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GUIDELINES FOR THE THICKNESS DESIGN OF BONDED WHITETOPPING PAVEMENT IN THE STATE OF COLORADO

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13. ABSTRACT (Maximum 200 words)

This report summarizes the development of procedures for the thickness design of bonded whitetopping pavements in the state of Colorado. Included in this report is an overview of the selected sites, design parameters, instrumentation, data acquisition and analysis. The parameters studied included joint spacing, concrete flexural strength, asphalt thickness, subgrade modulus of elasticity, concrete and asphalt modulus of elasticity, design equivalent single load application (ESLA) and temperature gradient.

Equations were developed to predict the critical stresses and asphalt strains. A mechanistic design procedure is described which allows the evaluation of trial whitetoping thickness and joint spacing. A modified procedure was also developed incorporating an empirical approach based on the number of ESLA. Based on this research a minimum subgrade modulus of 150 pci is required along with an asphalt thickness of 5 inch. As with AASHTO procedure, the method is not too sensitive to the number of ESLA.

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EXECUTIVE SUMMARY

This report summarizes the development of a procedure to be used for the design of whitetopping pavements. 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 (4 in. concrete or less) overlays require closely spaced joints (48 in. or less) and may not be practical for rehabilitating high volume traffic roadways as targeted by the State of Colorado. Slightly thicker concrete sections (5 to 7 in.) and wider joint spacings (up to 12 ft) were studied in this research project.

Three sites were evaluated in this study – a 1,000-ft-long frontage road on Santa Fe Drive near Denver constructed in May 1996, a one-mile-long section of State Road 119 in Longmont constructed in August 1996, and a three-mile-long section of US Rt. 287 near Lamar constructed in July, 1997. A total of 11 slabs were instrumented with strain gages. In general, gages were located at the center of the slab and along the longitudinal joint. Some slabs were instrumented with diagonal corner and transverse joint wheelpath gages. Typically, embedment strain gages were installed during construction at the asphalt/concrete interface and ½ in. above the interface in the concrete layer. Surface gages were installed prior to each load testing event.

The Santa Fe and Longmont sites were load tested at about 28 days and 1 year while the Lamar site was load tested at 28 days only. For load testing, CDOT trucks were loaded to provide a 20-kip single axle load. The load was applied statically at the strain gage locations (shown in appendix photos) and strain data was recorded at all depths of the pavement. To evaluate the effects of temperature and related curling, load tests were performed several times throughout the day for each load testing event. Thermocouples were installed at multiple depths in the pavement and monitored throughout each load testing event. Surface profile measurements were performed at several intervals during load testing by CDOT personnel. Profile data was compared to baseline measurements recorded shortly after slab construction to evaluate the degree of slab response to changing temperature gradients. CDOT personnel cast concrete cylinders and beams during construction. The samples were tested at ages corresponding to the load testing events so that strength parameters of the concrete were known. Cores were removed from the pavement to evaluate the shear strength between the layers as well as the asphalt properties.

The effect of surface preparation during construction was studied by including slabs constructed on milled and unmilled asphalt. Also, the effect of milling newly placed (repaired) or existing asphalt was evaluated. It was found that existing asphalt pavement should be milled and cleaned prior to concrete placement for an overall reduction of 25 percent in the critical load-induced stresses. However, new asphalt, such as that placed in repair patches, should not be milled prior to concrete placement to avoid a 50 percent increase in critical load-induced stresses.

Measured load-induced stresses at zero temperature gradient were compared to theoretical stresses calculated using the finite element program ILSL2. It was found during the PCA research that whitetopping pavements are partially bonded and cannot be directly modeled as fully-bonded or fully-unbonded sections. Therefore, since ILSL2 cannot model partially bonded layers, a correction coefficient was developed to equate theoretical (fully-bonded) stresses to measured stresses. A coefficient of 1.65 was calculated at the 95 percent confidence level. At the discretion of an experienced design engineer, the coefficient can be reduced based on the required reliability of the project.

A similar analysis was used to calculate the critical asphalt strain due to a partial, as opposed to full, bond. If the layers were fully bonded, the strain at the bottom of the concrete would be the same as the strain at the top of the asphalt. However, measured strains indicate that a coefficient of 0.842 is required to reduce the asphalt strains due to the partial bond between the layers.

The effect of temperature gradient was incorporated into the design procedure by comparing measured load- and temperature-induced stresses to theoretical stresses (converted using above coefficients) calculated at zero temperature gradient. If temperature does not effect whitetopping pavement, load-induced stresses would be equal throughout the day. However, it was found that the stress increases by a percentage equal to 4.56 multiplied by the temperature gradient (°F/in.). This stress increase is due to a loss of support at slab edges when curling occurs.

Equations were developed to predict the critical concrete stresses and asphalt strains. A mechanistic design procedure is described which allows the evaluation of trial whitetopping thicknesses and joint spacings. The procedure computes the concrete and asphalt fatigue life for specific material properties. Iterations are required to determine the appropriate parameters which provide the required design life for both concrete and asphalt layers. A modified procedure was also developed incorporating an empirical approach based on equivalent single axle loads (ESALs).

Sensitivity analyses provided characteristics of whitetopping. Based on this research, a minimum subgrade modulus of 150 pci may be required for some whitetopping. Also, an asphalt thickness of 5 in. is recommended. And finally, as with the AASHTO procedure, the method is not too sensitive to the number of ESALs. However, these conclusions need to be verified by future work and long-term performance of test sections.

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 [1]. The critical stress and strain prediction equations developed during this research are part of a first-generation design procedure which will be verified and/or modified with the collection of additional data from future research projects.

GUIDELINES FOR THE THICKNESS DESIGN OF BONDED WHITETOPPING PAVEMENT IN THE STATE OF COLORADO

by

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1.0 INTRODUCTION

Resurfacing existing asphalt concrete (AC) pavement with portland cement concrete (PCC) is not a new concept. The use of whitetopping for rehabilitating deteriorated asphalt pavements dates back to 1918. Whitetopping technology, however, has improved over the years as the concrete paving technology itself has improved.

There are several advantages to resurfacing asphalt pavements with portland cement concrete (whitetopping). Whitetopping can provide long-term benefits to the traveling public as well as the highway or airport agency. It significantly reduces time and delays accompanying the frequent maintenance of an asphalt surface. The proven durability and long-term performance of a PCC surface decreases the maintenance time and life cycle cost of the pavement. These advantages, in addition to the improvement in skid resistance and safety (especially under wet conditions), compare favorably to asphalt surfaces.

Design and construction procedures of whitetopping are well-established and explained in detail in Portland Cement Association (PCA) and American Concrete Pavement Association (ACPA) publications[1-3]. Features including minimum slab thickness, support characterization, and pre-overlay preparation are presented in these publications. Until very recently, there were no bonded 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, such as Georgia, Tennessee, Kentucky, and Colorado constructed whitetopping test sections on a trial and error basis. With the lack of guidelines, if the pavement is over-designed, the section performs well at a high construction cost. If the pavement is under-designed, the section requires maintenance or reconstruction, and diminishes the confidence in whitetopping pavement rehabilitation.

Therefore, there is a need for rationally developed whitetopping thickness design guidelines. Research testing conducted during this study allowed the development of a mechanistic whitetopping design procedure for the State of Colorado.

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2.0 BACKGROUND

Plain concrete, reinforced concrete, and fibrous (fiber reinforced) concrete have been used over the years for whitetopping, or resurfacing, flexible pavements[4-20]. 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 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 1992, NCHRP synthesis 204[14] listed 189 whitetopping projects constructed in the U.S. since 1918. These projects included streets, highways, and airfield pavements.

Prior to the design guidelines reported herein, PCA and ACPA sponsored a research study to develop thickness design guidelines for "ultrathin" whitetopping pavements[21]. The term, "ultra-thin whitetopping" or UTW, refers to the process of resurfacing existing asphalt pavements with concrete overlays with a maximum thickness of 4 in.[15]. For the PCA study, slabs located at the Spirit of St. Louis Airport in Chesterfield, MO were instrumented with strain gages and loaded using a 20 kip single axle load (SAL). A total of eight PCC slabs were instrumented and tested. Strain gages were located at potentially critical locations on the slabs. It was concluded that, for ultra-thin whitetopping with short joint spacing, the load location inducing the critical PCC stress is at the corner of the slab. The critical location inducing maximum asphalt strain occurs at the midpoint of the longitudinal joint.

To determine an adjustment factor increasing the stress due to the partially bonded condition, measured field load-induced flexural stresses were compared to fully bonded theoretical stresses. In the PCA study, a factor of 1.36 (36% increase in stress due to partial bonding) was determined based on the data collected in Missouri (average stress increase of 19% with a standard deviation of 17%). This adjustment factor was applied to stresses computed during a parametric study. Once the parametric study was complete and the stresses were converted and adjusted to simulate measured field behavior, linear regression techniques were used to develop equations predicting the critical stresses. The equations included parameters of the whitetopping pavement which have a significant impact on the induced concrete flexural stresses and asphalt flexural strains.

The PCA design procedure was developed as a guide for determining the PCC thickness of ultra-thin whitetopping to be used on low-volume roadways, intersections, offramps, etc. The maximum thickness and joint spacing included in the parametric study was 4 in. and 50 in., respectively. The State of Colorado is interested in using whitetopping as a technique for rehabilitating deteriorated asphalt highway pavements. The PCA design procedure did not include the thicknesses and joint spacing necessary for projects of this magnitude. Therefore, research has been conducted to develop bonded whitetopping design guidelines for the State of Colorado.

3.0 APPROACH

The general techniques used in the development of the PCA ultra-thin whitetopping design procedure were used to develop the Colorado guidelines. Field testing was conducted to evaluate critical load locations for the thicker PCC layer with larger joint spacings. The load-induced flexural strains were used to calibrate fully bonded stresses computed using finite element analysis techniques to partially bonded stresses measured in the field. Load testing was conducted throughout the course of a day in order to develop a temperature correction to be applied to the critical stresses derived for zero temperature gradient (zero slab temperature curling). Thickness design guidelines were established for partially bonded whitetopping using field calibrated theoretical stresses. Equations predicting the critical concrete flexural stresses and asphalt concrete strains for use in whitetopping design 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.

It was also requested that the developed mechanistic design procedure (based on an axle load distribution obtained from traffic monitoring data) 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 whitetopping thicknesses as 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 Colorado for 8 in. pavement thicknesses. 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.

4.0 FIELD TESTING PROGRAM

In order to develop design guidelines for bonded whitetopping pavement systems, field instrumentation and load testing was conducted at three different sites in Colorado. The objectives of the field testing were to:

- determine the critical load location of whitetopping pavements
- study the effects of different AC surface preparation techniques
- measure the response of whitetopping pavements under traffic loading
- evaluate interface bonding strength between the concrete and the asphalt layers
- investigate the effect of pavement age on load-induced stresses
- calibrate theoretical with measured stresses to develop thickness design guidelines

A total of three test pavement sites were investigated as part of this study. Each site had multiple test sections (slabs). The first two test sites were constructed during the summer of 1996. These sections 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.

4.1 CDOT Test Pavements

The first test project (CDOT1) was constructed on a frontage road to Santa Fe Drive in Denver, CO. This project had a total length of 1,000 feet, consisting of two 500-ft test sections. The first test section had 4-in.-thick concrete slabs placed on top of a 5-in.-thick newly placed asphalt pavement layer. No special asphalt surface preparation was attempted. The second section had 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. No dowel bars were used for transverse joints. Both longitudinal and transverse joints were sawcut to 1/3 of the concrete slab depth. Soil underneath the pavement was classified as A-7-6 and reportedly had a modulus of subgrade reaction (k) of approximately 150 pci.

The second whitetopping project in Colorado (CDOT2) involved rehabilitation of about a one-mile long, two lane existing asphalt pavement on State Road 119 near Longmont, CO. 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. For the east half of the pavement, a 1 ½ in. new asphalt layer was placed on top of the existing asphalt concrete pavement, with a concrete slab thickness of 5 in. On the passing lane of the east half of the pavement, 4 ½ in.-thick concrete slabs were placed directly on top of the existing asphalt pavement. For the traffic lane, the asphalt pavement was milled 1 ½ in., resulting in 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 was reportedly 340 pci.

The third whitetopping project in Colorado (CDOT3) involved rehabilitation of about a three-mile long section of two lane pavement on heavily truck-trafficked US Rt. 287 near Lamar, CO. Variables incorporated into six sections of this project included various concrete slab dimensions and joint reinforcement. Both the north and southbound lanes and shoulders were milled and thoroughly cleaned prior to concrete placement. The milled asphalt thickness was measured to be about 7 in. The design specified a 6 in. concrete whitetopping slab and was based on a 225 pci 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. After construction, 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.

4.2 Description of Test Slabs

Three slabs in the first project were instrumented and load tested at Santa Fe Drive (layouts and photographs shown in Appendix A). 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. No tie bars were used along joints between curbs and traffic lanes. Therefore, all three test slabs had a tied joint on the east side and a free edge on the west side.

Five slabs were instrumented with strain gages and load tested at Longmont (layouts and photos in Appendix B). Slabs had different dimensions, concrete slab thickness and concrete-asphalt interface conditions. Concrete design thicknesses ranged from 4.5 to 6 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.

Three slabs were instrumented with strain gages and load tested at Lamar (layouts and photos in Appendix C). 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. The existing asphalt surface was milled prior to concrete placement. Test slabs were located in the outside driving lane with tied concrete shoulders.

4.3 Instrumentation and Load Testing

All three test slabs of the Santa Fe Drive project were 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 top. 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 Santa Fe Drive project was conducted in August, 1996 and August, 1997.

Each Longmont site test slab was instrumented with 8 strain gages. Since the slabs at the Longmont 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 Santa Fe site (one on top of asphalt surface and the other in the concrete 1/2 in. above the asphalt top). Surface gages were also installed directly above the embedment gage locations. Load testing was performed at the Longmont test site in September, 1996 and August, 1997.

The project in 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 Lamar test site in September, 1997.

Thermocouple trees were installed before concrete pavement construction to monitor temperature gradients during load testing. Thermocouples were installed at five different depths in the pavement, at top of concrete slab, mid-depth of the concrete, concrete-asphalt interface, 2.5 in. in the asphalt, and near the bottom of the asphalt layer.

To document the relative temperature-induced curling deformation of the whitetopping slab surface, reference rods were driven into subgrade soil prior to construction. To minimize the effects of temperature on the movement of the reference rods, they were fabricated using invar steel with a very low coefficient of thermal expansion. The reference rods were positioned so that the first step of a dipstick profile measuring device would be from the rod to the corner of the test slab.

The dipstick was then used to record the relative elevations of the test slab by traversing a grid across the slab surface. For each site, initial surface profiles were measured using a dipstick, provided by CDOT, on selected slabs the following morning after concrete placement. These profiles were used as base lines for determining slab curl movements during load testing using this procedure each time the load testing was performed throughout the day. Curling profiles were recorded for the partially bonded whitetopping slabs.

Load testing was conducted several times throughout the day for each slab. Static and dynamic 20 kip single axle load flexural strains were measured. Air and pavement temperatures at different depths were recorded throughout the load testing period.

4.4 Laboratory Tests of Concrete Cylinders, Beams, and Pavement Cores

Concrete cylinders and beams were cast during pavement construction for the Santa Fe and Longmont test sites. Cylinders were tested at 28 and 365 days for their modulus of elasticity and compressive strength. The beams were tested at 28 days to determine their flexural strength. Pavement cores were drilled in conjunction with load testing, and two cores were typically taken from each test slab. Core thickness was measured, and direct shear[22] testing was conducted to determine the interface shear strength between the concrete and asphalt layers. Laboratory test results and test slab characteristics are presented in Table 4.4.1.

Concrete cylinders and beams were not available for testing from the Lamar site. Concrete cores were drilled in conjunction with the load testing performed at 28 days. Material characteristics were determined from core tests and are presented in Table 4.4.1.

4.5 The Effect Interface Preparation on Shear Strength and Load Induced Strains

The effects of interface preparation on load induced pavement response was studied at two of the three test projects evaluated. Project CDOT1 on Santa Fe Frontage Road was constructed with new asphalt. The joint spacing was 5 ft. Two of the test slabs instrumented with strain gages had a thickness of 5 in. and were constructed on a 4 in. asphalt base. For one of the slabs, the new asphalt concrete was milled prior to constructing the whitetopping. Also, two of the slabs constructed at Project CDOT2 in Longmont offered the opportunity to directly evaluate the effect of milling the asphalt surface. Both CDOT2 slabs had a joint spacing of 6 ft and were constructed on existing asphalt concrete pavement (one milled and one unmilled).

For all the test slabs at each project, cores were removed for testing. The interface shear strength was measured for each core removed and the average shear strength for each of the test slabs is shown in Table 4.4.1. 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 at 1 year.

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 4.5.1, load induced strains for newly placed asphalt were increased by an average of about 50 percent if the interface was milled. On the contrary, for existing asphalt pavements, the load induced strains were 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). While these observations are significant findings, more testing should be performed to verify the trends are constant for various joint spacings before incorporation into the design procedure.

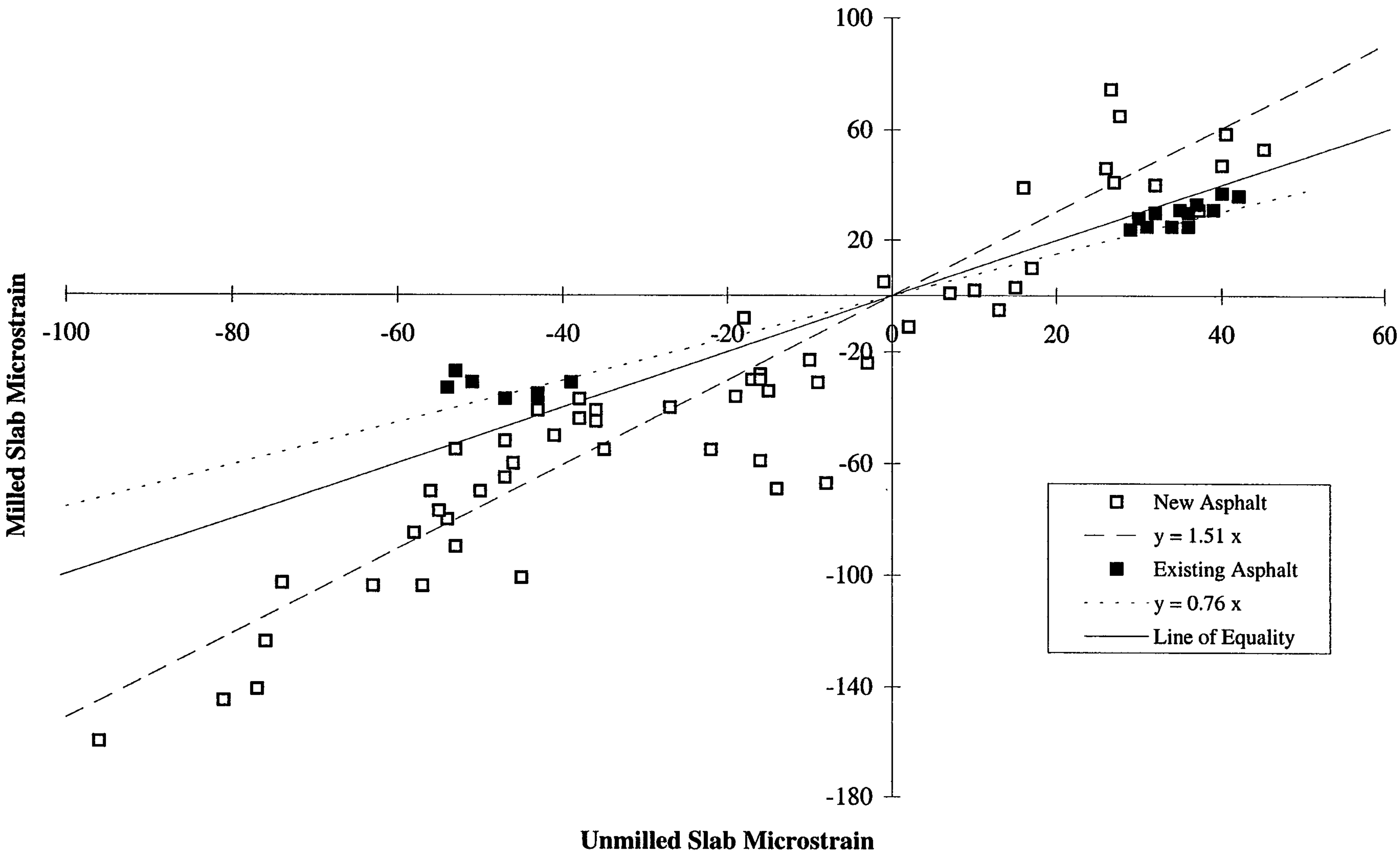
Table 4.4.1 - Test Slab Characteristics and Test Results

Site	Test Slab	PCC Thickness, in.	AC Thickness, in.	PCC 28 day Elastic Modulus, psi	PCC 365 day Elastic Modulus, psi	PCC 28 day Compressive Strength, psi	PCC 365 day Compressive Strength, psi	AC Resilient Modulus, psi	Modulus of Subgrade Reaction, psi/in.
Santa Fe	1	4.7	4.5	3,700,000	4,210,000	4290	5590	350,000	150
	2	5.8	5.9	3,700,000	4,210,000	4290	5590	350,000	150
	3	6.0	5.4	3,700,000	4,210,000	4290	5590	350,000	150
Longmont	1	5.1	3.3	3,280,000	4,000,000	4150	5540	800,000	340
	2	5.4	4.6	3,280,000	4,000,000	4150	5540	800,000	340
	3	6.3	3.4	3,280,000	4,000,000	4150	5540	800,000	340
	4	7.3	3.4	3,280,000	4,000,000	4150	5540	800,000	340
	5	6.8	2.8	3,280,000	4,000,000	4150	5540	800,000	340
Lamar	B	7.35	7	4,080,000	****	7470	****	800,000	225
	E	6.8	6.6	4,990,000	****	7200	****	800,000	225
	F	5.55	6.6	4,460,000	****	7400	****	800,000	225

Table 4.4.1 - Test Slab Characteristics and Test Results (continued)

Site	Test Slab	Longitudinal Joint Spacing, in.	Transverse Joint Spacing, in.	AC Surface Condition	28 Day Interface Shear Strength, psi	365 Day Interface Shear Strength, psi
Santa Fe	1	60	60	New	45	80
	2	60	60	New	30	60
	3	60	60	New Milled	10	80
Longmont	1	72	72	Existing	100	****
	2	120	144	New	60	105
	3	72	72	New	70	105
	4	72	144	Existing Milled	65	100
	5	144	144	Existing Milled	****	155
Lamar	B	144	120	Existing Milled	80	****
	E	72	72	Existing Milled	90	****
	F	72	72	Existing Milled	110	****

Figure 4.5.1- Effect of Interface Milling on Load Induced Microstrain



5.0 MECHANISTIC WHITETOPPING THICKNESS DESIGN PROCEDURE

Guidelines for bonded whitetopping were established from field calibrated flexural stresses and strains. This section includes the details of the steps followed during the development 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 process included the following elements:

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

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

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

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

5. To account for loss of support with temperature curling effects, an equation was derived to incorporate 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 covered.

    - Stepwise least squares linear regression techniques were used to derive equations predicting concrete stresses and asphalt strains from different pavement parameters and material properties.

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

    - The 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 Determination of Critical Load Location

The critical load location for the design of whitetopping pavement was determined by comparing the stress data collected for each load position. For the parameters studied, the critical load location inducing the highest tensile stress in the concrete layer was established 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. While it is reasonable that free edge loading produces the highest stress, it is more likely that the joints loaded by traffic will not be free edges. Therefore, for the design procedure, tied longitudinal joint loading was considered the critical load case, as shown in Figure 5.1.1. A relationship between free edge and tied edge stresses was developed for use in designs where free edge loading is likely (narrow truck entrances where slabs are not tied into concrete curb and gutter). The equation for data shown in Figure 5.1.2 is as follows:

$$\sigma_{FE} = 1.87 \times \sigma_{TE} \qquad \text{(Eq. 5.1.1)}$$

where,

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

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

5.2 Determination of Load-Induced Stress at Zero Temperature Gradient

Each of sites 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 course of a day. 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

Figure 5.1.1 - Location of Load Resulting in Maximum Stress

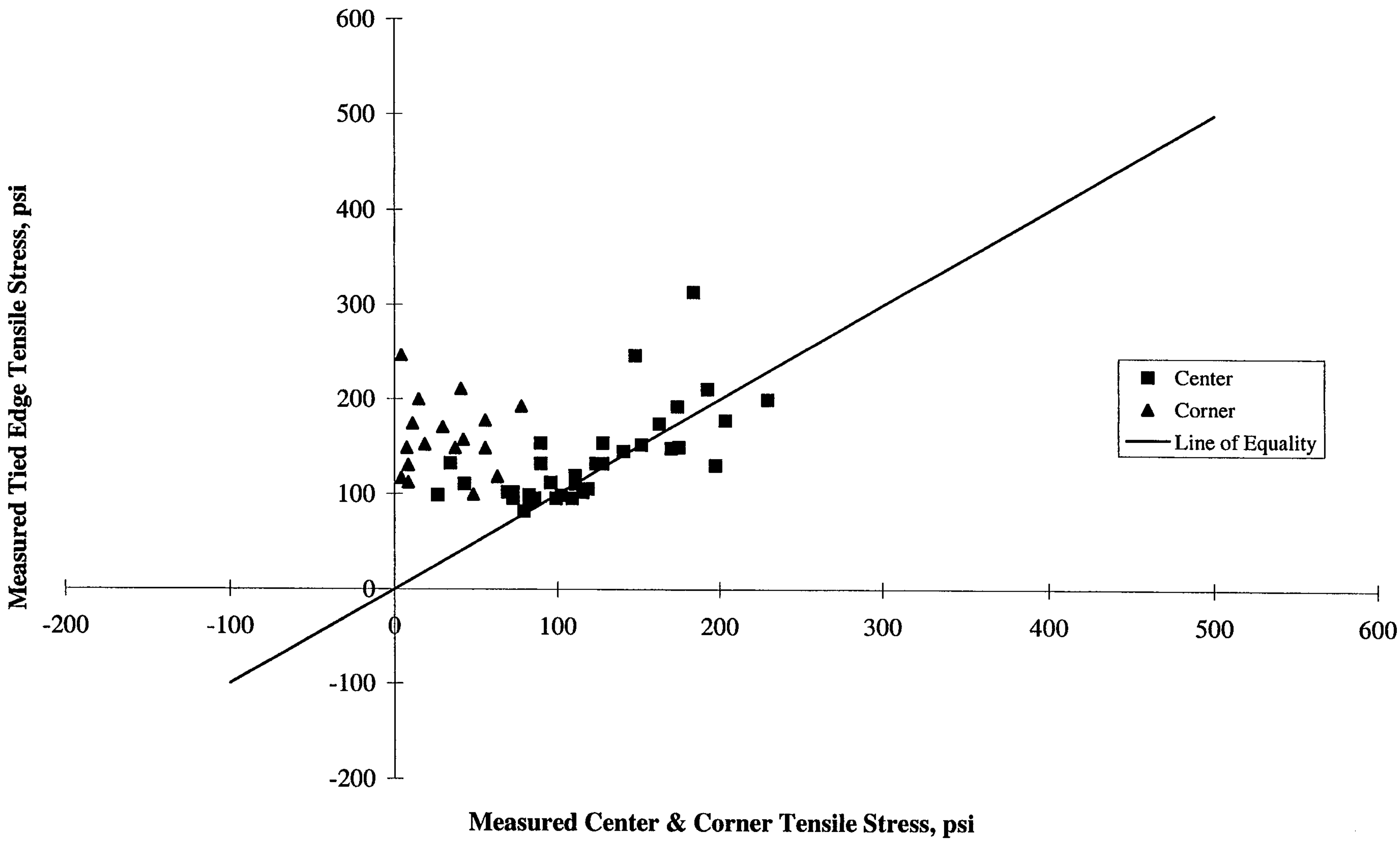
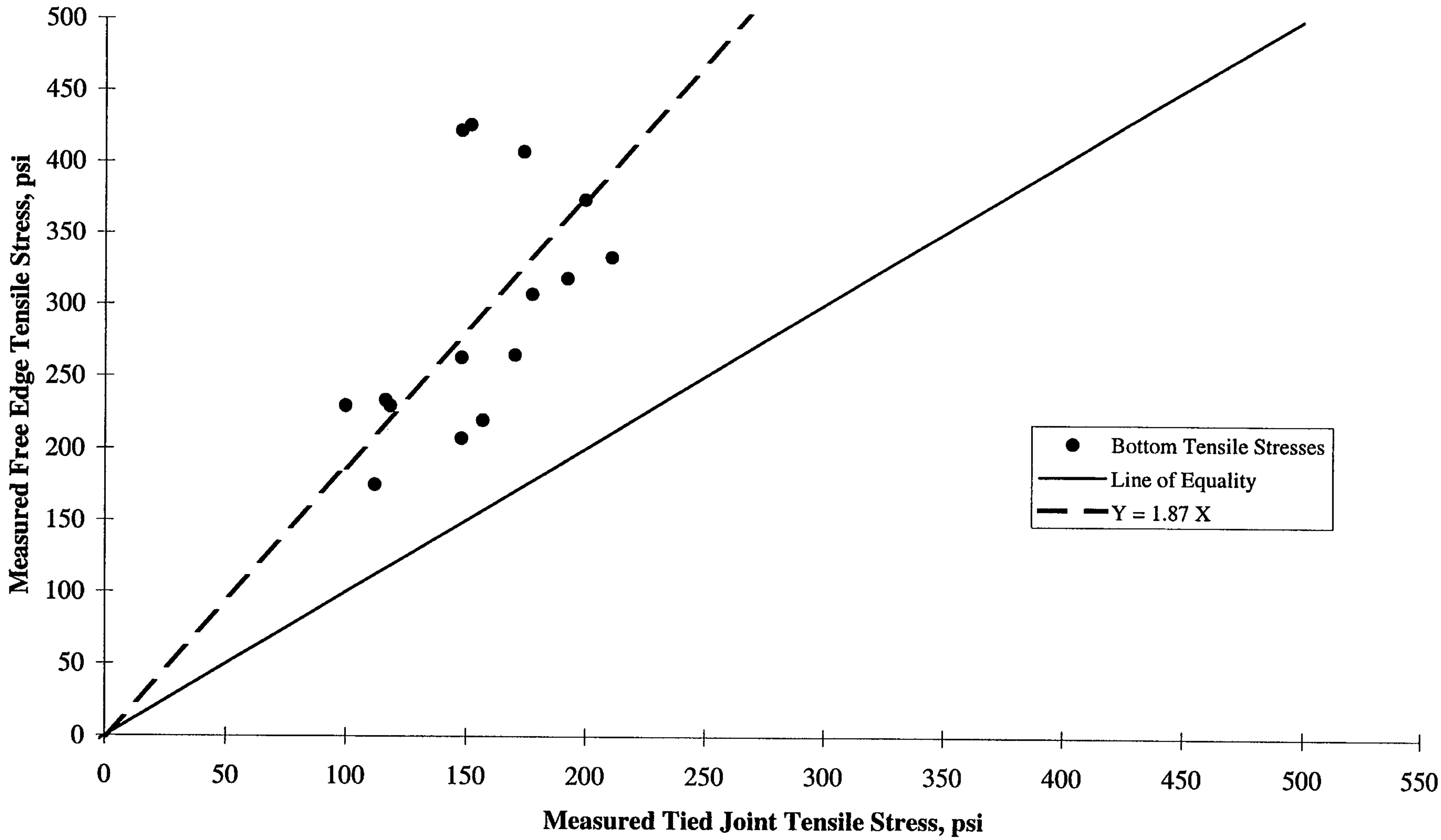


Figure 5.1.2 - Conversion From Tied Joint to Free Edge Stress



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.

