Continuously Reinforced Bonded Concrete Overlay of Distressed Jointed Concrete Pavements

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Texas Department of Transportation

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16. Abstract

There have been distresses and performance issues in jointed plain concrete pavement (JCP) in Sherman, Paris District. The pavement, 10 in. slab on 6 in. flexible base and lime treated subgrade, was built in 1984. Major distresses include transverse cracking and faulting in longitudinal joint between outside lane and outside shoulder. Forensic investigations revealed that the subbase support was neither uniform nor adequate. Substantial repair work has been done; however, the distresses continued to develop. Currently, there are no optimum rehabilitation strategies available for deteriorated JCP. To evaluate whether continuously reinforced concrete pavement (CRCP) bonded overlay can provide an optimum rehabilitation strategy, CRCP bonded overlay was constructed on half a mile section of JCP on US 75 southbound from Exit 64. Slab thickness and longitudinal steel reinforcement were 7 in. and 0.7 %, respectively. To optimize the performance of this rehabilitation method, the following requirements are incorporated in the specifications and design details; (1) maximum coefficient of thermal expansion (CTE) of concrete less than 4.6 microstrain/$^{\circ}$F, (2) wet-mat curing for at least 7 days, (3) the placement of longitudinal steel 4 in. from the concrete surface, (4) placement of non-woven fabric on transverse contraction joints, (5) installation of hook bars at the interface between old and new concrete at transition areas. Various gages were installed to investigate the behavior of CRCP overlay system due to environmental loading. Also, deflection testing was conducted before and after overlay with falling weight deflectometer (FWD). In-situ drying shrinkage of concrete was quite small, and in-situ CTE was close to the specification value. Deflection testing conducted before the overlay showed that many transverse joints had low load transfer efficiency (LTE) even though dowels were used. After the overlay, the deflections were reduced substantially, with the average of 1.02 mils. This deflection is comparable the deflections in 14 or 15 in.-thick normal CRCP. It is expected that this section will provide good long-term performance, and future performance will be monitored under TxDOT rigid pavement database project 0-6274.

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CONTINUOUSLY REINFORCED BONDED CONCRETE OVERLAY OF DISTRESSED JOINTED CONCRETE PAVEMENTS

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PRODUCTS
This report contains special provision to Item 360 "Concrete Pavement" and Item 421 "Hydraulic Cement Concrete" in Appendix D.

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Chapter 1: Introduction

1.1   Background

As of 2009, the Texas Department of Transportation (TxDOT) had about 4,000 lane miles of plain jointed concrete pavement (JCP or CPCD) in service.[1] Many of these pavements were built before TxDOT's policy of utilizing continuously reinforced concrete pavement (CRCP) as a preferred pavement type was put in place in 2002. Many miles of CPCD are approaching or have already exceeded their design lives and require rehabilitation. Several rehabilitation methods have been in use, including asphalt concrete (AC) overlay, crack and seat, rubblization, and removal and replacement. Among these rehabilitation methods, AC overlay has been the most widely utilized form of rehabilitation of CPCD in Texas, primarily due to the low cost of rehabilitation. On the other hand, AC overlay has the disadvantage of reflection cracking at existing transverse joints. Reflection cracking in AC overlays on JCP degrades the ride quality and often requires the removal and replacement of the AC layer within a few years. Crack and seat and rubblization with subsequent AC overlay are not always effective. Removal and replacement of JCP with another pavement type is quite expensive. A rehabilitation method for old JCP is needed that will provide a better long-term performance for a reasonable cost.

Findings from TxDOT research project 0-6274, "Project Level Performance Database for Rigid Pavements in Texas, Phase II" indicate superior performance of CRCP when the subbase support condition is adequate. Old deteriorated JCP could provide an excellent support to CRCP. CRCP bonded overlay on JCP could be a rehabilitation method for deteriorated JCP that will provide long-term good performance with a reasonable cost. The only unknown factor is whether reflection cracking will result at cracks and transverse joints. Even though longitudinal steel keeps transverse cracks in CRCP quite tight with no performance issues resulting from normal transverse cracks, it is not known whether the reflection cracking in CRCP will behave as normal transverse cracks in CRCP, which result from temperature and moisture variations in concrete. To prevent or retard reflection cracking, non-woven fabric could be placed at cracks and transverse joints in deteriorated JCP before CRCP overlay is placed.

