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INSTRUMENTATION AND FIELD TESTING OF THIN WHITETOPPING PAVEMENT IN COLORADO AND REVISION OF THE EXISTING COLORADO THIN WHITETOPPING PROCEDURE

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DEPARTMENT OF TRANSPORTATION
RESEARCH

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COLORADO DEPARTMENT OF TRANSPORTATION
RESEARCH BRANCH

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16. Abstract
This report summarizes the verification and revision of a thin whitetopping pavement mechanistic design procedure developed for the Colorado Department of Transportation. The original whitetopping procedure and design guidelines were developed during a 1998 study on thin whitetopping pavements in Colorado. This report includes information on the installation and construction of the test sections, instrumentation, field and laboratory testing, data acquisition, and data analysis.

The revised Colorado thin whitetopping pavement design procedure provides improved predictions of whitetopping load responses, and therefore should also provide more accurate insights into longer-term performance of thin whitetopping pavements for highway applications. The successful development and revision of a second-generation thin whitetopping design procedure provides an additional level of confidence for designers and highway agencies when considering this rehabilitation technique. Two different procedures were developed to calculate the thickness. One is a mechanistic approach incorporating finite element program, ILLI-SLAB to predict critical concrete stresses and asphalt strains. The second method is an empirical approach, incorporating the number of expected equivalent 18-kip single axle loads (ESLA).

Implementation:
This study validated and improved the predictions of load responses of the first generation of whitetopping procedures developed for Colorado during a 1998 study. These procedures are now being used by CDOT's pavement engineers and provide a more accurate insight into performance of thin whitetopping pavements for highway applications.

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INSTRUMENTATION AND FIELD TESTING OF THIN WHITETOPPING PAVEMENTS IN COLORADO AND REVISION OF THE EXISTING COLORADO THIN WHITETOPPING DESIGN PROCEDURE

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EXECUTIVE SUMMARY

Ultra-thin whitetopping (UTW), thin whitetopping (TWT) and conventional whitetopping (CWT) are portland cement concrete (PCC) resurfacing techniques for asphalt pavement rehabilitation. This report summarizes the verification and revision of a thin whitetopping pavement mechanistic design procedure developed for the Colorado Department of Transportation. The original whitetopping design guidelines and procedure was developed during a 1998 study on thin whitetopping pavements in Colorado.

Thin whitetopping pavement concrete thicknesses are typically considered to be between 4 and 8 in. Unlike the conventional whitetopping approach that has been used more extensively in the past, the TWT technique recognizes that certain bonding strength exists between the concrete overlay and the existing asphalt layer, which reduces concrete bottom flexural stresses. While whitetopping overlays have been constructed since 1918, design guidelines for this rehabilitative technique have not been available until recently. In 1994, the Portland Cement Association sponsored research to develop a procedure for the design of ultra-thin whitetopping pavement. However, these ultra-thin concrete overlays (4 in. or less) require closely spaced joints and are likely not practical for rehabilitating higher volume traffic roadways. To meet the highway traffic serviceability requirements of the Colorado Department of Transportation, slightly thicker concrete sections (4 to 8 in.) and wider joint spacings (up to 12 ft) were studied in both the 1998 original and this 2004 research project.

Three sites were evaluated in the 1998 study; the first is a U.S. 85 frontage road near Denver, Colorado; the second is along S.H. 119 near Longmont, Colorado; and the third is on U.S. 287 near Lamar, Colorado. Test slabs with various design features were constructed, instrumented and load tested at each of these thin whitetopping test pavements. Static load testing to collect pavement response measurements was typically performed at 28 days and one year after test section construction. The pavement response data were used to develop a first generation design procedure and general design guidelines for thin whitetopping pavement design for the Colorado Department of Transportation.

Included in the original project findings were recommendations for existing asphalt surface preparation prior to paving the concrete overlay (cold milling), minimum subgrade support conditions for using TWT (150 psi/in. or greater), minimum asphalt layer thicknesses necessary in TWT applications (5 in. or greater), concrete stress and asphalt strain interface partial bonding calibration factors (165% and 84% of fully-bonded modeled stress and strain responses, respectively), and pavement response prediction equations for performing a mechanistic-empirical analysis and design of thin whitetopping sections. Also among the original study findings was that additional investigation and verification of the thin whitetopping pavement performance characteristics and design guidelines developed was recommended.

A thin whitetopping rehabilitation of an existing asphalt pavement section on S.H. 121, Wadsworth Boulevard, near Denver, Colorado was scheduled for 2001. This provided an opportunity to construct, instrument and load test additional TWT test sections and use the data to calibrate and verify the observations and design procedure developed in the 1998 study. As a result, the Colorado Department of Transportation and the Federal Highway Administration co-sponsored this second study to further investigate thin whitetopping applications in Colorado and verify or modify the existing design procedure and guidelines.

The thin whitetopping pavement along Colorado S.H. 121 was constructed in 2001, and four TWT test sections with varied thicknesses and joint spacings were constructed and tested for this

study. The test slabs were constructed, instrumented and load tested using techniques similar to those performed for the original research project, and static load testing to collect pavement response measurements was performed at 28 days and two years after test section construction. The new pavement response data were used to verify and revise the original study findings and create a second-generation design procedure. In addition, the original thin whitetopping test pavements were revisited to perform condition surveys, collect cores and perform falling weight deflectometer measurements after the sections were in service for approximately 7 years. The overall performance of the original test sites appeared to be outstanding, with very minimal distress observed in any of the sections after multiple years of service. The 2004 project findings include revised calibration factors for modeled thin whitetopping concrete stresses and asphalt strains (151% and approximately 89% for stresses and strains, respectively), recommended joint spacings (6 ft.), elimination of the minimum required subgrade support condition and existing asphalt thickness limitations specified in the 1998 study, recommendations to include tied concrete shoulders, and recommendations for continued long-term performance monitoring of all thin whitetopping test sections included in the two studies.

This report presents information and discusses the instrumentation, construction, testing, analysis of data, findings and recommendations based on the research tasks performed related to the four Colorado thin whitetopping test sections included in the two studies. The revised Colorado thin whitetopping pavement design procedure presented provides improved predictions of whitetopping load responses, and therefore should also provide more accurate insights into longer-term performance of thin whitetopping pavements for highway applications. The successful development and revision of a second-generation thin whitetopping design procedure provides an additional level of confidence for designers and highway agencies when considering this rehabilitation technique.

IMPLEMENTATION STATEMENT

Whitetopping is quickly becoming a popular method used nationwide to rehabilitate deteriorated asphalt pavements. Since the flexible asphalt surface is replaced by rigid concrete, the technique offers superior service, long life, low maintenance, low life-cycle cost, improved safety, and environmental benefits. The critical stress and strain prediction equations developed and revised during this research are part of a second-generation design procedure, which can be verified and/or modified with the collection of additional data from future research projects.

INSTRUMENTATION AND FIELD TESTING OF THIN WHITETOPPING PAVEMENTS IN COLORADO AND REVISION OF THE EXISTING COLORADO THIN WHITETOPPING DESIGN PROCEDURE

by

Matthew J. Sheehan, Scott M. Tarr and Shiraz D. Tayabji*

1.0 INTRODUCTION

Ultra-thin whitetopping (UTW), thin whitetopping (TWT) and conventional whitetopping (CWT) are portland cement concrete (PCC) resurfacing techniques used for asphalt pavement rehabilitation that have gained considerable interest and greater acceptance in the last decade. Whitetopping techniques involve placing a concrete overlay on deteriorated or partially deteriorated asphalt pavements. Ultra-thin, thin and conventional whitetopping thicknesses are typically considered to be between 2 to 4 in., 4 to 8 in. and above 8 in., respectively. Unlike the conventional whitetopping approach that has been used more extensively in the past, the UTW and TWT techniques recognize that certain bonding strength exists between the concrete overlay and the existing asphalt layer [(1-4)]. The UTW and TWT pavements, therefore, behave as composite pavements. In addition, shorter joint spacings, typically from 2 to 12 ft depending on slab thickness, have been used for UTW and TWT pavements. The existence of interface bonding and the use of short joint spacings minimize slab bending, potential for shrinkage cracking, slab curling and warping, and reduce the required slab overlay thickness. Thin whitetopping pavements are often used for state and secondary highways subjected to moderate truck traffic, UTW pavements are typically intended for city streets or intersections with minimal truck traffic, and CWT pavements are usually intended for heavier traffic conditions [(1,2,4,5-9)].

Plain concrete, reinforced concrete, and fibrous (fiber reinforced) concrete have been used over the years for whitetopping pavements [(4,5,10,11)]. In the 1940's and 1950's, plain concrete was mainly used in airports, both civil and military. Thickness of concrete used in these projects ranged from 8 to 18 in. (200 to 460 mm). Since 1960, plain concrete has been used extensively to resurface existing highway pavements in states such as California, Utah and Iowa. Concrete thicknesses of these resurfacing projects typically ranged from 7 to 10 in. (175-250 mm). Continuous-reinforced concrete and fiber-reinforced concrete were also used on a limited number of projects. In 1994, NCHRP Synthesis 204 [(12)] listed 189 whitetopping street, highway and airfield pavement projects constructed in the United States since 1918. The increased interest is partially because whitetopping technology has improved over the years as concrete paving technology itself has improved [(4,13-15)].

Whitetopping asphalt pavements with portland cement concrete can provide long-term benefits to the traveling public as well as the highway or airport agency. The proven durability and long-term performance of a PCC surface decrease the maintenance time and life cycle cost of the pavement. This advantage significantly reduces the public agency time, cost and user delays accompanying the frequent required maintenance of an asphalt surface. These advantages, in

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addition to the improvement in skid resistance and safety (especially under wet conditions), make whitetopping pavements compare favorably to asphalt surfaces [17-20].

Design and construction procedures of conventional whitetopping are well established and explained in detail in Portland Cement Association (PCA) and American Concrete Pavement Association (ACPA) publications [2,4,6,7]. Features including minimum slab thickness, support characterization and pre-overlay preparation are discussed in these publications. Prior to these studies, there were no whitetopping guidelines to help the designer determine the required PCC thickness for the specific material and environmental parameters encountered. The pavement was either designed as a fully bonded or entirely unbonded concrete overlay. Many states, including Georgia, Tennessee, Kentucky and Colorado, constructed whitetopping test sections on a trial and error basis. Without design guidelines, if the pavement is over-designed, the section performs well at a high inital cost. If the pavement is under-designed, the section requires maintenance or reconstruction and diminishes the users confidence in whitetopping as a pavement rehabilitation technique. Therefore, there is a need for rationally developed whitetopping thickness design guidelines. Research testing conducted during this and the previous study [1] allowed the development of a mechanistic whitetopping design procedure for the State of Colorado.

2.0 BACKGROUND

Prior to the development of the 1998 Colorado thin whitetopping design guidelines and current revisions discussed in this report, the Portland Cement Association and American Concrete Pavement Association sponsored a research study to develop thickness design guidelines for ultra-thin whitetopping pavements [2]. As part of the PCA/ACPA study, thin slabs (2 to 4 in.) with short joint spacings (24 to 50 in.) located at the Spirit of St. Louis Airport in Chesterfield, Missouri were instrumented with strain gages and loaded using a 20 kip single axle load (SAL). The ultra-thin whitetopping test results indicated that the slab corner was the critical ultra-thin whitetopping load location, and the critical location inducing maximum asphalt strain occurred at the midpoint of the longitudinal joint.

The effects of whitetopping concrete and asphalt interface partial bonding were quantified using the PCA/ACPA study field testing results. Measured field load-induced flexural stresses were compared to fully bonded theoretical stresses to determine an adjustment factor increasing modeled ultra-thin whitetopping load stresses due to the partially bonded condition. A factor of 1.36 (36% increase in stress due to partial bonding) was determined based on the field data. This adjustment factor was applied to stresses computed during a parametric study to convert and adjust modeled stresses to simulate measured field behavior. Linear regression techniques were then used to develop prediction equations for the ultra-thin whitetopping section critical stresses. The equations included parameters of the whitetopping pavement that have a significant impact on the induced concrete flexural stresses and asphalt flexural strains. The PCA/ACPA design procedure was developed as a guide for determining the PCC thickness necessary for ultra-thin whitetopping applications in low-volume traffic situations such as intersections, streets and off ramps.

The State of Colorado was also interested in using thin whitetopping as a technique for rehabilitating deteriorated asphalt highway pavements. However, the PCA/ACPA design procedure did not include concrete thicknesses and joint spacings Colorado state officials considered acceptable for the highway projects being considered for whitetopping rehabilitation. Therefore, research was initiated to develop thin whitetopping design guidelines for the State of Colorado.

The Colorado DOT rationale for developing a thin whitetopping design procedure that incorporates stress correction factors was primarily to take advantage of the partial bonding phenomenon between the concrete and asphalt layers. This would allow the DOT to construct the most economical whitetopping pavements possible and increase the feasibility of thin whitetopping as a rehabilitation technique for asphalt pavements. Therefore, in 1996 the Colorado Department of Transportation (CDOT) and Portland Cement Association co-sponsored and initiated a research project to develop a mechanistic design procedure for thin whitetopping pavements [(1)]. This project involved construction of three TWT pavements containing many test sections with field instrumentation. Construction Technology Laboratories, Inc. (CTL) installed the instrumentation, conducted the load testing on the instrumented test sections, performed a theoretical analysis, and developed a thin whitetopping design procedure for CDOT. Many variables were considered in the construction of the test sections, including concrete overlay thickness, slab dimension, existing asphalt layer thickness, different asphalt surface preparation techniques, and the use of dowel and tie bars.

The general techniques used in the development of the PCA/ACPA ultra-thin whitetopping design procedure were also used to develop the original Colorado guidelines. Field testing was conducted to evaluate critical load locations for the thicker PCC layer and larger joint spacings. The load-induced flexural strains were used to calibrate fully bonded stresses computed using finite element analysis techniques with partially bonded stresses measured in the field. Load testing was also conducted throughout the course of a day to develop temperature corrections for the load responses. Equations predicting the critical concrete flexural stresses and asphalt concrete strains for use in whitetopping design were developed, and thickness design guidelines were established for partially bonded thin whitetopping pavements using field calibrated theoretical stresses.

In addition to the Colorado mechanistic thin whitetopping design procedure originally developed in 1998 (based on an axle load distributions obtained from traffic monitoring data), the Colorado Department of Transportation also requested that the procedure be converted so that the empirical theory of Equivalent Single Axle Loads (ESALs) could be used as the traffic input information. This required extrapolating AASHTO axle load conversion factors to include typical thin whitetopping thicknesses because the AASHTO design procedure does not suggest conversion factors for a pavement thickness below 6 in. Two ESAL conversion factors were developed based on actual traffic data (for Primary and Secondary Highways) supplied by CDOT for 8-in.-thick conventional pavements. In addition to ESAL conversion factors, a nonlinear relationship was realized for PCC thicknesses determined using the empirical (ESAL) and mechanistic (axle load) procedures. An additional conversion factor was derived to equilibrate the empirical and mechanistic design methods.

The 1998 Colorado study design guidelines have been regarded as a first-generation thin whitetopping pavement design procedure. However, the procedure needed to be further calibrated, verified and/or modified as more performance data became available. As stated in the 1998 report, there are several observations and conclusions regarding using TWT pavements for rehabilitation that should be examined more extensively. Among the original study items that require more investigation are the minimum existing asphalt thickness required for thin whitetopping applications and the required subgrade modulus of reaction. The results of the 1998 study indicate that a minimum of 5 in. of asphalt and a minimum subgrade k-value of 150 psi/in. is needed for thin whitetopping pavements to perform adequately. These minimum requirements cause concern because they are relatively common values encountered in pavement applications; many of the Colorado asphalt pavements that are potential candidates for thin whitetopping rehabilitation have thickness and support conditions near these minimum values. Therefore, additional investigation of these perceived limitations was desired by CDOT and recommended in the original report.

During the 2001 construction season, the Colorado Department of Transportation planned to construct a new four-mile long thin whitetopping pavement section on Colorado State Highway 121, Wadsworth Boulevard, near Denver, Colorado. This project provided an excellent opportunity to collect additional data that could be used for verification and modification of the original 1998 design procedure and guidelines. As a result, the Colorado Department of Transportation and the Federal Highway Administration (FHWA) co-sponsored a second study to further investigate thin whitetopping applications in Colorado and verify or modify the existing design procedure and guidelines.

The thin whitetopping pavement and test sections along Colorado S.H. 121 were constructed in the summer of 2001 and load tested in 2001 and 2003. This report presents information related to instrumentation, construction, testing and analysis of data from those thin whitetopping test sections. The final product of this study will be a verified or revised thin whitetopping pavement design procedure and a better understanding of longer-term performance of TWT pavements for highway applications.

Currently, the use of thin whitetopping pavements has still been limited to a few states, and many of the TWT pavements are still in the experimental stage. The successful development of a second-generation thin whitetopping design procedure should provide an additional level of confidence for designers when considering this technique. It will also ultimately provide state highway agencies another technical reference and engineering evaluation tool that may make them more comfortable selecting this technique for rehabilitation applications on a more routine basis.

3.0 OBJECTIVES AND SCOPE

The overall objectives of this project are to revise the current Colorado thin whitetopping pavement design guidelines and to further study thin whitetopping pavement behavior and performance in highway applications. These objectives will be accomplished by conducting the following scope of work.

- Literature and document review
- Instrumentation, construction and field load testing of newly constructed test sections
- Condition survey and performance evaluation of the test pavements included in the study
- Laboratory testing for material characterization and interface bond strength determination
- Verification and validation of the current design procedure using the new data
- Assessment and revision of current Colorado thin whitetopping design procedure.

The information documented and methods developed and revised during this project will contribute to the advancement of whitetopping technology through increased knowledge of techniques and considerations critical for constructing whitetopping pavements.

4.0 EXPERIMENTAL DESIGN APPROACH AND FIELD TESTING PROGRAM

Field instrumentation and load testing were conducted at four different sites in Colorado between 1996 and 2003 to develop, verify and revise design guidelines for bonded whitetopping pavement systems. The objective of the field testing was to collect data that could be used to:

- Determine and confirm the critical load location of whitetopping pavements
- Evaluate the effects of different AC surface preparation techniques
- Measure the response of whitetopping pavements under traffic loading
- Evaluate interface bond strength between the concrete and the asphalt layers
- Investigate the effect of pavement age on load-induced stresses
- Develop and verify the calibration for theoretical stress predictions with measured responses
- Develop, verify and revise thin whitetopping thickness design guideline equations.

Four test pavement sites were investigated as part of this study. Each site had multiple test sections and test slabs. The first two test sites were constructed during the summer of 1996 and were load tested at approximately 28 days and 1 year after construction. The third test section was constructed during the summer of 1997 and was only tested at 28 days after construction. The fourth section was constructed in 2001 and tested at 28 days and two years after construction.

The first three test sections included in the study were used to create the original thin whitetopping design guideline equations, and discussion of these sections will be included in the following report sections on a limited basis. The fourth test section will be the primary focus in the remainder of this report because data from this section are used to validate, confirm and revise the results and guidelines developed in the original study.

4.1 CDOT Pavement Test Sections

Four thin whitetopping test sections were constructed in Colorado between 1996 and 2001. The first two were constructed in 1996 and were located along a U.S. 85 Frontage Road near Denver, Colorado and S.H. 119 near Longmont, Colorado. The third was constructed in 1997 and was located on U.S. 287 south of Lamar, Colorado. The fourth section was constructed in 2001 on S.H. 121, Wadsworth Boulevard, near Denver, Colorado. Approximate traffic levels for each of these pavement sections are presented in Table 4.1. The original three sections will be discussed briefly and the fourth more extensively in the following subsections. Information regarding additional thin whitetopping sections constructed in Colorado is listed in Appendix A.

4.1.1. United States Route 85 Frontage Road, Santa Fe Drive, Denver, Colorado

The first test project (CDOT1) was constructed in 1996 on a frontage road to U.S. 85 Santa Fe Drive in Denver, Colorado. This project had a total length of 1,000 feet, consisting of two main 500-ft test sections and a third additional subsection.

The first test section at the U.S. 85 site had nominally 4-in.-thick concrete slabs paved on top of a 5-in.-thick newly placed asphalt pavement layer. No special asphalt surface preparation was attempted. The second section had nominally 5-in.-thick concrete slabs on top of a 4-in.-thick asphalt layer. A portion of the asphalt surface in the second test section was milled creating a third test section. All concrete slabs had a 60 in. joint spacing. Tie bars were installed along longitudinal joints, except those between curbs and traffic lanes, and no dowel bars were used in transverse joints. Both longitudinal and transverse joints were sawcut to 1/3 of the concrete slab depth. Soil underneath the pavement reportedly had a modulus of subgrade reaction (k) of approximately 150 psi/in. Table 4.2 presents a summary of the test sections and certain section characteristics.

Three slabs in the first project were instrumented and load tested at Santa Fe Drive. Slab 1 consisted of a 4-in.-thick concrete layer on top of a 5-in.-thick asphalt layer and slabs 2 and 3 had 5-in. of concrete on a 4-in.-thick asphalt layer. All test slabs were located in the southbound lane and were adjacent to the curbs. Since no tie bars were used along joints between curbs and traffic lanes, all three test slabs had a tied joint on the east side and a free edge on the west side.

Table 4.1. Approximate Traffic Levels for Colorado Whitetopping Test Sections

Route	Annual Average Daily Traffic (AADT)	Percent Trucks
U.S. 85	1,500	25%
S.H. 119	19,760	8%
U.S. 287	2,287	59%
S.H. 121	44,562	3%

Notes:
1. Source: Colorado Department of Transportation.
2. Data based on 2002 traffic surveys.
3. No data available for U.S. 85 frontage road; values are estimated.

4.1.2 Colorado State Highway 119, Longmont, Colorado

The second whitetopping test project in Colorado (CDOT2) involved a 1996 rehabilitation of the two eastbound lanes of an approximately one-mile long, divided four-lane existing asphalt pavement on S.H. 119 near Longmont, Colorado. Many variables were incorporated in this project, including various concrete slab dimensions and thicknesses, with different asphalt surface preparation. Three different asphalt surface preparation techniques were utilized. On the east half of the project, a 1-½ in. new asphalt layer

Table 4.2. Test Slab Characteristics and Test Results

Site	Test Slab	PCC Thickness, in.	AC Thickness, in.	Longitudinal Joint Spacing, in.	Transverse Joint Spacing, in.	AC Resilient Modulus, psi	AC Surface Condition	Modulus of Subgrade Reaction, psi/in.	28-Day Interface Shear Strength, psi	1-Year Interface Shear Strength, psi	2-Year Interface Shear Strength, psi	6/7-Year Interface Shear Strength, psi
U.S. 85 Santa Fe	1	4.7	4.5	60	60	350,000	New	150	45	80	****	****
	2	5.8	5.9	60	60	350,000	New	150	30	60	****	80
	3	6.0	5.4	60	60	350,000	New Milled	150	10	80	****	110
S.H. 119 Longmont	1	5.1	3.3	72	72	800,000	Existing	340	100	****	****	****
	2	5.4	4.6	120	144	800,000	New	340	60	105	****	140
	3	6.3	3.4	72	72	800,000	New	340	70	105	****	****
	4	7.3	3.4	72	144	800,000	Existing Milled	340	65	100	****	100
	5	6.8	2.8	144	144	800,000	Existing Milled	340	****	155	****	105
U.S. 287 Lamar	B	7.4	7.0	144	120	800,000	Existing Milled	225	80	****	****	****
	E	6.8	6.6	72	72	800,000	Existing Milled	225	90	****	****	130
	F	5.6	6.6	72	72	800,000	Existing Milled	225	110	****	****	****
S.H. 121 Wadsworth	1	4.1	5.3	48	48	398,000	Existing Milled	500	115	****	****	****
	2	4.4	5.5	72	72	288,000	Existing Milled	500	245	****	85	****
	3	7.0	4.6	72	108	334,000	Existing Milled	500	170	****	****	****
	4	6.3	5.0	72	72	394,000	Existing Milled	500	160	****	145	****

was placed on top of the existing asphalt pavement, with a nominal concrete slab thickness of 5 in. In the passing lane of the west half of the project, 4 ½ in.-thick concrete slabs were placed directly on top of the existing asphalt pavement. In the traffic lane, the asphalt pavement was milled 1 ½ in., resulting in a nominal concrete slab thicknesses of 6 in. No particular effort was made to clean the asphalt surface. However, all the asphalt pavement surfaces were washed prior to concrete placement. Tie bars were used for most of the longitudinal joints. Dowel bars were only installed along the transverse joints of slabs with longer joint spacings (12 ft). The modulus of subgrade reaction for the entire project was reportedly 340 pci.

Five slabs were instrumented with strain gages and load tested at the Longmont site. Slabs had different dimensions, concrete slab thickness and concrete-asphalt interface conditions. Concrete design thicknesses ranged from 4.5 to 6 in., although the as-constructed thicknesses ranged from about 5.1 to 7.3 in. Asphalt thicknesses ranged from approximately 3 to 5 in., and joint spacings ranged from 6 to 12 ft. The asphalt surface consisted of old asphalt concrete, new asphalt concrete and milled asphalt concrete. Test slabs were primarily located in the outside driving lane with tied concrete shoulders. Table 4.2 presents a summary of the test sections and certain section characteristics.

4.1.3 United States Route 287, Lamar, Colorado

The third whitetopping test project in Colorado (CDOT3) involved a 1997 rehabilitation of an approximately three-mile section of two-lane pavement on heavily truck-trafficked U.S. 287 near Lamar, Colorado. The two main variables included in six project test sections consist of concrete slab dimensions and joint reinforcement. Both the north and southbound existing asphalt lanes and shoulders were milled and thoroughly cleaned prior to concrete placement. The milled asphalt thickness was approximately 7 in. The design specified a 6 in. concrete whitetopping slab with concrete shoulders and was based on a 225-psi/in. modulus of subgrade reaction. Tie bars were used for all the longitudinal joints at varying spacing. Except for one section, dowel bars were installed at all transverse joints at varying spacing.

Three slabs were instrumented with strain gages and load tested at the Lamar site. Thicknesses ranged from 5.5 to 7.3 in. and 6.5 to 7.5 in. for the PCC and AC layers, respectively. Joint spacings ranged from 6 to 12 ft. Test slabs were located in the outside driving lane and all concrete shoulder joints were tied. Table 4.2 presents a summary of the test sections and certain section characteristics.

4.1.4 Colorado State Highway 121, Wadsworth Boulevard, Denver, Colorado

The S.H. 121 thin whitetopping pavement test section (CDOT4) was the primary focus of this study, which includes an effort to confirm or revise the findings from the original test sections. The test section was constructed in 2001 and is located on a 4-mile long TWT pavement project on S.H. 121, between Colorado State Highway Route C 470 and Park Hill Avenue, south of Denver, Colorado. This section of S.H. 121 is a four-lane divided secondary arterial with stoplights at the intersections. The general design of the TWT project included an undoweled whitetopping overlay of 6 in. with 6-ft joint spacings in both directions. The whitetopping section was designed to carry approximately 1.3 million 18-kip equivalent single axle loadings (ESALs) over a 10-year design period. The original asphalt concrete thickness for this pavement was nominally 5-1/2 inches, but the existing asphalt surface was milled to promote improved interface bonding between the existing asphalt and new concrete. The general design information for the TWT section is presented in Table 4.3, and as-constructed parameters are included in Table 4.2, which was already presented.

