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Rational Use of Terminal Anchorages in Portland Cement Concrete Pavements

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16. Abstract
It has long been stated that Portland cement concrete (PCC) pavements can grow and push bridges, resulting in damage to bridge structures. To protect bridge structures from damages due to the expansion of PCC pavements, three terminal systems are currently used in Texas: anchor lug (AL), wide-flange (WF), and expansion joint (EJ) systems. However, the effectiveness of these three systems has not been fully evaluated. This study investigated the parameters affecting the movements of continuously reinforced concrete pavement (CRCP) due to temperature variations near bridge terminal areas, whether thermal expansion of CRCP causes damage to bridge structures, and if it does, which terminal type is the most cost-effective. Field evaluations revealed that subbase friction plays an important role, and the movement of CRCP due to temperature variations was not excessive if the subbase friction is adequate and may not cause damage to the bridge structures. Most of the distresses near the bridge terminal areas were due to volume changes or instability in the embankment materials. The end movement of CRCP could be accommodated by a simple EJ system if there is adequate subbase friction. The benefits of WF and AL systems are doubtful considering their higher initial construction costs compared with that of a simple EJ system. On the other hand, it should be noted that in a few CRCP projects, observations were made of CRCP expanding beyond the thermal expansion limits, implying that there are other expansion mechanisms than thermal expansions. The investigation of CRCP expansions due to factors other than thermal volume changes was out of the scope of this project. Simple structural analysis showed that if CRCP expands beyond thermal expansion limits, it is practically impossible to restrain the slab expansions with known methods including the AL system.

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Chapter 1: Introduction

1.1 Background

Portland cement concrete (PCC), like any other material, undergoes volume changes when subjected to temperature variations. When the temperature of PCC is above the setting temperature, the concrete volume could expand beyond the volume at concrete setting. These thermal volume increases in PCC pavements, if not controlled or properly accommodated, have been cited as the cause of damage to bridge structures, including bridge approach slabs. At the same time, it is well known that drying shrinkage occurs in PCC, effectively reducing the volume of concrete in the long run. In order for the expansion of PCC to cause damage to bridge structures, slab expansions due to thermal volume increase of PCC should be greater than the sum of the slab contractions due to drying shrinkage of concrete and expansion joint width. From a theoretical standpoint, this mathematical relationship doesn't completely explain how thermal expansion of concrete could be so excessive as to cause damage to bridge structures. Other known mechanisms for concrete volume increase are alkali-silica reaction (ASR), delayed ettringite formation (DEF), and swelling due to the increase in moisture content. DEF requires a much higher curing temperature than the typical curing temperature in PCC paving; therefore, it is believed DEF is not a factor leading to excessive volume changes that cause damage to bridge structures. On the other hand, numerous field observations of distresses in approach slabs made in this study indicate that most of the distresses are due to volume changes or continued consolidation of embankment materials under the approach slabs. There is little field evidence that the expansion of PCC pavement due to temperature variations pushes the bridge structures and causes distresses. However, the Texas Department of Transportation (TxDOT) and many other state highway agencies (SHAs) still address the distresses in the terminal areas with the assumption that PCC pavement could expand and push the bridge structures, causing damage.

Terminal systems are intended to alleviate or prevent damage in bridge structures resulting from PCC pavement expansions. Currently, three terminal systems are used in Texas: anchor lug

(AL), wide-flange (WF), and expansion joint (EJ) systems (TxDOT, 2011). There are two different design philosophies utilized in the three systems. One design philosophy is that the slab movements at the end of pavements abutting bridge approach are not excessive and can be accommodated by providing adequate space between the end of the pavement slab and bridge approach slabs. EJ and WF systems are based on this philosophy. The other philosophy is that the slab movements could be excessive, and should be restrained by providing anchor lugs. The AL system is based on this philosophy. These two philosophies are not compatible and they are mutually exclusive. If slab movements at the end of the pavement are excessive and could cause damage to bridge structures, the philosophy used in EJ and WF systems is not correct and severe damage could occur in the bridges with those systems. On the other hand, if the slab movements at the end of the pavement are not excessive, the use of EJ and WF systems is more appropriate, and the use of the AL system will be a poor engineering practice, since the AL system is much more expensive than the other two systems. To resolve this incompatibility of the design philosophies made in the three systems in use at TxDOT, it is essential to understand the short-term and long-term slab movement behavior due to temperature variations.

1.2 Objectives

The primary objective of this study was to evaluate the effectiveness of the three terminal systems currently used in TxDOT: anchor lug (AL), expansion joint (EJ), and wide flange (WF.) Identifying and evaluating the most cost-effective and efficient system was the main objective of this study. The secondary objective was to develop guidelines for the rational use of terminal systems.

The findings from this study, which include fundamental information on the detailed slab movement behavior, both in the short-term and long-term, will provide the valuable information needed to develop more efficient and cost-effective terminal systems, which will result in savings not only in construction costs, but maintenance costs as well.

1.3 Limitations

Due to the complex nature of the mechanisms and factors related to this issue, not all the factors were included in this study and the resulting limitations are as follows:

1) All the test sections selected for field testing in this study had only one subbase type – asphalt stabilized base. The findings in this study cannot be applicable to CRCP system with different subbase types, such as granular, cement stabilized, or lean concrete bases.

2) The duration of slab movement observations was limited to less than three years, with most sections within 20 months. To investigate the slab movements of CRCP more comprehensively, observation periods should be extended.

3) All the sections included in this investigation were in rather flat terrain and the effect of longitudinal slope of CRCP on slab movements was not investigated.

4) Slab movement behavior of CRCP is quite complicated because of the numerous factors involved – subbase friction, creep of concrete, and transverse cracking. No attempts were made to quantify the relationships between these factors and slab movements.

5) There could be several mechanisms for the expansion of CRCP. In this study, the CRCP expansions due to temperature variations only were investigated. Other causes were not studied.

1.4 Report Organization

This research focuses on evaluating the effectiveness of terminal systems and developing guidelines for the rational use of terminal systems. This research report consists of six chapters and each chapter is summarized as follows:

Chapter 2 presents the overview of distresses at the bridge abutment areas surveyed in Texas and the potential causes are discussed.

Chapter 3 provides literature reviews with field observations. The factors related to terminal movements are described and the design standard of each terminal system is shown. Additionally, the field observations made in this research project are briefly discussed.

Chapter 4 depicts the field experimentations made in the project. The information on testing sites and how the end movement of CRCP was measured are included.

Chapter 5 presents the results of collected data in the field experimentation and analysis of data collected.

Chapter 6 shows the summary and recommendations from this research.

Chapter 2: Overview of Distresses at Bridge Abutment Areas and Potential Causes

It has been stated that the expansions in Portland cement concrete (PCC) pavement cause damage to bridge structures, mostly to the abutments and the areas near the abutments. Distresses caused by this damage quite often result in so-called "bumps in the bridge" and require expensive repairs. To alleviate the distresses, various methods have been used. A survey of bridge and pavement engineers at a number of state highway agencies (SHAs) indicates various opinions regarding the cause of the distresses. Some engineers believe the expansions of PCC pavement cause the distresses, while others believe the reverse is true – expansion and contraction of bridge beams and slabs cause damage to the pavement structure.

As will be discussed in the next chapter, the research effort to address this issue has been quite limited considering the significance of the problem. In this chapter, the problems observed in Texas are surveyed.

2.1 Bridges on US 69 in the Beaumont District

The bridges on US69 in the Beaumont District showed some of the most severe damage observed by the research team. Brian Merrill, P.E., of the Bridge Division informed the research team of the expansion of the pavement and resulting damage to the bridge structures. Figure 2.1 illustrates the damage in the abutment due to the approach slab pushing the armor joint. Figure 2.2 shows that the approach slab pushed the armor joint seven inches.

Figure 2.1 Damage in the abutment

Figure 2.2 Approach slab pushing bridge seven inches

This bridge is over Rolfe Christopher Drive, and the damage shown is in the northbound lane at the north end of the bridge. The bridge and the pavement were completed in May, 1971. The pavement is continuously reinforced concrete pavement (CRCP) with an eight-inch thick slab on six inches of cement stabilized sand shell and six inches of lime-treated subgrade.

The length between the vertical point of intersection of this bridge and that of the next bridge to the north, which is over Highland Avenue, is 4,130 ft. The net length of the pavement, including the bridge approach slabs from the abutment of this bridge to the abutment of the bridge over Highland Avenue is 3,600 ft. The elevation of this bridge is 48.80 ft. The elevation of the bridge over Highland Avenue is 35.58 ft. The elevation of the pavement section between the two bridges is about 20 ft. This bridge is about 29 ft above the pavement, with the slope at the embankment area at 4.0 percent, while the Highland Avenue bridge is about 16 ft higher than the pavement, with the slope at the embankment area at 3.5 percent.

For the terminal system in this project, three anchor lugs were installed at each end of the pavement. Figures 2.1 and 2.2 indicate that the anchor lug system was not able to restrain the slab movement. If it is assumed that the pavement pushed the approach slab by seven inches, the lugs were pushed by that amount.

At this point, it is not known what might have caused the approach slab to be pushed by seven inches. The maximum theoretical pavement expansion due to temperature increase was calculated with the following assumptions:

1) There was no friction between CRCP slabs and base.
2) The concrete setting temperature was 50 °F.
3) The coefficient of thermal expansion of the concrete is 6 microstrain/ °F.

These assumptions are quite conservative. As will be discussed in Chapter 5, substantial friction between pavement slab and base fully restrains the concrete slab movement except for the slab near the free end of the pavement. With the above assumptions, the maximum hypothetical slab expansion would be 9.1 inches when the concrete temperature is 120 °F. On the other hand, based on the information obtained in this study, approximately 200 ft of the CRCP slab contributes to the slab expansion, therefore the maximum slab expansion would be 1.0 in when the concrete temperature is 120 °F. There is a large difference in the maximum hypothetical slab expansion and a more realistic slab expansion. It is not feasible, at least at this point, to accurately identify the mechanism of the seven-inch push by the approach slab. The District plans to let the project in the near future. The research team plans to gather as much information as possible, including the evaluation of the condition of anchor lugs and the collection of concrete pieces for further evaluation.

During the field visit to this site in September 2010, it was observed that there were slope failures near the bridge approach slabs, as shown in Figure 2.3. The slope failure was repaired in April 2011, as shown in Figure 2.4, and embankment materials were collected from various locations. The embankment materials were high PI clay, with PI values over 50. It is not known at this point whether the volume changes in the embankment and/or natural soil under the pavement contributed to the approach slab being pushed by seven inches.

Figure 2.3 Slope failure in the embankment area

Figure 2.4 High PI clay in the embankment

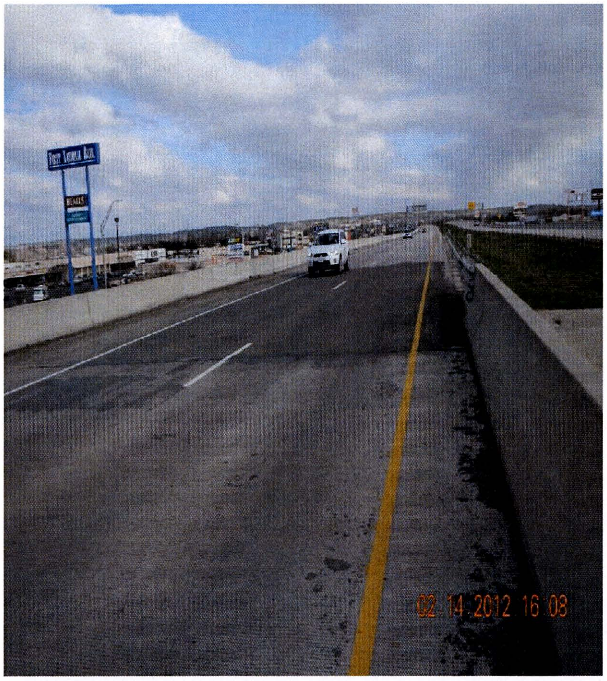
2.2 Bridge on SH 71 in Bastrop in the Austin District

SH 71 passing downtown Bastrop was built within the last 10 years. The pavement is hot mix asphalt pavement and this section has several bridges. It was observed that the bridge armor joint was closing up, and the bridge abutment was moving. The Area Office placed cold mix patching materials to even out the pavement surface near the bridges, as shown in Figure 2.5. Figure 2.6 shows consolidation of the embankment materials. Figure 2.7 is the other side of the rail shown in Figure 2.6. The settlement of the embankment materials is quite substantial. Figure 2.8 illustrates the damage in the traffic rail due to the relative movement between retaining wall and the traffic rail. This damage cannot be due to the expansion of the hot mix asphalt pavement, since the asphalt concrete has quite low stiffness, compared with that of concrete.

It is interesting to note that the distresses that needed repairs with cold mix asphalt and the damage in the traffic rail occurred where the pavement abutting the bridge was flexible pavement. If PCC pavement had been placed in this project, it is quite possible that there would be damage in the bridge approach slabs in the form of cracks. The distresses in this bridge illustrate that most, if not all, of the damage in the bridge structures including bridge approach slabs could be due to the instability or volume changes in the embankment materials, not necessarily the PCC pavement pushing the bridge approach slabs.

2.3 Bridge on IH 10 in Schulenburg in the Yoakum District

A bridge on IH 10 between mile post 676 and 677, three miles east of Schulenburg experienced distresses in the form of misaligned traffic rails, as shown in Figure 2.9. The pavement is AC overlay on CRCP. The terminal system used in this bridge is not known. Field evaluations indicated no damages to the bridge approach slabs or any other signs of distress in the bridge structures, except for the damages in the rail due to the relative movements between rails, as shown in Figure 2.10. It does not appear that the pavement was pushing the bridge structures, since distresses to this extent would have been observed in the bridge approach slabs.



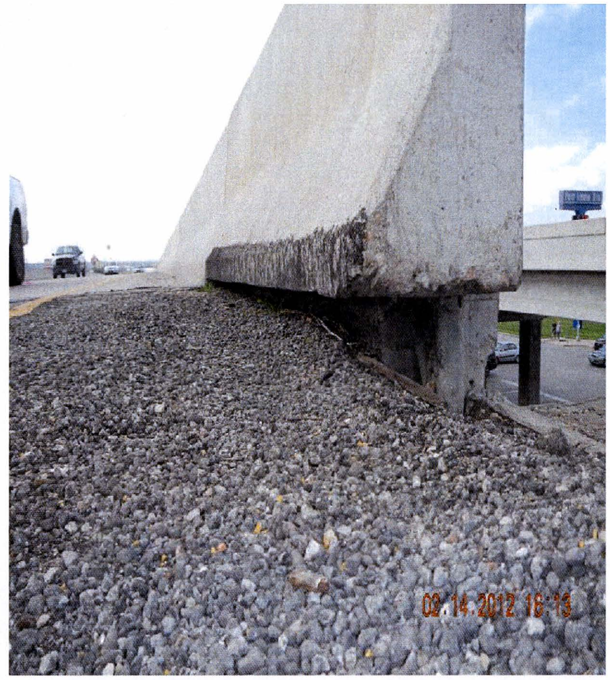


Figure 2.5 Repair of pavement near bridge

Figure 2.6 Subsidence of embankment material (1)

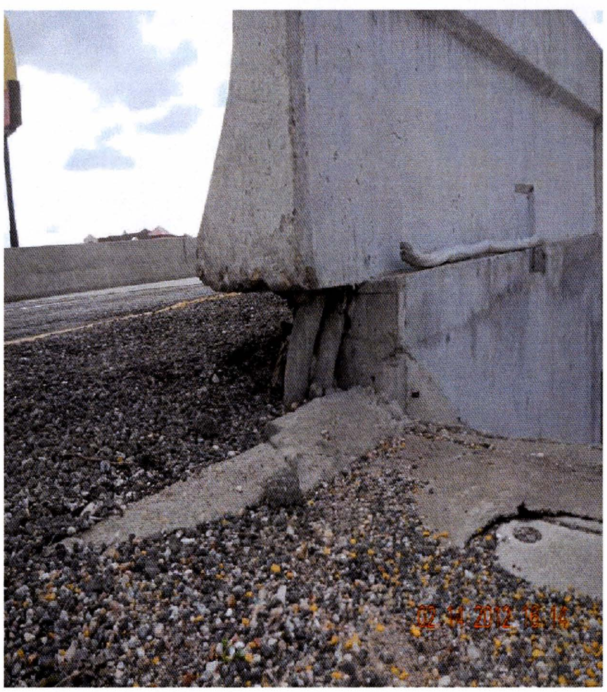


Figure 2.7 Subsidence of embankment material (2)

Figure 2.8 Damage in traffic rail

Figure 2.9 Misalignment of the traffic rails

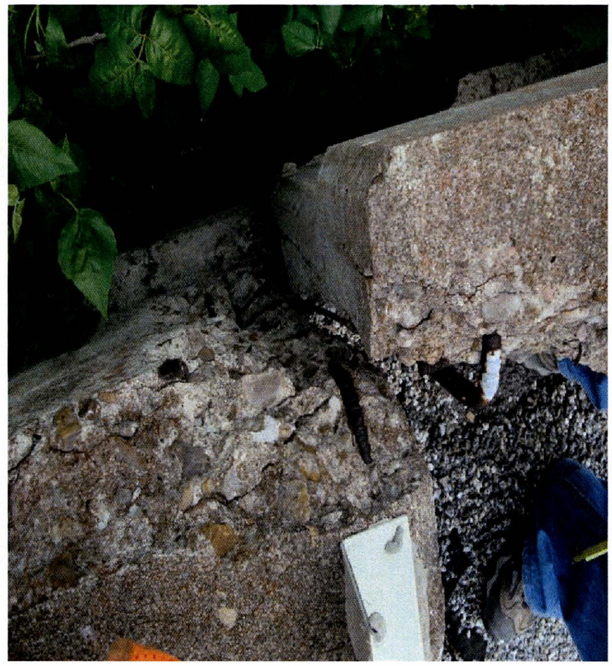


Figure 2.10 Damages in the traffic rails

Figure 2.11 indicates the relative displacement between girders and a bent cap. It appears that there were dowels or tie bars that caused the beams and bent cap to behave as one unit. The relative displacements resulted in spalling of the concrete in the bent cap. It is not known whether the beams or the bent cap initiated the relative movement.

Figure 2.11 Damage in bent cap

Figure 2.12 Variations in armor joint width

2.4 Bridge on IH 20 in the Fort Worth District

Figure 2.12 shows a variation in the widths of an armor joint in a bridge on IH 20 near IH 820 West. The terminal type was an xpansion joint system, and the pavement is CRCP with a 10 in thick slab. The large variation in the armor joint is indicative of either rotation of the abutment or pavement slab sliding. Figure 2.13 shows the misalignment of the traffic rail at the side of the roadway where the armor joint is closed. Figure 2.14 indicates that (1) the armor joint is completely closed up, (2) the approach slab slid toward the right side with respect to the bridge deck, and (3) there was a separation between the bridge approach slab and the traffic rail. Based on these observations, it is likely that the rotation of the abutment is a plausible cause for the variation in the joint width.