This system of CRCP bonded overlay on deteriorated JCP with non-woven fabric at cracks and transverse joints was implemented on US75 in Sherman, Paris District. The half-mile long JCP section on US75 southbound that was selected for the rehabilitation is located just south of Exit 64. Figure 1.1 illustrates the limits of the JCP section in Sherman and Figure 1.2 shows the location of the CRCP overlay project. The section was completed in 1984. The roadway consists of 10-in JCP with 15-ft joint spacing on 6-in flexible base and lime treated subgrade. Dowels were used at transverse joints for load transfer. The section has two lanes, 4-ft inside and 10-ft outside shoulders. Originally, shoulders were 1-in AC layer on 8-in flexible base. In 1998, tied concrete shoulders were retrofitted with 10-in CPCD with matching 15-ft joint spacing on 6-in flexible base.

When the pavement design was developed in the early 1980s for this section, the average daily traffic (ADT) was 11,000 and 20-year projected ADT (in 2004) was 16,200. The pavement was designed with 20-year design life and the above projected traffic. Actual ADT in 2010 was 51,000, which is more than 3-fold the projected design traffic for 2004.

Since 2002, the TxDOT Sherman Area Office has spent on average between half a

million and $1M per year for routine maintenance of the JCP section. Repairs on this section require various lane closures for an average of 3 months per year. With the assumption that the same level of repairs will be needed for the next 20 years, the cost of maintenance is projected to be 10 to 20 million dollars, even without adjustments for inflation. Road user cost due to lane closures at the current ADT and project ADT for the next 20 years is estimated at over 70 million dollars. Considering the financial constraints TxDOT is under, this level of cost is not acceptable and better rehabilitation method that is cost-effective and that will provide a long-term good performance with minimal maintenance is required.

To investigate the viability of CRCP bonded overlay on deteriorated JCP as a rather long-term, almost maintenance-free rehabilitation method, an implementation project was initiated on US75 in Sherman.

1.2   Objectives

The primary objective of this project was to develop pavement design, material and construction specifications, and pavement design details. Another important objective was to evaluate the viability of CRCP bonded overlay on deteriorated JCP in providing a long-term, maintenance-free pavement system. Due to the short amount of time allowed in this implementation project, long-term performance cannot be evaluated. Rather, long-term performance will be monitored under the current TxDOT rigid pavement database project, 0-6274.

Figure 1.1 Limits of JCP project	Figure 1.2 CRCP overlay project limits

Chapter 2: Evaluation of Existing Pavement & CRCP Overlay Design

The development of CRCP overlay design requires the functional and structural evaluations of the existing JCP. Functional evaluations for this project were conducted by visual survey and structural evaluations by deflection testing with falling weight deflectometer (FWD). This chapter presents the results of those evaluations along with the overlay design. Since this rehabilitation project (bonded CRCP overlay on deteriorated JCP) is quite unique, other considerations were given to design, materials, and construction details that will enhance the performance of this rehabilitation project.

2.1 Visual Survey

Figure 2.1-(a) shows the overall condition of the JCP section that will be overlaid with CRCP.   Figure 2.1-(b) illustrates typical distress and repairs in JCP. There were numerous cracks and repairs in this section. The slabs were numbered from Exit 64, southbound. For example, the slab shown in Figure 2.1-(b) is 29th slab from Exit 64.