Table 4.3. General Pavement Design Information

Roadway	Design Parameter	Value
SH 121	Highway Category	Secondary
(C 470 to Park Hill)	Design Life (years)	10
	Design Traffic (18-kip ESAL)	1,272,000
	Joint Spacing (in.)	72
	Concrete Elastic Modulus (psi)	3,400,000
	Concrete Poison's Ratio	0.15
	Existing AC Thickness (in.)	5-1/2
	AC Elastic Modulus (psi)	266,000
	AC Poison's Ratio	0.35
	Modulus of Subgrade Reaction (psi/in.)	500
	Design Concrete Overlay Thickness (in.)	6

Two primary experimental variables, concrete slab thickness and joint spacing, were considered in the S.H. 121 thin whitetopping test section construction. There were two slab thicknesses and two joint spacings for each thickness constructed, resulting in four different experimental combinations, as presented in Table 4.4. All other design parameters and material properties within project team control were kept constant.

Table 4.4. S.H. 121 Thin Whitetopping Project Primary Experimental Variables

S.H. 121 Test Section	Concrete Thickness, in.	Joint Spacing, ft
1	4	4 x 4
2	4	6 x 6
3	6	6 x 9
4	6	6 x 6

The test sections were located at the beginning of the project southbound lanes (north end of the project) from approximately station 187+00 to station 197+00. Each test section was 200 feet long, with a 200-ft-long transition zone between the 4-inch and 6-inch concrete sections. The 4-in.-thick sections were located at the northern end of the paving operation, and the 6-in.-thick sections were after the 4-in.-thick sections and the 200 ft transition zone.

In general, the pavement had 10-ft-wide outside and 4-ft-wide inside concrete shoulders. All concrete shoulders were constructed monolithically with the main pavement lanes. In addition, the entire lane was designed with a uniform cross slope across both shoulders and lanes.

The S.H. 121 rehabilitation project is representative of a typical situation when a TWT overlay could be considered. The traffic levels on this section of roadway are relatively high, but currently are limited to general vehicular and light truck traffic. The construction of a TWT overlay minimizes the amount of traffic interruption by expediting the construction and paving activities; using the existing asphalt as a base course facilitates the construction of a concrete pavement without requiring a more extensive and time consuming complete reconstruction project.

4.2 Pre-Construction Pavement Evaluations

The existing asphalt pavement condition was identified during the original project as critical to the subsequent performance of the thin whitetopping overlay. The first three test pavements were not visited by CTL prior to the thin whitetopping construction, but a pre-construction survey of the S.H. 121 existing asphalt pavement was conducted as part of this study. The evaluation was performed jointly by CTL and the Colorado DOT in April 2001 and included a visual condition survey, rutting measurements, coring and falling weight deflectometer (FWD) testing. Additional information regarding the pre-construction evaluation is presented in the project construction report [21] and Appendix B.

4.3 General Construction Approaches

The S.H. 121 thin whitetopping pavement was constructed in the summer of 2001. The general approach for the construction of the S.H. 121 test sections was in accordance with conventional slip-form paving operations where the two driving lanes and shoulders are placed monolithically. In contrast, the S.H. 119 and U.S. 287 sites were paved one lane at a time and the U.S. 85 section utilized the pre-placed concrete curb and gutter as a form on each side of the pavement. The asphalt milling, asphalt surface preparation, concrete mix design characteristics, concrete paving and control joint sawing construction procedures are further discussed in the project construction report [21] and Appendix C.

4.4 Instrumentation Installations and Load Testing

The instrumentation and testing approaches used at the S.H. 121 site are based on and consistent with the approaches utilized at the original three test sites. As discussed, the two primary variables to be evaluated in the current study were the slab thickness (two levels) and panel joint spacing (two levels for each thickness), resulting in four different combinations. Two replicate slabs were instrumented for each test section, resulting in eight total slabs. The overall purpose of the replicate installations was to obtain average data from replicate slabs to more accurately represent the responses of the slabs in the test sections. The following testing on the test sections was planned:

- Static load testing with strain measurements

- Surface profile measurements over daily temperature variations
- Joint opening measurements
- Temperature measurements
- Pavement coring and laboratory testing
- Ground penetrating radar testing for thickness estimation
- FWD tests.

In order to perform the field testing planned at the S.H. 121 site, instrumentation for each test section included the following:

- Embedded concrete strain gages
- Surface concrete strain gages
- Embedded thermocouples
- Retrofitted temperature sensors
- Reference rods
- Whitmore plugs.

A portion of the instrumentation required for this project had to be installed prior to construction of the TWT concrete overlay. This included the embedded concrete strain gages, reference rods and embedded thermocouples. Others were installed just prior to load testing activities, such as the surface strain gages and temporary temperature sensors. More detailed information regarding the instrumentation installations, including test slab locations and sensor placements, is presented in the project construction report [21] and Appendix D.

4.4.1 Slab Profile Measurements

During the field testing, surface profile measurements were collected on one test slab in each of the four S.H. 121 test sections. A dipstick provided by CDOT was used to record the relative elevations of each test slab by traversing the panel surface along the outside edge, transverse joint and diagonal. To define total slab deformations between different measurement periods, each profile measurement started and ended with the fixed reference rods installed prior to whitetopping pavement construction.

Initial baseline profile measurements were collected on each profile test slab within 24 hours of concrete paving, and subsequent profile measurements were collected at various times throughout the load testing periods. The baseline profiles were used as references for defining slab curling deformations during load testing periods and possible warping deformations that have occurred since construction. The profiles were recorded to define the slab curling and warping deformations and help confirm that the whitetopping pavement acts as partially bonded, two-layered system. The typical profile measurement location layout for each profile test slab is presented in Figure 4.1.

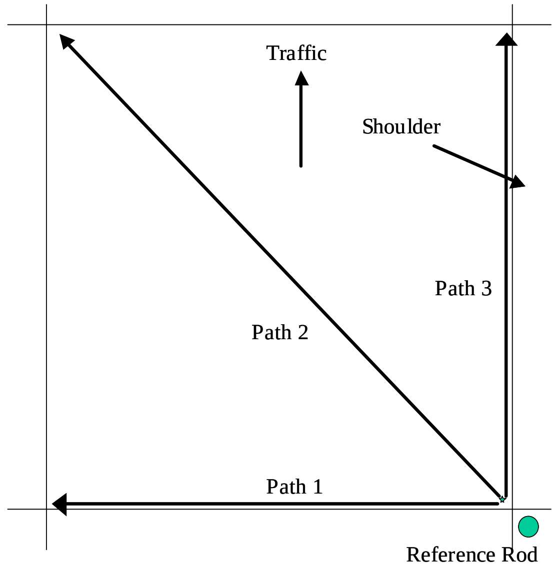

Figure 4.1. Typical Profile Measurement Locations

4.4.2 Static Load Testing

Two load testing sessions were performed on the S.H. 121 test sections; the first in July 2001, approximately 28 days following construction, and the second in July 2003, two years following construction. Two load testing sessions were performed to characterize pavement responses both before and after exposure to the effects of freeze-thaw cycling and substantial traffic repetitions.

Load testing was performed by placing the rear wheel of a loaded CDOT truck at various locations on the test slab surface based on the installed strain gage configurations. The typical wheel load placement locations are presented in Figure 4.2. Using a strain and switch box, static strain measurements were then recorded for the appropriate gages based on the wheel placements. Single axle trucks were used for the at the original test sites and the 28-day S.H. 121 load testing, so the original design guidelines and equations were based on single axle truck loadings. However, a tandem axle truck was included for the S.H. 121 two-year testing to evaluate the effect tandem axles have on the measured responses and evaluate whether the design guidelines are significantly influenced by tandem axle truck configurations. Figure 4.3 shows a rear truck wheel placed on a test slab during load testing activities.

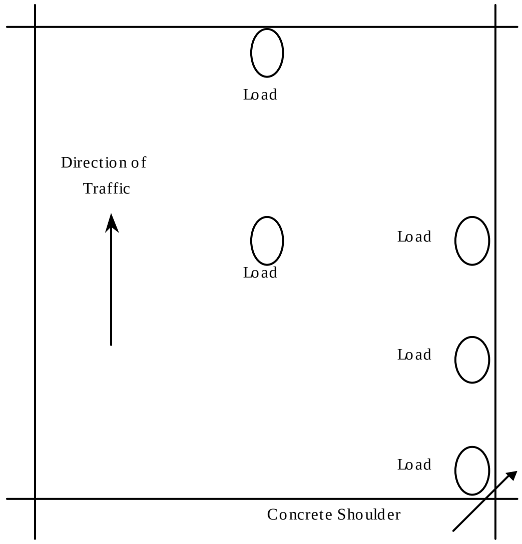

Figure 4.2. Typical Load Testing Wheel Locations

Figure 4.3. Truck Wheel Placement During Load Testing

Load testing was performed at various times throughout the test days to measure pavement responses when there were various temperature gradients through the pavement thickness.

Typically testing started at approximately 6 a.m. to capture negative nighttime temperature gradients through the pavement thickness, continued through the early morning when there would be a negligible or zero gradient, and was performed multiple times again throughout the day to capture measurements during multiple levels of positive daytime gradients.

4.4.3 Falling Weight Deflectometer Testing

Falling weight deflectometer (FWD) testing was not included in the load testing of the original test sites but was conducted on the four S.H. 121 test sections during the 28-day and two-year load testing efforts. Tests were performed on two slabs within each of the four test sections for a total of eight slabs. As with the static load testing, FWD testing was performed at various times throughout the test days to measure pavement responses when there were various temperature gradients through the pavement thickness. Typically testing started at approximately 6 a.m. to capture negative temperature gradients and continued throughout the test days to measure pavement responses when there were negligible and various positive temperature gradients through the pavement thickness. Three FWD drops were conducted at the slab center, transverse edge, longitudinal edge and corner locations.

4.5 Pavement Coring and Laboratory Testing

Numerous pavement cores were drilled as a portion of the S.H. 121 project. Twelve asphalt cores were drilled prior to construction and more than 40 cores were drilled through the thin whitetopping pavement in the two years following construction. The cores were primarily used to quantify the concrete and asphalt layer thicknesses and provide samples for performing direct shear testing on the asphalt-concrete interface in the laboratory. The Iowa 406-C test method [22] was used for performing the interface shear strength testing. The S.H. 121 core interface shear strength results are presented in Table 4.2.

Construction and testing of the original three thin whitetopping pavements included additional material property testing such as compressive strength, modulus of elasticity and flexural strength tests. Concrete cylinders and beams were cast during pavement construction for the U.S. 85 frontage road and S.H. 119 test sites, and pavement cores were drilled in conjunction with load testing. Concrete cylinders and beams were not available for testing from the U.S. 287 site, but concrete cores were drilled in conjunction with the load testing. Material property results for these test sections were presented in the 1998 report, but based on the limited amount these data were utilized for the original data analysis, similar testing efforts were not performed for the S.H. 121 site. However, the interface bond shear strength test results for the original sites are included in Table 4.2 of this report. Six- or 7-year interface shear strength results for the original three test sections are also included in Table 4.2 and are from samples drilled during revisits of those sites; revisits of the original sites were performed as a part of this study.

The S.H. 121 core interface shear strength results presented in Table 4.2 are relatively high compared to the original section strengths for 28 days following construction, but similar for later tests. The only concern regarding the interface bond strength results is the number 2003 cores drilled (two-year interface shear strength tests for the S.H. 121 site and 6/7 year strength tests for the original test site revisits) that were not extracted from the pavement intact. For example, twenty-eight days after S.H. 121 construction in 2001, 22 of 24 whitetopping cores drilled were removed from the pavement with the interface bond intact and suitable for testing. However, only 5 of the 18 shear test cores drilled in 2003 at the S.H. 121 sections were removed with the interface bond intact. In addition, only 11 of 30 cores at the three original sites during the 2003 revisits were removed with the interface bond intact. This may simply be a result of the coring operation, but based on the number of cores that were removed without the bond intact, it does raise concerns about the long-term interface bond performance. This is a whitetopping test section characteristic that should be monitored on a continued and long-term basis at all test pavement locations.

4.6 Revisiting Original Test Pavements

The three original whitetopping test sections were revisited during June 2003 to observe overall pavement performance after nearly seven years of service. The general tasks performed included crack mapping, core sampling, faulting measurements, joint width measurements, photographs and FWD testing. The site visit condition survey observations are discussed for each test section in the following subsections, and testing performed at the sites in 2003 is discussed in the final subsection.

4.6.1 United States Route 85 Frontage Road, Santa Fe Drive, Denver, Colorado

The overall condition of instrumented test slab areas at this pavement section was very good. Isolated longitudinal cracks were observed, but most appeared to be related to possible settlement or shifting of the adjacent concrete curb and gutter due to loss of support; most of these cracks were not located in the wheelpaths. In addition, a few corner cracks and transverse shrinkage cracks appeared to be located over longitudinal joint tie bars. However, on the south end of the test section there were a significant number of shattered slabs in the wheelpaths at a stop sign approach. The distressed area was in the approximately 80 feet where braking occurs leading up to the intersection (this distressed location coincides with the start of paving where the contractor was reportedly altering the concrete mixture to obtain a workable mix for paving operations). Additional investigation of the cause of this distress is recommended. Figures 4.4 and 4.5 present photographs of typical conditions observed at the site and the distressed area near the south stop sign approach, respectively. A summary of the distress observed at the sites is presented in Table 4.5.

Figure 4.4. Typical pavement Conditions Observed at the U.S. 85 Santa Fe Drive site

Figure 4.5. Stop Sign Approach Distressed Area Observed at the U.S. 85 Site

4.6.2 Colorado State Highway 119, Longmont, Colorado

The overall condition of instrumented test slab areas at this pavement section was also very good. Four of the five test sections exhibited minimal distress, but one section exhibited a considerable amount of longitudinal mid-panel cracking. This was Section No. 2, which had 10 by 12 ft panels and was the first section paved on the west end of the whitetopping site. One hundred seven of the 131 panels surveyed in this test section contained cracking. Many of the more severe, full panel length cracks had been filled with asphalt sealant, but there were also shorter, narrower cracks that did not extend the entire slab length and were not filled. No cracks were observed in Section No. 3 with 6 by 6 ft panels, No. 5 with 12 by 12 ft panels, or No. 1 with 6 by 6 ft panels. Only two longitudinal mid-panel cracks were observed in Section No. 4 with 6 by 12 ft panels.

Other than the longitudinal panel cracks in Section No. 2, the most frequent distress observed was minor joint spalling at various locations along the test sections, and a significant percentage of this spalling near the edge of the driving lane appeared to be from snow plow abrasion. The overall ride quality of these test sections was qualitatively observed as excellent. Figures 4.6 and 4.7 present photographs of typical conditions observed at the site and the slab cracking in Test Section No. 2, respectively. A summary of the distress observed at the sites is presented in Table 4.5.

Table 4.5. Summary of Distress Observed at the Original Whitetopping Test Sections

SITE	SECTION	NUMBER OF SLABS SURVEYED	NUMBER OF CRACKED SLABS	QUANTITY OF CRACKS						QUANTITY OF JOINT SPALLS		
				Transverse Mid-Panel	Corner	Longitudial Wheelpath	Longitudinal Mid-Panel	Other	Comments	Light	Moderate or Severe	Other/Comments
U.S. 85 Frontage Road	Strain Test Slab No. 1	45	8	****	4	****	****	2	Narrow trans. cracks over tie bars/across long. jt.	****	****	****
	Strain Test Slab No. 2	45	0	****	****	****	****	****	****	1	****	****
	Strain Test Slab No. 3	45	0	****	****	****	****	****	****	5	****	****
	Remainder of Pavement Section	369	94	2	35	2	12	26 39 2	Shattered slabs at stop sign approach Narrow trans. cracks over tie bars/across long. jt. Edge/corner settlement cracks	4	2	Some moderate spalling of shattered slab cracks & joints.
U.S. 287	Test Section B	64	5	****	****	2	****	5	Narrow trans. cracks over tie bars/across long. jt.	37	****	Light spalling observed in all three sections possibly due to debris in joints.
	Test Section F	200	33	2	2	20	4	9 5 4 11	Slabs removed and replaced Diagonal miscellaneous cracks Transverse miscellaneous cracks Concrete shoulder longitudinal cracks	10	****	
	Test Section E	200	8	1	****	4	****	3 4 2	Slabs removed and replaced Diagonal miscellaneous cracks Transverse miscellaneous cracks	12	****	
S.H. 119	Test Section No. 1	200	0	****	****	****	****	****	Diamond ground areas (from initial construction)	43	****	Most joint spalls or asphalt sealant repaired joint locations in all sections appear to be abrasive failures due to snow plow blade abrasion.
	Test Section No. 2	131	107	****	2	3	94	2	Miscellaneous cracks Over 50 percent of cracking/crack length is sealed	34	****	
	Test Section No. 3	286	0	****	****	****	****	****	****	15	****	
	Test Section No. 4	229	4	****	3	****	2	****	****	28	****	
	Test Section No. 5	50	0	****	****	****	****	****	Diamond ground areas (from initial construction)	53	****	

Figure 4.6. Typical Pavement Conditions Observed at the S.H. 119 Longmont Site

Figure 4.7. Typical Slab Cracking Observed in S.H. 119 Test Section No. 2

4.6.3 United States Route 287, Lamar, Colorado

The overall condition of instrumented test slab areas at this pavement section was also very good. The most frequent distress observed was minor transverse joint spalling at various locations along the test sections. Isolated longitudinal cracking was also observed in one of the 6 by 6 ft slab test sections, with approximately 33 of the 200 slabs surveyed cracked. Many of the cracks observed in this section appeared to approximately be located in the outside wheelpath of the southbound test lane. Most of the slab cracks observed had been sealed, but 9 slabs within the section had been removed and replaced. The other 6 by 6 ft slab test section only had eight cracked slabs in the approximately 200 slabs surveyed, and two slabs had been replaced. The 10 by 12 ft slab test section had five slabs with small cracks in the 64 slabs observed, and no slabs had been replaced. The overall ride quality of these test sections was qualitatively observed as excellent.

The slab removal and replacement repairs observed were reportedly performed shortly after initial construction when the northbound lane experienced a significant degree of cracking. This was attributed to placing the whitetopping on a hot asphalt surface accelerating the drying on the bottom of the concrete, which initiated shrinkage cracking. An attempt was made to keep the asphalt surface cool by spraying water during construction of the southbound driving lane and shoulder. Figures 4.8 and 4.9 present photographs of typical conditions observed at the site. A summary of the distress observed at the sites is presented in Table 4.5.

Figure 4.8. Typical Pavement Conditions Observed at the U.S. 287 Lamar Site

Figure 4.9. Typical Cracked Slab Conditions Observed at the U.S. 287 Lamar Site

4.6.4 Testing and Core Sampling

In addition to the general condition surveys of the original Colorado whitetopping test sections, core sampling, falling weight deflectometer testing, transverse joint faulting measurements and joint width measurements were performed during the 2003 site revisits.

Core sampling was performed to test the interface bond for long-term shear strength. The test results were presented earlier in Table 4.2. As has already been discussed, the only concern regarding the interface bond strength test results is the number 2003 cores drilled (two-year interface shear strength tests for the S.H. 121 site and 6/7 year strength tests for the original test site revisits) that were not extracted from the pavement intact. Only 11 of 30 cores at the three original sites during the 2003 revisits were removed with the interface bond intact. This may simply be a result of the coring operation, but based on the number of cores that were removed without the bond intact, it does raise concerns about the long-term interface bond performance. This is a whitetopping test section characteristic that should be monitored on a continued and long-term basis at all test pavement locations.

The transverse faulting measurements were performed at the sites with a faultmeter provided by CDOT. The survey consisted of approximately 30 transverse joint outside wheelpath faulting measurements at each test section. The results indicate that there is only minimal faulting in any of the test sections regardless of whether dowel bars were included in the transverse joints. Typical faulting measurements were less than 0.02 in.

Limited numbers of transverse and longitudinal joint width measurements were also collected during the test site revisits. The joint width measurements were collected to investigate whether the transverse and longitudinal pavement contraction control joints are cracked and working. Typically the measured joint widths were approximately 3/16-in.-wide, indicating that most joints were cracked and working based on a 1/8 in. initial sawcut width. The majority of joints being cracked also supports that using relatively short joint spacings appears to be a valid approach to control random cracking and minimize the effects of curling and warping stresses.

The FWD testing performed by CDOT at the sites was done to provide general pavement load capacity pavement comparisons between the test sections. Deflection testing was performed at the U.S. 85 frontage road and S.H. 119 sites, but it was not possible to perform FWD testing at the U.S. 287 site during 2003. The primary reason for performing the FWD testing was to establish some historical data regarding the current structural condition of the test sections; these data could be particularly useful if future failures are observed in the sections. Examples of the U.S 85 and S.H. 119 FWD test results are presented in Figure 4.10 and 4.11, respectively.

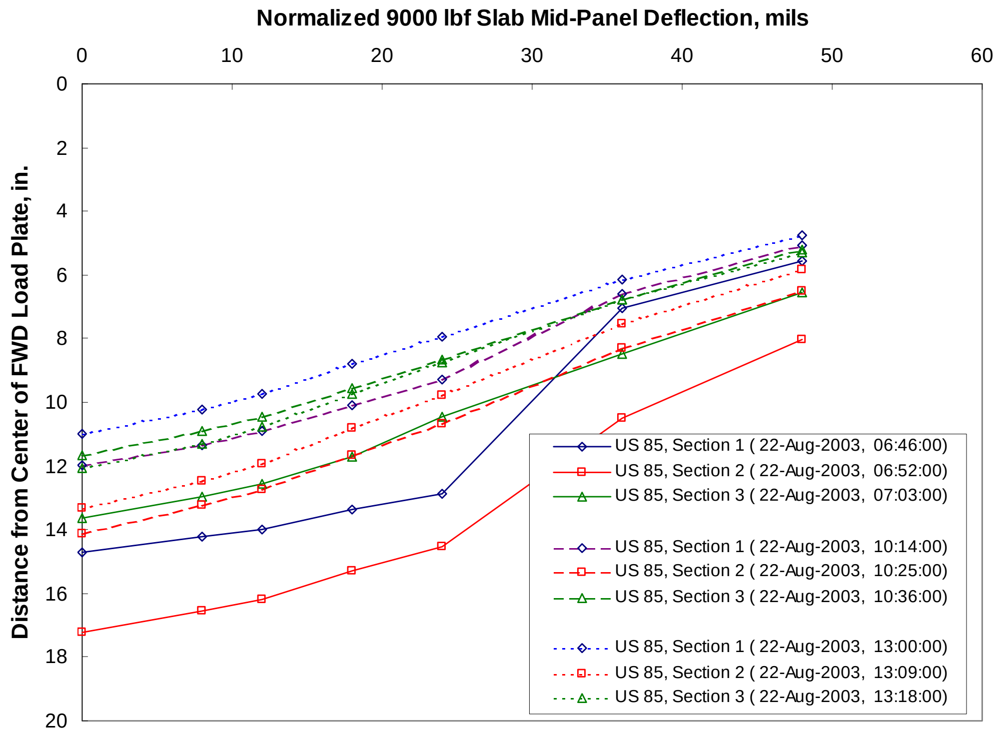

Figure 4.10. FWD Mid-Panel Deflections for U.S. 85 Frontage Road Site

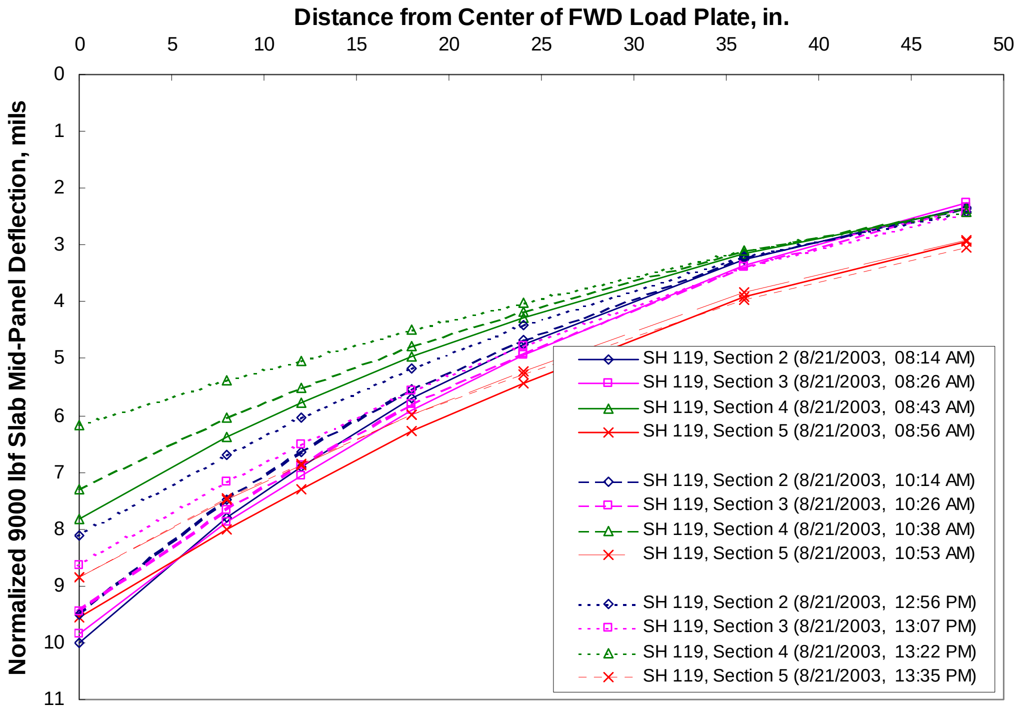

Figure 4.11. FWD Mid-Panel Deflections for S.H. 119 Site

Long-term continued monitoring is most important item related to the test sections. The long-term performance of the test sections is critical to the eventual verification of the design procedure developed during this study. The design procedure has been verified and revised as much as is currently possible, but the long-term performance of the test sections will ultimately verify whether the design procedure is valid.

5.0 MECHANISTIC WHITETOPPING THICKNESS DESIGN PROCEDURE

Guidelines for bonded whitetopping were established during the original 1998 study based on field calibrated flexural stresses and strains. This section includes the details of the steps followed during the original development and subsequent verification and revision of the design guidelines. Equations predicting the critical stresses and strains are provided. The rationale for incorporating stress correction factors, typical correction factors developed during this study, and recommendations for modifying the factors are also discussed. A detailed design example is also presented with the steps described and discussed.

The development and verification process included the following elements:

1. The critical load location for the design of whitetopping pavement was determined and verified by comparing the stress data collected for each load position.

2. Critical load-induced stresses were determined when there was approximately a zero temperature gradient.

3. An analysis between experimental and theoretical concrete stresses was made (no temperature gradient). The calibration factor originally developed to adjust theoretical fully bonded stresses to measured partially bonded concrete stresses was revised.

4. An adjustment factor originally developed to convert theoretical fully bonded maximum asphalt flexural strains to partially bonded strains was revised.

5. To account for loss of support with temperature curling effects, an equation was originally derived and presently reviewed that incorporates the percent change in stress (from zero temperature gradient) based on the unit temperature gradient (°F/in.).

6. The calculation of design concrete flexural stress and asphalt strain for a specific set of design parameters involves the following steps:

   - Maximum load-induced concrete stresses and asphalt strains were computed for fully bonded whitetopping pavements using the finite element program ILSL2 [23]. A wide range of pavement parameters and material properties were originally covered, but additional ILSL2 analyses were performed and incorporated into the current study.

