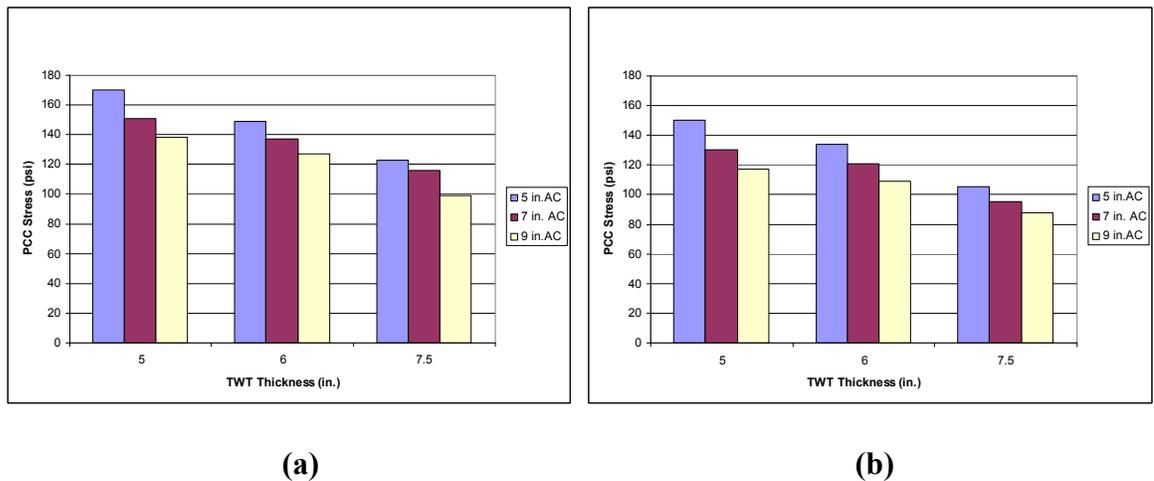
**Figure 4.11 PCC Stress vs. TWT Thickness Graph for Bonded Condition and**

**Unpaved Shoulder (a) AC Modulus 250 ksi and (b) AC Modulus 350 ksi**
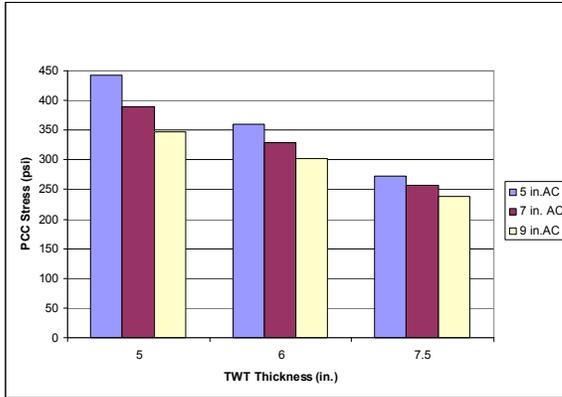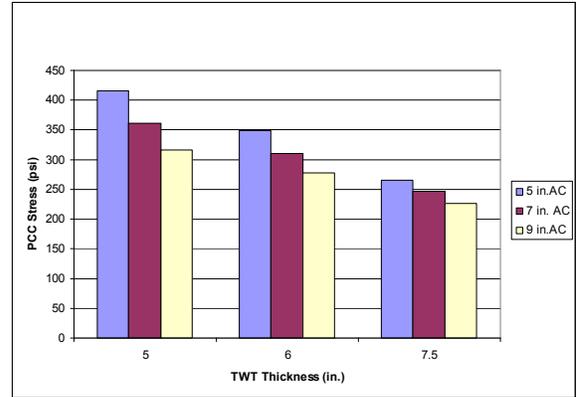
**(a)**             **(b)**

**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**
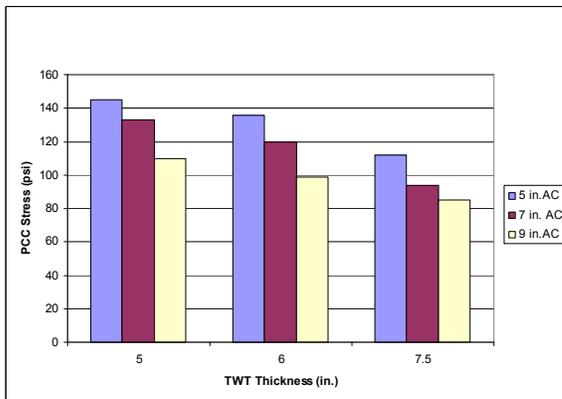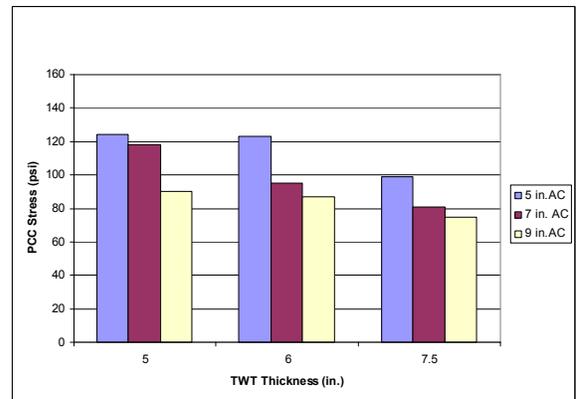


