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EVALUATION OF LONGITUDINAL JOINT TIE BAR SYSTEM

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16. Abstract

An adequate longitudinal joint tie bar system is essential in the overall performance of concrete pavement. Excessive longitudinal joint openings are believed to be caused by either inadequate tie bar size or spacing or improper tie bar installation. If designed and installed properly, tie bars prevent the joints from opening and consequently improve load transfer efficiency between slabs and between slabs and shoulders, resulting in increased load carrying capacity.

This study evaluated the longitudinal joint tie bar system currently used by CDOT, examining the criteria for proper use of tie bars and determining the maximum number of lanes that can be tied together without negatively impacting the concrete pavement structure. An improved mechanistic-empirical tie bar design method was developed. Tie bar design tables with recommended bar size and spacing were provided for each combination of pavement base types, CDOT concrete mixes, and weather stations. Field studies were conducted to investigate longitudinal joint performance and further evaluate the impact of factors related to design and construction practices. The experimental plan for this round of testing included the evaluation of tie bar alignment, measurement of joint load transfer, and measurement of relative slab movement at the joints. In addition, CDOT's current specifications and practices related to longitudinal joint construction and tie bar design and placement were compared with those of other state agencies.

Field testing results revealed that the measured joint openings at some tied longitudinal joints were in the typical range of non-tied slabs, implying that some tied joints performed as poorly as non-tied slabs. The results indicate the possibility of tie bar failure due to loss of concrete-steel bonding or yielding of tie bar steel. Another key finding was the possible impact of tie bar misalignment or misplacement on poor longitudinal joint performance. Testing indicated that the measured joint openings were wider when the tie bars did not connect to the other side of the joint, or when the embedment lengths were inadequate. On the other hand, tie bars with adequate embedment length on both sides of the joint, even when misaligned, appear to hold the joint tight.

Implementation
CDOT should adopt the mechanistic-empirical tie bar design procedure developed in this study. The research team recommends that CDOT conduct more rigorous experimental and field testing covering various base material types and concrete mixtures to obtain Colorado-specific model parameters for implementation.

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EXECUTIVE SUMMARY

Longitudinal joints are used in concrete pavements between traffic lanes, between the traffic lanes and shoulders, and other locations such as in the center of a wide ramp to control longitudinal cracking. These joints are used to relieve stresses that result primarily from movements in the concrete slab due to thermal and moisture changes through the slab thickness. Longitudinal joints can be formed by saw cutting a monolithic placement of concrete lanes and shoulders or can be smooth-faced butt joints resulting from two adjacent lane-lane or lane-shoulder placements. Tight joints over the life of the pavement are necessary to maintain joint integrity and to promote load transfer efficiency between concrete slabs (for saw cut joints). Wider joints could result in safety, durability, and longevity (structural) concerns for the concrete pavement system. Tie bars are primarily used across the longitudinal joints to hold the adjacent slabs tightly together. Field experience has shown that the longitudinal joints can widen excessively over time if tie bars are not designed adequately or installed improperly during construction.

The Colorado Department of Transportation (CDOT) uses standard design details for tie bar size and steel grade; the agency requires concrete pavements to have either tied concrete shoulders or tied concrete curb and gutter. The CDOT specification requires No. 4 tie bars (0.5-in.diameter), made with epoxy-coated Grade 40, deformed-steel, for pavements less than 8 in. thick; No. 5 bars (0.625-in. diameter) for pavements 8 to 10 in. thick; and No. 6 bars (0.75-in.diameter) for pavements greater than 10 in. thick. Tie bars are to be 30 in. long and placed at 30-in. centers, perpendicular to the longitudinal joint, at the mid-depth of the slab.

Field surveys have indicated the existence of longitudinal joint problems on some major highways in Colorado. The surveys showed that the longitudinal joint performance was highly variable, ranging from poor to excellent along the same roadway with seemingly similar lane configurations, geometry, base, and tie bar design. The joint openings at these sites often were wider than ½ in. to 1 in., and in one extreme case they were as much as 4 in. These issues were aggravated by joint faulting and longitudinal slippage between adjacent slabs at some sites.

CDOT's tie bar size recommendations are based on concrete slab thickness. CDOT's approach has apparent similarities with the tie bar design procedure presented in American Association of State Highway and Transportation Officials (AASHTO) 1993 *Guide for Design of Pavement Structures* and the design recommendations adopted by several other state DOTs. The AASHTO 1993 procedure is, in turn, based on the subgrade drag theory (SDT). Based on a simplistic friction model that assumes that friction is dependent on thickness of the slab, the SDT-based approach quantifies the amount of tie steel required to drag a concrete slab over an underlying layer without yielding or pulling out the steel bars. However, this method fails to consider several important factors that affect steel requirements in tie bar design. These include that friction between the slab and base depends on the base surface conditions and material stiffness or modulus, site-specific loading conditions, portland cement concrete (PCC) material properties, and actual temperature drop over time (concrete set temperature minus minimum temperature in winter. In addition, the SDT procedure depends heavily on the thickness of the PCC slab which can lead to very close unrealistic tie bar spacing for thick slabs with multiple tied lanes. To compensate for these deficiencies, several agencies, including CDOT, have adopted experience-based adjustments that often are built on the lessons learned from expensive joint failures.

An improved tie bar design procedure based on mechanistic-empirical concepts is proposed in this study. This procedure was developed under an American Pavement Concrete Association study which was supported by CDOT and is based on the premise that longitudinal joint performance can be controlled by limiting the tie bar yield stresses (similar to SDT) resulting from environmentally induced slab deformations due to drying shrinkage and temperature drop. Numerical solutions were developed using the ISLAB2005 finite element program for a variety of base materials, concrete mix design types, and climate statistics from 20 locations in Colorado. Using these solutions, tie bar design tables with recommended bar size and spacing were developed. Optimal tie bar design recommendations were developed based on the real design and site-specific environmental conditions.

Field studies were undertaken to evaluate the impact of design inadequacy and poor construction practices on longitudinal joint performance. Field testing was conducted in two rounds. Sites in and around the Denver area were selected in collaboration with CDOT personnel. Three sites were selected for the first round of testing. The experimental plan involved deflection testing to measure load transfer, as well as MIT Scan testing to evaluate tie bar alignment and to measure the relative slab movement at the joints.

The measured joint movements at all three sites were excessive, ranging up to 1 mm at a temperature difference of 10 °F and were roughly comparable to joint movements in non-tied slabs. This observation implies that some tied joints perform as poorly as non-tied slabs, and that tie bar failure may be caused by a loss of concrete-steel bonding or yielding of tie bar steel. Another key finding of the round one investigation was the possible role of tie bar misalignment (angular skew in longitudinal and or transverse directions) or misplacement (inadequate embedment or absence of tie bar on one side of the joint) in wider joint openings. However, the influence of design and construction implications on poor joint performance could not be determined at this time due to the limited availability of evidence.

Round two testing was conducted at five different sites to evaluate the impact of improper tie bar installation on longitudinal joint performance. MIT Scan testing revealed that the joint openings were wider at the time of measurement when either the embedment lengths of tie bars were inadequate or the tie bars were found on only one side of the longitudinal joint. The joint openings were tighter when the tie bars were embedded adequately on both sides of the longitudinal joint, even with apparent misalignment. Thus, the contribution of tie bar misalignment to wider joint opening could not be established with the existing field data.

CDOT's current specifications and practices related to longitudinal joint construction and tie bar design and placement were compared with those of other state agencies. Salient features evaluated included sampling requirements, equipment and methods used in tie bar testing, requirements of minimum pullout resistance of tie bars, tie bar size, spacing, placement depth, and alignment tolerances. It is recommended that CDOT consider these aspects for possible inclusion in revising their tie bar-related specifications.

The recommended tie bar design procedure and practical design tables are based on sound theoretical and engineering principles that take into account several impact factors; however, the inputs used in the analytical model are based on limited experimental studies and statistical validity. The research team recommends that CDOT conduct more rigorous experimental and field testing covering various base material types and concrete mixtures to obtain Colorado-specific model parameters for implementation.

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CHAPTER 1. INTRODUCTION

BACKGROUND

Longitudinal joints are needed in concrete pavements to (a) provide a contraction joint to relieve excessive thermal and moisture-induced stresses and deflections in the slab that may otherwise result in longitudinal cracking and (b) provide a construction joint to accommodate paving operations as needed to facilitate multiple lane and shoulder situations (Mallela et al. 2009).

Equally important to the provision of longitudinal joints is the provision of an adequate longitudinal joint tie bar system. Tie bars, if designed and installed properly, prevent the lane-lane and lane-concrete shoulder joints from opening excessively while at the same time preventing excessive restraint stresses in the concrete. A longitudinal joint that performs well improves load transfer efficiency (LTE) between concrete pavement slabs (more applicable for saw cut joints), resulting in increased load carrying capacity. In addition, it reduces moisture infiltration and enhances roadway safety.

COLORADO'S PRACTICES FOR TIE BARS AT LONGITUDINAL JOINTS IN CONCRETE PAVEMENTS

To reduce slab stress and extend service life, the Colorado Department of Transportation (CDOT) requires that concrete pavements with shoulders have either tied concrete shoulders, at least 3 ft wide, or a monolithic or tied concrete curb and gutter. When a traffic lane slab is paved wider than 12 ft to further reduce slab stress, the paint stripe marking the lane is placed at 12 ft and the longitudinal joint is sawed and tied at 14 ft. Requiring the longitudinal joint to coincide with the lane line is recommended in urban locations.

CDOT requires No. 4 tie bars (0.5-in. diameter) for concrete pavements less than 8 in. thick, No. 5 bars (0.625-in. diameter) for pavements 8 to 10 in. thick, and No. 6 bars (0.75-in. diameter) for pavements greater than 10 in. thick. Grade 40, deformed-steel, epoxy-coated tie bars are required. Tie bars in sawed (weakened-plane) longitudinal contraction joints are to be 30 in. long and placed at 30-in. centers, perpendicular to the longitudinal joint, at the mid-depth of the slab. For longitudinal construction joints in concrete slabs 8 in. thick or greater, a keyway is allowed to facilitate the use of bent Grade 40 tie bars or approved two-piece connectors. Longitudinal construction joints may be untied butt joints only if the concrete slab is less than 8 in. thick. These requirements are consistent with typical national practice (American Concrete Pavement Association, 2005). The only minor differences are that some other states do not require that tie bars be epoxy coated, some states allow Grade 50 or Grade 60 steel in addition to Grade 40, and some states allow tie bar lengths and spacings other that 30 in. (although 30 in. is typical).

In the event that tie bars are not placed in the concrete before it is hardened, they are to be placed in holes drilled in the hardened concrete to a diameter $^{1}/_{8}$ (0.125) in. greater than the required tie bar diameter, 15 to 16 in. into the concrete, spaced 30 in. on center, at the mid-depth of the slab. The holes for the tie bars are to be cleaned by brushing and with compressed air, and a tie bar is to be anchored into each hole with an amount of an approved epoxy sufficient to cover the bar and fill the hole at least 0.25 in. larger than the tie bar diameter.

When tie bars are placed in either plastic-state concrete or hardened concrete, if the engineer requires it, the contractor must conduct pullout tests of at least 15 bars to demonstrate that the average bar pullout resistance is at least 11,250 lb with slippage of $^{1}/_{16}$ (0.0625) in. or less. The concrete strength must be at least 2,500 psi before pullout tests are conducted. If two or more bars do not meet the required resistance, another set of 15 bars are to be tested, and if any of the second set of bars do not meet the required resistance, all of the remaining tie bars are to be tested. The contractor must conduct additional pullout tests and corrective actions as required by the engineer.

LONGITUDINAL JOINT-RELATED ISSUES IN COLORADO

Under this study, field surveys were conducted to locate and investigate in-service highways where lane separation and loss of load transfer had occurred. Information on candidate pavements was provided by Douglas County, CDOT, and the American Concrete Pavement Association (ACPA) Colorado/Wyoming chapter. A limited number of site visits were conducted to observe and record distresses. The primary goal of this exercise was to gather anecdotal evidence related to typical longitudinal joint problems experienced in the state.

The results of the informal field surveys are summarized in Table 1. As can be noted, the information gathered covered major arterials and primary and U.S. highways. Figure 1 through Figure 3 present examples of longitudinal joint distress observed at several sites in Colorado.

The following key observations are made based on an examination of the data gathered and interviews of CDOT staff:

- The longitudinal joint performance was highly variable, ranging from poor to excellent along the same roadway with seemingly similar lane configurations, geometry, and tie bar designs, indicating possible variability in placement or materials.
- Excessive openings (Figure 1) appear to be the predominant mode of longitudinal joint failures. Faulting of the longitudinal joint (Figure 2) was witnessed on two sites. In one instance, slippage between adjacent slabs has occurred (see Figure 3).
- The widths of the excessive joint openings often were greater than ½ in. to 1 in., and in one extreme case they were as much as 4 in.

FACTORS INFLUENCING PERFORMANCE OF LONGITUDINAL JOINTS

Tie bar sizes, spacing, embedment length, mechanical properties of tie bar steel, and steel coatings affect the integrity of the longitudinal joints in in-service pavements. In turn, the tie bar design details are influenced by many other factors including:

- Lane configuration of the roadway.
  - Number of lanes/shoulders.
  - Lane/shoulder widths.
  - Tied connection definition between lanes/shoulders.

Table 1. Longitudinal joint-tie bar system and multi-lane concrete pavement investigation.

State	Site Location	No. of Tied Lanes & Concrete Shoulders (Total tied width)	Construct-ion Date	Traffic Volume	Observed Longitudinal Joint Width	Pavement Cross Section	Tie Bar Design Details	Transverse Joint Spacing	Distresses
CO	Broadway, south of C-470, Douglas Co.	3-12 ft lanes + 8-ft tied shoulder (44 ft)	1981	ADT = 34,500	3 to 4 in.	7-in. JPCP over existing subgrade	No. 4 bars at 30-in. interval	15 ft centers	Longitudinal cracks, minor corner breaks, patches, and settlements
CO	Wildcat Reserve Parkway, south of Grace Blvd.	5-12 ft lanes + 2-10 ft shoulders (80 ft)	2002	ADT = 141,447	1 to 2 in. at some locations	6-7 in. JPCP over natural subgrade	No.4 bars at 30-in. interval	15 ft centers	Severe cracking, excessive longitudinal settlement
CO	Quebec Blvd., north of Timberline (NB)	7-12 ft lanes + 2-ft and 8-ft tied shoulders (94 ft)	1990	ADT= 29,475	1 to 2 in. at some locations	6-7 in. JPCP over natural subgrade	No. 4 bars at 30-in. interval	15 ft centers	Longitudinal cracks, minor corner breaks, patches, and settlement
CO	Quebec Blvd., south of Collegiate (NB)	7-12 ft lanes + 2-ft and 8-ft tied shoulders (94 ft)	1990	ADT= 29,475	1 to 2 in. at some locations	6-7 in. JPCP over natural subgrade	No.4 bars at 30-in. interval	15 ft centers	Moderate longitudinal cracking, corner breaks
CO	SH 119, south of US 287	2-12 ft lanes, curb & cutter (24 ft)	1983	ADT = 32,300 (%Trucks = 5.2)	1 to 2 in.	8-in. JPCP over class 6 materials (DGAB)	No. 5 bars at 30-in. interval	random at 12 to 20 ft	Moderate longitudinal cracking, corner breaks
CO	Quebec Blvd., north of Ashburn Lane (SB)	3-12 ft lanes + 8-ft shoulder (44 ft)	1990	ADT= 29,475	1 to 2 in. at some locations	6-7 in. JPCP over natural subgrade	No.4 bars at 30-in. interval	15 ft	Longitudinal cracking, corner spalling
CO	US 287, north of Fort Collins	1-12 ft NB + 10-ft shoulder (22 ft)	1991	ADT = 7600 (%Trucks = 15.1)	Over 1 in.	9 in. JPCP over DGAB	No. 5 bars at 30-in. interval	15 ft	Minor distresses

Table 1. Longitudinal joint-tie bar system and multi-lane concrete pavement investigation.

State	Site Location	No. of Tied Lanes & Concrete Shoulders (Total tied width)	Construct-ion Date	Traffic Volume	Observed Longitudinal Joint Width	Pavement Cross Section	Tie Bar Design Details	Transverse Joint Spacing	Distresses
CO	University Ave., west of Quebec (WB)	7-12 ft lanes + 2-ft and 8-ft tied shoulders (94 ft)	1989	ADT= 30,996	2 in. at some locations	6-7 in. JPCP over natural subgrade	No.4 bars at 30-in. interval	15 ft	Longitudinal cracking, spalling,
CO	I-25, MP 152-153	2-12 ft lanes SB + 10-ft and 4-ft tied shoulders (38 ft)	1987	ADT= 71,600 (%Trucks = 11.7)	1 in	7-in. unbonded JPCP overlay/chip seal/8-in JPCP	No. 5 bars at 30-in. interval	15 ft	Full depth longitudinal cracking, corner breaks, patches and settlement

NB = northbound; SB = southbound; WB = westbound; ADT = average daily traffic; JPCP = jointed plain concrete pavement; DGAB = dense graded asphalt base

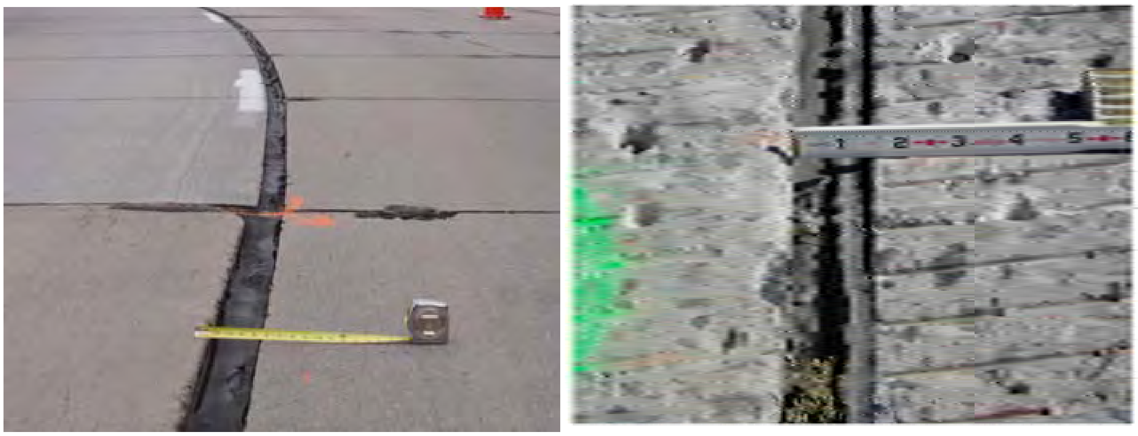

Figure 1. Examples of excessive longitudinal joint openings in in-service pavements.

Figure 2. A faulted longitudinal joint.

Figure 3. Slippage between lanes as evidenced by misaligned transverse joints in adjacent lanes.

- Pavement structure definition.
  - Thickness of the concrete layer.
  - Slab-base friction properties.
  - Base stiffness.
- Portland cement concrete (PCC) material properties.
  - Strength and modulus.
  - Coefficient of thermal expansion/contraction.
  - Shrinkage characteristics.
  - Unit weight of concrete.
- Ambient climatic conditions that influence slab movements.
- Construction factors.
  - Method for longitudinal joint formation, such as monolithically placed lanes that are saw cut, lanes placed separately (butt joint) during the same construction project.
  - Method of tie bar placement, such as automated inserters, manual placement, drilled and grouted, stabbed in place, etc.
- Other site factors.
  - Longevity and durability of pavement support conditions.
  - Side slope stability.
  - Roadway geometry—superelevation versus tangent sections.

As many of these factors as possible should be considered when developing recommendations for tie bar design and construction.

OBJECTIVES OF THE STUDY

Considering the variable performance of longitudinal joints with tie bars in Colorado, CDOT commissioned this study to investigate the adequacy of tie bar design and construction practices and modify them if necessary. The specific objectives of this study were to:

- Develop an improved design procedure and model for longitudinal joint tie bar system that consider critical factors and distresses impacting the performance of both the tie bar (between travel lanes and between the shoulder and travel lane) and concrete pavement, such as excessive joint opening and lane separation.
- Develop construction guidance and best management practices to ensure that the required pullout resistance of the tie bar is not compromised.

A separate but related objective was to theoretically examine the maximum number of lanes that can be tied together for various base course materials and climatic conditions in Colorado.

TECHNICAL APPROACH ADOPTED

This project involved an in-depth examination of the existing tie bar design and installation practices in Colorado. Field testing of longitudinal joint conditions involved joint movement measurement and a tie bar alignment scan for a limited number of sites. CDOT's current specifications and practices related to longitudinal joint construction and tie bar design were compared with those of other state agencies. An improved tie bar design method based on the mechanistic-empirical (M-E) approach developed under an ACPA study (Mallela et al. 2009) is proposed. Supplemental tie bar design tables were developed for Colorado climatic conditions and concrete mixes using this approach.

REPORT ORGANIZATION

Chapter 1 of this report presents the background material, a summary of Colorado's practices related to longitudinal joints and tie bars, and pertinent performance issues observed in in-service pavements.

Chapter 2 presents a technical overview of the proposed M-E tie bar design procedure, the analytical model used in the design procedure, and supplemental design tables to determine the recommended tie bar size and spacing for Colorado conditions.

Chapter 3 documents the field investigations conducted to investigate longitudinal joint performance and the impact of CDOT's design and construction practices. This chapter documents the results and findings from various test methods employed in this study.

Chapter 4 presents a comparison of CDOT's current specifications and practices with those of other state agencies. This chapter also presents the salient features identified in the standard specifications and standard plans of other state agencies.

Chapter 5 presents an overall summary of this study, conclusions, and recommendations.

Appendix A presents a step-by-step approach for identifying the appropriate tie bar size, spacing, and length using the M-E tie bar design method when the design tables presented in Chapter 2 are not used.