Figure 2.1-(a) Overview of JCP section	Figure 2.1-(b) Repair and distress in JCP
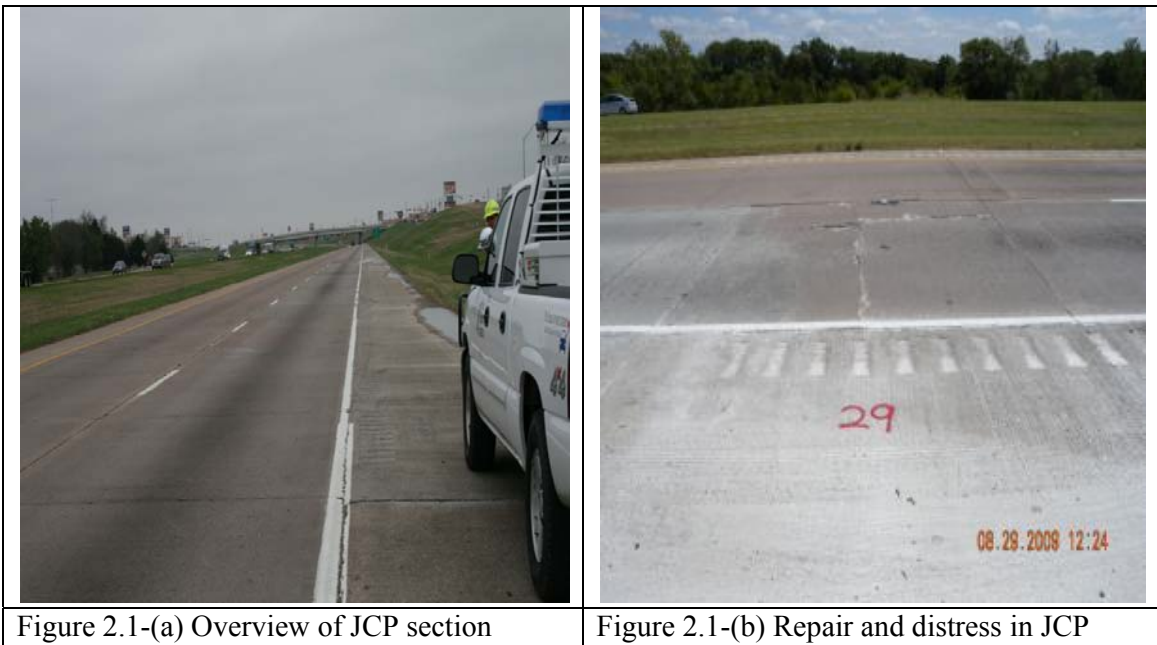
A detailed survey of cracks and repairs was conducted by TxDOT's CSTMP staff and is included in Appendix A.

2.2 Structural Evaluations

To evaluate the overall structural capacity and load transfer efficiency (LTE) at transverse joints and repair joints, deflection testing was conducted using FWD. All the slabs in the overlay project, from Slab #1 thru Slab #140, were evaluated. The testing was conducted at

three locations in each slab – upstream, middle of the slab, and downstream. If there were repairs, additional testing was conducted at the repaired transverse joints. Figure 2.2 illustrates the deflections in the first 10 slabs. In the x-axis, in SnU, S stands for Slab, n indicates the slab number, and U stands for Upstream (D is for Downstream). Where there were repair joints in the slabs, A or B is inserted between the slab number and the location of the drop (Upstream or Downstream). For example, S4AM indicates the drop at slab number 4, middle between upstream joint and first transverse repair joint. The crack map in Appendix A should be consulted when interpreting the deflection data. Figure 2.2 shows that there is a large variation in deflections, depending on the location of the FWD drops – smallest at the middle of the slab and largest at the upstream or downstream at transverse or repair joints. Two deflections with more than 20mils were both at repair joints, which indicates poor LTE at repair joints. The large deflections at transverse repair joints resulted most probably from poor tie bar insertion practice. Deflection information for the rest of the slabs is included in Appendix B.
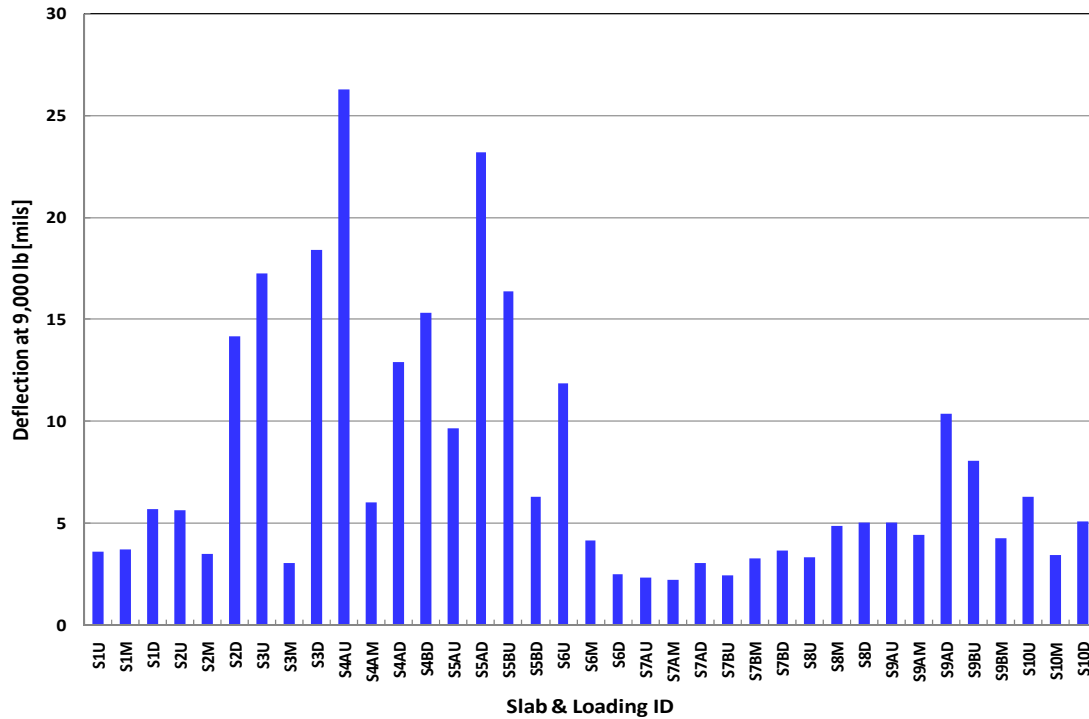
Figure 2.2 Slab deflections at various locations in Slab 1 thru Slab 10