**(a)**             **(b)**

**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**

|                | (a)                             | (b)                             |
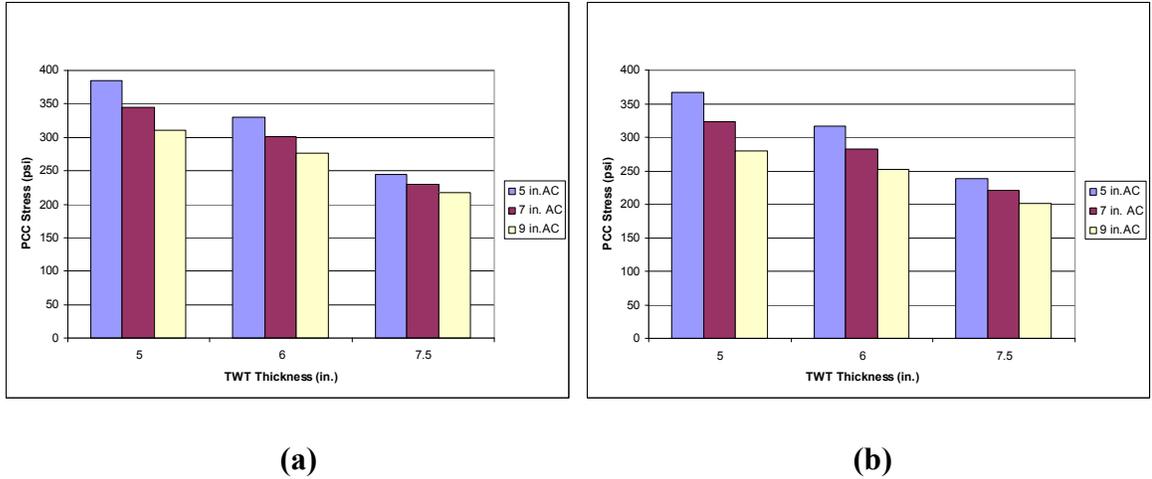
**Figure 4.14 PCC Stress vs. TWT Thickness Graph for Unbonded Condition and**

**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.

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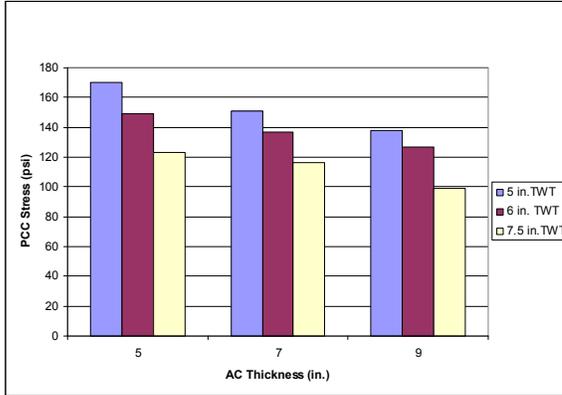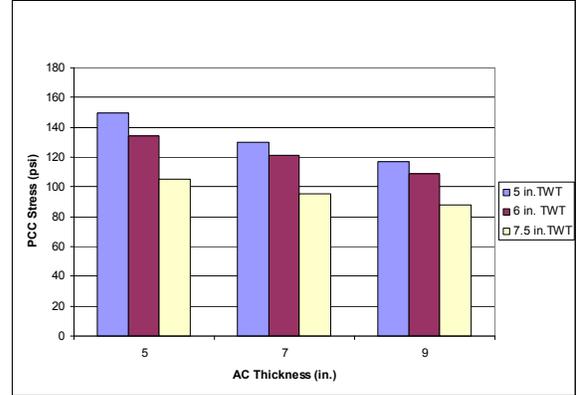
(a)                                          (b)

**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**
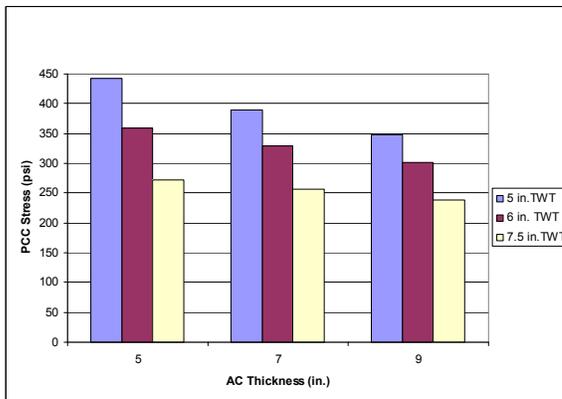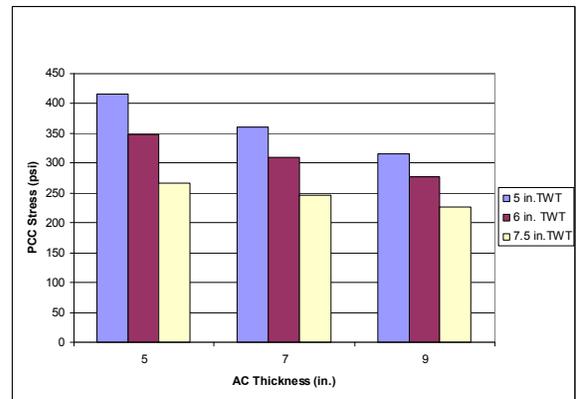