Figure 2.13 Misalignment of traffic rail (1)

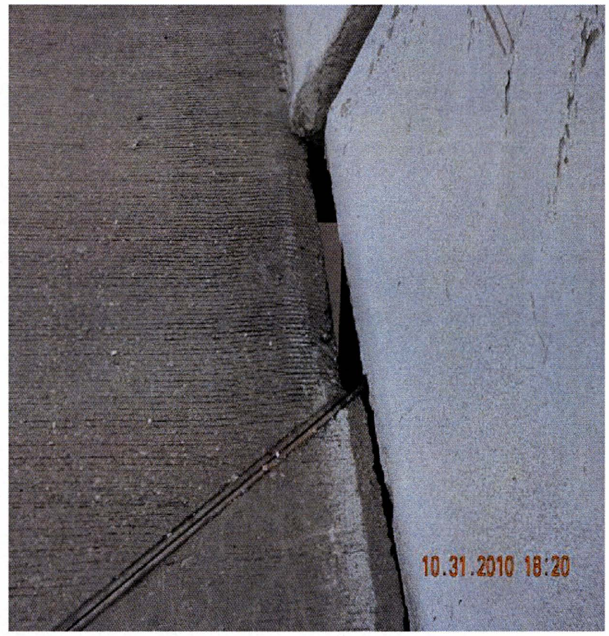


Figure 2.14 Misalignment of traffic rail (2)

Figure 2.15 illustrates the relative displacements between the approach slabs and traffic rail. This picture shows the other side of the bridge shown in Figure 2.13. It's interesting to note that the damage in the rail is limited to the bridge approach slab portion only. In other words, the rail and the bridge deck behaved as a single unit, while the rail and the approach slab did not. Figure 2.16 illustrates damage in the bridge superstructure. The abutment could have rotated, or the pavement could have pushed the approach slab. At the time of this visit, the expansion joint between the pavement and the approach slab was not closed up, even though it should be recognized that this evaluation was done in October, when the air temperature was not high.

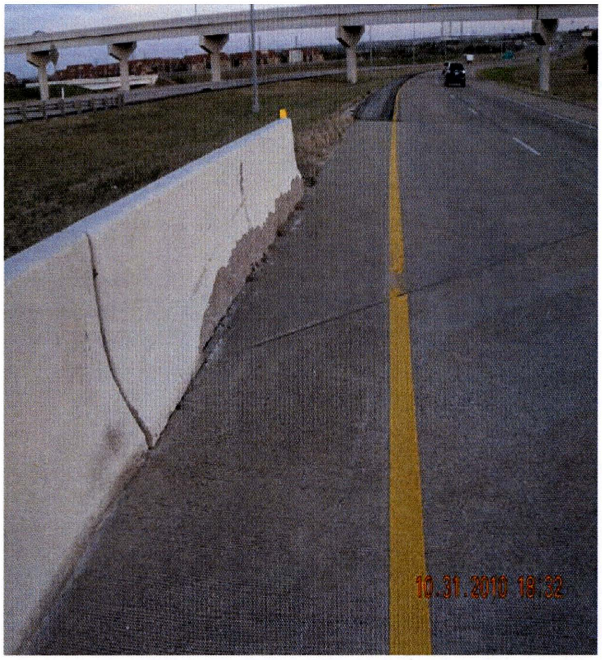


Figure 2.15 Damage in traffic rail

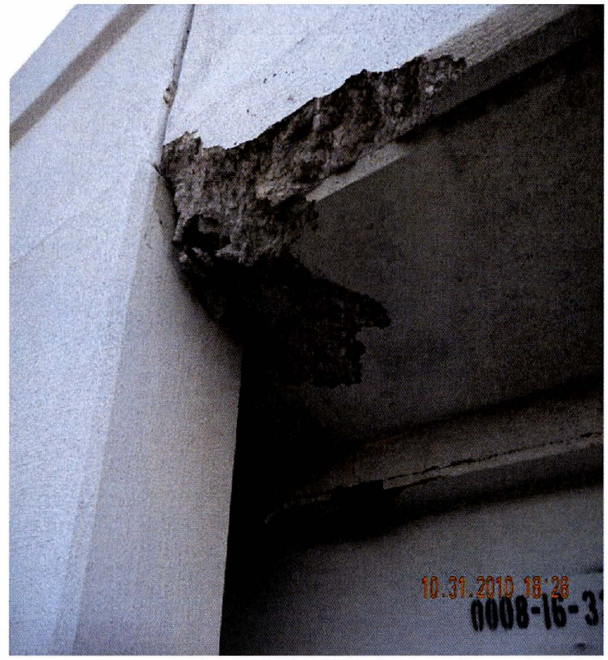


Figure 2.16 Damage in the bridge superstructure

2.5 Bridge on SH 31 in the Dallas District

A section on SH 31 near Trinidad is a plain jointed concrete pavement (CPCD) with 15-ft joint spacing. The section is in a good condition. The detailed information on this pavement section, such as pavement structure and what terminal type used, is not known. The Dallas District has traditionally used a simple expansion joint system, and since the pavement is CPCD, it is quite possible that an expansion joint terminal system was used. It is generally assumed among pavement engineers that the probability of CPCD pushing bridge structures is lower compared to CRCP, even though the logic behind this assumption has never been validated.

Figure 2.17 shows damage in the abutment, and Figure 2.18 illustrates the condition of armor joint and distresses in the bridge approach slab. Both distresses are quite common in bridge structures in Texas. The transverse cracks shown in Figure 2.18 are a good indicator of voids under the bridge approach slab behind the abutment. The portion of the concrete between armor joint and transverse cracks in Figure 2.18 nearly coincides with the width of the abutment.

The fact that there are damages in the bridge approach slab and abutment in a bridge where CPCD abuts the bridge indicate that these damages are due to the volume changes and/or consolidation of embankment materials.

Figure 2.17 Damages in abutment

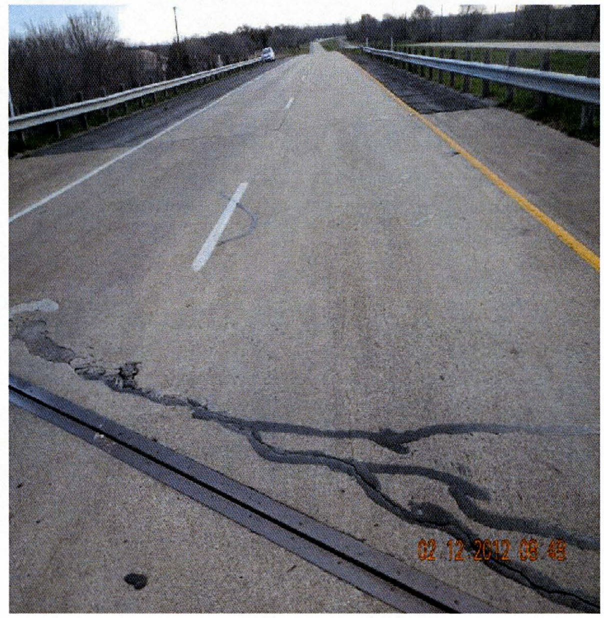


Figure 2.18 Transverse cracks near armor joint

2.6 Bridge on IH 35W in the Fort Worth District

During a widening project on IH 35W in the Fort Worth District , the side of the roadway, including bridge embankments, was cut and excavated. This provided a good opportunity to observe the condition of the embankment materials. The bridge was over Altamesa Blvd, between mile post 44 and 45. The pavement was 13 inches CRCP with an expansion joint terminal system. Figure 2.19 shows the side of the bridge embankment, approach slab and abutment.

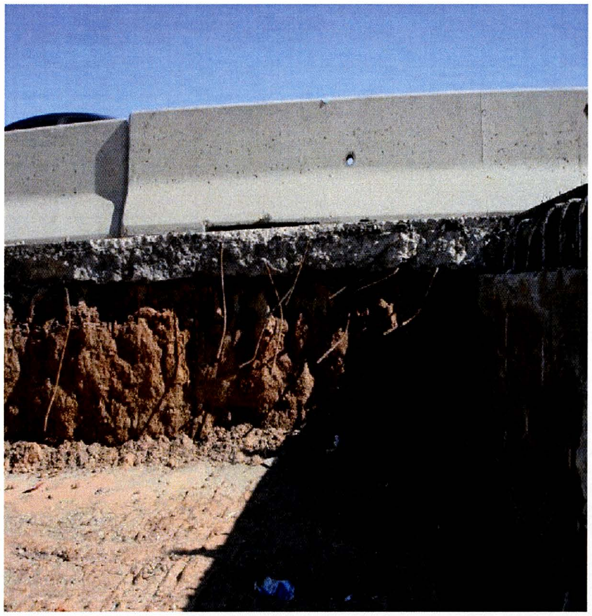


Figure 2.19 Bridge embankment and abutment

Figure 2.20 Void under bridge approach slab

Figure 2.20 shows a void of more than one inch under the bridge approach slab near the abutment, apparently due to the continued consolidation of the embankment materials. Figure 2.21 shows the undersealing material used in an effort to raise the bridge approach slab. Undersealing of the bridge approach slabs is quite frequent in Texas, indicating that there is an issue of continued consolidation of the embankment materials and voids under bridge approach slabs. Figure 2.22 illustrates transverse cracks in the bridge approach slab at the armor joint, which were apparently caused by the voids underneath. These cracks look similar to those cracks in Figure 2.18. It's interesting to note that the more than one in void under the bridge approach slab caused minor transverse cracks only under heavy traffic. One reason for the good performance of the bridge approach slab with the voids underneath could be due to the heavy reinforcement in the 13-inch slab. The longitudinal reinforcement in the bridge approach slabs is 1.5 percent, which is almost 2.5 times that of CRCP. The armor joint in Figure 2.22 is closed up, even though the expansion joint was in good condition as shown in Figure 2.23, indicating that the pavement did not push the approach slab. The condition of this armor joint indicates that the variations in the armor joint width are not caused by the expansion of the pavement; rather, they could be due to the displacements of the approach slabs caused by volume changes in the embankment materials.

Figure 2.21 Undersealing material

Figure 2.22 Transverse cracks due to voids

Figure 2.23 Expansion joint with support slab

Figure 2.24 Joint width of 1.5-in in wide flange terminal

2.7 Bridge on IH 45 in the Houston District

The pavement section on IH 45 near Spring Creek in the Houston District was built in February 1990. The CRCP has a 15-in thick slab on one-in ACP and six inches cement stabilized base. The wide flange terminal type was used. The paving concrete near the bridge was placed on February 21, 1990. The air temperature on that day varied from 52 °F at 9 am to 64 °F at 3 pm. The joint width at the wide flange terminal was measured on February 25, 2012, 22 years after the construction. The air temperature on February 25, 2012 varied from 52 °F at 9 am to 56 °F at 3 pm. Figure 2.24 illustrates that the joint width at the wide flange terminal system remains at 1.5 inches after 22 years of service. It is noted that the air temperatures on the day the pavement was constructed was very similar to that on the day the joint width was evaluated after 22 years. This indicates that, after 22 years of service, the length of the pavement at this location changed very little, or the net length of the pavement has remained the same after 22 years. It should be recognized that the wide flange terminal system lets the slab move freely.

2.8 Summary

Distresses have been observed in the areas near the bridge abutment, including the bridge abutment itself. It has been stated that Portland cement concrete (PCC) pushes the bridge structures, resulting in the distresses. However, the same type of distress was observed where the abutting pavement type was flexible pavement. A number of field evaluations of distresses in the terminal areas strongly indicate that volume changes in the embankment materials are partially responsible for the distresses. There could be other causes for the distresses. This issue of what is responsible for the distresses in the bridge terminal areas is quite complicated, partly due to the number of factors involved and the time it takes for the distresses to develop.

Chapter 3: Literature Review

3.1 Terminal Systems

To prevent problems due to pavement growth, terminal systems are installed at the bridge and pavement interface (TxDOT, 2011). As described in Chapter 1, there are three types of terminal systems in use: a) AL, b) WF, and c) EJ. TxDOT currently uses all three systems. These terminal systems are based on two different philosophies from the standpoint of how to accommodate concrete volume changes. As discussed in Chapter 1, the AL system tries to restrain concrete volume changes and minimizes the slab movements and potential for damage to bridge structures. The other two systems let the concrete slabs move rather freely, and provide a means to accommodate the concrete volume changes, i.e., expansion joints.

3.1.1 Expansion Joint System

The EJ system is a terminal system in which the pavement is allowed to expand, thus allowing it to release the stresses caused by expansion and contraction. A durable elastic filler material is filled in the space which accommodates the expansions. Figure 3.1 shows an EJ system.

3.1.2 Wide Flange System

The WF system consists of a wide flange beam partially set into a reinforced concrete sleeper slab. The top flange of the beam is flush with the pavement surface. The polyethylene foam (expansion material) sized to accommodate end movements is placed on the pavement side of the web portion along with a bond breaker between the pavement and the sleeper slab (FHWA, 1990). Figure 3.2 shows WF system.

3.1.3 Anchor Lug System

The AL system comprises of several (typically three or five) lugs that project into the ground and restrain the movement of the slab end. In order to design an AL system, the expected slab movement due to the change in temperature must be predicted reasonably accurately. AL design includes the calculation of the number of lugs as well as the dimensions (depth and width) of each lug.

Shelby and Ledbetter (1962) defined an AL system as "the anchorage system at the terminal joints of PCC pavements to help transfer the internal increase of the forces in the concrete due to temperature variations, into the soil mass through the passive bearing and shear resistance of the sub-soil." Figure 3.3 shows an AL system.

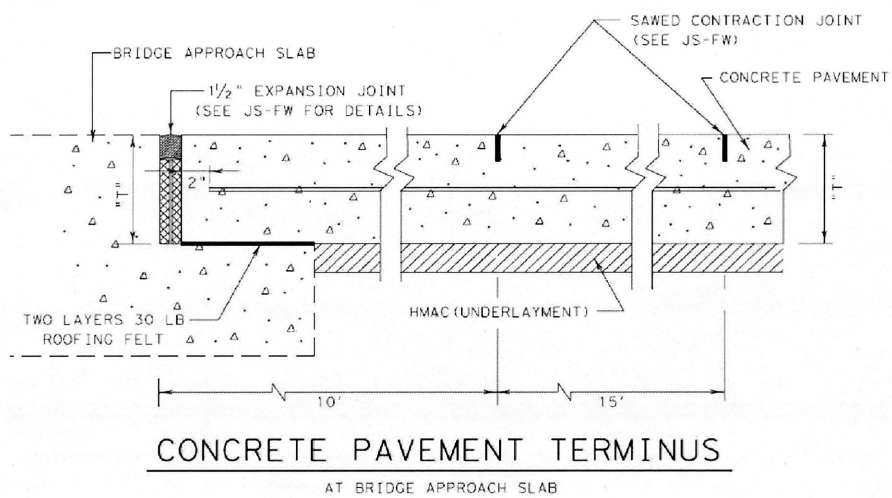


Figure 3.1 Typical expansion joint system (TxDOT, 2011)

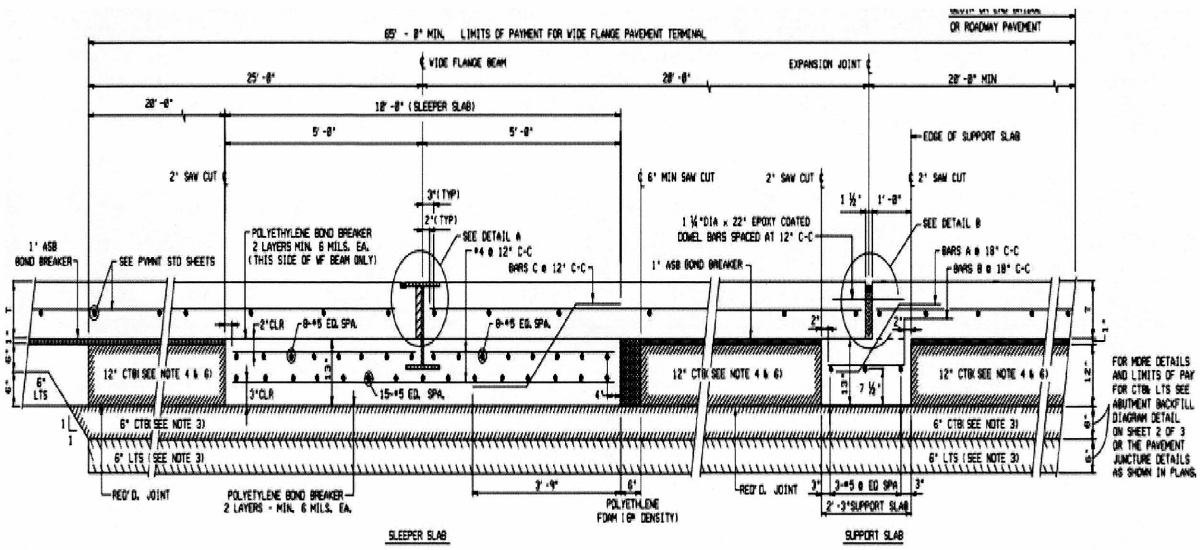


Figure 3.2 Wide-flange system (TxDOT, 2011)

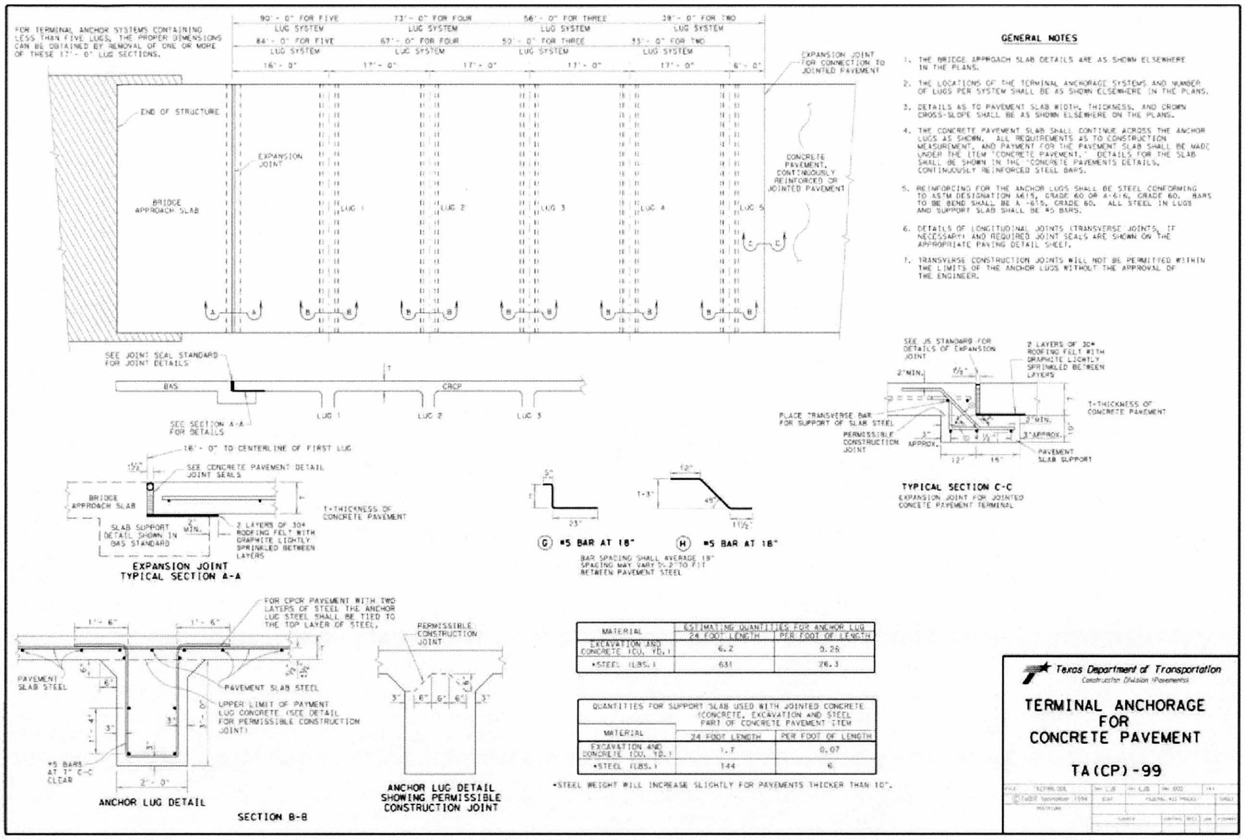


Figure 3.3 Anchor lug system (TxDOT, 2011)

3.2 Case Study

A literature review revealed few publications on the issue of slab expansions and terminal anchorage systems. Publications reviewed in this study were:

1. Mitchell R.A., 1963
2. McCullough B.F. and Sewell T.F., 1964
3. McCullough B.F., 1965
4. McCullough B.F., 1971
5. Dhamrait J.S. and Taylor R.K., 1977
6. Wesevich J.W. and McCullough B.F., 1987

7. McCullough B.F. and Wan-Yi Wu, 1992
8. Griffiths S. J., Bowmaker G.J and Bridge R. Q., 2005

3.2.1 Study by Mitchell, 1963

Mitchell conducted a full-scale experimental investigation of two different end anchor configurations; a cylindrical pile-shaped anchor and a rectangular solid-shaped anchor. A horizontal thrust was applied to four test anchor units by a jacking frame at the pavement slab level. Loads, deflections, and rotations were measured.