CHAPTER 2. TIE BAR DESIGN

OVERVIEW

This chapter presents a technical overview of the proposed M-E tie bar design procedure, tailored to Colorado conditions. As indicated earlier, this procedure was developed by Mallela et al. (2009) under an ACPA sponsored study supported by CDOT. This procedure can be applied to a variety of scenarios, such as different rigid pavement types (jointed plain, jointed reinforced, and continuously reinforced), base types (e.g., unbound materials, asphalt and cement treated base), lane configurations (slab width and number of longitudinal joints), and tie bar properties (bar length, diameter, and steel grade).

The M-E tie bar procedure is based on the premise that the performance of the longitudinal joint can be controlled by limiting the tensile stresses in the tie steel and the excessive stress build-up at the joint opening. This procedure takes into account the environmentally induced slab deformations due to drying shrinkage and temperature drop (from set temperature to mean minimum monthly temperature) that lead to joint opening in rigid pavements, thus providing context-sensitive designs to various microclimates and rigid pavement construction practices in Colorado.

In this Colorado study, a database of numerical solutions was developed using the ISLAB2005 finite element program for a pavement model with various base types commonly used in Colorado. These solutions were developed using the properties of two representative concrete mix design types (gap-graded and optimized) and climate statistics from 20 weather stations in Colorado. Using these solutions, practical tie bar design tables with recommended bar size and spacing were developed for each combination of pavement base types, CDOT concrete mixes, and weather stations.

DESCRIPTION OF THE ANALYTICAL MODEL

The analytical model used herein is a three-layered concrete pavement system (see Figure 4) that consists of PCC slabs with a tied longitudinal joint and a base layer resting on a Winkler foundation (i.e., an elastic layer with a linear relationship between the deflection and the applied wheel load pressure). The tie bars connecting the concrete slabs are represented as linear elastic springs, while the interface between the concrete slab and the base layer is modeled using two sets of linear elastic strings, one in the vertical direction and another in the horizontal direction. This analytical representation of the system of slabs and slab support is a fundamental shift from the subgrade drag theory (SDT), which assumes that a single slab is resting on a rigid base.

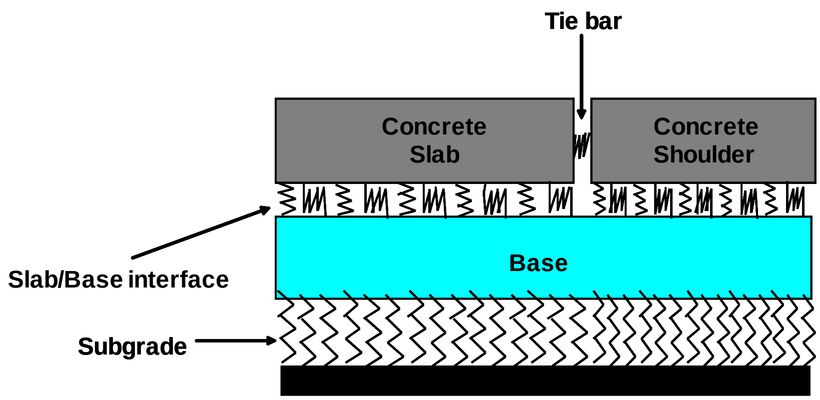

Figure 4. ISLAB2005 model of longitudinal joint and tie bar.

The analytical model used in the tie bar design procedure includes the following salient features:

- Slab-base interface: The stiffness at the interface between the concrete slab and base layer was modeled using two sets of linear elastic springs—the Totsky model in the vertical direction and the modified Coulomb friction model in the horizontal direction. The vertical spring models the relative vertical displacement difference between the bottom surface of the concrete slab and the top surface of the base, such as the curling deformation. The horizontal spring models the interlayer friction and sliding phenomenon at the contact surfaces of the two layers. These springs take into account the thickness and modulus of both the concrete and base layers in computing the interlayer stiffness. This is particularly significant in the context of incorporating the influence of base thickness and base modulus on joint opening and tie steel stresses in computations; in contrast, by attributing infinite stiffness to the base layer through the rigid base assumption, the SDT discounts the role of the base layer on joint performance.

- Pullout stiffness of tie bars: This approach uses a shear model, developed by Guo (1992), to model the pullout stiffness of tie bars. This model relates the vertical shear force transferred through the tie bar with the axial displacements at the contact points of adjacent concrete slabs with the tie bar. This model allows the user to establish a limiting pullout force on a tie bar of a given diameter, length, and steel grade based on the pullout bond-slip relationship developed by the Euro-International Concrete Committee (CEB-FIP 1990). Table 2 presents calculated values of pullout forces, bond stresses, and free edge slips for different tie bar sizes, grades, and embedded lengths. The free edge slip is the axial movement of reinforcement edge not embedded in concrete during pullout testing. The free edge slip is used in determining the maximum allowable joint opening for a given tie bar configuration.

Table 2. Bond properties for #4, #5, and #6 tie bars.

Tie Bar #	Tie Bar Embedment Length, in.*	Force at Steel Yield, lb		Bond Stress at Steel Yield, psi		Free Edge Slip at Steel Yield**, mil	
		Grade 40	Grade 60	Grade 40	Grade 60	Grade 40	Grade 60
4	12	7,875	11,775	417	625	3.1	8.7
	15	7,875	11,775	333	500	1.8	5.0
	18	7,875	11,775	278	417	1.1	3.1
5	12	12,266	18,398	521	781	5.5	15.1
	15	12,266	18,398	417	625	3.1	8.7
	18	12,266	18,398	347	521	2.0	5.5
6	12	17,663	26,494	625	938	8.7	23.8
	15	17,663	26,494	500	750	5.0	13.6
	18	17,663	26,494	417	625	3.1	8.7

*Embedment length was assumed to be one-half of the total tie bar length.
**Maximum allowable joint opening = 2 * free edge slip

- Environmentally induced loads: Two environmental factors are used in this model to define the critical stress conditions and contraction movements in the concrete slabs—drying shrinkage and a uniform temperature drop (the difference between the concrete temperature at set and the mean minimum monthly temperature for a given project location). The effect of drying and thermal shrinkage on concrete is quantified as equivalent free movement strains in the concrete. These two factors allow for incorporating the effects of concrete mix type, curing type, construction set temperature, and climate in the design procedure, thus providing context-sensitive designs.

Detailed discussion on the theoretical background of the analytical model used in the M-E tie bar design procedure can be found in the ACPA report (Mallela et al. 2009).

INPUT DATA FOR THE ANALYTICAL MODEL AND DESIGN

The input parameters required for the analytical model are as follows:

- Model geometry.
- Concrete material properties.
- Base properties.
- Concrete/base interface parameters.
- Subgrade properties.
- Tie bar properties.
- Inputs for computing environmentally induced stress/strains.

The assumptions used in the tie bar design procedure pertinent to Colorado conditions and CDOT practices are described in the following sections.

Model Geometry

The inputs required for model geometry include:

- Number of slabs in the direction perpendicular to traffic flow: Two, three, four, five, and six lanes/shoulders were used.
- Length of the slab along the direction of traffic (i.e., longitudinal direction): CDOT uses a joint spacing of 15 ft maximum for concrete pavement thicknesses over 6 in., 12 ft maximum for concrete thicknesses of 6 in. or less, and a minimum of 8 ft for any full-depth pavement.

- Width of the slab in the direction perpendicular to traffic flow (i.e., transverse direction): The width of each lane/shoulder was assumed to be 12 ft for a standard section and 14 ft for widened slab section.
- Thickness of pavement layers: The use of various PCC-base thickness combinations in the analytical model was deemed unnecessary due to the weak dependence of layer thicknesses on steel stress. Slab/base interface friction does not depend significantly on slab or base thickness but on the characteristics of the surfaces and base material properties. A PCC layer of 10 in. and a base layer of 6 in. were considered adequate in developing stress/strain solutions for a wide range of PCC-base thickness combinations used in practice.

Concrete Material Properties

The material-related inputs for the PCC layer required for developing analytical solutions include:

- Elastic modulus: Assumed to be 4,000,000 psi.
- Poisson’s ratio: Assumed to be 0.2.
- Unit weight: Assumed to be 150 pcf (this ensures that the PCC slab and base did not separate during response calculation.
- Modulus of concrete support: Assumed to be 500,000 psi/in.

The impact on tie bar design from these “standard” values was investigated and was found to be insignificant.

In addition, the PCC mix properties are required to compute the stress/strains induced by environmental factors. The required inputs include:

- Cementitious materials content: See Table 3.
- Coefficient of thermal expansion: See Table 3.
- Cement type: Types I and II are widely used in Colorado.
- Curing type: Curing compound is widely used in Colorado.

To obtain the Colorado-specific material inputs, two CDOT PCC mixes, one with an optimized gradation and another with a gap-graded gradation, were selected for this study. The properties of these mixtures are summarized in Table 3.

Table 3. Properties of CDOT PCC mixes.

Property	Optimized	Gap-graded
Type I Cement, lb	450	480
Fly Ash, lb	113	120
Water/Cementitious Ratio	0.36	0.44
Coefficient of Thermal Expansion, 1/°F	5.7	6.0

Base Properties

Six types of pavement base were considered in this study: unbound, soil cement, permeable cement treated (PCTB), cement treated (CTB), lean concrete (LCB), and asphalt treated bases (ATB). The properties of base materials used in the analytical model are summarized in Table 4. Note that the mid-range values of elastic modulus recommended in the Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO 2008) were used.

Table 4. Base types and their properties.

Base Type	Elastic Modulus (psi)	Poisson's Ratio
Unbound	50,000	0.35
Soil cement	500,000	0.25
Permeable cement treated	750,000	0.25
Cement treated	1,000,000	0.25
Lean concrete	2,000,000	0.25
Asphalt treated*	1,000,000	0.25

*A higher modulus value was used for ATB because critical conditions for tie bar typically are anticipated to occur in cold months.

Parameters for Concrete/Base Interface

The parameter values used in modeling the vertical and horizontal stiffness of the PCC-base interface are summarized in Table 5. These values were adopted from the HIPERPAV program (Ruiz et al. 2001).

Table 5. PCC-base interface parameters.

Base Type	Steady-state Frictional Stress (psi)	Stiffness in Stick (psi/in.)
Unbound	2	100
Soil cement	15	240
Permeable cement treated	15	240
Cement treated	15	240
Lean concrete	15	240
Asphalt treated or concrete over hot mix asphalt	6	240

Subgrade Properties

A modulus of subgrade reaction (k-value) of 200 psi/in. was assumed for the Winkler foundation.

Tie Bar Properties

The following inputs are required for defining the tie bar properties:

- Elastic modulus of steel: Assumed to be 29,000,000 psi.
- Poisson’s ratio of steel: Assumed to be 0.3.
- Steel grade: Grade 40 and Grade 60.
- Tie bar diameter: See Table 6.
- Tie bar embedded length: See Table 6.

Tie bar size and embedment lengths were obtained from CDOT’s Standard Drawing M-412-1, Sheet 5, Reinforcing Size Table. CDOT specifies different tie bar sizes based on the PCC layer thickness ranges, while the embedment length is kept at 30 in. for all thickness ranges (see Table 6).

Table 6. CDOT tie bar and embedment length requirements.

PCC Layer Thickness (in.)	Tie Bar Size	Embedment Length (in.)
< 8	No. 4 (¼ in.)	15
8 ≤ thickness ≤ 10	No. 5 (⅝ in.)	15
10 < thickness ≤ 15	No. 6 (¾ in.)	15

The CDOT specification of tie bar size apparently is based on the American Association of State Highway and Transportation Officials (AASHTO) 1993 Design Guide. According to this document, the PCC slab thickness heavily influences the required steel content to keep the joint opening tight. However, the ISLAB2005 analytical model places less emphasis on the PCC slab thickness. Figures 05 and 06 present the sensitivity of steel stresses and joining opening computed using the ISLAB2005 model for PCC slab thicknesses ranging from 6 to 14 in.

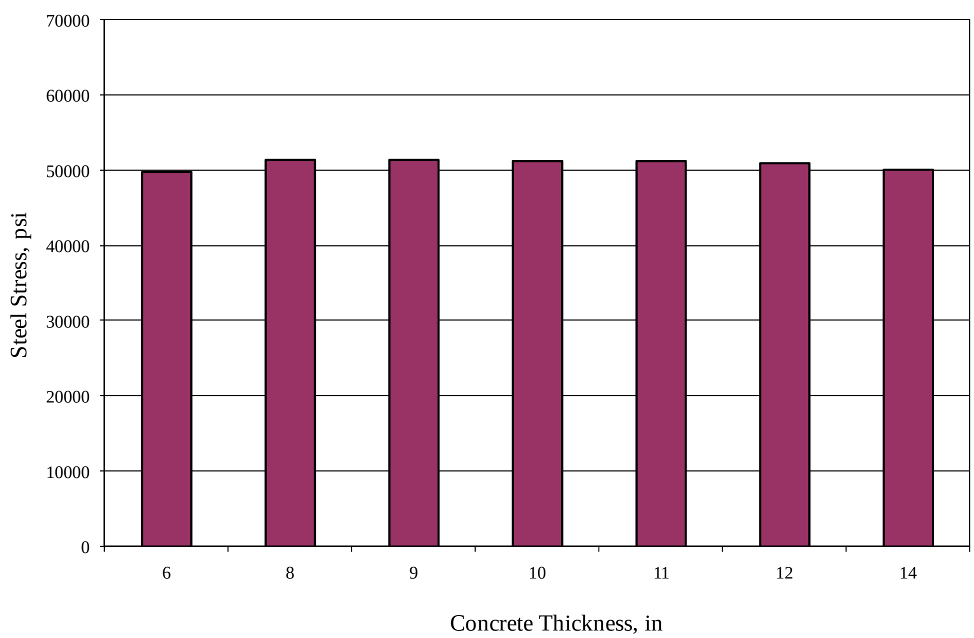

Figure 5. Effect of PCC slab thickness on steel stress (Mallela et al. 2009).

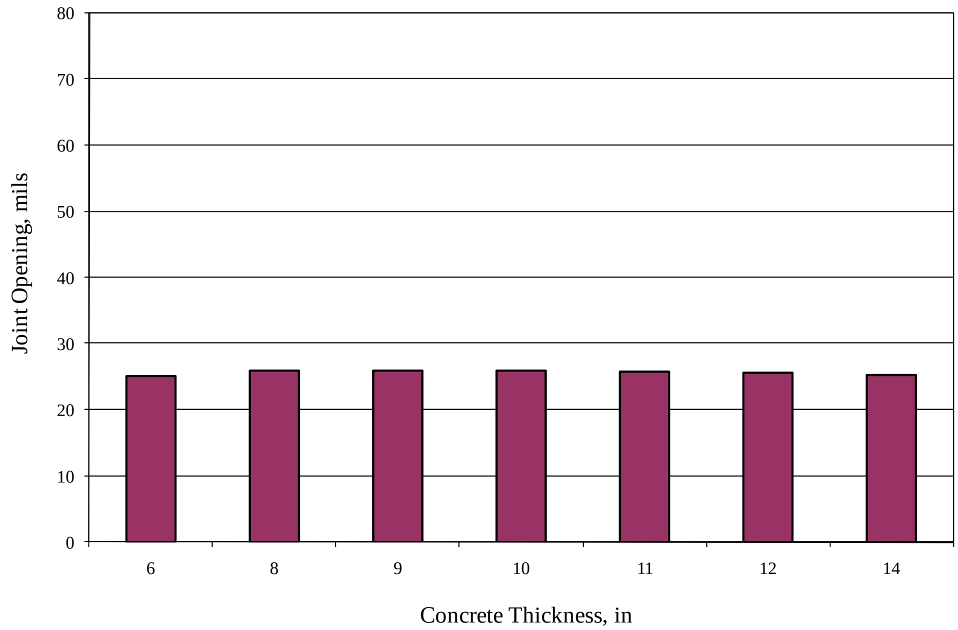

Figure 6. Effect of concrete thickness on joint opening (Mallela et al. 2009).

Inputs for Computing Equivalent Free Strains

The following inputs are required for computing equivalent free concrete slab strains caused by thermal and drying shrinkage of concrete:

- Project location.
- Mean ambient monthly temperature for the month of construction (assumed to be July).
- Mean minimum monthly temperature for the coldest month (assumed to be January).
- Ambient annual average relative humidity.

Twenty weather stations throughout Colorado were selected for this study. Relevant climate statistics were extracted from the Long Term Pavement Performance (LTPP) study database and are presented in Table 7. This information is used in quantifying the effects of thermal and drying shrinkage of concrete as equivalent free strains (i.e., critical strains caused by environmental factors).

Table 7. Colorado LTPP weather station information.

Station	County	Geographical Coordinates			Mean Monthly Temperature (°F)		Average Relative Humidity, (%)
		Latitude	Longitude	Elevation (ft)	July	January	
Akron	Washington	40.1	103.14	4661	74.6	29.9	60.5
Alamosa	Alamosa	37.26	105.52	7533	64.9	19.7	54.8
Aspen	Pitkin	39.13	106.52	7722	64.5	22.2	56.9
Burlington	Carson	39.14	102.17	4195	75.4	30.1	61.1
Col. Springs	El Paso	38.49	104.43	6180	71.4	31.6	53.2
Cortez	Montezuma	37.18	108.38	5896	72.8	29.5	53.8
Craig	Moffat	40.3	107.31	6189	68.6	19.8	63.9
Denver Intl	Denver	39.5	104.4	5379	74.1	31.7	53.7
Denver Centennial	Denver	39.34	104.51	5824	73.4	32.9	51.0
Durango	La Plata	37.08	107.46	6674	69.9	26.6	56.1
Grand Junction	Mesa	39.08	108.32	4823	79.8	30.5	49.1
La Junta	Otero	38.03	103.3	4190	78.5	31.6	54.8
Lamar	Prowers	38.04	102.41	3683	78.5	30.0	60.0
Leadville	Lake	39.14	106.19	9935	56.5	18.8	56.2
Limon	Lincoln	39.11	103.43	5347	71.8	27.5	61.4
Meeker	Rio Blanco	40.03	107.53	6330	69.0	23.8	58.4
Montrose	Montrose	38.31	107.54	5750	74.3	29.0	51.2
Pueblo	Pueblo	38.17	104.3	4652	76.8	31.5	52.9
Rifle	Garfield	39.32	107.44	5503	73.5	26.8	54.8
Trinidad	Las Animas	37.16	104.2	5738	74.8	34.6	49.5

Table 8 presents the equivalent free concrete strains and the concrete set temperature (at the time of construction) for each of the 20 weather station locations. The theory behind the computation of equivalent free to move (or unrestrained) concrete strains is discussed in the ACPA tie bar report (Mallela et al. 2009). Appendix A illustrates the computations with an example. Values presented in this table were calculated for Type I cement. In the case of Type II cement, equivalent free strains will be slightly less than for Type I (within 5 to 7 percent). Therefore, the same tie bar design may be recommended for both cement types.

Table 8. Equivalent free strains for Colorado weather stations.

Station	County	Estimated Temperature at Concrete Set, (°F)		Equivalent Free Strains, (microstrain)	
		Optimized	Gap-graded	Optimized	Gap-graded
Akron	Washington	106	108	720	755
Alamosa	Alamosa	94	95	712	746
Aspen	Pitkin	93	95	692	724
Burlington	Carson	107	109	723	759
Col. Springs	El Paso	102	104	696	729
Cortez	Montezuma	104	106	718	752
Craig	Moffat	98	100	727	762
Denver Intl	Denver	106	108	715	750
Denver Centennial	Denver	105	107	707	741
Durango	La Plata	100	102	709	743
Grand Junction	Mesa	113	115	770	808
La Junta	Otero	111	114	746	783
Lamar	Prowers	111	113	748	785
Leadville	Lake	82	84	649	678
Limon	Lincoln	103	105	711	746
Meeker	Rio Blanco	99	101	715	750
Montrose	Montrose	106	108	736	771
Pueblo	Pueblo	109	111	737	773
Rifle	Garfield	105	107	737	773
Trinidad	Las Animas	107	109	710	744

MODEL EXECUTION AND SOLUTION GENERATION

Using the inputs described in the previous section, the analytical model was executed using ISLAB2005. A finite element mesh of four-noded rectangular flat shell elements was used for discretizing the slabs and base of the concrete pavement for various slab-lane configurations. Various combinations of tie bar dimensions, tied lane configurations, and pavement cross-section details (including slab-base interface definitions) were run. For each of these combinations, the tensile stresses in the steel and the displacements of the slab edges (i.e., slab contraction) on either side of the critical longitudinal joint were computed and stored in a database that was used in developing the M-E tie bar design procedure.

For each permutation, joint openings were calculated as the difference between the horizontal displacements of slab edges. The computed joint opening then was compared with the maximum allowable joint opening (or two times the free edge slip) for a given tie bar size and embedment length. If the one-half of the computed joint opening exceeded the free edge slip for a given tie bar configuration, it was concluded that excessive yield stress will build up in the tie bar, leading toward its failure.

RECOMMENDED TIE BAR DESIGNS

The tie bar designs presented herein have been tailored to CDOT practices and conditions. Tables 9 through 20 present the recommended minimum tie bar size and maximum spacing for each combination of number of tied lanes, CDOT concrete mix type, and pavement base type for each of the 20 weather stations. Each tied shoulder was considered as an additional tied lane. These recommendations are based on the equivalent concrete free (unrestrained) strains, tie bar yield stresses, free edge slip, and ISLAB2005 computation of joint opening.

The recommendations are made based on the inputs and assumptions discussed in the inputs section of this chapter. Should the conditions deviate from these inputs and assumptions, practitioners can follow the procedure presented in Appendix A to determine the appropriate tie bar size and spacing for their situations.

SHEAR CAPACITY AT JOINTS

Tie bars are used primarily to hold the joint opening tighter so as to maintain the aggregate interlock at the joint. While the aggregate interlock promotes load transfer across the joint, the role of tie bars is to hold the abutting faces of the longitudinal joint tighter against thermal and moisture-induced movements. However, in some cases, the aggregate interlock could be absent due to the type of joint formation (separate placements of adjacent concrete lanes), deterioration of aggregate interlock due to wear, physical separation between adjacent slabs, or wider joint opening. In such cases, tie bars may be forced to act like dowels to transfer loads to the adjoining slab. Although this is not the primary purpose of the tie bar, lack of adequate shear capacity at the joint in such cases could lead to longitudinal joint faulting in the presence of other aggravating factors, such as heavy loads traversing the joints (as in urban settings) and lack of adequate support beneath the pavement (due to erosion or pumping).