(a)                                          (b)

**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**

64
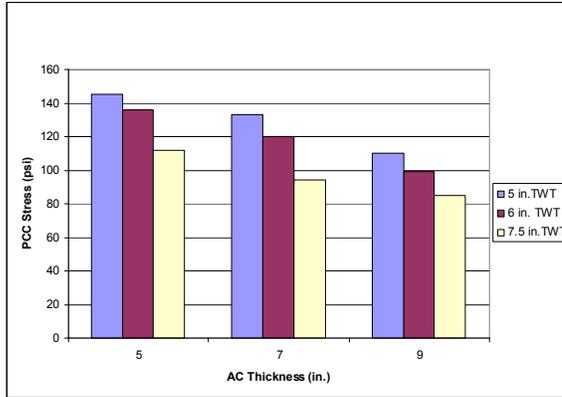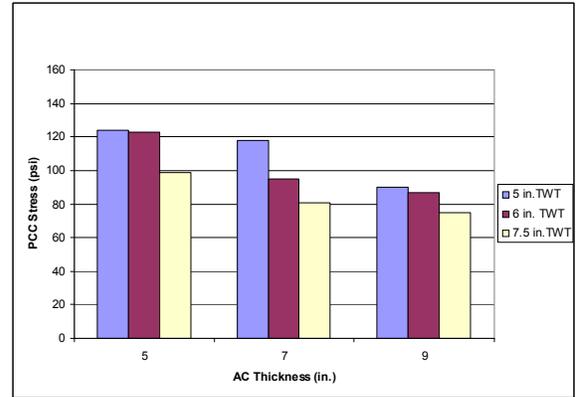
(a)                                                    (b)

**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**
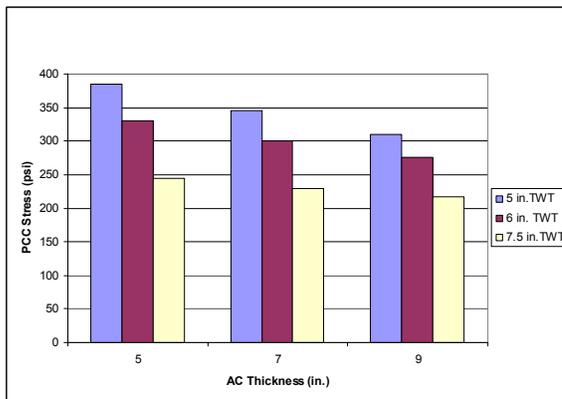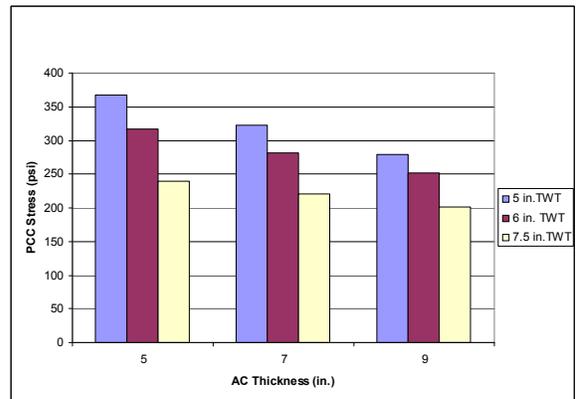


(a)                                                    (b)

**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

Existing AC modulus is representative of existing AC pavement condition. PCC stress vs. 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 the bonded one. For 250 ksi and 350 ksi AC