The objective of the study was to develop a rational analysis of end anchor units in series. A theoretical and an experimental investigation were conducted. In the theoretical phase, the elements of the problem that are of most interest to the designer were considered. The experimental phase, which was considerably more limited in scope than the theoretical phase, included a full-scale field loading test of two different configurations of anchor unit.

Mitchell developed a model with the following assumptions:

1. Uniform thickness, density, and linear elastic properties
2. Uniform volume change due to temperature, moisture, etc., if not restrained by forces external to the slab and
3. Uniform friction coefficient relative to subgrade on which it is supported.

The problem of shear and moment in the slab adjacent to the joint is more complex because a) the soil does not develop significant tensile strength, b) axial loads and transverse traffic loads are acting along the anchor loads, and c) bending and shear cracks might develop near the joint. The complexity of the problem would seem to justify a trial and error approach; that is, design should be adjusted on the basis of careful observation of the performance of anchor joints in the field. The analysis was proposed for design guidance, based heavily on intuition and simplifying assumptions, subject to modification as indicated by field observations. It was assumed that axial loads in the slab and traffic loads had negligible effect on the distribution of the shear and moment that was induced in the slab by the anchor moment. The analysis methods presented are believed to be suitable for design use in their present form, but they have thus far been correlated with field observations to a very limited degree.

3.2.2 Study by McCullough and Sewell, 1964

In this study, a number of continuously reinforced concrete pavement (CRCP) sections were constructed to develop and evaluate terminal designs. Numerous pavement ends were built using a different number of lugs, subbase type, percent grades and pavement lengths to evaluate the design. Data was collected for 2.5 years and analyzed to evaluate the parameters' effects on terminal movement. The various parameters evaluated in this study were:

A. Length of Slab

The field experimental sections had asphalt-treated subbase, -0.13 percent grade and five lugs. The results indicated that when the slab length increased to 1,000 feet, the rate of terminal movement increased. The rate of terminal movement was not affected if the length of the pavement exceeded 1,000 feet. When the rate of end movement increased with the increase in slab lengths up to 1,000 feet, then half of the total length of the slab contributed to end movement at both ends. That means only 500 feet of the CRCP contributed to the end movement at an expansion joint. However, this optimum length was just for asphalt-treated subbase.

B. Percent Grade

In the development of the design, the percent grade effect was analyzed theoretically only. The greater the grade, the more weight of the pavement acts down the plane in a direction parallel to the pavement surface. To evaluate percent grade, pavement lengths were taken more than 1,000 feet. Asphalt- treated subbase and five lugs with different graded end systems were considered for the analysis. It was determined that the rate of the end movement decreased as their corresponding grade increased. However, due to very few data selected for this analysis, the results were generalized and might not be true.

C. Coefficient of Friction

It was known that crushed sandstone, asphalt treated subbase, cement treated and surface treated subbase all differ in coefficient of friction. Surface treatment subbases have a higher coefficient of friction compared to asphalt stabilized subbase and crushed sandstone subbase. In this study, only theoretical values of friction were considered. Actual coefficient of friction values might be different from the values used in this study.

D. Number of Lugs

To study the effect of the number of lugs, pavement ends were built utilizing two, three, four, five and no lugs. From the data collected, the greatest rate of pavement movement was observed in two-lug systems. The four- and five-lug systems, even though having a lower rate of end movement than the two-lug system, had the same rate of end movement as the three-lug system. The two lug system experienced 1.4 inches of movement for a 100 °F temperature changes.

3.2.3 Study by McCullough, 1965

The objective of this investigation was to determine the extent of anchorage failures, and to perform the field observations and operations necessary to obtain data for use in re-evaluating the terminal anchorage design. A total of 152 anchor units were investigated and data collected. The coarse aggregate type in the concrete, joint spacing, type of base, and stabilization were the pavement structure-related factors included in the investigation.

It was concluded from these studies that terminal anchorage failures could be classified into two types: complete closure of all expansion joints, and faulting in the abutment walls. The factors that were related to failures were the pavement structure, age of anchor slab, distance from the lug to the abutment wall, the approach slab grade, and the presence of cracks in the anchor slab. The factors investigated in this study were:

A. Coarse Aggregate

It was observed that all failures were in sections that had concrete made with SRG coarse aggregate. This may be due to the higher thermal coefficient of expansion/contraction in SRG concrete. Concrete with the other coarse aggregate type, oyster shell, showed only about one-half the thermal coefficient of expansion of SRG concrete. Therefore, the shell concrete pavement would produce about one-half the force on the anchor system that would be obtained with pavement with SRG, making it much less likely to fail compared to concrete pavements with SRG.

B. Joint Spacing and Subbase Type

In the case of joint spacing, it was observed that failures occurred in both 15 ft and 61.5 ft joint spacing. Also, in considering the effect of subbase, it was noted that failure occurred with every type of subbase with the exception of cement-stabilized iron ore gravel. The absence of failures on the cement stabilized subbase might be due to high friction between the cement stabilized base and the concrete slab, reducing slab movement. However, it should be noted that at the time the observations were made, pavements with cement stabilized subbase were less than two years old.

C. Pavement Thickness and Distance from the Abutment Wall

Theoretically, the magnitude of the force generated due to pavement growth should be directly proportional to pavement thickness, but it was observed that sufficient forces were developed to cause failure of the anchor system in the thinnest pavements. No failures occurred in the anchor system when the anchor lug was at a distance of 75 ft or greater from the abutment wall. It was also observed that any anchor unit at a distance of less than 60 ft either failed, or was likely to fail. Therefore, it appeared that for embankment soils in the Houston area, a minimum distance of 60 to 75 ft was required between the header or abutment wall to the first anchor lug.

D. Slab Grade

Data also showed that anchor units on level grade showed relatively good performance and that all failures occurred on a grade of three percent or more. All positive grades were overpass structures that were constructed on embankments. It is possible that the anchor lug in the constructed embankment would not have had as high a resistance for slab movement as with the natural soil.

The report showed that 94 percent of the satisfactorily performing anchor systems had cracks in the anchor slabs, whereas only 56 percent of the anchor systems that were in a failure condition had cracks in their slabs. Six percent of units without cracks that were satisfactorily performing were less than two years old, and may not have been old enough to show any effects due to forces generated by pavement growth. It was concluded that the presence of transverse cracks in the anchor slabs was one of the positive sign indicating that the anchor system was performing satisfactorily. The cracks may have been volume change cracks caused by the presence of anchor lugs.

3.2.4 Study by McCullough, 1971

The effects of subbase type, length of the slab contributing to end movement, grade, and number of lugs on the slab end movement in the terminal systems were investigated. Based on the statistical analysis of the field data obtained, a design equation was developed with the assumption that a maximum of 500 ft of CRCP contributes to the end movement. Predicted slab movements from this equation for typical CRCP with asphalt stabilized subbase varies from 0.34 inches (1,000-ft length with five lugs) to 0.57 inches (5,280-ft with 1 lug) for 100 °F temperature variations.

3.2.5 Study by Dhamrait and Taylor, 1977

To evaluate the effectiveness of the systems and to develop information which will permit refinement of the design standards and specifications, two types of terminal systems were used for the evaluation, the AL and WF.

Extensive field evaluations were conducted by the staff engineers at the Illinois Department of Transportation. The average maximum movement of the slab at any WF system was less than 3/4 inch. The WF system performs satisfactorily, provides a smooth transition between the abutting pavements, and offer excellent shear transfer. The joints can probably be constructed without the sealant with no detrimental effects, as no apparent problems have developed because of the absence of the sealant. This report states that an excavation revealed voids of about 6 to 12 inches in the subbase adjacent to either side of a lug wall. That much gap between the lug and the soil indicates that at one point ALs had pushed and then moved back to form voids.

Visual inspections revealed ¾-inch to 3/8-inch gaps or voids between the pavement and the subbase, which indicates that the damage mechanism could be consolidation or volume instability of the embankment materials, and not PCC pavement slabs pushing the bridge structures. Both AL and WF showed issues related to the embankment material consolidation.

Limited data obtained in this study indicated that average annual winter to summer movements in the AL system did not exceed 3/4 inch, which is close to what was found in this study in Texas. The range of movement in both Illinois and Texas was 3/4 inch to one inch. Most problems occurred about five years after construction, with four lugs at 40 ft spacing; few problems were observed at locations with three lugs placed at 20 ft spacing. The dips between lugs were observed apparently caused by the rotation of the lugs.

3.2.6 Studies by Wesevich, Wimsatt and McCullough, 1987

The studies by Wesevich, Wimsatt and McCullough (1987) discuss the concepts of subbase friction, including three components: an adhesion, or gluing, component between the concrete pavement and the subbase; a bearing component that is influenced by the surface texture of the subbase; and a shearing component which is induced by the movement of the slab across the subbase. The frictional effects of stabilized subbases were not adequately pursued until this study.

The push-off tests experimentation on several subbases found that, on all but the cement stabilized subbases, the maximum frictional stresses ranged between 0.6 psi on an untreated clay subbase to 3.4 psi on a flexible subbase. Testing on two asphalt stabilized subbase layers indicated that the subbase thickness and internal temperature had substantial effect on the subbase frictional properties. It was also found that the asphalt stabilized subbase surface texture did not significantly affect the frictional restraint. Results of using the indirect tensile testing of subbase cores to estimate subbase friction are presented.

A significant result from this study was that the asphalt subbases had considerably higher frictional coefficients than their unbound counterparts, and that the polyethylene sheeting had a very low fricitonal coefficient. It was evident that cement-stabilized subbases caused relatively large tensile stresses in concrete due to temperature variations compared to other subbases and hence it was recommended to specify friction reducers or bond breakers between any concrete pavement and cement stabilized subbases.

3.2.7 Study by McCullough and Wu, 1992

In this study, the PSCP2 computer program developed at the Center of Transportation Research of the University of Texas at Austin was utilized for the analysis of the slab movement behavior due to temperature variations. The results of the analysis are divided into two parts. The first part investigates the effect of aggregate type, subbase friction force, season of placement, and slab thickness on the relationship between end movement and slab length. The second part studies the effect of end movement in relation to slab temperature.

This research indicated that thermal expansions were unavoidable but could be minimized by properly designing and placing the concrete at the appropriate time. The results obtained in this theoretical study are discussed below:

A. *Relationship between End Movement and Slab Length*

Figure 3.4 illustrates the relationship between total slab length, L, and total end movement 2ΔX. This study found that the maximum length of CRCP contributing to the slab end movement was 1,250 ft.

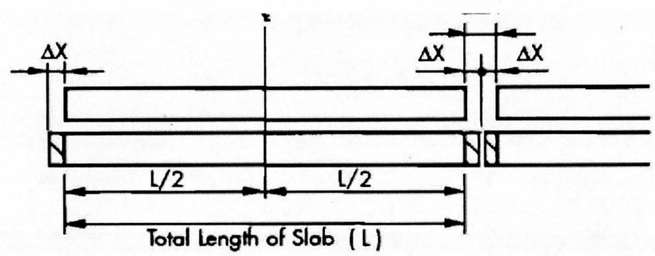


Figure 3.4 Relationship between total slab movement and total end movement (McCullough and Wu, 1992)

B. *Effect of Aggregate Type on the Slab Movement*

By holding other variables constant, a typical comparison of effects for two aggregate types, LS and SRG, was made. At the same condition, the end movement of the SRG was always greater than that of the LS. This is because the thermal coefficient of SRG is 1.33 times greater than that of LS. Figure 3.5 shows the typical seasonal movement in relation to slab length and a comparison between SRG and LS.

C. *Effect of Sub-Base Friction Force on Movement of Pavement*

Figure 3.6 illustrates that a smaller frictional force corresponds to greater end movement. It shows typical seasonal movement in relation to slab length, and a comparison among three different friction forces. As expected, as the friction force decreased, the pavement movement met less resistance so that the end movement increased. Thus, the friction force effect is significant for long CRC pavement slabs, but not for short ones. This is explained by the fact that, if the slab length is short, the friction force will be in the elastic range, and thus less sliding will occur.

D. *Effect of Slab Thickness on Movement of Pavement*

Figure 3.7 illustrates that more total movement can result from a thicker slab than a thinner one. The thicker slab has a larger cross-sectional area and produces larger forces, resulting in increased horizontal force and thus more movement.

E. *Slab Grade*

The rate of end movement decreases with increasing slab grade. This trend is the same irrespective of the pavement having a plus or minus grade, a variable number of lugs, or different types of subbases.

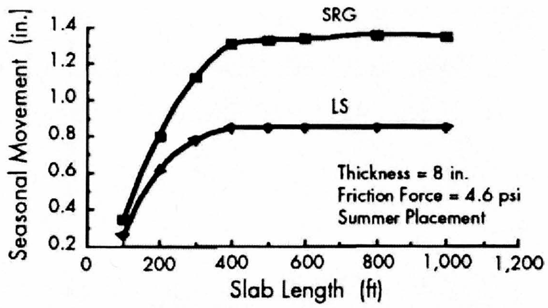


Figure 3.5 Typical seasonal movement in relation to slab length and a comparison between SRG and LS (McCullough and Wu, 1992)

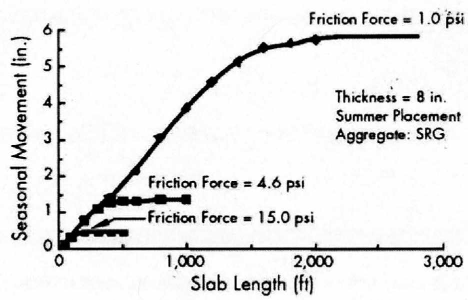


Figure 3.6 Typical seasonal movement in relation to slab length, and a comparison among three different friction forces (McCullough and Wu, 1992)

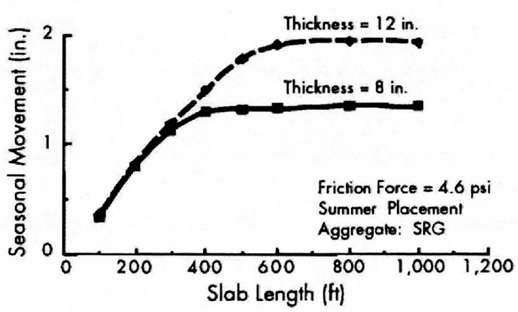


Figure 3.7 Typical seasonal movement in relation to slab length and a comparison between 8-inch and 12-inch slabs (McCullough and Wu, 1992)

The analysis results were found from the computer model and were compared with the field results from three sites whose friction was not known. The number of sites investigated was very few. The end movements obtained in field were not comparable with the PSCP2 model results. The terminal anchor systems could not be evaluated in the program and hence the systems were not taken into consideration.

Chapter 4: Field Experimentation

To evaluate slab movement at the end of CRCP with accuracy and consistency, a field testing protocol was developed. Since long-term evaluation of slab movement is needed in this study, concrete displacement gages that provide accuracy, high resolution, and long-term stability should be used. Gages based on measuring resonant frequency of vibrating wire provide accurate and reliable long-term displacement measurement (Window and Hollister, 1982; Larive et al., 1995). Because this type of gage measures the frequency, which is imperturbable and can be transmitted over long cable length, it has shown the stable measurements over long periods regardless of changes of resistance or length of leads. The concrete displacement gage installed in the field testing were crackmeters (Geokon Model 4420), strain gages (Geokon Model 4202), and Jackout assembly earth pressure cell (Geokon Model 4820) which is based on vibrating wire strain gage (VWSG) technology.

4.1 Instrumentation of Gages

4.1.1 Crackmeter Installation Procedure

Proper selection of the location of gage installation is important, as there are several transverse joints in terminal system in CRCP. Regardless of the terminal systems in place, the gage should be installed a few inches in the pavement side from the transverse joint that is the farthest from the armor joint.

For this study, a transverse hole was drilled on the edge of the pavement slab to install the sensor anchor. The vertical location of the hole was located about two inches to three inches below the surface of the slab as shown in Figure 4.1. To anchor the crackmeter to the fixed point, a vertical hole was drilled on the ground about three inches away from the edge of the pavement slab as shown in Figure 4.2. The depth of the vertical hole should be deep enough to go through any stabilized subbase. The three-ft long invar, which has minimum thermal expansion, was inserted into the ground through the vertical hole. One end of the crackmeter was connected to the sensor anchor and the other to the invar as shown in Figure 4.3. This system measures the longitudinal movement of the slab relative to the ground. After installing the crackmeter, a protective metal cover box was installed to ensure the long-term stability of the measurement system. Wires from the crackmeter were connected to the data logger. Figure 4.4 shows a one-channel datalogger used in this study.

Figure 4.1 Drilling of transverse hole to install a groutable anchor

Figure 4.2 Drilling of a vertical hole to install invar

Figure 4.3 One end of crackmeter is connected to groutable anchor and the other to the invar

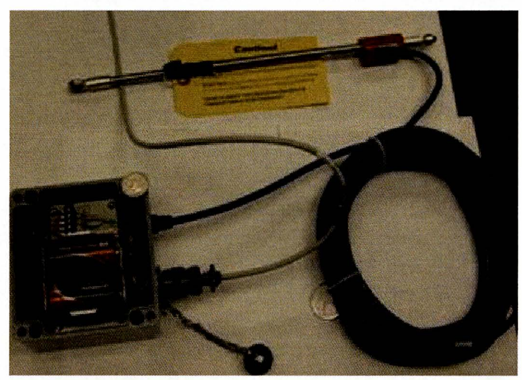


Figure 4.4 Wires from crackmeter connected to data logger

4.1.2 Vibrating Wire Strain Gage Installation Procedure

To conduct in-depth evaluations of overall anchor lug behavior, six-inch concrete embedment strain gages were used in one section. Strains are measured using the vibrating wire principle: a length of steel wire is tensioned between two end blocks that are firmly in contact with the mass concrete. Deformations in concrete will cause the two end blocks to move relative to one another, altering the tension in the steel wire. This change in tension is measured as a change in the resonant frequency of vibration of wire. The primary means of gage placement is direct embedment in concrete by pre-attaching the gage to rebar and then concrete is poured. To provide good support at rebar, the strain gage is tied with a thin block and tied with tie wires as shown in Figure 4.5.