In this study, the Friberg model was used to conduct stress analysis to evaluate the load transfer capabilities of tie bars. Much like dowels, the design of tie bars is governed by the bearing stress between concrete and steel. The actual bearing stress between a tie bar and concrete should be less than the allowable bearing stress.

Table 9. Tie bar design for jointed plain concrete pavement (JPCP) (CDOT optimized mix) on unbound base.

Climate Station	Number of Tied Lanes					
	2		3		4	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#4	45	#4	45	#5	45
Alamosa	#4	45	#4	45	#5	45
Aspen	#4	45	#4	45	#5	45
Burlington	#4	45	#4	45	#5	45
Colorado Springs	#4	45	#4	45	#5	45
Cortez	#4	45	#4	45	#5	45
Craig	#4	45	#4	45	#5	45
Denver International	#4	45	#4	45	#5	45
Denver Centennial	#4	45	#4	45	#5	45
Durango	#4	45	#4	45	#5	45
Grand Junction	#4	45	#4	45	#5	45
La Junta	#4	45	#4	45	#5	45
Lamar	#4	45	#4	45	#5	45
Leadville	#4	45	#4	45	#4	45
Limon	#4	45	#4	45	#5	45
Meeker	#4	45	#4	45	#5	45
Montrose	#4	45	#4	45	#5	45
Pueblo	#4	45	#4	45	#5	45
Rifle	#4	45	#4	45	#5	45
Trinidad	#4	45	#4	45	#5	45

Note: Only Grade 60 steel is recommended.

Table 10. Tie bar design for JPCP (CDOT gap-graded mix) on unbound base.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#4	45	#4	45	#5	45
Alamosa	#4	45	#4	45	#5	45
Aspen	#4	45	#4	45	#5	45
Burlington	#4	45	#4	45	#5	45
Colorado Springs	#4	45	#4	45	#5	45
Cortez	#4	45	#4	45	#5	45
Craig	#4	45	#4	45	#5	45
Denver International	#4	45	#4	45	#5	45
Denver Centennial	#4	45	#4	45	#5	45
Durango	#4	45	#4	45	#5	45
Grand Junction	#4	45	#4	45	#5	45
La Junta	#4	45	#4	45	#5	45
Lamar	#4	45	#4	45	#5	45
Leadville	#4	45	#4	45	#5	45
Limon	#4	45	#4	45	#5	45
Meeker	#4	45	#4	45	#5	45
Montrose	#4	45	#4	45	#5	45
Pueblo	#4	45	#4	45	#5	45
Rifle	#4	45	#4	45	#5	45
Trinidad	#4	45	#4	45	#5	45

Note: Only Grade 60 steel is recommended.

Table 11. Tie bar design for JPCP (CDOT optimized mix) on soil cement base.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#5	30	#6	36	#6	36
Alamosa	#5	30	#6	36	#6	36
Aspen	#5	36	#6	36	#6	36
Burlington	#5	30	#6	36	#6	36
Colorado Springs	#5	36	#6	36	#6	36
Cortez	#5	30	#6	36	#6	36
Craig	#5	30	#6	36	#6	36
Denver International	#5	30	#6	36	#6	36
Denver Centennial	#5	30	#6	36	#6	36
Durango	#5	30	#6	36	#6	36
Grand Junction	#5	30	#6	36	#6	36
La Junta	#5	30	#6	36	#6	36
Lamar	#5	30	#6	36	#6	36
Leadville	#5	36	#5	36	#5	36
Limon	#5	30	#6	36	#6	36
Meeker	#5	30	#6	36	#6	36
Montrose	#5	30	#6	36	#6	36
Pueblo	#5	30	#6	36	#6	36
Rifle	#5	30	#6	36	#6	36
Trinidad	#5	30	#6	36	#6	36

Note: Only Grade 60 steel is recommended.

Table 12. Tie bar design for JPCP (CDOT gap-graded mix) on soil cement base.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#5	30	#6	36	#6	36
Alamosa	#5	30	#6	36	#6	36
Aspen	#5	30	#6	36	#6	36
Burlington	#5	30	#6	36	#6	36
Colorado Springs	#5	30	#6	36	#6	36
Cortez	#5	30	#6	36	#6	36
Craig	#5	30	#6	36	#6	36
Denver International	#5	30	#6	36	#6	36
Denver Centennial	#5	30	#6	36	#6	36
Durango	#5	30	#6	36	#6	36
Grand Junction	#5	30	#6	36	#6	36
La Junta	#5	30	#6	36	#6	36
Lamar	#5	30	#6	36	#6	36
Leadville	#5	36	#6	36	#6	36
Limon	#5	30	#6	36	#6	36
Meeker	#5	30	#6	36	#6	36
Montrose	#5	30	#6	36	#6	36
Pueblo	#5	30	#6	36	#6	36
Rifle	#5	30	#6	36	#6	36
Trinidad	#5	30	#6	36	#6	36

Note: Only Grade 60 steel is recommended.

Table 13. Tie bar design for JPCP (CDOT optimized mix) on PCTB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	36	#6	36
Alamosa	#6	36	#6	36	#6	36
Aspen	#6	36	#6	36	#6	36
Burlington	#6	30	#6	36	#6	36
Colorado Springs	#6	36	#6	36	#6	36
Cortez	#6	36	#6	36	#6	36
Craig	#6	36	#6	36	#6	36
Denver International	#6	36	#6	36	#6	36
Denver Centennial	#6	36	#6	36	#6	36
Durango	#6	36	#6	36	#6	36
Grand Junction	#6	36	#6	36	#6	36
La Junta	#6	36	#6	36	#6	36
Lamar	#6	36	#6	36	#6	36
Leadville	#5	36	#6	36	#6	36
Limon	#6	36	#6	36	#6	36
Meeker	#6	36	#6	36	#6	36
Montrose	#6	36	#6	36	#6	36
Pueblo	#6	36	#6	36	#6	36
Rifle	#6	36	#6	36	#6	36
Trinidad	#6	36	#6	36	#6	36

Note: Only Grade 60 steel is recommended.

Table 14. Tie bar design for JPCP (CDOT gap-graded mix) on PCTB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	36	#6	36
Alamosa	#6	36	#6	36	#6	36
Aspen	#6	36	#6	36	#6	36
Burlington	#6	36	#6	36	#6	36
Colorado Springs	#6	36	#6	36	#6	36
Cortez	#6	36	#6	36	#6	36
Craig	#6	36	#6	36	#6	36
Denver International	#6	36	#6	36	#6	36
Denver Centennial	#6	36	#6	36	#6	36
Durango	#6	36	#6	36	#6	36
Grand Junction	#6	36	#6	36	#6	36
La Junta	#6	36	#6	36	#6	36
Lamar	#6	36	#6	36	#6	36
Leadville	#6	36	#6	36	#6	36
Limon	#6	36	#6	36	#6	36
Meeker	#6	36	#6	36	#6	36
Montrose	#6	36	#6	36	#6	36
Pueblo	#6	36	#6	36	#6	36
Rifle	#6	36	#6	36	#6	36
Trinidad	#6	36	#6	36	#6	36

Note: Only Grade 60 steel is recommended.

Table 15. Tie bar design for JPCP (CDOT optimized mix) on CTB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	36	#6	30
Alamosa	#6	36	#6	36	#6	30
Aspen	#6	36	#6	36	#6	36
Burlington	#6	36	#6	36	#6	30
Colorado Springs	#6	36	#6	36	#6	36
Cortez	#6	36	#6	36	#6	30
Craig	#6	36	#6	36	#6	30
Denver International	#6	36	#6	36	#6	30
Denver Centennial	#6	36	#6	36	#6	30
Durango	#6	36	#6	36	#6	30
Grand Junction	#6	36	#6	36	#6	22.5
La Junta	#6	36	#6	36	#6	30
Lamar	#6	36	#6	36	#6	30
Leadville	#5	36	#6	36	#6	36
Limon	#6	36	#6	36	#6	30
Meeker	#6	36	#6	36	#6	30
Montrose	#6	36	#6	36	#6	30
Pueblo	#6	36	#6	36	#6	30
Rifle	#6	36	#6	36	#6	30
Trinidad	#6	36	#6	36	#6	30

Note: Only Grade 60 steel is recommended.

Table 16. Tie bar design for JPCP (CDOT gap-graded mix) on CTB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	36	#6	22.5
Alamosa	#6	36	#6	36	#6	30
Aspen	#6	36	#6	36	#6	30
Burlington	#6	36	#6	36	#6	22.5
Colorado Springs	#6	36	#6	36	#6	30
Cortez	#6	36	#6	36	#6	22.5
Craig	#6	36	#6	36	#6	22.5
Denver International	#6	36	#6	36	#6	30
Denver Centennial	#6	36	#6	36	#6	30
Durango	#6	36	#6	36	#6	30
Grand Junction	#6	36	#6	36	#6	22.5
La Junta	#6	36	#6	36	#6	22.5
Lamar	#6	36	#6	36	#6	22.5
Leadville	#6	36	#6	36	#6	36
Limon	#6	36	#6	36	#6	30
Meeker	#6	36	#6	36	#6	30
Montrose	#6	36	#6	36	#6	22.5
Pueblo	#6	36	#6	36	#6	22.5
Rifle	#6	36	#6	36	#6	22.5
Trinidad	#6	36	#6	36	#6	30

Note: Only Grade 60 steel is recommended.

Table 17. Tie bar design for JPCP (CDOT optimized mix) on LCB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	26	--	--
Alamosa	#6	36	#6	26	--	--
Aspen	#6	36	#6	30	--	--
Burlington	#6	36	#6	26	--	--
Colorado Springs	#6	36	#6	30	--	--
Cortez	#6	36	#6	26	--	--
Craig	#6	36	#6	26	--	--
Denver International	#6	36	#6	26	--	--
Denver Centennial	#6	36	#6	26	--	--
Durango	#6	36	#6	26	--	--
Grand Junction	#6	36	#6	30	--	--
La Junta	#6	36	#6	26	--	--
Lamar	#6	36	#6	26	--	--
Leadville	#6	36	#6	36	--	--
Limon	#6	36	#6	26	--	--
Meeker	#6	36	#6	26	--	--
Montrose	#6	36	#6	26	--	--
Pueblo	#6	36	#6	26	--	--
Rifle	#6	36	#6	26	--	--
Trinidad	#6	36	#6	26	--	--

Note: Only Grade 60 steel is recommended.

Table 18. Tie bar design for JPCP (CDOT gap-graded mix) on LCB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	22.5	--	--
Alamosa	#6	36	#6	26	--	--
Aspen	#6	36	#6	26	--	--
Burlington	#6	36	#6	22.5	--	--
Colorado Springs	#6	36	#6	26	--	--
Cortez	#6	36	#6	22.5	--	--
Craig	#6	36	#6	22.5	--	--
Denver International	#6	36	#6	26	--	--
Denver Centennial	#6	36	#6	26	--	--
Durango	#6	36	#6	26	--	--
Grand Junction	#6	36	#6	22.5	--	--
La Junta	#6	36	#6	22.5	--	--
Lamar	#6	36	#6	22.5	--	--
Leadville	#6	36	#6	30	--	--
Limon	#6	36	#6	26	--	--
Meeker	#6	36	#6	26	--	--
Montrose	#6	36	#6	22.5	--	--
Pueblo	#6	36	#6	22.5	--	--
Rifle	#6	36	#6	22.5	--	--
Trinidad	#6	36	#6	26	--	--

Note: Only Grade 60 steel is recommended.

Table 19. Tie bar design for JPCP (CDOT optimized mix) on ATB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	36	#6	30
Alamosa	#6	36	#6	36	#6	30
Aspen	#6	36	#6	36	#6	36
Burlington	#6	36	#6	36	#6	30
Colorado Springs	#6	36	#6	36	#6	36
Cortez	#6	36	#6	36	#6	30
Craig	#6	36	#6	36	#6	30
Denver International	#6	36	#6	36	#6	30
Denver Centennial	#6	36	#6	36	#6	30
Durango	#6	36	#6	36	#6	30
Grand Junction	#6	36	#6	36	#6	30
La Junta	#6	36	#6	36	#6	30
Lamar	#6	36	#6	36	#6	30
Leadville	#5	36	#6	36	#6	36
Limon	#6	36	#6	36	#6	30
Meeker	#6	36	#6	36	#6	30
Montrose	#6	36	#6	36	#6	30
Pueblo	#6	36	#6	36	#6	30
Rifle	#6	36	#6	36	#6	30
Trinidad	#6	36	#6	36	#6	30

Note: Only Grade 60 steel is recommended.

Table 20. Tie bar design for JPCP (CDOT gap-graded mix) on ATB.

Climate Station	Number of Tied Lanes					
	2		3		More than 3	
	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)	Tie Bar Size Designation	Tie Bar Space, (in.)
Akron	#6	36	#6	36	#6	22.5
Alamosa	#6	36	#6	36	#6	30
Aspen	#6	36	#6	36	#6	30
Burlington	#6	36	#6	36	#6	22.5
Colorado Springs	#6	36	#6	36	#6	30
Cortez	#6	36	#6	36	#6	22.5
Craig	#6	36	#6	36	#6	22.5
Denver International	#6	36	#6	36	#6	30
Denver Centennial	#6	36	#6	36	#6	30
Durango	#6	36	#6	36	#6	22.5
Grand Junction	#6	36	#6	36	#6	22.5
La Junta	#6	36	#6	36	#6	22.5
Lamar	#6	36	#6	36	#6	22.5
Leadville	#6	36	#6	36	#6	36
Limon	#6	36	#6	36	#6	30
Meeker	#6	36	#6	36	#6	30
Montrose	#6	36	#6	36	#6	22.5
Pueblo	#6	36	#6	36	#6	22.5
Rifle	#6	36	#6	36	#6	22.5
Trinidad	#6	36	#6	36	#6	30

Note: Only Grade 60 steel is recommended.

The allowable bearing stress recommended by the American Concrete Institute (ACI) can be determined using the following equation (Yoder and Witczak, 1975):

$$f_b = \left(\frac{4-d}{3.0}\right) f_c'$$

Where

f_c' = compressive strength of concrete, psi

d = tie bar diameter, in.

The maximum bearing stress on concrete can be determined using the following equation:

$$\sigma = \frac{KP}{4\beta^3 EI}(2 + \beta z)$$

Where

K = modulus of bar support, pci

P = transferred load, lb

E = modulus of elasticity of steel, psi

I = moment of inertia of tie bar, in.^{4} = $(\pi d^4)/64$

z = joint opening, in.

β = relative stiffness of the tie bar embedded in concrete

$$\beta = \sqrt[4]{\frac{Kd}{4EI}}$$

A sensitivity analysis was conducted to check the adequacy of tie bars using the following assumptions:

- Modulus of bar support (K) = 1,000,000 pci.
- Elastic modulus of steel (E) = 29,000,000 psi.
- Compressive strength of concrete f'_c = 5,000 psi.
- Dual wheel load (P) = 9,000 lb.
- Design load (P) was defined as:

$$P = \frac{9000 * LTE}{\eta}$$

Where

LTE = load transfer efficiency of tie bar, assigned to be 50%

η = coefficient of tie bar group action

(η = 2 for spacing of 30 in.; η = 1.5 for spacing of 45 in.)

Using the Friberg model, bearing stresses in concrete were computed as a function of joint opening for No. 4, 5, and 6 tie bars at spacings of 30 and 45 in. (see Figures 7 through 9). It was found that only the No.6 tie bar at 30-in. spacing satisfies the allowable bearing stress criteria for loading transfer functions in the absence of aggregate interlock. Hence, for longitudinal joints where there is heavy truck traffic and no aggregate interlock (e.g., smooth construction butt joints), if significant load shear transfer is desired, larger tie bars may be needed. This recommendation needs to be tempered with the demands of the longitudinal design requirements for the pavement section under consideration because the primary purpose of the tie steel is to hold maintain the joint integrity by resisting axial movements that tend to pull it apart.

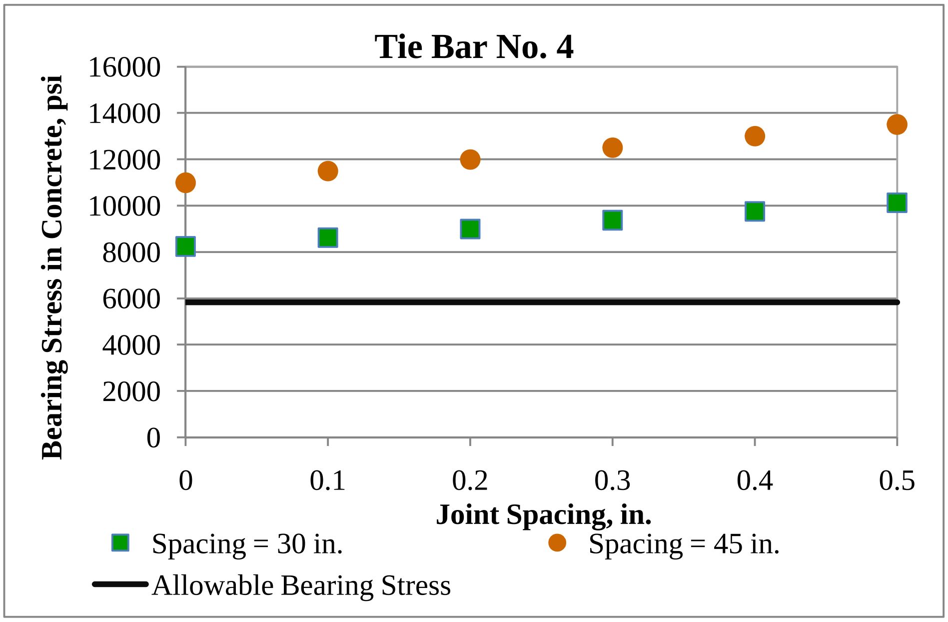

Figure 7. Bearing stresses for No. 4 tie bars.

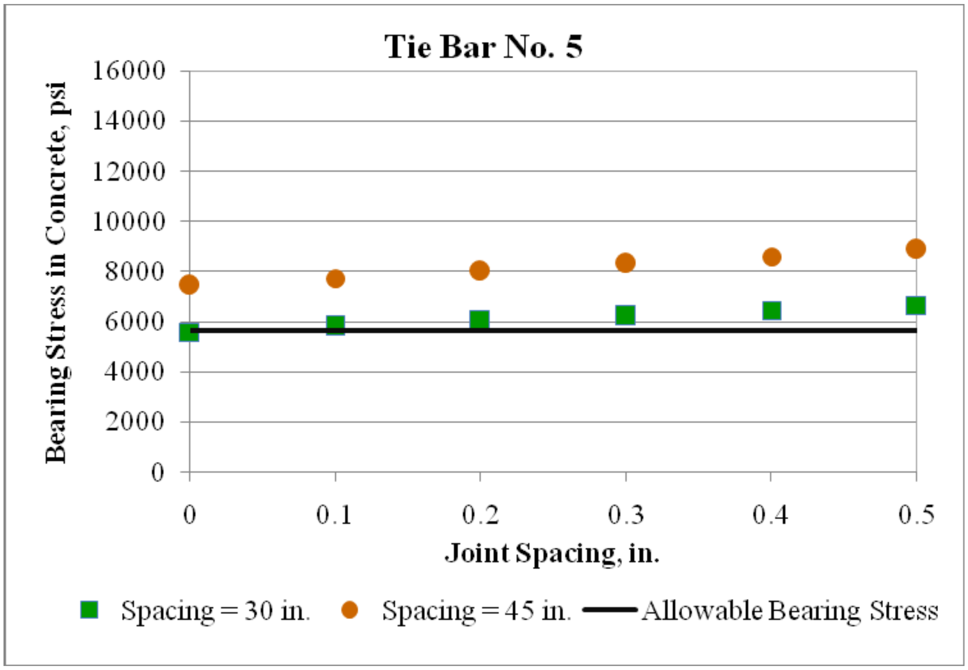

Figure 8. Bearing stresses for No. 5 tie bars.

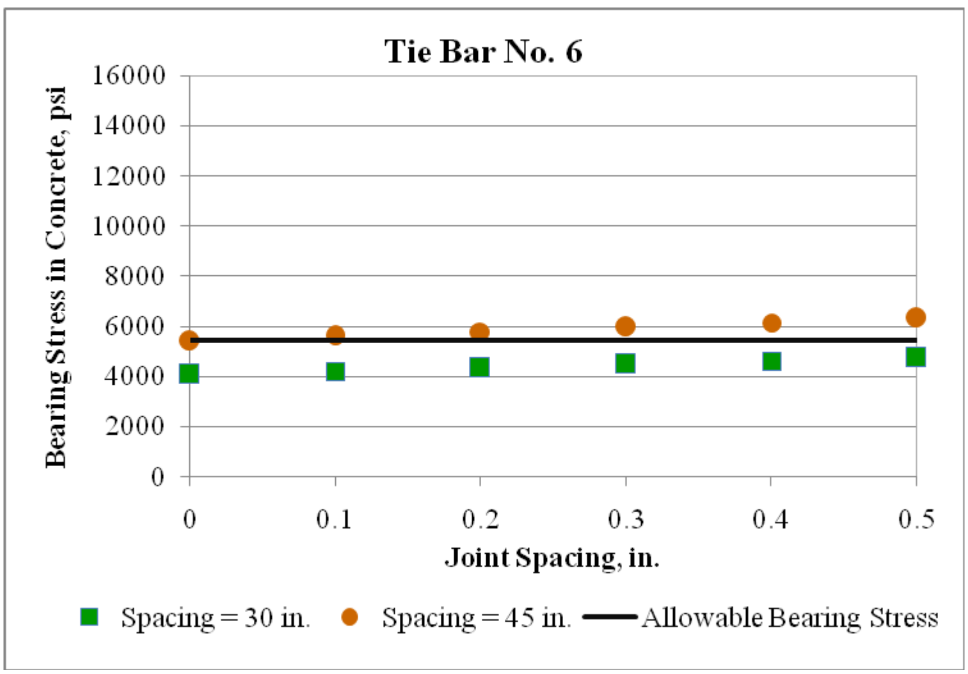

Figure 9. Bearing stresses for No. 6 tie bars.