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moduli, tensile stresses were observed to be 442 psi and 416 psi for 5-in. unbonded TWT, 5-in. AC layer, and 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)
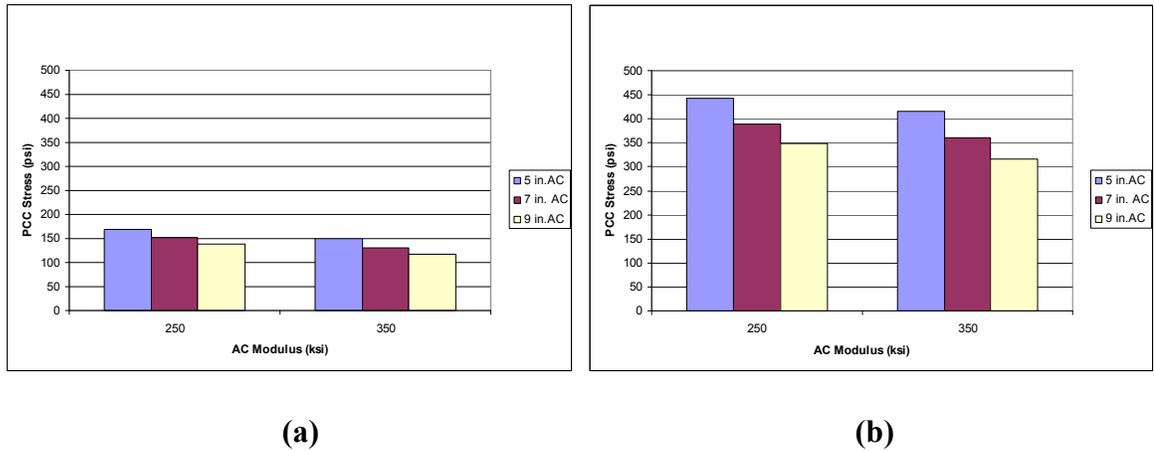
**Figure 4.19 PCC Stress vs. Existing AC Modulus Graph for 5-in. TWT and Unpaved Shoulder (a) Bonded and (b) Unbonded**



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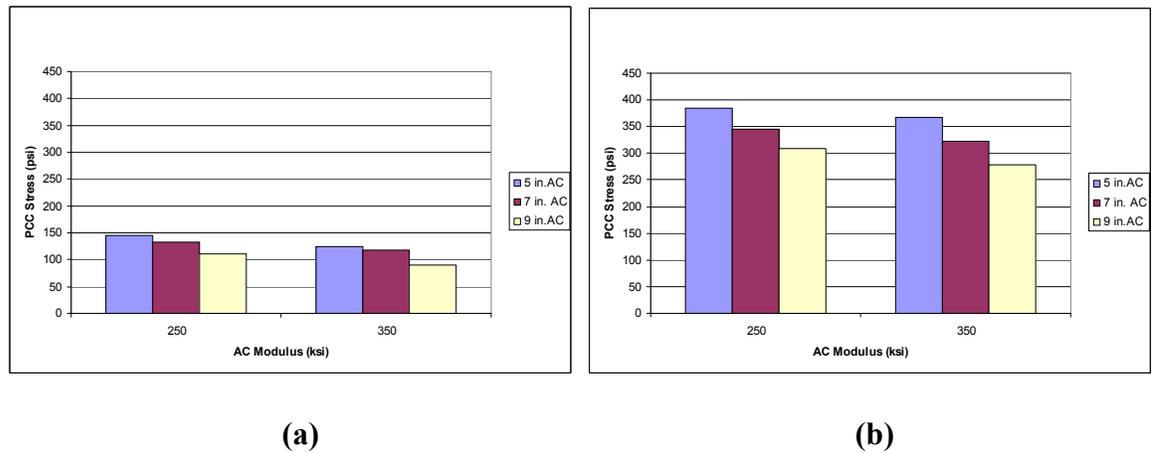
**Figure 4.20 PCC Stress vs. Existing AC Modulus Graph for 5-in. TWT and Paved Shoulder (a) Bonded and (b) Unbonded**
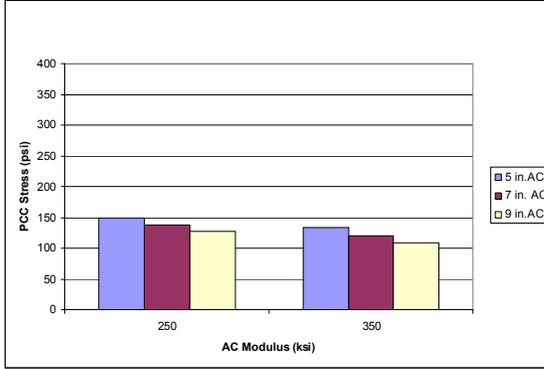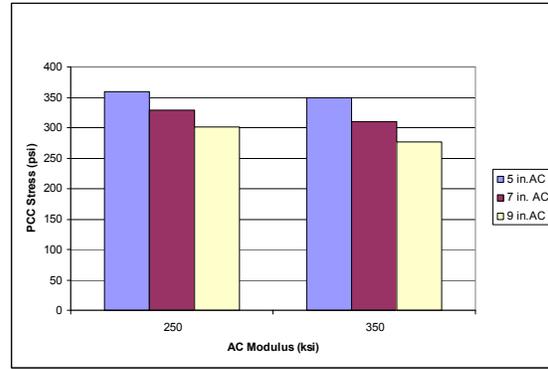
(a)                                                            (b)