4.1.3 Pressure Cell

Model 4820 Jackout assembly earth pressure cells were installed to measure the soil pressure in contact with the concrete in the anchor lug. The pressure cell was installed one ft from the bottom of a lug to properly evaluate the behavior of the AL system. The pressure transducer housing is connected directly and perpendicularly to the thick back plate, as shown in Figure 4.6. A seating was made to arrange the pressure cell in proper position when installing the lug. The contact area of the soil in the lug was properly smoothed by applying a thin coat of plaster of Paris. When the pressure cell assembly was placed in position at the bottom of the lug, the jack was activated so the two plates are moving towards the soil and in good contact as shown in Figures 4.7 to 4.10.

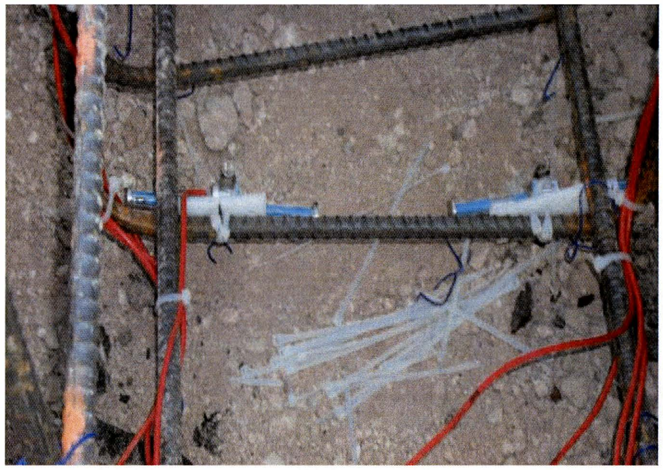


Figure 4.5 Installation of concrete strain gage

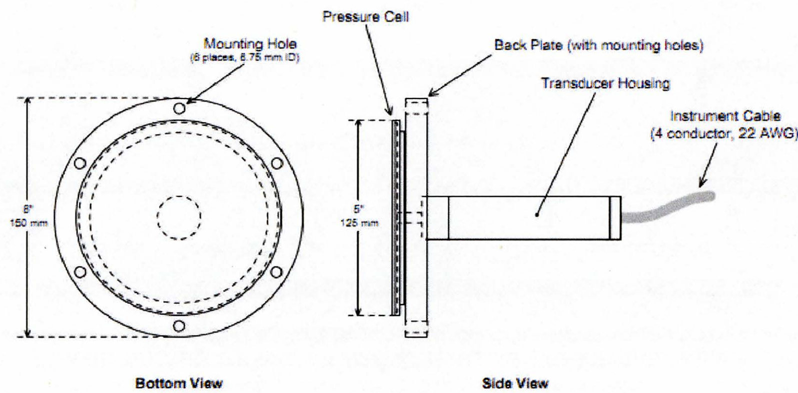


Figure 4.6 Detail view of the pressure cell

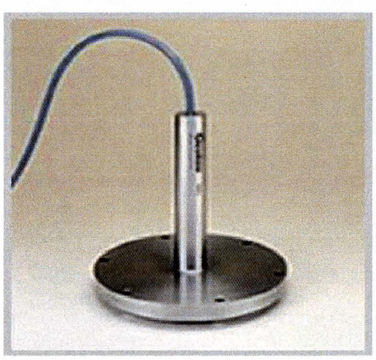


Figure 4.7 Pressure cell

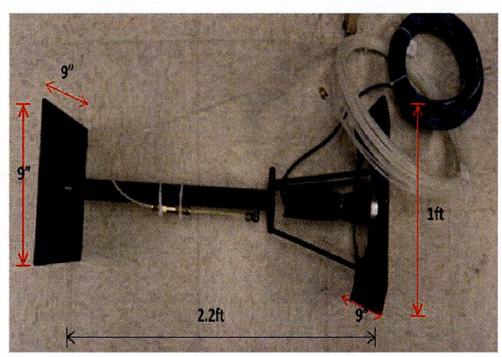


Figure 4.8 Jackout pressure cell assembly

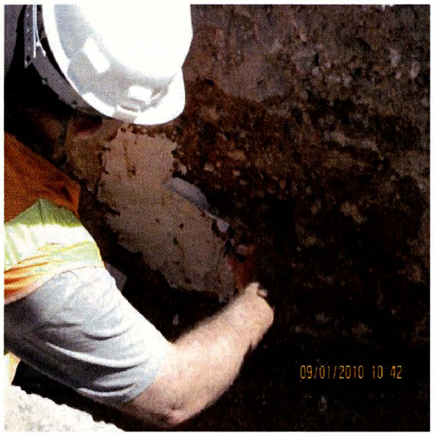


Figure 4.9 Applying plaster of Paris for good contact between pressure cell and soil

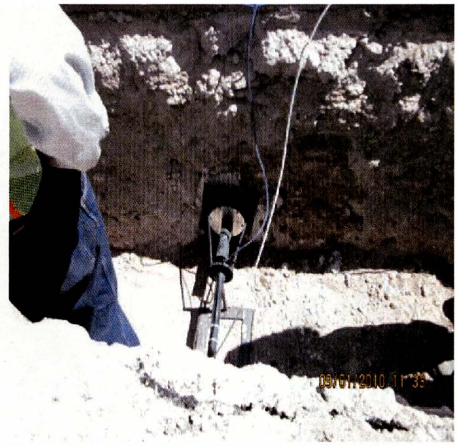


Figure 4.10 Installation of pressure cell in the AL

4.2 Selection of Sites

A total of eight projects were selected to measure slab movements. For the test section selection, two variables were considered: terminal type and geographical locations. Table 4.1 lists the test sections selected for slab movement evaluations and Figure 4.11 shows the location of the test sections.

Table 4.1 Summary of Test Sections

Test Section location	Highway	Terminal Type	Coarse Aggregate Type	Length of Pavement, miles	Slab Thickness, inches	Subbase Type	Date of Installation
El Paso	Spur 601	EJ	LS[a]	0.80	13	3 in. ASB[c]	4/21/09
Wichita Falls	US 82	EJ	LS	0.50	13	3 in. ASB	3/18/09
Linden	US 59	WF	SRG[b]	> 1.00	13	4 in. ASB	10/1/09
Houston	US 290	WF	LS	1.05	11	1 in. ASB + 6 in. CSB[d]	10/04/07
Texarkana	LP 151	AL	SRG	0.45	13	4 in. ASB	5/22/07
Lubbock1	US 82	AL	SRG	0.45	12	4 in. ASB	3/18/09
Lubbock2	Loop 289	AL	LS	0.45	10	1 in. ASB + 6 in. CSB[d]	9/2/10
Waco (PTCP)	IH 35	Armor Joint	LS	0.06	9	4 in. ASB	2/23/09

[a]: limestone; [b]: siliceous river gravel; [c]: asphalt stabilized base, [g] : cement stabilized base

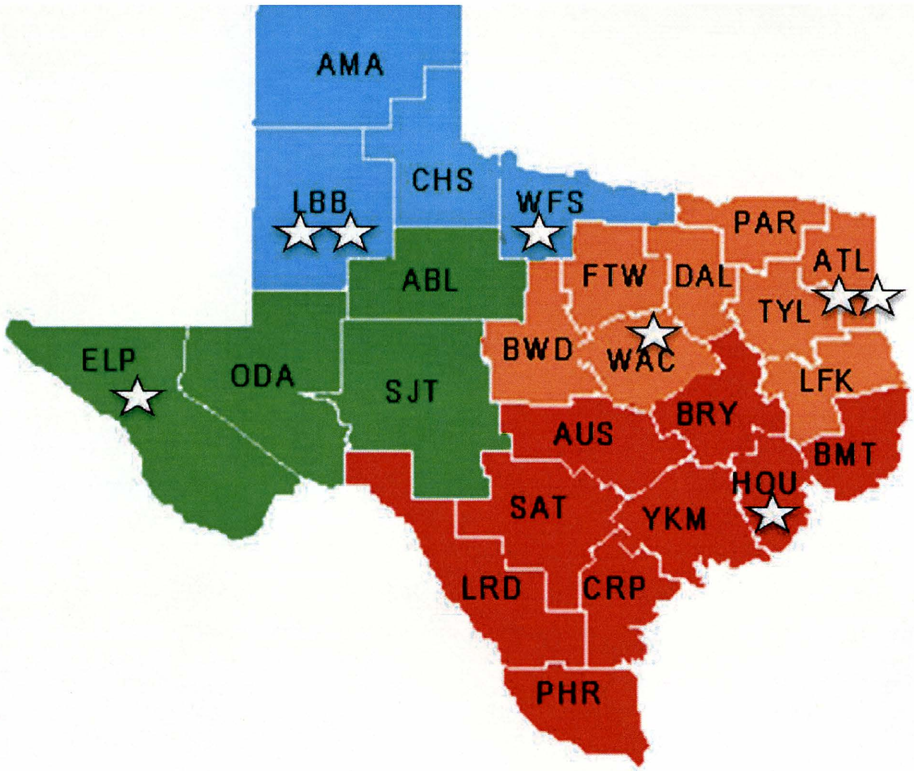


Figure 4.11 Map showing all sections

4.2.1 El Paso

The El Paso site was located at Spur 601 in El Paso. EJ was used as the terminal system to accommodate concrete slab movement. The slab thickness of the concrete pavement was 13 inches. The length of pavement was 0.80 mile. Subbase type used was three inches ASB. Aggregate type was LS. Crackmeters were installed at this site on April 21, 2009. Figures 4.12 and 4.13 show the location of the bridge and the crackmeter position. A protection box was used to protect the crackmeter and the datalogger as shown in Figures 4.14 and 4.15.

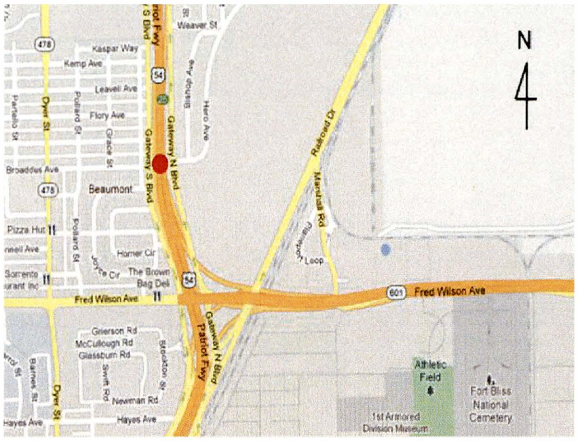


Figure 4.12 Map showing the location of the site

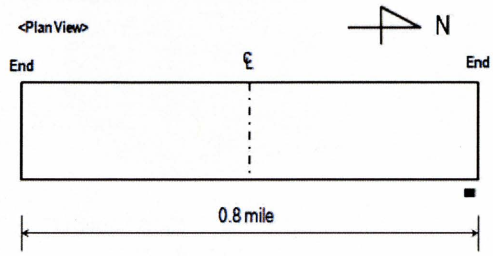


Figure 4.13 Location of the crackmeter

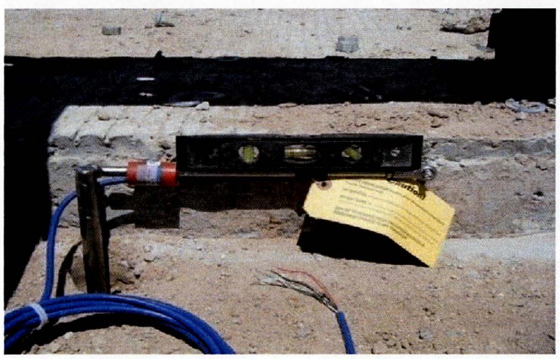


Figure 4.14 Installation of the crackmeter

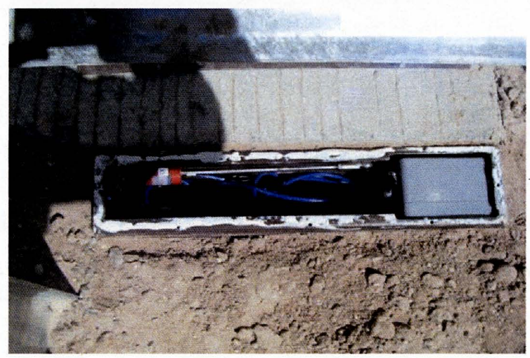


Figure 4.15 Setup of the crackmeter with the protection box

4.2.2 Wichita Falls

The Wichita Falls site was located on US 82 in Wichita Falls. EJ was used as the terminal system to accommodate concrete slab movement. The slab thickness of the concrete pavement was 13 inches. The length of pavement was 0.50 mile. Subbase type used was three inches ASB Type B. The aggregate type used was LS. Four crackmeters were installed at this site on March 18, 2009. Figures 4.16 and 4.17 show the location of the pavement and crackmeter positions where they were installed. After damage to the gages, new crackmeters were installed on May 11, 2010 (1, 5, 7, 8 in Figure 4.17) and four new crackmeters were installed on March 4, 2011 (2, 3, 4, 6 in Figure 4.17). Figure 4.18 illustrates the installation of crackmeters. Protection boxes were installed at all the crackmeter locations as shown in Figure 4.19.

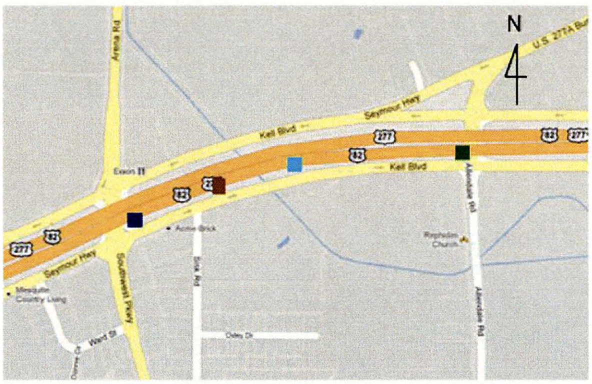


Figure 4.16 Location of the bridges and crackmeters

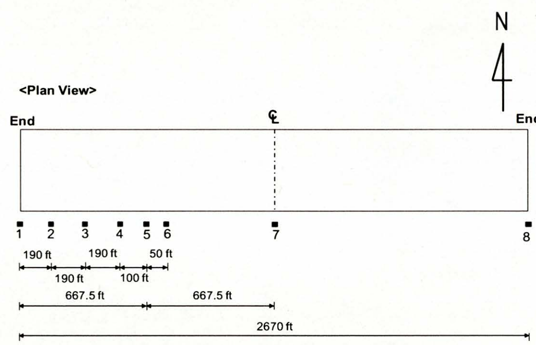


Figure 4.17 Location of eight crackmeters

Figure 4.18 Installation of the crackmeter

Figure 4.19 Crackmeter setup with protection box

4.2.3 *Linden*

The Linden site is located on US 59 northbound as shown in Figure 4.20. WF was used as the terminal system to accommodate concrete slab movement. The slab thickness of the concrete pavement was 13 inches. The length of pavement was greater than a mile. Subbase type was open graded 4 inches ASB. Aggregate type was SRG. The construction and placement of WF system are shown in Figures 4.21, 22, 23 and 24. A crackmeter was installed at this site on October 1, 2009. Figure 4.25 shows the installation of a crackmeter near the WF at the end of pavement. A protection box was also placed to protect the crackmeter and the datalogger as shown in Figures 4.26. The construction was completed on October 1, 2009 as shown in Figure 4.27.

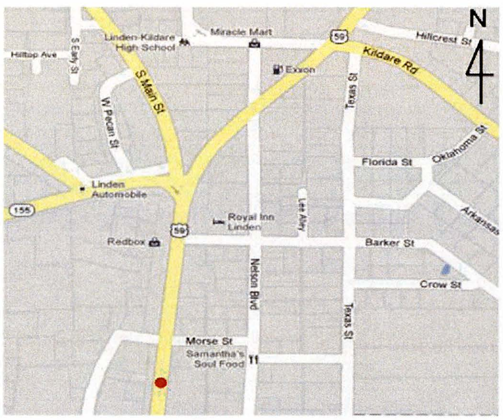


Figure 4.20 Map showing the location of the pavement on US 59

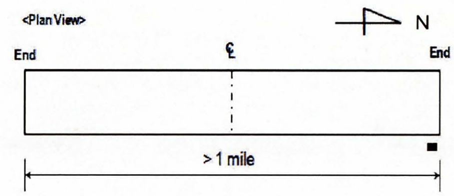


Figure 4.21 Crackmeter location at the end of the pavement near WF

Figure 4.22 Pouring of concrete at the section

Figure 4.23 Finishing of the concrete

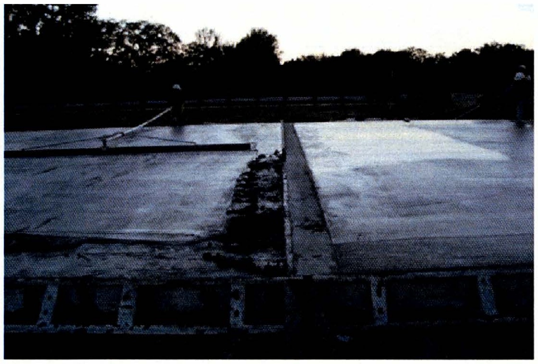


Figure 4.24 WF system placed

Figure 4.25 Installation of the crackmeter with protection box

Figure 4.26 Setup of the datalogger and the crackmeter

Figure 4.27 Pavement construction completed

4.2.4 Houston

The Houston site was located on US 290 as shown in Figure 4.28. WF system was used as the terminal system to accommodate concrete slab movement. The slab thickness of the concrete pavement was 10 inches. The length of pavement was 1.05 mile. Subbase was Type D one-inch ASB plus six inches CSB. Aggregate type was LS. The section was constructed in 1995 and a crackmeter was installed at this site on April 10th 2007. The location of the WF and the crackmeter are shown in Figures 4.29 to 4.32.

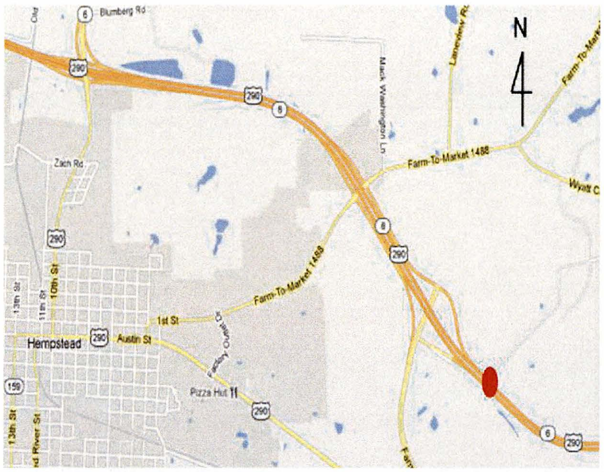


Figure 4.28 Map showing the location of the site

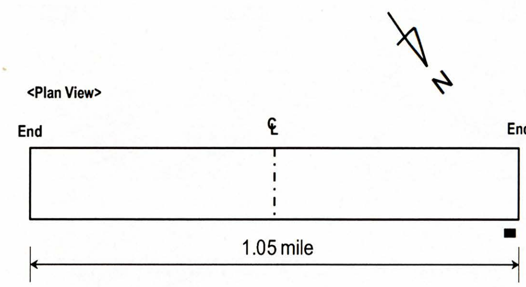


Figure 4.29 Location of crackmeter at the end of pavement near WF

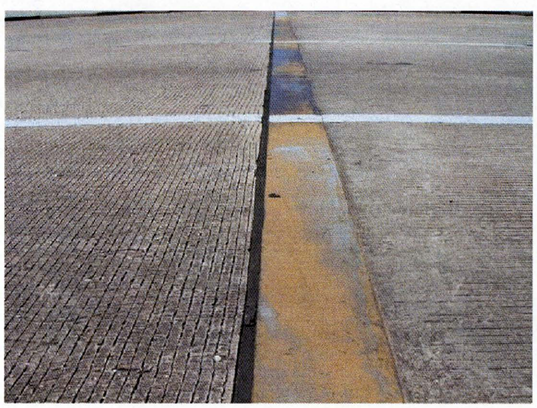


Figure 4.30 WF system

Figure 4.31 Location of crackmeter installation

Figure 4.32 Crackmeter installed with the datalogger box

4.2.5 Texarkana

The Texarkana site was located on LP 151 as shown in Figure 4.33. This pavement is not a new section. An AL system was used as the terminal system to restrain movements. The slab thickness of the concrete pavement was 13 inches. The length of pavement was 0.45 mile. Subbase type was four inches ASB. Aggregate type was SRG. A crackmeter was installed at this site on May 22, 2007 as shown in Figure 4.34. Figures 4.35 and 4.36 show the datalogger setup at the guardrails and the overview of the pavement where the crackmeter was installed, respectively.