CHAPTER 3. IDENTIFICATION OF FIELD SECTIONS OF INTEREST AND FIELD SURVEYS

INTRODUCTION

Field survey data were gathered for the ACPA tie bar study (Mallela et al. 2009) regarding the in-service performance of longitudinal joints from 24 sites in various states, including Colorado. The survey acquired information on characteristics such as roadway geometry, traffic volume, lane configuration, and tie bar design to determine any factors that may contribute to longitudinal joint distresses.

The evidence revealed that excessive joint openings and loss of load transfer appeared to be the predominant modes of joint failures. Furthermore, the joint performance was highly variable, sometimes ranging from excellent to poor on the same segment of roadway with apparently similar characteristics. While concluding that the current state of the practice for tie bar design may not be adequate, the findings of the survey identified inadequate design or poor construction practices as plausible causes for joint failures. However, this survey, conducted through telephone interviews, did not provide an adequate level of detail to draw definite conclusions on longitudinal joint performance. Factors such as load transfer at the joints, tie bar spacing and alignment, and joint movement at the opening due to cyclical temperature changes should be taken into account. Hence, there is a need for more detailed investigations to understand the behavior of longitudinal joints relating to the adequacy of tie bar design, placement practices, and the role of environmental factors such as cyclical temperature changes and pavement shrinkage.

SITE SELECTION

For this project, an ideal field site was regarded as one that allowed the researchers to conduct side-by-side investigation of longitudinal joints, with and without tie bars, and joints connecting lane-to-lane and lane-to-shoulder, placed on various base material types. It was considered important to identify sites with good and poorly performing joints so as to understand better the impact of design inadequacy and poor construction practices on performance differences. Moreover, reference joint openings (i.e., joint openings at the time of construction) would be needed to arrive at definitive conclusions. However, considering the practical constraints associated with the availability of such sites and measurements and factors such as weather patterns and operational issues, the project team identified in-service field sections that matched, inasmuch as practical, the definition of an ideal site. The following additional criteria were used in site selection:

- Test sites with a high degree including such factors as tangent sections, avoiding on ramps, and level topography with maximum sight distance.
- Each test section was to include both closed joints and joints with openings 1 in. or more plus distress such as settlement.
- Ideally, each test section would have a single uniform pavement and base design.
- Test sites with high traffic levels with a substantial percentage of trucks.
- For practicality, data collection was limited to a 1.0-mile lane closure with adequate room in adjacent lanes for oversized loads and local traffic volume.

To maximize the amount of data collected and efficiency of the field efforts, the main highways and interstates in and around the greater Denver area were scouted. The initial site selection process also involved visual surveys and review of the highway designs.

Field testing was carried out in two rounds: one in fall 2008 and another in fall 2010. In the first round of testing, various field testing methods were employed to establish trends of longitudinal joint behavior. Round two testing built on the findings of the round one testing.

FIELD TESTING PLAN AND TEST METHODS

The field testing plan used in this study included the following. Anywhere from 8 to 13 sets of measurements were obtained at each site.

- Movement of the longitudinal joints: The width the joint openings was measured over several hours from early morning until mid-afternoon. The measurement period was considered adequate to capture the diurnal variations, as the joint movement is open the widest during early morning hours and narrowest during the early afternoon period.

- Falling Weight Deflectometer (FWD) measurements: Deflection testing was conducted at the identified lane-shoulder or lane-lane joint at various times during the day (simultaneously with joint movement measurement). In most instances, the testing was restricted to lane-shoulder joints due to traffic control issues.
- PCC slab temperature measurements: The PCC slab temperature was measured, simultaneously with joint movement measurement and FWD testing, at three different depths—1 in. below the surface, mid-depth, and 1 in. from the bottom of the concrete layer.
- Dowel bar position and alignment: A MIT Scan device was used to locate the position and alignment of tie bars.

The first round of testing included all the tests mentioned above, while in the second round, only MIT Scan testing was conducted.

Longitudinal Joint Movement Measurement

Joint movements were measured in accordance with the protocols of the LTPP Seasonal Monitoring Program (FHWA, 1994). The testing involves drilling ½-in. diameter holes on both sides of the joint at mid-length and near the ends of the joint, placing ½-in. high modulus steel snap rings in the holes below the pavement surface, and measuring the joint movement using a digital caliper. The snap rings improve repeatability and accuracy of the measurement by providing a sharp edge for the caliper jaws to touch each time a measurement is taken. Figure 10 illustrates a typical snap ring set-up at a joint. Carbon blocks of known widths were used as standards and were checked with the calipers periodically to ensure calibration before testing was conducted.

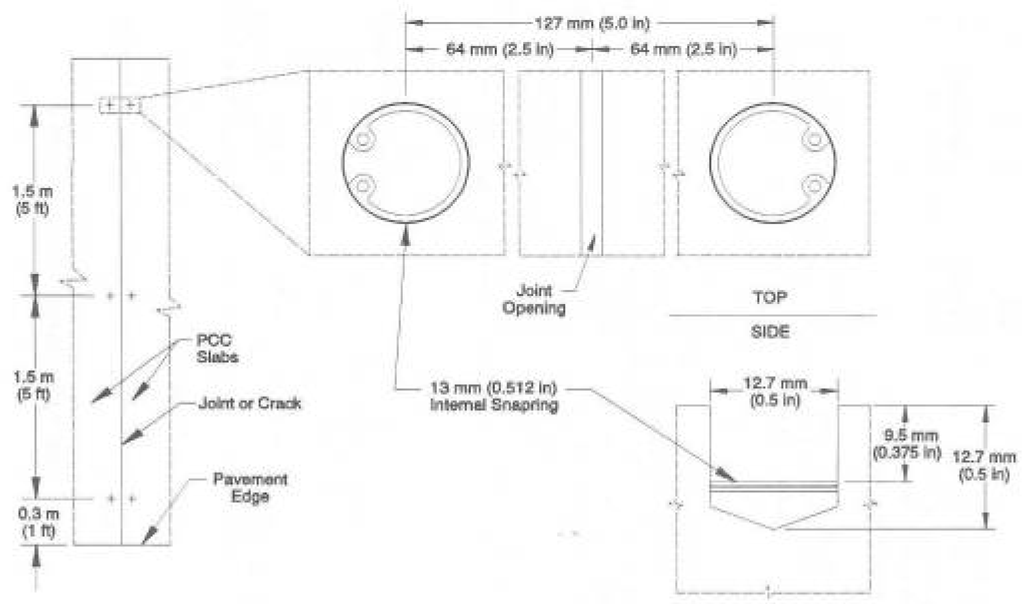

Figure 10. Schematic of a snap ring testing set up for joint movement measurements (FHWA, 1994).

Joint movements were measured at each location with an electronic digital caliper having +/- 0.39-mil precision. The reported joint movement is the average value of the three measurements from the mid-length and near the ends of the joint.

Falling Weight Deflectometer Testing

For this study, CDOT conducted testing using a JILS FWD. The device was configured with a side sensor to enable the measurement of deflections across the longitudinal joints and their LTEs.

During testing, the FWD load plate was positioned tangentially on one side of the longitudinal joint (usually at the edge of the outside or driving lane), and the deflections were measured at both sides of the joint. Figures 11 and 12 show the FWD positioned to test longitudinal joint LTE at the lane-shoulder joint. The resulting deflections at the center of the load plate and across the joint were measured and recorded to compute the joint LTE.

Figure 11. FWD configured for longitudinal joint LTE testing.

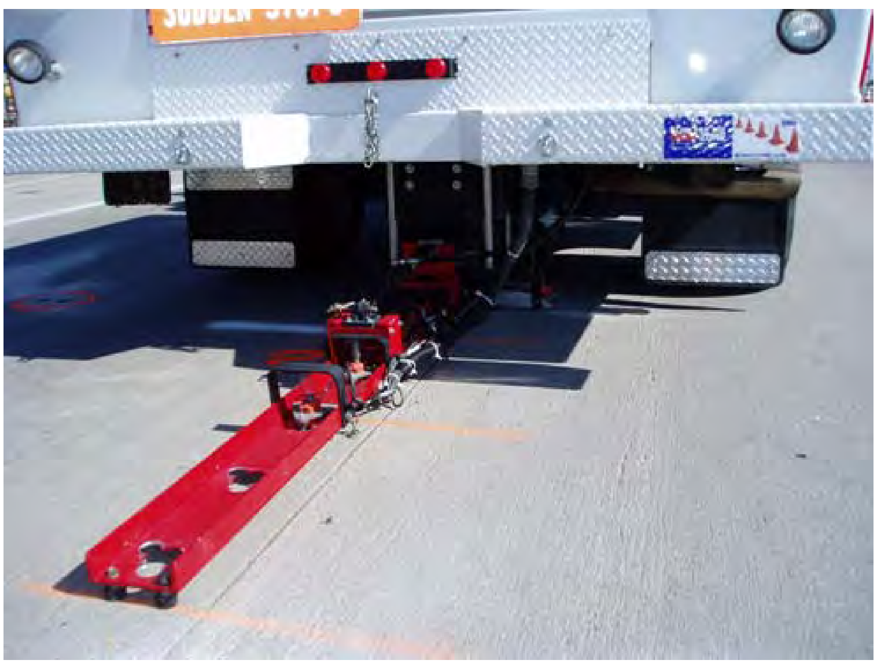

Figure 12. Close-up of FWD load plate and side sensor.

The drop sequence used in the testing followed the standard LTPP load sequence for rigid pavement testing (see Table 21).

Table 21. LTPP FWD loading sequence for longitudinal joint LTE testing.

FWD Drop Position Number	Number of Drops	Target Load (lb)	Acceptable Range (lb)
2	4	9,000	8,100 to 9,900
3	4	12,000	10,800 to 13,200
4	4	16,000	14,400 to 17,600

Pavement Temperature Measurement

During testing, three ½-in. diameter holes were drilled for each joint location near the center of the concrete slab for pavement temperature monitoring. After drilling was completed, the holes were cleared of dust, and 0.07 oz of mineral oil measured with an oral syringe was dispensed into them to provide thermal conductivity between the pavement and the temperature probe. Temperatures were taken at three different depths with a +/- 0.18 °F k-type digital probe at the same time the joint measurements were taken. The mid-depth temperature was used for data analysis. The top and bottom slab temperatures were used for error checking the mid-depth slab temperatures.

MIT Scan Testing

MIT Scan testing was conducted to determine the tie bar position relative to the longitudinal joint being tested. The measuring process involves setting rails on the joint to be scanned, entering pavement information such as concrete thickness and tie bar size into the on-board computer, and pulling the unit along the joint. During testing, the device emits a weak, pulsating magnetic signal and detects the transient magnetic response signal induced in the metal tie bars. The response signals are measured with high precision using special receivers in the device. Methods of tomography are used to determine the position and/or alignment of the tie bars. Figure 13 illustrates MIT Scan testing of a transverse joint for dowel bar detection.

Figure 13. Pulling the MIT Scan device along the rail system.

Round One Testing

Three sites were chosen for the first round of testing:

- US 40 (Hugo site).
- I-70 Mile Post 289 (Gun Club site).
- I-70 Mile Post 316 (Byers site).

CDOT provided traffic control and FWD testing while ARA recorded longitudinal joint movement and pavement temperatures.

<u>US 40, Hugo Site</u>

Longitudinal joint testing was conducted on US 40 just east of the town of Hugo on September 29, 2008. Lane width was 12 ft, and slab length was 15 ft. The pavement was 9.5-in. PCC over a 7 to 10-in. hot mix asphalt concrete (HMAC) base on a 10-in. silty sand subbase. Six joints were identified between the two westbound lanes; three closed joints were designated as Control Joints A, B, and C, and three open joints were designated as Test Joints A, B, and C. The width of joint opening in test joints ranged from 1 to 1½ in.

Figure 14 shows Control Joint A with a low-serverity crack above a tie bar, typical of all joints in the control section. Also shown is Test Joint B, showing the sealant detached and stretched beyond usefulness.

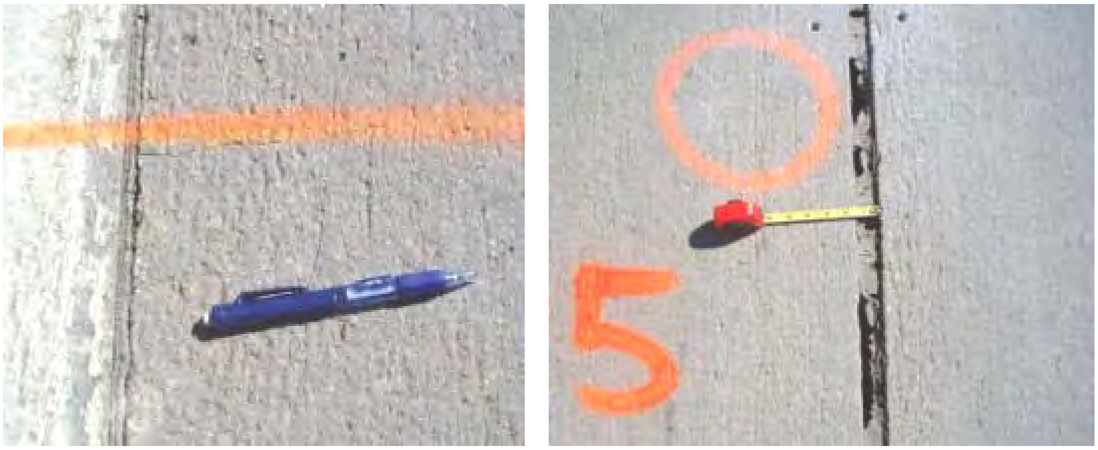

Figure 14. Control Joint A (left) with low-severity crack above a tie bar and Test Joint B (right) showing the sealant detached and with 1¼-in. opening.

LTE for the control joints remained relatively constant as the pavement temperature changed, whereas LTE for the test joints increased with increased pavement temperature, as shown in Figure 15. This change could be explained by the impact of increasing of aggregate interlock due to reduction of joint opening. Joint movement was twice as much for the test joints as for the control joints at a temperature change of about 10 °F, as shown in Figure 16.

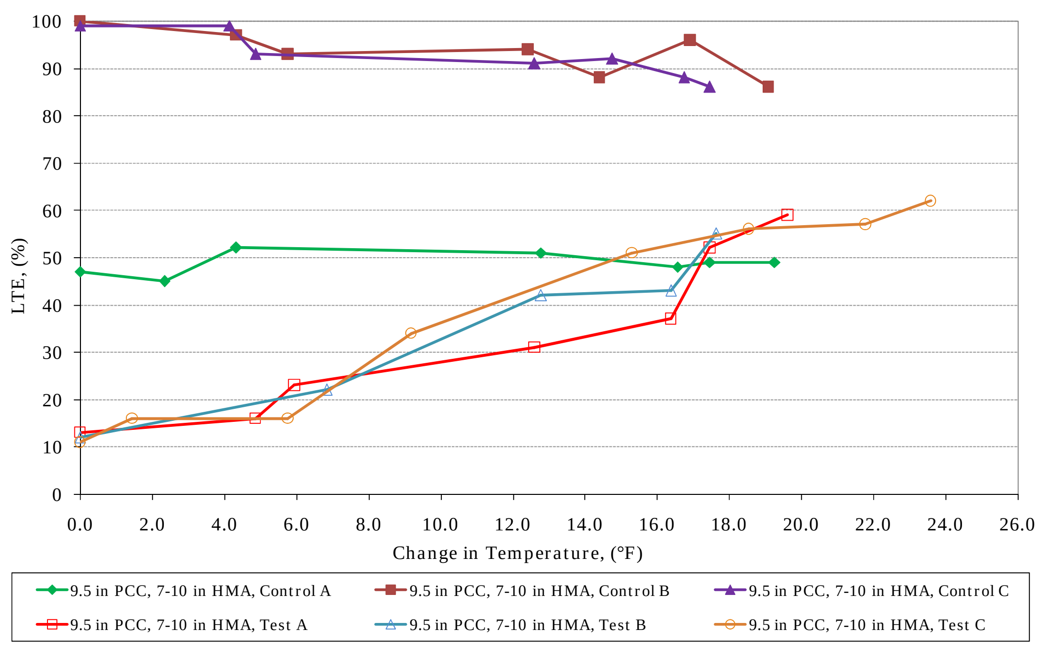

Figure 15. LTE at Hugo site.

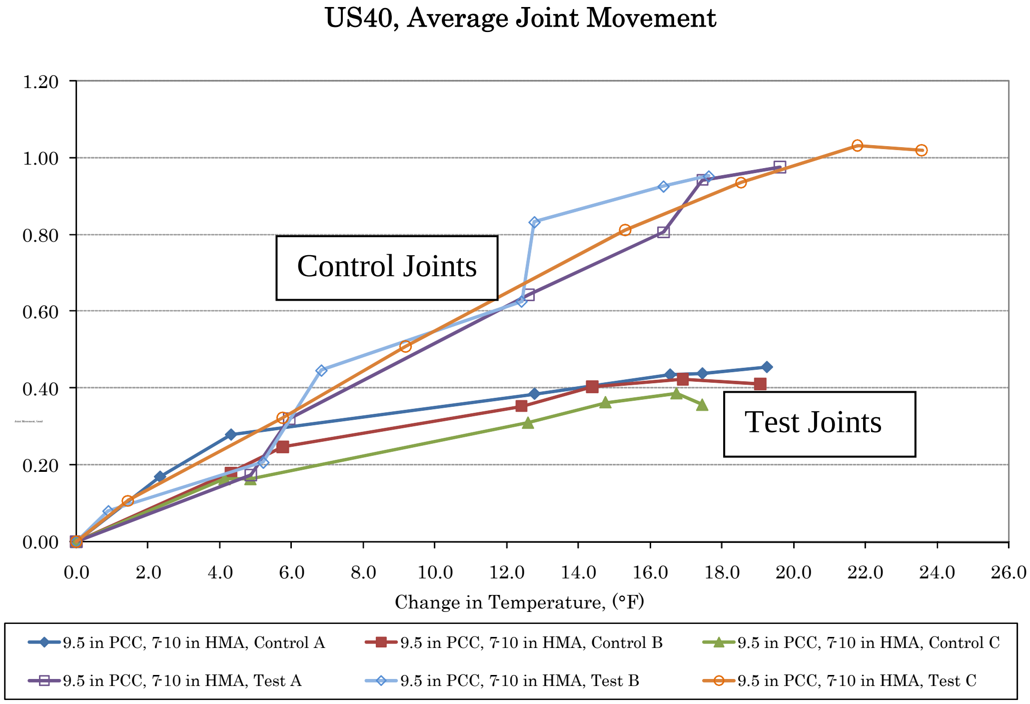

Figure 16. Joint movement at Hugo site.

The MIT Scan test results were used to evaluate the alignment of tie bars at these joints and the uniformity of the depth of tie bar placement. Figure 17 shows the MIT Scan contour images of Control Joints A, B, and C (arranged from left to right), which indicate that the tie bars in the control joints are aligned properly with no tangible evidence of depth non-uniformity. The MIT Scan of the test joints, however, produced indiscernible images, indicating severe misalignment, or blank images, indicating the device's inability to detect tie bars. MIT Scan tests show the mean depth to tie bar was 3.1 inches in the control joints, shallower than the mid-thickness depth of 4.75 in.

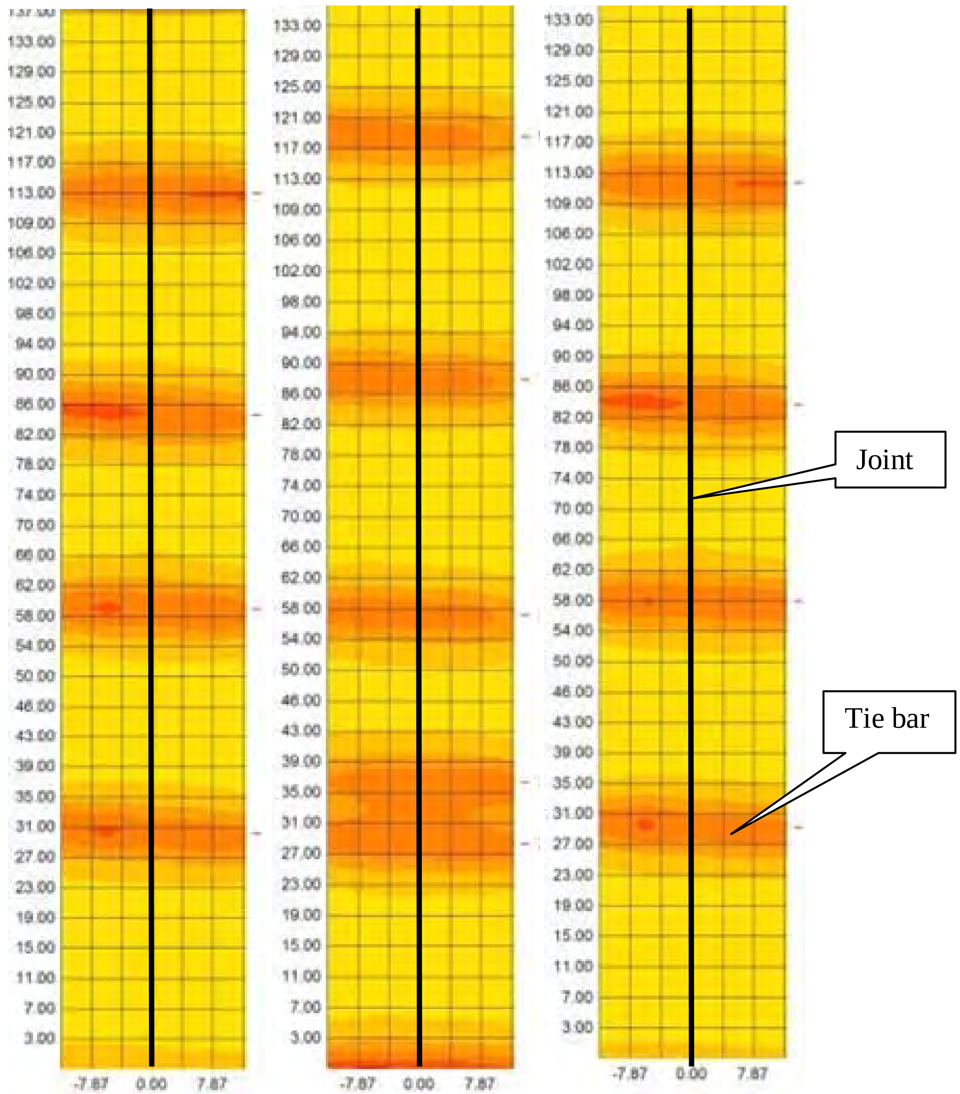

Figure 17. MIT Scan images of control joints at Hugo site (A left, B center, C right all show tie bars in place).