   - Stepwise least squares linear regression techniques were used to derive the original equations to predict concrete stresses and asphalt strains from different pavement parameters and material properties. These equations have been re-derived and the new equations presented based on the current study

   - Theoretical load-induced concrete stresses are increased to account for the partially bonded condition (step 3 above).

   - Theoretical load-induced asphalt strains are decreased to account for the partially bonded condition (step 4 above).

   - The increased load-induced concrete stresses are adjusted to account for loss of support with temperature curling effects (step 5 above).

7. Whitetopping concrete thicknesses are established by limiting both the concrete flexural stresses and asphalt flexural strains within safe limits under anticipated traffic and environmental conditions during the pavement's design life. The procedure uses fatigue concepts to evaluate the concrete and asphalt layers separately. Therefore, for a given set of pavement parameters and material properties, the concrete or the asphalt layer may govern the design.

5.1 The Effect of Interface Preparation on Shear Strength and Load Induced Strains

The effect of interface preparation on load induced pavement response was studied at two of the three original test projects evaluated. However, the existing asphalt surface preparation for all S.H. 121 test sections was identical, so interface preparation was not investigated extensively during this study. The only concern regarding interface bond and surface preparation is related to the results of the interface shear strength coring that has already been discussed. Based on the results of the current study, it is recommended that continued monitoring of interface shear strengths be performed. The original study results recommend cold milling existing asphalt and not milling newly placed asphalt prior to concrete paving, and the following paragraphs briefly summarize the findings from the original study.

The varied asphalt preparations used at the three original test sites offered a good opportunity to evaluate the effect of asphalt surface preparation on load induced strains and interface shear strengths. The U.S. 85 frontage road project was constructed with new asphalt, and in one of the test sections the new asphalt concrete was milled prior to whitetopping construction. Two S.H. 119 test sections were constructed over new asphalt, two were constructed over existing milled asphalt, and one was constructed over existing asphalt with no surface preparation. The test sections at the U.S. 287 site were all constructed over milled existing asphalt.

Cores were removed from all original test sections for interface shear strength testing, and the average test results are presented in Table 4.2. For each of the test slabs, regardless of interface condition, the shear strength increased between approximately 28 days and 1 year. For newly placed asphalt, the interface shear strength increased by an average of 80 and 590 percent for unmilled and milled surfaces, respectively. The higher percentage for milled surfaces is somewhat misleading, however, because the 28-day shear strength was the lowest measured at about 10 psi. Existing milled asphalt shear strength increased by approximately 54 percent over the first year of service. Unfortunately, due to the necessity to close multiple lanes, the existing unmilled asphalt was unable to be tested beyond the 28-day tests.

A comparison of the load-induced strains for milled relative to unmilled interfaces revealed a significant difference between new and existing asphalt pavements rehabilitated with whitetopping concrete. As shown in Figure 5.1, load induced strains for newly placed asphalt increased by an average of about 50 percent if the interface was milled. On the contrary, for existing asphalt pavements, the load induced strains decreased by approximately 25 percent when interface milling was performed. The data shown in the figure includes all strains collected from gages placed at multiple depths and locations (edge, center, corner) of the test slabs. Therefore, some of the strains are positive (tensile) and some are negative (compressive).

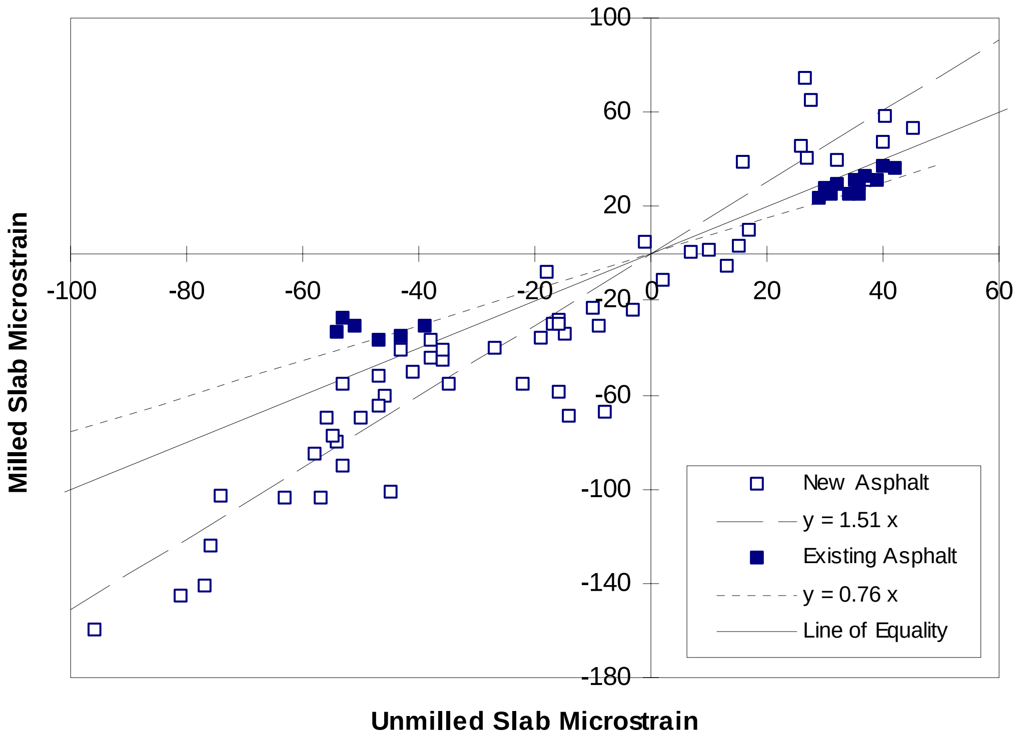

Figure 5.1. Effect of Interface Milling on Load Induced Strain [(1)]

5.2 Determination of Critical Load Location

The critical load location for the design of whitetopping pavement was determined during the original study by comparing the stress and strain data collected for each load position. The critical load location inducing the highest tensile stress in the concrete layer was when the load was centered along a longitudinal free edge joint. For whitetopping pavement, a free edge joint occurs when both the asphalt and concrete are formed against a smooth vertical surface such as a formed concrete curb and gutter. It is reasonable that free edge loading produces the highest stress, but it is likely more common that the joints loaded by traffic will not be free edges. Therefore, for the design procedure, tied longitudinal joint loading was originally considered the critical load case, as shown in Figure 5.2, and this was verified during the current study as the data in Figure 5.3 shows. Although the edge versus center strain data presented in Figure 5.3 is very similar, the edge data is for a wheel load placed 3 in. from the slab edge. When the wheel load is placed directly at the slab edge, the expected strains would be larger and the critical load location more exaggerated than this data suggests.

A relationship between free edge and tied edge stresses was originally developed for use in designs where free edge loading is likely (narrow truck entrances where slabs are not tied into concrete curb and gutter). Since the S.H. 121 test sections did not include free edges, this equation could not be further verified, but the equation for original data shown in Figure 5.4 is as follows [(1)]:

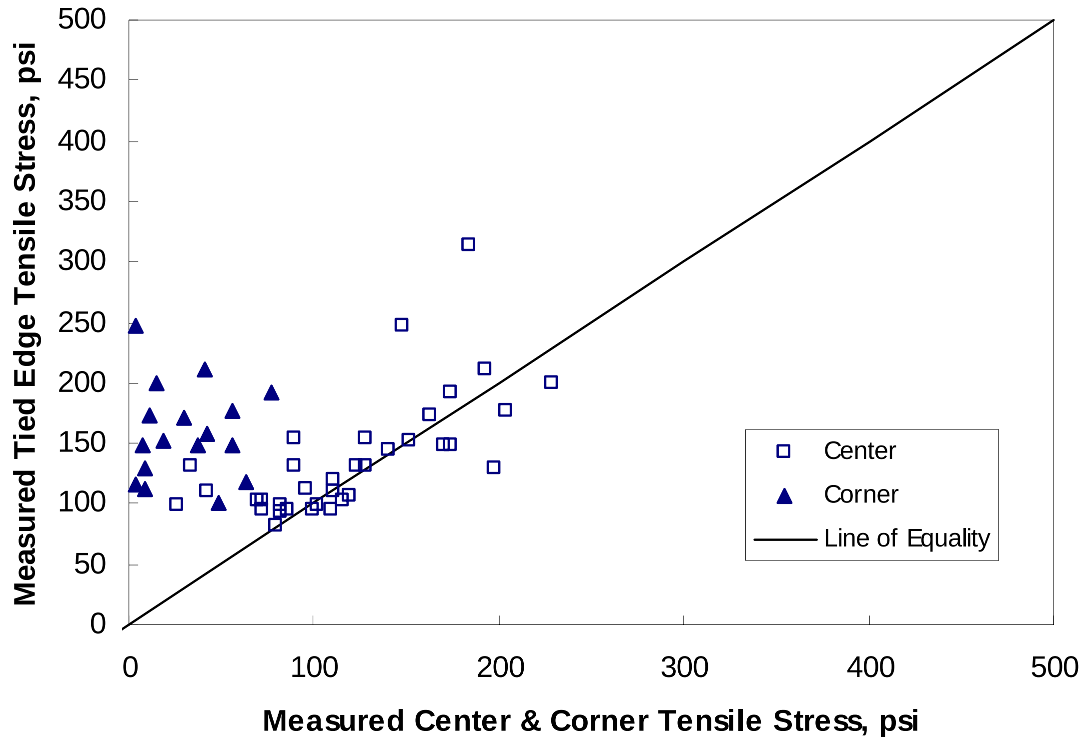

Figure 5.2. Location of Load Resulting in Maximum Stress

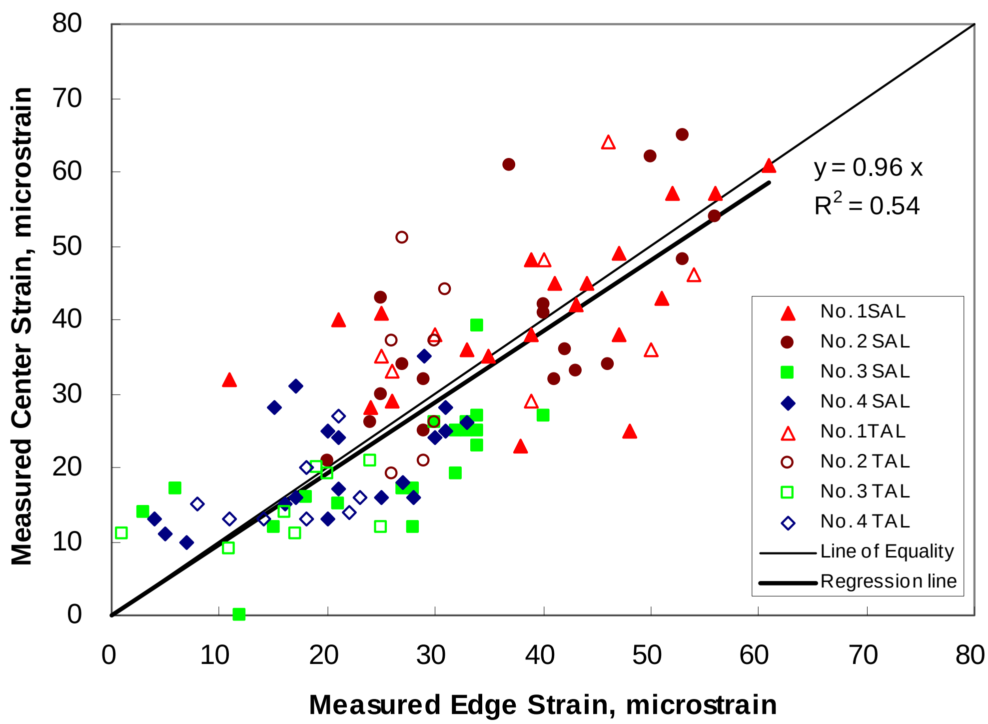

Figure 5.3. Measured Center Versus Edge Strains

$$\sigma_{FE} = 1.87 \times \sigma_{TE} \qquad \text{(Eq. 5.1)}$$

where,

σ_{FE} = load-induced stress at a longitudinal free joint, psi
σ_{TE} = load-induced stress at a longitudinal tied joint, psi

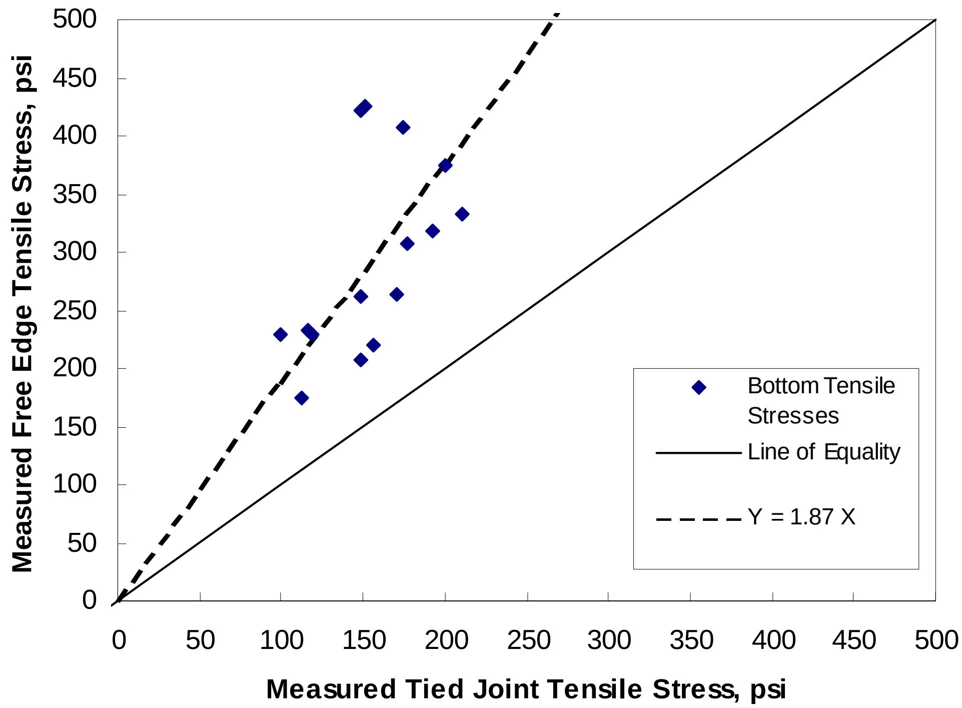

Figure 5.4. Conversion From Tied Joint to Free Edge Stress

5.3 Determination of Load-Induced Stress at Zero Temperature Gradient

Each test site included in this study had multiple slabs instrumented for load testing. A variety of material parameters, joint configurations, and interface preparation treatments were studied. Each slab instrumented was load tested multiple times during the testing days. Load testing was scheduled for relatively hot summer days where the temperature gradient through the concrete would be significant. The first loading of each slab was performed shortly after sunrise when the temperature gradient was still negative (surface cooler than slab bottom). Several additional load tests were performed throughout the day to evaluate the effects of various temperature gradient conditions on load-induced stresses. Load-induced stresses were plotted against the measured temperature differentials throughout the day to establish stress corresponding to a temperature gradient of zero. Zero gradient stresses were compared with theoretical stresses. This comparison allowed for a partial bond calibration factor to be applied to fully bonded theoretical stresses.

5.4 Analysis of the Effect of Interface Bond on Load-Induced Concrete Stress

The effect of interface bonding was evaluated by comparing measured stresses for zero temperature gradient conditions to the computed stresses for fully bonded pavement systems. Stresses caused by loads at mid-joint and slab corner were computed using the finite element computer program ILLISLAB (ILSL2) [23], assuming fully bonded concrete-asphalt interface. ILLISLAB was developed in 1977 for the Federal Aviation Administration (FAA) for structural analyses of concrete pavement systems. The program is based on plate bending theory for a medium-thick plate placed on a Winkler or spring foundation [24]. It is capable of computing stresses and deflections for panels with doweled, keyed, or aggregate interlock load transfer at the joints. However, it is not capable of modeling the partially bonded interface between whitetopping pavement layers.

Measured tied edge loading partial bond stresses were plotted as a function of theoretical fully bonded edge stresses in Figure 5.5. Typically, the measured stresses are greater than theoretical stresses. The slope of the original least squares linear regression line was 1.54 which represents a 54 percent increase in the stress due to the partial bond condition. However, once the S.H. 121 test site data is included, the line slope is reduced to 1.35, representing a 35 percent increase in stresses due to the partial bonding condition. This reduction in the partial bonding concrete stress factor is consistent with the asphalt surface preparation performed at the S.H. 121 site. All the S.H. 121 test sections were existing asphalt milled prior to concrete placement, and based on the previous study this is the best approach for promoting bond for existing asphalt substrate conditions.

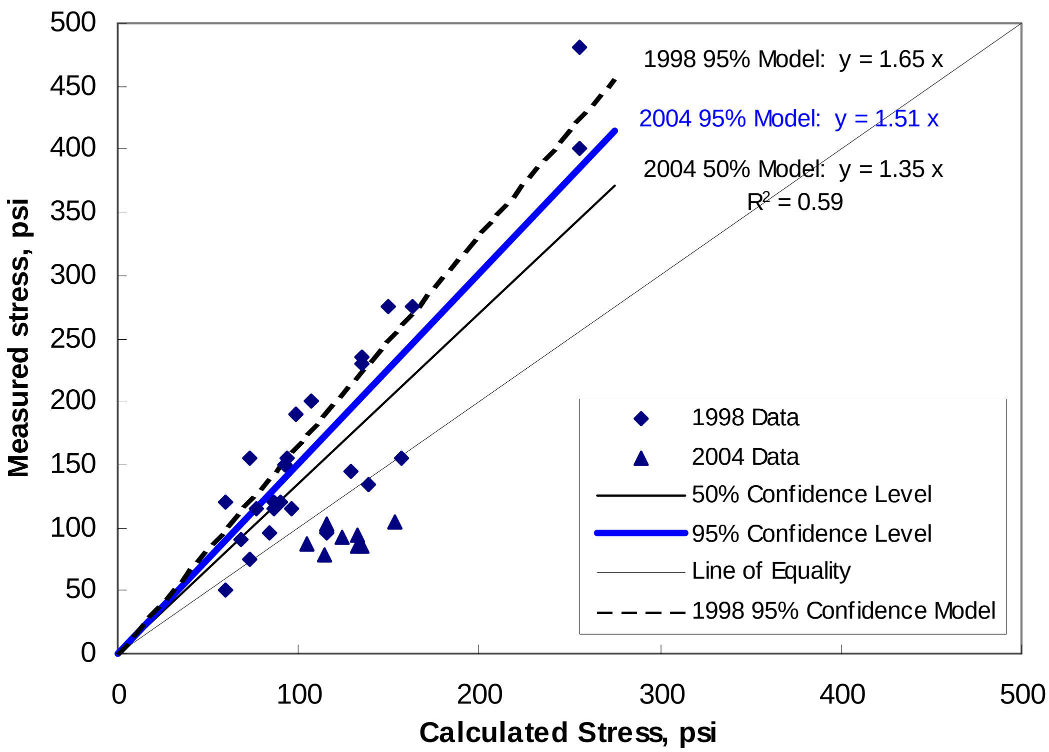

Figure 5.5. Measured Versus Calculated Bottom Concrete Stress

Due to the measured data variability, the calculated standard deviation of the coefficient was incorporated to establish the 95 percent confidence level for the interface bond stress increase model. These lines, also plotted on Figure 5.5, represent a 51 percent increase in the bottom edge fully bonded tensile stress calculated for the current study rather than the 65 percent increase determined in the original study. The original and revised equations are as follows:

1998 Original Model [1]:

$$\sigma_{ex} = 1.65 \times \sigma_{th} \qquad \text{(Eq. 5.2)}$$

2004 Adjusted Model:

$$\sigma_{ex} = 1.51 \times \sigma_{th} \qquad \text{(Eq. 5.3)}$$

where,

σ_{ex} = measured experimental partially bonded stress, psi
σ_{th} = calculated fully bonded stress, psi

The 2004 coefficient can be reduced to 1.48 or 1.44 for confidence levels of 90 or 75%, respectively. Depending on the design, the engineer may opt to select a lower confidence. For example, for a high volume roadway, the engineer would likely select a higher confidence level than for a low-volume residential pavement.

5.5 Analysis of the Effect of Interface Bond on Load-Induced Asphalt Strain

The effect of interface bond on the load-induced asphalt surface strain was also studied using field-collected data. Prior to construction, the surface of the asphalt was instrumented with strain gages placed at locations corresponding to concrete joint edges and centers. Concrete embedment gages were also installed between 1/2 and 1 in. above the asphalt gages prior to concrete placement. Finally, concrete surface gages were installed at these locations just prior to load testing. Gages at the interface were used to evaluate the transfer of strain from the concrete bottom to the asphalt surface. The strain at the bottom of the concrete was calculated extrapolating the concrete surface strain and the embedded strain gage measurement. If slabs were fully bonded, the concrete bottom strain would equal the asphalt surface strain. Figure 5.6 shows a comparison of asphalt and concrete strains for the tied edge loading case. Asphalt strains are generally less than the concrete strains, which is the result of slippage between the layers. The equations representing the loss of strain are as follows:

1998 Original Model [1]:

$$\varepsilon_{ac} = 0.842 \times \varepsilon_{pcc} \qquad \text{(Eq. 5.4)}$$

2004 Adjusted Model:

$$\varepsilon_{ac} = 0.897 \times \varepsilon_{pcc} - 0.776 \qquad \text{(Eq. 5.5)}$$

where,

ε_{ac} = measured asphalt surface strain, microstrain
ε_{pcc} = measured concrete bottom strain, microstrain

Due to the measured data variability, the 95 percent confidence level regression model was also selected for representing the asphalt strain decrease due to partial interface bonding. Based on this model, there is approximately a 10 percent loss of strain transfer from the concrete to the asphalt due to the partial bond between the layers, decreased from the original 15 percent determined in the original study. Again, this reduction in the strain slippage is consistent with the existing asphalt surface milling performed at the S.H. 121 site. Stresses and strains at the bottom of the asphalt layer decrease with loss of bond. The design procedure assumes that average strain reductions reflecting partial bond at the interface are equally reflected at the bottom of the asphalt layer.

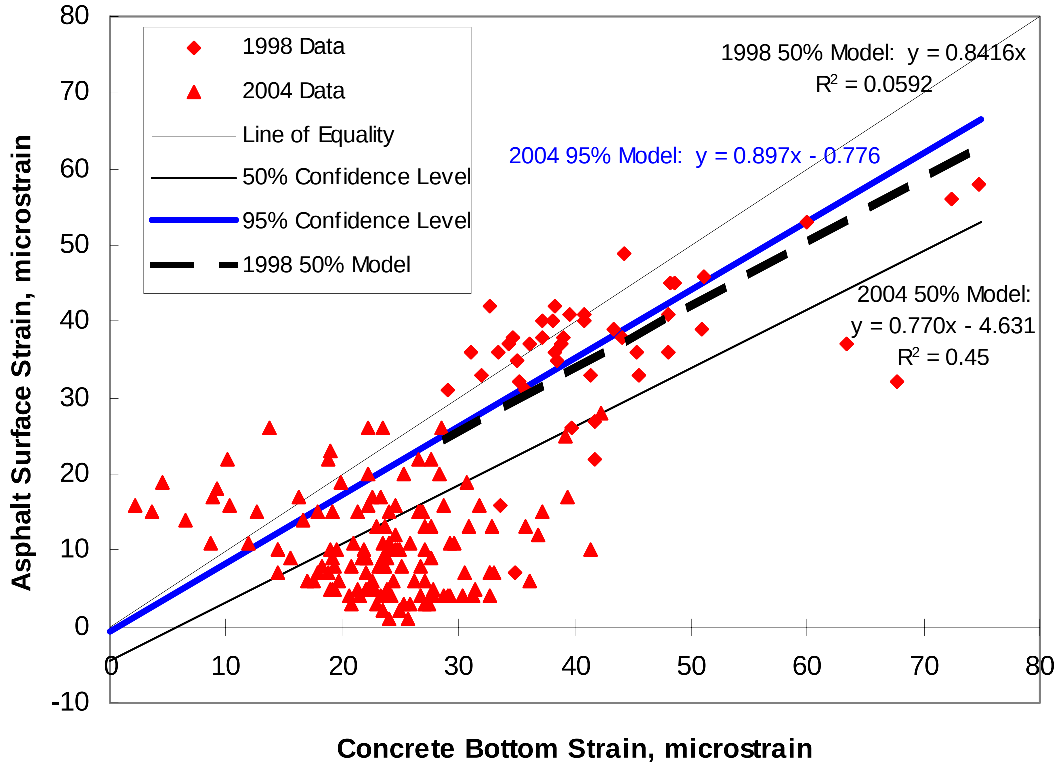

Figure 5.6. Asphalt Surface Strain Versus Concrete Bottom Strain

5.6 Analysis of Temperature Effects on Load-Induced Stresses

Load testing was repeated throughout the test days to monitor the effects of changing temperature gradients on the load induced stresses. If the temperature gradients were not significant enough to produce curling and subsequent loss of support at slab edges, measured load-induced stresses would not significantly change during the day. Temperature gradients throughout load testing ranged from -2 to 6 °F/in. Measurable stress changes occurred with changing temperature gradient, which indicates that restraint stresses are present and raises concern that there could be loss of support conditions. Falling weight deflectometer testing and slab profile measurements were performed to assist in evaluating any issues with loss of support, and those results will be discussed in subsequent sections of this report. However, minimizing the whitetopping joint spacings is recommended (typically using 6 ft by 6 ft panels) for minimizing the effects of curling and warping restraint stresses and possible loss of support.

Once the theoretical load-induced stresses are adjusted for the partial bonding condition, the effect of the temperature-induced curling are applied. Figure 5.7 shows the percent change in measured stress over the range of gradients tested. The relationships derived between the change in stress and measured temperature gradient is as follows:

1998 Original Model [1]:

$$\sigma_{\%} = 4.56 \text{ x } \Delta_T \qquad \text{(Eq. 5.6)}$$

2004 Adjusted Model:

$$\sigma_{\%} = 3.85 \text{ x } \Delta_T \qquad \text{(Eq. 5.7)}$$

where,

$\sigma_{\%}$ = percent change in stress from zero gradient
Δ_T = temperature gradient, °F/in.

This relationship is applied to the partial bond stresses to account for the effect of temperature-induced slab curling and loss of support effects on the load-induced concrete stresses.