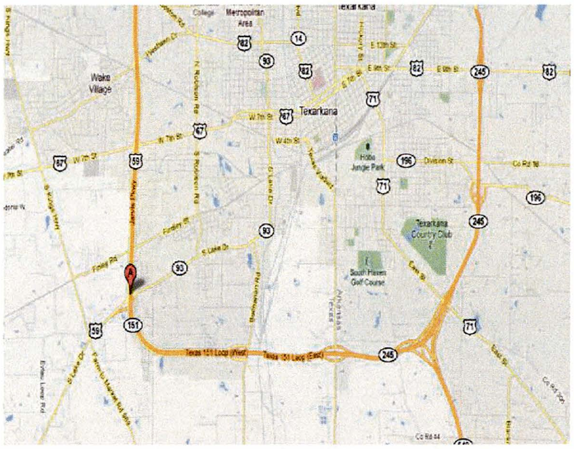


Figure 4.33 Map showing the location of the pavement

Figure 4.34 Crackmeter installation

Figure 4.35 Datalogger installation

Figure 4.36 Pavement shown at the location

4.2.6 Lubbock 1

The Lubbock 1 site was located on US 82 Marsha Sharp Freeway as shown in Figures 4.37 and 4.38. An AL system with four lugs was used as the terminal system to restrain movement. The slab thickness of the concrete pavement was 12 inches. The length of pavement was 0.45 mile. Subbase type was four inches ASB. Aggregate type was SRG. A crackmeter was installed at this site on March 18, 2009 as shown in Figure 4.39.

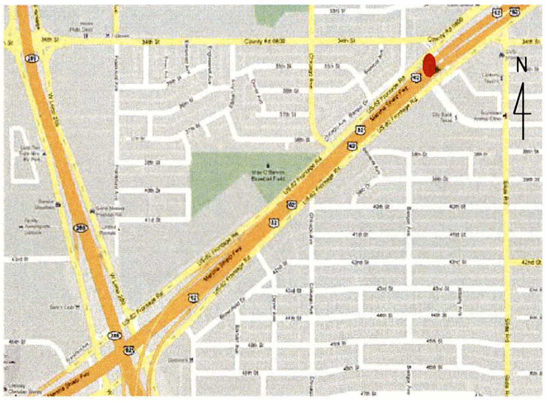


Figure 4.37 Map showing the location

Figure 4.38 Pavement at US82 under construction

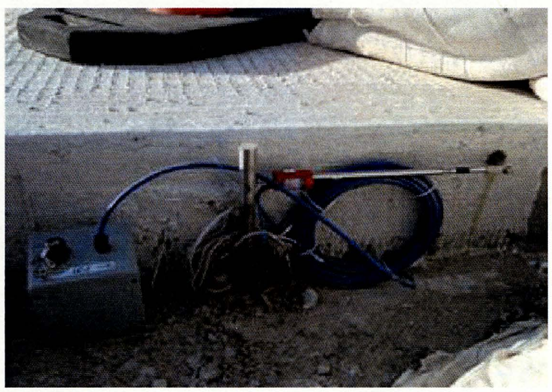


Figure 4.39 Installation of the crackmeter and the datalogger

4.2.7 Lubbock 2

The Lubbock 2 site was located on Loop 289 as shown in Figure 4.40. An AL system consisting of four anchor lugs was used as the terminal system. The slab thickness of the concrete pavement was 10 inches. The length of pavement was 0.45 mile. Subbase was Type D one-inch ASB plus six inches CSB as shown in Figures 4.41 and 4.42. Aggregate type used was LS.

Detailed behavior of the AL for temperature variations was evaluated using vibrating wire strain gages, crackmeters, and pressure cells. The strain distribution along the depth of the lug provides useful information on the resistance provided by the surrounding soil and subbase. This information was essential to know whether the AL could restrain movement of pavement for a long period. One pressure cell was installed. The purpose of installing pressure gages was to evaluate soil pressure exerted on the soil by lug movement.

The nearest (AL1: anchor lug 1) and farthest (AL4: anchor lug 4) anchor lugs from the bridge approach slab were selected as the instrumentation area in the test section as shown in Figures 4.43 and 4.47. As there was no place for installing the datalogger box in the area of the pavement, it was decided to use PVC conduits to send the cables of the gages and to arrange the datalogger box at the bottom of the retaining wall. Figure 4.44 shows the arrangement of the pipes towards AL1 and AL4.

Figure 4.45 shows the anchor lugs area excavated on August 31, 2009. In the night of August 31, all gages were brought to the site, and the cables were sent through the PVC conduit. On September 1, the steel cages were assembled at the job site as shown in Figure 4.46. Wooden planks were used to fix the gages in position. Eight VWSGs were installed in the two lugs. Figure 4.47 illustrates the section and gage installation scheme. Two pressure cells were installed in opposite directions in the AL4, which is away from the bridge as shown in Figures 4.47 and 4.50. All the VWSGs were installed eight ft from the retaining wall. To ensure a good contact of pressure cells to the soil, the surface of the walls were flattened, and a thin layer of plaster of Paris was applied as shown in Figure 4.48. Figure 4.49 shows the gages installed in the AL near the bridge (AL1). Concrete was poured in this lug in the morning of September 2, 2010 (Figures 4.51 and 4.52). The two pressure cells were installed 13 ft from the edge of the retaining wall. The main lane paving was done on September 30, 2010 as shown in Figure 4.53. The concrete placement started at 11 am. Concrete strain gages in the pavement slab (L1, L2, R1, and R2 in Figure 4.47) were installed in the CRCP at this time. The two crackmeters were also installed on November 8, 2010 when the shoulder was placed as shown in Figure 4.54. The shoulder was placed with concrete at 1 pm as shown in Figure 4.55.

Figure 4.40 Location of the pavement section

Figure 4.41 Subbase type 6 inches CSB

Figure 4.42 1 inch ASB on 6 inches CSB

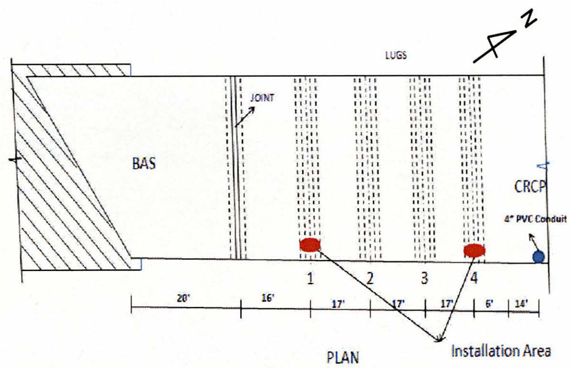


Figure 4.43 Installation location of all the gages

Figure 4.44 Pipes to protect gage cables

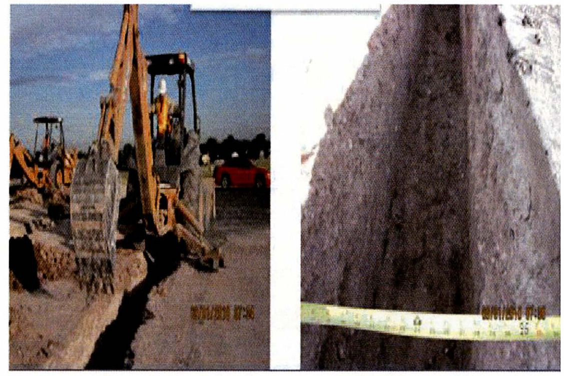


Figure 4.45 Excavation of the lugs

Figure 4.46 Cage tied and placed in the excavated area

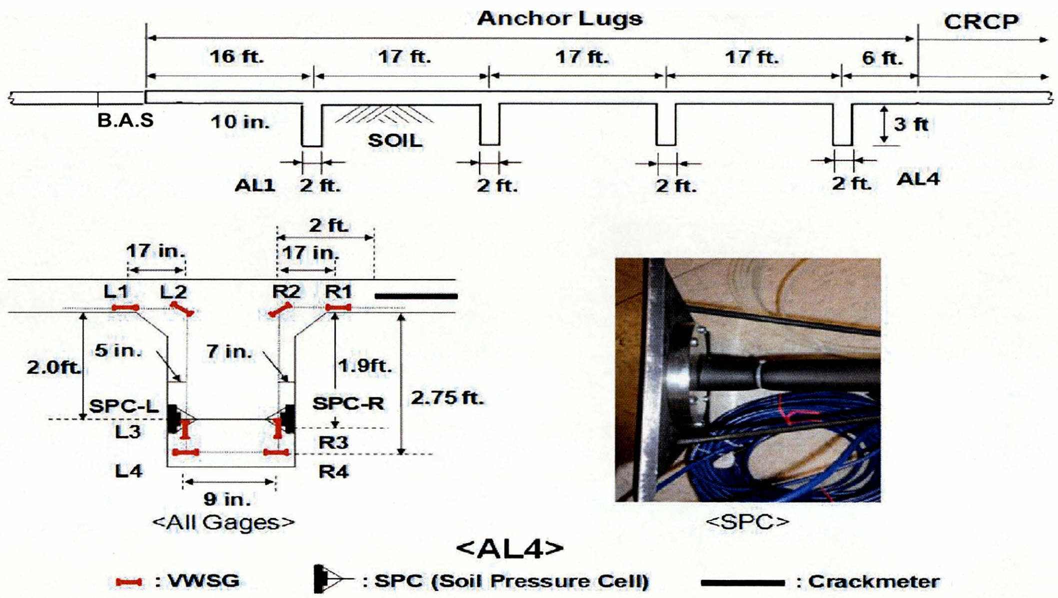


Figure 4.47 Section view of the anchor lugs, gages installation shown, jack out pressure cell

Figure 4.48 Plaster of Paris applied for the good contact between pressure cell and the soil

Figure 4.49 Installation of VWSGs in AL1

Figure 4.50 Installation of jackout pressure cells

Figure 4.51 Concrete pouring in the lugs

Figure 4.52 Covering the gages with concrete

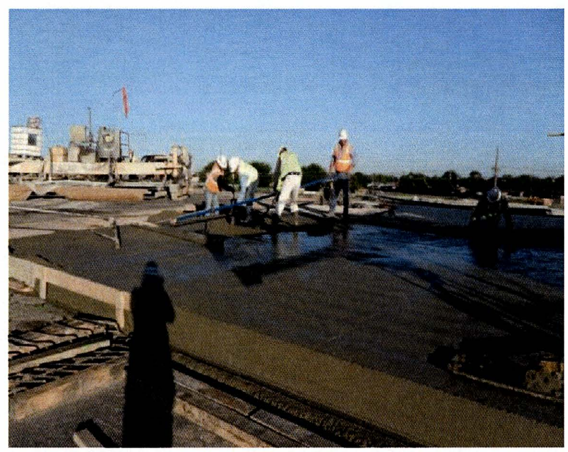


Figure 4.53 Concrete placement on the AL

Figure 4.54 Crackmeter installation with protection

Figure 4.55 Concrete placement in the shoulder area

4.2.8 Waco

Cast-in-place concrete post-tensioned concrete pavement was constructed on IH 35 in Hillsboro in the Waco District from 2007 to 2009. To investigate the effect of subbase friction on slab movement, the information on the slab movement of this pavement section was utilized. The slab thickness of the pavement was nine inches. The length of the pavement slab between armor joints was 300 ft. Subbase type was four inches ASB. Aggregate type was LS. Two layers of polyethylene sheet were placed between the subbase and concrete slab to reduce the friction of the slab as shown in Figures 4.56 and 4.57. Figure 4.58 shows the concrete placement. Crackmeters were installed at this site on February 23, 2009 as shown in Figure 4.59. At the free edge of the slab, five crackmeters were installed at different distances from the slab center as shown in Figure 4.60.

Figure 4.56 Polyethylene sheet placed between subbase and CRCP

Figure 4.57 Construction procedure

Figure 4.58 Pouring of concrete

4.59 Installation of crackmeter

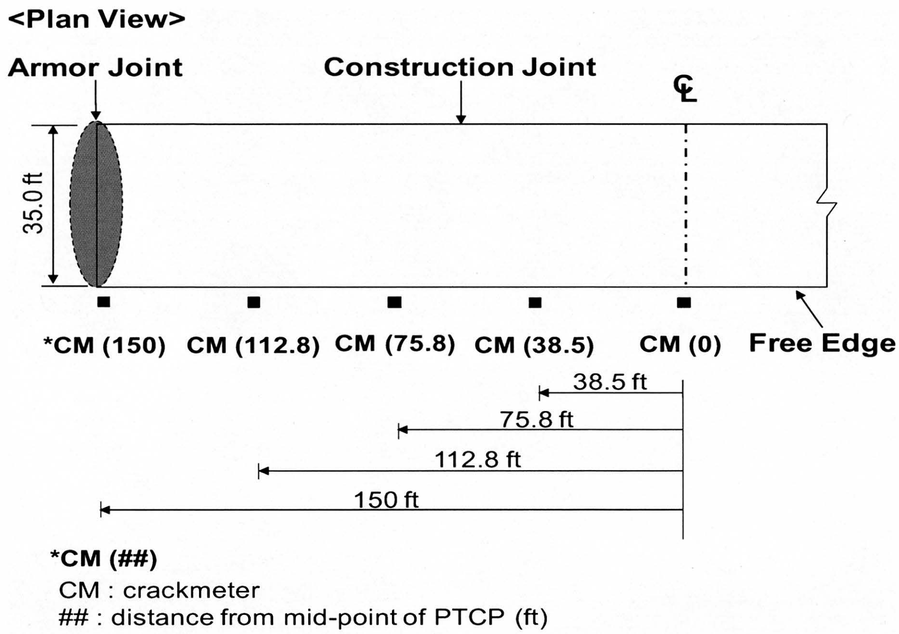


Figure 4.60 Positions of the crackmeters

Chapter 5: Collection and Analysis of Data

The eight sites described in Chapter 4 were visited on a periodic basis for site data collection. The following relationships were studied for the sections.

1. Slab movement vs. concrete temperature variations
2. Daily and yearly rate of slab movement
3. Concrete strains vs. concrete temperature variations (Lubbock 2 only)
4. Concrete strains vs. time (Lubbock 2 only)
5. Stress of the soil vs. temperature variations (Lubbock 2 only)

5.1 El Paso (EJ system)

Figure 5.1 shows a good correlation between slab movement and temperature variations. While the daily movement rate was observed to be 0.56 mils/°F for daily temperature variation, the seasonal movement rate was observed to be 9.59 mils/°F from winter (25 °F) to summer (90 °F). The seasonal slab movement was slightly larger than 0.8 inches. The rate of seasonal movement was much greater than the daily rate. This difference was likely caused by friction with the subbase, which was not elastic when the slab was slowly expanding or contracting. Due to the larger time-rate of daily temperature variations, the interface between the concrete slab and subbase acted as an elastic material and the subbase friction provided great resistance, resulting in a smaller daily slab movement rate. On the other hand, subbase friction did not remain elastic when the seasonal rate of temperature variation was quite small, and thus, friction became less effective, resulting in a larger seasonal slab movement rate.

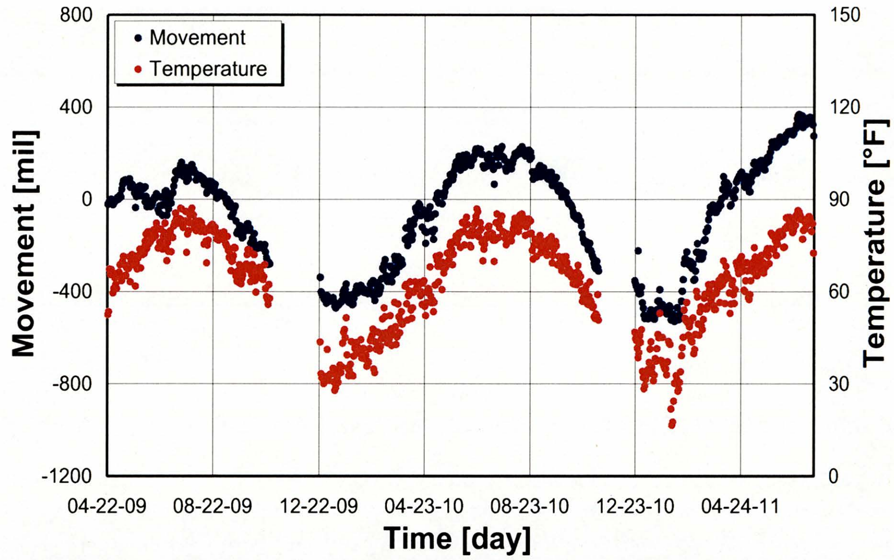


Figure 5.1 Concrete slab movement vs. concrete temperature April 22, 2009 to August 22, 2011

5.2 Wichita Falls (EJ system)

The total length of the test section between adjacent bridges was 2,670 ft. In order to evaluate the distribution of CRCP movements along the pavement, four crackmeters were installed initially in this section, and four additional crackmeters were installed later.

Figures 5.2 and 5.3 indicate that the slab movements were largest near the ends of the CRCP section while the movements at other locations were negligible, indicating that not all the pavement length contributed to the movements at the end of the pavement. It was observed that the daily and seasonal movement rates at the west end (Location 1) were 0.56 mils/°F and 9.69 mils/°F, respectively. Figure 5.4 shows the distribution of seasonal movement rate along the distance from the mid-point of the CRCP section. The seasonal movement rate was calculated based on measured data with crackmeters as shown in Figure 5.2. There is a large difference in the movement rate among the locations. The movement rate at the end of the pavement is much larger than those at other locations. This distribution is possibly due to the subbase friction. If the subbase friction was non-existent or quite small, the slab movements should be approximately proportional to the distance from the mid-point of the CRCP section. However, the subbase friction restrained the slab movement and resulted in the nonlinear distribution of slab movement. Figure 5.5 shows the locations of transverse cracks at the west end of CRCP. It is noted that crack spacing is large near the end of CRCP and decreases as the distance from the end increases. This presents strong evidence of a subbase friction effect. Near the end of CRCP, the cumulative subbase friction is small, and the slab is relatively free to move, resulting in less stress and fewer cracks. On the other hand, as the distance from the CRCP end increases, the crack spacing decreases. The information in Figures 5.4 and 5.5 provides a strong evidence of a significant role of friction on slab movement and crack development.

Air temperature was quite high in the summer of 2011. The widths of expansion joints were measured on August 17, 2011. The air temperature was 106 ºF. The joint widths varied from 0.90 to 1.25 inches in the west end, and from 1.25 to 1.50 inches in the east end (Figures 5.6 to 5.9). Since the initial joint width during the construction was about 1.5 inches, the slab expansions varied approximately from zero to 0.6 inches.