I-70, Gun Club Site

Longitudinal joint testing was conducted on I-70 near the entrance ramp from Gun Club Road on September 30, 2008. The outer lane width was 12 ft, and slab length was 15 ft. The pavement was 11-in. PCC over a 6-in. aggregate base for both the lane and the shoulder. Three joints were selected between the eastbound outer lane and the outer shoulder. All these joints were identified as performing poorly, with joint openings ranging between ⅜ and ½ in. No control joints were available at this site.

Figure 18 shows a close-up view of Test Joint A with the joint sealant deeply recessed and partially attached to the face of the joint. Figure 19 indicates the angular skew (as indicated by the direction of tape measure in the left image) of the tie bar as discovered by coring.

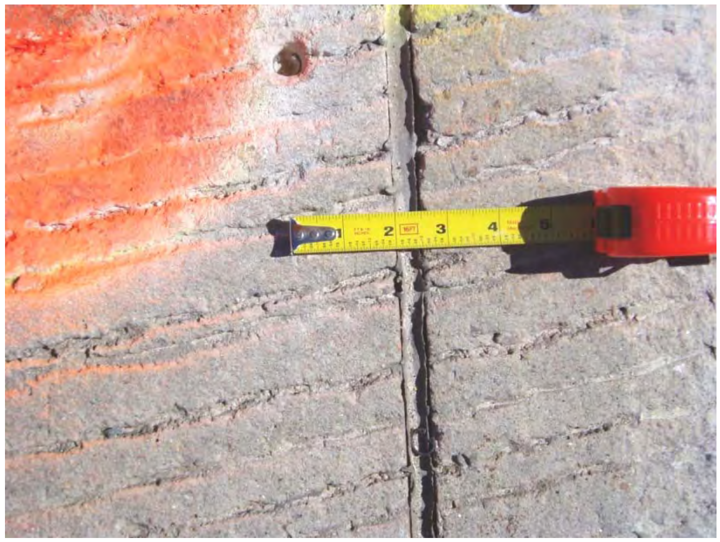

Figure 18. Close-up view of Test Joint A at Gun Club Road site.

Figure 19. Diagonal orientation of tie bar across joint (left) and the 11-in. core (right).

All three joints are relatively the same in terms of construction and tie bar placement; however, as shown in Figure 20, the load transfer on Test Joint A remained steady with temperature variations, while the other joints showed an increase in LTE as the temperature increased. Figure 21 shows a halted but similar trend in joint movement with temperature change.

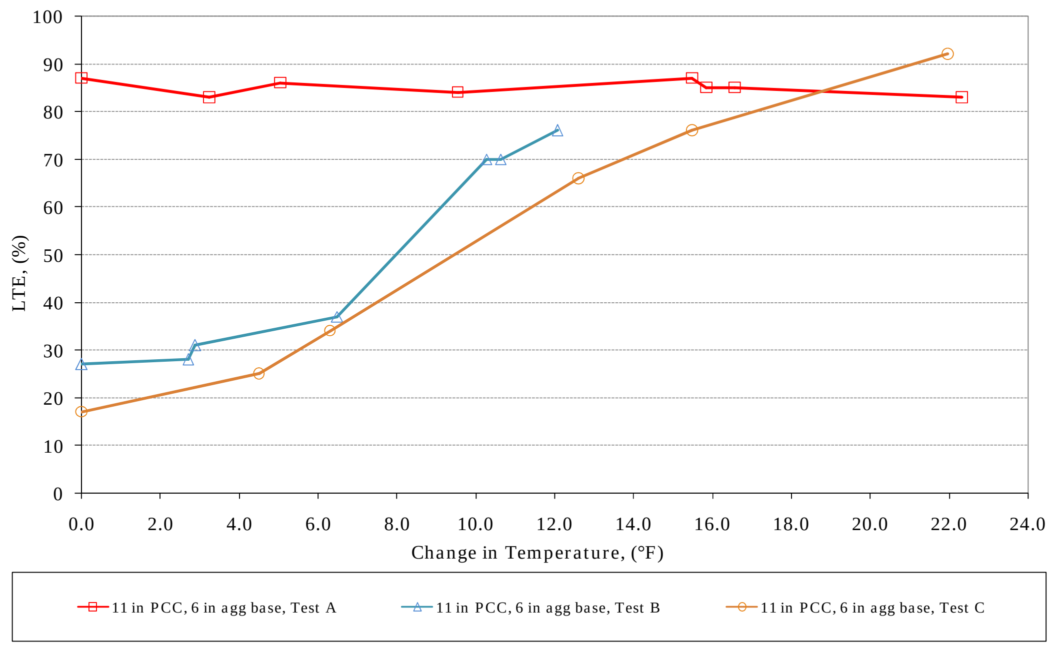

Figure 20. LTE at Gun Club Road site.

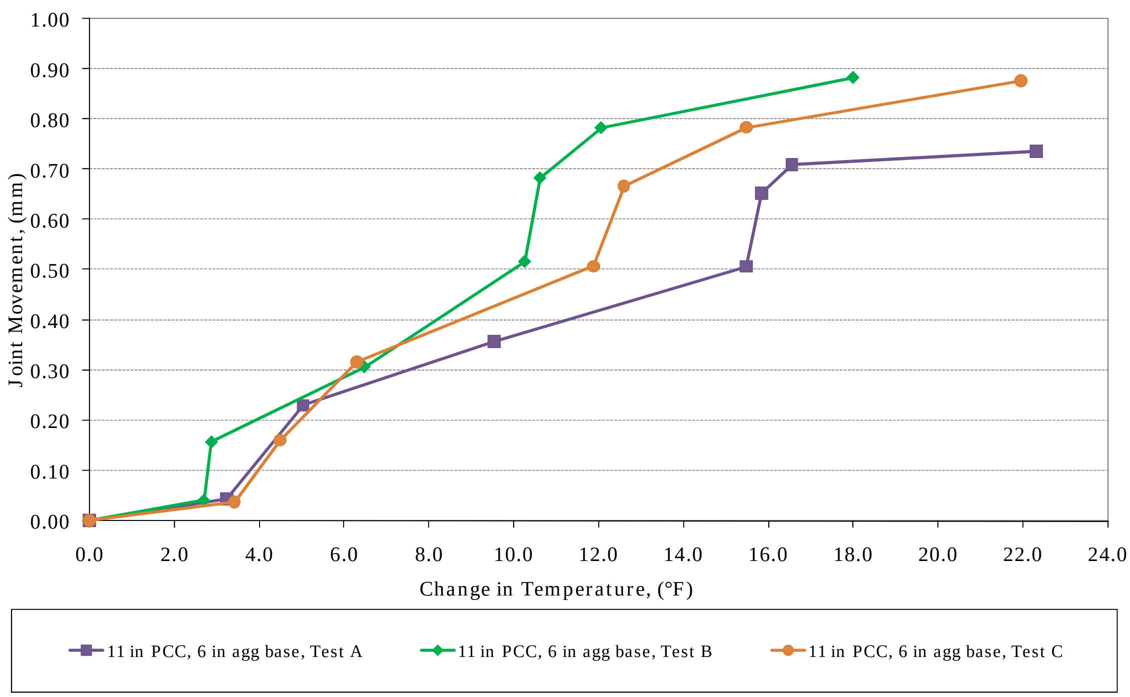

Figure 21. Joint movement at Gun Club Road site.

Tie bar depth could not be determined because the tie bars were too severely skewed for the MIT Scan to generate alignment data. Figure 22 shows only very weak contours (indiscernible images) of the tie bars positioned diagonally across the joints. Heavy contours at the lower portion of each scan result from the interference effect of dowel bars at the transverse joint.

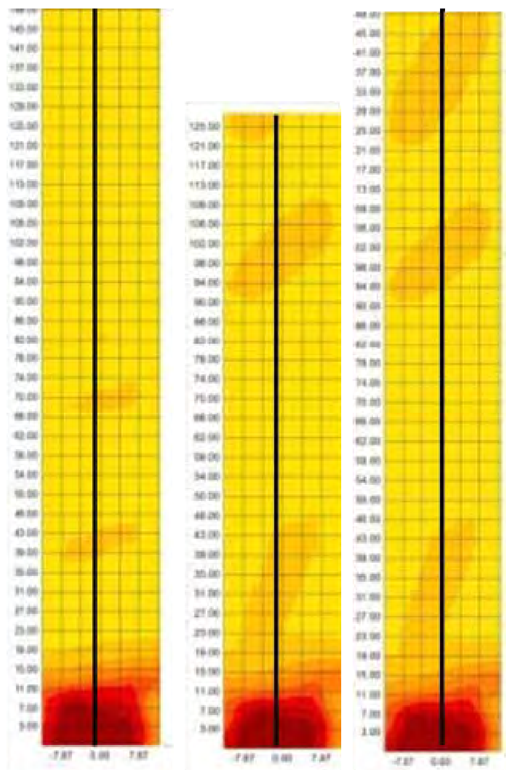

Figure 22. MIT Scan images of longitudinal joints at Gun Club Road site.

<u>I-70, Byers Site</u>

Longitudinal joint testing was conducted on I-70 near the exit ramp to Byers on October 1, 2008. The pavement was 10.5-in. PCC over dense graded crushed stone base with variable spaced (17- to 20-ft) transverse joints and 11.8-ft lane width. Three poorly performing joints, designated as Test Joints A, B, and C, were identified between the eastbound inner and outer mainline lanes. The width of joint openings ranged between 1 and 1½ in. Figure 23 shows Test Joint A and a close-up of the widely open joint filled with fines. Joint sealant in the test section was not attached to the face of the joints or was not present. Figure 24 shows coring operations and a 10.5-in. core extracted from Test Joint A.

Figure 23. Test Joints A, B, and C (left) and close-up of Test Joint A (right).

Figure 24. Cutting core sample (left) and the 10.5-in. core (right) from Test Joint A.

Control Joints A, B, and C, shown in Figure 25, were esablished 133 ft east in a slightly newer constructed section of I-70 with 14-in. PCC over dense graded crushed stone base with 12-ft lane width and 15-ft slab length. Figure 26 shows images of the core location and 14-in. core from Control Joint C.

Figure 25. Control Joints A, B, and C (left) and close-up of Control Joint B (right).

Figure 26. Core from Control Joint C (left) and the extracted 14-in. core (right).

The results of LTE testing and joint movements are shown in Figures 27 and 28, respectively. Both the test and control joints generally show a slight increase in LTE as temperature increases. The test and control joints show similar trends in increased joint movement ranging from 0.45 mm to 0.70 mm at a temperature change of about 10 °F.

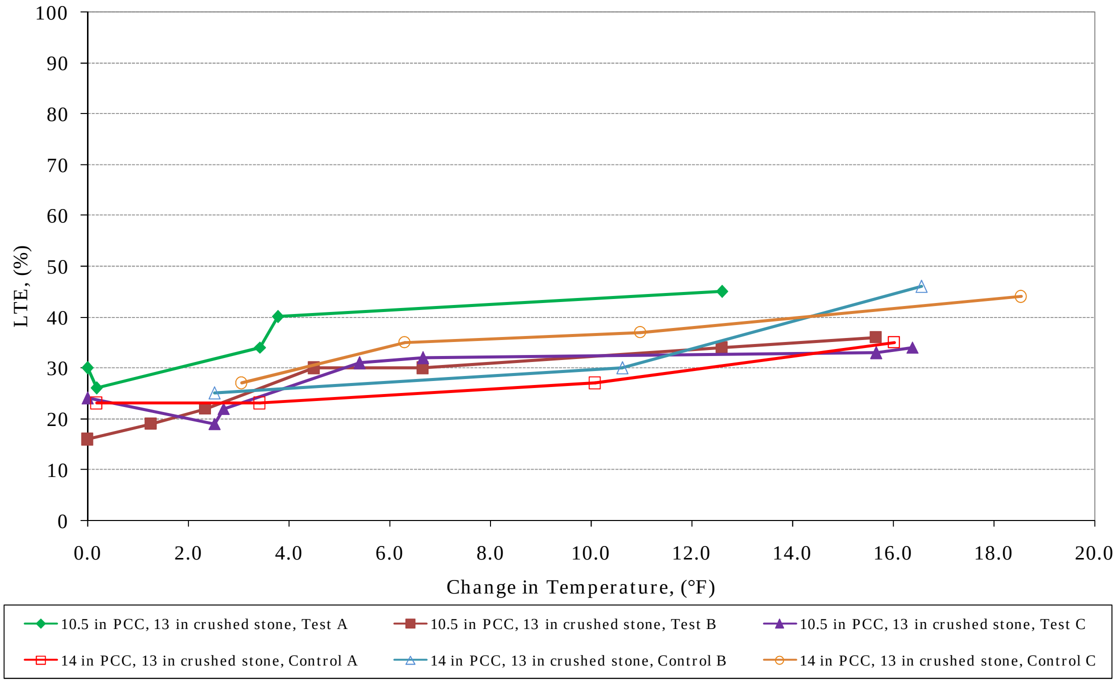

Figure 27. LTE at Byers site.

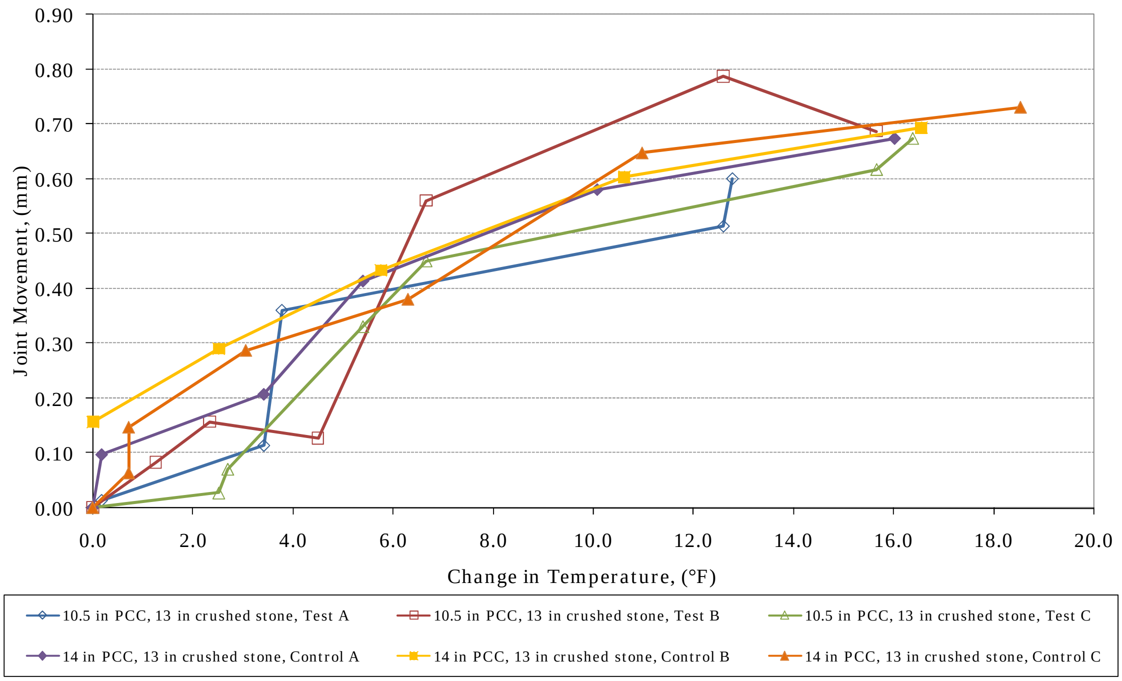

Figure 28. Joint movement at Byers site.

Figure 29 shows MIT Scan contour images for Test Joints A and B (first and second images from the left) and Control Joints A, B, and C (third, fourth, and fifth images from the left). The contours for the test joints indicate that the tie bars are either missing or out of alignment, as only one end of the bars are visible. The contour plot for Test Joint C was blank, indicating the tie bars were misplaced completely. The contours for the control joints indicate that the tie bars were normally aligned with an average placement depth of 6 in. from the surface. However, the contours of the control joints for this site were not as sharp as those from the Hugo site, likely indicating non-uniform placement depth of tie bars at these joints.

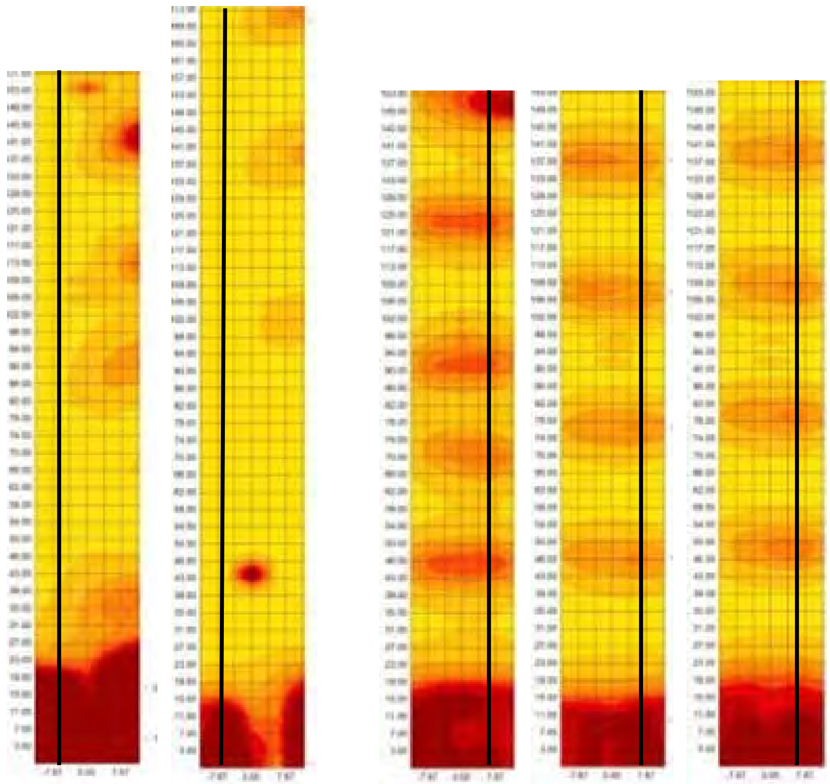

Figure 29. MIT Scan images of longitudinal joints at Byers site.

Observations from Round One Testing

The results of the first round of field testing indicate the following:

- LTE generally increases with increasing temperature for test joints, as the joint opening width tends to get tighter during the mid-afternoon. Test Joint A at the Gun Club Road site was an exception, where the LTE remained steady with temperature changes.

- For control joints, the change in LTE with temperature is either negligible (as in the case of the Hugo site) or less pronounced than for the test joints (as in the case of the Byers site).
- The trends indicate that the joint movement, as expected, is a function of pavement temperature—joints tend to close as temperature rises. However, the trends indicating the difference in joint movements between control and test joints were mixed. At the Hugo site, the joint movements of control joints were less than half those of test joints, while the joint movements were the same at the Byers site.
- The measured joint movement ranged from 0.25 in. to 0.7 in. at a temperature difference of 10 °F. These values were excessive and similar to joint movement of non-tied slabs. This observation implies that some tied joints perform as poorly as non-tied slabs, thus indicating the possibility of tie bar failure due to loss of concrete-steel bonding or yielding of tie bar steel.
- Tie bars were severely misaligned for all test joints at all three sites. In some cases, the MIT Scan produced blank images, as the device was unable to detect the presence of tie bars. The MIT Scan indicated the presence of normally aligned tie bars for all control joints at all three sites.
- The results of MIT Scan testing further indicated a possible impact of tie bar misalignment (angular skew in longitudinal and or transverse directions) or misplacement (inadequate embedment or absence of tie bar on one side of the joint) on poor longitudinal joint performance.

Round Two Testing

The purpose of the second round of testing was to evaluate the impact of tie bar misalignment and misplacement on poor longitudinal joint performance. Following discussions with CDOT personnel, five sites were selected for this round:

- I-70 Mile Post 308.2 (Byers site).
- I-70 Mile Post 316 (Byers site).
- I-70 Mile Post 324 (Byers site).
- I-225 Mile Post 8.5 (Aurora site).

- I-225 Mile Post 10.5 (Aurora site).

Based on the visual assessment of MIT Scan images, the tested joints were grouped into three categories--I, II, and III. The scans of joints identified as Category I typically showed evenly spaced tie bars crossing the longitudinal joint, and the joint opening ranged from 0.3 to 1.1 in. at the time of measurement. Category II joints had the same joint openings as those of Category I, but the scan indicated missing or misaligned tie bars that did not cross the joint in all cases. Category III joints had wider joint openings ranging from 0.3 to 2.15 in. The tie bars at Category III joints did not cross the joints and were severely misaligned or even missing in some cases.

<u>I-70 Mile Post 308.2, Byers Site</u>

MIT Scan testing was conducted on the two eastbound lanes of I-70 at mile post 308.2. Twenty-three lane-lane joints were tested. The lane at this site width was 12 ft. The pavement was 14-in. PCC over either lean concrete or silty sand base. The tie bars were No. 6 size at 30-in. spacing.

This site had 10 Category I joints that measured 0.3 in. wide. The scans revealed four evenly spaced tie bars per joint. The transverse joints in this section were saw cut every 15 ft perpendicular to the longitudinal joint. Figure 30 is a typical scan showing returns of the four tie bars evenly spaced along the joint.