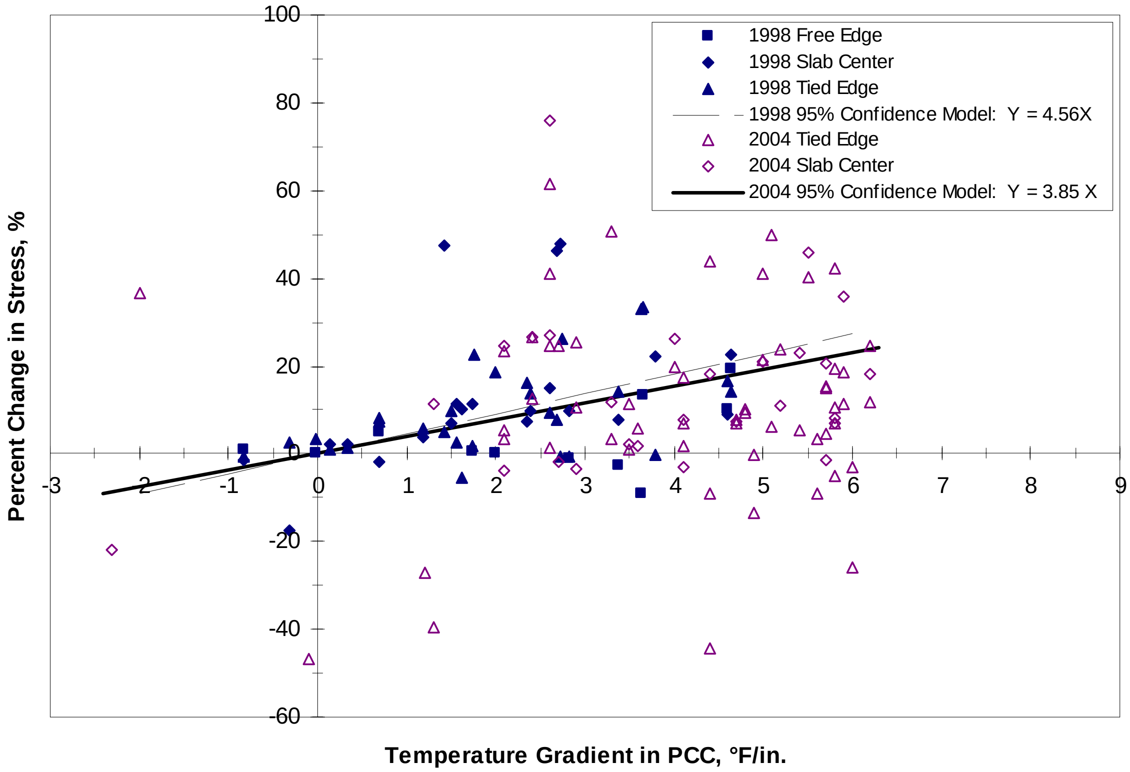

Figure 5.7. Increase in Load Stress Due to Curling Loss of Support

Falling weight deflectometer testing and slab profile measurements were performed and may assist in evaluating any issues with loss of support. Figures 5.8 and 5.9 present FWD data used to assist in the verification of temperature effects. As expected and presented in Figure 5.8, the mid-panel FWD deformations measured are variable based on the time of day the pavement is tested. The relatively small differences in deflection magnitude differences between the mid-panel and slab edge throughout the day as presented in Figure 5.9, suggests that the variations in deflection due to curling are much less significant than the overall differences that result from uniform temperature variations. The magnitudes of the deflection basin variations suggest that the differences are likely more a result of the changing asphalt temperature and corresponding deformation characteristics than slab curling deformations.

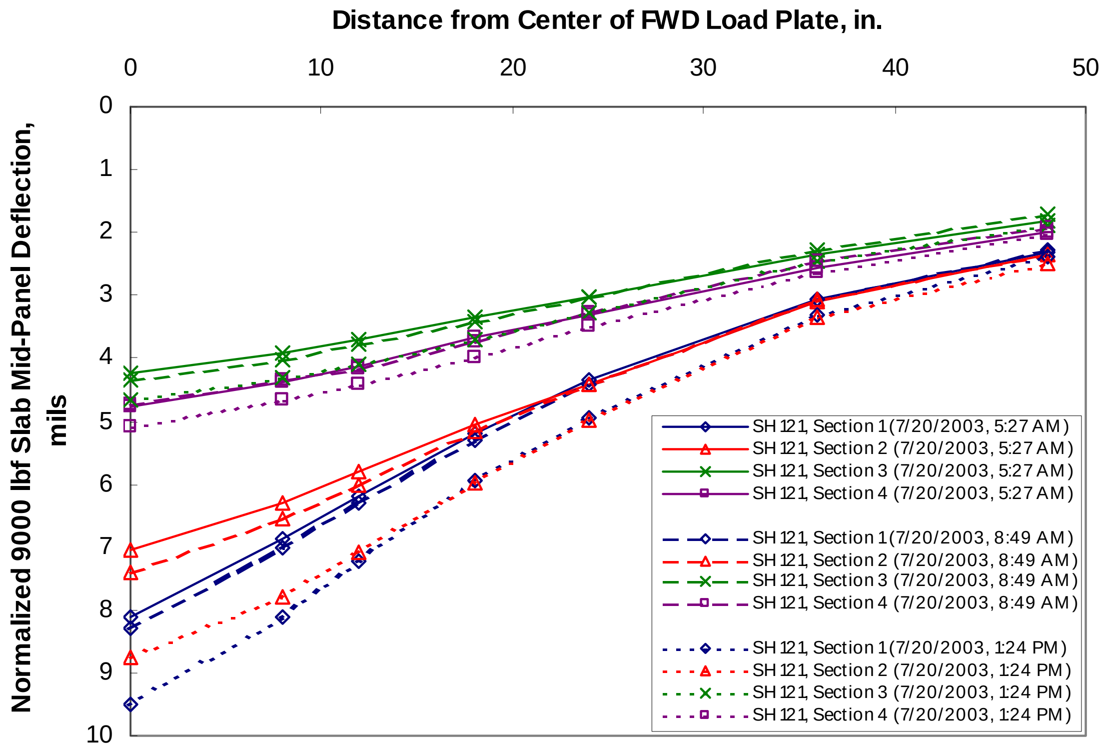

Figure 5.8. Typical FWD Measured Deflection Basins for S.H. 121 Test Sections

Figures 5.10 and 5.11 present example measured slab diagonal profile deformations for the two S.H. 121 test sections with 6 ft by 6 ft joint spacings. The profiles presented are considerably different between 28-day and two-year test periods, but that effects is likely due to concrete surface changes due to service conditions and the limitations associated with re-establishing the exact measurement traverse locations.

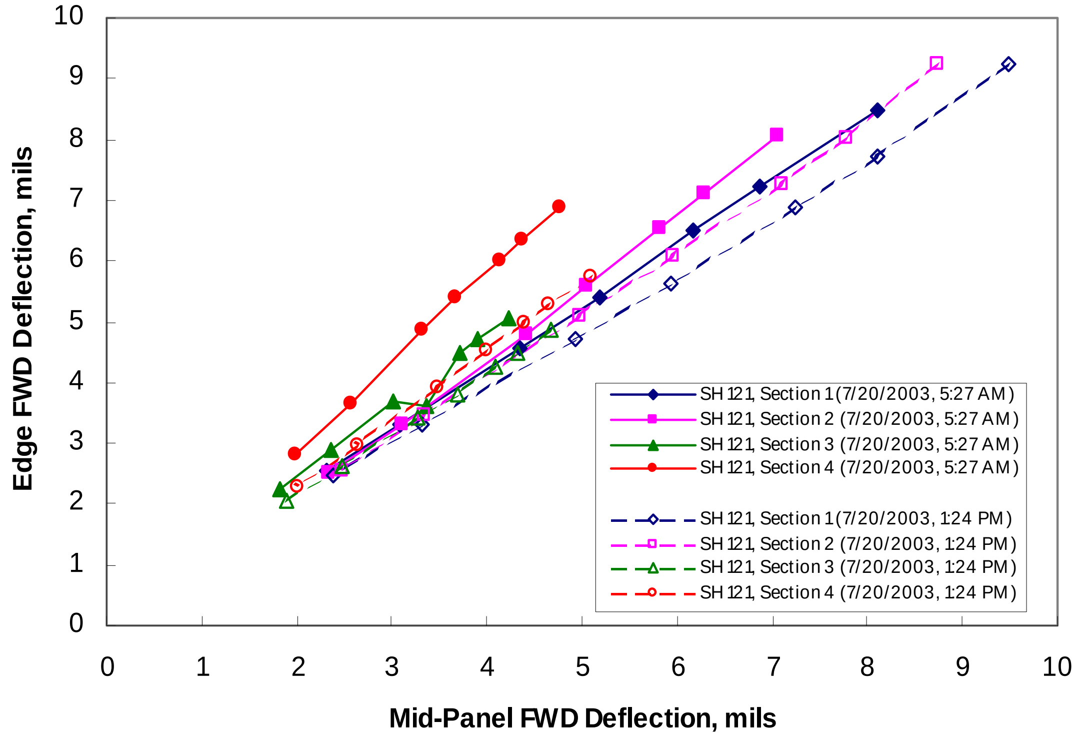

Figure 5.9. Comparison of Typical Mid-Panel and Edge Deflection Basins

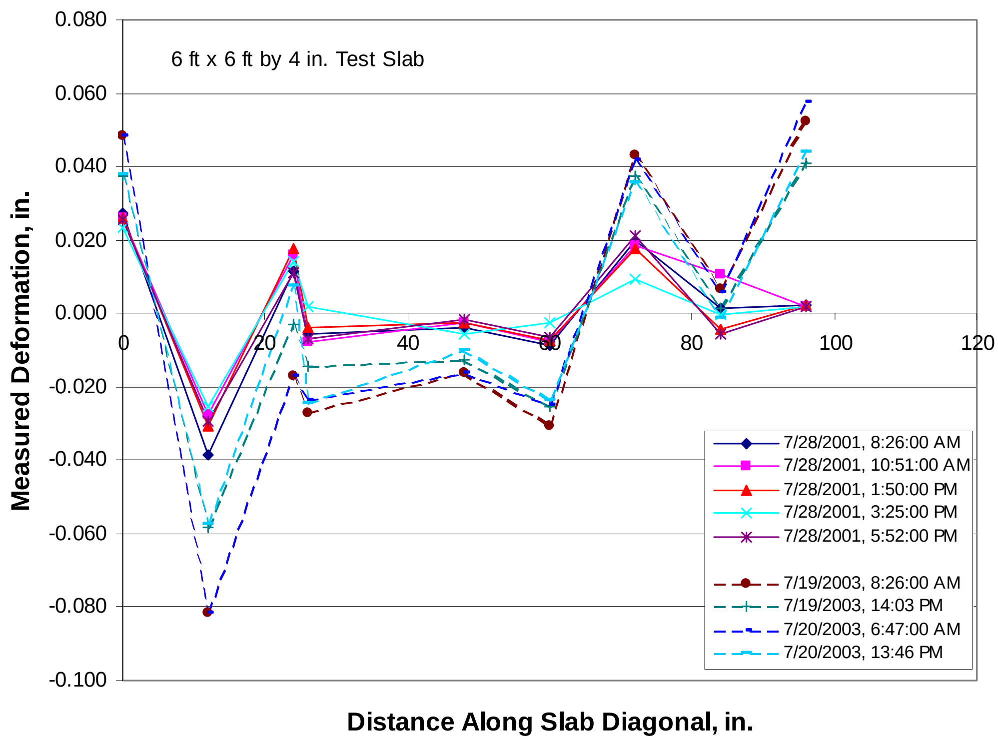

Figure 5.10. Diagonal Profile Measurements for S.H. 121 Test Section 2

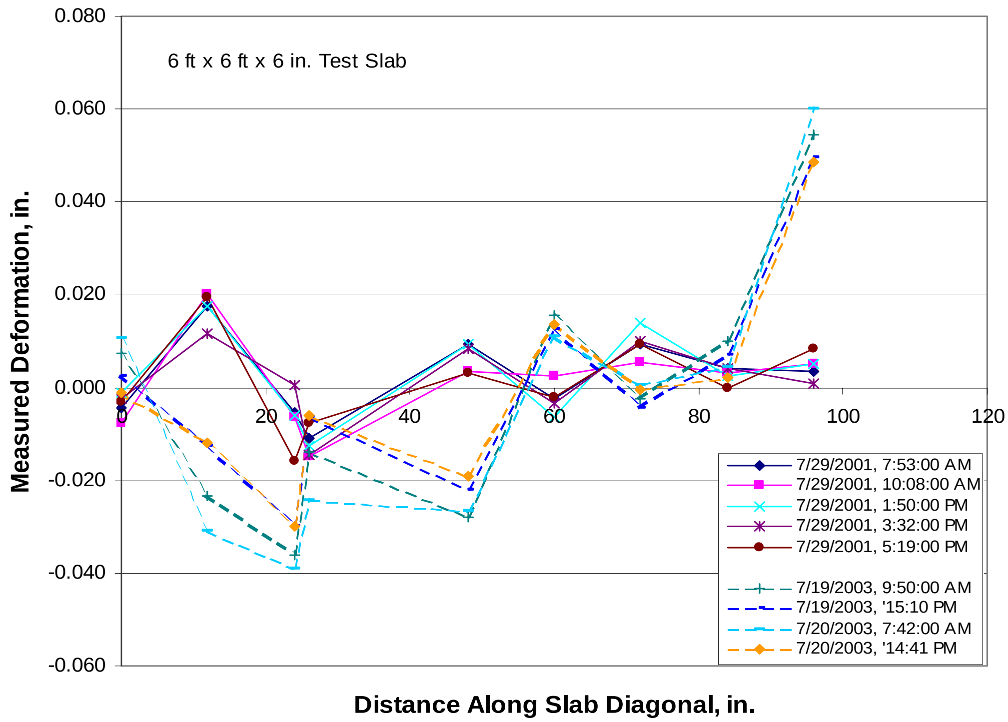

Figure 5.11. Diagonal Profile Measurements for S.H. 121 Test Section 4

The profile measurements indicate that there are minimal test slab curling deformations occurring throughout the test day despite the changing gradient conditions, which supports that there are curling restraint stresses present. However, based on comparing the 28-day and two-year measurements, it also appears that there may be a long-term deformation potentially related to slab warping effects. The two-year test period measurements still exhibit very little deformation change throughout the test day, which suggests that there is still partial bonding maintained and not a lack of support issue present based on the data from either testing period. The additional warping deformation apparently present during the two-year measurements could have been accommodated by permanent asphalt deformations, especially during higher temperature periods. This would maintain the partial bond between layers and account for the minimal deformations measured for each testing period. Based on the FWD and profile measurement data, minimizing the whitetopping joint spacings is still recommended for minimizing the effects of curling and warping restraint stresses and possible loss of support.

5.7 Verification and Revision of Design Equations

Two different modes of distress typically exist in whitetopping pavements, corner cracking caused by corner loading and mid-slab cracking caused by joint loading. Both of these types of failure were considered in Illislab analyses performed for developing, verifying and revising design equations.

5.7.1 Stress Computation Using the Finite Element Program ILSL2

For the corner and tied edge loading conditions, the following combinations of parameters were investigated for single and tandem truck axle configurations (the bold parameters are the values added or changed in 2004), resulting in nearly 4000 Illislab (ILSL2) analysis runs being performed:

Joint spacing, L	48, 72, and 144 in.
Concrete slab thickness, t_{pcc}	4, 5, 6 and **7** in.
Asphalt layer thickness, t_{ac}	3, 6, and 9 in.
Concrete modulus of elasticity, E_{pcc}	4 million psi
Asphalt modulus of elasticity, E_{ac}	0.05, **0.25**, 0.5, **0.75** and 1 million psi
Concrete Poisson's ratio, μ_{pcc}	0.15
Asphalt Poisson's ratio, μ_{ac}	0.35
Modulus of subgrade reaction, k	**50, 150, 300 and 500** psi/in.
Truck axle configuration	Single (SAL) &Tandem (TAL)
Slab loading locations	Corner & Longitudinal Edge

Stresses were computed using Illislab for each combination load configuration and analysis parameter stated above. A 20-kip single axle load (SAL) and a 40-kip tandem axle load (TAL) were positioned at the analysis slab longitudinal tied edge longitudinal mid-point and corners. Maximum tensile stresses at the bottom of each layer were calculated for both the concrete and asphalt. Maximum concrete flexural stresses and asphalt strains typically occurred for the joint loading condition, but the maximum values determined for each combination of load configuration and analysis parameter stated above was used for developing and revising the design equations.

Curling and warping restraint stresses were not incorporated into the parametric analysis based on the information collected during the load testing events and the uncertainty of modeled curling and warping restraint stress predictions. As shown by the measured profile deformations in the current and original study, the slab surface was never observed to be in a curled downward condition, even for the highest daytime temperature gradients of 6° F/in. Slab upward warping effects due to moisture differentials (surface drier than bottom) appeared greater than measured downward temperature curling effects. As a result, the maximum edge loading condition tensile stress occurs at the bottom of the concrete layer. At this location, the combined temperature curling and moisture warping restraint stress is in compression. The inclusion of restraint stresses would decrease the load-induced stresses and their omission is conservative for the edge loading case.

Certain combinations of parameters (high stiffness) may result in the maximum load-induced stress occurring during corner loading. In this case, the combined temperature and moisture restraint stresses would be additive to load-induced stresses and would be included in a conservative design procedure. However, for high slab stiffness values, the resulting concrete stress are low; typically in the range of about 100 to 150 psi when corner loading conditions are critical. It is unlikely that restraint stresses would exceed 200 psi resulting in a combined stress of about 300 to 350 psi. It is also likely that concrete flexural strength will exceed 600 psi. resulting in a stress ratio near 0.50. In the fatigue loading studies of concrete, maintaining a stress ratio of about 0.50 would result in nearly an unlimited number of load repetitions for that load category. Therefore, corner loading condition restraint stresses would likely not contribute to excessive consumption of the fatigue life and were not incorporated into the thickness design procedure.

5.7.2 Original Prediction Equations for Design Stresses and Strains

Prediction equations were derived for computing design concrete flexural stresses and asphalt flexural strains. A total of four equations [(1)] were developed as follows:

Concrete Stress For 20-kip SAL

$$\sigma_{pcc} = 919 + 18{,}492 / l_e - 575.3 \log k + 0.000133\, E_{ac} \qquad \text{(Eq. 5.8)}$$
$$R^2_{adj.} = 0.99$$

Concrete Stress For 40-kip TAL

$$\sigma_{pcc} = 671.2 - 0.000099\, E_{ac} - 437.1 \log k + 1.582 \times 10^4 / l_e \qquad \text{(Eq. 5.9)}$$
$$R^2_{adj.} = 0.99$$

Asphalt Strain For 20-kip SAL

$$1/\varepsilon_{ac} = 8.51114 \times 10^{-9}\, E_{ac} + 0.008619\, l_e/L \qquad \text{(Eq. 5.10)}$$
$$R^2_{adj.} = 0.99$$

Asphalt Strain For 40-kip TAL

$$1/\varepsilon_{ac} = 9.61792 \times 10^{-9}\, E_{ac} + 0.009776\, l_e/L \qquad \text{(Eq. 5.11)}$$
$$R^2_{adj.} = 0.99$$

where,

σ_{pcc} = maximum stress in the concrete slab, psi
ε_{ac} = maximum strains at bottom of asphalt layer, microstrain
E_{pcc} = concrete modulus of elasticity, assumed 4 million psi
E_{ac} = asphalt modulus of elasticity, psi
t_{pcc} = thickness of the concrete layer, in.
t_{ac} = thickness of the asphalt layer, in.
μ_{pcc} = Poissons ratio for the concrete, assumed 0.15
μ_{ac} = Poissons ratio for the asphalt, assumed 0.35
k = modulus of subgrade reaction, pci
l_e = effective radius of relative stiffness for fully bonded slabs, in.

$$= \{E_{pcc} * [t_{pcc}^3 / 12 + t_{pcc} * (NA - t_{pcc} / 2)^2] / [k * (1 - \mu_{pcc}^2)] + E_{ac} * [t_{ac}^3 / 12 + t_{ac} * (t_{pcc} - NA + t_{ac} / 2)^2] / [k * (1 - \mu_{ac}^2)]\}^{1/4}$$

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

$$= [E_{pcc} * t_{pcc}^2 / 2 + E_{ac} * t_{ac} * (t_{pcc} + t_{ac} / 2)] / [E_{pcc} * t_{pcc} + E_{ac} * t_{ac}]$$

L = joint spacing, in.

Each of the original equations developed to calculate the critical stresses and strains in a whitetopping pavement are dependent on the effective radius of relative stiffness of the layered system. The relative stiffness of a concrete slab and subgrade was defined by H.M. Westergaard [(24)] to include the contribution of the supporting medium stiffness as well as the flexural stiffness of slab in resisting load-induced deformation. The radius of relative stiffness appears in many of the equations dealing with stresses and deflections of concrete pavements. Whitetopping pavements include an additional structural layer of asphalt concrete. The stiffness contribution of the asphalt layer is incorporated into the effective radius of relative stiffness equation shown above.

5.7.3 Evaluation of the Original Stress and Strain Prediction Design Equations

The results from the parametric analysis performed were used to compare the stress and strain responses predicted by the 1998 equations for each modeling case to the actual modeled results. The following Figures 5.12 through 5.15 present the predicted versus actual results for the 1998 prediction equations and 2004 actual modeled responses. The overall poor prediction observed for the single axle load stress prediction model appears to be a concern, especially considering that this model is the one primarily relied upon by CDOT for the design procedure that incorporates ESALs. The other models appear to predict stress and strain responses considerably more reliably than the SAL stress model, but could potentially still be improved.

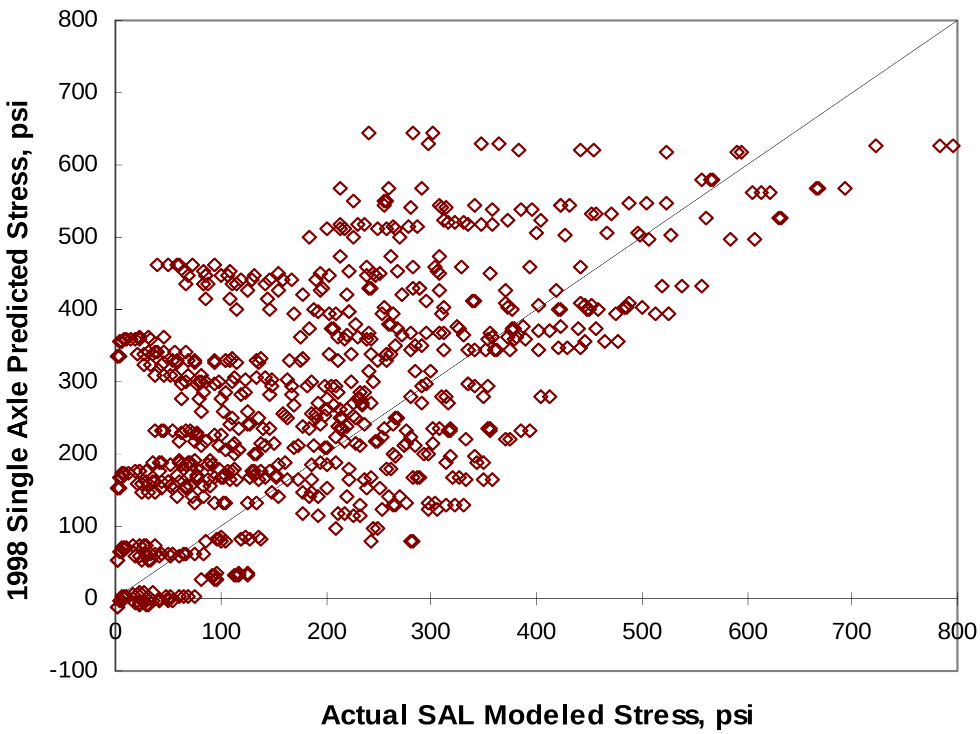

Figure 5.12. Original 1998 SAL Model Predicted Stresses Versus Actual Modeled Stresses

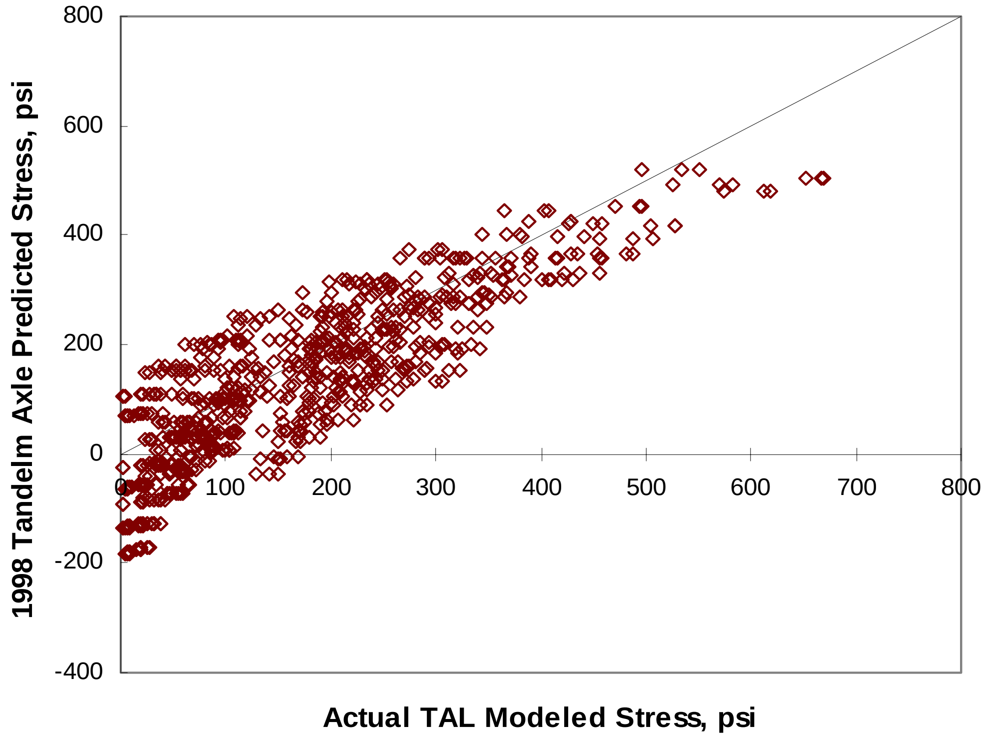

Figure 5.13. Original 1998 TAL Model Predicted Stresses Versus Actual Modeled Stresses

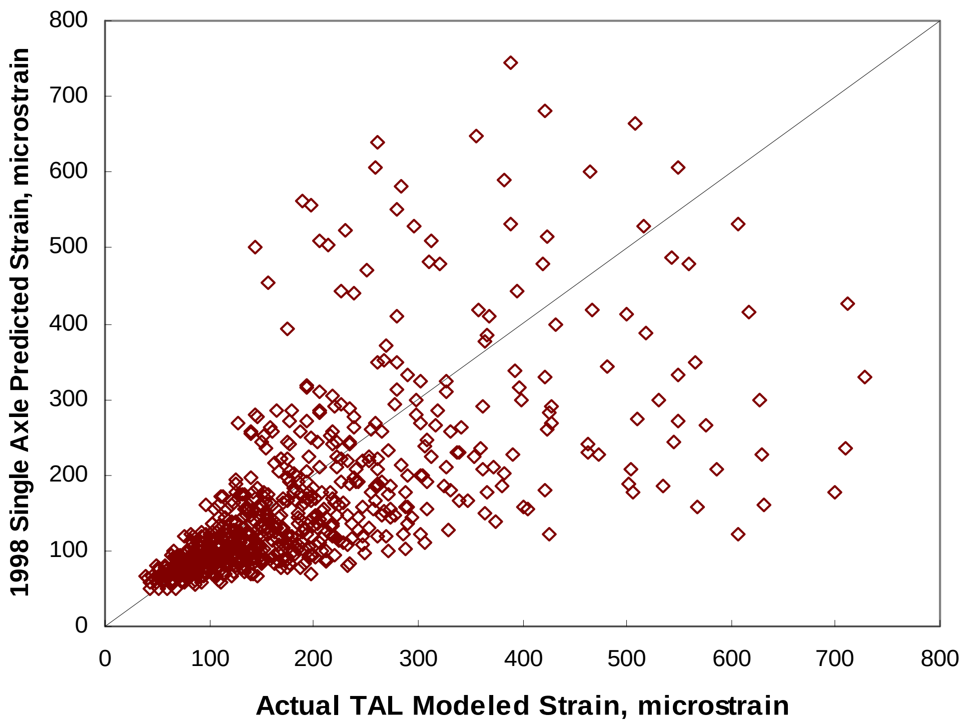

Figure 5.14. Original 1998 SAL Model Predicted Strains Versus Actual Modeled Strains