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

The effect of interface bonding was initially quantified by comparing measured stresses (zero temperature gradient) 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), 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 medium-thick plate on a Winkler (spring) foundation bending theory. 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 5.3.1. In general, measured stresses are greater than theoretical stresses. The slope of the "best fit" line is 1.54 which represents a 54 percent increase in the stress due to the partial bond condition. However, to account for the variability in the collected data, the calculated standard deviation of the coefficient was used to derive an equation representing a 95 percent confidence for the increase in stress due to the partial interface bond. This line, plotted in the figure, representing a 65 percent increase in the bottom edge fully bonded tensile stress calculated is as follows:

$$\sigma_{ex} = 1.65 \times \sigma_{th} \qquad \text{(Eq. 5.3.1)}$$

where,

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

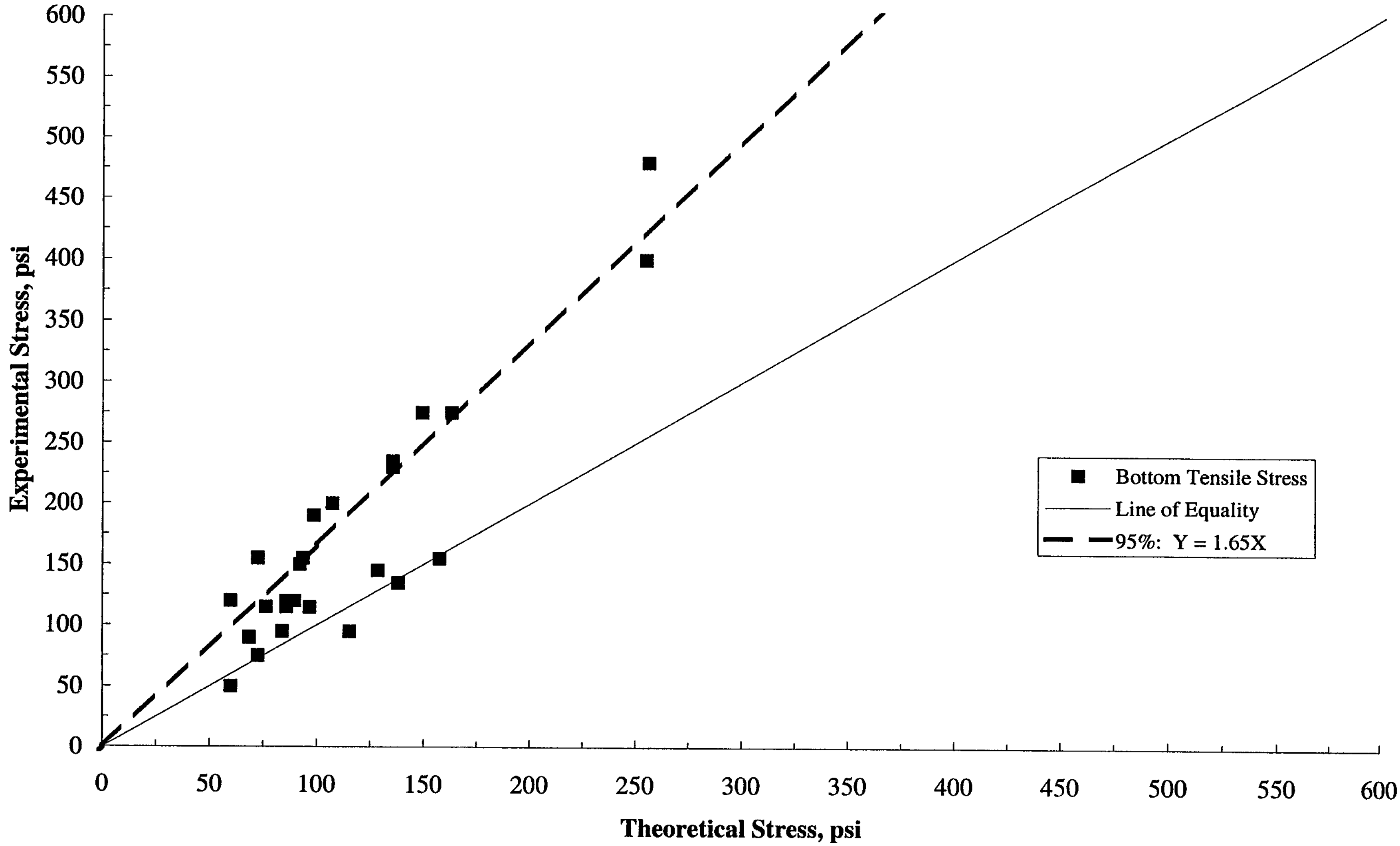
σ_{th} = calculated fully bonded stress, psi

The coefficient can be reduced to 1.63 or 1.59 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.

5.4 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 studied by evaluating data collected in the field. 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 1/2 in. above the asphalt

Figure 5.3.1 - Increase in Critical Load Stress Due to Partial Bonding Condition



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 strain measured 1/2 in. from the concrete bottom. If slabs were fully bonded, the concrete bottom strain would equal the asphalt surface strain. Figure 5.4.1 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 equation representing the loss of strain is as follows:

$$\varepsilon_{ac} = 0.842 \text{ x } \varepsilon_{pcc} \qquad \text{(Eq. 5.4.1)}$$

where,

ε_{ac} = measured asphalt surface strain, microstrain

ε_{pcc} = measured concrete bottom strain, microstrain

There is approximately a 15 percent loss of strain transfer from the concrete to the asphalt due to the partial bond between the layers. 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.

5.5 Analysis of Temperature Effects on Load-Induced Stresses

Load testing was repeated throughout the course of the day 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 course of the day. Temperature gradients throughout load testing ranged from -1 to 5 °F/in. A significant change in stress occurred with changing temperature gradient. This is a significant deviation from ultrathin whitetopping. Ultrathin whitetopping, by design, requires a short joint spacing to minimize restraint stress cracking. The PCA study recommends limiting the joint spacing to 48 in. For relatively short joint spacings, the loss of support due to curling is less likely. Loss of support effects in the thickness design procedure should be accounted for when joint spacings exceed 4 ft.

Once the theoretical load-induced stresses are adjusted for the partial bonding condition, the effect of the temperature-induced curling are applied.. Figure 5.5.1 shows the percent change in measured stress over the range of gradients tested. The relationship derived between the change in stress and measured temperature gradient is as follows:

$$\sigma_{\%} = 4.56 \text{ x } \Delta_T \qquad \text{(Eq. 5.5.1)}$$

where,

$\sigma_{\%}$ = percent change in stress from zero gradient

Δ_T = temperature gradient, °F/in.

Figure 5.4.1 - Asphalt Surface Strain vs. Concrete Bottom Strain

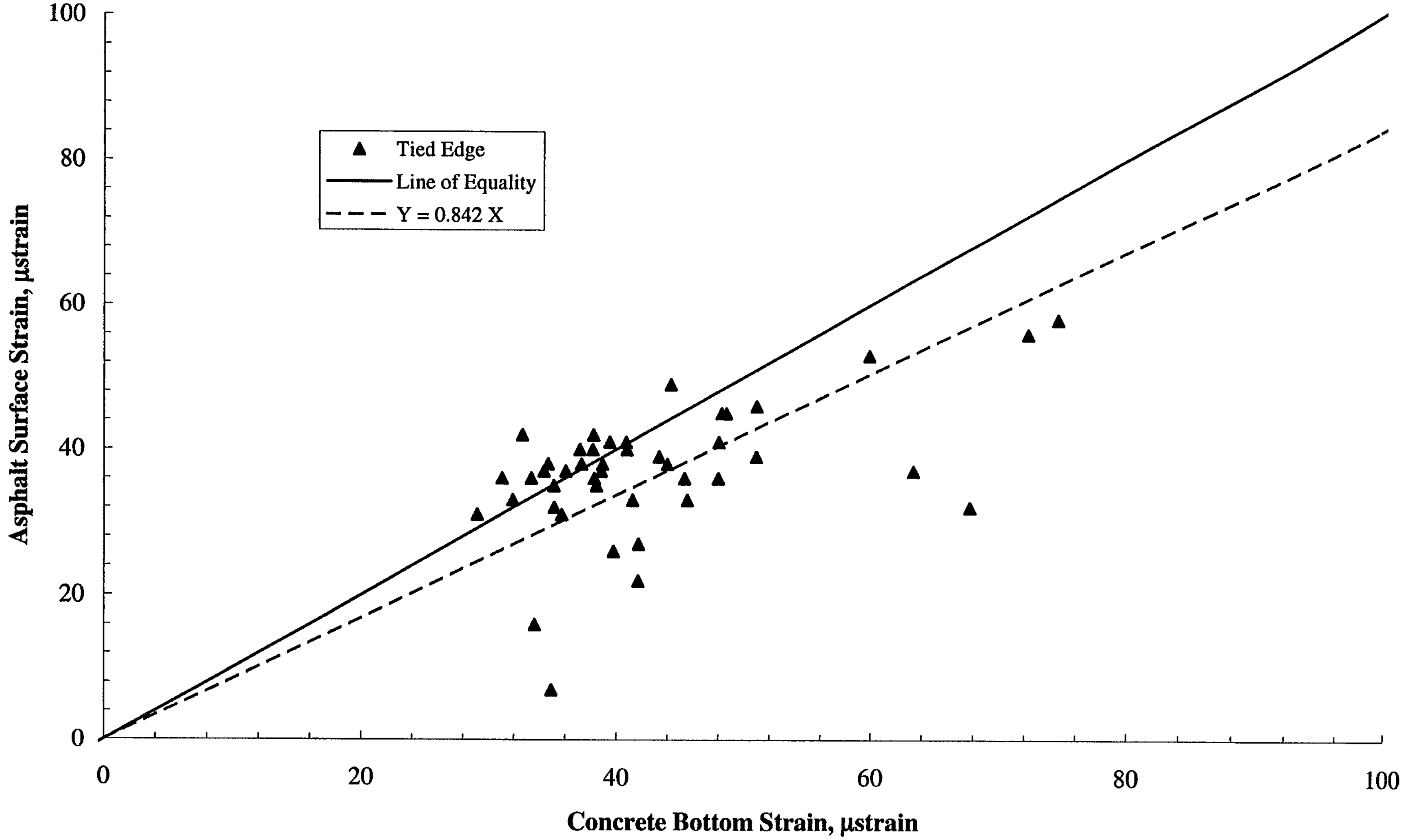
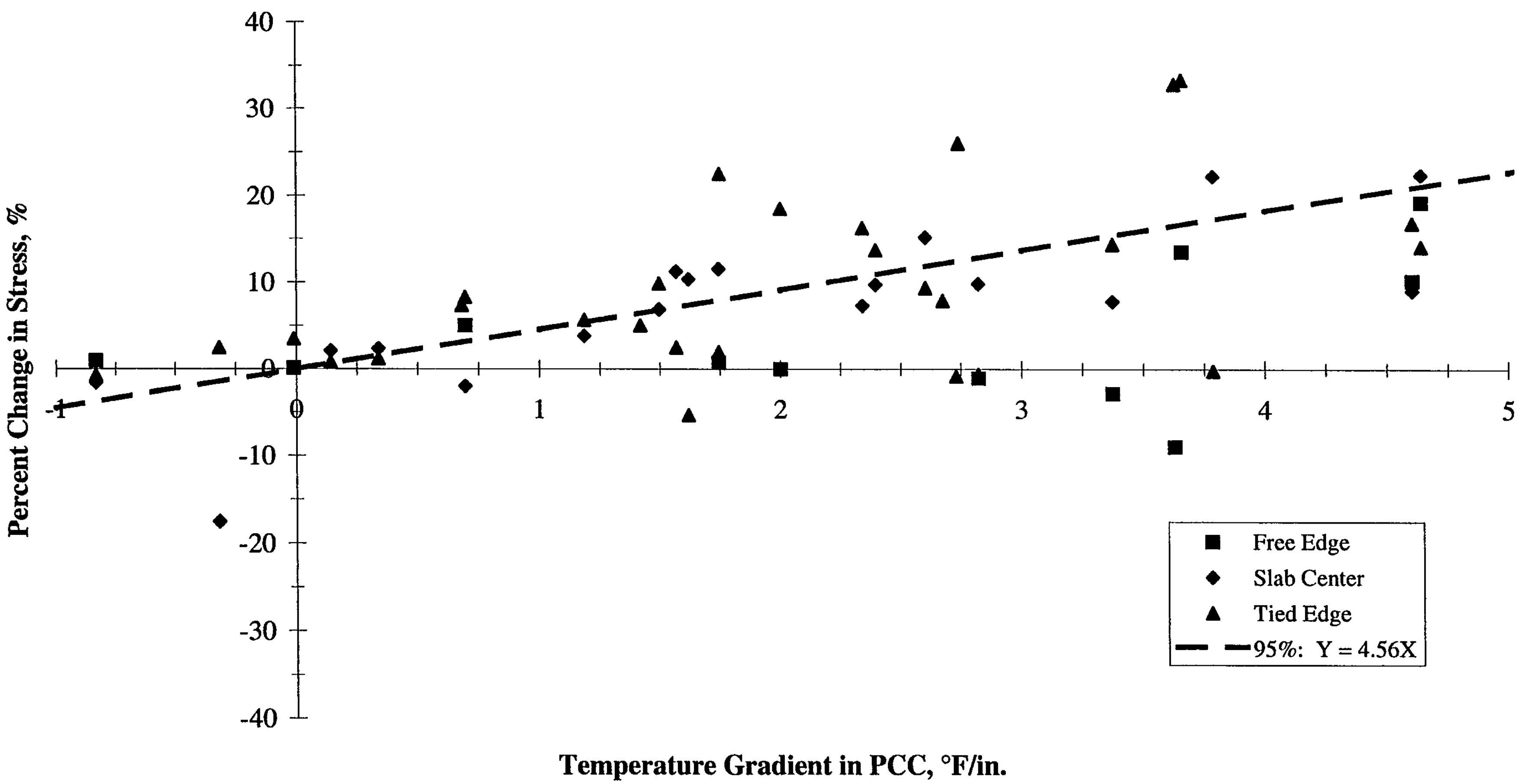


Figure 5.5.1 - Increase in Load Stress Due to Curling Loss of Support



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.

5.6 Development of Design Equations

Two different modes of distress may 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 developing the design equations.

5.6.1 Stress Computation Using the Finite Element Program ILSL2

For the corner and tied edge loading conditions, the following combinations of parameters were investigated:

Joint spacing, L	48, 72, and 144 in.
Concrete slab thickness, t_{pcc}	4, 5, and 6 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, 0.5, 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	75, 200, and 400 pci

Mid-joint loading. Load-induced longitudinal joint stresses for a 20-kip single axle load (SAL) and a 40-kip tandem axle load (TAL) were computed. Maximum tensile stresses at the bottom of each layer were calculated for both the concrete and asphalt. Maximum asphalt strains used in generating the design equations occurred for the joint loading condition. In most cases, the joint loading condition also produced the maximum stress at the bottom of the concrete layer. However, this may not be the case for ultrathin whitetopping as found during the PCA study(21).

Corner loading. Both a 20-kip SAL and a 40-kip TAL were applied to whitetopping slab corners. The corner loading case was found to produce the maximum concrete stress for relatively few conditions. In general, the corner loading case governed at higher values of the effective radius of relative stiffness. As the stiffness increases, the load-induced stress decreases. When the corner load case governed, relatively lower stresses resulted. The maximum stress, whether edge or corner, was used in the derivation of the concrete stress prediction equations.

Temperature restraint stress. Based on the information collected during the load testing events, temperature restraint stresses were not incorporated into the design procedure. As shown on the plots in the appendix, the surface profile was never observed to be in a curled down condition, even for the highest temperature gradients of 6° F/in. Slab upward warping effects due to moisture differentials (surface drier than bottom) were greater than measured downward temperature curling effects.

For the edge loading condition, the maximum 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.

As previously discussed, for certain combinations of parameters (high stiffness), the maximum load-induced stress occurs at the corner. 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 stress is 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 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. Corner loading condition restraint stresses probably would not contribute to excessive consumption of the fatigue life and were not incorporated into the thickness design procedure.

5.6.2 Development of 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 were developed as follows:

Concrete Stress For 20-kip SAL

$$\sigma_{pcc} = 919 + 18{,}492 / l_e - 575.3 \log k + 0.000133 E_{ac} \qquad \text{(Eq. 5.6.1)}$$

$$R^2_{adj.} = 0.99$$

Concrete Stress For 40-kip TAL

$$\sigma_{pcc} = 671.2 - 0.000099 E_{ac} - 437.1 \log k + 1.582 \times 10^4 / l_e \qquad \text{(Eq. 5.6.2)}$$

$$R^2_{adj.} = 0.99$$

Asphalt Strain For 20-kip SAL

$$1/\varepsilon_{ac} = 8.51114 \times 10^{-9} E_{ac} + 0.008619\, l_e/L \qquad \text{(Eq. 5.6.3)}$$

$$R^2_{adj.} = 0.99$$

Asphalt Strain For 40-kip TAL

$$1/\varepsilon_{ac} = 9.61792 \times 10^{-9} E_{ac} + 0.009776\, l_e/L \qquad \text{(Eq. 5.6.4)}$$

$$R^2_{adj.} = 0.99$$

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 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.

5.6.3 Adjustment of the Stress Predictions

Equations were developed to adjust the theoretical stresses and strains to account for conditions such as partial bond and loss of support due to temperature-induced slab curling. The stresses computed were 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 65 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 15 percent to account for the partial bonding condition. Effects of temperature-induced slab curling on load-induced stresses were also included in the thickness design procedure.

5.7 PCC and Asphalt Concrete Fatigue

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. In a design, the total fatigue should not exceed 100%. The concrete fatigue criterion was incorporated as follows:

For SR > 0.55

$$\mathrm{Log}_{10}(N) = (0.97187 - SR) / 0.0828 \qquad \text{(Eq. 5.7.1)}$$

For 0.45 ≤ SR ≤ 0.55

$$N = (4.2577 / (SR - 0.43248))^{3.268} \qquad \text{(Eq. 5.7.2)}$$

For SR < 0.45

$$N = \text{Unlimited} \qquad \text{(Eq. 5.7.3)}$$

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. For whitetopping pavements, when the asphalt layer is covered by concrete slabs, pavement rutting will not be the governing distress. Therefore, the 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:

$$N = C * 18.4 * (4.32 \times 10^{-3}) * (1/\varepsilon_{ac})^{3.29} * (1/E_{ac})^{0.854} \qquad \text{(Eq. 5.7.4)}$$

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.

5.8 A 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 50 percent of the asphalt concrete fatigue life has been consumed to date. 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 Tables 5.8.1 and 5.8.2 for the expected loads (Column 1 in Table 5.8.1) and expected number of repetitions (Column 8 in Table 5.8.2). Parameters and material properties used in the design are the following:

asphalt modulus of elasticity, E_{ac} = 600,000 psi
asphalt thickness, t_{ac} = 7 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 = 5 ½ in.
joint spacing, L = 72 in.
existing asphalt fatigue = 50 percent

Procedure Steps:

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

$$l_e = 32.77$$

$$L/l_e = 2.20$$

Table 5.8.1 - Design Example

Axle Load, kips	Critical Concrete Stresses and Asphalt Strains					
	Load Induced		Bond Adjustment		Loss of Support Adjustment	
	Stress, psi	Microstrain	Stress, psi	Microstrain	Stress, psi	Microstrain
1	2	3	4	5	6	7

Single Axles **le = 32.77** **L/le = 2.20**

22	263	135	434	115	494	115
20	239	111	395	94	449	94
18	215	111	355	94	404	94
16	191	98	316	84	359	84
14	167	86	276	73	314	73
12	144	74	237	63	269	63
10	120	62	197	52	224	52
8	96	49	158	42	180	42
6	72	37	118	31	135	31
4	48	25	79	21	90	21
2	24	12	39	10	45	10

Tandem Axles

44	98	120	161	102	183	102
40	89	98	146	83	166	83
36	80	98	132	83	150	83
32	71	87	117	74	133	74
28	62	76	103	65	117	65
24	53	65	88	55	100	55
20	44	54	73	46	83	46
16	35	43	59	37	67	37
12	27	33	44	28	50	28
8	18	22	29	18	33	18
4	9	11	15	9	17	9

Table 5.8.2 - 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: 50

22	200	0.760	367	54.5	115	4,211,093	0.0
20	600	0.690	2,502	24.0	94	8,149,273	0.0
18	2,500	0.621	17,069	14.6	94	8,149,273	0.0
16	5,000	0.552	116,451	4.3	84	12,006,341	0.0
14	7,500	0.483	1,921,269	0.4	73	18,629,627	0.0
12	25,000	0.414	unlimited	0.0	63	30,935,640	0.1
10	550,000	0.345	unlimited	0.0	52	56,359,276	1.0
8	875,000	0.276	unlimited	0.0	42	117,435,486	0.7
6	1,250,000	0.207	unlimited	0.0	31	302,585,276	0.4
4	1,750,000	0.138	unlimited	0.0	21	1,148,650,837	0.2
2	5,000,000	0.069	unlimited	0.0	10	unlimited	0.0

Tandem Axles

44	5	0.282	unlimited	0.0	102	6,329,613	0.0
40	50	0.256	unlimited	0.0	83	12,249,016	0.0
36	500	0.230	unlimited	0.0	83	12,249,016	0.0
32	1,500	0.205	unlimited	0.0	74	18,046,501	0.0
28	5,000	0.179	unlimited	0.0	65	28,001,837	0.0
24	50,000	0.154	unlimited	0.0	55	46,498,769	0.1
20	75,000	0.128	unlimited	0.0	46	84,712,551	0.1
16	500,000	0.102	unlimited	0.0	37	176,515,034	0.3
12	750,000	0.077	unlimited	0.0	28	454,810,146	0.2
8	1,000,000	0.051	unlimited	0.0	18	unlimited	0.0
4	1,250,000	0.026	unlimited	0.0	9	unlimited	0.0
		Total Concrete Fatigue, % =		**97.8**	**Total Asphalt Fatigue, % =**		**53.2**

2. Using the calculated l_e and L/l_e along with the modulus of subgrade reaction, k, Equation 5.6.1 is used to compute the load-induced critical concrete stresses (Col. 2 in Table 5.8.1) and Equation 5.6.3 is used to compute the load-induced critical asphalt strains (Col. 3 in Table 5.8.1) 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 5.8.1.

3. Repeat step 2 for the anticipated tandem axle loads (TAL). Use Equation 5.6.2 to compute the concrete stresses and Equation 5.6.4 to compute the asphalt strains for a 40-kip TAL shown in the lower portion of Columns 2 and 3 in Table 5.8.1.

4. Using Equations 5.3.1 and 5.4.1, 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 5.8.1, respectively.

5. Use Equation 5.5.1 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 5.8.1 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 5.7.1 to 5.7.3, 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 5.7.4 and enter these values into Column 13.

12. The percent fatigue for the asphalt layer and the total asphalt fatigue damage is computed the same way as used for the concrete fatigue computation in Step 9 accept there is the addition of the fatigue damage 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, the concrete fatigue analysis governed. The critical stresses at the extreme fiber of the concrete slab cause greater damage than the critical strains at the bottom extreme fiber of the asphalt layer. For the existing asphalt and subgrade conditions, a concrete whitetopping thickness of 5 ½ in. with a joint spacing of 72 in. is shown to be sufficient to carry the anticipated traffic loading.

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, IL in the late 1950's and early 1960's. Traffic (frequency of axle loadings) is represented by the concept of the 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 alone. 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.

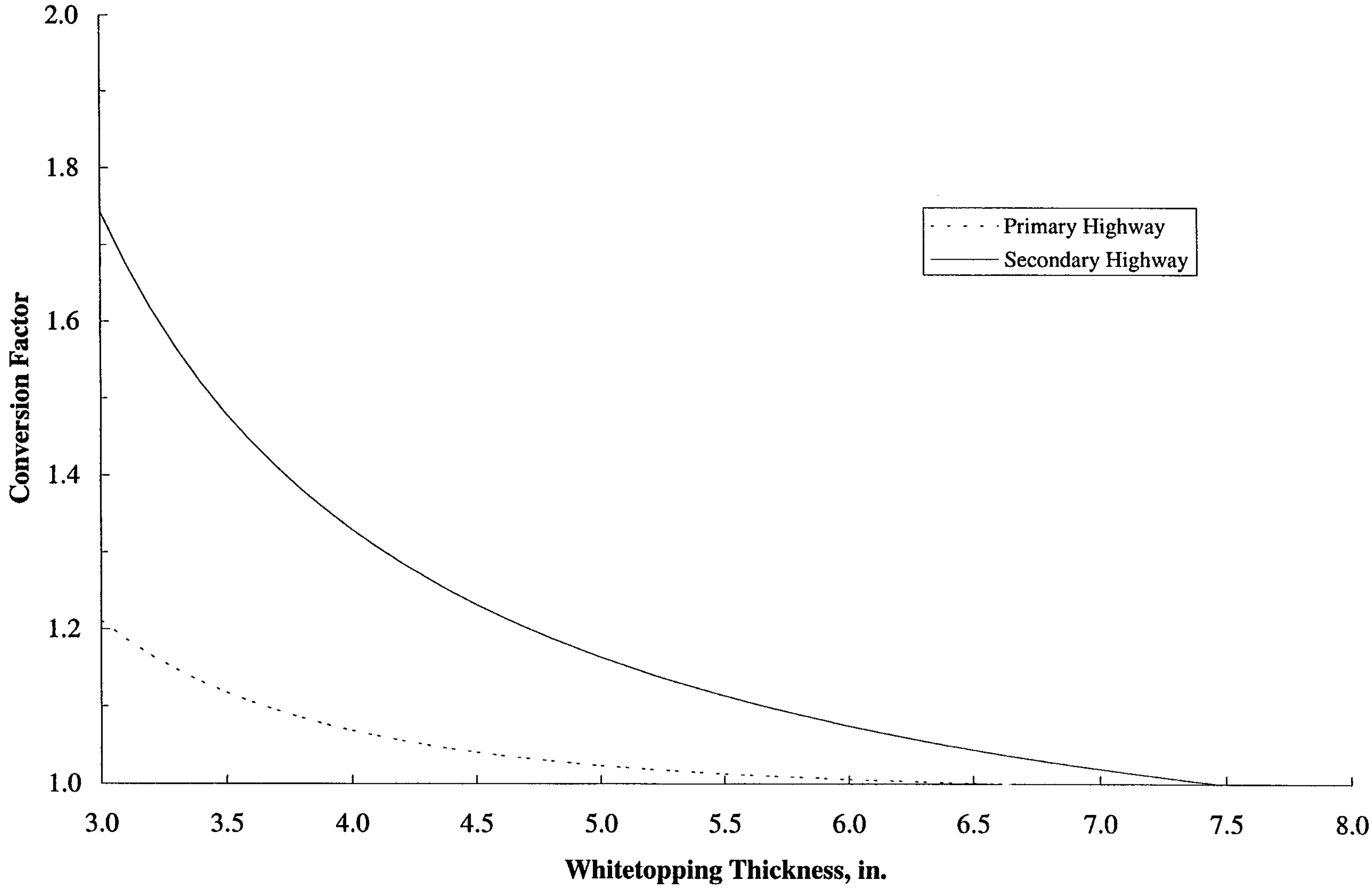
6.1 Converting Estimated ESALs to Whitetopping ESALs

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 6.1.1 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:

Primary Highway: $$F_{ESAL} = 0.985 + 10.057 * (t_{pcc})^{-3.456} \qquad \text{(Eq. 6.1.1)}$$

Secondary Highway: $$F_{ESAL} = (1.286 - 2.138 / t_{pcc})^{-1} \qquad \text{(Eq. 6.1.2)}$$

Figure 6.1.1 - Conversion of 8-in.-thick ESALs to Whitetopping ESALs



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, say 750,000, should be converted to 925,000 using the secondary highway conversion equation (Eq. 6.1.1).