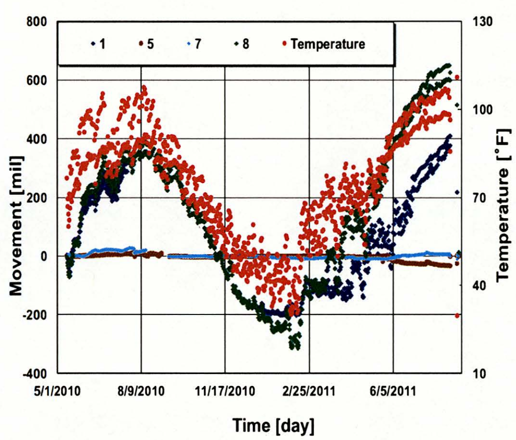


Figure 5.2 Concrete slab movement vs. concrete temperature for locations 1, 5, 7 and 8 (Refer to 4.17).

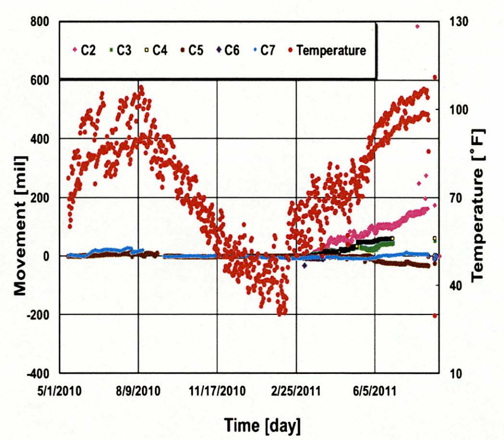


Figure 5.3 Concrete slab movement vs. concrete temperature for locations 2, 3, 4, 5, 6, and 7 (Refer to 4.17).

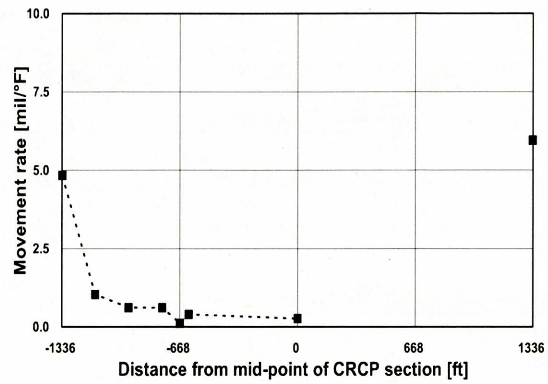


Figure 5.4 Distribution of seasonal movement rate at different locations

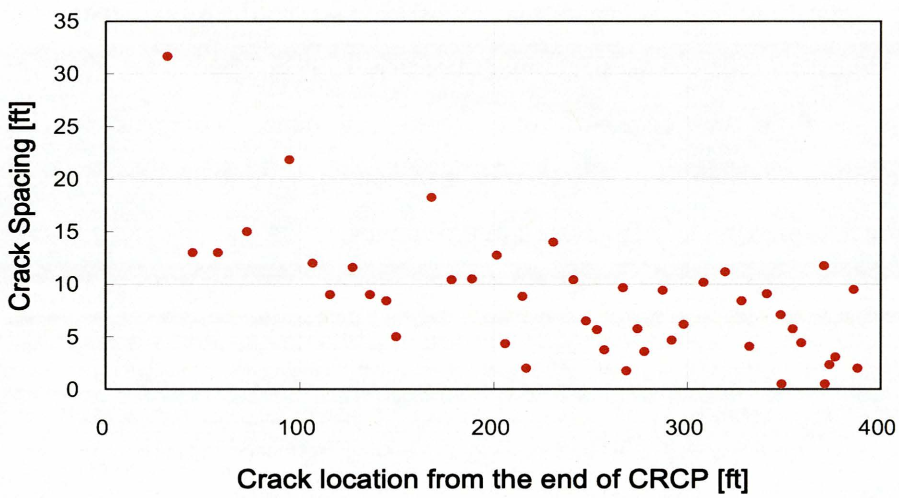


Figure 5.5 Crack spacing and crack location from the west end of the CRCP

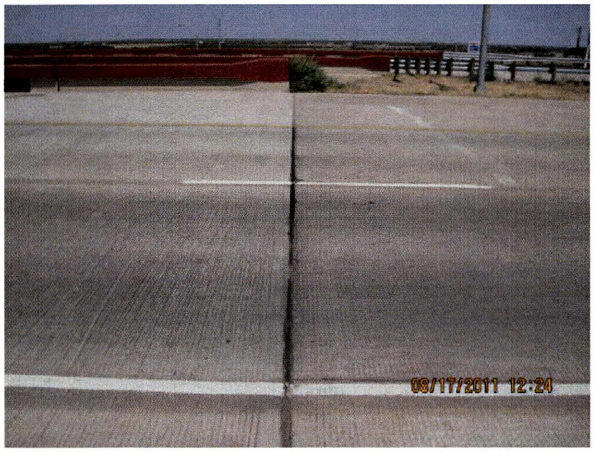


Figure 5.6 Expansion joint at the west end of the CRCP

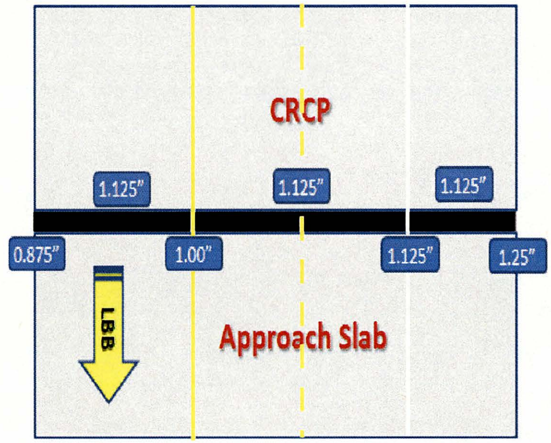


Figure 5.7 Joint widths at different locations of the expansion joint (West)

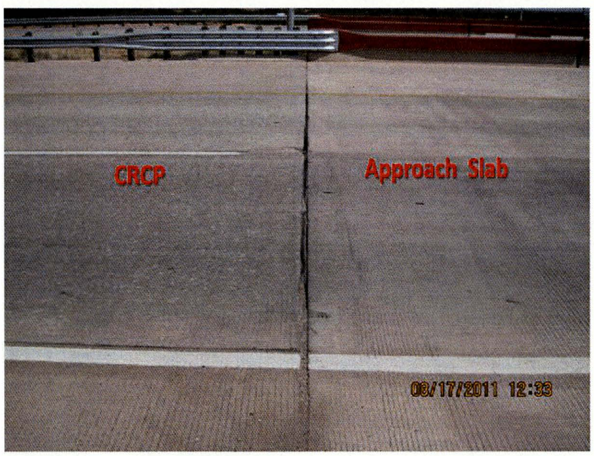


Figure 5.8 Expansion joint at the east end of the CRCP

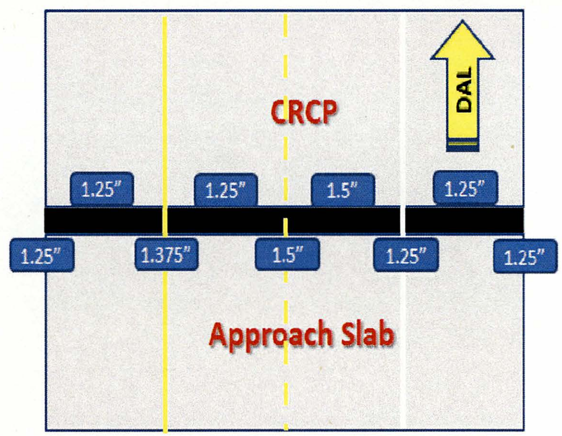


Figure 5.9 Joint widths at different locations of the expansion joint (East)

5.3 Linden (WF system)

Figure 5.10 presents the slab movements over time in the section in Linden, showing a good correlation between slab movement and temperature variations.

Figure 5.11 shows slab movements from the construction at a fixed temperature of 60 °F, indicating that the pavement contracted up to about two months. The data indicates the effect of drying shrinkage of concrete. For example, the slab contracted about 120 mils between the fifth day and 40th day.

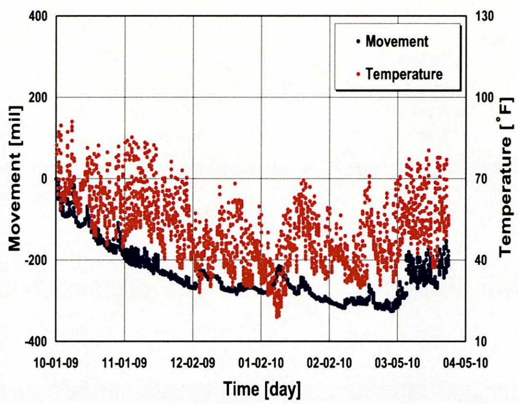


Figure 5.10 Concrete slab movement vs. concrete temperature

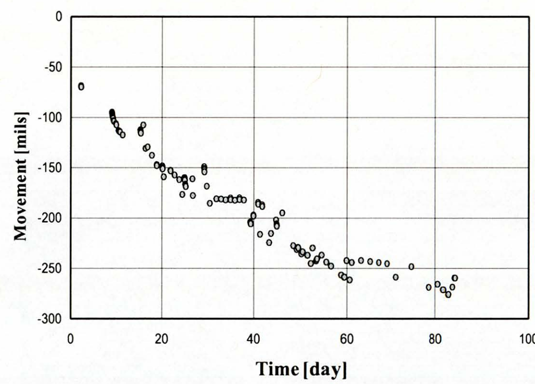


Figure 5.11 Drying shrinkage of concrete at 60 °F as fixed temperature

5.4 Houston (WF system)

This section on US 290 in Hempstead was built in 1995 and the slab movement was evaluated in 2007 and 2008. Figure 5.12 shows a good correlation between temperature and slab movement. The daily movement rate of the pavement was about 0.4 mils/°F. The rate of seasonal movement was much larger than the daily rate shown in Figure 5.13.

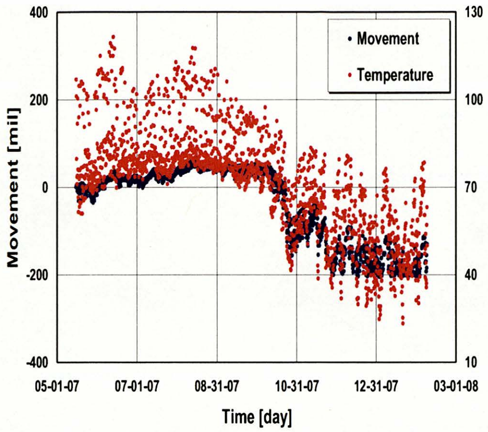


Figure 5.12 Slab movement vs. concrete temperature

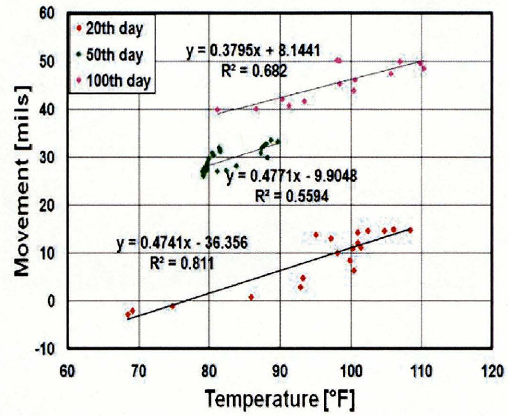


Figure 5.13 Daily movement rate of concrete slab

5.5 Texarkana (AL system)

The Texarkana section was built in 2004. Figure 5.14 shows change of the movement and temperature. The seasonal movement rate was observed to be 3.56 mils/°F (370 mils/ 104 °F). From Figure 5.15, the daily movement rate ranged between 0.5 to be 1.25 mils/°F. As in other sections, the seasonal movement rate was much higher than the daily movement rate. As shown in Table 5.1, the seasonal movement rate is larger than the other section (Texarkana Section) with an AL system. The Texarkana section was about seven years old during the time of this study, while other sections were new. This might indicate the diminishing effectiveness of the AL system with time.

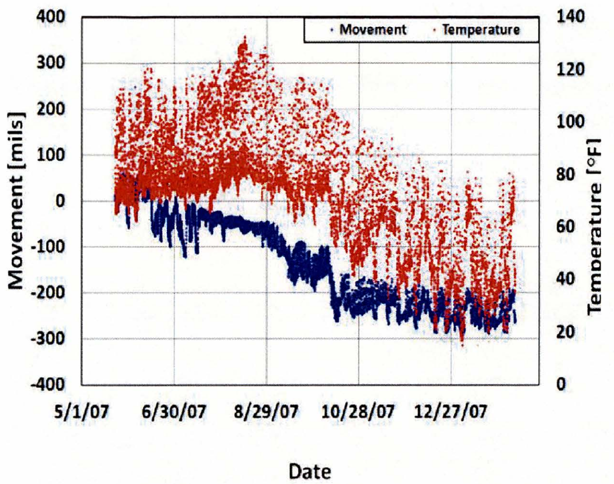


Figure 5.14 Slab movement vs. concrete temperature

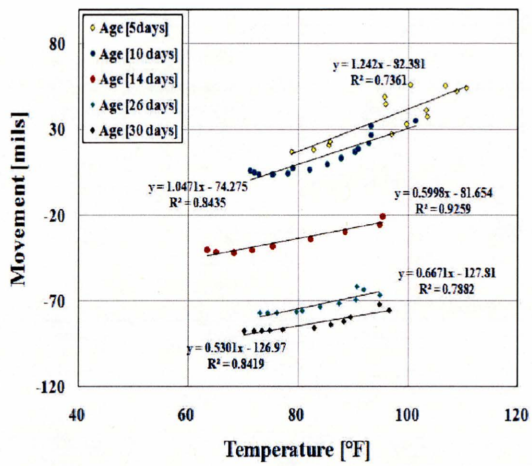


Figure 5.15 Daily movement rate of concrete slab at different ages

5.6 Lubbock 1 (AL system)

Figure 5.16 shows that there is a good correlation between slab movement and temperature. Unfortunately, the crackmeter in this section stopped working six months after installation. Figure 5.17 shows a daily movement rate of 0.33 mils/°F at different ages after the construction of the pavement slab. Figure 5.18 shows the slab movement at a constant temperature of 60 °F. The concrete shrank about 23 mils until 60 days after construction. After 60 days, at the same temperature the concrete expanded slightly, indicating that the effect of drying shrinkage to the concrete was completed in 60 days. Anchor lugs appear to restrain the concrete movement fairly well as a drying shrinkage of 20 mils was observed within 60 days, compared to drying shrinkage of 250 mils in the Linden section (Figure 5.11). It appears that AL system restrains slab movements effectively at the early age, probably because of good contact between lug walls and embankment materials, and possibly more transverse cracks in pavements near the anchor lug system.

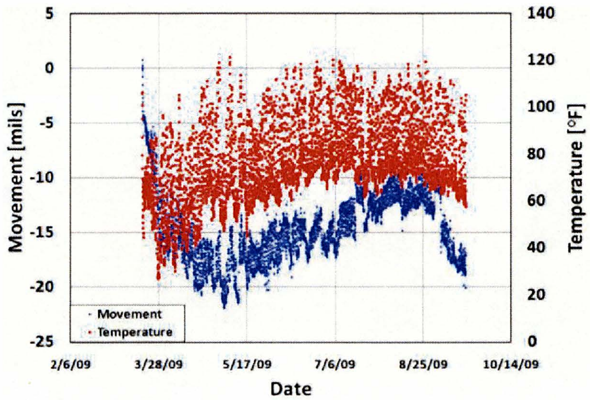


Figure 5.16 Concrete slab movement vs. concrete temperature

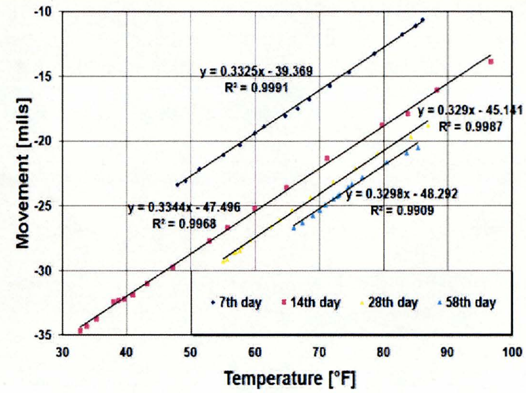


Figure 5.17 Daily rate of movement of the concrete slab at different ages of concrete

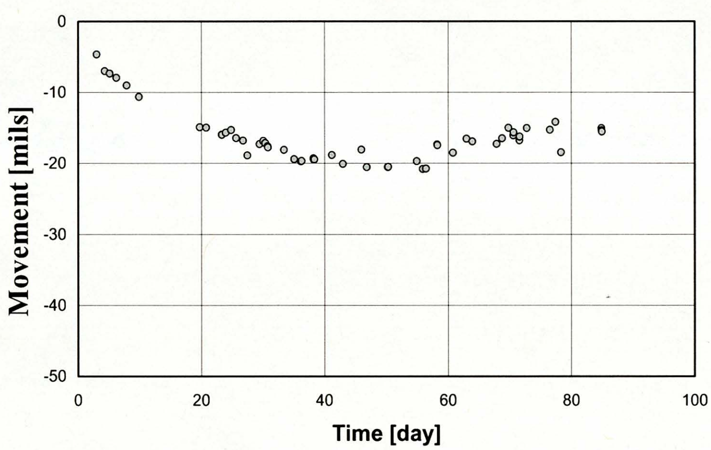


Figure 5.18 Drying shrinkage of concrete slab after construction at 60 °F fixed temperature

5.7 Lubbock 2 (AL system)

Figure 5.19 shows the location of gages in AL1 and AL4 in the Lubbock 2 section. Figure 5.20 shows the slab movements measured with crackmeters installed in AL1 and AL4. The movements are, in general, quite small. The movements at AL1 are larger than those at AL4, probably because AL1 is the free end of the pavement and is free to move, and the long CRCP section restrains the movement at AL4. The daily movement rate at AL1 was 0.06 mils/°F and the seasonal movement rate was 0.52 mils/°F, which is quite small when compared to the other sites, indicating that immediately after concrete placement, the anchor lugs restrain the movements well. It is not clear if the lugs will behave as effectively after some years. Figure 5.21 shows a continually increasing daily movement rate over one year. The rate became more than doubled that at 1.5 months after the pavement placement. This might indicate the diminished effectiveness of AL system over time in restraining slab movement.

To evaluate the lug behavior due to temperature variations, concrete strains at different depths of the lug were investigated. Figure 5.22 shows the concrete strains measured in AL1. As the temperature decreased continuously, the strains also decreased at all locations. Restraining CRCP slab movement would produce bending behavior in anchor lugs and thus result in variations of strains at different depths. However, strain measurements from the gages installed in AL1 showed negligible differences among gages. Similar trend of strain variations was observed in AL4. The main lane was paved on September 30th 2011. The temperature variations before the concrete placement were larger. This is because, after the concrete placement, the gages are not open to the environment and are completely covered with concrete.

Figures 5.23 and 5.24 show the comparison of strains measured in AL1 with those in AL4. If anchor lugs restrain the slab movements, the restraining force would be larger in AL4 than in AL1, which would result in a difference in strain variations between AL1 and AL4. However, the measured data exhibits that the strain variations in vertical and longitudinal directions in AL1 were similar to those in AL4. Similar variations in concrete strains in AL1 and AL4 potentially indicate not all the concrete elements in the lugs resist CRCP movement.