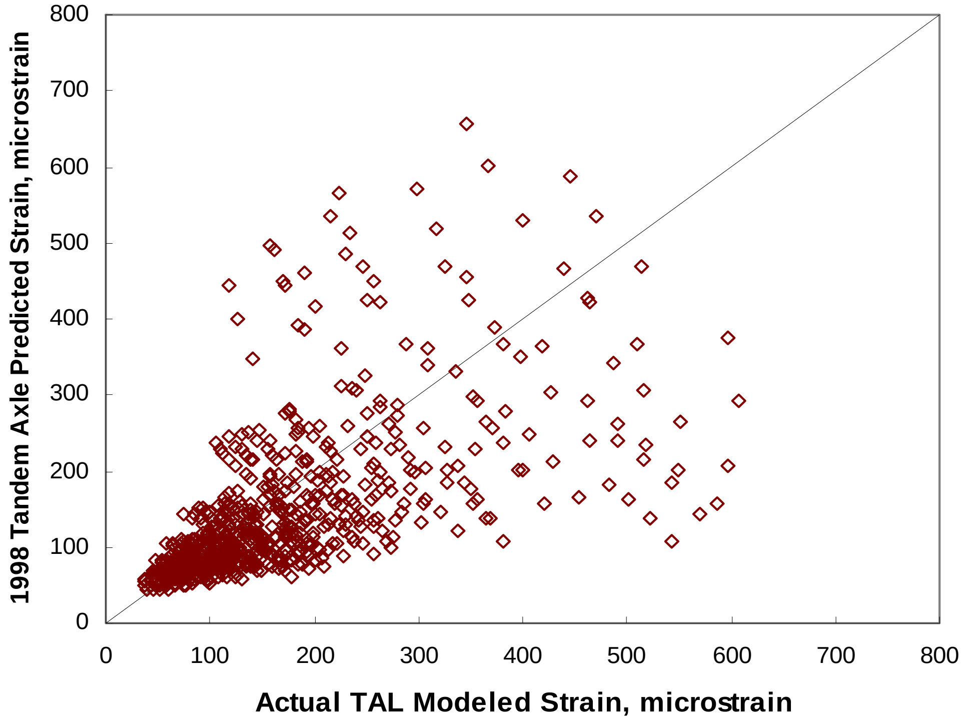

Figure 5.15. Original 1998 TAL Model Predicted Strains Versus Actual Modeled Strains

5.7.4 Revision of the Stress and Strain Prediction Design Equations

Since the single axle load concrete stress prediction model in particular does not appear to provide satisfactory predictions of the modeled stresses for the Illislab parametric analysis performed for this study, revised equations were developed. The newly revised response prediction equations are least squares linear regression equations with multiple predictors, as were the original equations. The four redeveloped prediction equations for computing design concrete flexural stresses and asphalt flexural strains are listed below:

2004 Concrete Stress For 20-kip SAL

$$(\sigma_{pcc})^{1/2} = 18.879 + 2.918\ t_{pcc} / t_{ac} + 425.44 / l_e - 6.955\text{x}10^{-6}\ E_{ac} - 9.0366 \log k + 0.0133\ L \qquad \text{(Eq. 5.12)}$$

$$R^2_{adj.} = 0.91$$

2004 Concrete Stress For 40-kip TAL

$$(\sigma_{pcc})^{1/2} = 17.669 + 2.668\ t_{pcc} / t_{ac} + 408.52 / l_e - 6.455\text{x}10^{-6}\ E_{ac} - 8.3576 \log k + 0.00622\ L \qquad \text{(Eq. 5.13)}$$

$$R^2_{adj.} = 0.92$$

2004 Asphalt Strain For 20-kip SAL

$$(\varepsilon_{ac})^{1/4} = 8.224 - 0.2590\, t_{pcc} / t_{ac} - 0.04419\, l_e - 6.898\text{x}10^{-7}\, E_{ac} - 1.1027 \log k \quad \text{(Eq. 5.14)}$$

$$R^2_{adj} = 0.81$$

2004 Asphalt Strain For 40-kip TAL

$$(\varepsilon_{ac})^{1/4} = 7.923 - 0.2503\, t_{pcc} / t_{ac} - 0.04331\, l_e - 6.746\text{x}10^{-7}\, E_{ac} - 1.0451 \log k \quad \text{(Eq. 5.15)}$$

$$R^2_{adj} = 0.82$$

where
σ_{pcc} = maximum stress in the concrete slab, psi
ε_{ac} = maximum strains at bottom of asphalt layer, microstrain
E_{pcc} = concrete modulus of elasticity, assumed 4 million psi
E_{ac} = asphalt modulus of elasticity, psi
t_{pcc} = thickness of the concrete layer, in.
t_{ac} = thickness of the asphalt layer, in.
μ_{pcc} = Poissons ratio for the concrete, assumed 0.15
μ_{ac} = Poissons ratio for the asphalt, assumed 0.35
k = modulus of subgrade reaction, pci
l_e = effective radius of relative stiffness for fully bonded slabs, in.

$$= \{E_{pcc} * [t_{pcc}^3 / 12 + t_{pcc} * (NA - t_{pcc} / 2)^2] / [k * (1 - \mu_{pcc}^2)] + E_{ac} * [t_{ac}^3 / 12 + t_{ac} *(t_{pcc} - NA + t_{ac} / 2)^2] / [k * (1 - \mu_{ac}^2)]\}^{0.25}$$

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

$$= [E_{pcc} * t_{pcc}^2 / 2 + E_{ac} * t_{ac} * (t_{pcc} + t_{ac} / 2)] / [E_{pcc} * t_{pcc} + E_{ac} * t_{ac}]$$

L = joint spacing, in.

Each of the revised equations to calculate the critical stresses and strains in a whitetopping pavement are dependent on the effective radius of relative stiffness of the layered system and asphalt layer modulus as were the original equations. Modulus of subgrade reaction was included in the original concrete stress prediction equations, but it is included in all of the new equations. However, all of the revised equations also utilize asphalt and concrete thickness as predictors, and the concrete stress equations use joint spacing as an additional predictor.

Figures 5.16 through 5.23 present the predicted versus actual results for the 2004 prediction equations and 2004 actual modeled responses. These figures present pairs of plots for each prediction equation (i.e., Figures 5.16 and 5.17 are the two predicted versus actual figures for the SAL stress prediction equation, etc.). Pairs of predicted versus actual plots are presented for each prediction equation due to the transformations performed during the regression analysis to remove curvature trends in the data and improve overall prediction quality; the equations developed to predict stresses actually predict the square root of the stresses and the strain prediction equations predict the fourth root of the strains. Therefore, the first plot in each pair presents the square root or fourth root values for each respective model, and the second plot presents the predicted versus actual values once the predictions are transformed back to stresses and strains.

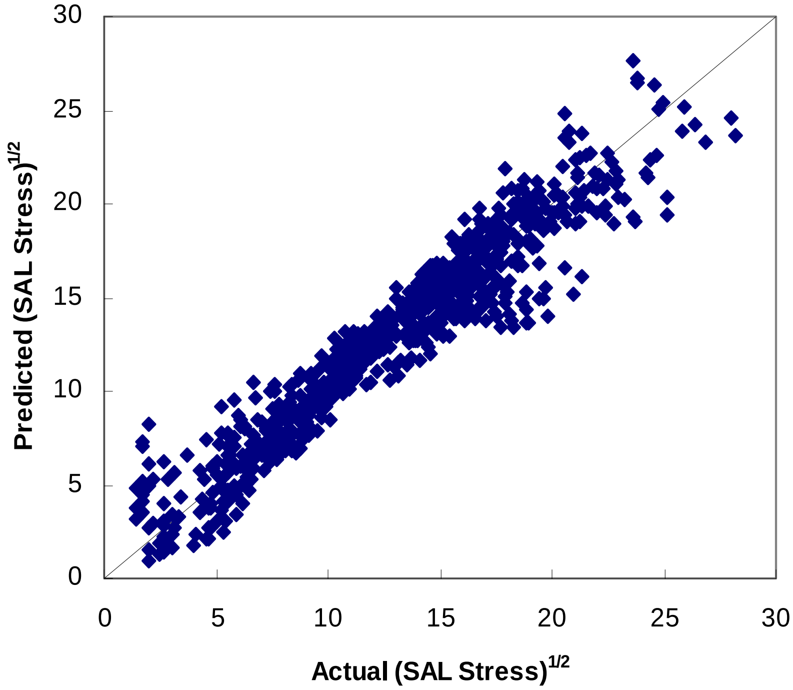

Figure 5.16. Revised SAL Concrete Stress Model Predicted Versus Actual Plot

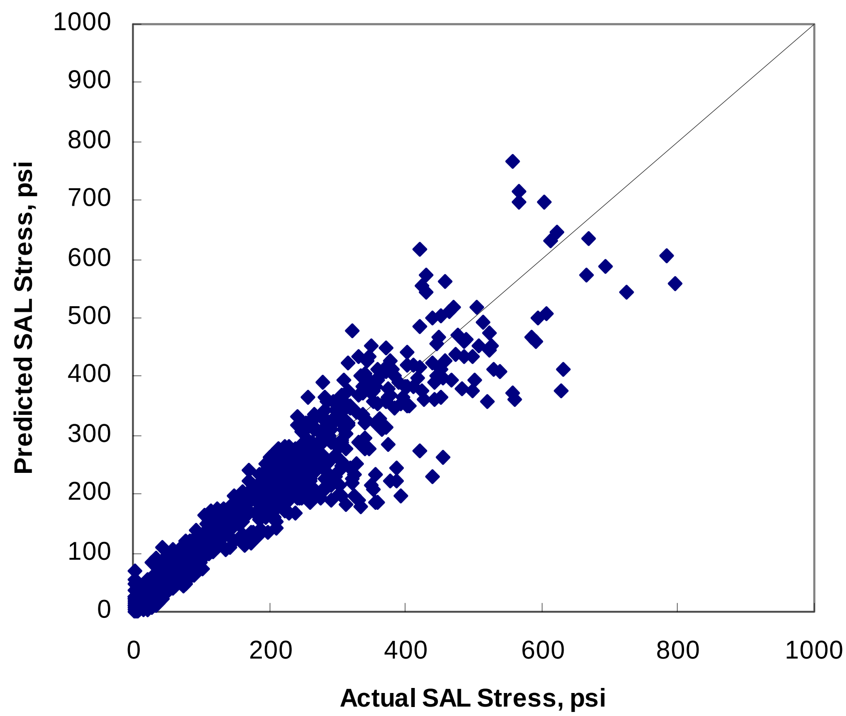

Figure 5.17. Revised SAL Concrete Stress Model Predicted Versus Actual Plot

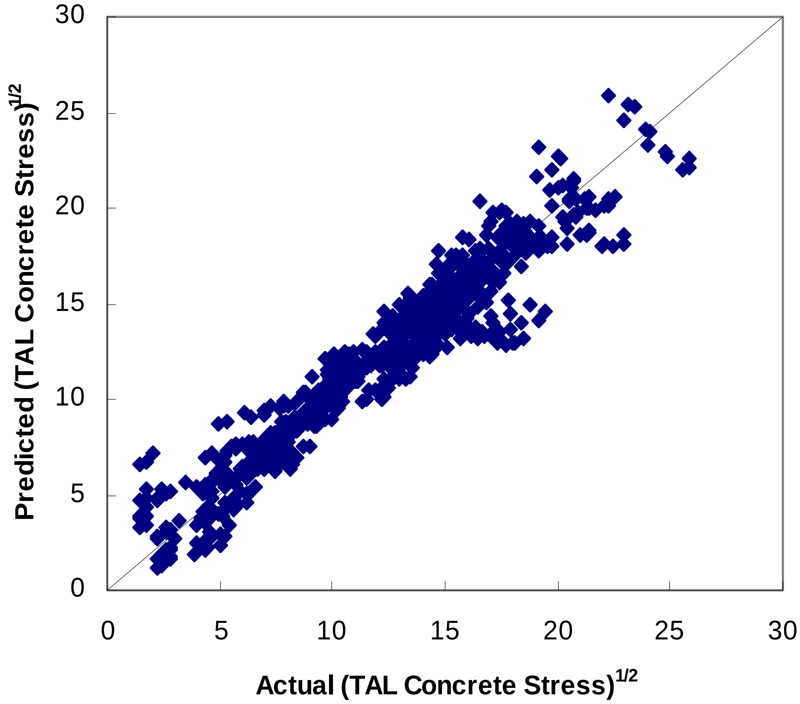

Figure 5.18. Revised TAL Concrete Stress Model Predicted Versus Actual Plot

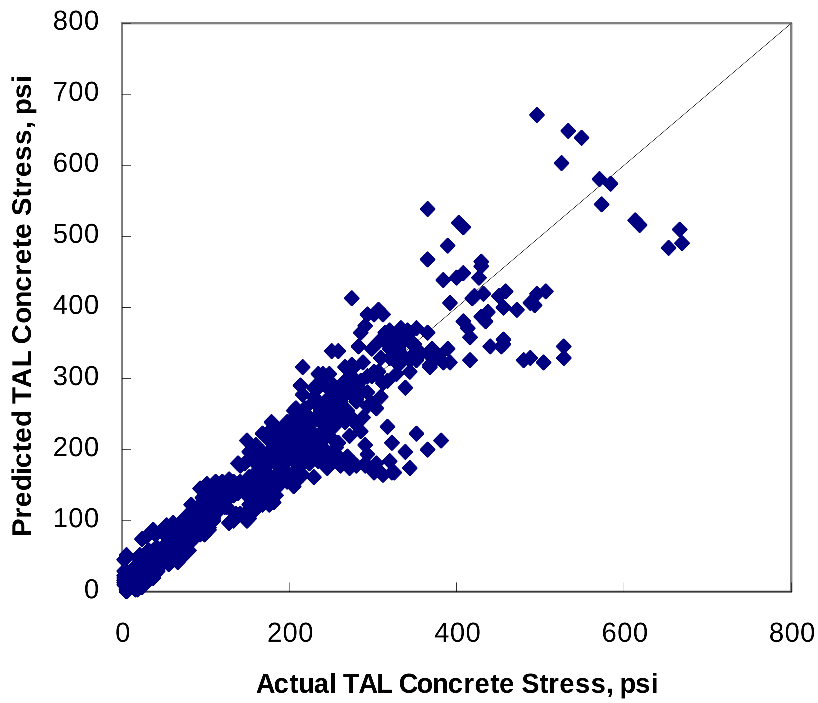

Figure 5.19. Revised TAL Concrete Stress Model Predicted Versus Actual Plot

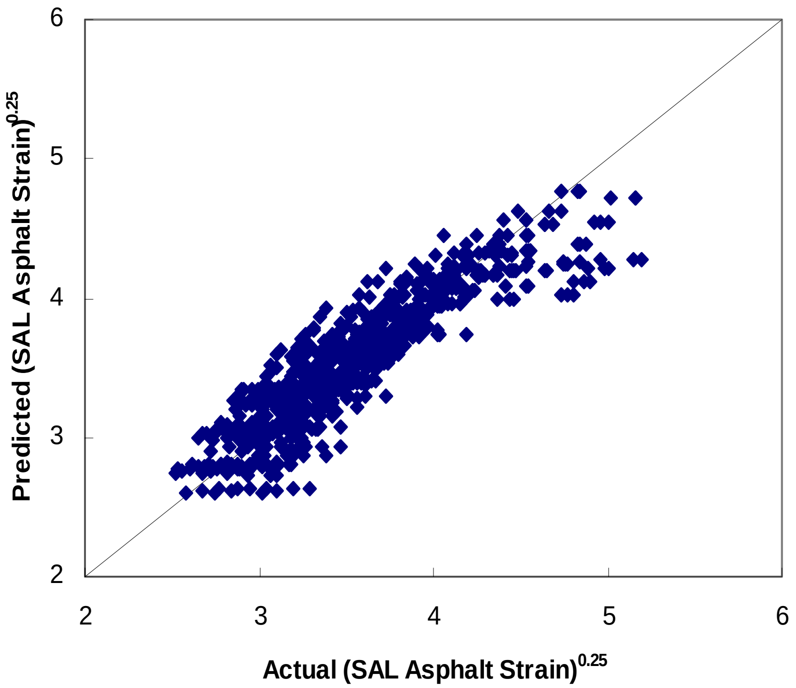

Figure 5.20. Revised SAL Asphalt Strain Model Predicted Versus Actual Plot

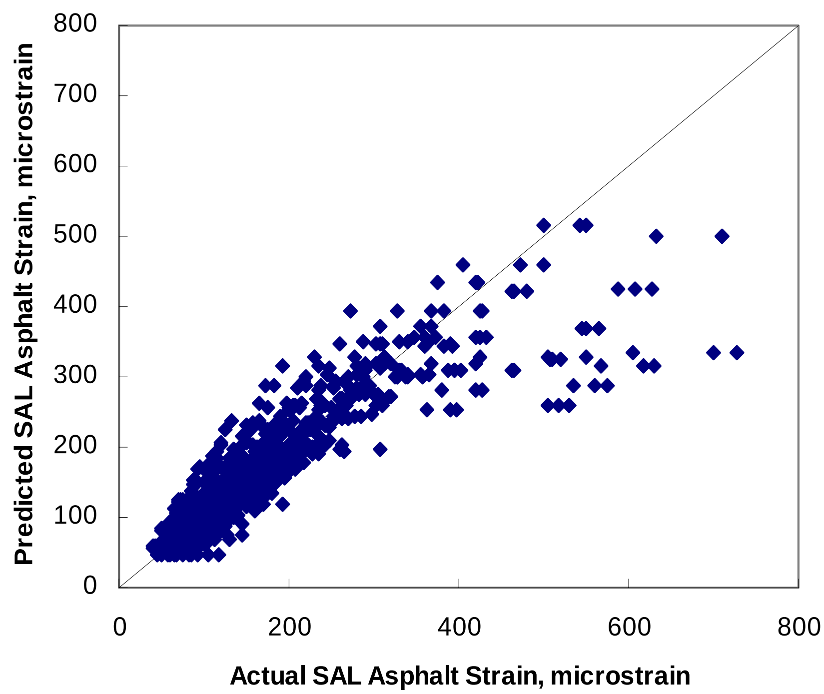

Figure 5.21. Revised SAL Asphalt Strain Model Predicted Versus Actual Plot

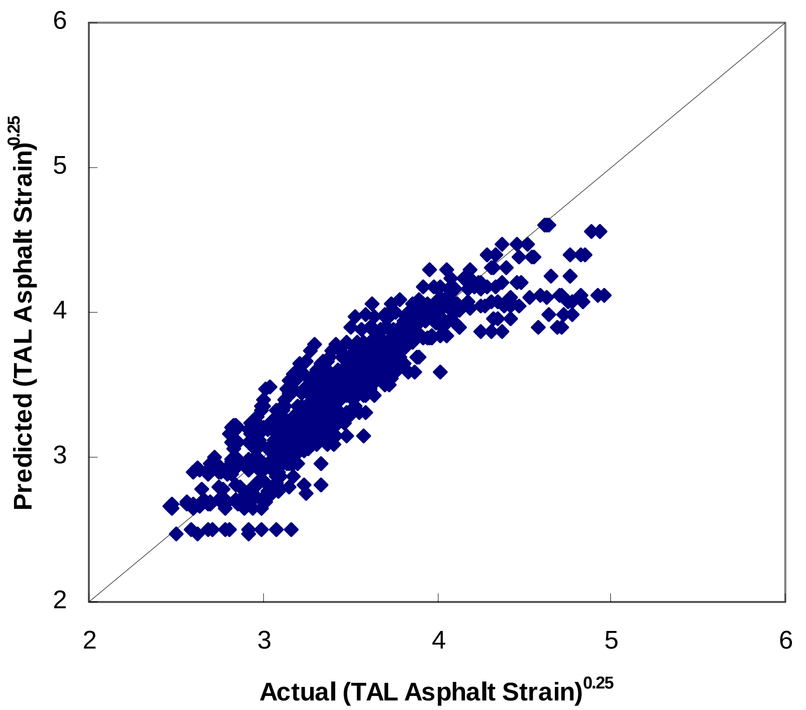

Figure 5.22. Revised TAL Asphalt Strain Model Predicted Versus Actual Plot

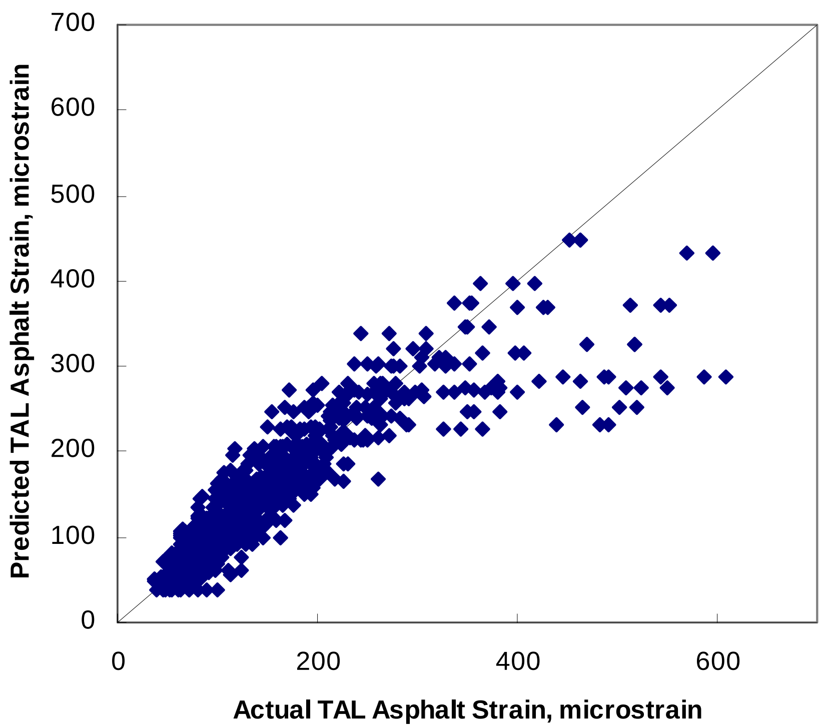

Figure 5.23. Revised TAL Asphalt Strain Model Predicted Versus Actual Plot

As discussed, the first of each of the pair of predicted versus actual plots presents the actual transformed prediction values, and the second plot presents the corresponding converted stress or strain value. Comparing the predicted versus actual plots for each of the revised response prediction models to the corresponding plots for the original models (Figures 5.12 through 5.15), it appears that all of the revised models provide considerably improved predictions of the corresponding stress or strain responses. The improvement is particularly significant for the SAL concrete stress prediction model because it is the primary model used for the ESAL design approach, which will be discussed in subsequent sections of this report.

5.7.5 Revised Adjustments of the Stress and Strain Predictions

The original 1998 equations developed to adjust theoretical stresses and strains to account for conditions such as partial bond and loss of support due to temperature-induced slab curling were revised during this study. The stresses modeled are for whitetopping pavements with fully bonded concrete and asphalt layers. Field tests and theoretical analysis have shown that whitetopping pavements are partially bonded composite pavements. As previously presented, an increase in concrete flexural stress of 51 percent from fully bonded pavements would be required to account for the loss of bonding at the 95 percent confidence level. Asphalt strains are decreased by approximately 10 percent to account for the partial bonding condition at the 95 percent confidence level. Effects of temperature-induced slab curling on load-induced stresses were also included in the thickness design procedure, and all of the original 1998 adjustments for these stresses and strains were also revised during the current study.

5.8 PCC and Asphalt Fatigue Equations

The Portland Cement Association (PCA) developed a fatigue criterion [25] based on Miner's hypothesis [26] that fatigue resistance not consumed by repetitions of one load is available for repetitions of other loads. Revisions of this portion of the 1998 procedure are not necessary for the current study; the information included in this report section was presented in the 1998 report and is again included in this report to provide a comprehensive summary of the revised Colorado thin whitetopping design procedure. In a design, the total fatigue should not exceed 100%. The concrete fatigue criterion was incorporated as follows [25,26]:

For $SR > 0.55$

$$\log_{10}(N) = (0.97187 - SR) / 0.0828 \qquad \text{(Eq. 5.16)}$$

For $0.45 \leq SR \leq 0.55$

$$N = (4.2577 / (SR - 0.43248))^{3.268} \qquad \text{(Eq. 5.17)}$$

For $SR < 0.45$

$$N = \text{Unlimited} \qquad \text{(Eq. 5.18)}$$

where,

SR = flexural stress to strength ratio
N = number of allowable load repetitions

Asphalt pavements are generally designed based on two criteria, asphalt concrete fatigue and subgrade compressive strain. Subgrade compressive strain criterion was intended to control pavement rutting for conventional asphalt pavements. Since concrete slabs cover the asphalt layer in whitetopping pavements, pavement rutting should not be the governing distress. Therefore, asphalt concrete fatigue was used as the design criterion in this procedure. The asphalt concrete fatigue equation developed by the Asphalt Institute [(27)] was employed in the development of the whitetopping design procedure. The asphalt concrete fatigue equation is as follows [(27)]:

$$N = C * 18.4 * (4.32 \text{ X } 10^{-3}) * (1/\varepsilon_{ac})^{3.29} * (1/E_{ac})^{0.854} \qquad \text{(Eq. 5.19)}$$

where,

N = number of load repetitions for 20% or greater AC fatigue cracking
ε_{ac} = maximum tensile strain in the asphalt layer
E_{ac} = asphalt modulus of elasticity, psi
C = correction factor = 10^M
$M = 4.84 * [(V_b / (V_v + V_b) - 0.69]$
V_b = volume of asphalt, percent
V_v = volume of air voids, percent

For typical asphalt concrete mixtures, M would be equal to zero. The correction factor, C, would become one, and was omitted from the equation. However, since whitetopping is designed to rehabilitate deteriorated asphalt pavement, the allowable number of load repetitions (N) needs to be modified to account for the amount of fatigue life consumed prior to whitetopping construction. Therefore, the calculated repetitions must be multiplied by the fractional percentage representing the amount of fatigue life remaining in the asphalt concrete. For example, if it is determined that 25 percent of the asphalt fatigue life has been consumed prior to whitetopping, the calculated allowable repetitions remaining must be multiplied by 0.75.

The whitetopping pavement thickness design involves the selection of the proper concrete slab dimension and thickness. Two criteria were used in governing the pavement design; asphalt and concrete fatigue under joint or corner loading. Temperature and loss of support effects were also considered in the design procedure. A design example is presented in next section to illustrate how to use the developed procedure to calculate the required whitetopping concrete thickness.