LTE was calculated at both transverse and longitudinal joints using Equation 2.1. Here, d_{load} is the deflection under the loading plate (Sensor #1) and d_{unload} is the deflection of the Sensor #2 which is on the other side of the transverse joint in the upstream case. In the downstream case, d_{load} is the deflection under the loading plate (Sensor #1) and d_{unload} is the deflection of the Sensor #4 which is on the other side of the transverse joint and also at the other side of the loading plate.

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$$LTE(\%) = \frac{d_{unload}}{d_{load}} \times 100 \qquad\qquad (2.1)$$

where, d_{unload} : Deflection of unloaded slab (in)

d_{load} : Deflection of loaded slab (in)

Figure 2.3-(a) illustrates LTE at transverse contraction and repair joints. It shows that LTEs were low in many joints, even though dowels were used. The low LTEs are of concern because the potential for reflection cracking at transverse and repair joints will be more likely, unless things are done to prevent or minimize the reflection cracking. Figure 2.3-(b) shows LTE along the longitudinal joint between outside lane and outside shoulder, which was retrofitted in 1998.

Figure 2.3-(a) LTE at transverse/repair joints	Figure 2.3-(b) LTE at longitudinal joint
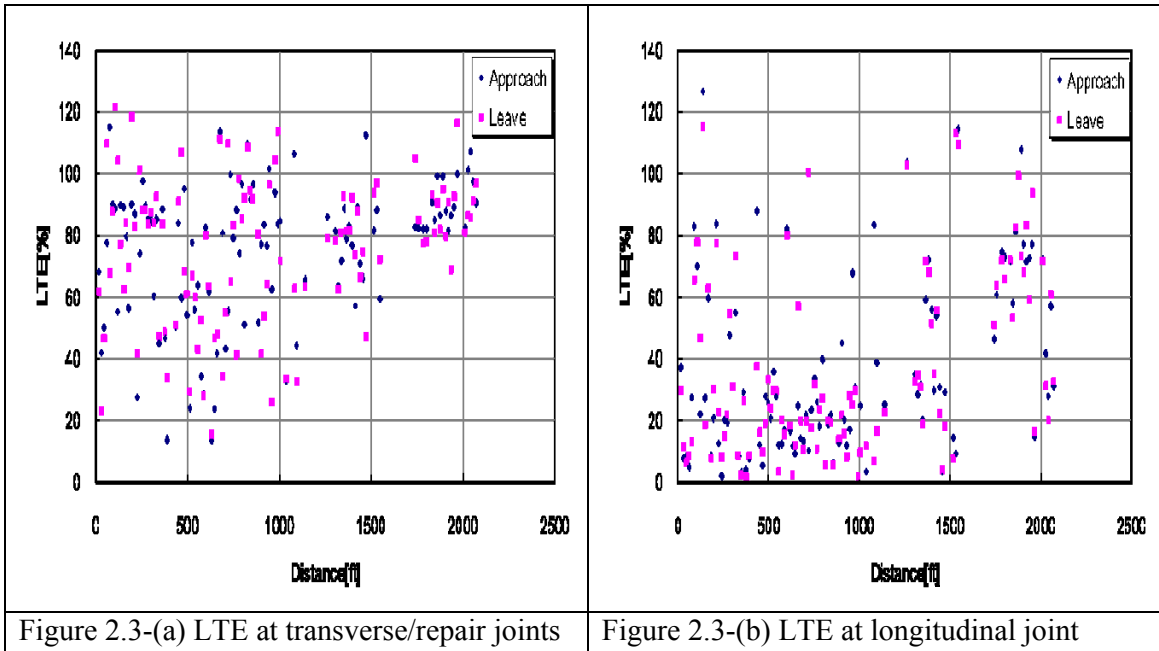
2.3 CRCP Design