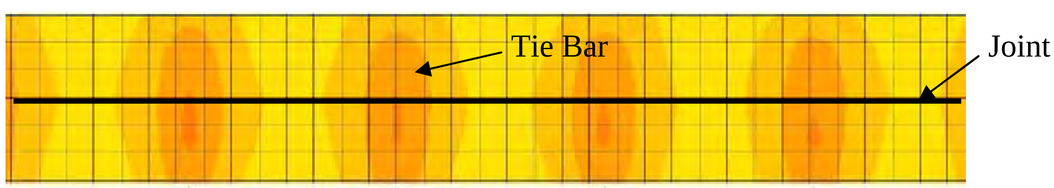

Figure 30. Scan of a typical Category I joint at the I-70 mile post 308.2 site.

An adjacent set of 13 slabs at the same test site had an AASHTO Classification A-2 granular base with skewed transverse joints at variable longitudinal joint lengths of 17, 18, 19, and 20 ft. The joint widths varied from 0.3 to 0.5 in. despite being classified as Categories II and II. Three Category II joints were identified, with the scan of each joint showing five moderately to severely misaligned tie bars (Figure 31) that in some cases did not fully cross the joint.

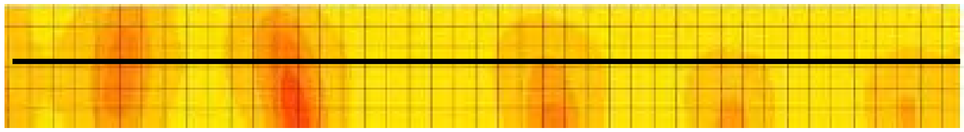

Figure 31. Scan of a typical Category II joint at the I-70 mile post 308.2 site.

The remaining 10 joints were classified as Category III. The scans indicated that most of these joints had no tie bars. In one joint, as shown in Figure 32, the tie bars were severely misaligned and appeared to be found near the bottom of the pavement. None of the tie bars crossed the joint. The alignment of the tie bars in Category III could not be calculated.

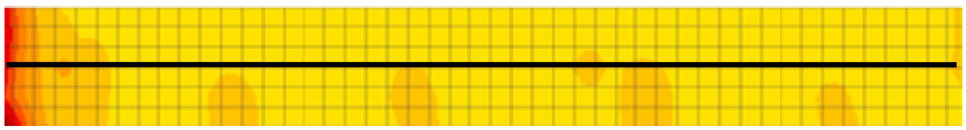

Figure 32. Scan of a Category III joint at the I-70 mile post 308.2 site.

As indicated in Table 22, Category I joints had bars positioned on average 1.19 in. above the mid-depth, while for Category II joints, the tie bars were positioned on average 4.18 in. below the mid-depth. In this site, though the tie bars were misaligned (i.e., Category II), the joint opening between the two categories were almost the same. However, the joint openings for Category III joints were wider.

Table 22. Tie bar alignment summary table for the I-70 mile post 308.2 site.

Joint Number	Category	Joint Opening (in.)	Average Depth Deviation (in.)	Average Side Shift (embedment) (in.)	Average Misalignment (in.)	Average Horizontal Misalignment (in.)	Average Vertical Misalignment (in.)
1-10	I	0.3	1.19	0.92	0.70	0.30	0.60
11-13	II	0.3	4.18	1.45	2.24	1.63	1.20
14-23	III	0.3 - 0.5	--	--	--	--	--

<u>I-70 Mile Post 316, Byers Site</u>

MIT Scan testing was conducted on the two eastbound lanes of I-70 at mile post 316. Twenty lane-lane joints were tested. Lane width was 12 ft. The slab was placed monolithically and joints were sawcut. The transverse joints were skewed at 17, 18, 19, and 20 ft. The tie bars were No. 6 size at 30-in. spacing. The pavement was 11-in. PCC over silty sand base.

This site had 12 Category II joints, each 0.4 in. wide, and 8 Category III joints open 1.4 to 2.15 in. The scans of the Category II joints show the tie bars several inches below the pavement mid-depth, and in nearly all scans the bars cross the joint but were severely misaligned, as shown in Figure 33.

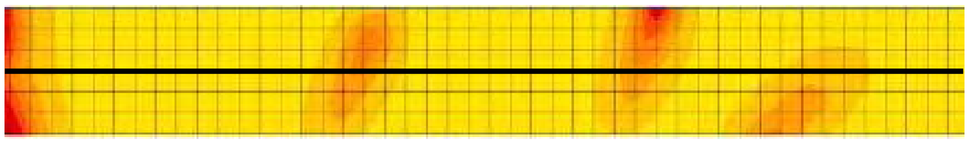

Figure 33. Scan of a typical Category II joint at the I-70 mile post 316 site.

The scans of the Category III joints show each joint with one to five severely misaligned tie bars, none of which cross the joint. Figure 34 is a typical scan in which four tie bars are shown embedded in only one side of the joint, with none crossing the joint. The alignment data for the Category II joints are summarized in Table 23. The alignment of the tie bars in Category III joints could not be calculated.

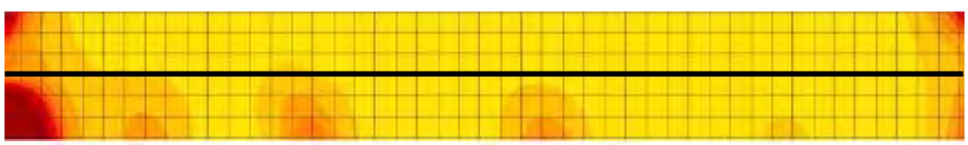

Figure 34. Scan of a typical Category III joint at the I-70 mile post 316 site.

Table 23. Tie bar alignment summary table for the I-70 mile post 316 site.

Joint Number	Category	Joint Opening (in.)	Average Depth Deviation (in.)	Average Side Shift (embedment) (in.)	Average Misalignment (in.)	Average Horizontal Misalignment (in.)	Average Vertical Misalignment (in.)
1-12	II	0.4	3.23	2.59	6.03	5.44	1.52
13-20	III	1.4 - 2.15	--	--	--	--	--

Though misaligned and positioned several inches below mid-depth, the joint openings at Category II joints at the time of measurement were smaller. However, for joints where the tie bars went undetected in MIT Scan testing, the joint opening was much wider, ranging between 1.4 and 2.15 in.

I-70 Mile Post 324, Byers Site

MIT Scan testing was conducted on the two eastbound lanes of I-70 at mile post 324. The lane width was 12 ft. The slab was placed monolithically, and joints were sawcut. The transverse joints were skewed at 17, 18, 19, and 20 ft. The tie bars were No.6 size at 30-in. spacing. The pavement was 11.5-in. PCC over silty sand base.

Eighteen lane-lane joints were scanned at this site. The first eight joints were classified as Category I, with joint openings 0.3 to 0.35 in. wide. The scans show either five or six tie bars per joint, depending on joint length, and in all but one scan the tie bars were spaced evenly. Four of the eight scans show returns from the tie bars plus unusual returns (Figure 35), suggesting interference from an unknown metallic source in or below the pavement. A typical contour scan without any interference from a metallic source is shown in Figure 36.

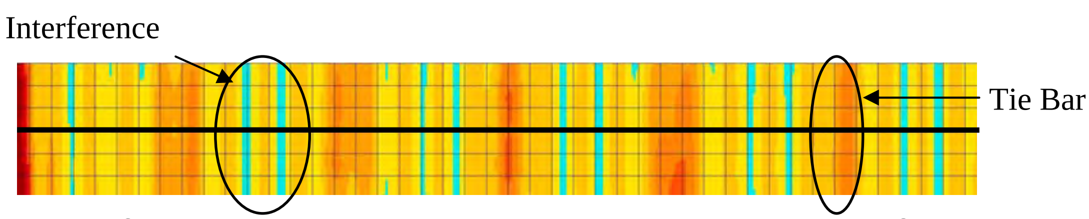

Figure 35. Scan of a typical Category I joint showing the tie bars and interference at the I-70 mile post 324 site.

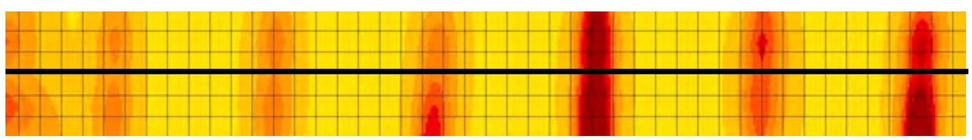

Figure 36. Scan of a typical Category I joint without interference at the I-70 mile post 324 site.

Of the remaining 10 joints, 9 were still identified as Category I, as the joint opening ranged from 0.4 and 1.1 in. and the tie bars were normally aligned and evenly spaced. The tenth joint, with an opening of 1.0 in., had several tie bars missing, and hence was identified as Category II.

Nine Category I joints had five, six, or seven evenly spaced tie bars per joint, depending on the joint length. The MIT Scan software produced images of the tie bars but was unable to calculate the tie bar positions. Judging by the strong and weak returns of the tie bars in the scans, such as illustrated in Figure 37, the tie bars likely were positioned several inches above or below the mid-depth of the pavement. The alignment data are summarized in Table 24.

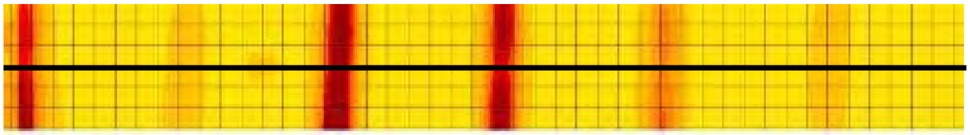

Figure 37. Scan of a typical Category I joint at the I-70 mile post 324 site.

Table 24. Tie bar alignment summary table at the I-70 mile post 324 site.

Joint Number	Category	Joint Opening (in.)	Average Depth Deviation (in.)	Average Side Shift (embedment) (in.)	Average Misalignment (in.)	Average Horizontal Misalignment (in.)	Average Vertical Misalignment (in.)
1-8	I	0.3 - 0.35	3.77	0.89	1.44	0.92	0.87
9	II	1.0	--	--	--	--	--
10-18	I	0.4 - 1.1	--	--	--	--	--

The width of the joint openings in this test section ranged from 0.3 to 1.1 in. at the time of measurement. In nearly all cases, the bars were located well above mid-depth in the top third of the slab, crossed the joint, and on average displayed less than 1 in. of horizontal or vertical misalignment. The presence, location, and alignment of the bars was reasonable, suggesting the possibility of inadequate tie bar design in wider joint openings. However, the inadequacy of tie bars could not be confirmed without verifying the construction records for the as-built joint opening at the time of construction.

I-225 Mile Post 8.5, Aurora Site

MIT Scan testing was conducted on the three northbound lanes of I-225 at mile post 8.5. Fifteen lane-lane joints were tested. The pavement was six slabs wide, and the lane width was 12 ft. The transverse joint spacing was 15 ft. The tie bars were No.6 size at 30-in. spacing. The pavement was 13.5-in. PCC over sand and gravel base. MIT Scan testing was performed at the center joints.

All 15 joints were identified as Category I joints, with 5 opened 0.3 in. wide and 10 opened 0.5 to 0.55 in. wide. All scans at this site were similar and showed four evenly spaced tie bars positioned as shown in Figures 38 and 39. Generally, the average misalignments of the tie bars were similar regardless of the width of the joint opening. The alignment data for all joints are summarized in Table 25.

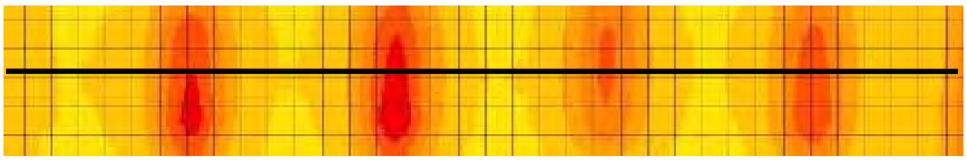

Figure 38. Scan of a typical joint with 0.3-in. opening at the I-225 mile post 8.5 site.

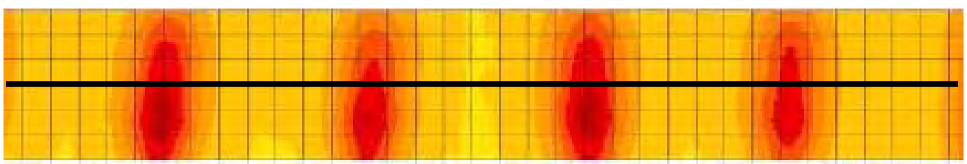

Figure 39. Scan of a typical joint with 0.5-in. opening at the I-225 mile post 8.5 site.

Table 25. Tie bar alignment summary table for the I-225 mile post 8.5 site.

Joint Number	Category	Joint Opening (in.)	Average Depth Deviation (in.)	Average Side Shift (embedment) (in.)	Average Misalignment (in.)	Average Horizontal Misalignment (in.)	Average Vertical Misalignment (in.)
1-5	I	0.3	3.10	1.13	1.15	0.46	0.87
6-15	I	0.5 - 0.55	2.62	0.28	1.22	0.73	0.89

The tie bars were relatively well positioned, suggesting that the current tie bar design may not be adequate for tying six slabs. However, the design inadequacy could not be confirmed without calculating the as-built joint opening at the time of construction.

I-225 Mile Post 8.5, Aurora Site

MIT Scan testing was conducted on the four northbound lanes of I-225 at mile post 8.5. Twenty lane-outer shoulder joints were tested. The lane width was 12 ft. The transverse joint spacing was 15 ft. The tie bars were No.6 size at 30-in. spacing. The pavement was 12.5-in. PCC over silty sand base. MIT Scan testing was conducted at the joints between the far outside lane and shoulder.

Eleven of the 20 joints were classified as Category I and were open 0.3 in. wide. All but one scan showed four evenly spaced tie bars positioned, as shown in Figure 40. One scan showed one of the joints to have only one tie bar.

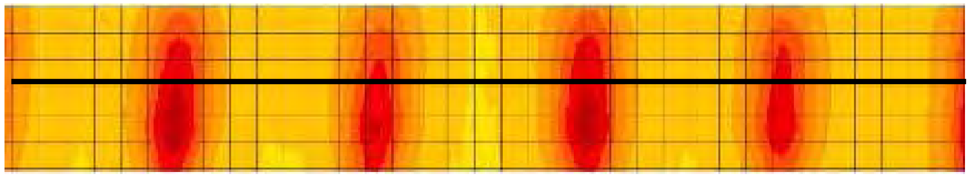

Figure 40. Scan of a typical Category I joint at the I-225 milepost 10.5 site.

The remaining nine joints were Category III and open 1.2 to 1.6 in. wide. The scans show five of the nine joints have four or five tie bars per joint (see Figure 41), but the tie bars do not fully cross the joint. The four remaining scans show no tie bars. The alignment data are summarized in Table 26.

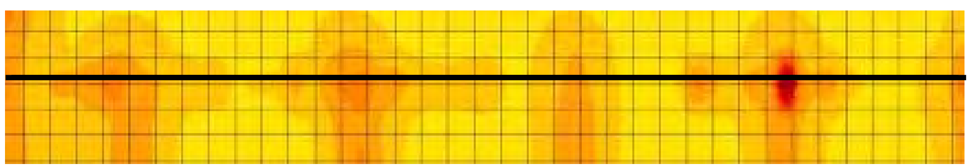

Figure 41. Scan of a typical poorly performing joint at the I-225 mile post 10.5 site.

Table 26. Tie bar alignment summary for the I-225 mile post 10.5 site.

Joint Number	Category	Joint Opening (in.)	Average Depth Deviation (in.)	Average Side Shift (embedment) (in.)	Average Misalignment (in.)	Average Horizontal Misalignment (in.)	Average Vertical Misalignment (in.)
1-11	I	0.3,	2.73	1.14	1.58	0.87	1.16
12-20	III	1.2 – 1.6	2.88	3.98	3.49	1.76	2.71

At this site, the Category I joints showed tighter joint opening at the time of measurement. On the other hand, as in the case of Category II joints, where the joint construction was poor, with none of the tie bars crossing the joint, the joint opening was much wider, ranging between 1.2 and 1.6 in.

Field Observations from Round Two Testing

Based on the MIT Scan measurements in round two, the following observations were made:

- The vertical placement of the tie bar relative to the mid-depth of the slab did not seem to impact joint opening. However, this observation should be verified. A typical minimum cover depth for steel is recommended and used by most agencies.
- In most cases, the construction issues relating to the quality of tie bar placement seem to control the performance of longitudinal joints. The failure to place the tie bars across the joint, thereby assuring minimum embedment, appears to contribute more to wider joint opening than the degree of skewness of the bars. Once again, more testing is recommended to confirm this finding.
- Joint openings were wider when the tie bars did not connect to the two slabs or when the embedment lengths were inadequate. Tie bars with adequate embedment length on both sides of the joint, even when misaligned, appear to work.
- At the I-70 mile post 324 site and the I-225 mile post 8.5 site, all the tie bars were found in place, while the joint opening was higher than the expected. The possible reasons include the inadequacy of tie bar design (e.g., the use of Grade 40 steel) and the amount of joint opening formed at the time of construction. Sufficient data were not available to differentiate the as-built joint opening from the impacts of design inadequacy.

CHAPTER 4. REVIEW OF CDOT'S TIE BAR DESIGN AND CONSTRUCTION PRACTICES

The results of the field investigations indicated design inadequacy and improper installation as plausible reasons for poor performance of longitudinal joints in concrete pavements. Hence, it is necessary to evaluate the adequacy of CDOT's tie bar size and spacing requirements and the specification requirements for proper installation. CDOT's tie bar design guidance was compared with the AASHTO 1993 and M-E tie bar design procedures. Similarly, the CDOT specifications were compared with those of other state DOTs. This chapter discusses the findings of these comparisons.

EVALUATION OF CDOT'S TIE BAR DESIGN PRACTICES

Comparison of CDOT and AASHTO 1993 Tie Bar Design Procedures

Colorado's Standard Plan M-412-1 provides guidelines on tie bar size, center to center spacing, and embedment length. Section 709.03 specifies that tie bars for longitudinal and transverse joints shall conform to AASHTO M 284 and shall be Grade 40, epoxy-coated, and deformed.

CDOT's tie bar design approach is thickness dependent—in other words, larger tie bar sizes are required for thicker concrete slabs. This approach is similar to the tie bar design procedure specified in the AASHTO 1993 *Guide for Design of Pavement Structures*. In addition to the concrete slab thickness, the AASHTO 1993 method takes into account the distance between the free longitudinal edges (a longitudinal joint with no tie bars) in determining the tie bar size and spacing. Figure 42 presents a comparison of tie bar spacing and the maximum allowable distance to the closest free edge for different tie bar sizes. The maximum allowable distance to the closest free edge for a given tie bar size and spacing was computed using the design procedure presented in the AASHTO 1993 Design Guide.

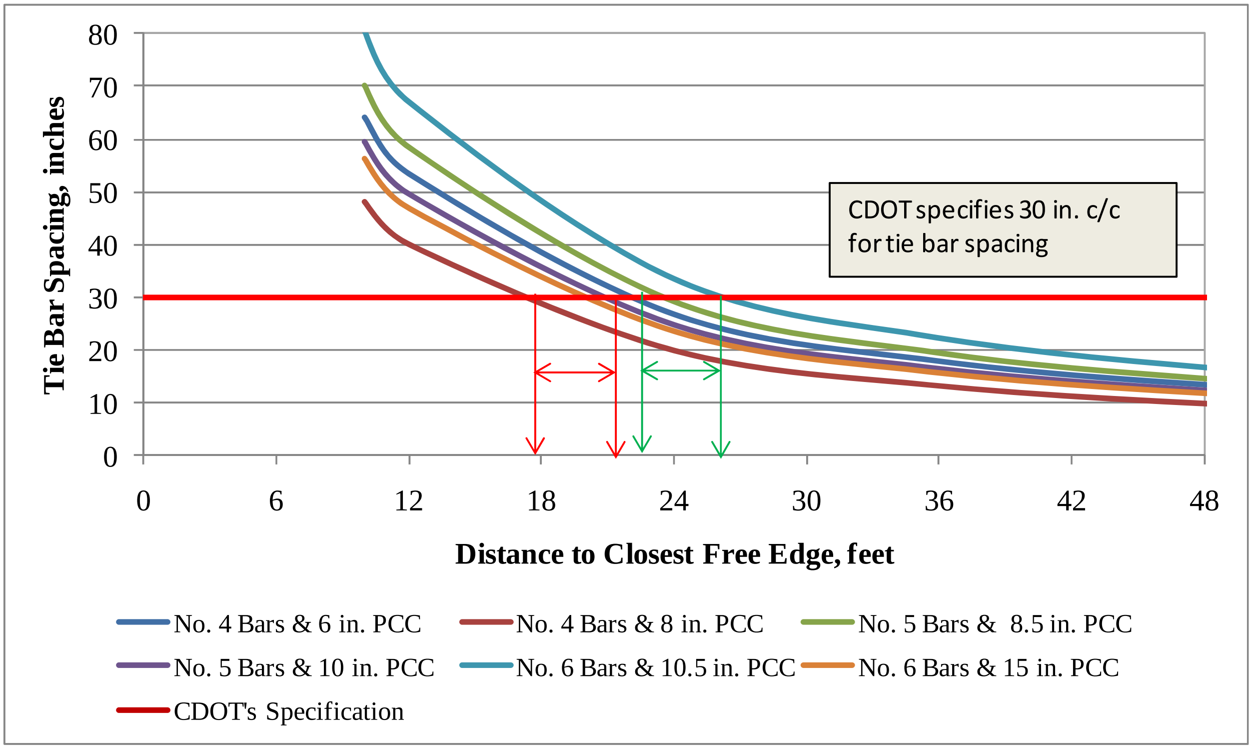

Figure 42. Comparison of CDOT and AASHTO 1993 tie bar design.

(Assuming Grade 40 steel and a subgrade friction of 1.5)

Figure 42 provides both conservative (red arrows) and unconservative (green arrows) estimates of maximum allowable distance to the closest free edge for the CDOT-specified thickness ranges for a given tie bar size. For example, CDOT specifies the use No. 4 bars for concrete slab thicknesses between 6 and 8 in. For a CDOT-specified tie bar spacing of 30 in., following the curve “No. 4 Bars and 6 in. PCC” in Figure 42 indicates an unconservative estimate of 21.3 ft, while the curve “No. 4 Bars and 8 in. PCC” indicates a conservative estimate of 16.0 ft.