6.2 Modified Whitetopping Thickness Design Conversion

Converting the traffic distribution to ESALs and using that 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 axle load distribution. For instance, in the example shown in Tables 5.8.1 and 5.8.2, for the axle load traffic distribution given, the required whitetopping thickness is 5.5 in. Using AASHTO 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. Inputting this number of expected repetitions for the 18-kip axle load and setting all other axles loads to zero repetitions results in about 1,440 percent fatigue life consumed. For the ESALs computed, the required thickness is calculated to be over 6.5 in. Therefore, a conversion was developed 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:

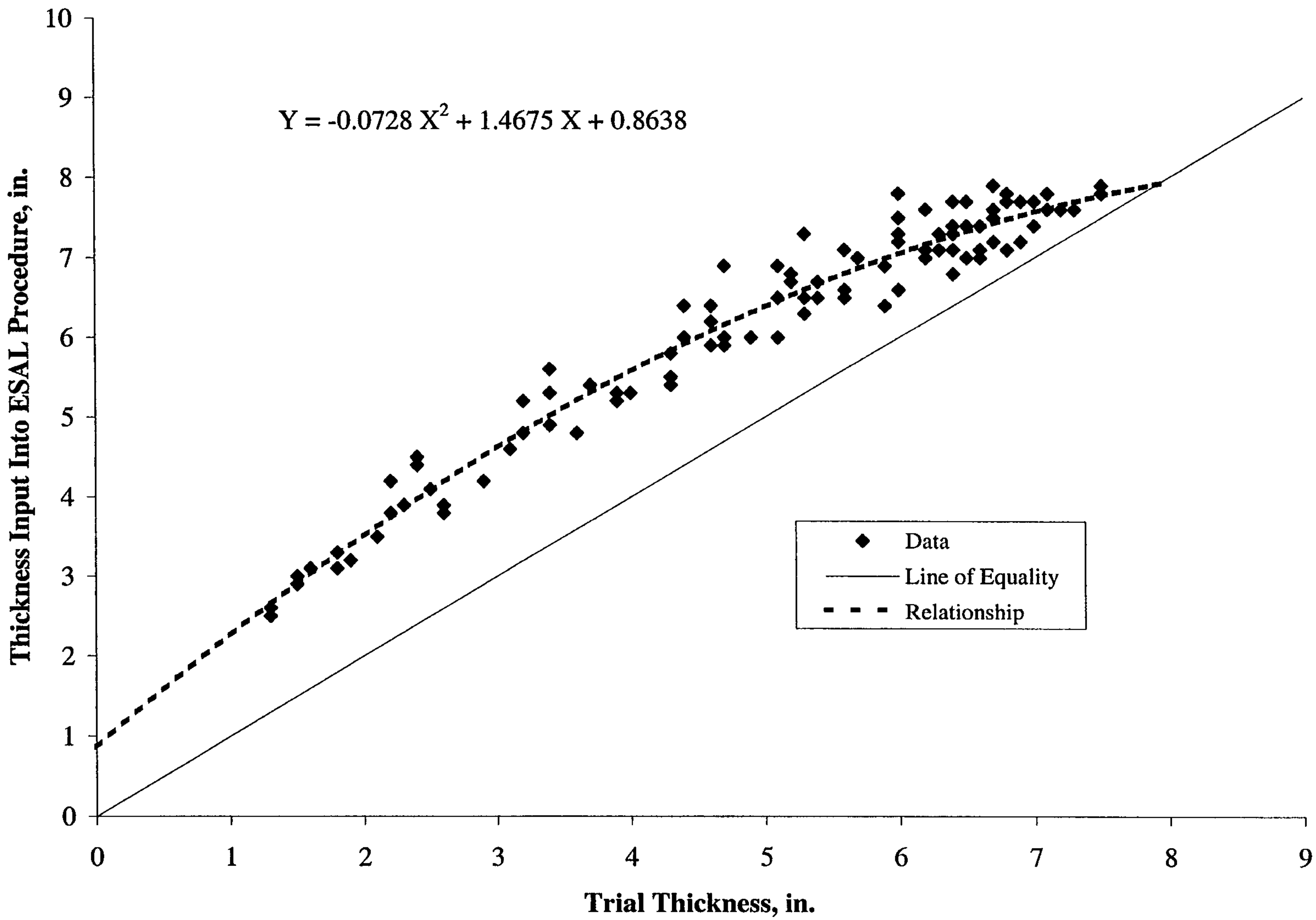
asphalt modulus of elasticity, E_{ac} = 50,000 to 1,000,000 psi
asphalt thickness, t_{ac} = 3 to 9 in.
existing modulus of subgrade reaction, k = 100 to 400 pci
concrete modulus of rupture, MR = 550 to 750 psi

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 6.2.1. The mechanistic procedure utilizes axle load distribution and the empirical procedure uses ESALs. As shown by the line of equality, the two design methods do not result in equal design thicknesses. However, the trend suggests that a relationship exists between the two procedures to allow a trial thickness to be converted

Figure 6.2.1 - Thickness Conversion from Mechanistic Procedure to Empirical Procedure



prior to being input into the ESAL design procedure. The equation developed to convert the trial whitetopping thickness to an input thickness is as follows:

$$t_{INPUT} = -0.0728 * (t_{TRIAL})2 + 1.4675 * (t_{TRIAL}) + 0.8638 \qquad \text{(Eq. 6.2.1)}$$

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 6.2.1, this correlation was developed for whitetopping thickness below 8 in. and should not be extrapolated further. Field data was collected on a maximum PCC thickness of about 7 in. and the design procedure equations were developed from theoretical stresses for concrete with a maximum thickness of 6 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 in.

Equations 5.6.1 and 5.6.3 were modified as follows to calculate the stress and strain due to an 18-kip Single Axle Load:

Concrete Stress For 18-kip SAL

$$\sigma_{pcc} = 18/20 * (919 + 18{,}492 / l_e - 575.3 \log k + 0.000133\ E_{ac}) \qquad \text{(Eq. 6.2.2)}$$

Asphalt Strain For 18-kip SAL

$$1/\varepsilon_{ac} = 18/20 * (8.51114 \times 10^{-9}\ E_{ac} + 0.008619\ l_e/L) \qquad \text{(Eq. 6.2.3)}$$

Figure 6.2.2 shows the calculations for the design example presented in Tables 5.8.1 and 5.8.2. As shown, a required thickness of 5 ¼ in. is the result of the modified design approach incorporating ESALs. While this is slightly different from the 5½ in. thickness required by the mechanistic procedure, it is within the standard deviation typically achieved by slip-form pavers.

7.0 SENSITIVITY ANALYSIS

Sensitivity analyses were conducted for calculated whitetopping thickness. 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.

As shown in Figures 7.0.1 and 7.0.2, the minimum concrete thickness is somewhat sensitive to lower moduli of subbase/subgrade reaction. Figure 7.0.1 shows, for a wide range of asphalt thicknesses, the required concrete thickness significantly increases at a subgrade modulus less than 150 pci. This sensitivity is more apparent in Figure 7.0.2 for

Figure 6.2.2 - Design Example Incorporating ESALs for Traffic Input

Whitetopping Input Parameters

Highway Category (Primary or Secondary)*	Secondary
Joint Spacing, in.	72
Trial Concrete Thickness, in.	5.23
Converted Concrete Thickness, in.	6.55
Concrete Flexural Strength, psi	650
Asphalt Thickness, in.	7
Asphalt Elastic Modulus, psi	600,000
Asphalt Fatigue Life Previously Consumed, %	50
Subgrade Modulus, pci	200
Temperature Gradient, °F/in.	3
Design ESALs	245,544
ESAL Conversion Factor	1.0418
Concrete Elastic Modulus, psi	4,000,000
Concrete Poisson's Ratio	0.15
Asphalt Poisson's Ratio	0.35
Neutral Axis	4.21

le = 35.05

L/le = 2.05

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
182	**97**	**301**	**82**	**342**	**82**

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
2.6E+05	**0.526**	**2.6E+05**	**98.1**	**82**	**1.3E+07**	**2.0**

Concrete Fatigue, % = 98.1 **Asphalt Fatigue, % = 52.0**

Required Whitetopping Thickness = 5.25 in.

Figure 7.0.1 - PCC Thickness Sensitivity to Modulus of Subbase/Subgrade Reaction

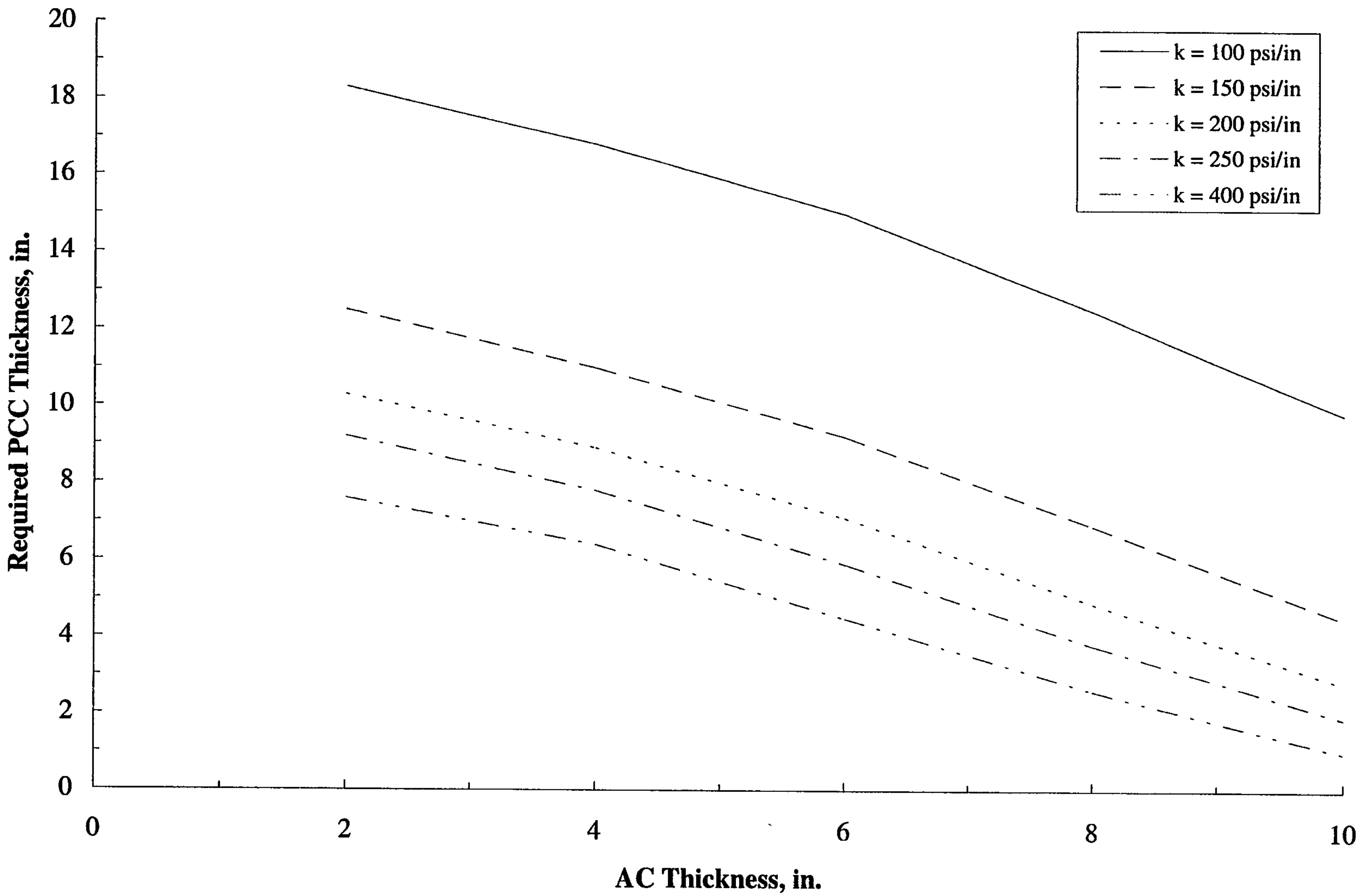
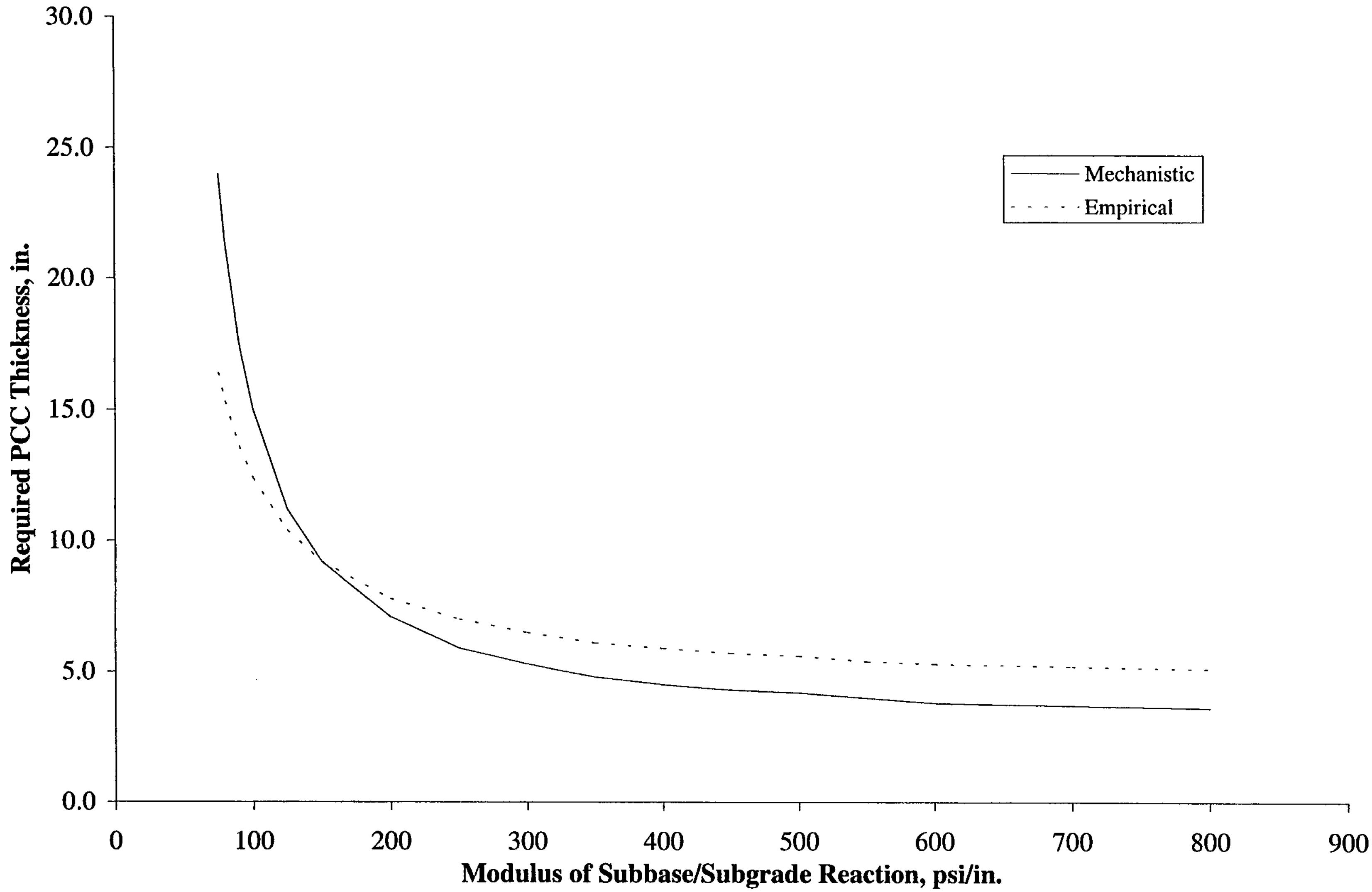


Figure 7.0.2 - Thickness Sensitivity to Subgrade Modulus



the mid-range 6-in. asphalt thickness. Therefore, it is recommended that this design procedure be used only when the modulus of subbase/subgrade reaction exceeds 150 pci. However, as a rehabilitation procedure for existing asphalt pavements, it is likely that the modulus of subbase/subgrade reaction will be higher than 150 pci.

Figure 7.0.3 shows the minimum concrete thickness sensitivity to asphalt modulus of elasticity. While the required thickness is fairly sensitive at very low asphalt moduli (50,000 psi), the minimum thickness of the asphalt layer should not be lower than about 5 in. At asphalt thicknesses below 5 in., the required concrete thickness, as calculated using this design procedure, increases with an increase in the asphalt modulus of elasticity. This phenomenon should be investigated during the verification process of this design procedure. A range of asphalt moduli and thicknesses should be included in the whitetopping design verification study.

Whitetopping thickness sensitivity as a function of the concrete flexural strength and temperature gradient is shown in Figures 7.0.4 and 7.0.5, 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 7.0.5.

Whitetopping thickness sensitivity to the expected number of 18-kip ESALs are shown in Figures 7.0.6 to 7.0.8. Thicknesses are not too sensitive to the number of ESALs above 1 million except under various levels of asphalt modulus of elasticity shown in Figure 7.0.7. It is shown that the required concrete thickness is not sensitive to asphalt modulus at traffic loading below 4 million ESALs. However, above 4 million ESALs, the limitations of the asphalt modulus are become apparent. For example, whitetopping should not be specified when the asphalt modulus is below 400,000 psi and the expected number of ESALs exceeds 5 million. The effect of a lower asphalt thickness is shown in Figure 7.0.6 supporting the suggested minimum thickness of about 5 in. The detrimental effect of relatively low moduli of subbase/subgrade reaction is also shown on Figure 7.0.8.

While the sensitivity of whitetopping will be better understood with continuing research, based on Figures 7.0.1 to 7.0.8, the design of whitetopping concrete for the rehabilitation of deteriorated asphalt concrete pavements is sensitive to certain existing in situ conditions. The greatest sensitivity of the minimum required concrete thickness is to the modulus of subbase/subgrade reaction. The effect of lower subgrade moduli was observed in all cases studied. The procedure is also sensitive to existing asphalt thickness and, in certain cases, modulus of elasticity. The design engineer should pay particular attention to these limiting parameters when using this procedure to design whitetopping pavement. However, in any case, the procedure was based on whitetopping thicknesses in the range of 4 to about 7 in. For thicknesses above 7 in., it is recommended that the engineer compare whitetopping to a design based on the AASHTO procedure. Also, it is recommended that an AASHTO design be specified if this procedure results in a design whitetopping thickness in excess of 8 in.