2.3.1 Slab Thickness Design

Currently, there are no design procedures available for CRCP bonded overlay on JCP. From a purely mechanistic standpoint, the design should be based on the critical concrete stress that would result in punchouts in the overlaid pavement system. Finite element modeling of the overlaid system will be quite complicated due to the discontinuity and accurate assessment and modeling of interface at transverse joints. No effort was made in this project for modeling of the system for pavement design. Rather, the design procedures for bonded concrete overlay in the AASHTO 93 Guide were utilized.[2]

The inputs needed for bonded overlay in the AASHTO 93 Guide and the values selected for this project are:

1) Joints and cracks adjustment factor: 0.94 (from "Deteriorated transverse joints and

cracks per mile: 28")
2) Durability adjustment factor: 0.88
3) Fatigue adjustment factor: 0.94 (from "27 % Slabs cracked")

For the number of cracks used for the above adjustment factor, please refer to the crack map in Appendix A.

With the design traffic of 82 million ESALs for the next 30 years, the design thickness of CRCP if it is placed in the existing subbase would be 14.2-in. With the adjustment factors determined above, the effective slab thickness of the existing 10-in JCP becomes:

$D_{eff} = 0.94*0.88*0.94*10 = 7.8$-in.

Accordingly, the required slab thickness is $14.2 - 7.8 = 6.4$-in. Based on this estimation, 7-in slab was selected for the CRCP slab thickness.