**Figure 4.21 PCC Stress vs. Existing AC Modulus Graph for 6-in. TWT and Unpaved**

**Shoulder (a) Bonded and (b) Unbonded**
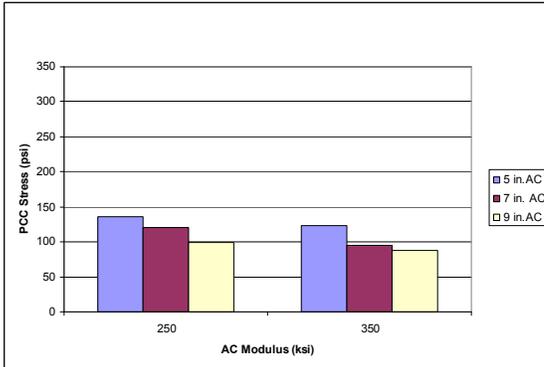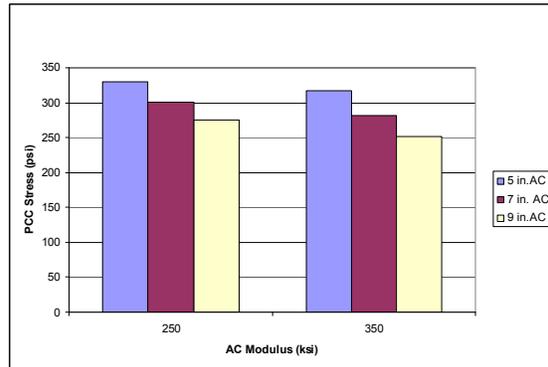


(a)                                                            (b)

**Figure 4.22 PCC Stress vs. Existing AC Modulus Graph for 6-in. TWT and Paved**

**Shoulder (a) Bonded and (b) Unbonded**
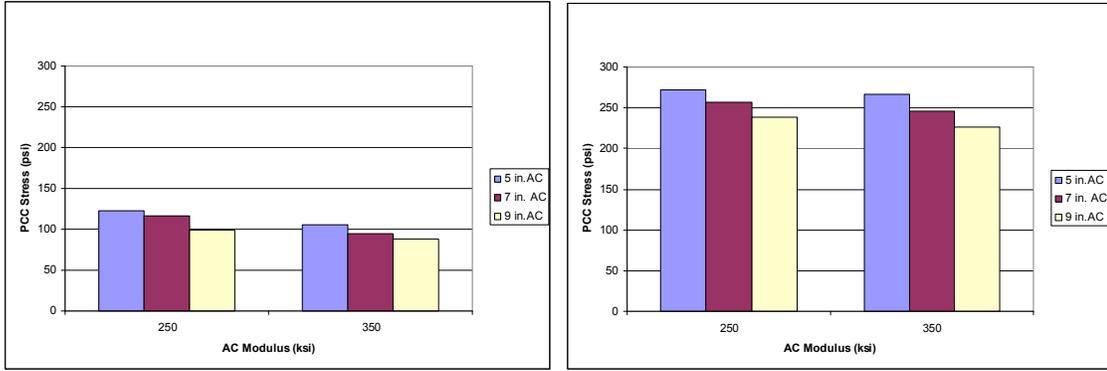
**(a)**                                         **(b)**

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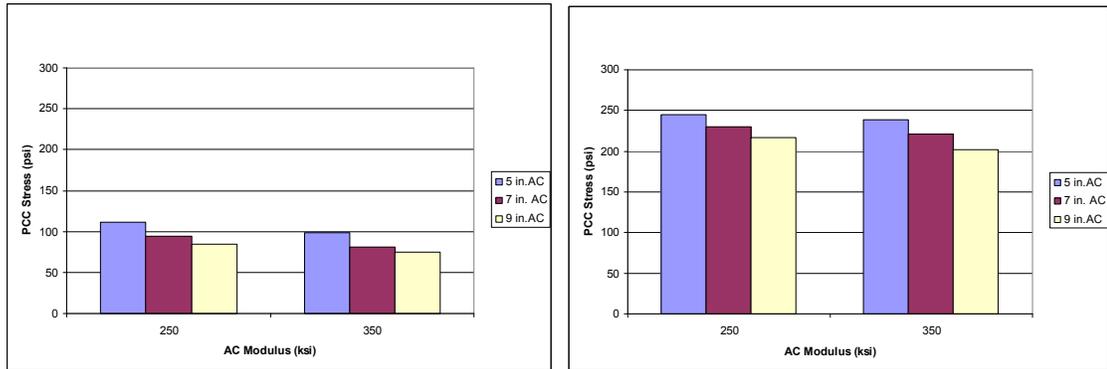


**(a)**                                         **(b)**

**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 in Figure 4.25 to Figure 4.30. For unbonded 5-in. TWT, 5-in. AC layer, and 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.
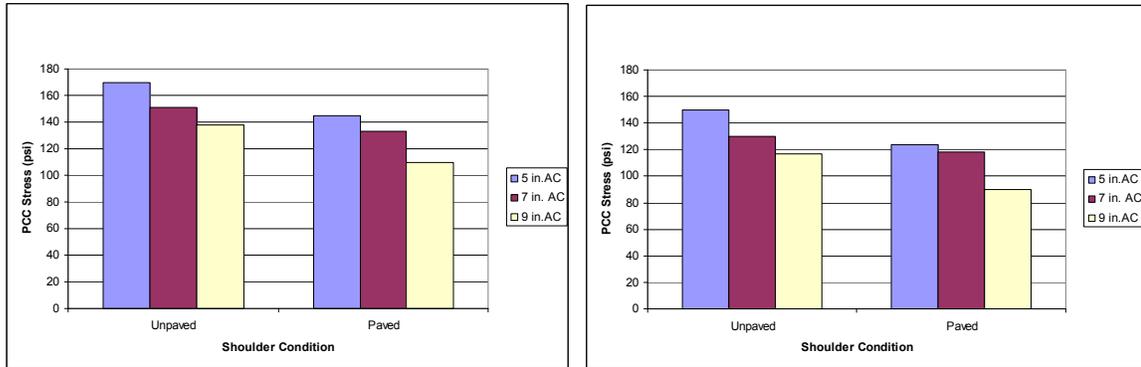