5.9 Whitetopping Pavement Design Example

An example problem is presented to illustrate the steps involved in the design procedure. The example represents the design of a whitetopping project for a secondary roadway. Based on traffic surveys, it was determined that approximately 25 percent of the asphalt concrete fatigue life has been already consumed. Visual inspection of the existing pavement indicates that asphalt fatigue cracking is not too severe (magnitude and quantity) and supports the decision to use a whitetopping rehabilitation. Results are presented in Table 5.1 for the expected loads (Column 1 in Table 5.1) and expected number of repetitions (Column 8 in Table 5.1). Parameters and material properties used in the design are the following:

Asphalt modulus of elasticity, E_{ac} = 350,000 psi
Asphalt thickness, t_{ac} = 5-1/2 in.
Existing modulus of subgrade reaction, k = 200 pci

Concrete modulus of elasticity, E_{pcc} = 4,000,000 psi
Concrete modulus of rupture, MR = 650 psi
Concrete Poisson's ratio, μ_{pcc} = 0.15
Asphalt Poisson's ratio, μ_{ac} = 0.35
Temperature differential, Δ_T = 3° F per in. throughout the day
Trial concrete thickness = 4 in.
Joint spacing, L = 72 in.
Existing asphalt fatigue = 25 percent

Procedure Steps:

1. Determine le and L/le for the set of design parameters.

$$l_e = 24.41$$

$$L/l_e = 2.95$$

2. Using the calculated l_e and L/l_e along with the modulus of subgrade reaction, k, Equation 5.12 is used to compute the load-induced critical concrete stresses (Col. 2 in Table 5.1) and Equation 5.14 is used to compute the load-induced critical asphalt strains (Col. 3 in Table 5.1) for anticipated 20-kip single axle loads (SAL). Stresses and strains for the remaining axle loads are computed as ratios of the 20-kip SAL load. Results are presented in the upper portion of Table 5.1.

3. Repeat step 2 for the anticipated tandem axle loads (TAL). Use Equation 5.13 to compute the concrete stresses and Equation 5.15 to compute the asphalt strains for a 40-kip TAL shown in the lower portion of Columns 2 and 3 in Table 5.1.

4. Using Equations 5.3 and 5.5, compute the partial bond adjustment to the computed fully bonded concrete stresses and asphalt strains. Adjust the stresses and strains accordingly as shown in Columns 4 and 5 of Table 5.1, respectively.

5. Use Equation 5.7 to adjust the concrete stress to account for the loss of support due to temperature-induced concrete slab curling. There is no adjustment for the asphalt strains. Therefore, Columns 6 and 7 of Table 5.1 reflect the total concrete stresses and asphalt strains due to the anticipated loading and temperature gradient.

6. With the total concrete stresses and asphalt strains known, the fatigue analyses are conducted. Separate fatigue analyses must be done for the concrete and asphalt layers. For a given set of parameters, one of the two analyses will govern and determine the required concrete thickness for the selected joint spacing.

7. Compute the concrete stress ratio, SR, in Column 9, by dividing the total concrete stresses in Column 6 by the design concrete modulus of rupture.

8. Using the stress ratio and Equations 5.16 to 5.18, determine the allowable repetitions for the concrete layer in Column 10.

9. Compute the percent fatigue in Column 11 by dividing Column 8 by Column 10, multiplying by 100, and totaling the concrete fatigue damage for all axle loadings.

10. Enter the maximum asphalt microstrain from Column 7 into Column 12 as shown.

11. Using the existing asphalt modulus of elasticity and the microstrains in Column 12, compute the allowable load repetitions for the asphalt layer from Equation 5.19 and enter these values into Column 13.

12. The percent fatigue for the asphalt layer and the total asphalt fatigue damage is computed in the same manner as used for the concrete fatigue computation in Step 9 except for the addition of fatigue damage already consumed prior to whitetopping construction. Sum the percent fatigue for the given load cases as well as the percentage previously consumed to compute the total asphalt fatigue damage at the bottom of Column 14.

Example Summary. In this case, both the concrete and asphalt fatigue analyses dictated the required whitetopping thickness. For the existing asphalt and subgrade conditions, a concrete whitetopping thickness of 4 in. with a joint spacing of 72 in. is shown to be sufficient to carry the anticipated traffic loading.

Table 5.1. Design Example

Axle Load, kips	Multiplied by LSF	Critical Concrete Stresses and Asphalt Strains					
		Load Induced		Bond Adjustment		Loss of Support Adjustment	
		Stress, psi	Microstrain	Stress, psi	Microstrain	Stress, psi	Microstrain
1	1	2	3	4	5	6	7
Single Axles				**le =**	**24.41**	**L/le =**	**2.95**
22	22	287	372	434	333	484	333
20	20	**261**	**305**	**394**	**273**	**440**	273
18	18	235	305	355	273	396	273
16	16	209	271	315	242	352	242
14	14	183	237	276	212	308	212
12	12	157	203	237	181	264	181
10	10	131	169	197	151	220	151
8	8	104	135	158	121	176	121
6	6	78	102	118	90	132	90
4	4	52	68	79	60	88	60
2	2	26	34	39	30	44	30
Tandem Axles							
44	44	258	326	389	292	434	292
40	40	**234**	**267**	**353**	**239**	**394**	239
36	36	211	267	318	239	355	239
32	32	187	237	283	212	315	212
28	28	164	208	247	186	276	186
24	24	140	178	212	159	237	159
20	20	117	148	177	132	197	132
16	16	94	119	141	106	158	106
12	12	70	89	106	79	118	79
8	8	47	59	71	52	79	52
4	4	23	30	35	26	39	26

Table 5.1. Design Example (continued)

Axle Load, kips	Expected Repetitions	Concrete Fatigue Analysis			Asphalt Fatigue Analysis		
		Concrete Stress Ratio	Allowable Repetitions, N	Fatigue Percent, %	Asphalt microstrain	Allowable Repetitions, N	Fatigue Percent, %
1	8	9	10	11	12	13	14
Single Axles		**Percent Asphalt Concrete Fatigue Life Previously Consumed:**					**25**
22	200	0.744	563	35.6	333	302,460	0.1
20	600	0.677	3,691	16.3	273	586,315	0.1
18	2,500	0.609	24,222	10.3	273	586,315	0.4
16	5,000	0.541	160,469	3.1	242	864,831	0.6
14	7,500	0.474	3,863,057	0.2	212	1,343,938	0.6
12	25,000	0.406	unlimited	0.0	181	2,236,180	1.1
10	550,000	0.338	unlimited	0.0	151	4,085,411	13.5
8	875,000	0.271	unlimited	0.0	121	8,548,812	10.2
6	1,250,000	0.203	unlimited	0.0	90	22,182,962	5.6
4	1,750,000	0.135	unlimited	0.0	60	85,410,299	2.0
2	5,000,000	0.068	unlimited	0.0	30	871,988,534	0.6
Tandem Axles							
44	5	0.667	4,770	0.1	292	467,008	0.0
40	50	0.607	25,771	0.2	239	905,507	0.0
36	500	0.546	139,508	0.4	239	905,507	0.1
32	1,500	0.485	1,698,113	0.1	212	1,335,870	0.1
28	5,000	0.425	unlimited	0.0	186	2,076,368	0.2
24	50,000	0.364	unlimited	0.0	159	3,455,854	1.4
20	75,000	0.303	unlimited	0.0	132	6,316,223	1.2
16	500,000	0.243	unlimited	0.0	106	13,224,754	3.8
12	750,000	0.182	unlimited	0.0	79	34,350,779	2.2
8	1,000,000	0.121	unlimited	0.0	52	132,526,818	0.8
4	1,250,000	0.061	unlimited	0.0	26	1,361,391,514	0.1
		Total Concrete Fatigue, % =		**66.2**	**Total Asphalt Fatigue, % =**		**69.7**

6.0 MODIFIED DESIGN PROCEDURE INCORPORATING ESALS

The State of Colorado currently designs pavements using the procedure developed by the American Association of State Highway and Transportation Officials (AASHTO) [28]. This empirical procedure is based on pavement performance data collected during the AASHO Road Test in Ottawa, Illinois in the late 1950's and early 1960's. Traffic (frequency of axle loadings) is represented by the concept of an Equivalent 18-kip Single Axle Load (ESAL). Factors are used to convert the damage caused by repetitions of all axles in the traffic mix (single and tandem) to an equivalent damage due to 18-kip ESALs. Because the relative damage caused by ESALs is a function of the pavement thickness, a series of ESAL conversion factors have been developed for a range of concrete thicknesses. However, the minimum concrete thickness included in the AASHTO design manual is 6 in. Since whitetopping thicknesses below 6 in. are anticipated, it was necessary to develop correction factors to convert ESAL estimations based on thicker concrete sections. Also, because the ESAL method of design appears to overestimate the required PCC thickness, it was necessary to develop a conversion factor, which would make the empirical and mechanistic procedures more compatible. It was not necessary to revise the majority of this portion of the 1998 study during the current study. Much of the information discussed in this report section was presented in the 1998 report [1] and is again included to provide a comprehensive summary of the revised Colorado thin whitetopping design procedure.

6.1 Converting Estimated ESALs to Whitetopping ESALs

The State of Colorado provided axle distributions for two highway categories (Primary and Secondary) anticipated as typical whitetopping traffic loading. The ESAL conversion factors were for an 8-in.-thick concrete pavement and a terminal serviceability of 2.5. The conversion factors were extrapolated for pavement thicknesses as low as 4 in. and the total ESALs were computed for a range of possible whitetopping thicknesses. For each highway category, ESAL conversions were developed as a percentage of the total ESALs computed for an 8-in.-thick concrete pavement. Figure 6.1 shows the curves developed for converting total estimated ESALs based on an assumed concrete thickness of 8 in. With these conversions, the designer only needs to obtain the design ESALs based on an assumed concrete thickness of 8 in. For each trial whitetopping thickness, the total ESAL estimation is adjusted based on the following conversion equations [1]:

Primary Highway: $$F_{ESAL} = 0.985 + 10.057 * (t_{pcc})^{-3.456} \quad \text{(Eq. 6.1)}$$

Secondary Highway: $$F_{ESAL} = (1.286 - 2.138 / t_{pcc})^{-1} \quad \text{(Eq. 6.2)}$$

where,

F_{ESAL} = Conversion factor from ESAL estimation based on assumed 8-in.-thick concrete pavement

t_{pcc} = thickness of the concrete layer, in.

For example, for the design of a 4 ½-in.-thick whitetopping for a secondary highway, the estimated ESALs based on an assumed 8-in.-thick pavement should be converted using the secondary highway conversion equation (i.e. 750,000 ESALs converts to 925,000 thin whitetopping ESALs using Equation 6.1).

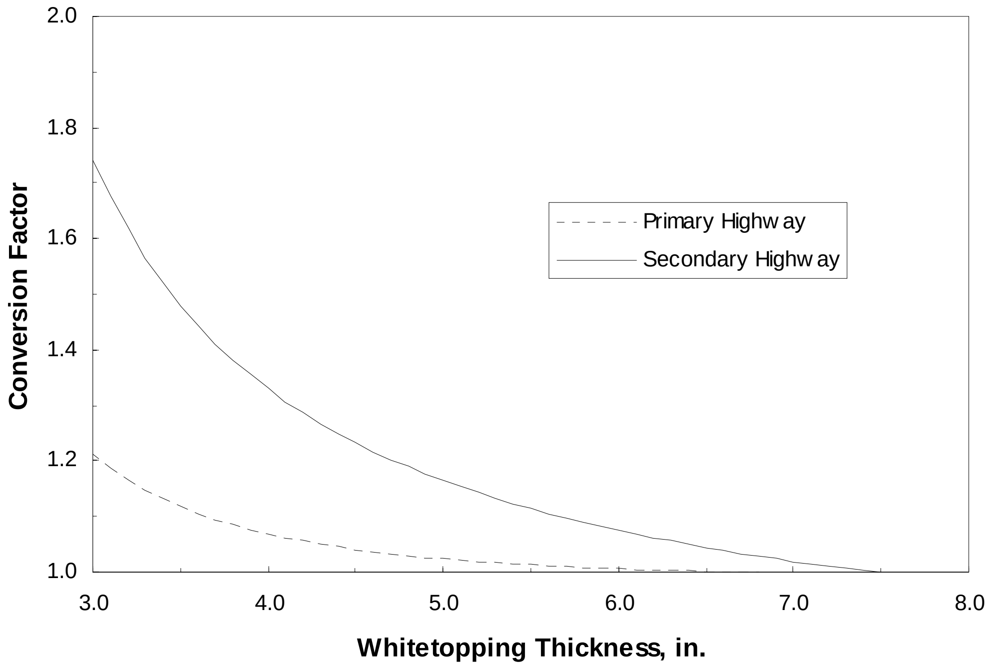

Figure 6.1. Conversion of 8-in.-thick ESAL's to Whitetopping ESAL's

6.2 Modified Whitetopping Thickness Design Conversion

Converting the mechanistic approach traffic distribution to ESALs and using the calculated ESAL value as the expected number of 18-kip axle load repetitions (and setting all other axle loads to zero repetitions) does not result in a design thickness equal to that calculated for the original mechanistic axle load distribution. For instance, in the example shown in Table 5.1, for the axle load traffic distribution given, the required whitetopping thickness is 4 in. Using AASHTO conversion factors developed based on the original conversion factors for an assumed 8-in.-thick pavement, and the secondary highway conversion discussed in the previous section, the estimated number of ESALs is 245,544. Using this number of expected repetitions for the 18-kip axle load in Table 5.1 and setting all other axles loads to zero repetitions results in about 1,000 percent fatigue life consumed. For the ESALs computed, the required thickness is calculated to be over 5 in. Therefore, a conversion was developed for the 1998 design procedure to equate the two design procedures.

Comparative designs were calculated for a series of input parameters for the two procedures. Ranges of input parameters were as follows:

Asphalt modulus of elasticity, E_{ac} = 50,000 to 1,000,000 psi
Asphalt thickness, t_{ac} = 3 to 9 in.
Existing modulus of subgrade reaction, k = 100 to 400 pci
Concrete modulus of rupture, MR = 550 to 750 psi

Input parameters kept constant were the following:

Concrete modulus of elasticity, E_{pcc} = 4,000,000 psi

Concrete Poisson's ratio, $\mu_{pcc} = 0.15$
Asphalt Poisson's ratio, $\mu_{ac} = 0.35$
Temperature differential, $\Delta_T = 3°$ F per in. throughout the day
Joint spacing, L = 72 in.

A comparison of the required PCC thickness calculated by both design procedures is shown in Figure 6.2. The mechanistic procedure utilizes axle load distribution and the empirical procedure uses ESALs, and the differences observed between the two methods is primarily a result of the conversions necessary to transform mechanistic load categories into theoretical whitetopping ESALs. However, the relationship for both the mechanistic and empirical procedures is very sensitive to all input parameters (i.e., particularly traffic levels and pavement physical characteristics). Therefore, the recommended approach for the design of thin whitetopping rehabilitation projects is to evaluate the each project using both the mechanistic and empirical techniques and compare the results.

The trend in the Figure 6.2 data suggests that a relationship exists between the two procedures and that a correlation could be developed to convert the trial thickness prior to being input into the ESAL design procedure. An equation was developed for this purpose during the original study, and the updated equation based on the revised models is as follows:

$$t_{INPUT} = 1.1251\,(t_{TRIAL}) + 0.6299 \qquad \text{(Eq. 6.3)}$$

where,

t_{INPUT} = converted concrete thickness to be input into the ESAL design procedure calculations
t_{TRIAL} = trial concrete thickness which becomes whitetopping thickness specified

As shown in Figure 6.2, this correlation was developed for whitetopping thickness below 8 in. and should not be extrapolated further. Field data were collected on a maximum PCC thickness of about 7-1/2 in. and the design procedure equations were developed from theoretical stresses for concrete with a maximum thickness of 8 in. Load-induced stresses for thicker concrete sections have not been verified by field testing and, therefore, it is not recommended that this procedure be used to design whitetopping sections greater than about 7-1/2 to 8 in.

Revised equations 5.12 and 5.14 were modified as follows to calculate the stress and strain due to an 18-kip Single Axle Load:

2004 Concrete Stress For 18-kip SAL

$$\sigma_{pcc} = 0.9*(18.879 + 2.918\; t_{pcc} / t_{ac} + 425.44 / l_e - 6.955\text{x}10^{-6}\, E_{ac} - 9.0366 \log k + 0.0133\, L)^2 \qquad \text{(Eq. 6.4)}$$

2004 Asphalt Strain For 18-kip SAL

$$\varepsilon_{ac} = 0.9*(8.224 - 0.2590\; t_{pcc} / t_{ac} - 0.04419\; l_e - 6.898\text{x}10^{-7}\, E_{ac} - 1.1027 \log k)^4 \qquad \text{(Eq. 6.5)}$$

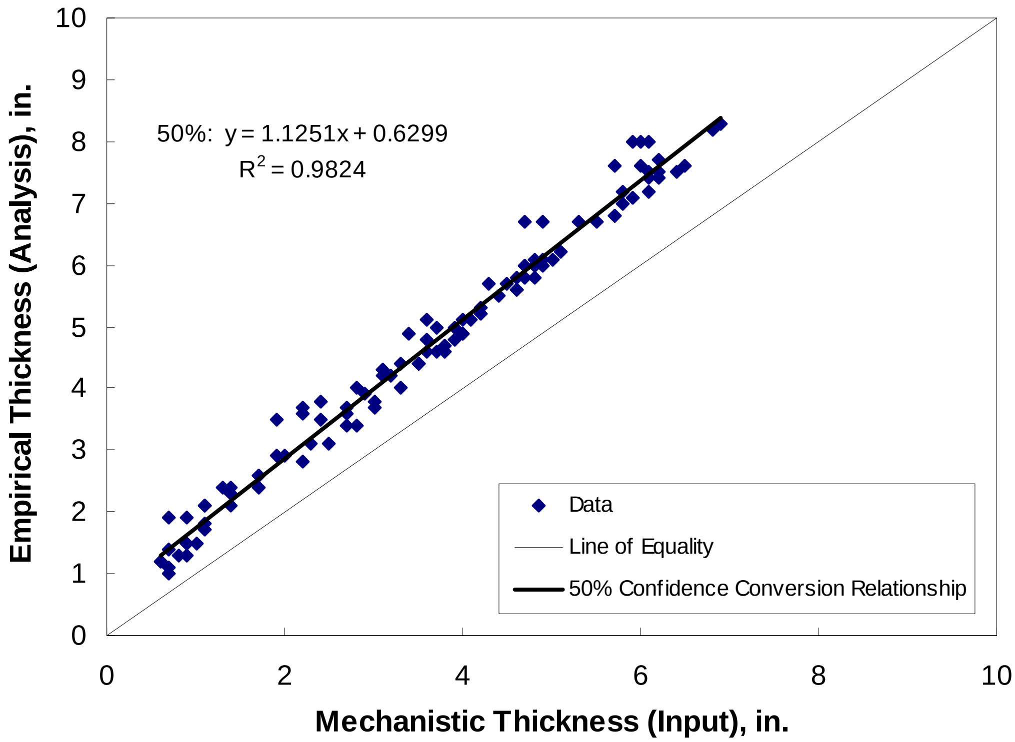

Figure 6.2. Thickness Conversion from Mechanistic Procedure to Empirical Procedure

6.3 Whitetopping Pavement Design Example

Figure 6.3 shows example empirical design method calculations including the relationship defined by Equation 6.3 for the mechanical design method example previously discussed and presented in Table 5.1. As shown, a required thickness of 4-1/4 in. (rounded up to the nearest 1/4 in. for a 4.1 in. trial thickness) is the result of the modified design approach incorporating ESALs. While this is slightly different from the 4 in. thickness required by the mechanistic procedure, it is within the standard deviation typically achieved by slip-form pavers. Also, since this procedure is meant for whitetopping thicknesses between approximately 4 and 8 in., using a minimum 4 in. concrete thickness and comparing results for thicker designs to results obtained using conventional design procedures is recommended.

Whitetopping Input Parameters

Highway Category (Primary or Secondary)*	Secondary
Joint Spacing, in.	72
Trial Concrete Thickness, in.	4.1
Concrete Flexural Strength, psi	650
Concrete Elastic Modulus, psi	4,000,000
Concrete Poisson's Ratio	0.15
Asphalt Thickness, in.	5.5
Asphalt Elastic Modulus, psi	350,000
Asphalt Poisson's Ratio	0.35
Asphalt Fatigue Life Previously Consumed, %	25
Subgrade Modulus, pci	200
Temperature Gradient, °F/in.	3
Design ESALs	245,544

Converted Concrete Thickness, in. =	5.24
ESAL Conversion Factor =	1.3072
Neutral Axis =	3.07
le =	27.36
L/le =	2.63

Critical Concrete Stresses and Asphalt Strains					
Load Induced		Bond Adjustment		Support Adjustment	
Stress, psi	μstrain	Stress, psi	μstrain	Stress, psi	μstrain
1	2	3	4	5	6
201	**228**	**303**	**204**	**338**	**204**

ESAL Fatigue Analysis						
No. of 18-kip ESALs	Concrete Fatigue Analysis			Asphalt Fatigue Analysis		
	Stress Ratio	Allowable ESALs	Fatigue, %	Asphalt μstrain	Allowable ESALs	Fatigue, %
7	8	9	10	11	12	13
3.2E+05	**0.520**	**3.2E+05**	**99.9**	**204**	**1.5E+06**	**21.0**

Concrete Fatigue, % = 99.9 **Asphalt Fatigue, % = 46.0**

Required Whitetopping Thickness = 4.25 in.

Figure 6.3. Design Example Incorporating ESALs for Traffic Input

7.0 SENSITIVITY ANALYSIS

Sensitivity analyses were conducted for calculated whitetopping thicknesses. Parameters studied for sensitivity include asphalt thickness, modulus of subbase/subgrade reaction, asphalt modulus of elasticity, concrete flexural strength and the expected number of 18-kip ESALs. Each of the figures presented in this section include the original sensitivity curves based on the 1998 procedure and the updated curves based on the 2004 revised procedure. The curves may change slightly based on the combination of design inputs for a specific pavement being considered for thin whitetopping, but the curves present the general relationships established for each parameter indicated. Also, since using the procedure to design whitetopping pavements with concrete thicknesses greater than 8 in. is not recommended, the 2004 sensitivity curves are not extended beyond concrete thicknesses of 8 in.

As shown in Figure 7.1, the minimum concrete thickness for the 1998 model was relatively sensitive to lower moduli of subbase/subgrade reaction. Therefore, the 1998 study results indicated that the thin whitetopping design procedure be used only when the modulus of subbase/subgrade reaction exceeds 150 psi/in. This limitation was a concern because 150 psi/in. and below are commonly encountered pavement subgrade support conditions. However, the sensitivity analysis for the 2004 revised design procedure appears to indicate that this issue has been resolved. The 2004 sensitivity curves presented in Figure 7.1 are much less sensitive to subgrade modulus. In addition, the sensitivity curve shapes are more consistent with a general relationship that could be expected between concrete thickness and subgrade support (i.e., relatively non-sensitive at higher concrete thicknesses, and leveling out at lower thicknesses based on an inverse relationship with lower support conditions).

Figure 7.2 presents the minimum concrete thickness sensitivity to asphalt modulus of elasticity. Based on the 1998 study, the required thickness appeared to be fairly sensitive at very low asphalt moduli (50,000 psi), and there appeared to be a minimum asphalt layer thickness of about 5 in. necessary for the design procedure to be valid. Again, this minimum asphalt thickness issue was a concern because 5 in. of asphalt is relatively thick and anticipated as a relatively common thickness for asphalt pavements potentially being considered for whitetopping rehabilitation. However, the 2004 sensitivity analysis appears to also indicate that the minimum asphalt thickness issue has been resolved using the 2004 revised design procedure. The 2004 sensitivity curves presented in Figure 7.2 are still relatively sensitive to subgrade modulus. However, the sensitivity curve shapes are more consistent with a general relationship that could be expected between concrete thickness and asphalt modulus (i.e., relatively non-sensitive at higher concrete thicknesses, and leveling out at lower thicknesses based on an inverse relationship with lower modulus conditions).

Whitetopping thickness sensitivity as a function of the concrete flexural strength and temperature gradient is shown in Figures 7.3 and 7.4, respectively. While the thickness is somewhat sensitive to the flexural strength, it is likely flexural strengths of 650 psi can be specified and achieved for use in whitetopping construction. Thickness is not very sensitive to anticipated concrete temperature gradients as shown in Figure 7.4. These issues were not particularly of concern based on the 1998 design procedure. However, as was the case for subgrade support conditions and asphalt modulus, the 2004 sensitivity curves shapes appear to be more consistent with the relationship that would be expected for flexural strength and temperature gradient (i.e., leveling out at lower thicknesses based on an inverse relationship with lower concrete strength and temperature gradients).