Figure 5.25 shows the pressures measured from soil pressure cells. The pressures decreased with time and were in good correlation with the longitudinal concrete strains near the bottom level of AL4. Contrary to expectations, the variation of pressures measured with SPC-L (left side, Figure 5.19) was very close to that with SPC-R (right side, Figure 5.19). Pressures measured by (or registered in) both gages were quite small, indicating that the restraining force between anchor lugs and surrounding soils was not substantial. The summer during this study was very hot and dry. Due to this dry weather, the shrinkage of the soil might have been sufficient enough for the contact between the soil and the wall to be lost and the pre-pressures which were kept on plate of both cells were not enough to compensate for the shrinkage.

Analysis of AL4 gages L1 and R1 shown in Figures 5.26 and 5.27 show that the crack occurred through the gage R1 on 30th September 2011 just after the concrete placement. Before the crack occurred, the concrete strains in location R1 were near zero. When the crack occurred, the strain jumped from this minimum to 1,200 $\mu\varepsilon$ as seen in Figures 5.28 and 5.29.

Figure 5.30 shows the strain variations in L1 and R1 in AL4 (see Figure 5.26). First, R1 strain is much larger due to the crack. Second, until the end of January, the directions of the concrete strain between L1 and R1 are opposite. In other words, when the temperature goes down, the gage R1 through which a transverse crack occurred registered tensile strain as expected. However, L1 registered compressive strain, which can be only explained if the lugs are moving, potentially the third lug moving towards AL4 (Figure 5.31). On Feb 3, 2011, the temperature in Lubbock went down to 20 °F. After that, both L1 and R1 strains moved in the same directions. This is quite possible when AL4 actually moves due to the pulling from CRCP (Figure 5.32).

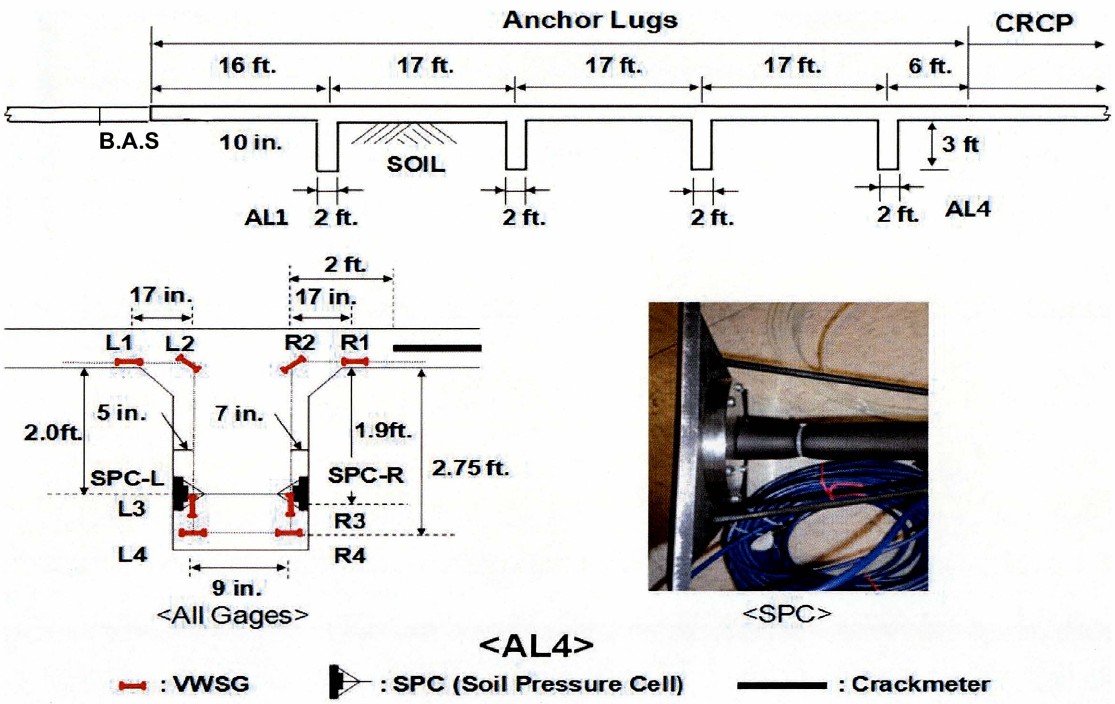


Figure 5.19 Location of all gages in anchor lugs 1 and 4

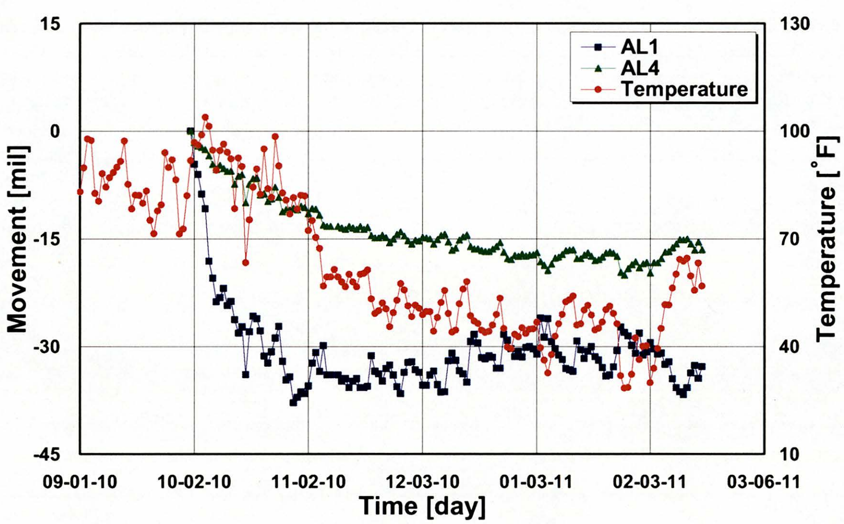


Figure 5.20 Comparison of AL1 movement with AL4 movement from the crackmeters

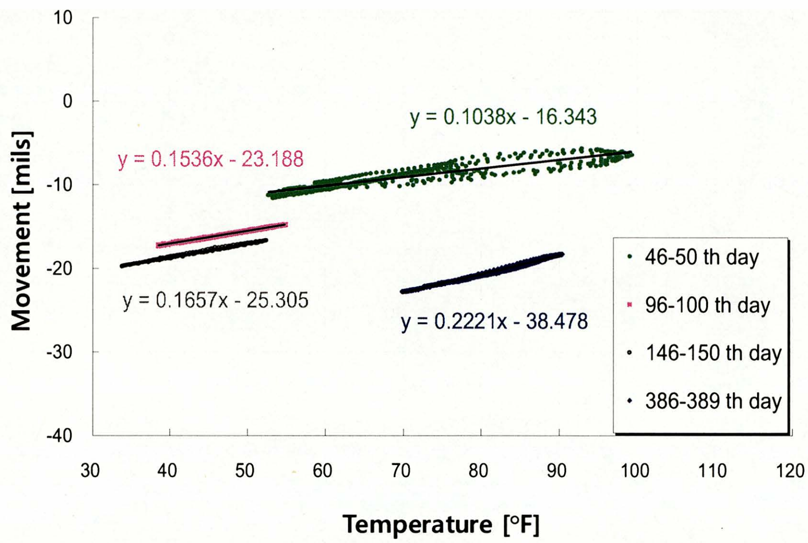


Figure 5.21 Movement rate of concrete at different ages at AL1

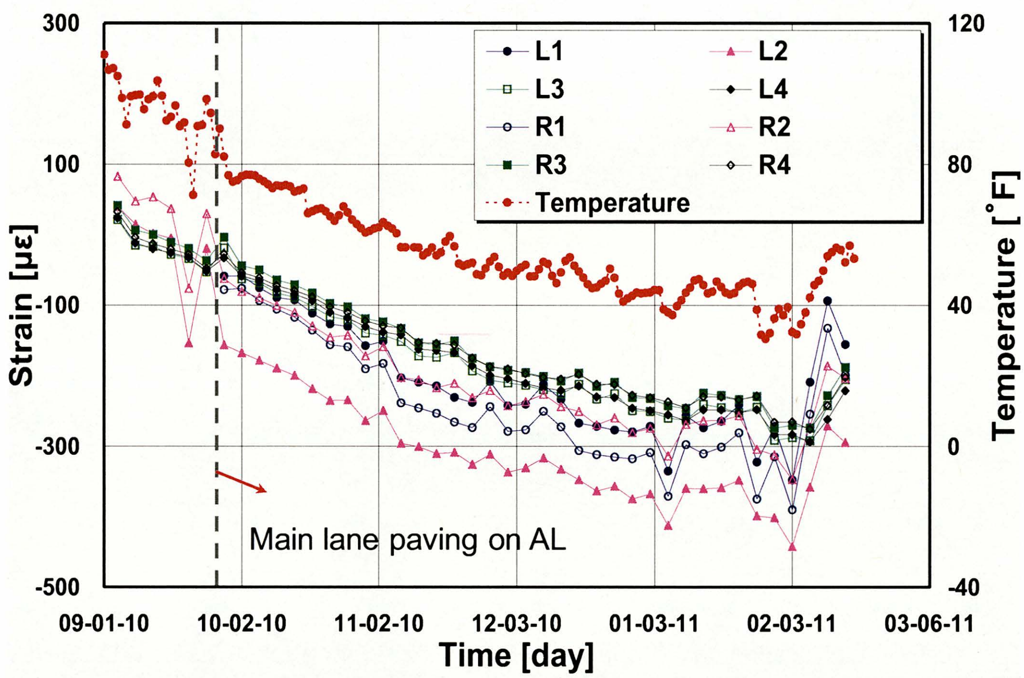


Figure 5.22 Concrete strains at different depths in AL1

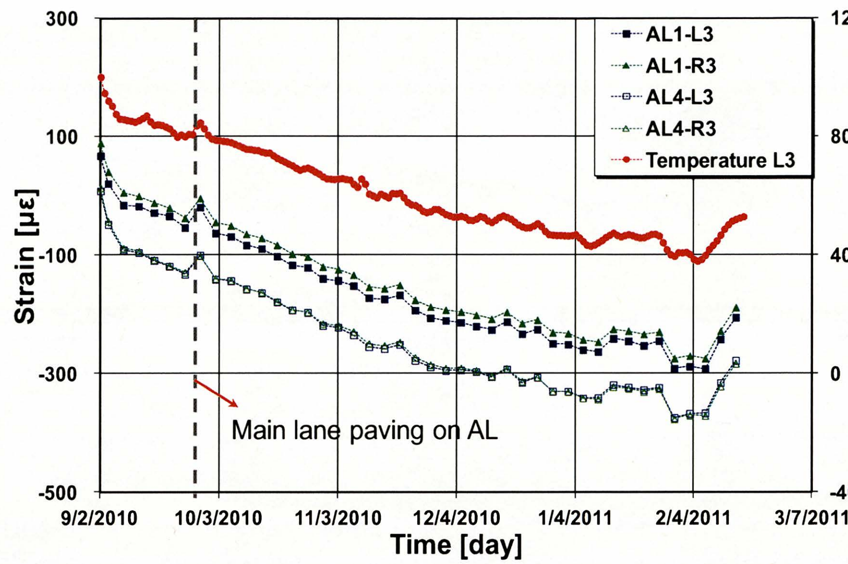


Figure 5.23 Comparison of AL1 behavior with AL4 behavior – vertical strain

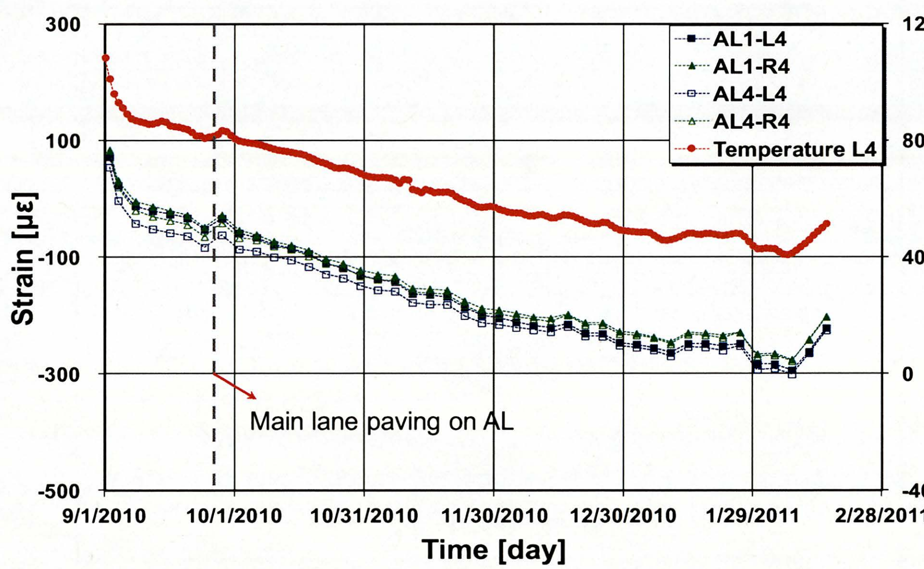


Figure 5.24 Comparison of AL1 behavior with AL4 behavior – longitudinal strain

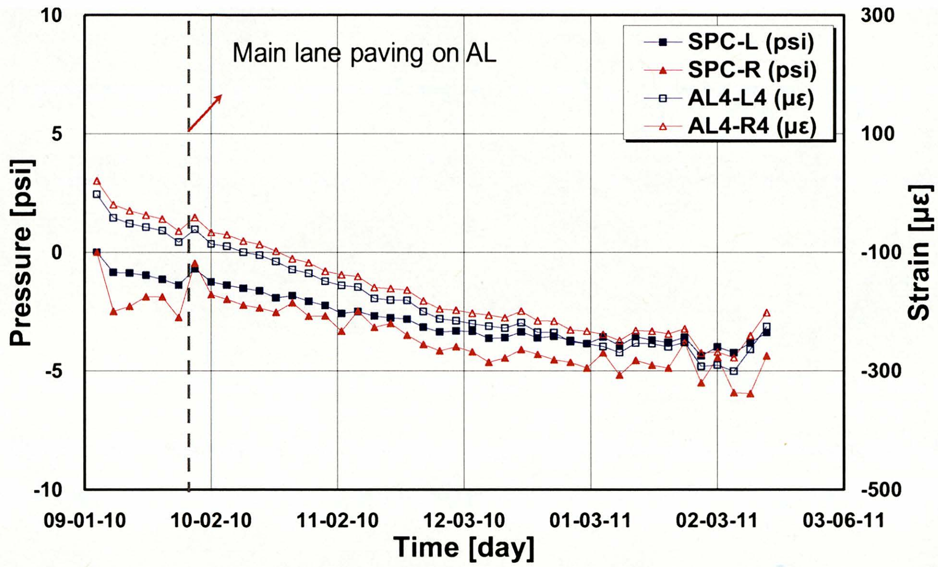


Figure 5.25 Correlation of soil pressure with longitudinal concrete strains

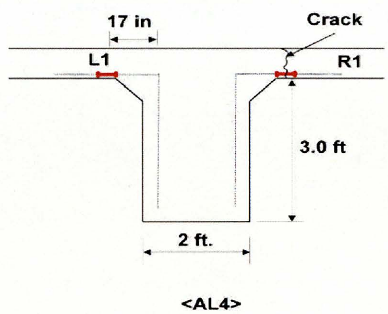


Figure 5.26 Location of L1, R1 and the crack in AL4

Figure 5.27 Location of the crack in AL4 at R1

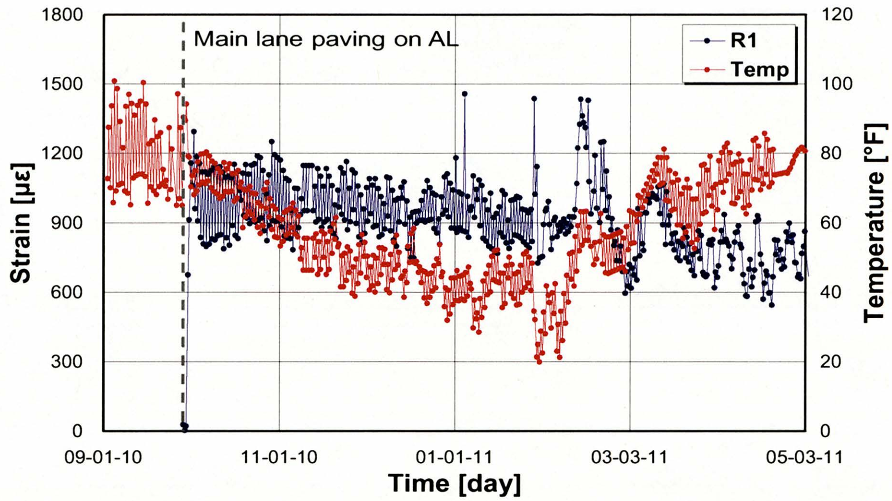


Figure 5.28 R1 movement vs. concrete temperature

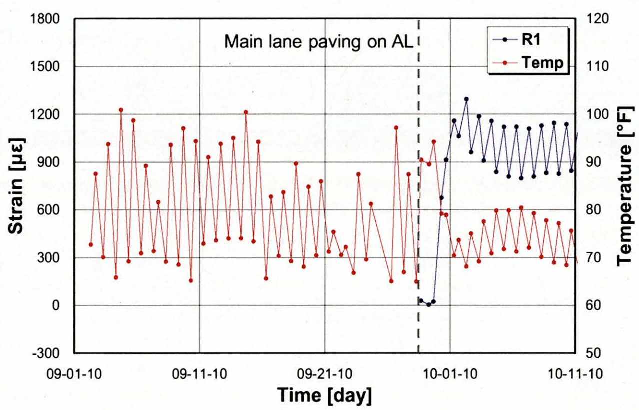


Figure 5.29 R1 movement vs. concrete temperature at the beginning of concrete placement

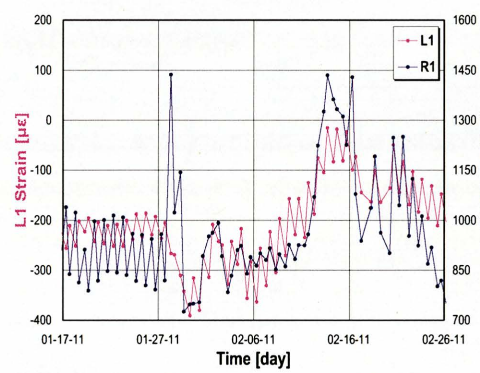


Figure 5.30 Jan. to Feb. L1 and R1 strain comparison

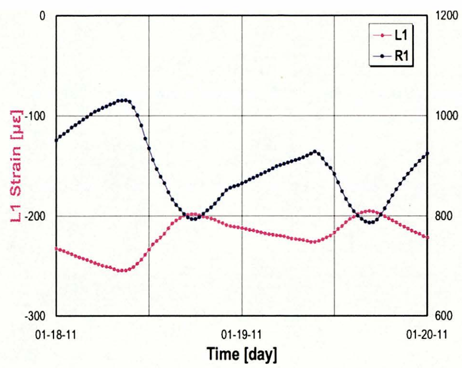


Figure 5.31 L1 and R1 strains prior to cold temperature

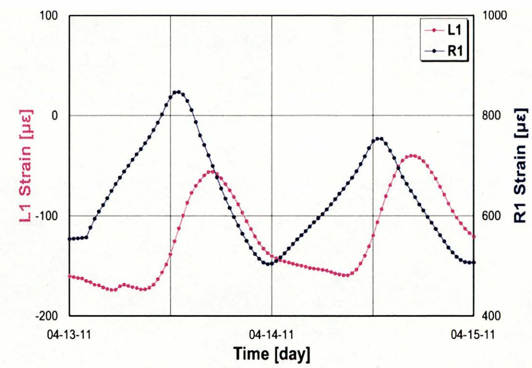


Figure 5.32 L1 and R1 strain after cold temperature

5.8 Waco (Cast-in-Place Post-Tensioned Concrete Pavement)

In this project, to minimize subbase friction and prestress loss, two layers of polyethylene plastic sheet were placed over the asphalt stabilized subbase. The slab length was 300 ft. At the free edge of the slab, five crackmeters were installed at different distances from the slab center as shown in Figure 5.33. Figures 5.34 and 5.35 present longitudinal slab movements at different distances from the slab center. These figures show that the movements correlate well with the temperature. The longitudinal slab movements became larger with the increased distance from the center of the slab. Figure 5.36 shows that the rate of daily movement at the end of the slab ranged from 5.6 to 6.8 mils/°F. This level of daily rate is almost 10 times larger than those at Linden and Wichita Falls, even though the slab lengths in Linden and Wichita Falls are much larger. This difference in the rate of daily slab movements signifies the effect of subbase friction, since polyethylene sheeting was placed under the concrete slab in this section, whereas an asphalt stabilized base was used in Linden and Wichita Falls.