(a)                                                (b)

**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**
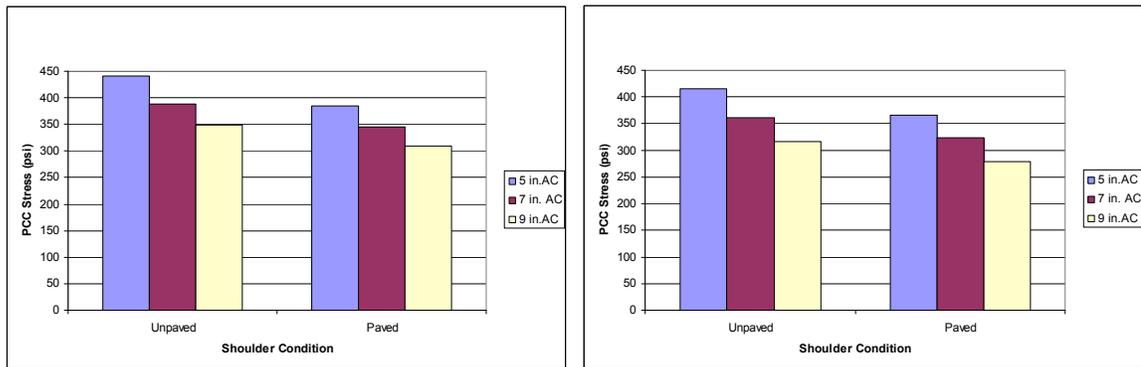


(a)                                                (b)

**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**
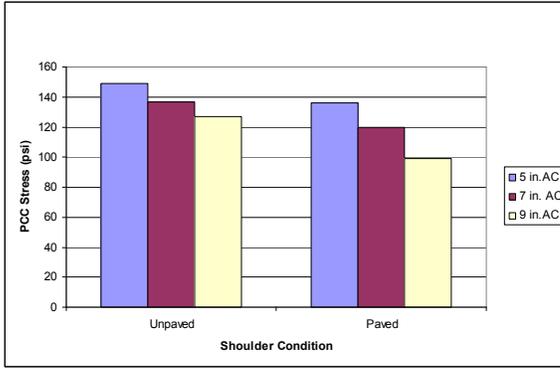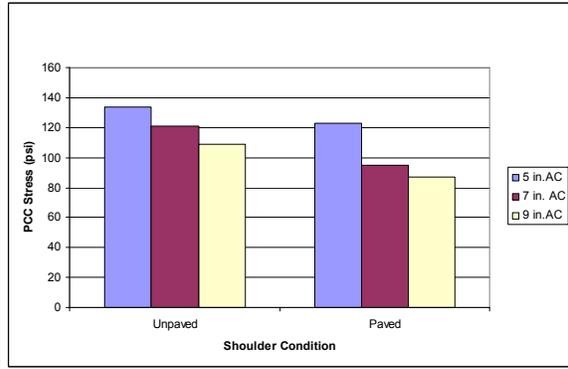
**(a)**                                        **(b)**

**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**
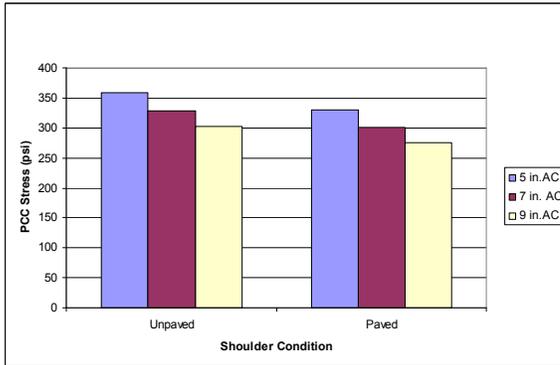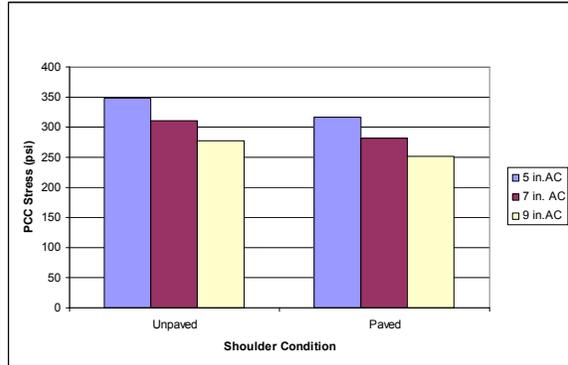