Table 27 presents the maximum allowable distance to the closest free edge and the corresponding number of lanes that can be tied together for a range of concrete slab thicknesses using the CDOT guidance on tie bar size and spacing. These estimates, computed using the AASHTO 1993 tie bar design procedure, indicate that the CDOT tie bar designs are comparable with the AASHTO design only when two or three lanes tied are together. In other words, the CDOT tie bar design, if assumed to be based on the AASHTO 1993 tie bar design, seems to be inadequate when four or more lanes are tied together.

Table 27. Maximum allowable distance to closest free edge estimates for CDOT tie bar designs.

Tie bar size at 30 in. spacing	Concrete Slab Thickness (in.)	Max Allowable Distance to Free Edge (in.)	Allowable Number of Tied Lanes
No. 4	6	256	3
No. 4	7	219	3
No. 4	8	192	2
No. 5	9	265	3
No. 5	10	238	3
No. 6	11	307	4
No. 6	12	282	3
No. 6	13	260	3
No. 6	14	241	3
No. 6	15	226	3

Evaluation of AASHTO 1993 Guide Tie Bar Design Procedure

The longitudinal joint tie bar design method in the AASHTO 1993 Design Guide is based on SDT. This approach determines the tie bar size and spacing requirements based on the amount of steel required to pull the slab across the base without yielding the steel. The procedure was developed primarily for determining the amount of distributed steel required to control shrinkage crack widths in long concrete slabs on grade which proved to be inadequate due to many wide failed transverse cracks in long jointed reinforced concrete pavements. The SDT is based on a number of assumptions that may not be practical, and the AASHTO tie bar design procedure inherits these deficiencies. The drawbacks of the AASHTO 1993 procedure are as follows:

- This procedure does not consider the effects of curling and warping stresses of concrete and load stresses at the longitudinal joint.
- Increased slab thickness results in increased slab/base friction and thus increased reinforced content. In reality, the interface condition of the slab and base along with base modulus are much more significant in affecting joint opening.
- Other important factors influencing the movement of joint opening, such as temperature and moisture variations at the highway location (e.g., concrete set temperature minus minimum monthly temperature), concrete mix type (e.g., drying shrinkage, coefficient of thermal contraction), and construction practices (e.g., curing type) are not factored into the procedure.

- This procedure does not consider the greatly increased frictional resistance of modern stabilized base courses.
- This procedure is not particularly sensitive to the base material modulus, taking into account only the coefficient of friction at the slab-base interface and not the base modulus.

Improvements in M-E Tie Bar Design Procedure

The approach recommended in this study addresses the deficiencies of the SDT and offers an improved tie bar design procedure. This approach takes into account various influencing factors that affect longitudinal joint performance:

- Slab-base interface friction parameters.
- Number of lanes/shoulders tied together.
- Climatic variables (temperature, humidity).
- Base material type and modulus/stiffness.
- Concrete mix properties (cement type, cementitious materials content, drying shrinkage, and the coefficient of thermal expansion [CTE]).
- Curing method.

A major distinction with the M-E based tie bar design is the consideration of base material type and much less dependence on the concrete slab thickness. A stabilized base typically is stiff and offers greater frictional resistance at slab-base interface when the adjacent slab edges tend to pull away from a tied joint due to environmentally induced strains. Thus, stabilized bases tend to increase the tensile stresses in the concrete slabs significantly, requiring more steel content and closer spacing to mitigate longitudinal cracking. On the other hand, a less stiff base, such as an unbound aggregate base, tends to offer much less resistance to this opposing movement and results in smaller joint openings. Therefore, using an unbound base course results in lower tensile stresses and requires less steel content and wider spacing.

Another major distinction is the way the M-E based approach incorporates the significance of the number of tied joints in tie bar design. Mallela et al. (2009) found that the tensile stresses in the concrete slab increase as the distance from the joint to the nearest free edge increases. When multiple slabs are tied together, the critical tensile stress occurs in the center of the centermost tied slab. If this stress is high enough to exceed the allowable threshold, longitudinal cracking may result.

Figure 43 shows the average tensile stresses in concrete, computed using the ISLAB2005 analytical model at an equivalent free concrete strain value of 800 microstrains, for various base types and numbers of tied lanes or shoulders. No. 6 tie bars with 36-in. spacing were used in the model. The equivalent free strains for various locations in Colorado are listed in Table 8. As indicated in Figure 43, the average tensile stress through the slab increased significantly as the number of tied lanes increased from two to three. However, this increase in tensile stress was not significant after three lanes. Also indicated in this figure, the average tensile stress for pavement with lean concrete base could not be computed for four lanes or more (using the specified tie bars and spacing), as larger diameter tie bars and/or closer tie bar spacing were required. At the same time, this inadequacy may not be critical, as the increase in tensile stress was not significant for four or more tied lanes.

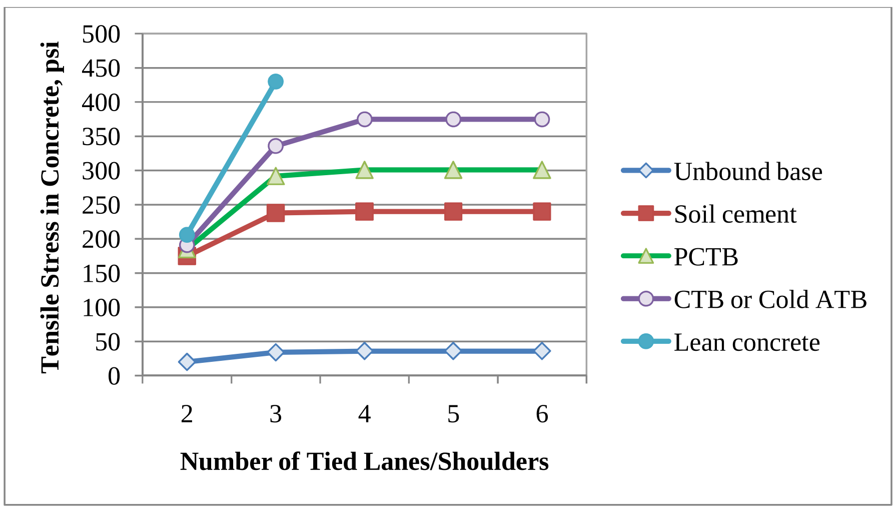

Figure 43. Tensile stress in the concrete slab vs. the number of tied lanes/shoulders.

Comparison of Tie Bar Designs

Tie bar design recommendations (bar size, spacing, and steel grade) determined using the CDOT, AASHTO 1993, and M-E based procedures were compared for 17 sites in Colorado. Table 28 presents the details of these comparisons. The table also presents site-specific information such as the PCC thickness, base type, lane configuration, observed joint width at the time of measurement, and location for use in the design procedure.

Table 28. Comparison of tie bar design recommendations using the CDOT, AASHTO 1993, and M-E procedures.

Site Location	No. of Tied Lanes & Concrete Shoulders	Observed Longitudinal Joint Width	Pavement Cross Section	CDOT			AASHTO			ME*		
				Size	Spacing	Grade	Size	Spacing	Grade	Size	Spacing	Grade
Broadway, south of C-470, Douglas Co.	Three 12-ft lanes + 8-ft tied shoulder (44 ft)	3 to 4 in.	7-in. JPCP over existing subgrade	No. 4	30	40	No. 4	23	40	No. 5	36	40
Wildcat Reserve Parkway, south of Grace Blvd	Five 12-ft lanes + two 10-ft shoulders (80 ft)	1-2 in. at some locations	6- to 7-in. JPCP over natural subgrade	No. 4	30	40	No. 4	14	40	No. 5	36	40
Quebec Blvd., north of Timberline (NB)	Seven 12-ft lanes + 2-ft and 8-ft tied shoulders (94 ft)	1-2 in. at some locations	6- to 7-in. JPCP over natural subgrade	No. 4	30	40	No. 4	12	40	No. 5	36	40
Quebec Blvd., south of Collegiate (NB)	Seven 12-ft lanes + 2-ft and 8-ft tied shoulders (94 ft)	1-2 in. at some locations	6- to 7-in. JPCP over natural subgrade	No. 4	30	40	No. 4	12	40	No. 5	36	40
SH 119, south of US 287	Two 12-ft lanes, curb & cutter (24 ft)	1 to 2 in.	8-in. JPCP over DGAB	No. 5	30	40	No. 5	48**	40	No. 6	36	60
Quebec Blvd., north of Ashburn Lane (SB)	Three 12-ft lanes + 8-ft tied shoulder (44 ft)	1-2 in. at some locations	6- to 7-in. JPCP over natural subgrade	No. 4	30	40	No. 4	23	40	No. 5	36	40
US 287, north of Fort Collins (NB)	One 12-ft lane + 10-ft shoulder (22 ft)	Over 1 in.	9-in. JPCP over DGAB	No. 5	30	40	No. 5	46	40	No. 6	36	60
University Ave., west of Quebec (WB)	Seven 12-ft lanes + 2-ft and 8-ft tied shoulders (94 ft)	2-in at some locations	6- to 7-in. JPCP over natural subgrade	No. 4	30	40	No. 4	12	40	No. 5	36	40
I-25, MP 152-153 (SB)	Two 12-ft lanes +10-ft & 4-ft tied shoulders (38 ft)	1 in	7-in. unbonded JPCP overlay/chip seal/8.-in JPCP	No. 5	30	40	No. 5	32	40	No. 6	36	60

Table 28. Comparison of tie bar design recommendations using the CDOT, AASHTO 1993, and M-E procedures.

Site Location	No. of Tied Lanes & Concrete Shoulders	Observed Longitudinal Joint Width	Pavement Cross Section	CDOT			AASHTO			ME*		
				Size	Spacing	Grade	Size	Spacing	Grade	Size	Spacing	Grade
US 40 East of Hugo	Four 12-ft lanes & 10-ft shoulder on each side + curb (52ft)	1 to 1½ in.	9.5-in. PCC over 7- to 10-in. HMAC base	No. 5	30	40	No. 5	24	40	No. 6	36	60
I-70 near Exit 289, Gun Club Road (EB)	Two 12-ft lanes + 12-ft inside shoulder + 12-ft outside shoulder (48 ft)	⅜ to ½ in.	11-in. PCC over 6-in. aggregate base	No. 6	30	40	No. 6	34	40	No. 5	36	40
I-70 near Exit 316, Byers (EB)	11.8-ft + 12-ft lanes + 4-ft inside shoulder + 12-ft outside shoulder (39.8 ft)	1 to 1½ in.	10.5-in. PCC over dense graded crushed stone base	No. 6	30	40	No. 6	33	40	No. 5	36	40
I -70 MP 308.2 (EB)	Two 12-ft lanes + 12-ft outside shoulder + 4-ft inside shoulder - placed monolithic (40 ft)	0.3 to 0.5 in.	14-in. PCC over lean concrete base and silty sand base	No. 6	30	40	No. 6	24	40	No. 6	26	60
I-70 MP 316 (EB)	Two 12-ft lanes +4-ft inside shoulder + 12-ft outside shoulder-placed monolithic (40 ft)	0.4 to 2.15 in.	11-in. PCC over silty sand base	No. 6	30	40	No. 6	35	40	No. 5	36	40
I-70 MP 324 (EB)	Two 12-ft lanes + 4-	0.3 to 1.1 in.	11.5-in. PCC over	No. 5	30	40	No. 5	33	40	No. 5	36	60

Table 28. Comparison of tie bar design recommendations using the CDOT, AASHTO 1993, and M-E procedures.

Site Location	No. of Tied Lanes & Concrete Shoulders	Observed Longitudinal Joint Width	Pavement Cross Section	CDOT			AASHTO			ME*		
				Size	Spacing	Grade	Size	Spacing	Grade	Size	Spacing	Grade
	ft inside shoulder + 12-ft outside shoulder -placed monolithic (40 ft)		silty sand base									
I-225 MP 8.5	Six 12-ft lanes NB, including shoulders (72 ft)	0.3 to 0.55	13.5-in. PCC over sand and & gravel base	No. 6	30	40	No. 6	17	40	No. 5	36	40
I-225 MP 10.5	Five 12-ft lanes northbound, including shoulders (60 ft)	0.3 to 1.6	12.5-in. PCC over silty base	No. 6	30	40	No. 6	22.5	40	No. 5	36	40

*The M-E design recommendations correspond to a factor of safety closer to 1.0.
**The AASHTO 1993 method specifies a maximum tie bar spacing of 48 in.

The CDOT tie bar designs generally compared well with the designs using the AASHTO 1993 procedure. Both procedures provided the same tie bar size and steel grade recommendations for all 17 sites. Significant differences in spacing requirements are observed between the CDOT and the AASHTO procedures, particularly for the sites that have multiple tied lanes. For example, Quebec Boulevard, Wildcat Reserve Parkway, University Avenue, and I-225 mile post 8.5 and mile post 10.5 have five or more lanes tied together. The maximum allowable spacing computed using the AASHTO 1993 procedure is 7.5 to 18 in. closer than the CDOT recommended spacing. On the other hand, the CDOT recommended spacing was more conservative than the AASHTO recommendations for cases with two tied lanes.

Significant differences are observed between the design recommendations using the CDOT and M-E design procedures. For sites with more than 24-ft tied together, the M-E procedure's recommended tie bar sizes were one size higher than the CDOT recommended tie bar size, albeit with larger required tie bar spacing (overall however, for these cases, the M-E tie bar procedure required a greater amount of steel). In some cases, a higher steel grade also was recommended because, according to the stress strain calculations of the ISLAB 2005 model, higher bond strength and tie bar pullout resistance are required for the longitudinal joint tie bar system to control the joint performance effectively. The use of No.4 Grade 40 steel may not be adequate to overcome the stresses mobilized in the longitudinal joint tie bar system, particularly for the range of environmentally induced stresses encountered in Colorado. Note that joint failures were observed on several of these sites where the current steel design was lower than the M-E procedure's required steel amount. This is not to say that all the failures are attributable to design alone. As noted in Chapter 3, construction deficiencies were noted for some of the sites that were investigated, namely, I-70 near Exit 289 and I-70 near mile post 308.

The differences between the current CDOT procedure and the M-E procedure are more pronounced for sites with stabilized bases, where the resistance offered by the stiffer base against the tightening of joint opening is higher and, hence, steel with higher pullout resistance is required. Examples include the I-25 site (unbonded concrete base), US 40 east of Hugo (HMAC base), and I-70 mile post 308.2 (lean concrete base). Note that the maximum allowable slip at the joint, bearing stresses, and pullout resistance increase with increasing bar size and better steel grade.

There were sites where the current CDOT steel requirements were greater than those required by the M-E procedure, including I-70 mile post 316 (eastbound), I-70 mile post 324 (eastbound), I-225 mile post 8.5, and I-225 mile post 10.5. In all these cases, thickness of the pavement drove the higher steel requirements for the CDOT procedure, while the M-E procedure is relatively insensitive to this parameter.

EVALUATION OF CDOT'S TIE BAR CONSTRUCTION PRACTICES

The differences in design recommendations between the CDOT and M-E procedures indicate the need for tie bar steel with better properties (higher size and/or grade) to counter the impacts of environmentally induced stresses, multiple lanes tied together and stiffer bases. However, the field investigations revealed that there were significant performance differences (i.e., wider joint openings) between joints with properly and improperly installed tie bars. As discussed in Chapter 3, the wider openings measured at these joints could have been caused by misalignment and misplacement of tie bars during installation. Hence, there was a need to evaluate and identify gaps, if any, in the current CDOT specifications relating to tie bar installation practices and material quality.

Current CDOT Tie Bar Installation Specifications

Section 412.13(a) of the CDOT standard specifications (2005) presents requirements for longitudinal joints and tie bars. This section presents key aspects related to method specifications for installation, sampling requirements, testing, and pass/fail criteria for material quality. In addition, CDOT Standard Plan M-412-1 (2006) specifies the embedment length (30 in.) and placement depth (mid-depth of concrete slab) of tie bars for installation. However, these specifications lack any specific language regarding the installation control and related placement tolerances.

Comparison with Specifications of Other State Agencies

CDOT specifications were compared with those of various other state DOTs, focusing on the following aspects:

- Sampling requirements of tie bars.
- Test equipment and method.
- Tie bar pullout resistance requirement.
- Tie bar spacing and depth of placement.
- Installation tolerances.

Section 412.13(a) of the CDOT standard specifications requires that the contractor test at least 15 of the tie bars before installation. If two or more tie bars fail to meet the passing criteria, then another 15 tie bars should be tested, and if any of the second set of 15 tie bars fails to meet the passing criteria, then all remaining tie bars should be tested.

CDOT's specification is similar to those of the Pennsylvania DOT (2011) and the Illinois DOT (2007), except that Illinois follows a different sampling plan. Illinois DOT requires that 5 percent of the first 500 tie bars and 0.5 percent of the bars installed after the initial 500 should be tested.

The research team does not recommend any changes to CDOT's sampling requirements; however, published literature on the statistical process behind these requirements could not be found. Therefore, it is recommended that CDOT reevaluate the statistical validity and update the current requirements, if necessary, using historical data (for determining expected variance) and approved statistical methods.

CDOT specifications do not mention the test equipment and method to be used for testing tie bars. Pennsylvania DOT (2011) specifies the use of a center-pull hydraulic jack with a load measuring gage and bearing ring capable of testing each tie bar to 12,000 lb or to a 1/32-in. slippage, while Illinois (2007) and Texas (2004) specify that the equipment and method to be used in tie bar testing should be in accordance with the requirements of ASTM E488 (2003).

This ASTM standard provides procedures for determining tensile and shear strengths of post-installed and cast-in-place anchorage systems in structural members made with concrete. ASTM E1512 (2007) provides instructions for testing the adhesive bond developed between a steel reinforcement bar or anchor and the surface of a hole in concrete. This test method also can be used to assess the effects of factors such as moisture, freezing, and thawing on bond performance. The research team suggests that CDOT further evaluate adopting the ASTM static tension method as a protocol for testing tie bars.

CDOT specifications require that the average pullout resistance of tie bars should be at least 11, 250 lb with a slippage of 1/16 in. or less when tested using approved methods. Illinois DOT specifies minimum pullout strength values of 11,000 and 19,750 lb for No. 6 and No. 8 bars, respectively, whereas Pennsylvania DOT specifies the average pullout strength of tie bars should be at least 12,000 lb or a maximum slip of 1/32 in., whichever occurs first. Pennsylvania also recommends minimum pullout resistance based on the tied width of pavement (see Table 29). As the distance from the joint to the nearest free edge is more than 12 ft (i.e., widened slab or multiple slabs tied together), the required pullout resistance for tie bars is increased.

Table 29. Pennsylvania DOT specification for tie bars.

Tied Width of Pavement (Distance from Joint Being Constructed to Nearest Free Edge)	Pullout Resistance (lb/ft)*
< 12 ft	2,200
12-17 ft	3,200
> 17 ft	4,500

*Pennsylvania DOT specifies "pounds per foot" for pullout resistance. To convert this value to pounds, multiply this value by the tie bar spacing using appropriate units.

CDOT tie bar pullout requirements are comparable with those specified by Illinois and Pennsylvania. The adequacy of a passing criterion between 11,000 and 12,000 lb was evaluated using the pullout force at steel yield and the design recommendations. A minimum value of tie bar pullout resistance in this range is expected to be adequate for pavement systems with less stiff bases and fewer tied lanes, while the adequacy of this value remains questionable for stiffer bases and multiple tied lanes. Further studies are needed to establish a passing criterion for a range of scenarios involving various combinations of base types, number of tied lanes, and concrete mix properties. Moreover, the phenomenon of allowable slippage should be taken into account in conjunction with the threshold pullout resistance value.

CDOT Standard Plan M-412-1 specifies a center-to-center tie bar spacing of 30 in. CDOT's spacing requirements were found comparable with other state DOT specifications. Tables 30 and 31 provide the tie bar spacing requirements specified in Wisconsin (2010) and Ohio (2008) standard specifications, respectively.

Table 30. Wisconsin DOT specification for tie bars.

Pavement Depth (in.)	Clear Cover (in.)	Max Tie Bar Spacing (in.)	
		Pavement Width	
		24 or 26 ft	> 30 ft
6, 6 ½	3 ± ½	48	42
7, 7 ½	3 ¼ ± 1	45	36
8, 8 ½	3 ¾ ± 1	39	30
9, 9 ½	4 ¼ ± 1	33	27
10, 10 ½	4 ¾ ± 1	30	24
11, 11 ½	5 ¼ ± 1	27	21
12	5 ¾ ± 1	24	21

Table 31. Ohio DOT specification for tie bars.

Thickness of Pavement (in.)	Transverse Joint Spacing (ft)	Number of Tie Bars per Slab	Max. Spacing between Tie Bars (in.)
10 or less	15	7	26
	21	10	25
> 10	15	9	20
	21	13	20

CDOT Standard Plan M-412-1 also specifies that tie bars are placed at mid-depth of the concrete slab, which is comparable with national practice.

Like most state DOTs, CDOT does not specify any construction-related tolerances for tie bar placement. Such requirements include tolerances for depth of placement, embedment length on both sides, and angular skew in the longitudinal and transverse directions. Based on a limited review of agency specifications, Washington State DOT (2010) was found to specify the following:

> Tie bars shall be placed at the mid depth of the concrete slab, centered over the joint, perpendicular to centerline, and parallel to the Roadway surface.
>
> Placement tolerances for tie bars
>
> - ±1-inch of the middle of the concrete slab depth.
> - ±1-inch of being centered over the joint.
> - ±1-inch from perpendicular to the centerline.
> - ±1-inch from parallel to the Roadway surface.
>
> The horizontal position of tie bars may be adjusted to avoid contact with existing tie bars in the longitudinal joint where panel replacement takes place.