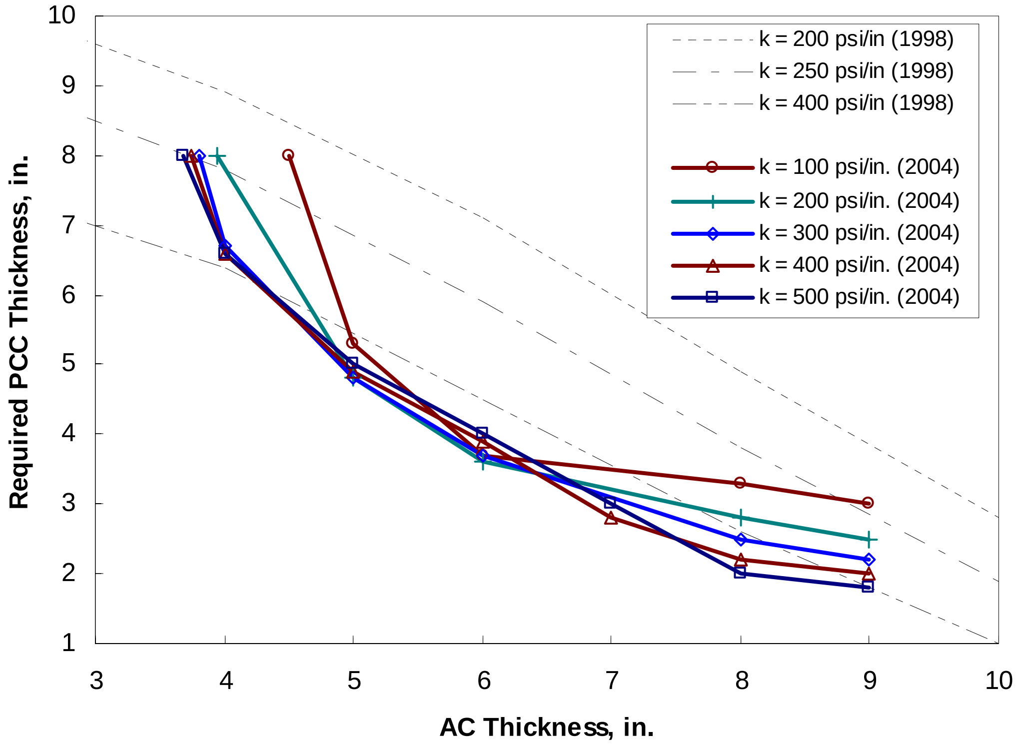

Figure 7.1. PCC Thickness Sensitivity to Modulus of Subbase/Subgrade Reaction

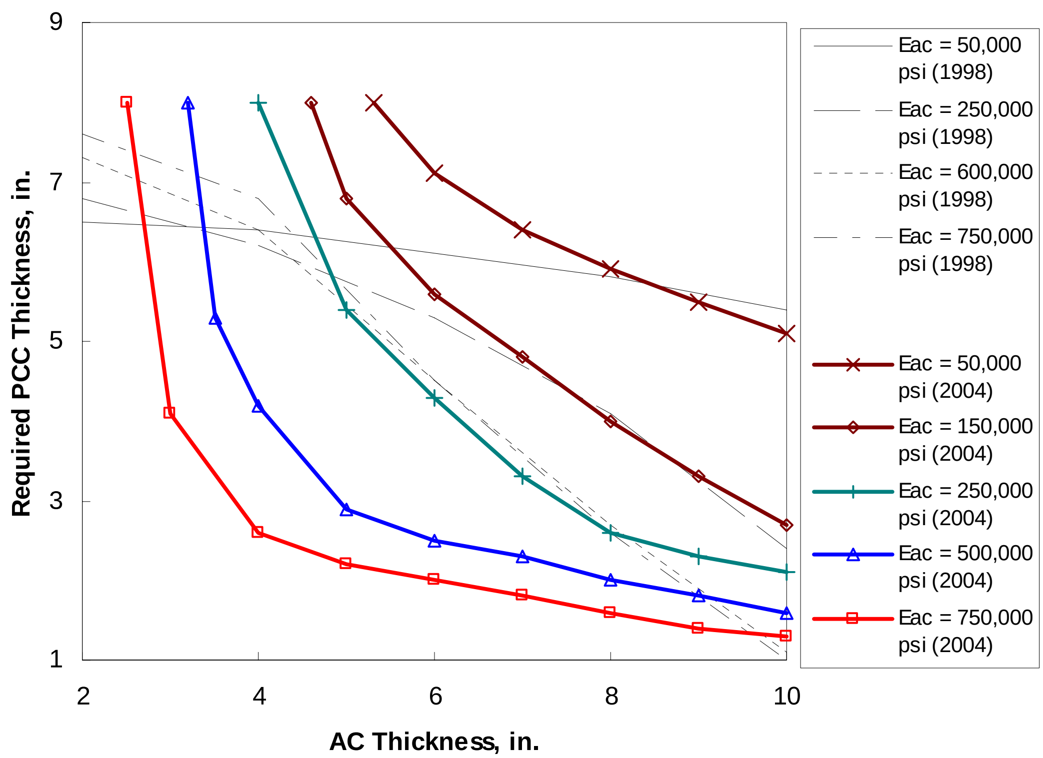

Figure 7.2. PCC Thickness Sensitivity to AC Modulus of Elasticity

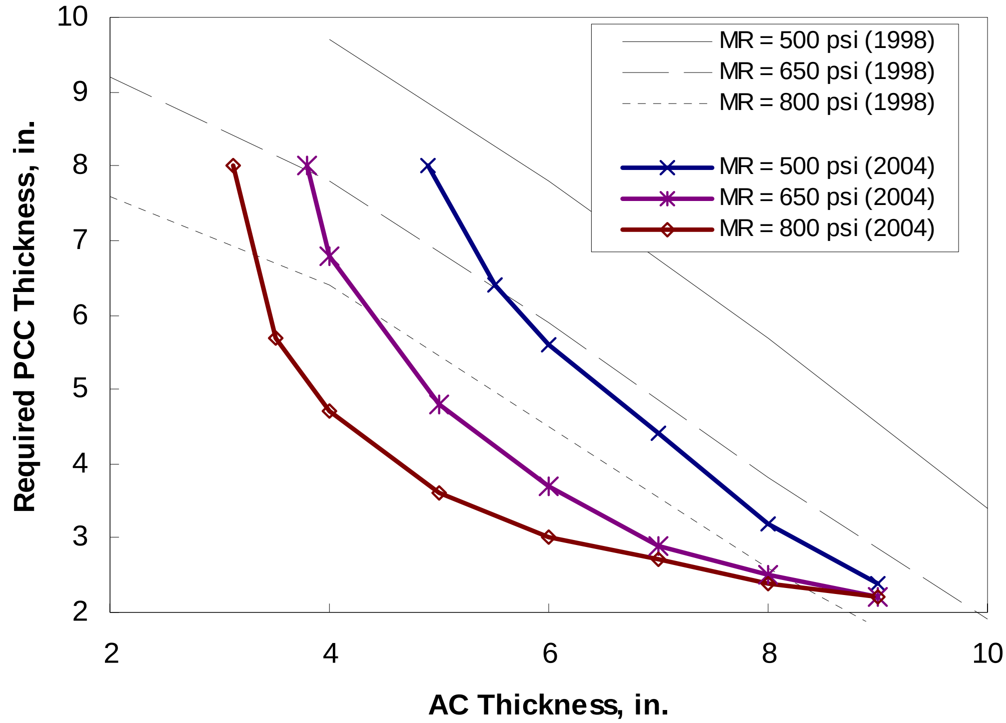

Figure 7.3. PCC Thickness Sensitivity to Concrete Modulus of Rupture

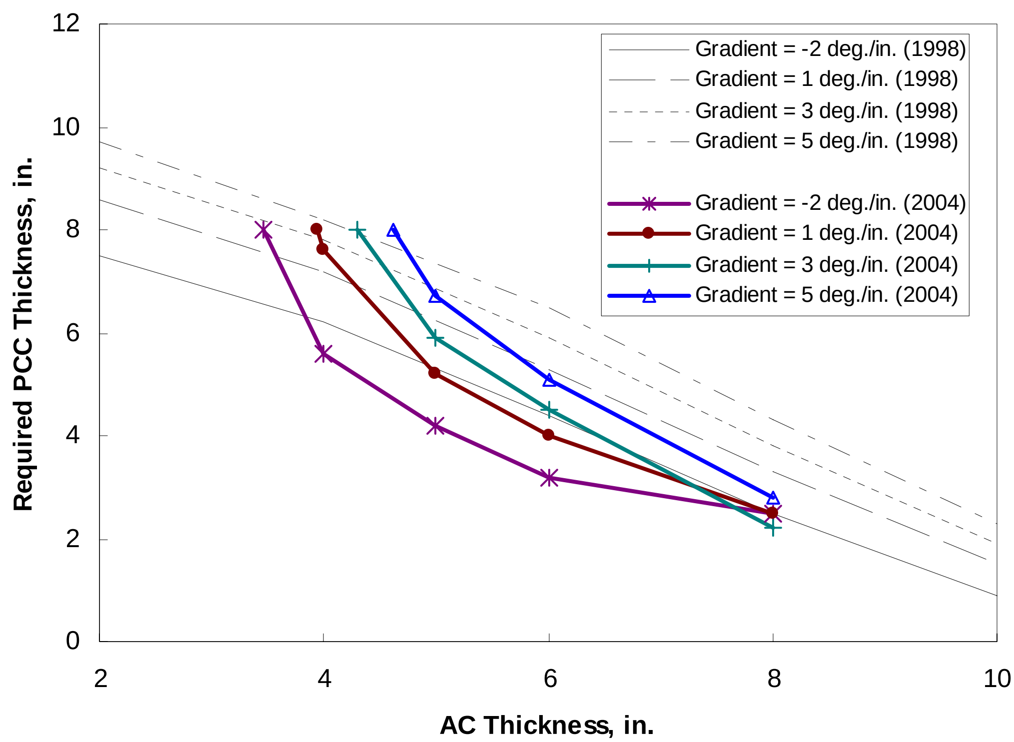

Figure 7.4. PCC Thickness Sensitivity to Temperature Gradient

Whitetopping thickness sensitivity to the expected number of 18-kip ESALs based on asphalt thickness, asphalt modulus and subgrade support conditions are shown in Figures 7.5 to 7.7, respectively. Required concrete thicknesses based on the 1998 design procedure did not appear to be overly sensitive to the number of ESALs above 1 million except under various levels of asphalt modulus of elasticity as shown in Figure 7.6. However, the 2004 revised design procedure appears to be more sensitive to traffic levels for each of the design variables presented in Figures 7.5 to 7.7. This overall relationship was anticipated because thin whitetopping has not particularly been considered a rehabilitation alternative for highway pavements with extremely large volumes of expected ESALs. As also anticipated, the sensitivity of whitetopping thicknesses to traffic levels is a function of the existing physical characteristics of the subgrade and asphalt. This further emphasizes the importance of evaluating and quantifying the existing pavement conditions to get a realistic estimate of required whitetopping thickness using the 2004 revised design procedure.

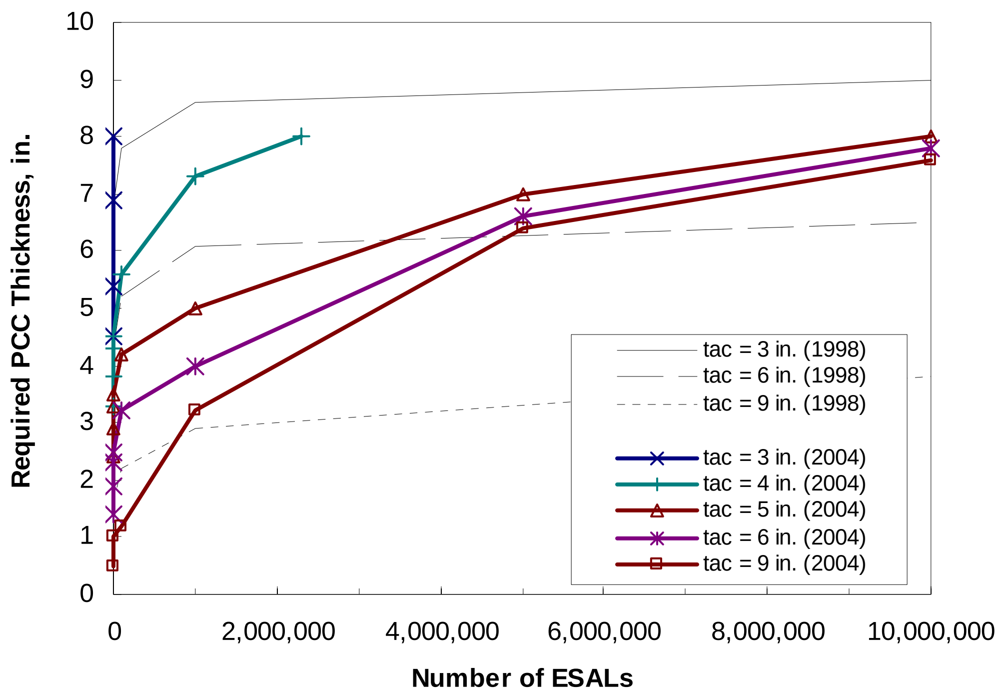

Figure 7.5. PCC Thickness Sensitivity to Asphalt Thickness

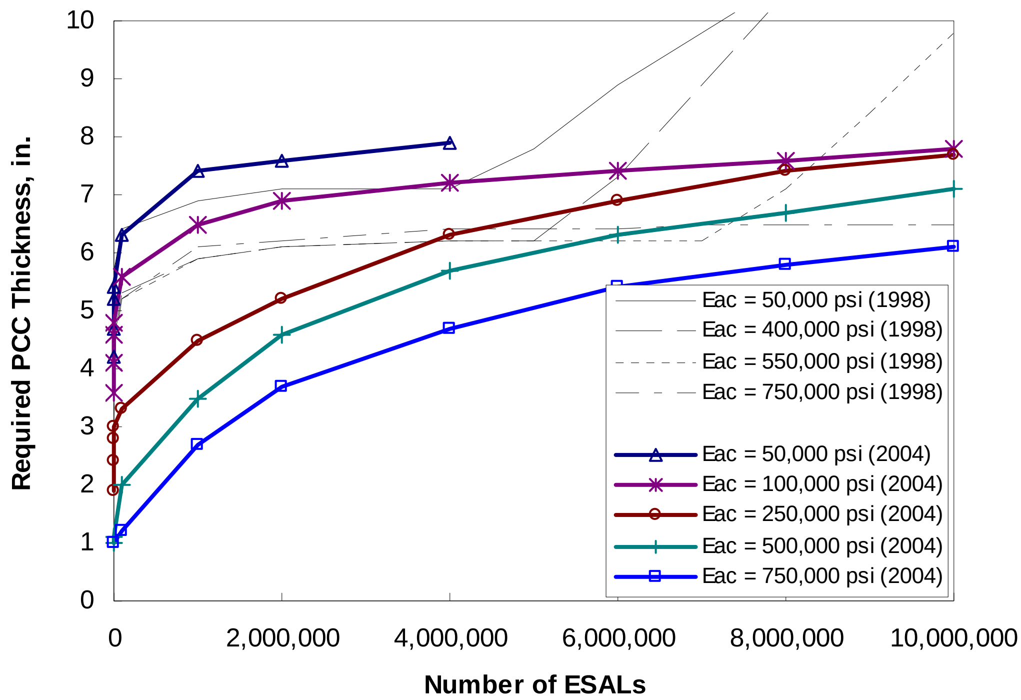

Figure 7.6. PCC Thickness Sensitivity to Asphalt Thickness

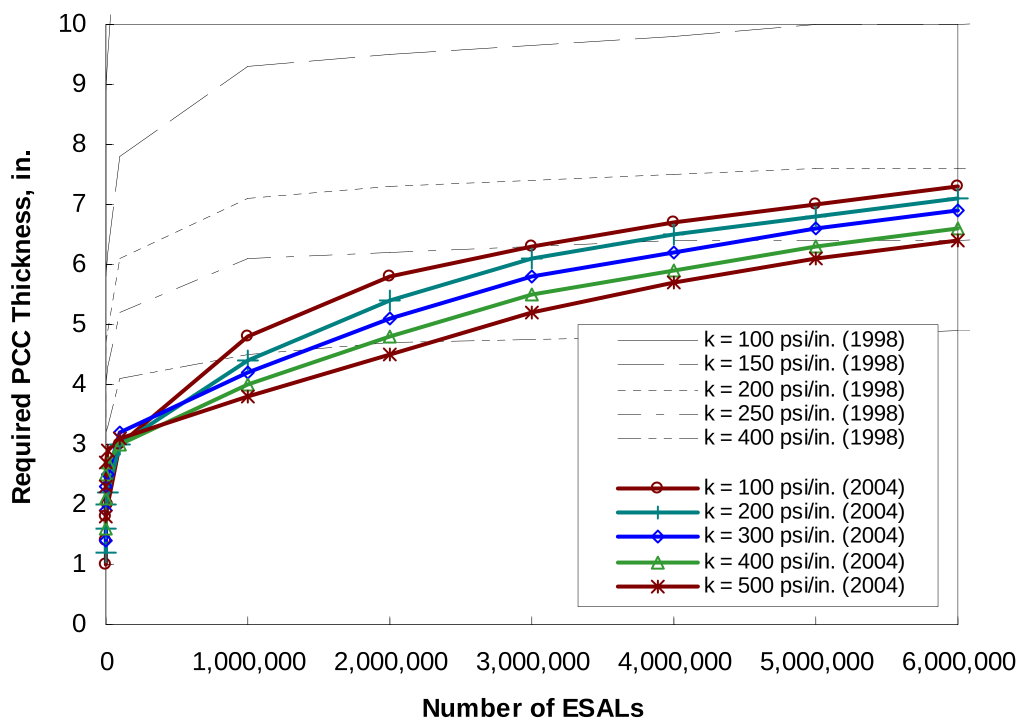

Figure 7.7. PCC Thickness Sensitivity to Modulus of Subbase/Subgrade Reaction

8.0 CONCLUSIONS AND RECOMMENDATIONS

A mechanistic pavement design procedure for thin whitetopping was developed and revised through comprehensive studies involving extensive field load testing and theoretical analysis of whitetopping pavement responses. Two types of pavement failure were considered in the design procedure; portland cement concrete fatigue under joint or corner loading and asphalt concrete fatigue under joint loading. Temperature induced stresses and strains were not included in the design procedure. The mechanistic procedure developed was also modified to incorporate the number of expected Equivalent 18-kip Single Axle Loads (ESALs) currently used by the State of Colorado for the design of concrete pavements.

The methods outlined in this report are intended as a second-generation thin whitetopping design procedure. The design examples presented in Table 5.1 and Figure 6.3 show that the procedures appears to provide reasonable results. However, the design procedure can continue to be refined as more field performance data (especially long-term performance data) become available. Based on the field testing and theoretical analyses conducted during this study, the following conclusions and recommendations can be made:

1. Whitetopping pavements behave as partially bonded systems and should be designed accordingly.

2. A good bond within the concrete/asphalt interface is essential for successful whitetopping performance.

3. For existing asphalt pavement being rehabilitated, the asphalt surface should be milled and well cleaned prior to concrete placement. Milling reduces strain (and corresponding stress) in the whitetopping by approximately 25 percent when asphalt surface milling is performed.

4. For new asphalt pavement being constructed as a whitetopping base, the new asphalt should not be milled prior to concrete placement. The strain (and corresponding stress) in the whitetopping is increased by approximately 50 percent when newly placed asphalt is milled prior to concrete placement.

5. If existing asphalt pavement patching is necessary prior to concrete paving, mill the existing pavement first and perform patching work after the milling has been completed.

6. Due to the partial bonding condition, the tensile stress in the bottom of the concrete layer is approximately 51 percent higher than that of a fully bonded slab system.

7. Due to the partial bonding condition, the tensile strain in the bottom of the asphalt layer is approximately 10 percent lower than that of a fully bonded slab system.

8. The recommended joint spacing for thin whitetopping pavements is 6 ft in both directions. At joint spacings greater than 4 ft, temperature gradients in the concrete layer increase the load-induced tensile stress. An equation was developed to calculate the percent increase in stress due to a temperature gradient.

9. Including dowel bar load transfer devices at transverse contraction control joints does not appear critical to attain satisfactory thin whitetopping pavement performance based on the performance of existing Colorado thin whitetopping test sections. However, load transfer

devices will affect pavement performance if the asphalt deteriorates or the amount of curling in the concrete layer becomes excessive. These are long-term processes that should be monitored.

10. Tied concrete shoulders on thin whitetopping pavements appear to provide substantial stress reducing and performance benefits based on the field testing and analyses performed for this study. Tied concrete shoulders are recommended for thin whitetopping pavements that are expected to carry significant traffic levels.

11. The 2004 revised thin whitetopping design procedure is more sensitive than the original procedure to existing pavement subgrade characteristics, asphalt properties and future traffic volumes. There does not appear to be specific minimum values for subgrade support or asphalt thickness necessary (as was the case for the original study), but required values for these parameters may be dictated by the existing material characteristics and properties of each specific project.

12. Due to the sensitivity of the revised thin whitetopping procedure to design procedure input parameter values, it is critical to evaluate and quantify the existing pavement conditions (i.e., subgrade support, existing asphalt modulus and thickness, remaining asphalt concrete fatigue life, anticipated traffic volumes and distributions, etc.) to get a realistic estimate of required whitetopping thickness using the revised design procedure. It is also critical to comprehensively evaluate these characteristics along the entire pavement project length to account for any non-uniformity in the existing conditions.

13. The 2004 revised thin whitetopping design procedure should be used as a guideline for designing thin whitetopping pavements in Colorado. It should not be used to design ultra-thin whitetopping pavements (concrete thicknesses less than 4 in.) or conventional whitetopping pavements (concrete thicknesses greater than about 7-1/2 to 8 in.). When the design procedure indicates that concrete thicknesses above approximately 7-1/2 inches may be required, it is recommended that the design be crosschecked with conventional concrete pavement and conventional whitetopping design methods.

14. The mechanistic Colorado thin whitetopping design procedure developed in 1998 and revised during this 2004 study is the recommended technique for performing thin whitetopping design calculations. The empirical procedure based on AASHTO ESALs also presented can be used, but because the results of this study are based on actual pavement response data rather than empirical performance data, the mechanistic approach is the preferred design technique.

15. Frequent monitoring of the interface bond strength should also be performed for all test sections. The most recent pavement coring efforts suggest that long-term bond strength may be an issue, and this needs to be further investigated because maintaining bond strength between the concrete and asphalt is critical to long-term performance of the thin whitetopping pavements.

16. Continued long-term monitoring of the test sections must be performed. The long-term performance of the test sections is critical to the eventual verification of the design procedure developed during this study. The design procedure has been verified and revised as much as is currently possible, but the long-term performance of the test sections will ultimately verify whether the design procedure is valid. Any additional information obtained should be incorporated into the design procedure if possible.

17. The design method outlined in this report was developed based on information collected from four thin whitetopping pavement test sites. While an attempt was made to study a range of parameters, it is recommended that additional studies be conducted to further validate the current design procedure.

This report presents information related to instrumentation, construction, testing and analysis of data from thin whitetopping test sections in Colorado. The revised Colorado thin whitetopping pavement design procedure presented provides improved predictions of whitetopping load responses, and therefore should also provide more accurate insights into longer-term performance of thin whitetopping pavements for highway applications. The successful development and revision of a second-generation thin whitetopping design procedure provides an additional level of confidence for designers and highway agencies when considering this rehabilitation technique.

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APPENDIX A – ADDITIONAL THIN WHITETOPPING PAVEMENTS IN COLORADO

APPENDIX A – ADDITIONAL THIN WHITETOPPING PAVEMENTS IN COLORADO

The Colorado Department of Transportation has constructed a number of thin whitetopping pavements during the last 15 years. All of these thin whitetopping pavement sections are considered to be performing exceptionally well. There is also reportedly one 5-year-old ultra-thin whitetopping section that has exceeded its anticipated design life but currently exhibits distress and is scheduled for rehabilitation. Table A.1 lists TWT pavements constructed and some of the characteristics of those sections as provided by CDOT, but the ultra-thin section is not included in the table.

Table A.1. Thin Whitetopping Pavement Sections in Colorado

Year Constructed	CDOT Region	Location	Concrete Thickness, in.	Pavement Quantity, sq. yds.	Unit Cost, $/sq. yd.
1990	4	S.H. 68 (Harmony Rd.) Fort Collins	5	2,333	See Note 1
1994	1	S.H. 83 near Franktown	5	2,637	$18.00
1996	**6**	**U.S. 85 Santa Fe Frontage Road**	**5**	**2,370**	**$31.50**
1996	**4**	**S.H. 119, S. of Longmont**	**4.5, 5 & 6**	**22,300**	**$11.48**
1997	**2**	**U.S. 287, Campo - North and South**	**6**	**25,813**	**$19.43**
1997	1	I-70, Eisenhower Tunnel - West	6	1,335	$35.00
1997	4	S.H. 6, Fleming to East of Haxtun	5.5	186,858	$13.65
1997	1	S.H. 83 (Parker Road) Pine Lane to Arapahoe	5	64,700	$14.00
1997	1	S.H. 40 in Hugo	6	905	$30.00
1999	6	S.H. 83, Rice to Orchard	5	91,614	$16.25
2001	**6**	**S.H. 121: C470 to Parkhill**	**6**	**148,556**	**$20.00**
2001	1	I-70, Eisenhower Tunnel (EJMT Complex)	6	6,934	$40.60
2002	1	S.H. 83, Jamison Ave.	6	97,684	$20.00

Notes:

1. S.H. 68 project expenses were donated by American Concrete Pavement Association (ACPA) and Colorado Ready Mix Concrete Association (CRMCA) members.
2. Projects listed in **bold** text are the test sections instrumented for this study.
3. All information presented was provided by the Colorado Department of Transportation (CDOT).

APPENDIX B – PRE-CONSTRUCTION PAVEMENT EVALUATIONS

APPENDIX B – PRE-CONSTRUCTION PAVEMENT EVALUATIONS

The existing asphalt pavement condition was identified during the 1998 project as critical to the subsequent performance of the thin whitetopping overlay. The first three test pavements were not visited by CTL prior to the thin whitetopping construction, but a pre-construction survey of the S.H. 121 existing asphalt pavement was conducted as part of this study. The evaluation was performed jointly by CTL and the Colorado DOT in April 2001 and included a visual condition survey, rutting measurements, coring and falling weight deflectometer (FWD) testing.

B.1 Visual Condition Survey

Severe fatigue cracking distress in both left and right wheel paths was detected in S.H. 121 Test Sections 1, 2 and the transition area. The presence of potholes was also quite evident throughout the two sections. Figure B.1 shows the typical fatigue cracking for these two test sections. However, as presented in Figure B.2, distresses in Test Sections 3 and 4 were minor and were in the form of longitudinal cracking near the pavement centerline.

Although severe distress was observed in Test Sections 1 and 2, it appeared that a large portion of the distressed surface material was removed through milling nominally ½ to 2 in. of asphalt during construction.

B.2 Rutting Measurements

Rut-depth measurements were taken at 50-foot intervals within the S.H. 121 test sections in the left-wheel path (LWP) and in the right-wheel-path (RWP) for both inside and outside lanes, as presented in Figure B.3. The measured rutting was considered in the low range, with the average ranging from 1/8 in. to 3/8 in for the four test sections, and was essentially eliminated during the asphalt milling efforts. Table B.1 shows the average rut-depth for the four test sections.

B.3 Pavement Coring

Twelve asphalt pavement cores were drilled at 50-foot intervals through each of the four S.H. 121 test sections. In each test section, the first and third cores were taken in the driving lane right wheel path and the second core taken in the middle of the lane. Cores were used to verify the asphalt pavement thickness in all four sections. As shown in Table B.2, the existing thickness of the first and the second test sections ranged from 5-½ to 6 inches, and the existing thickness of the third and the fourth test sections ranged from 6-½ to 8 inches.

Figure B.1. Fatigue Cracking Observed in Test Sections 1 and 2

Figure B.2. Typical Conditions Observed in Test Sections 3 and 4

Figure B.3. Rutting Measurements on Existing Asphalt Pavement

Table B.1. Average Rut Depth of the Existing Asphalt Pavement

Test Section	Measture Rut-Depth, in.			
	Traffic Lane		Passing Lane	
	RWP	LWP	RWP	LWP
1	3/8	3/8	1/8	3/8
2	3/8	3/8	1/8	3/8
3	1/8	3/8	1/8	2/8
4	3/8	3/8	3/8	1/8

B.4 Falling Weight Deflectometer Testing

Falling weight deflectometer (FWD) tests were conducted on the four S.H. 121 test sections prior to milling or construction on April 24, 2001. Tests were performed at 20-ft intervals within the test sections, with three drops conducted at each location.

Table B.2. Existing Asphalt Pavement Core Thickness

Test Location, ft	Asphalt Layer Thickness, in.			
	Test Section			
	1	2	3	4
50	6.0	5.8	7.5	7.5
100	6.0	6.0	8.0	7.0
150	5.8	5.5	7.3	6.5
Average	5.9	5.8	7.6	7.0

The average 9000 lb load center plate deflections are 13.19, 15.14, 13.20, and 13.82 mils for Test Sections 1, 2, 3, and 4, respectively. The deflection data were also used to backcalculate the pavement layer moduli. From construction records, the existing asphalt pavement structure consisted of the AC layer, a CDOT Class 6 aggregate base of 4 in. and a Class 1 aggregate subbase of 10 in. The pavement was treated as a two-layer system, an AC layer and a foundation, in the backcalculation process. The backcalculated pavement layer moduli for the four test sections are summarized in Table B.3. The asphalt elastic modulus used in the design was reportedly 266,600 psi.

Table B.3. Summary of the Estimated Layer Moduli of the Existing Asphalt Pavement

Back-Calculated Layer Moduli, psi	Test Section							
	1		2		3		4	
	AC	Base	AC	Base	AC	Base	AC	Base
Maximum	596,400	24,200	466,400	26,100	479,200	24,200	686,300	22,100
Minimum	260,500	20,900	164,000	18,600	237,500	17,200	205,800	15,100
Average	398,700	22,500	288,600	21,800	334,600	19,100	394,200	18,500
Standard Deviation, psi	88,900	1,100	91,300	2,700	64,900	2,100	151,200	2,100
Coefficient of Variation, %	22	5	32	12	19	11	38	11

APPENDIX C – GENERAL CONSTRUCTION APPROACHES

APPENDIX C – GENERAL CONSTRUCTION APPROACHES

The S.H. 121 thin whitetopping pavement was constructed in the summer of 2001, and Interstate Highway Construction, Inc. (IHC) from Denver, Colorado was the paving contractor. The general approach for the construction of the S.H. 121 test sections was in accordance with conventional slip-form paving operations where the driving lanes and shoulders are placed monolithically. In contrast, the S.H. 119 and U.S. 287 sites were paved one lane at a time and the U.S. 85 section utilized the pre-placed concrete curb and gutter as a form on each side of the pavement.