Figure 7.0.3 - PCC Thickness Sensitivity to AC Modulus of Elasticity

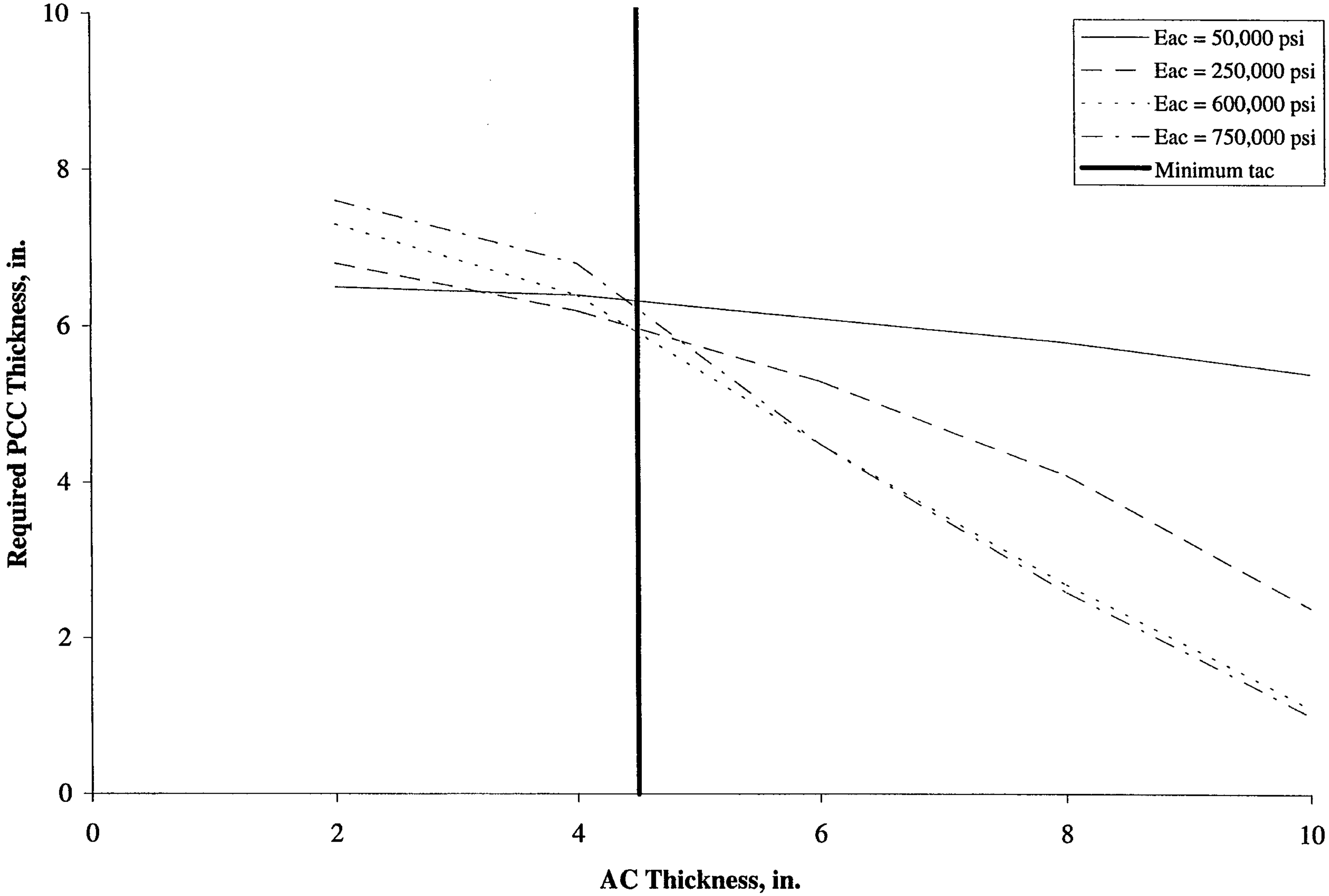


Figure 7.0.4 - PCC Thickness Sensitivity to Concrete Flexural Strength

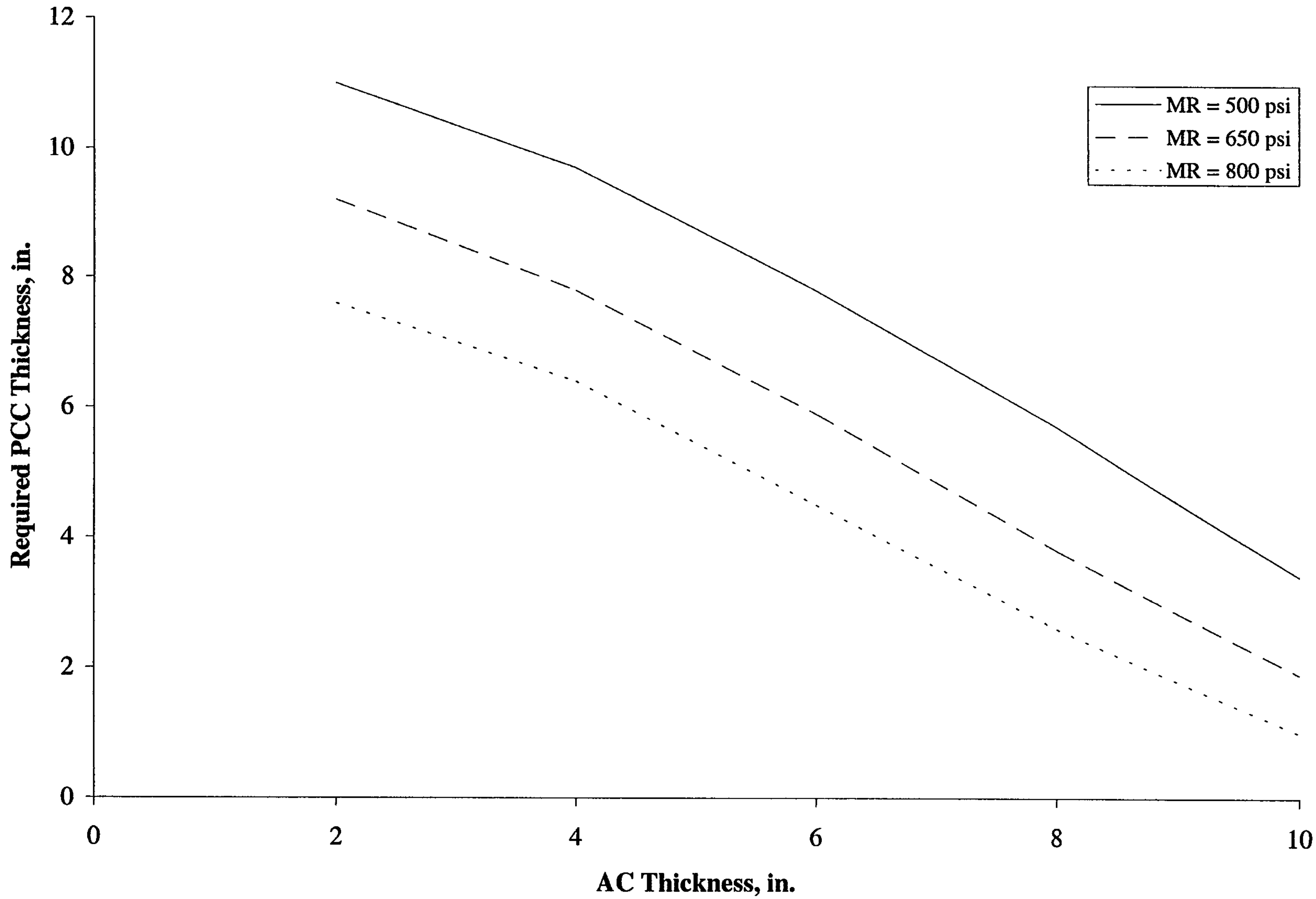


Figure 7.0.5 - PCC Thickness Sensitivity to Temperature Gradient

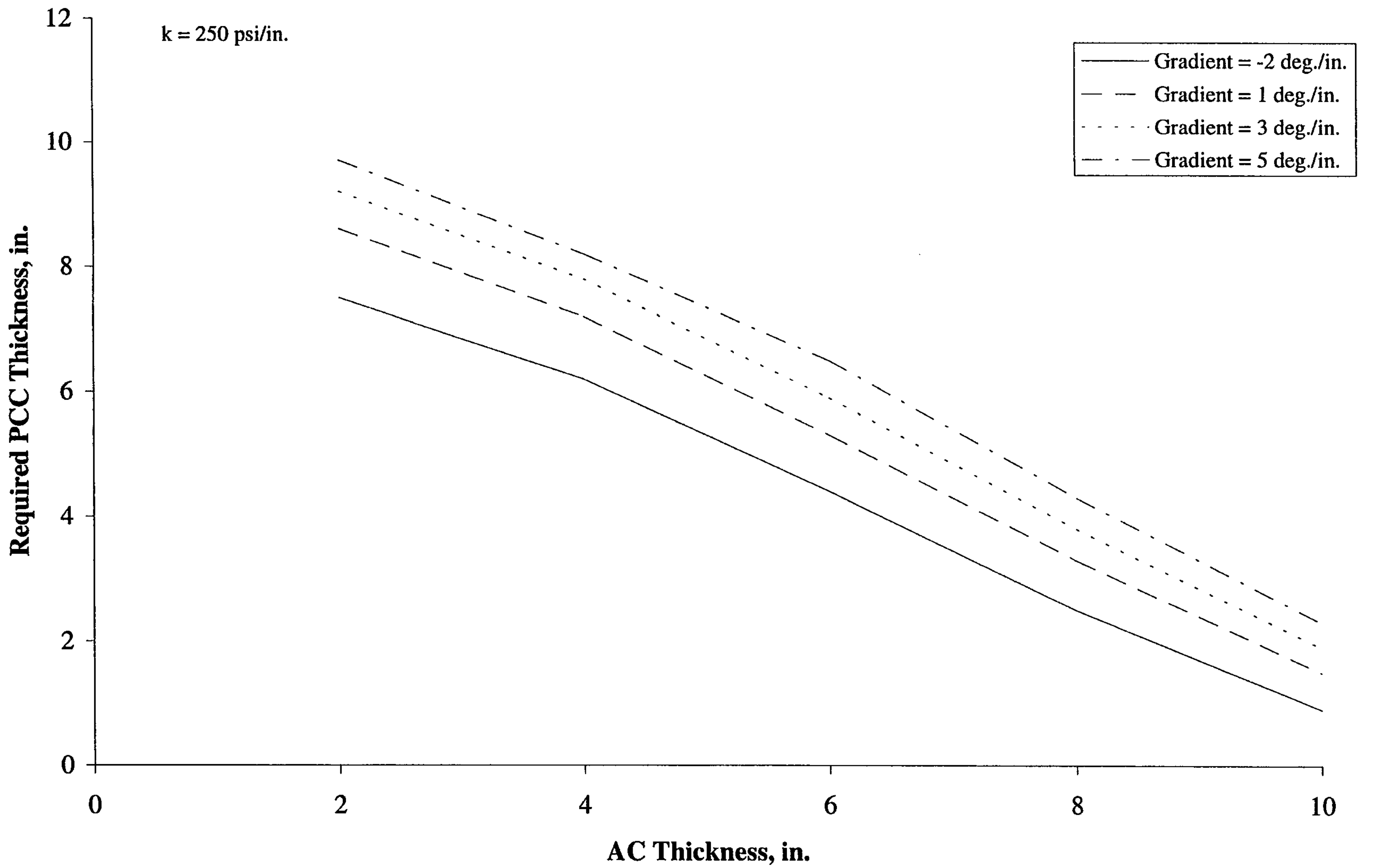


Figure 7.0.6 - PCC Thickness Sensitivity to Asphalt Thickness

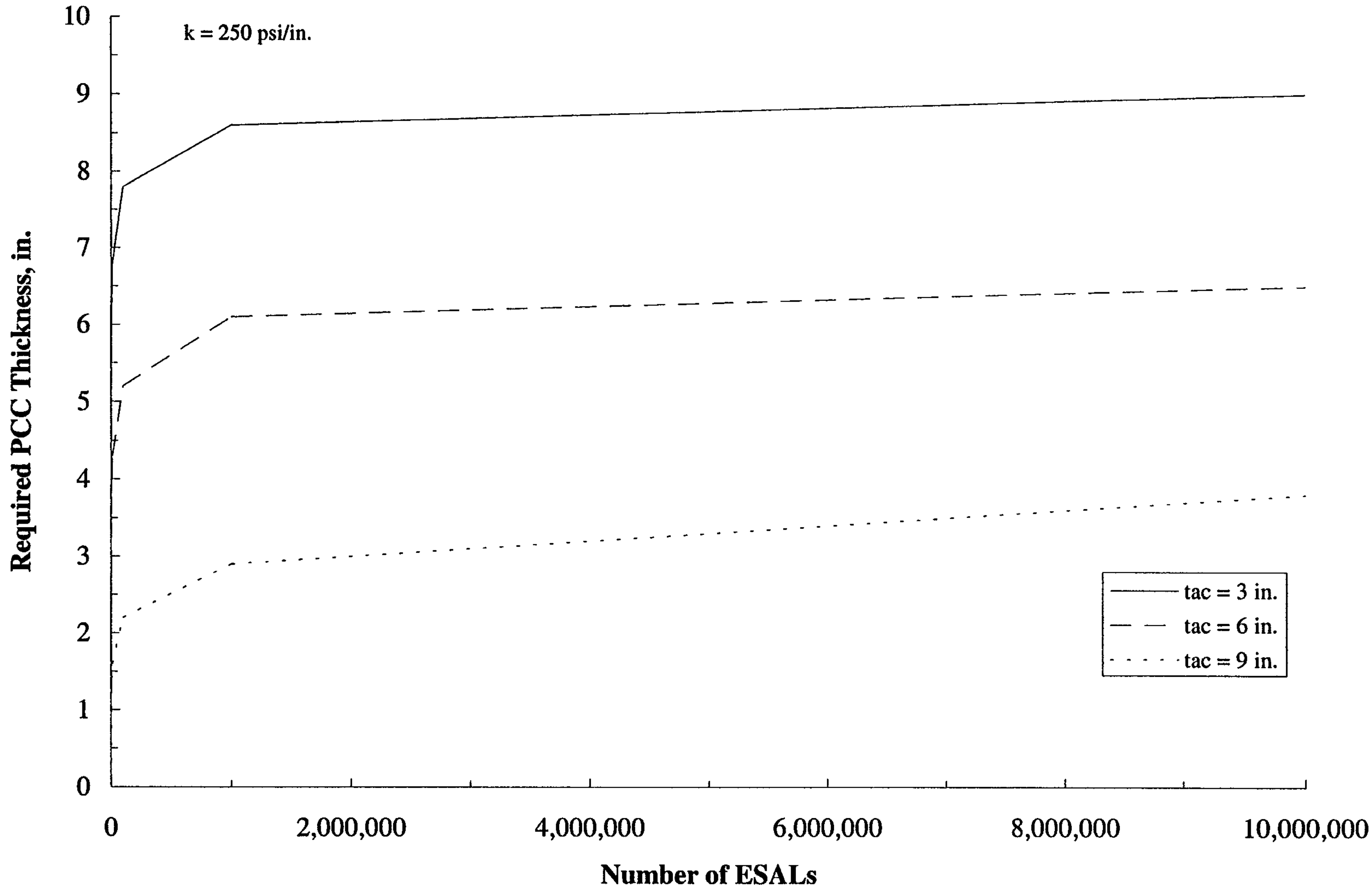


Figure 7.0.7 - PCC Thickness Sensitivity to Asphalt Thickness

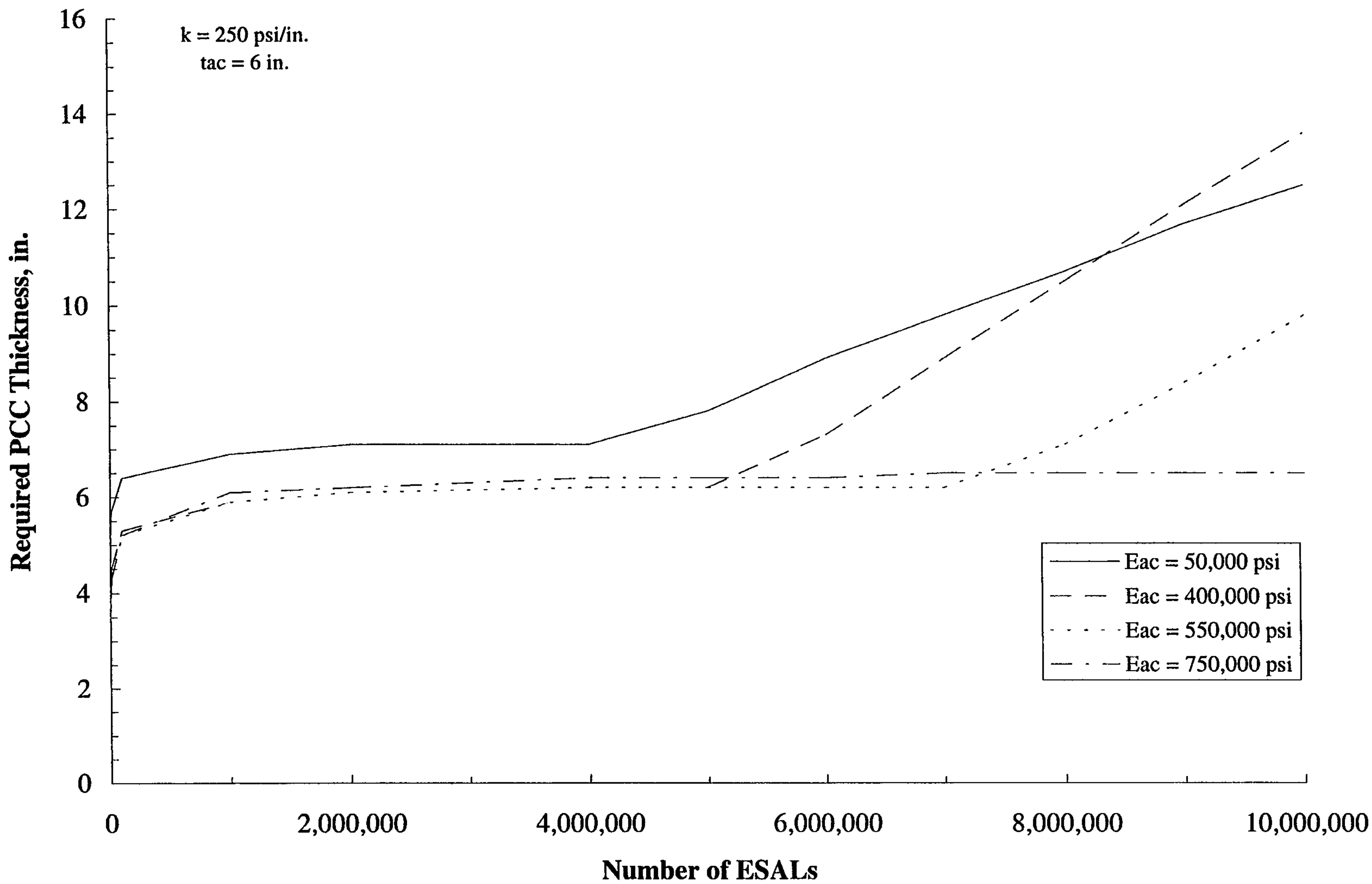
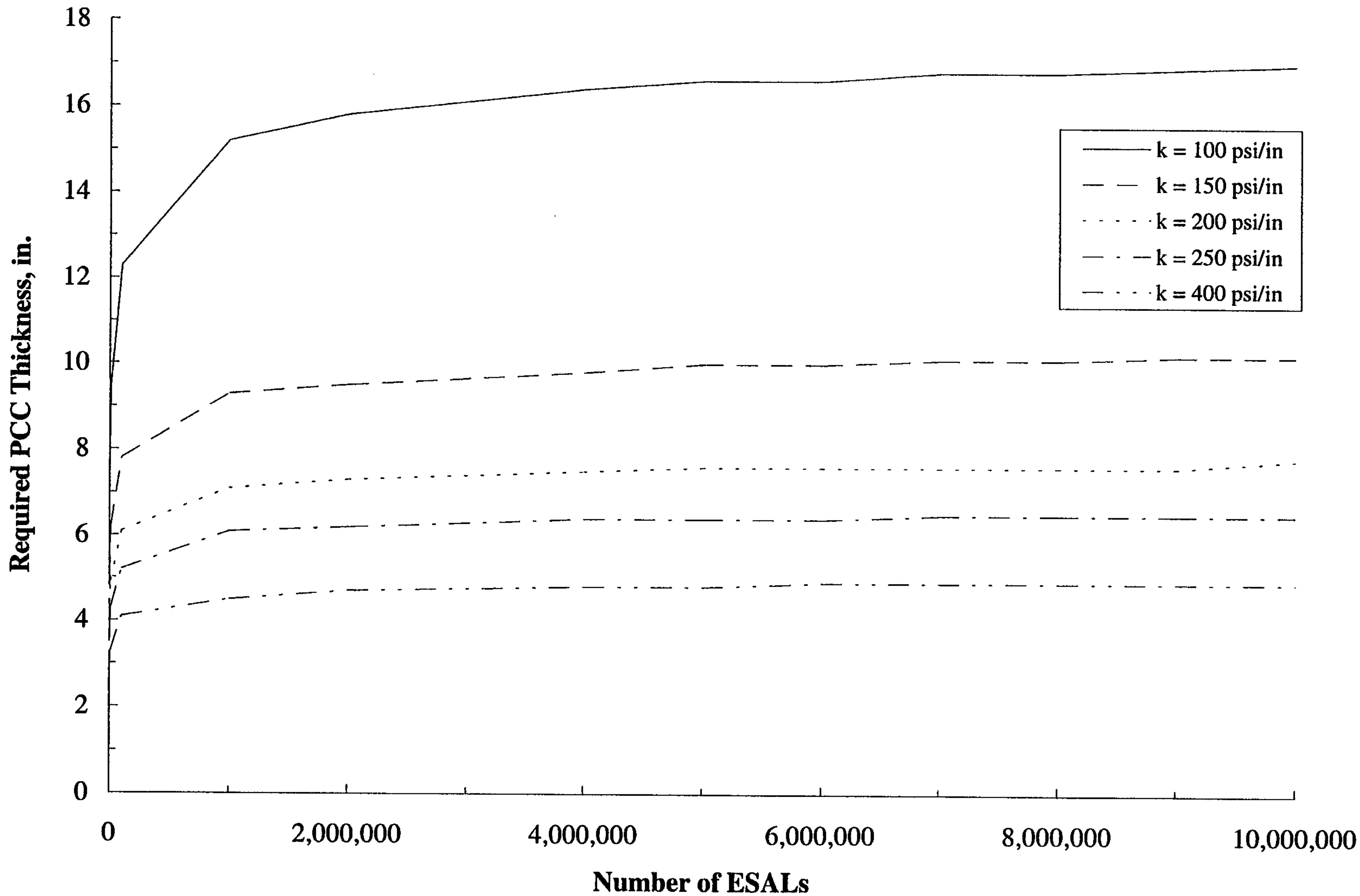


Figure 7.0.8 - PCC Thickness Sensitivity to Modulus of Subbase/Subgrade Reaction



8.0 SUMMARY AND CONCLUSIONS

A mechanistic pavement design procedure for whitetopping pavement was developed through a comprehensive study involving extensive field load testing and theoretical analysis of whitetopping responses. Two types of pavement failure were considered in this 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 developed procedure 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 design examples presented in Tables 5.8.1 and 5.8.2 and Figure 6.2.2 showed that this procedure gave reasonable results. However, the design procedure is viewed as the first generation design method. It should be refined as more field performance data (especially long-term performance data) becomes available.

Based on the field and theoretical analyses conducted during this study, the following conclusions 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 strain (and corresponding stress) in the whitetopping is reduced by approximately 25 percent when the asphalt is milled prior to concrete placement. The opposite was found for new asphalt placed as a whitetopping base. The strain (and corresponding stress) in whitetopping on new asphalt is increased by approximately 50 percent when the asphalt is milled prior to concrete placement.
4. Due to the partial bonding condition, the tensile stress in the bottom of the concrete layer is 65 percent higher than that of a fully bonded slab system.
5. Due to the partial bonding condition, the tensile strain in the bottom of the asphalt layer is approximately 15 percent lower than that of a fully bonded slab system.
6. At joint spacing greater than 48 in., 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.
7. Based on data collected, it appears that whitetopping pavements should be constructed when the modulus of subbase/subgrade reaction exceeds 150 pci.
8. A minimum asphalt thickness of 5 in. (after milling) is recommended for whitetopping pavement.
9. The current design procedure does not appear to be highly sensitive to the number of expected ESALs. However, this should be verified by long-term experience.

The methods outlined in this report are intended as a first generation whitetopping design procedure. The following recommendations for future work are provided:

1. This method was developed based on information collected during three field evaluations. While an attempt was made to study a range of parameters, it is highly recommended that further studies be conducted to validate the design procedure outlined.
2. The effect of partial bonding on concrete stress and asphalt strain should be studied further. The effects of milling need to be investigated further. An increase in the bonding of the layers will lower the 65 percent stress increase currently incorporated into the design procedure. Likewise, the asphalt strain may be affected (positively or negatively) by an increased interface bond strength.
3. The effect of load transfer devices were investigated during this study. No significant effects were observed in the newly constructed whitetopping pavements. However, load transfer devices will effect the performance of the pavement if the asphalt deteriorates or the amount of curling in the concrete layer becomes excessive. These are long term processes which should be monitored.
4. The maximum joint spacing could not adequately be determined due to the relatively few joint spacings evaluated. In fact, the joint spacing variable could not be included in the equations developed to calculate the maximum stress. At this time, a joint spacing of 72 in. seems reasonable.
5. The effects of lower moduli of subbase/subgrade reaction should be investigated further. The minimum subgrade modulus studied during this evaluation was 150 pci. It is recommended that lower moduli be investigated.
6. The long-term performance of whitetopping should be evaluated. Periodic surveys of whitetopping pavements should be performed to document the occurrence of distress. This information can be incorporated into the design procedure once it becomes available.

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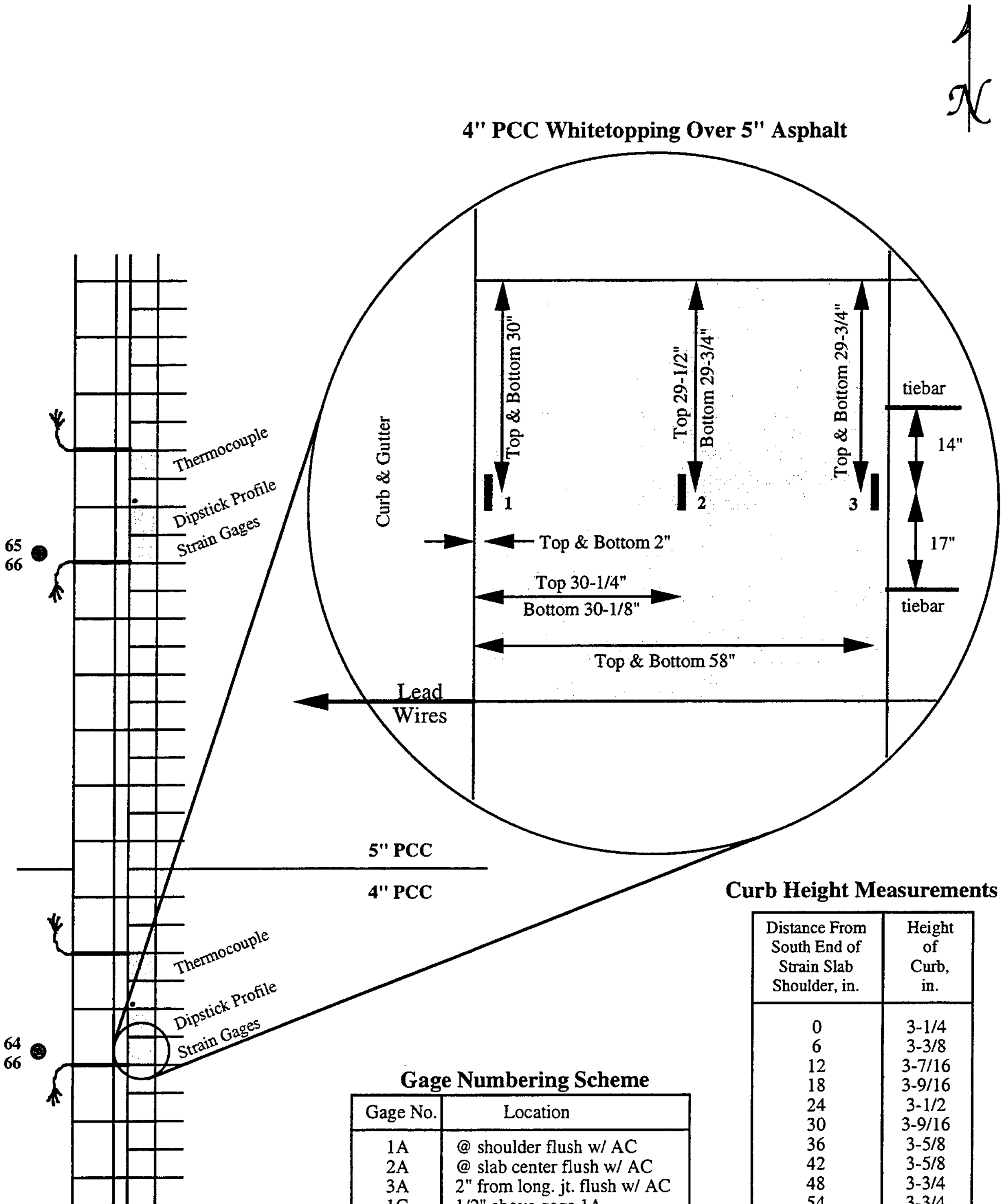
Appendix A:
CDOT1 Layout, Photos, Temperatures, and Profiles



Curb Height Measurements

Distance From South End of Strain Slab Shoulder, in.	Height of Curb, in.
0	3-1/4
6	3-3/8
12	3-7/16
18	3-9/16
24	3-1/2
30	3-9/16
36	3-5/8
42	3-5/8
48	3-3/4
54	3-3/4
60	3-13/16

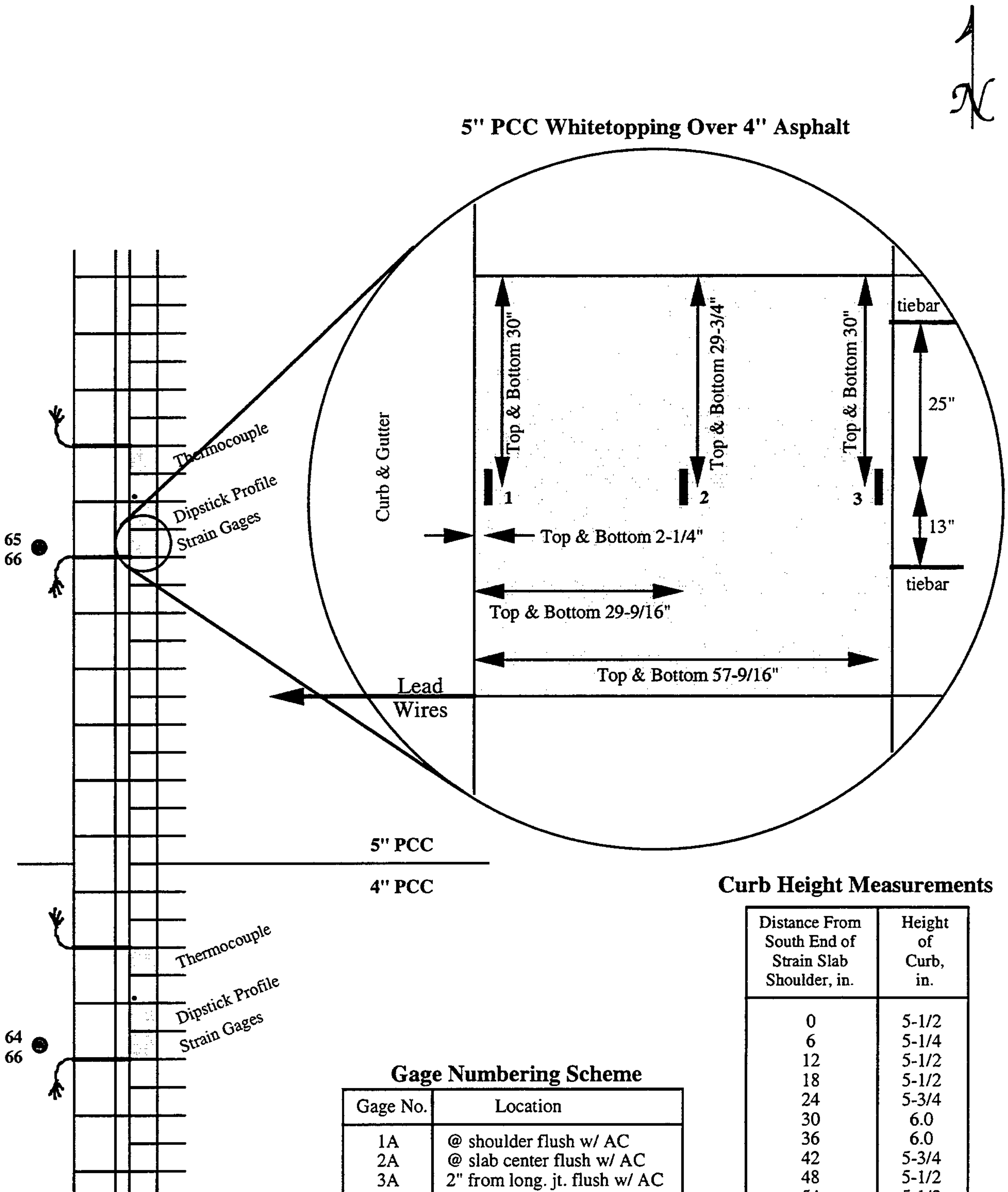
Gage Numbering Scheme

Gage No.	Location
1A	@ shoulder flush w/ AC
2A	@ slab center flush w/ AC
3A	2" from long. jt. flush w/ AC
1C	1/2" above gage 1A
2C	1/2" above gage 2A
3C	1/2" above gage 3A

*Note: All tiebars used at longitudinal joints - 30"

Figure A.1 - Test Slab 1 Layout for CDOT Project #1.

5" PCC Whitetopping Over 4" Asphalt



Curb Height Measurements

Distance From South End of Strain Slab Shoulder, in.	Height of Curb, in.
0	5-1/2
6	5-1/4
12	5-1/2
18	5-1/2
24	5-3/4
30	6.0
36	6.0
42	5-3/4
48	5-1/2
54	5-1/2
60	5-1/2

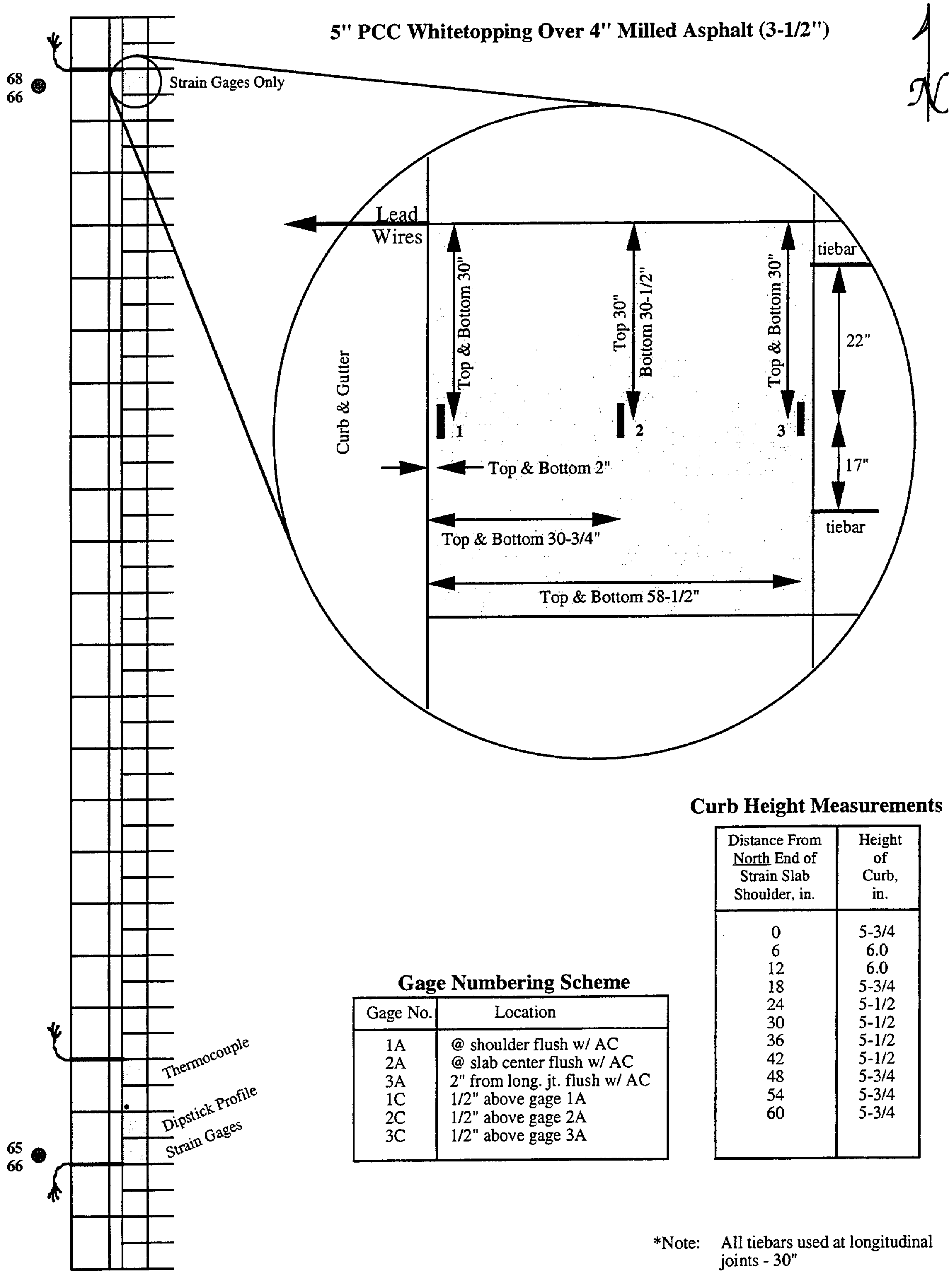
Gage Numbering Scheme

Gage No.	Location
1A	@ shoulder flush w/ AC
2A	@ slab center flush w/ AC
3A	2" from long. jt. flush w/ AC
1C	1/2" above gage 1A
2C	1/2" above gage 2A
3C	1/2" above gage 3A

*Note: All tiebars used at longitudinal joints - 30"

Figure A.2 - Test Slab 2 Layout for CDOT Project #1.