**(a)**                                        **(b)**

**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**
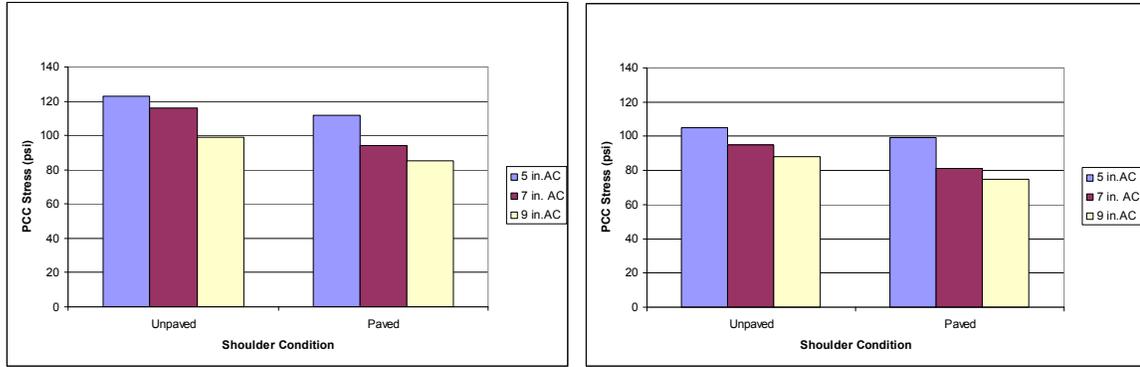
(a)                                                                (b)

**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**
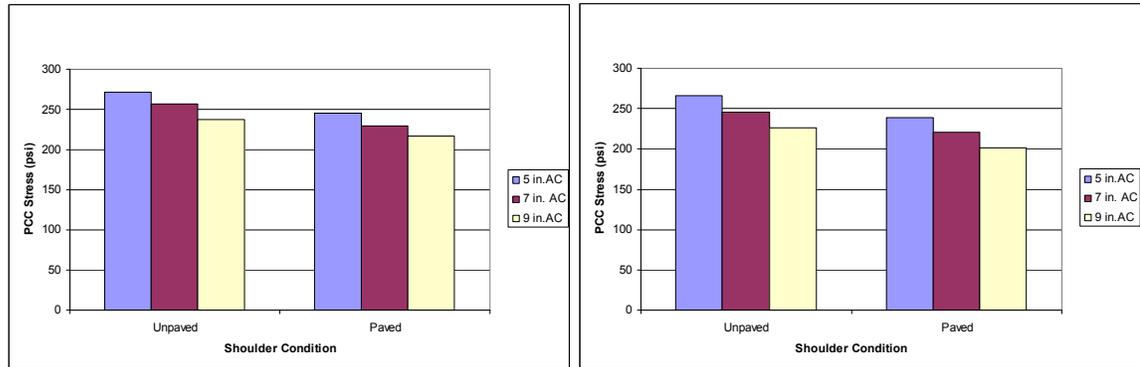


(a)                                                                (b)

**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 $1.5^0$F/in and $3^0$F/in on TWT were also examined. The contour plot of curling stress has been shown in Figure 4.31. Curling stresses for different TWT thicknesses have been shown in Figures 4.32 and 4.33. 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

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bonding condition and AC modulus have no effect on curling stress. For 5-in., 6-in., and 7.5-in. TWT, curling stresses for $1.5^0$F/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%.
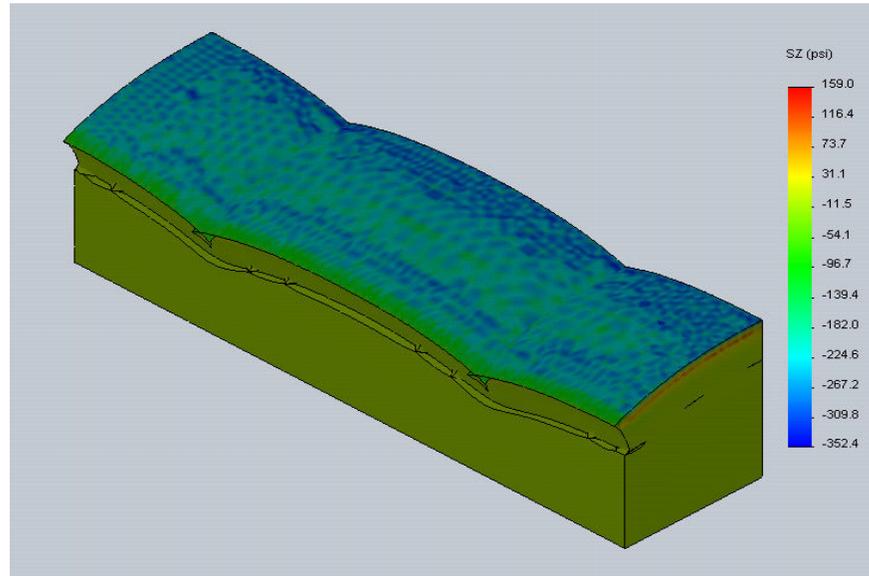