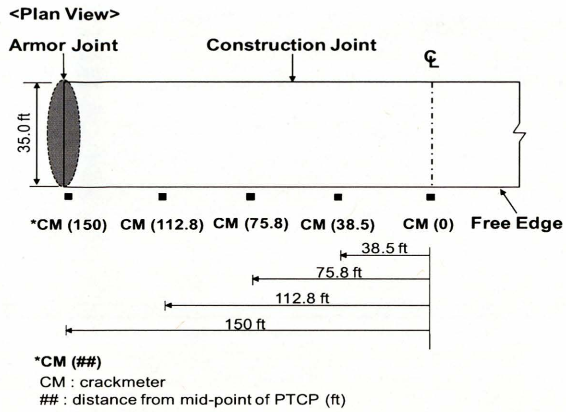


Figure 5.33 Location of the gages

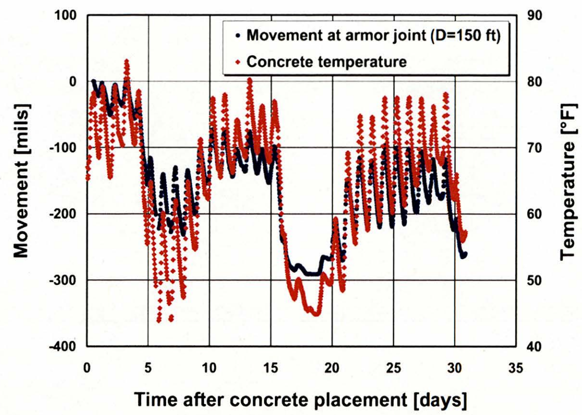


Figure 5.34 Concrete slab movement vs. concrete temperature

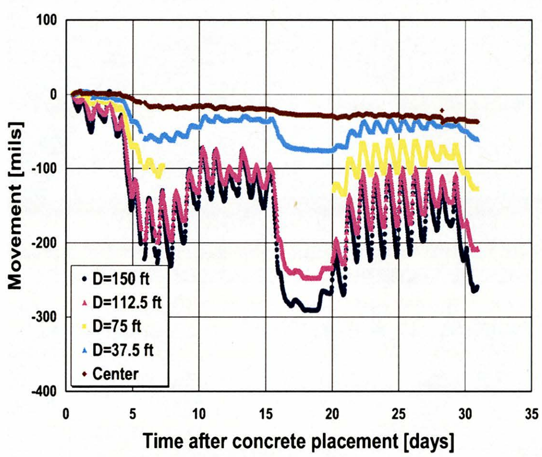


Figure 5.35 Slab movement vs. concrete temperature at distances from center of slab

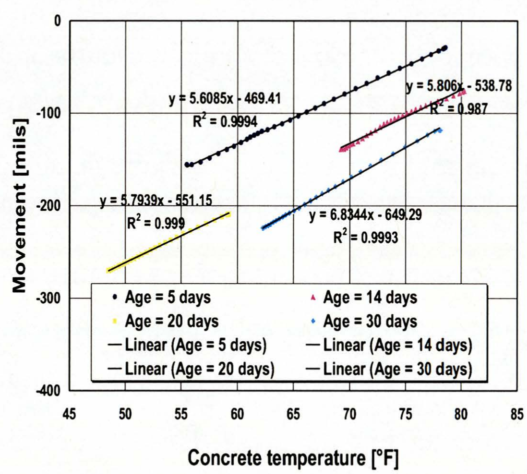


Figure 5.36 Daily movement rate of the concrete slab at different ages of the concrete

Table 5.1 Summary results of Test Sections

Test Section location	Terminal Type	Coarse Aggregate Type	Length of Pavement, miles	Subbase Type	Cons-truction Year	Daily Mvt Rate (mil/°F)	Seasonal Mvt Rate (mil/°F)
El Paso	EJ	LS[a]	0.80	3 in. ASB[c]	2009	0.56	9.59
Wichita Falls	EJ	LS	0.50	3 in. ASB	2009	0.56	9.69
Linden	WF	SRG[b]	> 1.00	4 in. ASB	2009	0.51	4.53
Houston	WF	LS	1.05	1 in. ASB + 6 in. CSB[d]	1995	0.40	2.63
Texarkana	AL	SRG	0.45	4 in. ASB	2004	0.61	3.56
Lubbock1	AL	SRG	0.45	4 in. ASB	2009	0.33	*
Lubbock2	AL	LS	0.45	1 in. ASB + 6 in. CSB[d]	2010	0.07	0.51
Waco (PTCP)	Armor Joint	LS	0.06	4 in. ASB	2008	6.83	*

[a]: limestone; [b]: siliceous river gravel; [c]: asphalt stabilized base, [g] : Cement stabilized base

* No data available for seasonal movement rate measurement

5.9 Further discussion

From the analysis results shown in Table 5.1, it is observed that among the three AL sections, the seasonal movement rate at the Texarkana section is much larger than the other two sections in Lubbock. The reason could be that the sections in Lubbock are less than two years old, whereas Texarkana section is about seven years old. It could be that, in the Texarkana section, continued slab movements over seven years created voids between anchor lug vertical walls and surrounding soil, resulting in diminished resistance of AL system against slab movements. Regardless of the terminal types, the end movement of CRCP sections was not large enough to cause damage to the bridge structures.

Figure 5.37 shows an example of a 10-inch thick CRCP with a five-lug AL system. If the CRCP exerts horizontal compressive stress, the force (F_H) exerted on the anchor lugs will be the product of this stress and the area of the pavement. For example, a horizontal compressive concrete stress of 500 psi due to temperature increase or other causes would result in 5,000 lbs of force (500 psi x 10 in x 1 in (unit width)). This force is resisted by the compressive stresses in the soil at the left side of the anchor lugs as shown in Figure 5.37. The total surface area of the left side of anchor lugs is 180 sq. in. (5 x 36 in x 1 in). Recall that the height of the lug is 3 ft. Accordingly, the compressive stress in the soil at lug walls is 28 psi (5,000 lbs/180 sq. in.) Table 5.2 shows soil stress values corresponding to various concrete compressive stresses.

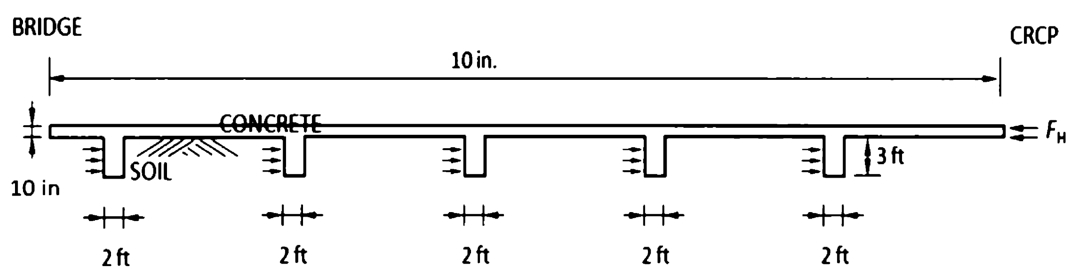


Figure 5.37 Example of CRCP with AL system

Table 5.2: Soil restraint stresses at lug walls for different stresses in concrete

Concrete Compressive Stress (psi)	Stress in Soil at Lug Wall (psi)
500	28
1000	56
2000	111

The stresses generated in soil due to slab expansion at lug wall as shown in Table 5.2 are large enough to result in permanent deformations in embankment soils adjacent to the lug walls. As a result, when the lug moves back due to pavement contraction, the soil does not retract with it, but rather creates a void between the lug wall and the soil. These plastic deformations result in voids between soil and lug walls as observed in Illinois, diminishing the effectiveness of AL system restraining slab movements.

Chapter 6: Conclusions and Recommendations

In this research study, the effectiveness of three terminal systems – expansion joint (EJ), wide-flange (WF), and anchor lug (AL) – was evaluated by field measurements of slab movements in CRCP. The issue of damage to bridge structures due to CRCP expansions is quite complicated in nature, and not all the factors were included in this study. Also, the damage to the bridge structures from CRCP expansions is a long-term process, and it was not feasible to investigate this long-term process in this study. Due to the complex nature of the mechanisms and factors related to this issue, not all the factors were included in this study and the resulting limitations need to be stated in the findings of this study:

1. The findings made in this study are applicable to CRCP only.

2. All the test sections selected for field testing in this study had only one subbase type – asphalt stabilized base. The findings in this study cannot be applicable to CRCP systems with different subbase types, such as granular, cement stabilized, or lean concrete bases.

3. The duration of slab movement observations was limited to less than three years, with most sections observed for 20 months. To investigate the slab movements of CRCP more comprehensively, observation periods should be extended.

4. All the sections included in this investigation were in rather flat terrain and the effect of longitudinal slope of CRCP on slab movements was not investigated.

5. Slab movement behavior of CRCP is quite complicated primarily because of the numerous factors involved – subbase friction, creep of concrete, and transverse cracking. No attempts were made to quantify the relationships between these factors and slab movements.

6. There could be several mechanisms for the expansion of CRCP. In this study, the CRCP expansions due to temperature variations only were investigated. Other causes were not

studied.

With these limitations stated, the findings made in this study can be summarized as follows:

1) It appears that slab movements at the end of CRCP are substantially restrained by subbase friction.

2) Considering CRCP systems with asphalt base typically used in Texas, the slab length that significantly contributes to the slab movements is limited to not more than few hundred feet from the end of CRCP.

3) Seasonal slab movement rates due to temperature variations are larger than daily slab movement rates. However, seasonal slab movements at the end of CRCP are not excessive.

4) Among the three terminal systems, seasonal slab movement rates were the highest in expansion joint systems and lowest in anchor lug systems. This is probably because the AL system effectively restrains movement at the early ages.

5) Slab movements at the end of CRCP can be accommodated by EJ systems with subbase friction that can be achieved with typical asphalt stabilized base. The use of AL systems is not needed to restrain concrete slab movements. The benefits of WF and AL systems are doubtful considering the higher cost of those systems compared with the cost of EJ systems.

Even though the investigation made in this study is not comprehensive, the findings indicate that of the three terminal systems studied, expansion joint system is the most cost-effective. AL and WF systems might not provide the benefits they are currently perceived to have. It is recommended that TxDOT utilize EJ systems as a bridge terminal system. Design standards for EJ systems are included in Appendix A.

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APPENDIX A

Guidelines for the Rational Use of Terminal Systems and Design Standards

It is a common practice to install terminal systems between bridge approach slabs and the end of Portland cement concrete (PCC) pavement. The objective of the terminal systems is to protect bridge structures from damage due to PCC pavement expansions, even though the exact mechanisms of PCC pavement expansions have not been fully understood.

There are two types of PCC pavement used in Texas. One is jointed plain concrete pavement (concrete pavement contraction design (CPCD)) and the other is continuously reinforced concrete pavement (CRCP). These two pavement types address concrete volume changes and resulting cracking potential in different ways. In CPCD, transverse contraction joints are provided at every 15-ft and concrete volume changes due to temperature and moisture variations are allowed. Accordingly, the stresses in concrete due to temperature and moisture variations are maintained at a low level and the probability of cracking becomes quite small. On the other hand, volume changes in concrete due to temperature and moisture variations are severely restrained in CRCP by longitudinal reinforcing steel, resulting in numerous transverse cracks. General consensus among practitioners is that slab expansions in CPCD is not large enough to cause damage to bridge structures, whereas the slab expansions in CRCP could be large enough to cause damage to bridge structures and need to be controlled with a proper terminal system. This document provides guidelines on a rational terminal system for CRCP, even though it could be used for CPCD.

There could be various causes for CRCP expansions. In this research study, CRCP expansions due to only temperature increase were investigated. Accordingly, the findings from this study and resulting recommended terminal system are not applicable to CRCP expansions due to causes other than temperature increase.

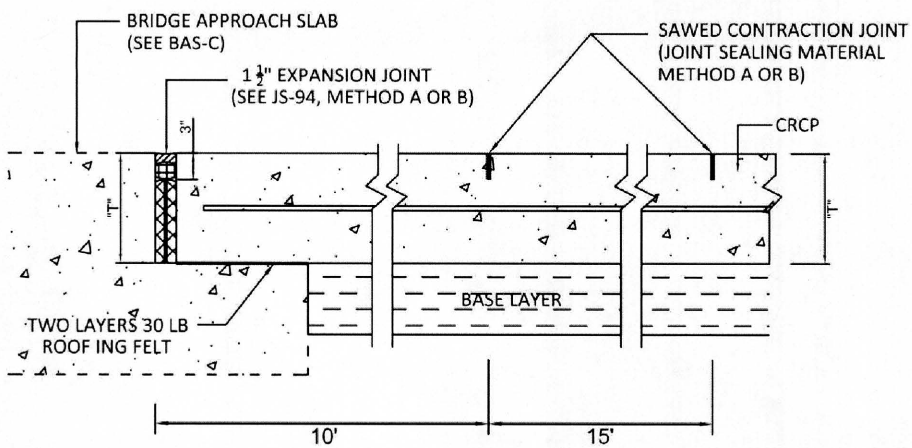
Traditionally, three different terminal systems have been in use in TxDOT. They are the expansion joint (EJ) system, the wide flange (WF) system, and the anchor lug (AL) system. Expansion joint and wide flange systems operate on the assumption that slab expansions are not excessive and can be accommodated by providing a 1.5-in expansion joint. On the other hand, anchor lug systems operate on the different assumption that slab expansions could be large and need to be restrained. These two assumptions are not compatible. If CRCP expansions are more than 1.5-in, which is the width of an expansion joint for EJ and WF systems, damage could result to the bridge structures with EJ or WF systems. On the other hand, if CRCP expansions are limited to less than 1.5-in, the use of AL system will be an unwise use of valuable financial resources. In order to identify a rational terminal system, it is important to estimate the potential maximum CRCP slab expansions at the end of the pavement.

Research findings from this study can be summarized as follows:

1) With the hot mix asphalt layer typically used in Texas under the concrete slab, CRCP expansions due to temperature variations are not excessive. For a seasonal temperature increase from winter to summer of 100 °F, CRCP expansions of about 1-in are expected. Even when the CRCP is constructed in the winter, providing 1.5-in joint width would have 50 percent reserve.
2) Simple one-dimensional analysis shows that AL systems with five lugs could restrain CRCP expansions equivalent to 15 °F temperature increase. With a greater than 15 °F temperature increase, embankment materials will begin yielding, potentially creating voids between lug walls and embankment materials and reducing long-term effectiveness.

The WF system would provide performance comparable to the EJ system. Since the installation of the WF system is more complicated than the EJ system and the cost of the WF system is higher than that of the EJ system, the EJ system with 1.5-in expansion joint width is recommended for a terminal system.

The use of AL system might be beneficial in preventing the sliding of CRCP on a steep slope. However, the effectiveness of the AL system on controlling the sliding of CRCP was not investigated in this study. For the design of CRCP on a long, steep slope, contact the Rigid Pavement and Concrete Materials Branch at the Construction Division.



"T" = CRCP SLAB THICKNESS

GENERAL NOTES

1. THE BRIDGE APPROACH SLAB DETAILS ARE AS SHOWN ELSEWHERE IN THE PLANS.
2. THE LOCATION OF THE TERMINAL EXPANSION JOINT SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
3. DETAILS AS TO PAVEMENT SLAB WIDTH, THICKNESS, AND CROWN CROSS-SLOPE SHALL BE AS SHOWN ELSEWHERE ON THE PLANS.
4. DETAILS OF LONGITUDINAL JOINTS (TRANSVERSE JOINTS, IF NECESSARY) AND REQUIRED JOINT SEALS ARE SHOWN ON THE APPROPRIATE PAVING DETAIL SHEETS.
5. THE SAW CUT DEPTH FOR THE SAWED CONTRACTION JOINT SHALL BE ONE FOURTH OF THE SLAB THICKNESS.
6. LONGITUDINAL AND TRANSVERSE REINFORCEMENT SHALL BE IN ACCORDANCE WITH GOVERNING CRCP DESIGN STANDARDS.

Texas Department of Transportation
Construction Division (Pavements)

TERMINAL EXPANSION
JOINT FOR
CONCRETE PAVEMENT

TEJ (CP) - 12

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APPENDIX B

SPECIAL PROVISION

368

Concrete Pavement Terminals

For this project, Item 368, "Concrete Pavement Terminals," of the Standard Specifications, is hereby amended with respect to the clauses cited below, and no other clauses or requirements of this item are waived or changed hereby.

Article 368.5. Payment. The first paragraph is voided and replaced with the following:

The work performed and the materials furnished in accordance with this Item and measured as specified under "Measurement" will be paid for at the unit price bid for "Wide Flange Pavement Terminals," "Anchor Lugs Pavement Terminals" or "Junction Transition Terminals,"

APPENDIX C

Guidance for Determining the Most Efficient Uses of Terminal Systems

At the beginning of the research study, it was assumed that (1) CRCP expansions due to temperature variations could be proportional to the pavement lengths between bridges, and (2) the resistance by an anchor lug system could restrain CRCP expansions substantially enough to prevent damage to bridge structures. During the course of research, it was determined that these assumptions were not correct for the pavement systems typically used at TxDOT, which includes the use of asphalt stabilized layer under the concrete slab.

The length of the CRCP that contributes to the expansions of the pavement was limited to a few hundred feet from the end of the pavement section. The movements of CRCP located beyond a few hundred feet from the end of the pavement were negligible. It was construed that, as long as the CRCP section is longer than few hundred ft between bridges, the CRCP expansions would be similar. This finding reveals that the number of anchor lugs needed should not depend on the CRCP section length. If the anchor lug system is used to prevent or minimize the slab sliding potential when CRCP is constructed at a long, steep slope, the number of lugs needed should be related to the length and the slope of the CRCP section. The number of lugs needed versus the length and slope of CRCP was not investigated in this study. It is considered that projects where the anchor lug system is needed to prevent CRCP sliding due to the length and slope of the pavement are quite rare in Texas. If situations arise where this is the case, design engineers are encouraged to contact the Rigid Pavements and Concrete Materials Branch of CSTMP in Austin for guidance.

Structural analysis of an anchor lug system revealed that the resistance from the lugs against CRCP movements is quite minimal, due to the low strength capacity of embankment materials compared with that of Portland cement concrete. Field observations confirm the inability of anchor lug systems to resist CRCP slab movements.

Based on the findings of this study, the use of anchor lug systems is not recommended to protect bridge structures from CRCP expansions due to temperature increase. The use of expansion joint system is recommended because of the system's lower cost, simple construction, and potentially lower maintenance cost.

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