5" PCC Whitetopping Over 4" Milled Asphalt (3-1/2")



Curb Height Measurements

Distance From North End of Strain Slab Shoulder, in.	Height of Curb, in.
0	5-3/4
6	6.0
12	6.0
18	5-3/4
24	5-1/2
30	5-1/2
36	5-1/2
42	5-1/2
48	5-3/4
54	5-3/4
60	5-3/4

Gage Numbering Scheme

Gage No.	Location
1A	@ shoulder flush w/ AC
2A	@ slab center flush w/ AC
3A	2" from long. jt. flush w/ AC
1C	1/2" above gage 1A
2C	1/2" above gage 2A
3C	1/2" above gage 3A

*Note: All tiebars used at longitudinal joints - 30"

Figure A.3 - Test Slab 3 Layout for CDOT Project #1.

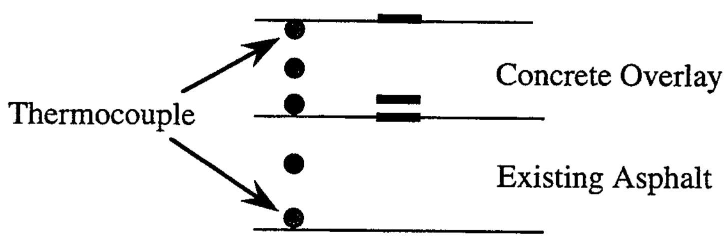


Figure A.4 - Typical Section View of Strain Gage and Thermocouple Layout

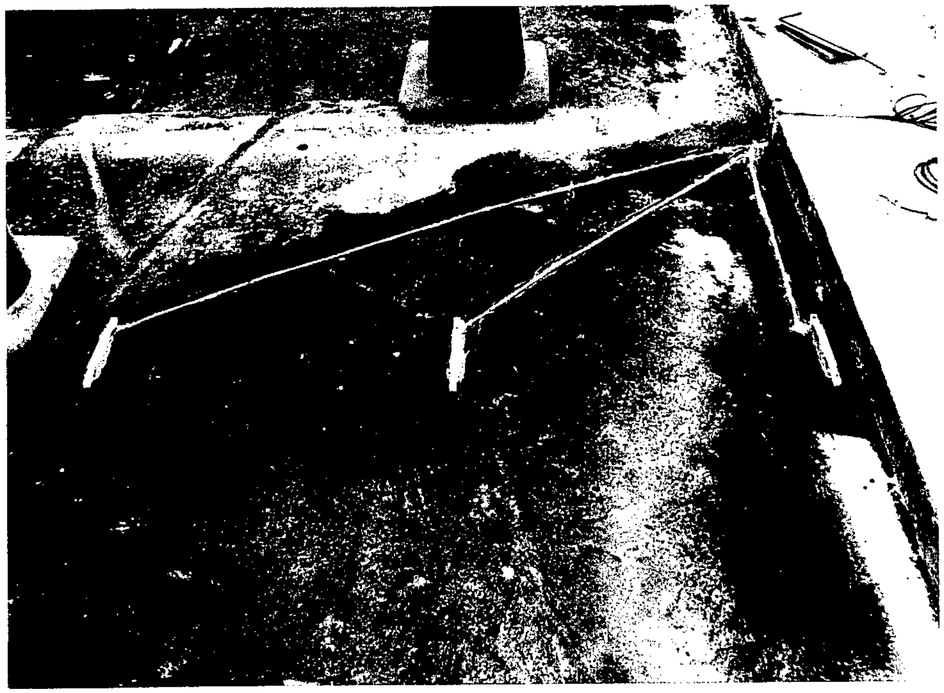


Figure A.5 - Asphalt Surface and Concrete Embedment Gages Prior to Concrete Placement

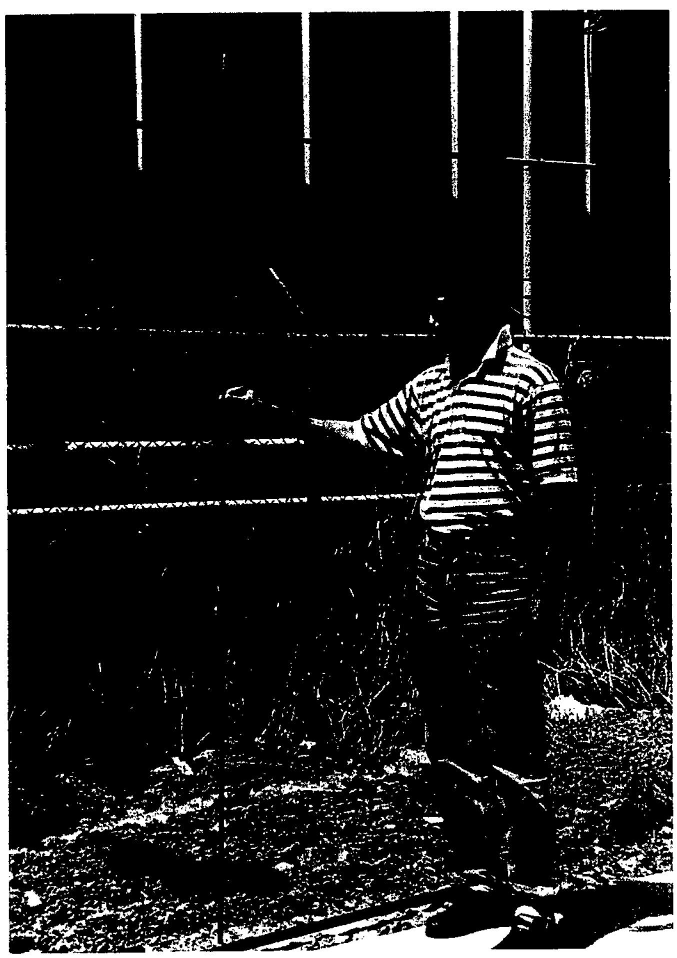


Figure A.6 - Typical Invar Rod Used for Profile Elevation Reference Prior to Embedment

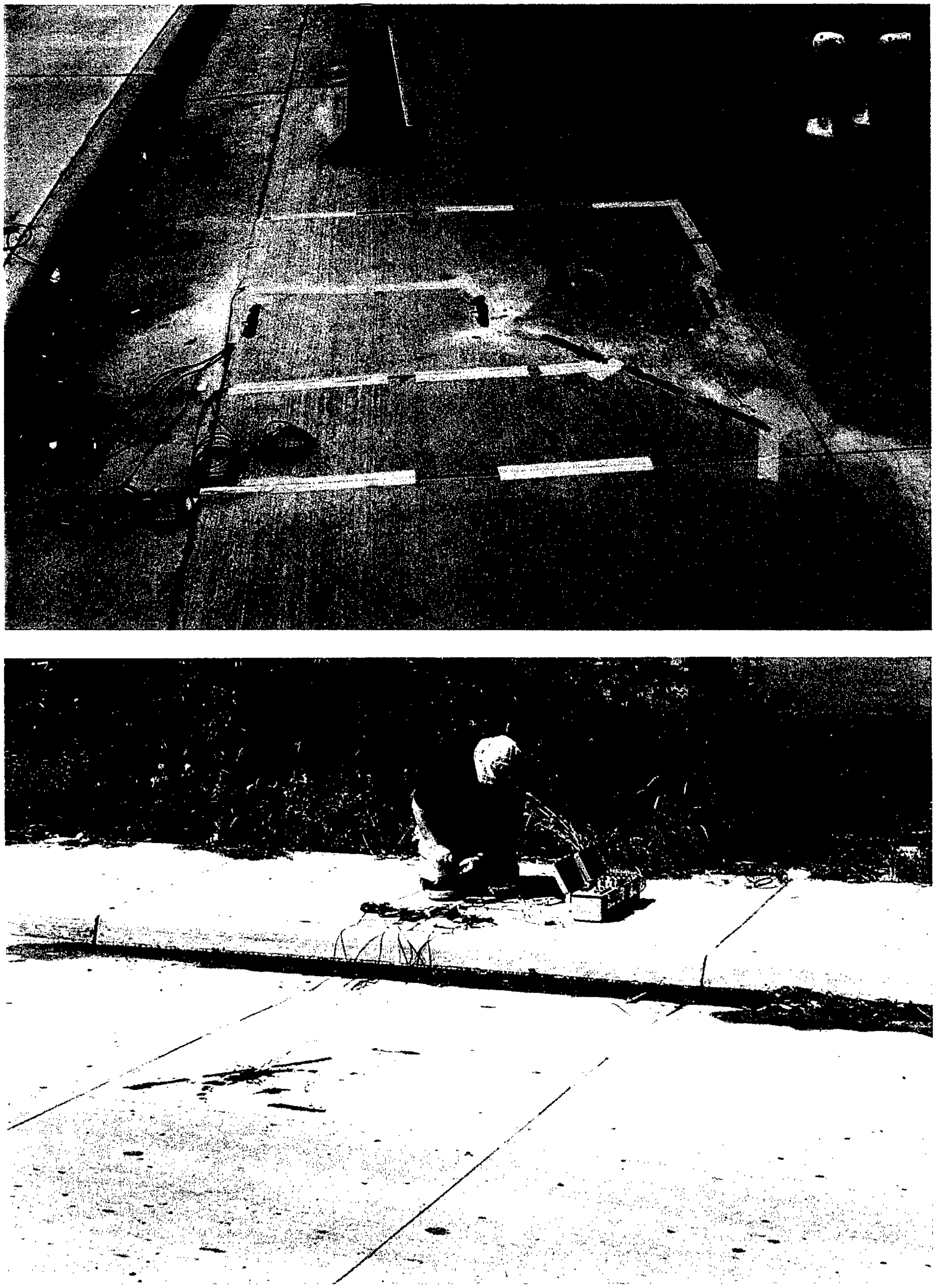
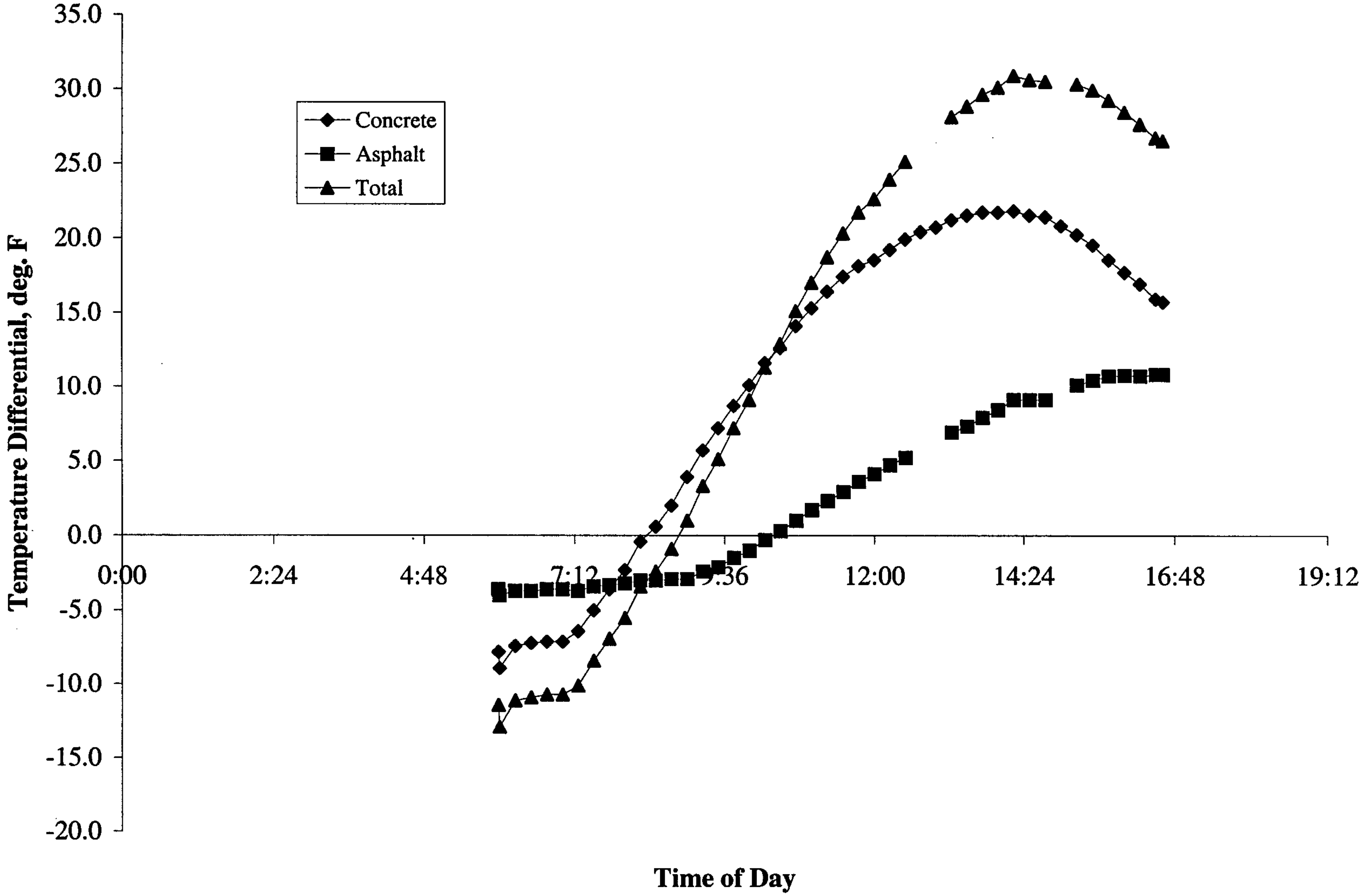


Figure A.7 - Typical Strain Gage Layout and Load Testing Setup at CDOT Project #1

Figure A.8 - Load Testing at CDOT Project #1

Figure A.9 - Typical Placement of Truck Tires Adjacent to Strain Gages for Load Testing

Figure A.10 - Typical Temperature Gradient in CDOT1 Pavement Layers



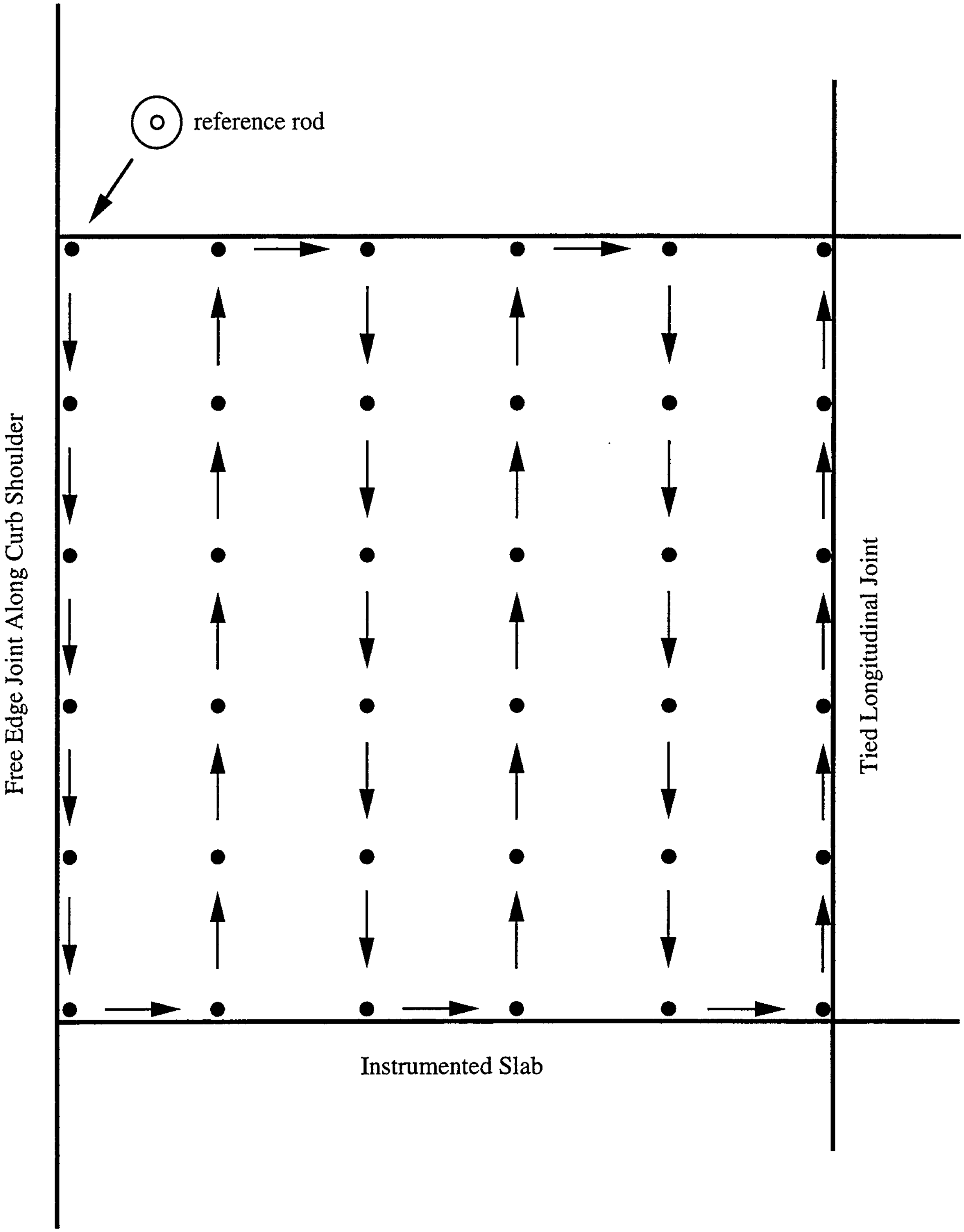


Figure A.11 - Typical Dipstick Profile Layout for CDOT Project #1.

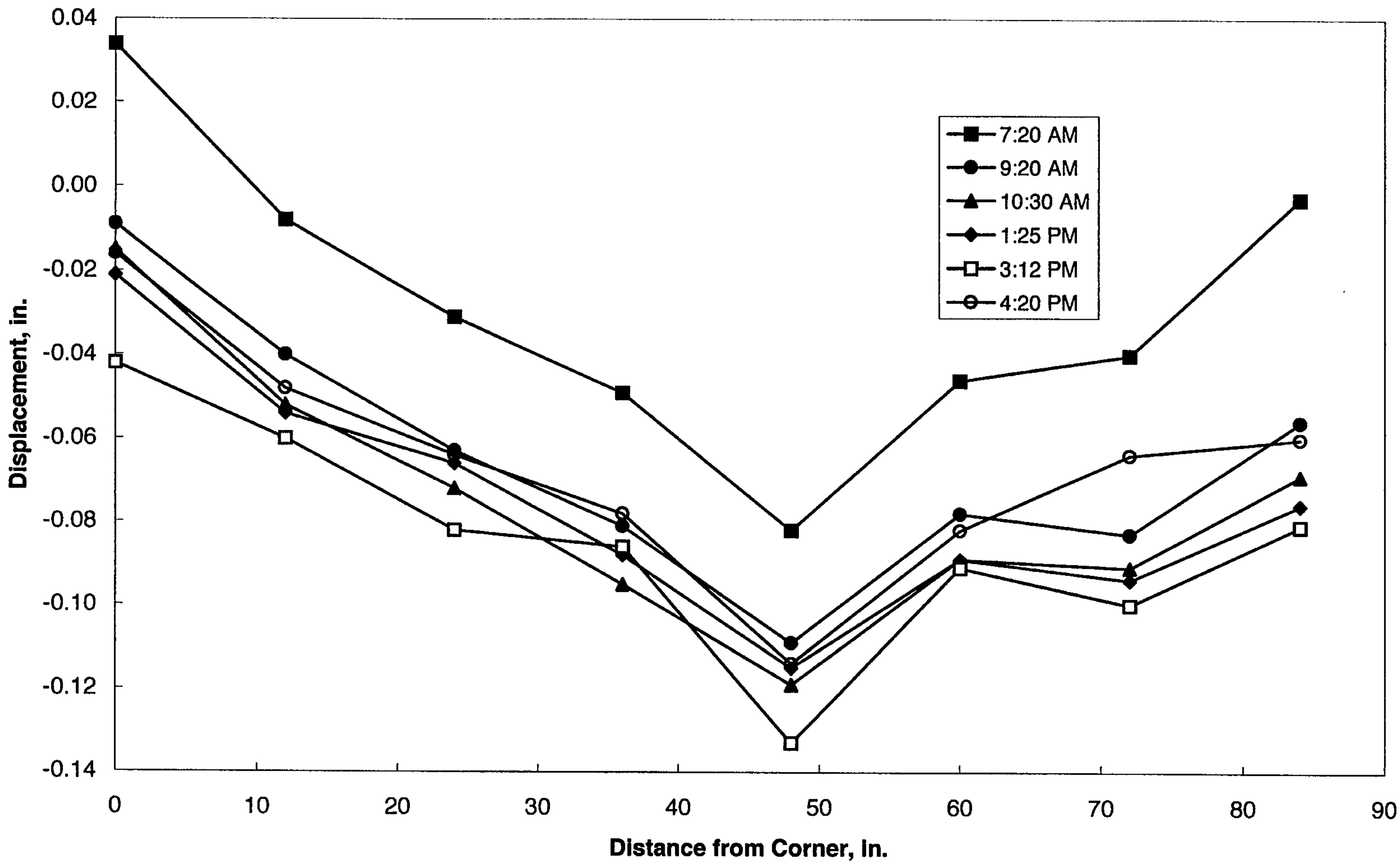


Figure A.12 - Typical Surface Profiles Along 4-in. Slab Diagonal, CDOT Project #1 28-Day Testing

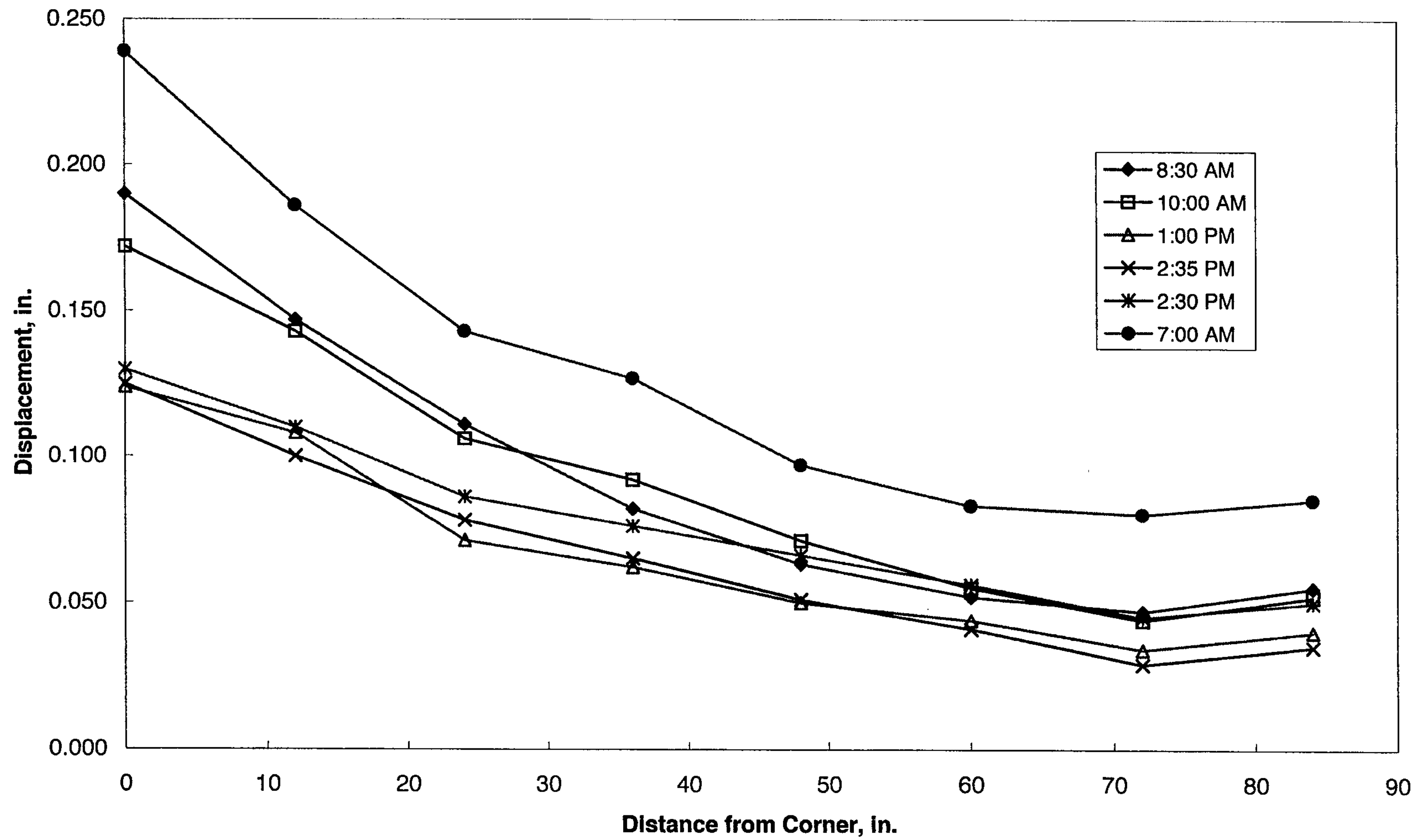


Figure A.13 - Typical Surface Profiles Along 5-in. Slab Diagonal, CDOT Project #1 28-Day Testing