**Figure 4.31 Contour Plot of Curling Stress**
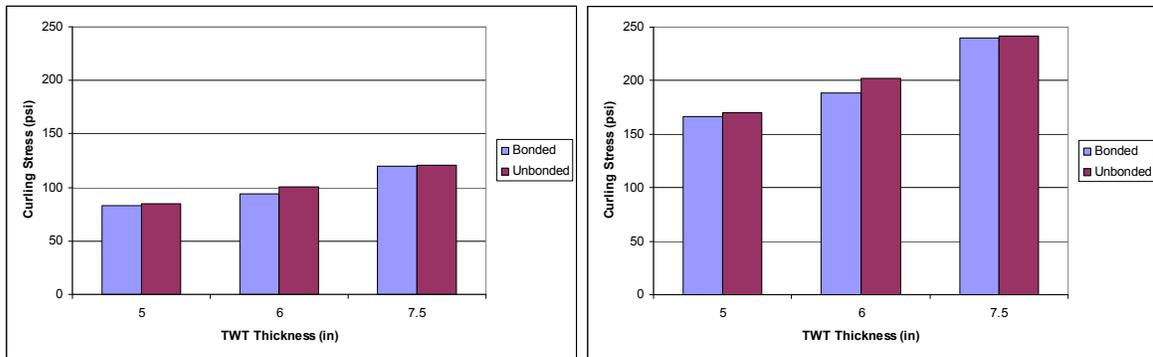


(a)                                                                          (b)

**Figure 4.32 PCC Stress vs. TWT Thickness for AC Modulus 250 ksi (a) Temp. Diff. of**

**$1.5^0$F/in and (b) Temp. Diff. $3^0$F/in**
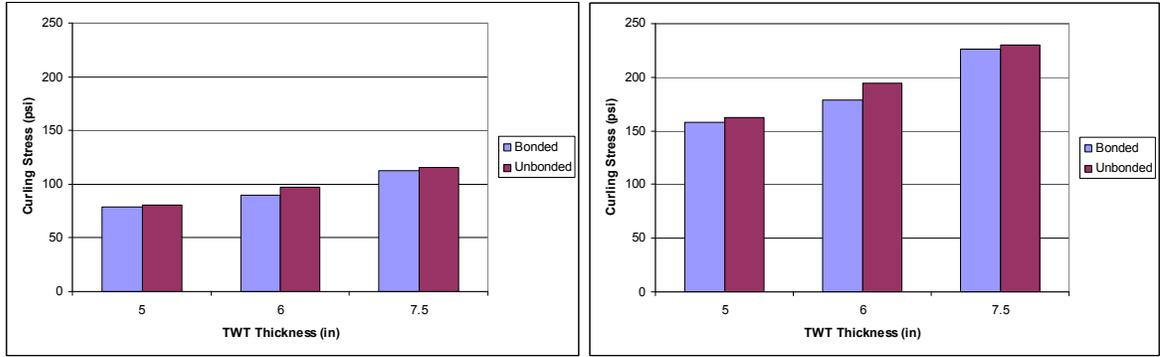
<div align="center">(a)                               (b)</div>

**Figure 4.33 PCC Stress vs. TWT Thickness for AC Modulus 350 ksi (a) Temp. Diff. of 1.5$^0$F/in and (b) Temp. Diff. 3$^0$F/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 4.1 shows the comparison of stresses obtained from SolidWorks and ANSYS. Fourteen to 19 percent differences in stresses were found for different TWT thicknesses. The employed version of ANSYS limits the user to create a mesh with up to 32,000 nodes. But 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 4.2 shows the comparison of stresses obtained from SolidWorks and KENSLABS. Twenty to 25 percent differences were found for different TWT thicknesses.

<div align="center">73</div>

For two layers model, KENSLABS ignores the properties of the bottom/foundation layer. In the current study, KENSLABS can not 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 rapture of concrete) ratio, and

N = 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, considering durability. For different parameters considered in this study, Tables 4.3, 4.4, and 4.5 represent the design catalogs of TWT for zero, 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 | | Unpaved | 5 | 10 | 10 | 10 | 10 |
| 7 | | Bonded | | 6 | 10 | 10 | 10 | 10 |
| 9 | 350 | | Paved | 7.5 | 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 |
| | 350 | | | 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 | | Unpaved | 5 | 10 | 10 | 10 | 10 |
| 7 | | Bonded | | 6 | 10 | 10 | 10 | 10 |
| 9 | 350 | | Paved | 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 | | Unpaved | 5 | 10 | 10 | 10 | 10 |
| 7 | | Bonded | | 6 | 10 | 10 | 10 | 10 |
| 9 | 350 | | Paved | 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 asses 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 existing AC layer is the most important factor that affects 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. Thus, partial bonding at the interface should be investigated in detail.

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