The research team suggests that CDOT include construction tolerances for tie bar placement. The findings of field investigations revealed that the joint openings were wider when tie bar placement was poor. As observed in several Category III joints, the joint openings generally exceeded 1 in. when the tie bars were not embedded properly across the joint.

The findings of field testing further revealed that the joint opening widths were not impacted significantly by the depth of tie bar placement and longitudinal and transverse skew; however, this observation was made based on limited field data. More rigorous field investigations, backed by theoretical analyses and statistical validity, are needed to evaluate the effects of these geometric nonconformities.

DISCUSSION ON CDOT'S TIE BAR PRACTICES

The following observations were made from the evaluation of CDOT's tie bar design and construction practices:

- CDOT tie bar requirements are comparable with the AASHTO 1993 procedure for the scenarios where only two or three lanes tied are together. While the tie bar sizes determined using these methods are of similar size, the AASHTO procedure recommends closer tie bar spacing for multiple tied lanes, and the CDOT required spacing is shorter (conservative) for three or fewer tied lanes.
- The M-E tie bar procedure requires larger tie bars than the CDOT procedure, and a higher steel grade in some cases, while the spacing requirements were slightly more relaxed (i.e., greater spacing in the M-E method than in the CDOT method). The differences between the M-E and CDOT design requirements are more pronounced for pavement systems with multiple lanes tied together or stabilized (asphalt and cement) bases.
- While CDOT's practices on tie bar spacing, placement depth, and sampling were similar to those of other state agencies, CDOT specifications lack sufficient criteria to control tie bar misalignment and misplacement during installation.
- CDOT's minimum pullout criterion for tie bars is comparable to the criteria specified in Pennsylvania and Illinois. However, the adequacy of this criterion to ensure longitudinal joint performance under various scenarios is yet to be established.

There are several questions yet to be answered. The longitudinal joint tie bar system is a complex one, where factors such as concrete mix properties, tie bar properties, slab-base interface friction, climatic variations, and construction practices come into play. The M-E based approach is a step forward from the SDT-based approach. The analytical model used in the proposed M-E approach is based on sound theoretical and engineering principles and provides a reasonably accurate representation of concrete steel interactions in this complex system. However, the parameters used in the analytical model are based on limited experimental studies and statistical validity. It should be cautioned that the model predictions will only be as good as the inputs. For adoption and implementation, the model parameters should be updated with representative Colorado-specific values.

In addition, the M-E model is built on the axial load condition; it does not address the faulting conditions where the joint lacks or loses its shear capacity due to smooth faced joints or physical separation along with very heavy truck traffic loadings, and the tie bars are forced to act as load transfer devices.

Hence, it is suggested that CDOT validate pavement systems which include the tie bar recommendations from this study by undertaking a comprehensive experimental program involving field monitoring of joint movements and load transfer for various combinations of number of tied lanes and stiffer bases, laboratory testing, and additional numerical modeling. Field investigations may involve instrumentation to monitor the movement of joint opening, strain measurement on the axis of tie bars at joint, strain measurement at the top and bottom of the PCC slab, FWD testing for measuring LTE, and distress surveys. Laboratory testing may involve steel pullout tests to measure and validate pullout resistance at yield and corresponding slip. The bonding characteristics of steel (with different sizes, grade, and embedment length) and concrete (with different mix types and strengths) need to be evaluated. These field and laboratory data would be invaluable for refining the theoretical model with additional numerical modeling efforts.

CHAPTER 5. SUMMARY, CONCLUSIONS, AND FUTURE RESEARCH

SUMMARY

An adequate longitudinal joint tie bar system properly constructed is essential in the overall performance of concrete pavement. Excessive longitudinal joint openings are believed to be caused by inadequate tie bar size and spacing and by improper tie bar installation. If designed and installed properly, tie bars prevent the joints from opening and improve LTE between slabs and between slabs and shoulders, resulting in increased load carrying capacity.

This study evaluated the longitudinal joint tie bar system currently used by CDOT, examining the CDOT criteria for proper use of tie bars.

The current CDOT recommendations for tie bar size are based on PCC thickness only. This approach, a simplified variant of the AASHTO 1993 tie bar design procedure, is based on the subgrade drag theory (SDT). This theory has several major deficiencies, as it fails to take into account the effects of actual temperature drop, drying shrinkage, proper slab/base friction, and loading conditions in tie bar design.

An improved tie bar design method is recommended in this study. This procedure, based on fundamental engineering M-E principles, considers critical factors such as the temperature drop from concrete set to minimum monthly and base type. It also considers the maximum number of lanes that can be tied together for various base course materials and climatic conditions in Colorado. Using numerical solutions obtained using ISLAB2005, tie bar design tables with recommended bar size and spacing have been developed for each combination of pavement base types, CDOT concrete mixes, and weather stations.

Field studies were conducted to investigate longitudinal joint performance of concrete pavements in Colorado and further evaluate the impact factors related to design and construction practices. Field testing was carried out in two rounds. The experimental plan for the first round of testing included the evaluation of tie bar alignment (using MIT Scan), measurement of joint load transfer (using the FWD), and measurement of the relative slab movement at the joints. The second round of testing involved only MIT Scan testing.

In addition, CDOT's tie bar design recommendations were compared with the AASHTO 1993 and M-E tie bar design procedures. CDOT's specifications and practices related to longitudinal joint construction and tie bar design and placement were compared with those of other state agencies. The scope of this review included the sampling requirements, equipment, and methods used in tie bar testing, requirements of minimum pullout resistance of tie bars, tie bar size, spacing, placement depth, and alignment tolerances.

CONCLUSIONS

The following conclusions were drawn based on the field investigations and review of CDOT's practices with respect to longitudinal joint tie bar system:

- CDOT's current tie bar design approach inherits the deficiencies of SDT and could possibly be one reason for the existing longitudinal joint problems (excessive opening and loss of LTE) in concrete pavements in Colorado.
- In the first round of field testing, excessive movements were identified in tied joints of all three sites. The measured joint movements were in the typical range for non-tied slabs, implying that some tied joints performed as poorly as non-tied slabs, and thus indicating the possibility of tie bar failure due to loss of concrete-steel bonding or yielding of tie bar steel.
- The information gathered in the first round of testing indicated some possible impact of tie bar misalignment or misplacement on poor longitudinal joint performance.
- In the second round of testing, it was observed that the construction issues relating to the quality of tie bar placement seem to control the performance of longitudinal joints. Joint openings were wider when the tie bars when the embedment lengths were inadequate. Tie bars with adequate embedment length on both sides of the joint, even when misaligned; appear to be in good order.
- The influence of design and construction implications on poor joint performance could not be determined due to limited availability of data. The amount of built-in joint opening

at the time of construction is required to differentiate the impact of design and construction implications.

- CDOT tie bar recommendations are comparable with the AASHTO 1993 procedure for the scenarios where only two or three lanes tied are together. While the tie bar sizes determined using these methods are similar, the AASHTO procedure recommends tighter tie bar spacing for multiple tied lanes, and the CDOT-recommended spacing is conservative for three or fewer tied lanes.
- The M-E tie bar procedure requires larger bars than the CDOT procedure and a higher steel grade in some cases, while the spacing requirements were slightly more relaxed. The differences between the M-E and CDOT design recommendations are more pronounced for pavement systems with stabilized bases.
- While CDOT's practices on tie bar spacing, placement depth and sampling were similar to those of other state agencies, the construction specifications lack sufficient criteria to control tie bar misalignment and misplacement during installation.
- CDOT's minimum pullout criterion for tie bars is comparable to the criteria specified in Pennsylvania and Illinois. However, the adequacy of this criterion to ensure longitudinal joint performance under various scenarios is yet to be established.

RECOMMENDATIONS

A series of supplemental look-up tables, specific for Colorado conditions, were developed using the proposed M-E tie bar design approach to help designers to determine the tie bar spacing and size for combinations of base material types, concrete mix types, and number of tied lanes.

One of the major limitations of this study was the lack of more current experimental data to establish the model input parameters in the analytical model. Further research is necessary for establishing the following model parameters:

- A systematic investigation of the bond behavior of tie bars and paving concrete. The impact of tie bar pullout stiffness on the concrete pavement longitudinal joint design developed in this study is significant, especially in the case of pavements on stabilized bases. Guidance from the Euro-International Concrete Committee was adopted in this research, since it was the best available and most practical source of information. This recommended investigation requires obtaining pullout force-slip curves for various tie bar sizes, embedment lengths, and typical Colorado concrete mixes.

- Slab/supporting layer friction characteristics for various base types in accordance with modern friction theories. The last published data on this topic are from the mid-1980s.

Another impediment to this research was the lack of sufficient field data, such as the initial joint opening at the time of concrete set, to further investigate the impacts of tie bar design and placement practices on longitudinal joint performance. As conducted in the LTPP studies, it is recommended that CDOT undertake a comprehensive field program to further validate and evaluate the longitudinal joint behavior in concrete pavement systems that incorporate the tie bar recommendations from this research through monitoring of joint opening and temperature variations.

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APPENDIX A: TIE BAR DESIGN GUIDANCE

INTRODUCTION

This appendix presents a step-by-step approach for identifying the appropriate tie bar size, spacing, and length using the M-E tie bar design method. A tie bar design tool available on the ACPA website (http://apps.acpa.org/apps/METiebar.aspx) also utilizes the calculations illustrated here.

The example presented herein illustrates the steps involved in the computation of equivalent free strains in concrete. These strains represent the cyclical effect of temperature changes and drying shrinkage on unrestrained concrete. Upon computing the free strains, the practitioner should use the design tables presented at the end of this appendix to determine the recommended tie bar configuration. These design tables were prepared for two, three, and four tied standard width (12-ft) lanes and a widened (14-ft) lane placed on six different base types (PCTB, CTB, soil cement, LCB, ATB, and unbound bases).

STEPS INVOLVED IN TIE BAR DESIGN

Step 1. Obtain Design Inputs

- Location: Aspen, CO.
- Geometry: Two traffic lanes, 12-ft wide and 14-ft wide slabs, 10-in. JPCP atop 6-in. unbound base.
- Concrete Mix: CDOT Optimized.
- Concrete CTE (α): $5.7*10^{-6}$/°F.
- Concrete slab construction month: July.
- Cementitious materials content: 563 lb/yd^3.
- Cement type: Type I.
- Curing procedure: Application of curing compound.

Step 2. Estimate Design Temperature Drop and Thermal Strain

1. Mean ambient temperature in July: 64.5 °F (Table 7).
2. Mean minimum monthly temperature (January), T_{min}: 22.2 °F (Table 7).
3. Calculate the concrete set temperature, $T_{constr.}$ The concrete set temperature can be estimated from the expected construction monthly ambient temperature and cementitious materials content using the following equation:

$$T_{constr} = (CC\left[-0.015917 + \left(0.001416 + \frac{1}{CC}\right) * MMT - 0.000006 * MMT^2\right] \tag{A-1}$$

Where

CC = cementitious materials content, lb/yd^3

MMT = mean ambient monthly temperature for the month of construction, °F

Concrete set temperature, T_{constr} = 92.9 °F

4. Total temperature drop: 92.9 °F – 22.2 °F = 70.7 °F.
5. Calculate thermal strain in unrestrained concrete due to a uniform temperature drop. The concrete thermal strain, $\varepsilon_{thermal}$, due to a uniform temperature drop is computed as:
   - $\varepsilon_{thermal} = \alpha * (T_{constr} - T_{\min})$ (A-2)
   - $\varepsilon_{thermal} = (5.7 * 10^{-6}) * (70.7) = 403.02 * 10^{-6}$ or 403 microstrain.

Step 3. Compute Drying Shrinkage Strain

1. Calculate ultimate shrinkage strain, ε_{su}:
   - Typical shrinkage strain, recommended value 650 microstrain.
   - C_1 = Cement type factor = 1.0 for Type I cement.
   - C_2 = Type of curing factor = 1.2 because cured by curing compound.
   - Ultimate shrinkage:
     - $\varepsilon_{su} = C_1 \cdot C_2 \cdot \varepsilon_{ts}$ (A-3)
     - $\varepsilon_{su} = 650 * 1 * 1.2 = 780$ microstrain.
2. Calculate drying shrinkage strain at the bottom of the slab, $\varepsilon_{sh,b}$:
   - Ultimate shrinkage strain, ε_{su} = 780 microstrain.
   - Recommended time for shrinkage calculation: 365 days.

- Recommended value for the time to develop 50 percent of the ultimate shrinkage: 35 days.
    - $$S_t = \frac{Age}{n + Age} \qquad \text{(A-4)}$$
    - S_t = 365/(35+365) = 0.9125
- Minimum relative humidity factor for RH of 90 percent:
    - $$S_{mean} = \begin{cases} 3 - 0.03 \cdot RH_i \;\; if \;\; RH_i > 80\% \\ 1.4 - 0.01 \cdot RH_i \;\; if \;\; 30 < RH_i < 80\% \\ 1.1 \;\; if \;\; RH_i \leq 30\% \end{cases} \qquad \text{(A-5)}$$
    - S_{mean} = 0.3
- Shrinkage strain at the bottom of the slab:
    - $$\varepsilon_{sh,b} = \varepsilon_{su} S_t S_{mean} \qquad \text{(A-6)}$$
    - $\varepsilon_{sh,b}$ = 780 * 0.9125 * 0.3 = 214 microstrain.

3. Calculate shrinkage strain at the top of the slab, $\varepsilon_{sh,t}$:

- Ultimate shrinkage, ε_{su} = 780 microstrain.
- Mean RH in Aspen: 56.9 percent (Table 7).
- Mean relative humidity factor for RH of 56.9 percent.
    - S_{mean}= 0.83 (from equation A-5).
- Shrinkage strain at the top of the slab:
    - $$\varepsilon_{sh,t} = \varepsilon_{su} \cdot S_t \cdot S_{mean} \qquad \text{(A-7)}$$
    - $\varepsilon_{sh,t}$ = 780 * 0.9125 * 0.83 = 591.5 microstrain .

4. Mean drying shrinkage strain, $\varepsilon_{sh,\,m}$, through the concrete slab:

- Shrinkage strain at the bottom of the slab, $\varepsilon_{sh,b}$ = 214 microstrain.
- Shrinkage strain at the top of the slab, $\varepsilon_{sh,t}$ = 591.5 microstrain.
- Thickness of the shrinkage zone (e.g., the driest portion of the slab, near the surface), h_d = 2 in (assumed)
- Thickness of PCC layer H_{PCC} = 12

- Mean drying shrinkage strain
  - $$\varepsilon_{sh,m} = \varepsilon_{sh,b} + \left(\varepsilon_{sh,t} - \varepsilon_{sh,b}\right) \cdot \frac{h_d}{H_{PCC}} \quad \text{(A-8)}$$
  - $\varepsilon_{sh,\,m}$ = 214 + (591.5-214)*2/12 = 289 microstrain.

Step 4. Compute Total Equivalent Free Strain in Concrete

The total equivalent micro units of free strain, ε_{eq}:

- $$\varepsilon_{Eq} = \varepsilon_{thermal} + \varepsilon_{sh,m} \quad \text{(A-9)}$$
- ε_{eq} = 403 + 289 = 692 microstrain.

Step 5. Determine Tie Bar Design from Standard Tables

Upon computing the total equivalent free strains, the user should identify the appropriate design tables (presented at the end of this appendix as Tables A-1 through A-38) for the given traffic lane width, number of tied lanes, and base types. These design tables, prepared based on ISLAB2005 runs, present recommended tie bar configurations based on:

- Number of tied lanes.
- Base types and thickness.
- Lane/slab width (standard or widened slab).

For the inputs used in this example (6-in. unbound base, 12-ft lane tied to 14-ft lane), the recommendations presented in Table A-2 are used. For the estimated free strain of 692 microstrain (see row for 700 microstrains), the following tie bar configuration is required:

- #4 tie bar, Grade 60 steel OR #5 tie bar, Grade 40 steel.
- 45-in. spacing.
- 24-in. total tie bar length (12-in. embedment length).

This design will ensure that the yield stress is below the yield strength of the steel to ensure the long-term integrity of the longitudinal joint.

Table A-1. Tie bar design for two tied 12-ft lanes on a 6-in. unbound base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#4	45	24	40
550	#4/#5	45	24	60/40
600	#4/#5	45	24	60/40
650	#4/#5	45	24	60/40
700	#4/#5	45	24	60/40
750	#4/#5	45	24	60/40
800	#4/#5	45	24	60/40

Table A-2. Tie bar design for tied 12-ft width lanes and 14-ft width lane on 6-in. unbound base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#4/#5	45	24	60/40
550	#4/#5	45	24	60/40
600	#4/#5	45	24	60/40
650	#4/#5	45	24	60/40
700	#4/#5	45	24	60/40
750	#4/#5	45	24	60/40
800	#4/#5	45	24	60/40

Table A-3. Tie bar design for two tied 14-ft lanes on a 6-in. unbound base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#4/#5	45	24	60/40
550	#4/#5	45	24	60/40
600	#4/#5	45	24	60/40
650	#4/#5	45	24	60/40
700	#4/#5	45	24	60/40
750	#4/#5	45	24	60/40
800	#4/#5	45	24	60/40

Table A-4. Tie bar design for three tied 12-ft lanes on a 6-in. unbound base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#4/#5	45	24	60/40
550	#4/#5	45	24	60/40
600	#4/#5	45	24	60/40
650	#4/#5	45	24	60/40
700	#4/#5	45	24	60/40
750	#4/#5	45	24	60/40
800	#4/#5	45	24	60/40

Table A-5. Tie bar design for more than three tied 12-ft lanes on a 6-in. unbound base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#4/#5	45	24	60/40
550	#4/#5	45	24	60/40
600	#4	45	24	60
650	#5	36/45	24	40/60
700	#5	36/45	24	40/60
750	#5	36/45	24	40/60
800	#5	45	24	60

Table A-6. Tie bar design for two tied 12-ft lanes on a 6-in. soil cement base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5	36	24	60
750	#5/#6	30/36	24	60
800	#5/#6	30/36	24	60

Table A-7. Tie bar design for two tied 12-ft lanes on a 5-in. soil cement base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5	36	24	60
750	#5	36	24	60
800	#5	36	24	60

Table A-8. Tie bar design for two tied 12-ft lanes on a 4-in. soil cement base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5	36	24	60
750	#5	36	24	60
800	#5	36	24	60

Table A-9. Tie bar design for tied 12-ft wide lanes and 14-ft wide lane on 4-in. soil cement base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5	36	24	60
750	#5/#6	30/36	24	60
800	#6	36	24	60

Table A-10. Tie bar design for two tied 14-ft lanes on a 6-in. soil cement base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5/#6	30/36	24	60
750	#5/#6	30/36	24	60
800	#6	36	24	60

Table A-11. Tie bar design for three tied 12-ft lanes on a 6-in. soil cement base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5/#6	30/36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-12. Tie bar design for more than three tied 12-ft lanes on a 6-in. soil cement base.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5/#6	30/36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-13. Tie bar design for two tied 12-ft lanes on a 6-in. PCTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5/#6	30/36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-14. Tie bar design for two tied 12-ft lanes on a 5-in. PCTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5	36	24	60
750	#5/#6	30/36	24	60
800	#6	36	24	60

Table A-15. Tie bar design for two tied 12-ft lanes on a 4-in. PCTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5	36	24	60
700	#5	36	24	60
750	#5	36	24	60
800	#6	36	24	60

Table A-16. Tie bar design for tied 12-ft wide lanes and 14-ft wide on a 6-in. PCTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5/#6	30/36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-17. Tie bar design for two tied 14-ft lanes on a 6-in. PCTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5/#6	30/36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-18. Tie bar design for three tied 12-ft lanes on a 6-in. PCTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5/#6	30/36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-19. Tie bar design for more than three tied 12-ft lanes on a 6-in. PCTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5/#6	30/36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-20. Tie bar design for two tied 12-ft lanes on a 6-in. CTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-21. Tie bar design for two tied 12-ft lanes on a 5-in. CTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5/#6	30/36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-22. Tie bar design for two tied 12-ft lanes on a 4-in. CTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#5/#6	30/36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-23. Tie bar design for tied 12-ft wide lanes and 14-ft wide lane on a 6-in. CTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5/#6	30/36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-24. Tie bar design for two tied 14-ft lanes on a 6-in. CTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-25. Tie bar design for three tied 12-ft lanes on a 6-in. CTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-26. Tie bar design for more than three tied 12-ft lanes on a 6-in. CTB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#6	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	30	24	60
800	#6	22.5	24	60

Table A-27. Tie bar design for two tied 12-ft lanes on a 6-in. LCB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5/#6	30/36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-28. Tie bar design for two tied 12-ft lanes on a 5-in. LCB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5/#6	30/36	24	60
600	#5/#6	30/36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-29. Tie bar design for two tied 12-ft lanes on 4-in. LCB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5/#6	30/36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-30. Tie bar design for tied 12 feet wide lanes and 14 feet width lane on 6-in. LCB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#6	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-31. Tie bar design for two tied 14-ft lanes on a 6-in. LCB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#6	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-32. Tie bar design for three tied 12-ft lanes on a 6-in. LCB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#6	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	30	24	60
750	#6	26	24	60
800	#6	22.5	24	60

Table A-33. Tie bar design for more than three tied 12-ft lanes on a 6-in. LCB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#6	36	24	60
550	#6	36	24	60
600	#6	22.5	24	60

Table A-34. Tie bar design for two tied 12-ft lanes on a 6-in. ATB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5	36	24	60
600	#5	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-35. Tie bar design for tied 12-ft width lanes and 14-ft width lane on 6-in. ATB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#5/#6	30/36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-36. Tie bar design for two tied 14-ft lanes on a 6-in. ATB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-37. Tie bar design for three tied 12-ft lanes on a 6-in. ATB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#5	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	36	24	60
800	#6	36	24	60

Table A-38. Tie bar design for more than three tied 12-ft lanes on a 6-in. ATB.

Total Equivalent Free Strain, Microstrain	Tie Bar Size Designation	Tie Bar Space, in.	Tie Bar Length, in.	Steel Grade
500	#6	36	24	60
550	#6	36	24	60
600	#6	36	24	60
650	#6	36	24	60
700	#6	36	24	60
750	#6	30	24	60
800	#6	22.5	24	60