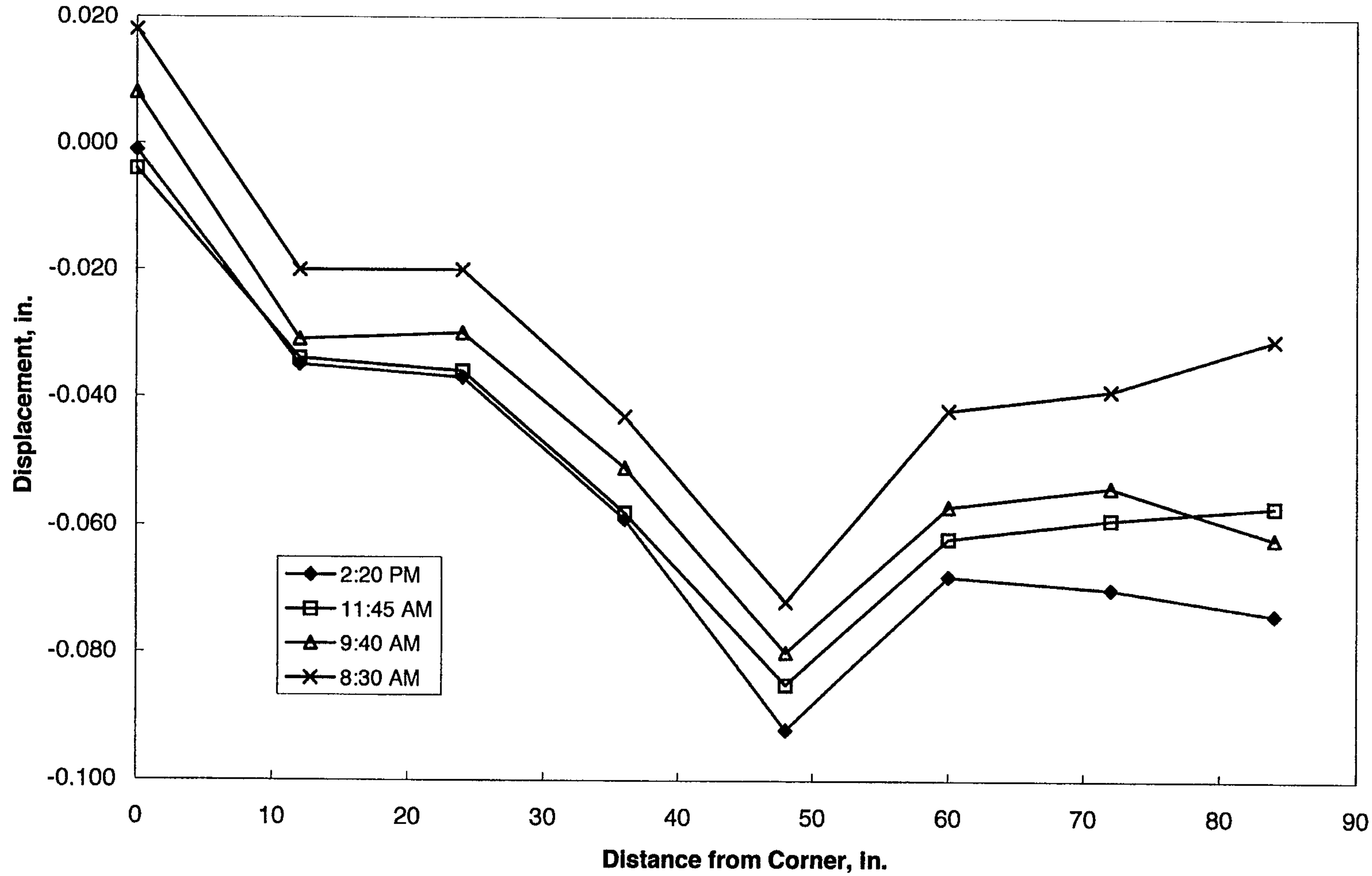


Figure A.14 - Typical Surface Profiles Along 4-in. Slab Diagonal, CDOT Project #1 365-Day Testing

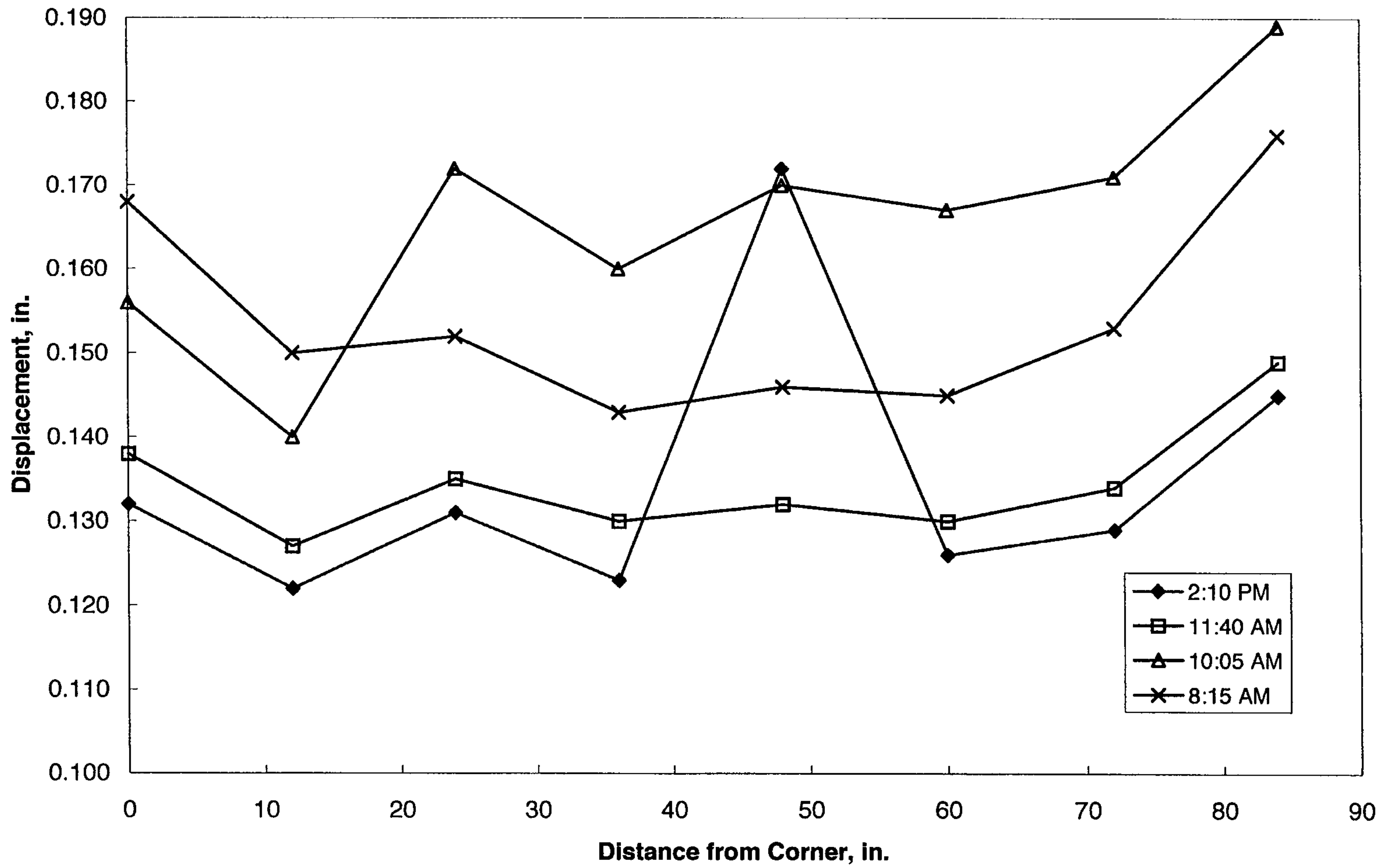


Figure A.15 - Typical Suface Profiles Along 5-in. Slab Diagonal, CDOT Project #1 365-Day Testing

Appendix B:
CDOT2 Layout, Photos, Temperatures, and Profiles

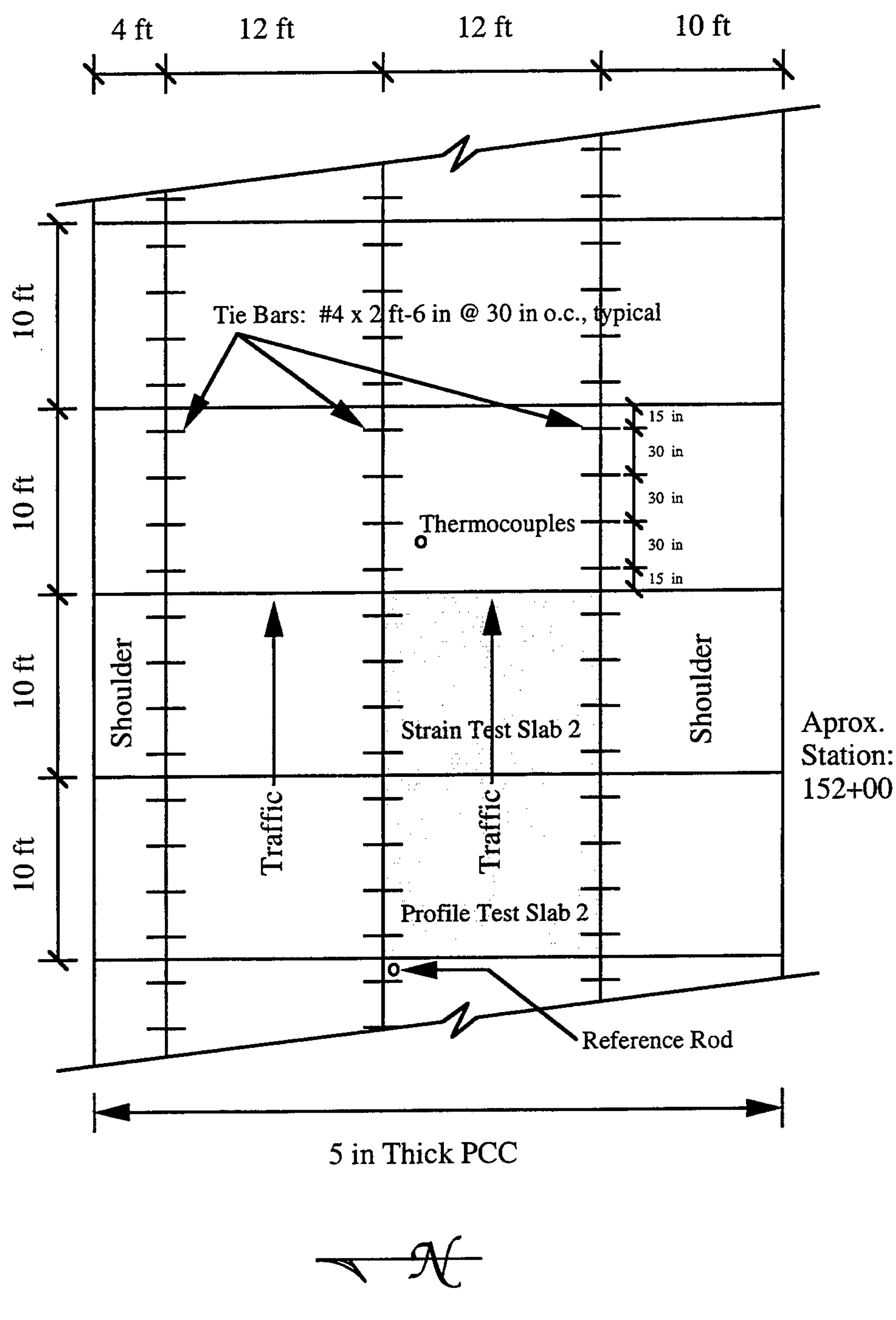


Figure B.1 - Test Slab 2 Layout for CDOT Project #2.

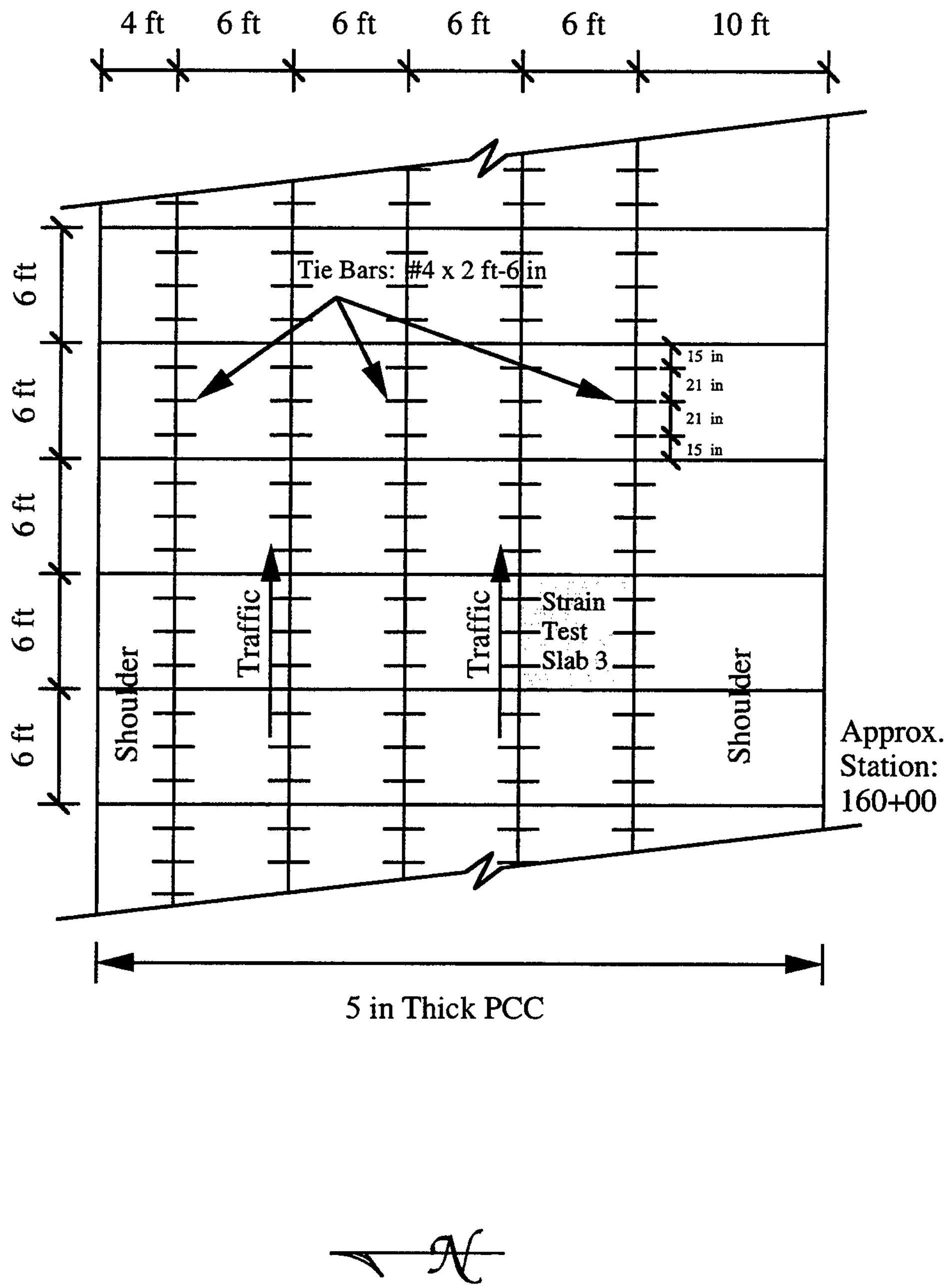


Figure B.2 - Test Slab 3 Layout for CDOT Project #2.

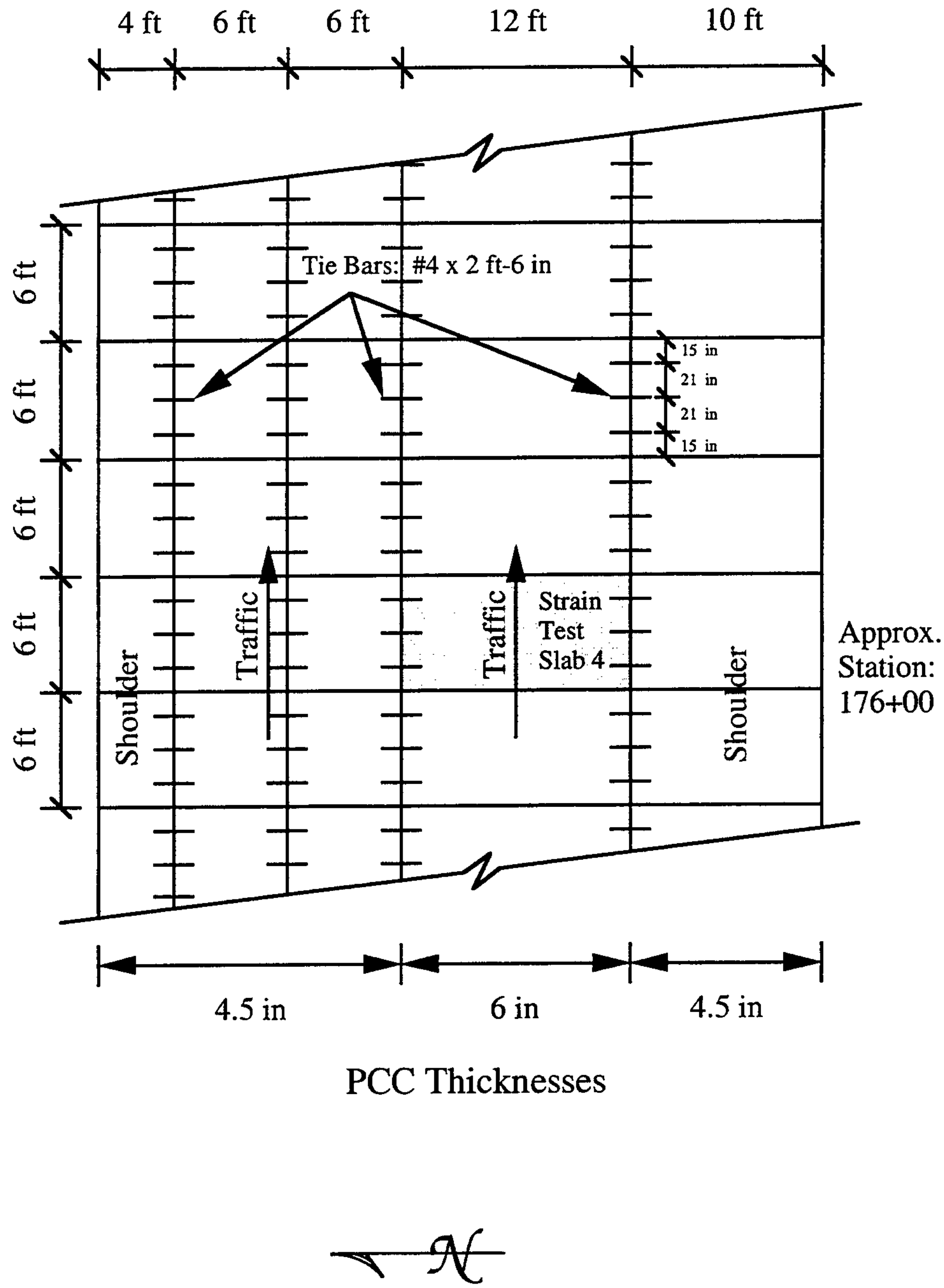


Figure B.3 - Test Slab 4 Layout for CDOT Project #2.

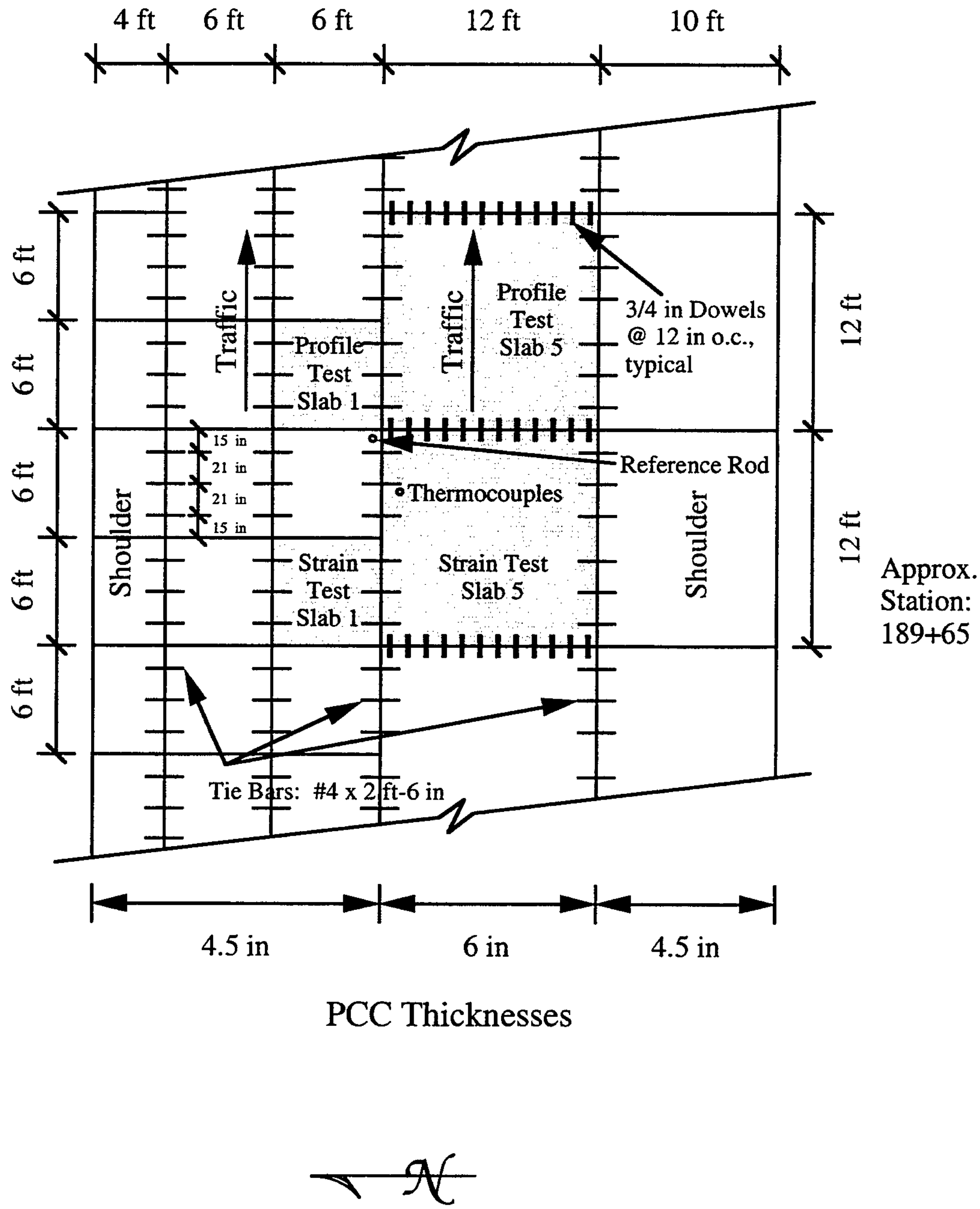
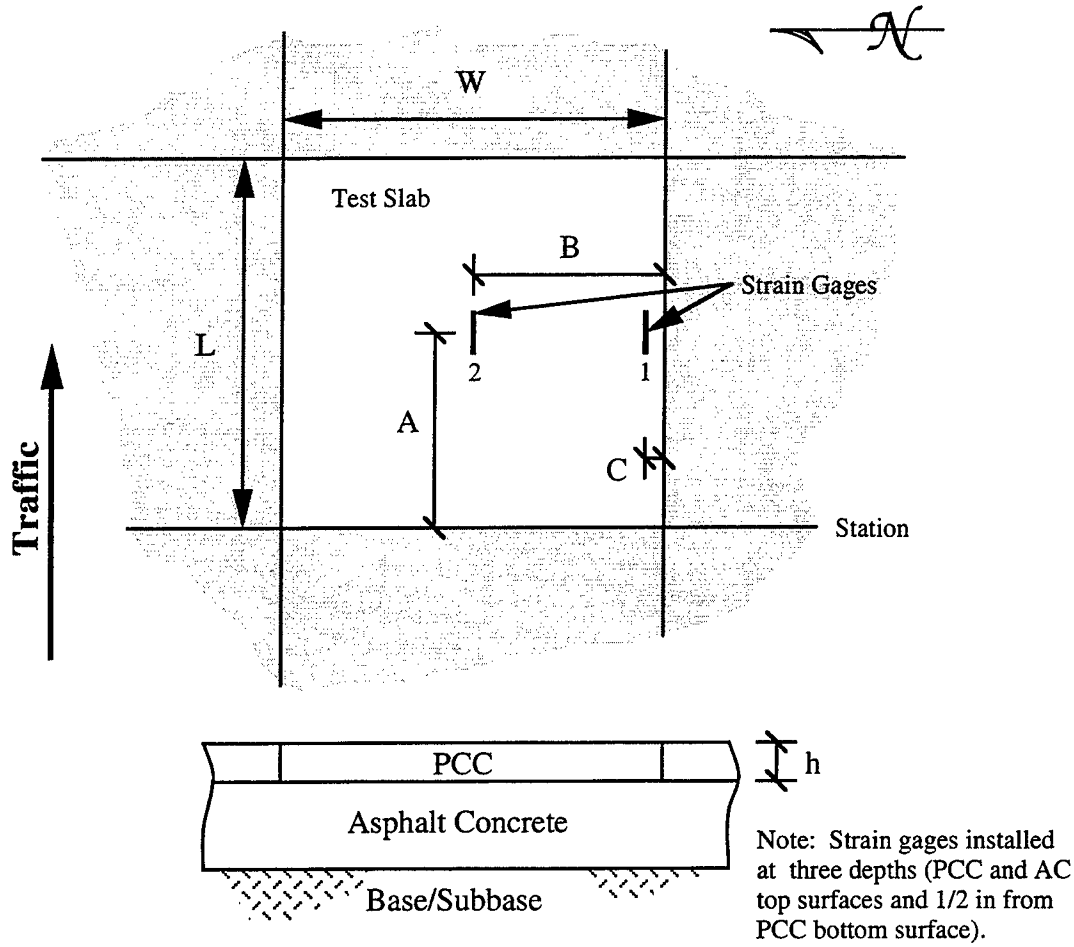


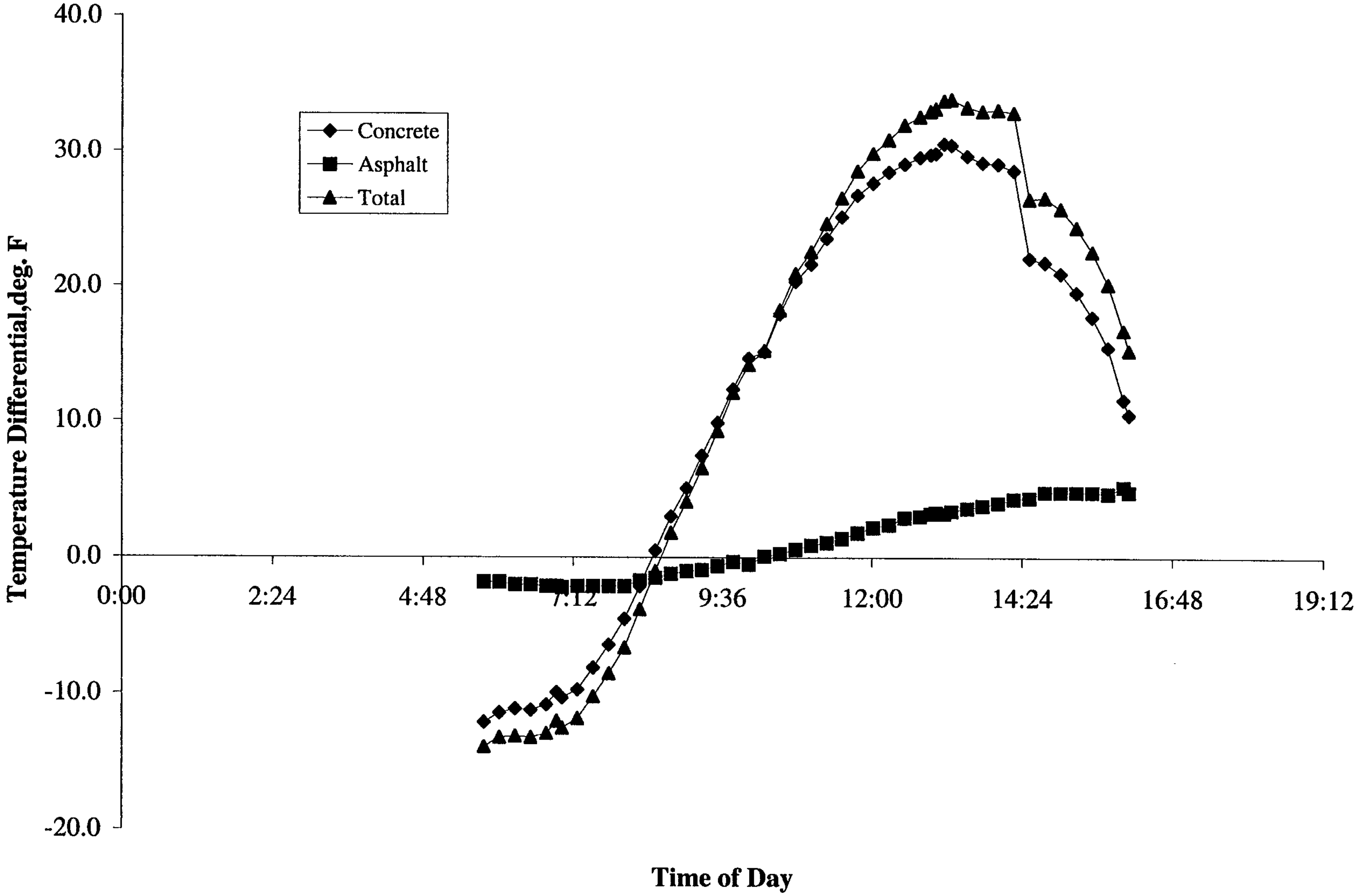
Figure B.4 - Test Slabs 1 and 5 Layout for CDOT Project #2.



Test Slab	Approximate Station	Design Dimensions: Thickness, in (h)	Design Dimensions: Width, in (W)	Design Dimensions: Length, in (L)	As-built Dimensions: Width, in (W)	As-built Dimensions: Length, in (L)	Strain Gage Location Dimensions: A, in	Strain Gage Location Dimensions: B, in	Strain Gage Location Dimensions: C, in
1	189+65	4.5	72.0	72.0	73.0	73.0	45.0	36.0	2.0
2	152+00	5	120.0	144.0	149.5	121.0	60.0	77.0	7.5
3	160+00	5	72.0	72.0	74.5	71.5	36.0	41.0	8.0
4	176+00	6	72.0	144.0	149.75	72.0	36.0	78.25	8.0
5	789+65	6	144.0	144.0	149.75	154.5	73.0	78.25	8.25

Figure B.5 - Slab Dimensions and Strain Gage Locations for CDOT Project #2.