C.1 Asphalt Milling and Surface Preparation

The existing asphalt surface was cold milled by IHC on June 15 and 16, 2001. The asphalt milling removed ½ to 2 in. of the asphalt concrete to create a surface that would promote enhanced interface bonding between the concrete and the asphalt layers. Previous studies [1,2] have indicated that cold milling the existing asphalt surface promotes a stronger mechanical interface bond between the two layers and promotes the formation of a composite pavement section to carry load induced stresses. The milled asphalt surface was also swept multiple times, air blasted to remove any remaining debris or dust, and wetted prior to concrete placement. Each of these tasks was performed to provide a clean asphalt surface and promote mechanical bond at the interface between the asphalt and new concrete overlay. Figure C.1 shows the rough asphalt surface after milling and cleaning.

The test sections were located in the southbound lanes at the north end of the project, and a 15-ft-long, full pavement width area of asphalt at the very beginning of the 4-in.-thick test sections (the north end of the paving operation where the 4-in.-thick Test Sections 1 and 2 were located) was milled 2 inches deeper than the remaining areas of the pavement. This additional milling provided a thicker (6 in. thick) area at the beginning of the TWT where the pavement transitions from existing asphalt to new thin whitetopping concrete. Past experiences have indicated that this is often an area susceptible to increased amounts of panel cracking and deterioration, and that constructing a thickened area at this location would help to eliminate the occurrence of cracking and distress.

Figure C.1. Milled Surface of Existing Asphalt Pavement

C.2 Concrete Mix Design

The concrete mixture used for the S.H. 121 TWT overlay was reportedly typical for a slip-form paving mixture used in Colorado, with the exception that it included fiber reinforcement. The specified compressive strength for the mixture was 4,200 psi at 28-days. The concrete supplier was also Interstate Highway Construction located in Denver, Colorado. Table C.1 presents the concrete mixture proportions provided to CTL.

Table C.1. Concrete Mix Design

Cement	585 lb
Fly Ash (Class F)	113 lb
Coarse Aggregate	1,614 lb
Sand	1,320
AEA	2.5 oz
Water	264 lb
Polypropylene Fiber	3 lb

Note: Based on one cubic yard SSD Batch Weight

C.3 Concrete Paving

The S.H. 121 thin whitetopping test sections were paved on June 22, 2001. Paving began at the north end of the project southbound lanes where the thickened approach started at approximately Station 196+25. The 4-in.-thick test sections were paved immediately following the thickened approach, a 200-ft transition area followed, and all remaining pavement was designed to be nominally 6 in. thick.

Paving started at approximately 6:30 A.M. on June 22, 2001. The paver started at approximately Station 196+25 and paved the southbound lanes in the direction of traffic. The first and second test sections (4-in.-thick test slabs) were paved at approximately 6:45 A.M. and 7:15 A.M., respectively. Test Sections 3 and 4 (6-in.-thick test sections) were paved starting at approximately 8:30 A.M. and were finished by 9:15 A.M. A photograph of general thin whitetopping paving operation is presented in Figure C.2.

Dowel bars were not used in transverse control joints in the S.H. 121 TWT pavement construction. However, tie bars were placed at 30 inches on-center along all longitudinal contraction joints. The paver was equipped with an Automatic Tie Bar Inserter and placed all tie bars automatically, except for Test Section 1 where tie bars were placed manually using tie bar chairs fastened to the asphalt. Chairs were necessary in Section 1 because of the 4 ft by 4 ft joint spacing; all other test sections had a 6-ft spacing between longitudinal joints matching the Automatic Tie Bar Inserter settings. The chairs and tie bar inserter were set so the tie bars would be at the mid-depth of the concrete slabs.

The test slab locations in each test section were marked to prevent concrete trucks from damaging the instrumentation as they were backing in to deliver concrete. As the paver approached each set of test slabs, concrete was placed by hand around the embedded strain gages and thermocouples to ensure proper consolidation around the instrumentation and to reduce the possibility that the gages would be damaged by the paver passing over the sensor locations. The instrumentation could be damaged if the paver was set low enough to reach the gages or if a large amount of concrete was being pushed ahead of the paver as it passed the instrumentation locations. Since this section of pavement passes through densely populated areas, traffic noise was a major concern. To minimize traffic noise, final surface texture was provided by Astroturf drag.

Figure C.2. General View of the Thin Whitetopping Paving Operation

C.4 Transverse and Longitudinal Control Joint Sawing

Once the concrete gained sufficient strength to support people walking on the surface, the locations of the gages and test slab joints were identified and marked using reference points established prior to paving. The joint sawing subcontractor marked out the remaining control joints prior to initiating sawing activities.

The transverse joints were sawed prior to the longitudinal joints. The joint sawcutting subcontractor performed trial sawcuts at the beginning of the paving to determine when the concrete had gained sufficient strength to allow for sawcutting without raveling of the sawcut edges. The transverse sawing started at approximately 2:00 P.M. on June 22, 2001, about 7½ hrs after paving. Two self-propelled conventional saws were used to perform the transverse sawcuts, as shown in Figure C.3. Soffcut saws were on site but only used for the 4-ft by 4-fy test sections.

A train of walk-behind and self-propelled diamond blade concrete saws spaced using a guide bar ahead of the saws was used to cut the 6-ft longitudinal joints. This approach was utilized because the subcontractor felt the assembly was the most efficient approach to maintain the proper spacing between saws and make straight and evenly spaced longitudinal sawcuts. A photograph of the longitudinal sawing operation is presented in Figure C.4, and the finished pavement surface is shown in Figure C.5.

Figure C.3. Sawing Transverse Control Joints

Figure C.4. Sawing Longitudinal Control Joints

Figure C.5. Pavement Surface After Joint Sawing

APPENDIX D – TEST SECTION INSTRUMENTATION

APPENDIX D – TEST SECTION INSTRUMENTATION

As discussed in previous sections of this report, the two primary variables to be evaluated in the study were the slab thickness (two levels) and panel joint spacing (two levels for each thickness), resulting in four different combinations. Two replicate slabs were instrumented for each test section, resulting in eight total slabs. Three replicate slabs per test section were originally planned, but the surface gage instrumentation tasks associated with three test slabs proved to be more extensive than the available traffic lance closure times would permit. However, the embedded gages for all three replicate slabs per section were still installed prior to paving to provide redundancy if the paving operation damaged any of the installations. The overall purpose of the replicate installations was to obtain average data from replicate slabs to more accurately represent the responses of the slabs in the test sections. The following testing on the test sections was planned:

- Static load testing with strain measurements
- Surface profile measurements over daily temperature variations
- Joint opening measurements
- Temperature measurements
- Pavement coring and laboratory testing
- Ground penetrating radar testing for thickness estimation
- FWD tests.

Two sets of field tests were performed; the first set was conducted about 28 days after TWT pavement construction and the second two years after construction. Performing these 28-day and two-year load tests allows for the inclusion of the test section responses following the pavement being exposed to extended traffic repetitions freeze/thaw cycles.

In order to perform the field testing planned at the S.H. 121 site, instrumentation for each test section included the following:

- Embedded concrete strain gages
- Surface concrete strain gages
- Embedded thermocouples
- Retrofitted temperature sensors
- Reference rods
- Whitmore plugs.

A portion of the instrumentation required for this project had to be installed prior to construction of the TWT concrete overlay. This included the embedded concrete strain gages, reference rods and embedded thermocouples. Others were installed just prior to load testing activities, such as the surface strain gages and temporary temperature sensors.

The test slab locations were in the outside wheelpath of the traffic lane in all test sections. The specific slabs selected were near the center of each 200-ft-long test section. Each of the section test slabs was separated in the longitudinal direction from the following test slab by at least two concrete panels. Figure D.1 presents the typical layout of test slabs within each test section.

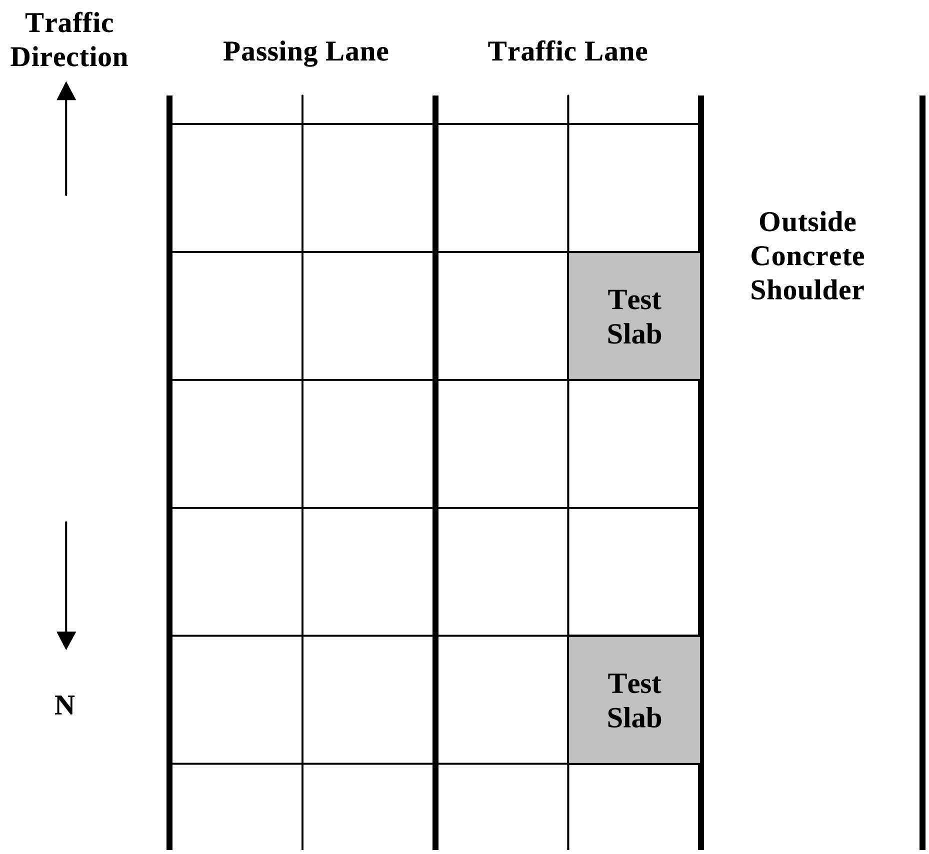

Figure D.1. Typical Layout of Test Slabs within Each Test Section

D.1 Embedded Strain Gages

The typical layout of the strain gages for the four S.H. 121 test sections is shown in Figure D.2, but this basic configuration is typical for all four test pavements included in the study. In general, gages were placed at the slab center, along longitudinal joints adjacent to the concrete shoulder, along longitudinal joints on the concrete shoulder, and the transverse joint center. Also, as shown in Figure D.3, multiple gages were used at designated locations. These multiple gages were installed on the concrete slab surface, 1 in. above the existing asphalt surface, and on the asphalt surface. There were six embedded strain gages on each test slab for a total of 48 for the S.H. 121 project.

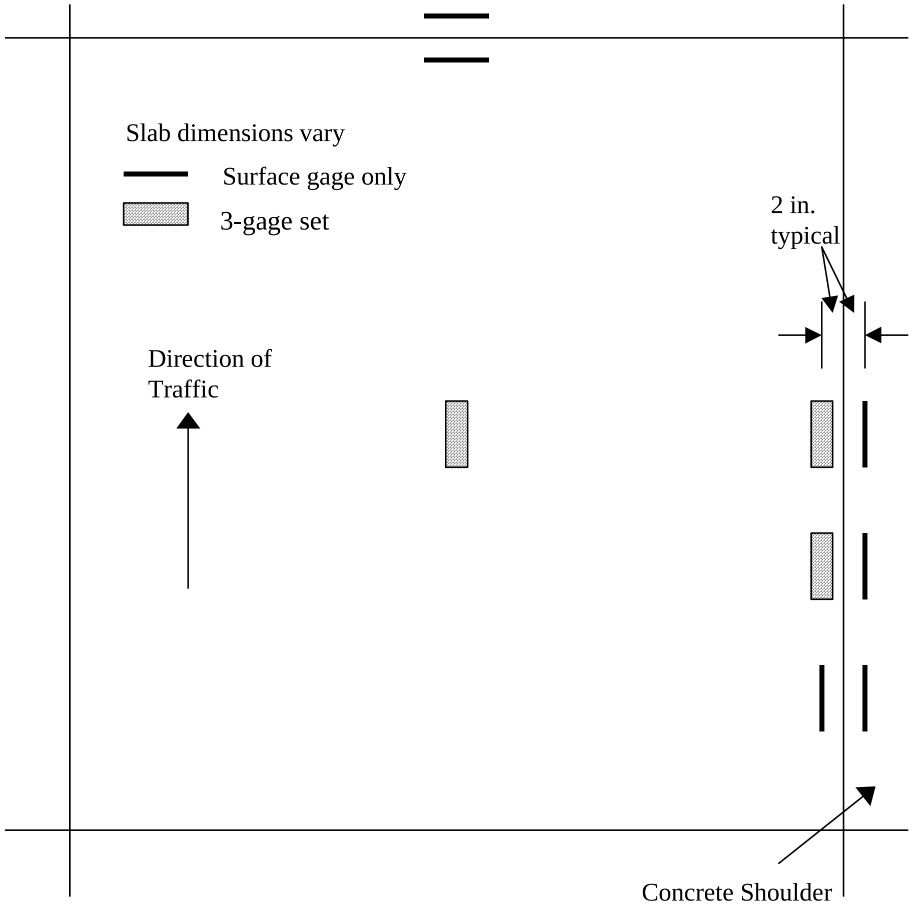

Figure D.2. Typical Test Slab Strain Gage Layout – Plan View

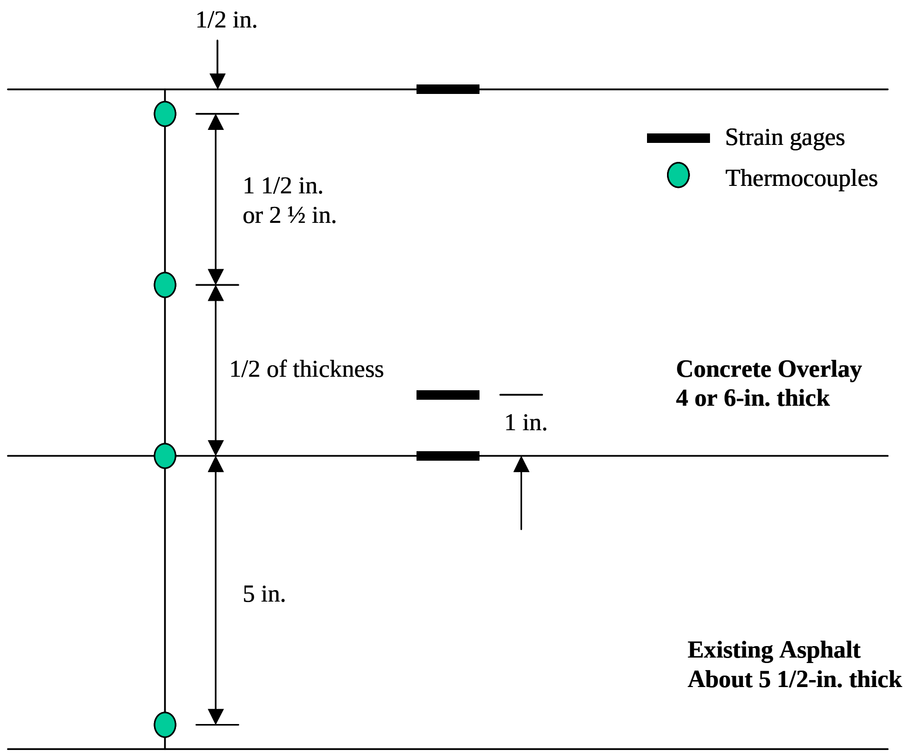

Figure D.3. Typical Strain Gage and Thermocouple Layout – Section View

The embedded strain gages were fabricated and tested for stability in the CTL laboratory prior to arriving at the project site. They consisted of ½-in.-long gages epoxied to the prepared, smooth surface of No. 3 steel bars. The gages installed at the concrete-asphalt interface were mounted on 12-in.-long bars and the embedded gages located at one inch above the asphalt-concrete interface gages were mounted on 16-in.-long bars. The following is the sequence of the installation process:

- **Identification and Marking of Test Slab and Gage Locations** – The location of each test slab and gage was identified using the edge of the outside concrete shoulder and pavement centerline, which were provided by CDOT representatives. Also, after marking all test slabs and gage locations, the locations of all gages and joints were triangulated out to multiple reference points outside the roadway so the gage and joint locations could be accurately re-established following concrete paving.

- **Asphalt Surface Preparation** – To enhance bonding between the concrete overlay and the existing asphalt, the asphalt surface was milled, resulting in rough surfaces. Therefore, a diamond grinder was used to cut grooves in the asphalt surface for installing the interface gages. Holes were also drilled into the asphalt layer and were used to anchor the concrete embedment gage chairs.

- **Gage Installation** – The grooves in the asphalt were cleaned with acetones and the interface gages were then epoxied into the prepared grooves. For installation of the concrete gages one inch above the interface, chairs were made from threaded rods and the gages were attached to the chairs. The concrete gages were positioned directly above the interface gages and the one-inch spacing between the interface and embedded gages was maintained, as shown in Figure D.4. Lead wires connected to the gages were recessed into the asphalt and run to the edge of the pavements to protect them from the construction vehicles. The lead wires were individually labeled at the end for identification purpose and were buried at the pavement edge to further protect them during construction activities. All installed gages were then checked for functionality.

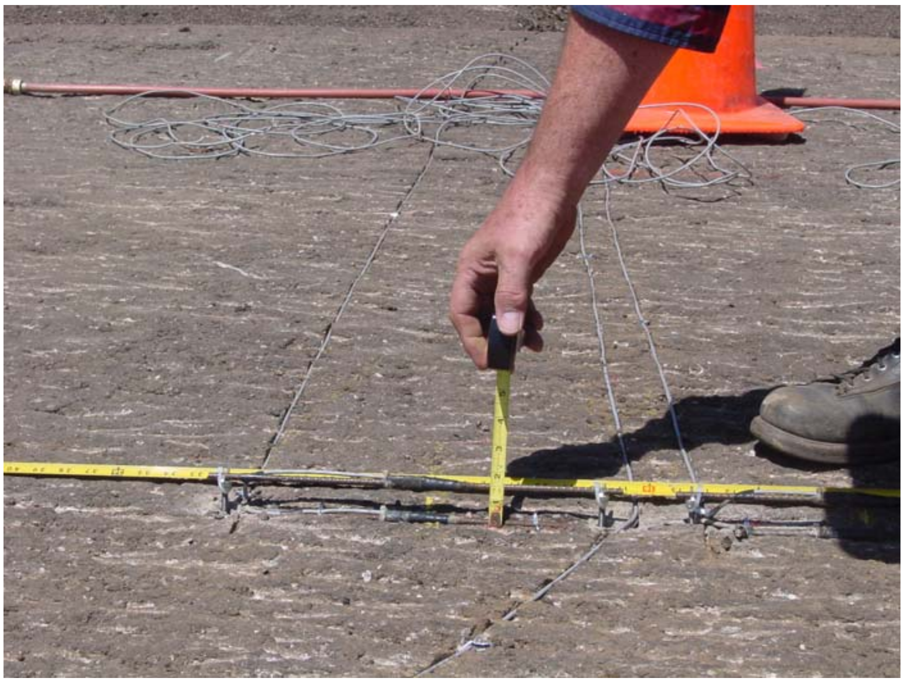

Figure D.4. Typical Embedded Strain Gage Installation

D.2 Reference Rods

To serve as a fixed elevation reference for thin whitetopping slab surface profile measurements, one 6-ft-long steel reference rod was installed in each S.H. 121 test section. The reference rods were located on the concrete shoulder adjacent to the longitudinal joint between the traffic lane and the shoulder.

To install the reference rods, cores were drilled through the asphalt layer, and the rods were installed in the empty core hole locations by first pounding a steel pipe approximately 4 ft long into the ground. Through the pipe, the steel reference rod was then driven into the ground about two feet beyond the depth of the protective pipe. A machined cap was screwed to the top of the reference rod to provide a consistent surface for the elevation measurement instrument to rest on when collecting slab deformation measurements. A protective polyvinyl chloride (PVC) pipe assembly was used to protect the portion of the reference rod assembly above the asphalt grade from the TWT concrete and concrete paver. This type of installation was utilized to minimize the affect of frost movement during the winter. Figure D.5 shows the installed reference rod with the protective PVC assembly. The reference rod PVC pipe sleeves were covered by approximately 3/8 in. of concrete following paving, so following the completion of the control joint sawing, the locations of the reference rods were identified and the PVC protective sleeves were exposed.

Figure D.5. Reference Rod Assembly Installation

D.3 Embedded Thermocouples

Thermocouples were installed at different depths in the S.H. 121 concrete and asphalt layers (see Figure D.3) to monitor the temperature gradients through the pavement section during load testing activities. Two test slabs were instrumented, one in the 4-in. and one in the 6-in. thick test sections. Prior to concrete construction, embedded thermocouples located five inches into the asphalt layer and at the asphalt-concrete interface were installed. The remaining thermocouples were installed just before load testing by drilling ½ in holes in the concrete and placing a small amount of mineral oil in the hole bottoms. A thermocouple wire was placed in the mineral oil and the temperatures from all thermocouple wires were recorded during load testing activites.

D.4 Whitmore Plugs

Also included in the testing plan was the installation of Whitmore plugs at different locations across both transverse and longitudinal joints. These plugs were installed to help determine if the contraction control joints were cracked and working as designed. Ten plugs were installed in one test slab from each of the four experimental test sections, and Measurements were collected between the plugs using a digital caliper. Typical installation layouts for Whitmore plugs are shown in Figure D.6.

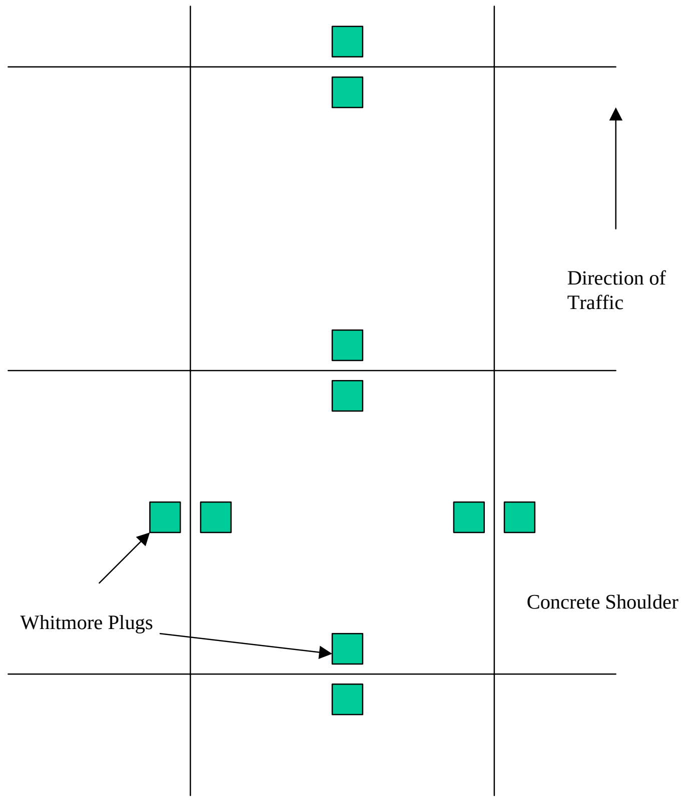

Figure D.6. Typical Whitmore Plug Positions for Each Test Section

D.5 Surface Strain Gages

Surface gages were installed just prior to the load testing activities, or approximately 28-days after the pavement construction. Tokyo Sokki PL-120-11 strain gages were used. Nine surface gages were installed on each test slab, and the typical layout of the surface gages is presented in Figure D.2. The surface gages were placed directly over the embedded gages in locations indicated in Figure D.2. Gages near the joints were typically two inches from the contraction control joints as also indicated in Figure D.2.

Instrumenting three test slabs per test section, or 12 slabs total, was originally proposed. The twelve slabs were instrumented with embedded gages during pavement construction. However, because of the traffic volumes on this roadway and local regulation, the pavement section could only be closed between 8:30 A.M. and 3:30 P.M. for the follow up instrumentation. This time restriction would not allow for installation of surface gages and load testing of the 12 slabs. After discussion with and permission from Mr. Ahmad Ardani, the Colorado DOT project manager for this project, only two slabs were load tested for each combination (for a total of 8 slabs). It was felt that this would still provide sufficient data for analysis purposes.

The installation of the surface gages included the following:

- Cutting recessed slots into the concrete surface at each strain gage location
- Cleaning the recessed slots using Acetones
- Attaching gages to the recessed slots using fast-setting epoxy
- Cutting grooves to the control joint locations for running the lead wires to the pavement edge
- Soldering leads to the installed gages
- Recessing the leads and running the leads to the pavement edge where the embedded gages were located
- Checking installed gage functionality
- Applying hot wax over the gage and solder connections to protect them from moisture intrusion during the testing period.

In this project, the embedded and surface strain gages were used to measure strains induced by static truckloads placed on the pavement surface in selected locations. A typical test slab with installed surface gages is shown in Figure D.7.

Figure D.7. Typical Test Slab with Surface Gages Installed

D.6 Instrumentation and Testing of the Original Test Sections

The construction, instrumentation and testing approaches discussed for the S.H. 121 site are based on and consistent with the approaches utilized at the original test sites. The following paragraphs briefly describe the instrumentation and testing efforts performed at the original sites during the first thin whitetopping study.

The three test slabs at the U.S. 85 frontage road project were partially instrumented before the concrete pavement construction. Each test slab was instrumented with 12 strain gages. Three sets of two prepared embedment gages were installed, one on top of asphalt surface and the other in the concrete 1/2 in. above the asphalt surface. These gages were located at the longitudinal edges and center of the slab along the transverse centerline. For each slab, a free edge joint and a tied joint were instrumented. Surface gages were also installed before load testing, including one on top of each of the three sets of embedment gages and three gages along one corner diagonal line. Load testing on the U.S. 85 frontage road project was conducted in August 1996 and August 1997.

Each S.H. 119 site test slab was instrumented with eight strain gages. Since the slabs at the site did not include a free edge, sets of two prepared gages were installed at one tied longitudinal edge and at the slab centers. The vertical gage locations were identical to the locations at the U.S. 85 frontage road site (one on top of asphalt surface and the other in the concrete 1/2 in. above the asphalt surface). Surface gages were also installed directly above the embedment gage locations. Load testing was performed at the S.H. 119 test site in September 1996 and August 1997.

The U.S. 287 project near Lamar only included surface gages and did not include embedment gages. Two of the four surface gage locations were identical to the gage locations at the Longmont site (along the longitudinal joint and at the slab center). An attempt was made to investigate a maximum surface tensile stress due to a corner loading by installing two additional surface gages along a longitudinal and transverse joint 18 in. from the corner. Load testing was performed at the U.S. 287 test site in September 1997.

Thermocouple trees were installed before concrete pavement construction at all original test sections to monitor temperature gradients during load testing. Reference rods were also installed and dipstick profile measurements collected during load testing activities. Static load testing with a 20-kip single axle load was conducted several times throughout the day at all the original test sections.