Figure B.6 - Typical Temperature Gradient in CDOT2 Pavement Layers



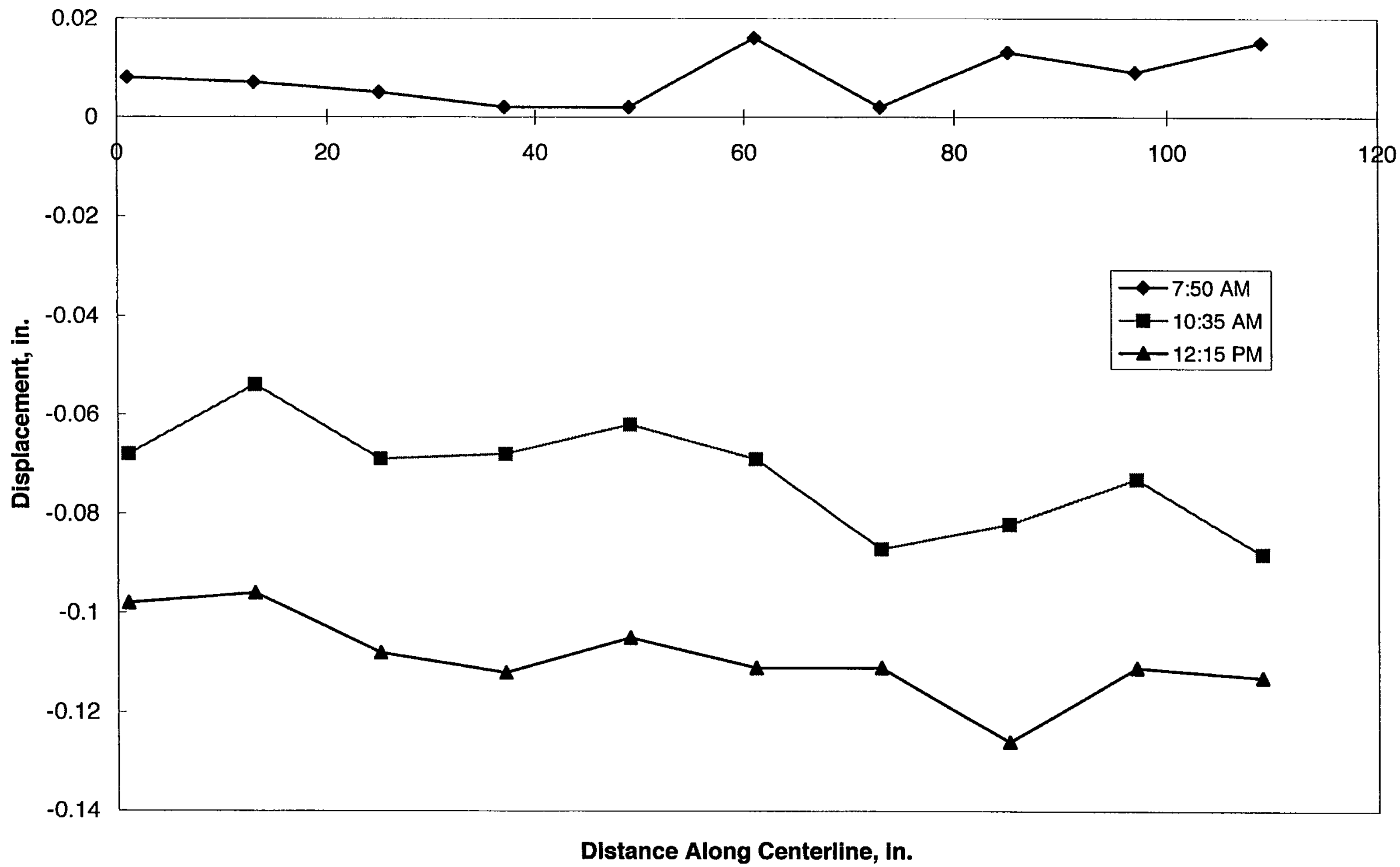


Figure B.14 - Typical Surface Profiles Along Centerline of Slab 2, CDOT Project #2 28-Day Testing

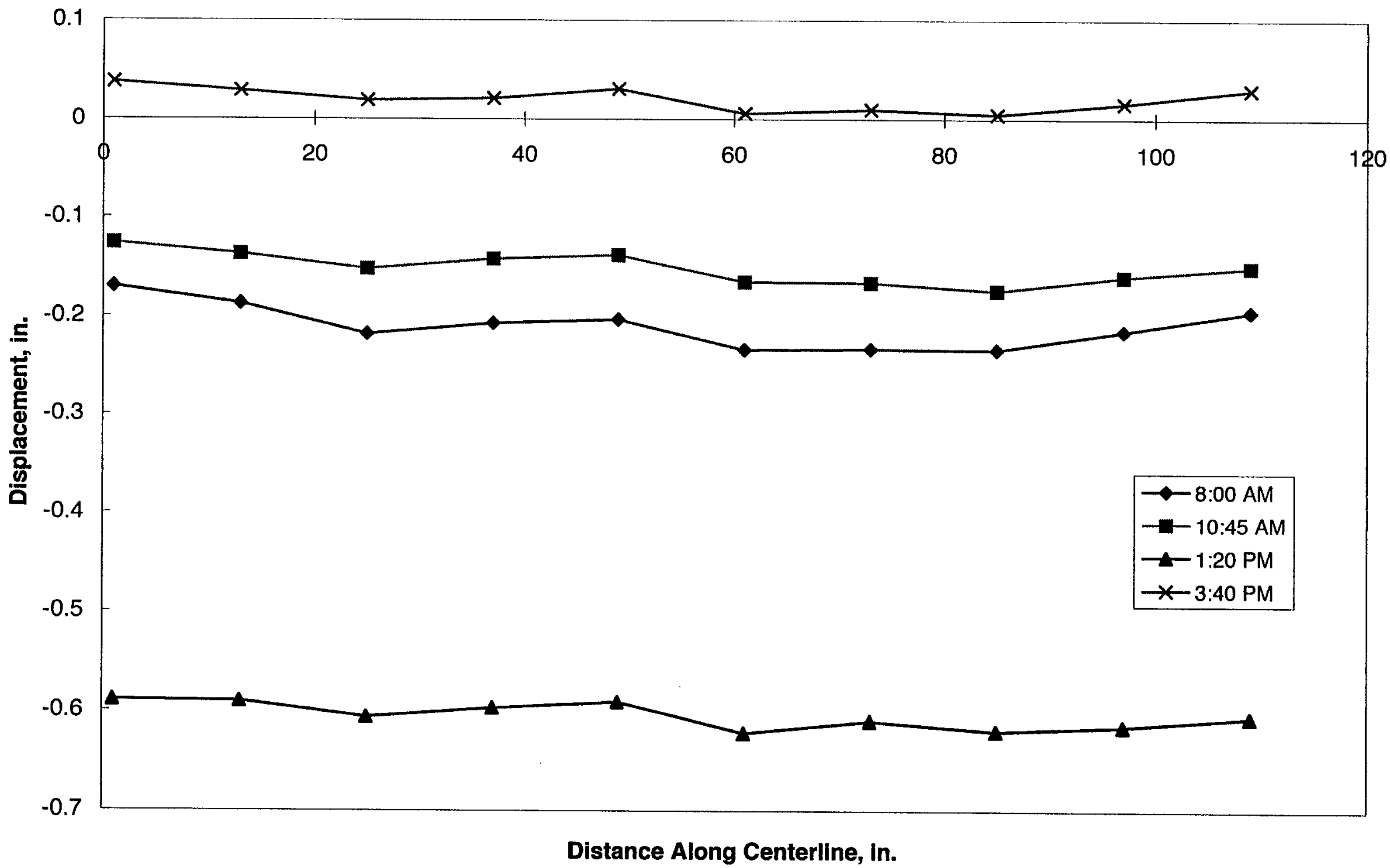


Figure B.15 - Typical Surface Profiles Along Centerline of Slab 2, CDOT Project #2 365-Day Testing

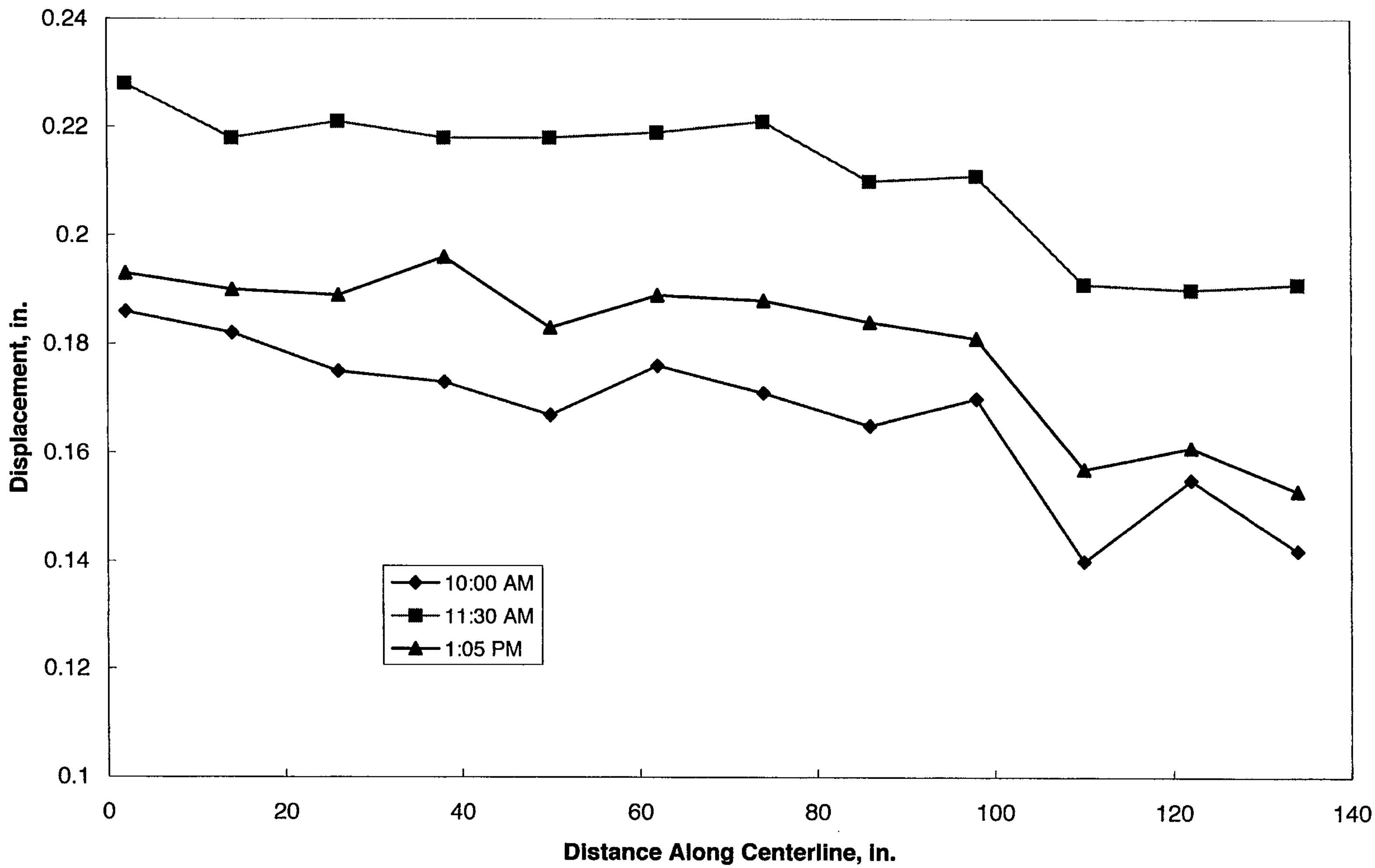


Figure B.16 - Typical Surface Profiles Along Centerline of Slab 5, CDOT Project #2 28-Day Testing

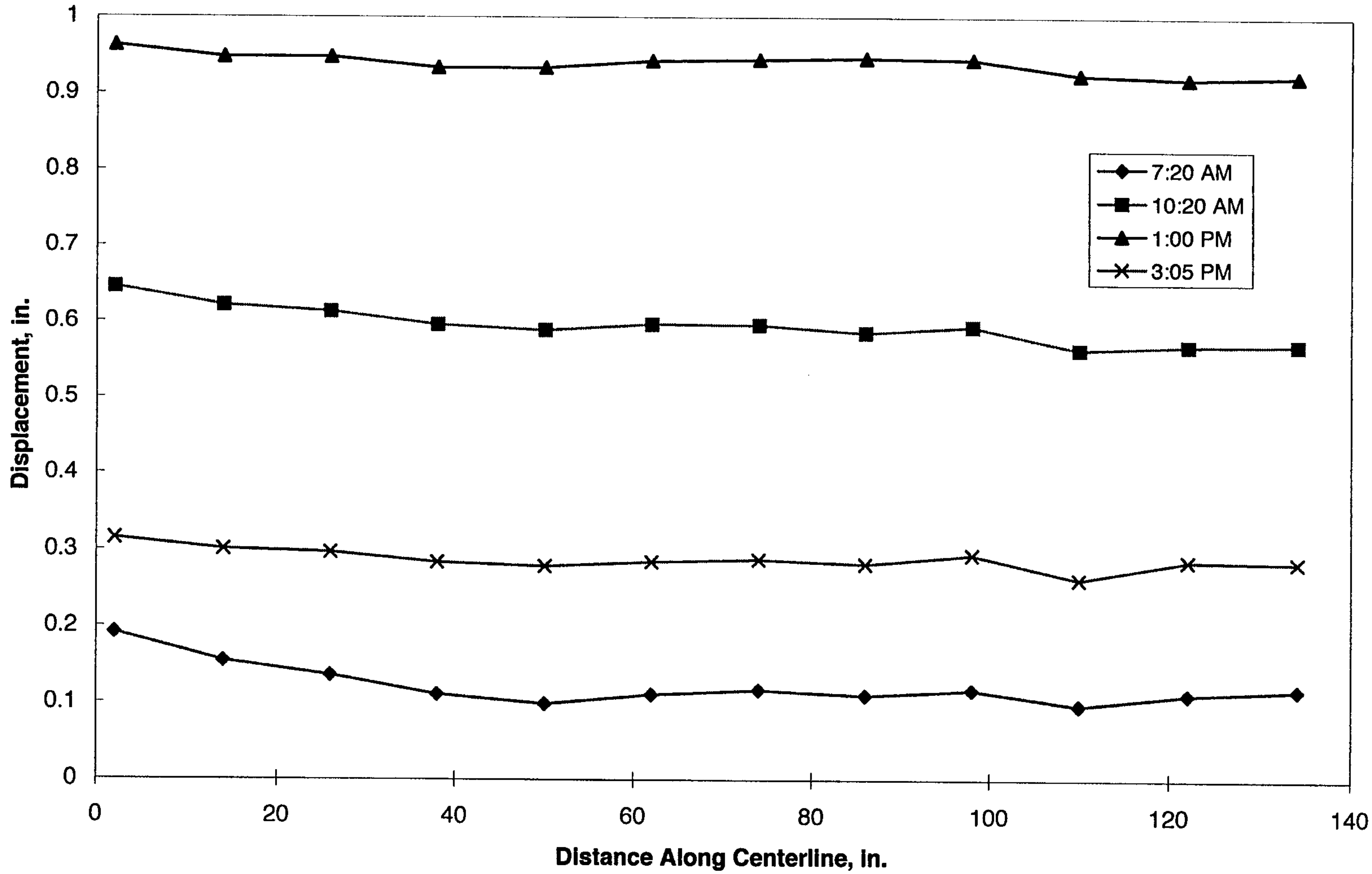


Figure B.17 - Typical Surface Profiles Along Centerline of Slab 5, CDOT Project #2 365-Day Testing

Appendix C:
CDOT3 Layout, Photos, Temperatures, and Profiles

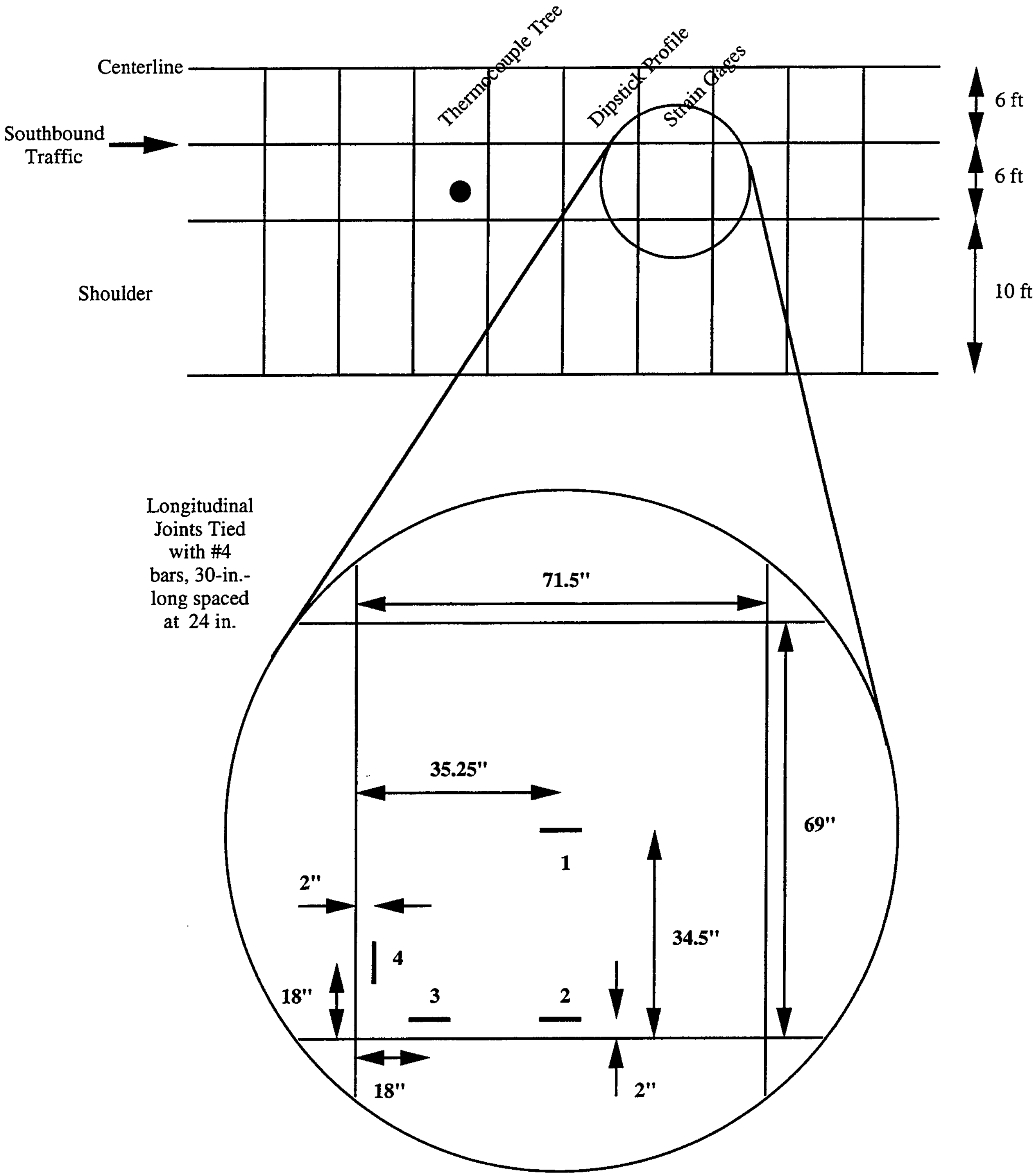


Figure C.1 - Test Slab F Layout for CDOT Project #3.

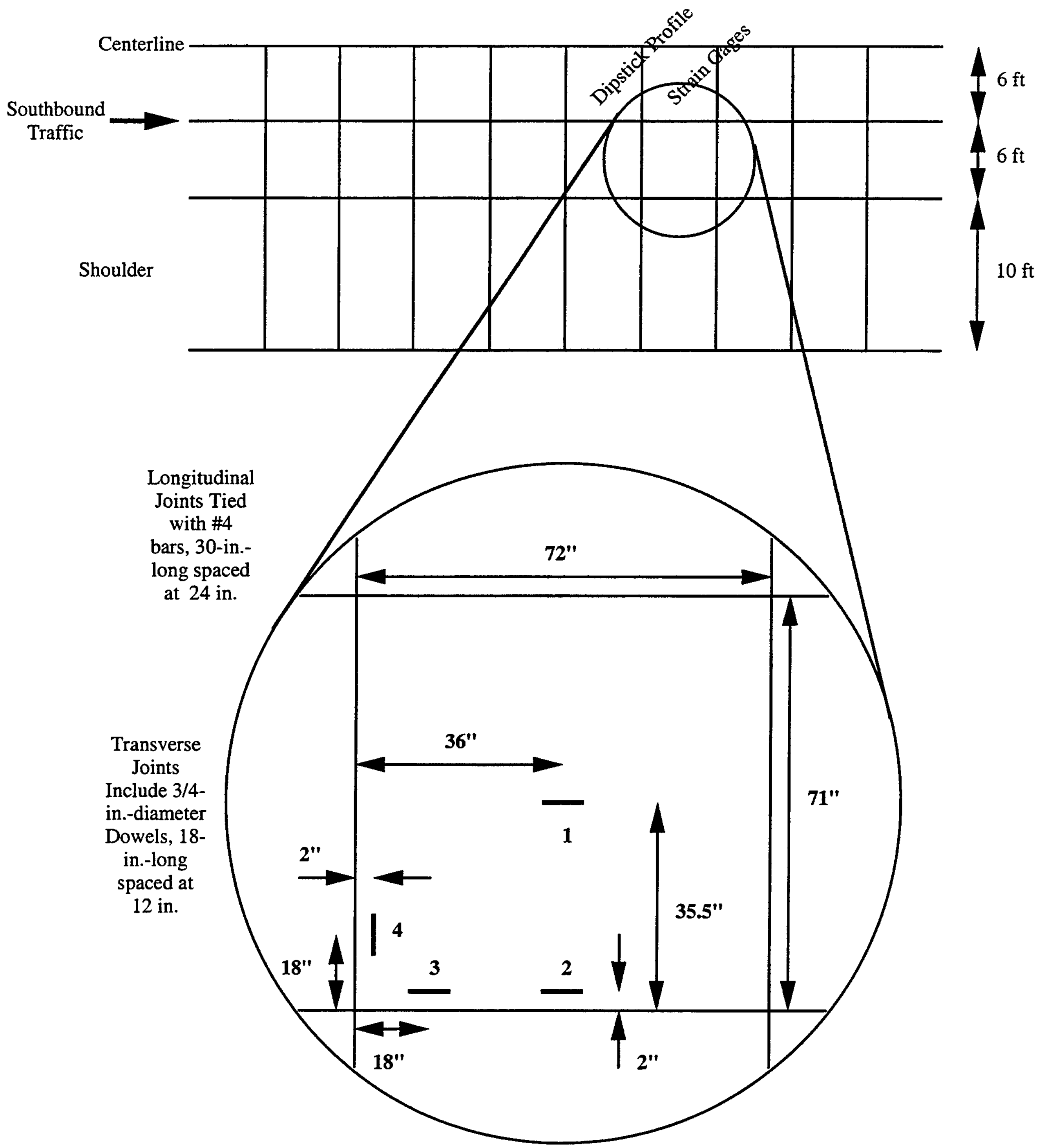


Figure C.2 - Test Slab E Layout for CDOT Project #3.

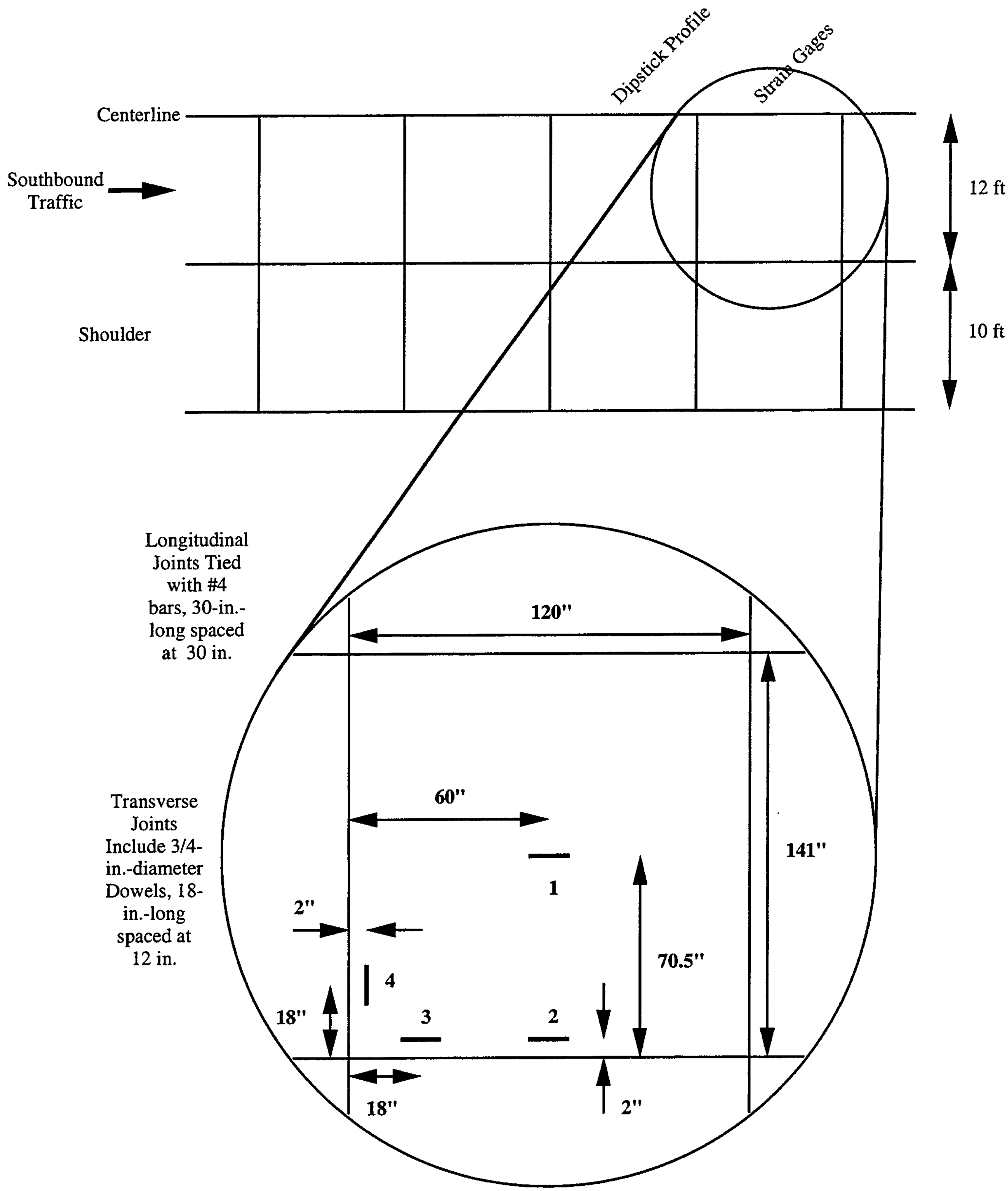
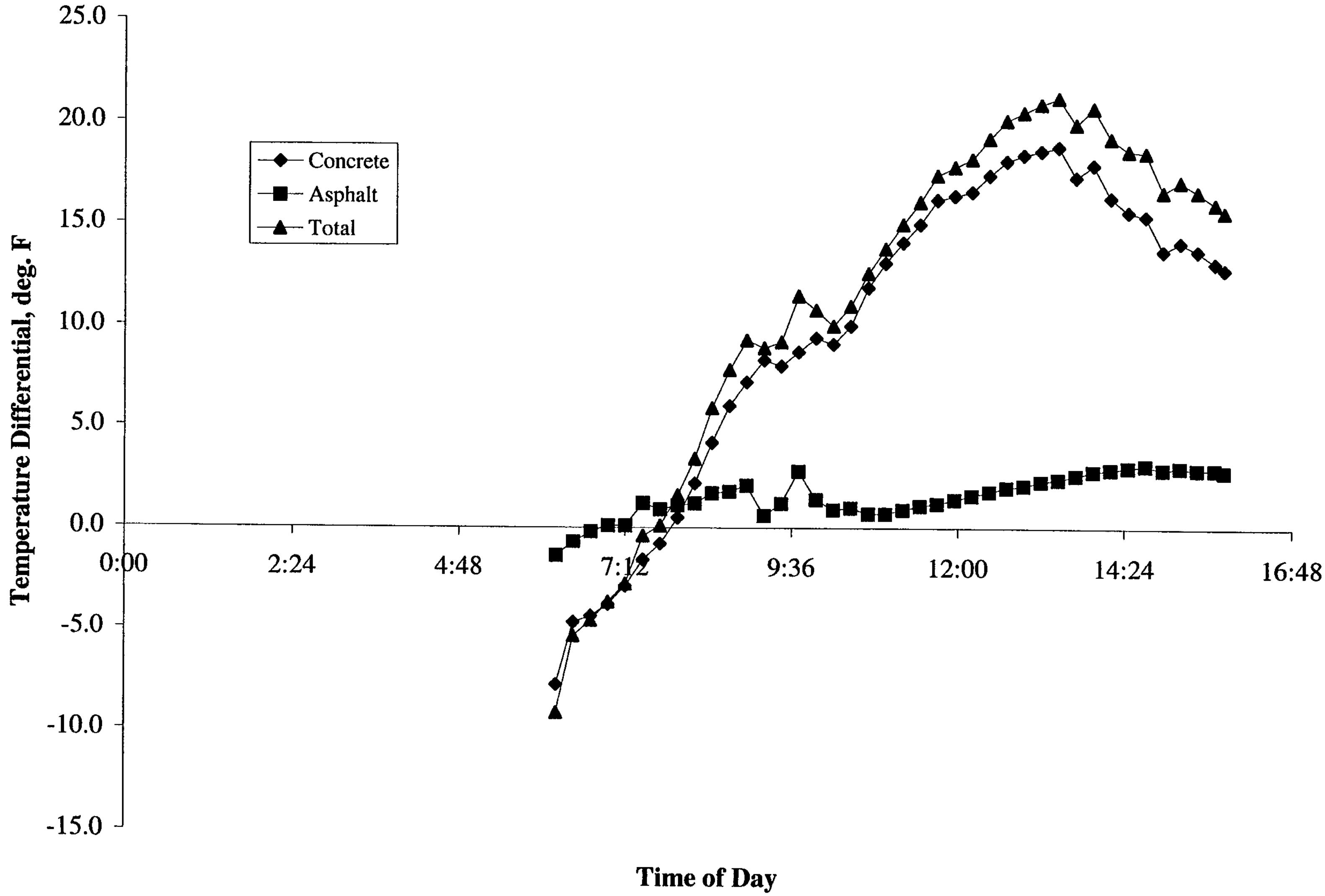


Figure C.3 - Test Slab B Layout for CDOT Project #3.

Figure C.4 - Typical Temperature Gradient in CDOT3 Pavement Layers



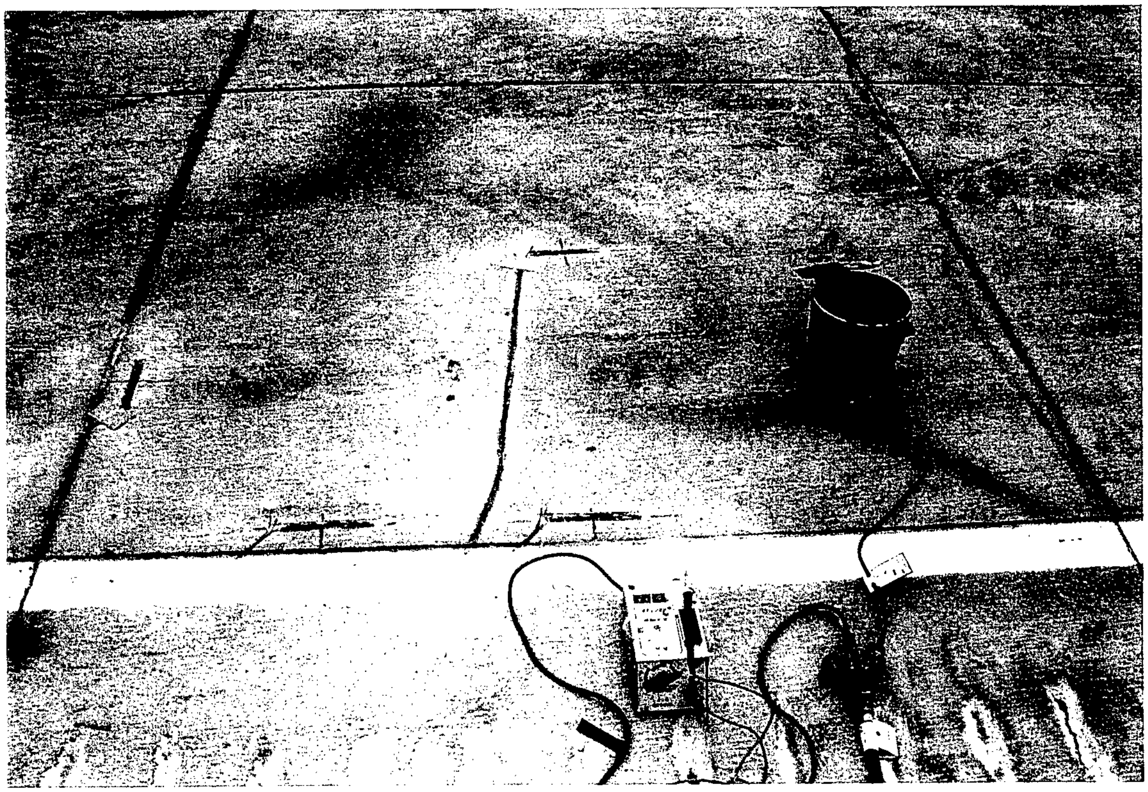


Figure C.5 - Typical Strain Gage Layout at CDOT Project #3

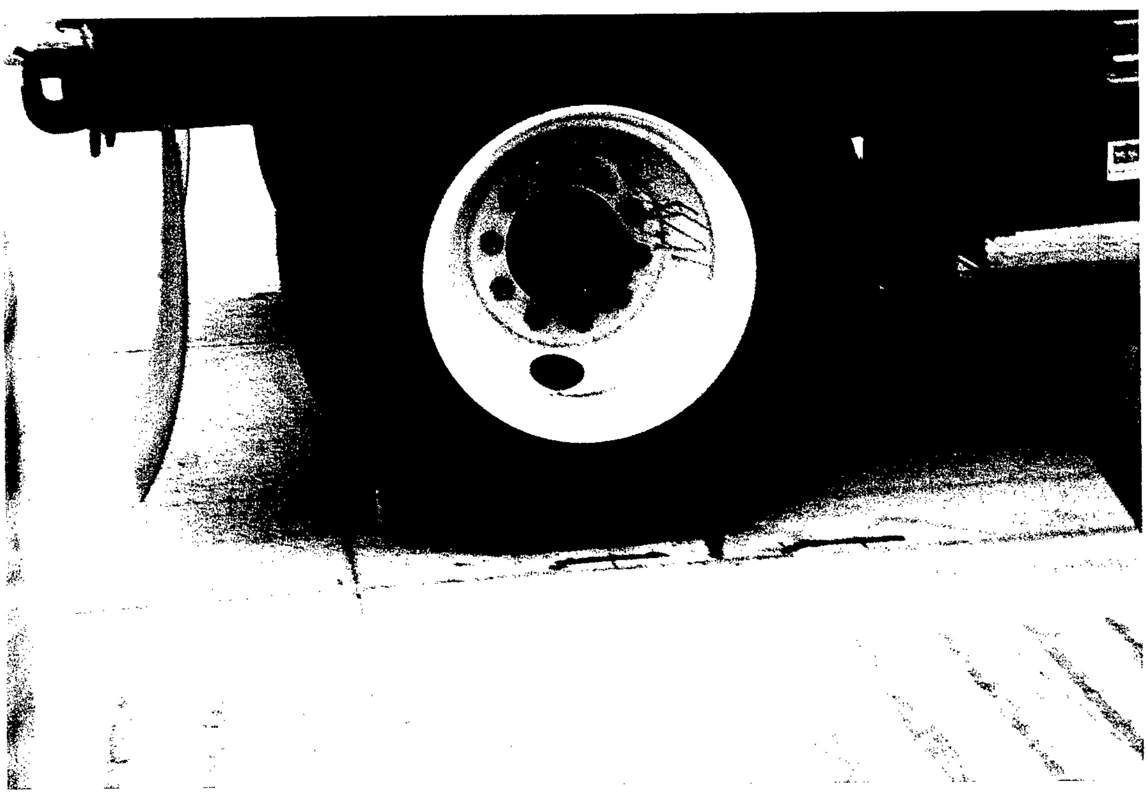


Figure C.6 - Typical Placement of Truck Tires Adjacent to Strain Gages for Load Testing

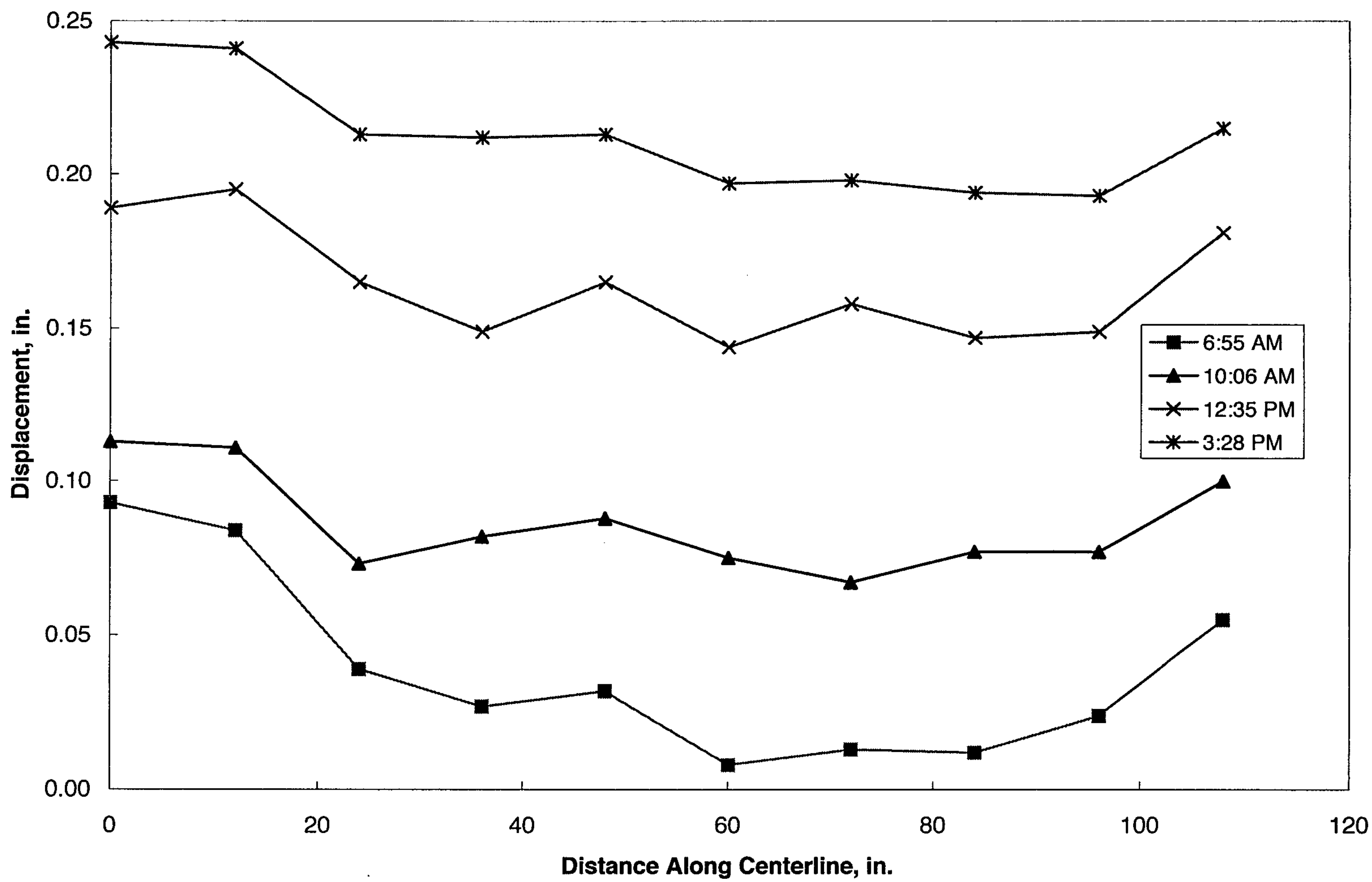


Figure C.7 - Typical Surface Profiles Along Section B Slab Centerline, CDOT Project #3

Figure B.7 - Installing Strain Gages on the Asphalt Concrete Surface at CDOT Project #2

Figure B.8 - Installed Invar Reference Rod Prior to Paving

Figure B.9 - Preparing Concrete Surface for Strain Gages at CDOT Project #2

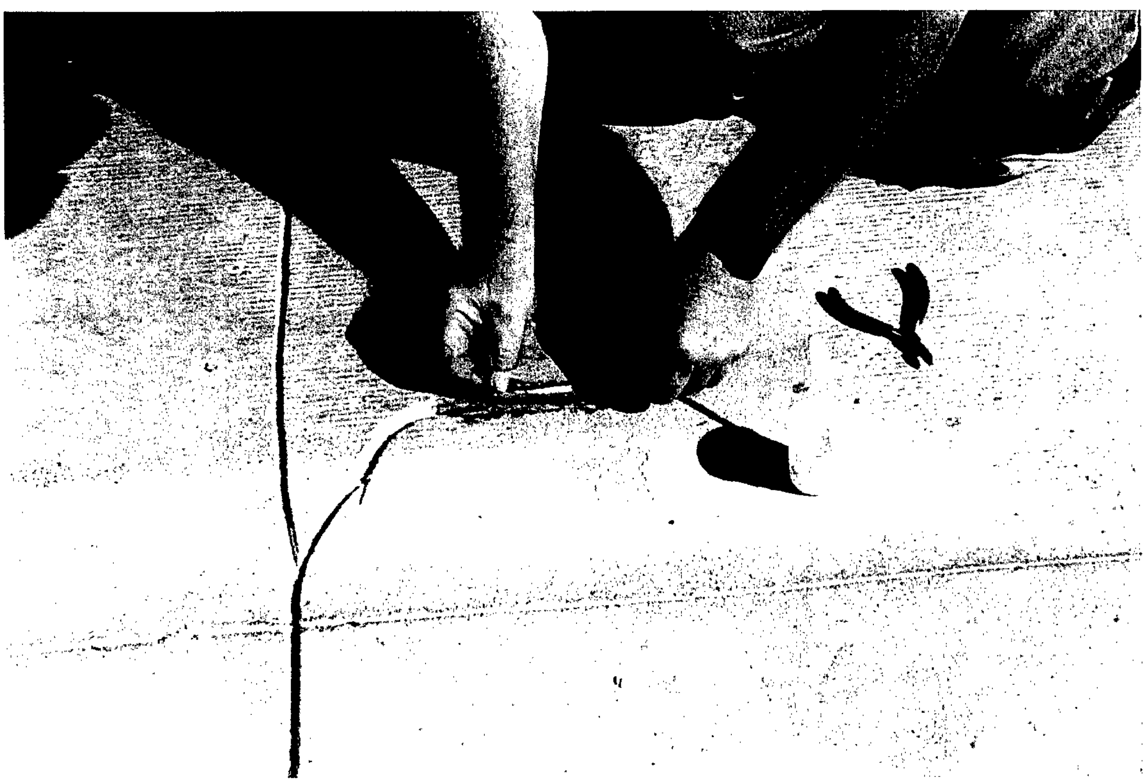


Figure B.10 - Installing Concrete Surface Gages at CDOT Project #2

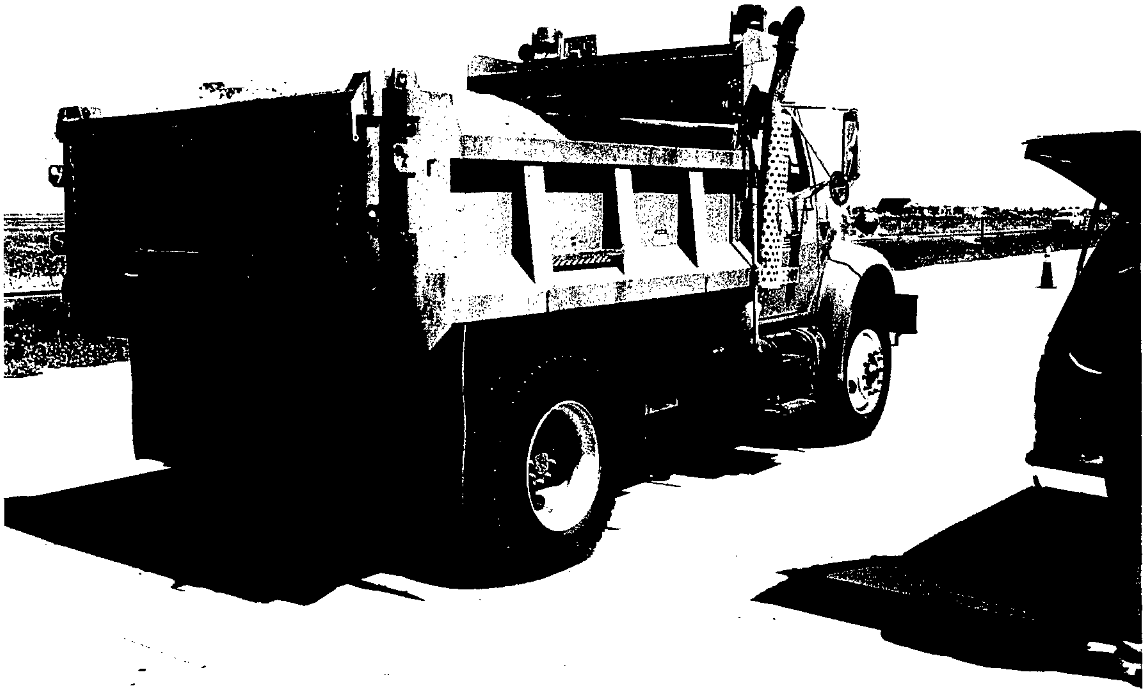


Figure B.11 - Load Testing at CDOT Project #2

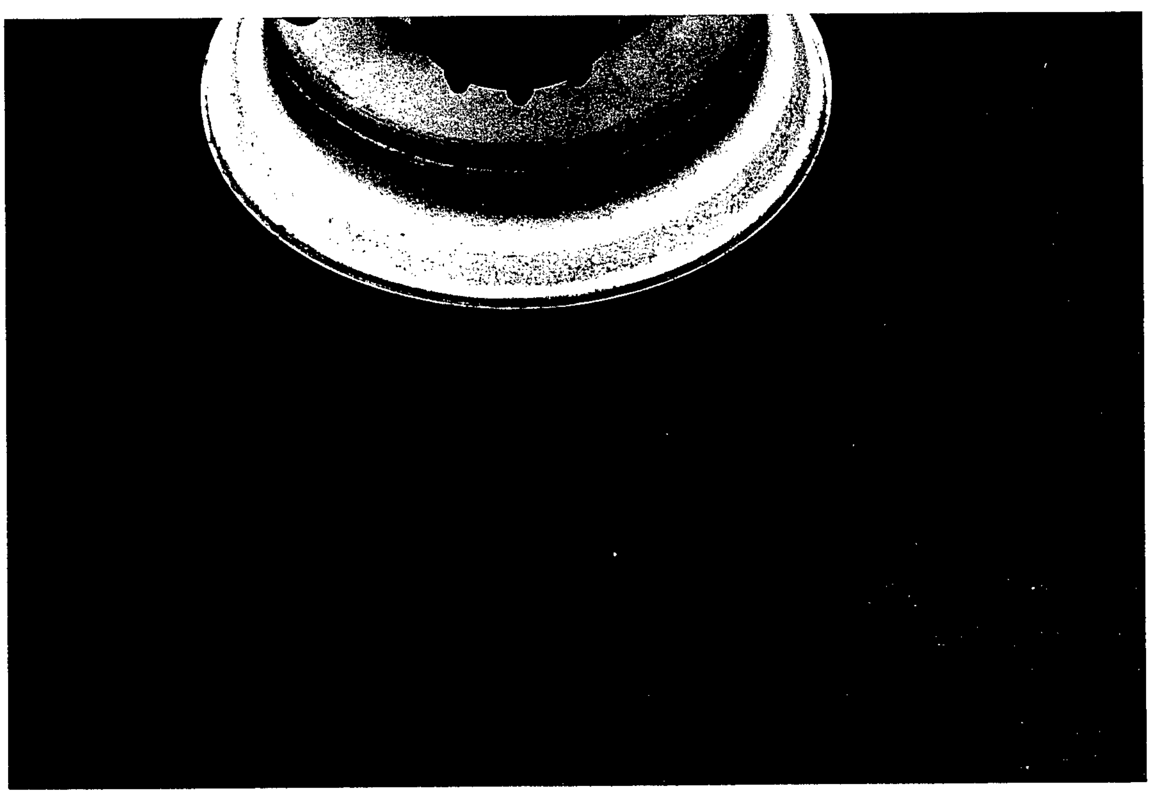


Figure B.12 - Typical Placement of Truck Tires Adjacent to Strain Gages for Load Testing

Figure B.13 - Collecting Profile Elevation Measurements Using the Dipstick at CDOT Project #2

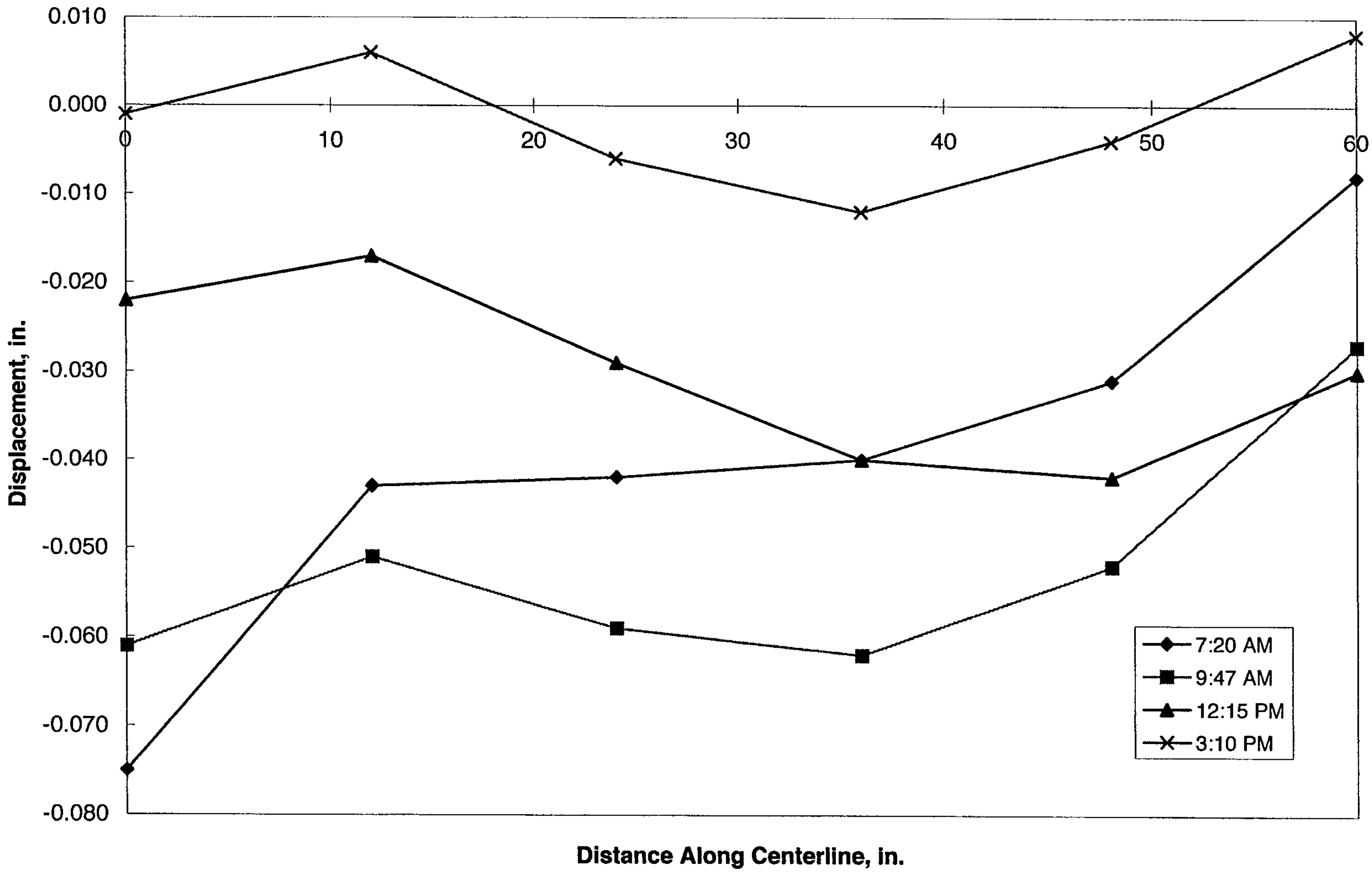


Figure C.8 - Typical Surface Profiles Along Section E Slab Centerline, CDOT Project #3

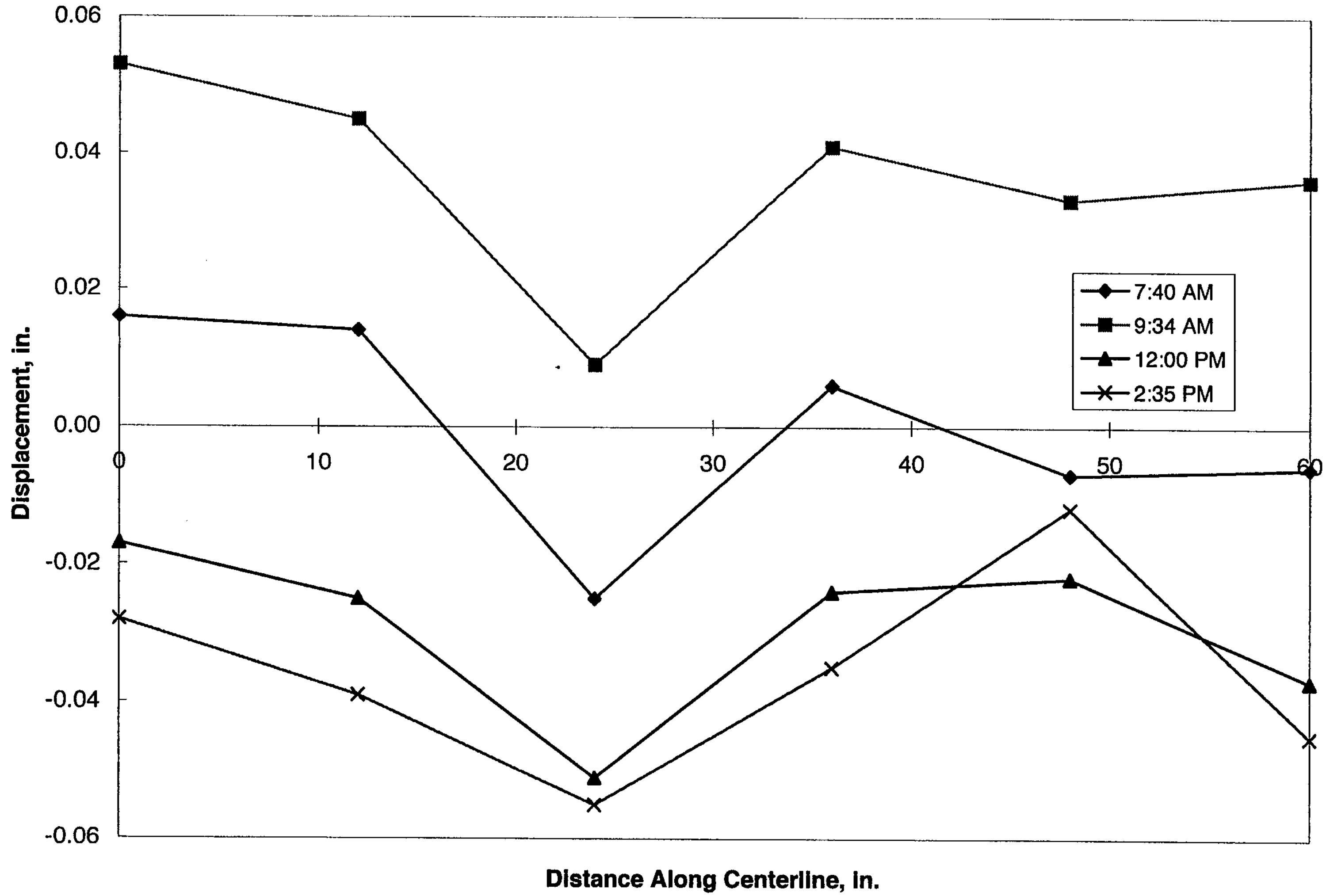


Figure C.9 - Typical Surface Profiles Along Section F Slab Centerline, CDOT Project #3

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