2.3.2 Steel Design

Longitudinal steel was determined based on the experience in Texas. Currently in TxDOT, 0.68% of longitudinal steel is used for 15-in CRCP. The combined slab thickness was 17-in and it was decided that 0.7% steel would be used. In new CRCP, this steel percentage is based on the cross-sectional area of the whole section. In the bonded overlay, a decision has to be made as to whether the combined cross-sectional area or just the new CRCP overlay cross-sectional area should be used. The purpose of the longitudinal steel in CRCP is to induce transverse cracks due to temperature and moisture variations in concrete and hold them tight for good load transfer. It can be assumed that there will be little further drying shrinkage that will develop in the existing CPCD, since the concrete is more than 15 years old. Also, after 7-in concrete over the CPCD, the temperature variations in CPCD will be minimal. Most of the transverse cracks occur in places other than transverse joints. Accordingly, the load transfer efficiency at cracks in CRCP will be enhanced. As a result, it was decided that the use of CRCP cross-sectional area was appropriate for the estimation of the amount of longitudinal steel. Number 6 bars were used for longitudinal steel. Steel was placed 4-in from the surface of CRCP, which is 0.5-in lower than mid-depth. The reason for lowering the steel was to provide more restraint on concrete volume changes in the lower half of the CRCP slab, which will enhance the bonding of new concrete to old concrete. The spacing for transverse steel was at 3-ft with No. 6 bars. Transverse steel was placed below longitudinal steel, as in normal CRCP construction. Figure 2.4 shows the steel reinforcement design for this project.
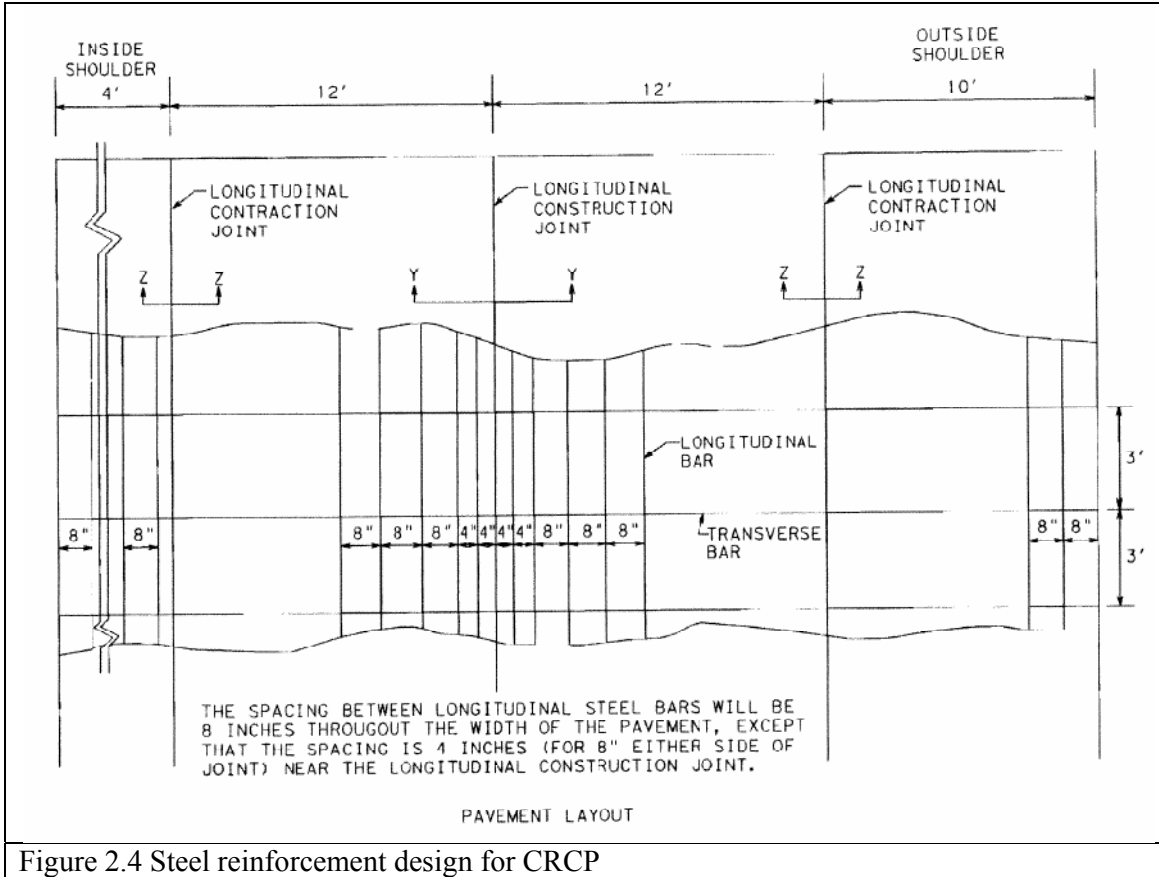
Figure 2.4 Steel reinforcement design for CRCP

2.4 Other Design/Materials/Construction Considerations

In bonded CRCP over JCP, aspects than other slab thickness and steel reinforcement need to be considered for optimum performance. This rehabilitation system is quite unique and has been rarely used. Developing an optimum system based solely on theoretical analysis would be quite difficult, since certain constitutive relations at the interface between JCP and CRCP are neither well understood nor properly quantified. In addition, it would be quite difficult to estimate, while developing pavement designs, accurate environmental conditions during construction such as temperature, relative humidity and wind velocity, which will have substantial effects on the behavior and performance of this rehabilitation system.

Over the years the research team, while conducting research studies on concrete pavement, especially 0-4893 "Performance of Old Concrete under Thin Overlays," identified design/materials/construction elements that would improve bonded concrete overlays. For example, concrete with a lower coefficient of thermal expansion (CTE) and modulus of elasticity provided much better performance than concrete with higher values for those properties.

The research team put together all the knowledge and experience gained over the years to develop a rehabilitation system that will provide optimum performance. There are three areas that affect the performance of this rehabilitation system – design, construction, and materials. Even though these three areas are, to some extent, inter-dependent, descriptions are provided on the efforts made for each area.

2.4.1 Design

There are other design elements than slab thickness and longitudinal reinforcement that will affect the performance of this rehabilitation system. One element is the design of transition area. It is well known that concrete shrinks due to moisture loss, especially at the early ages. Drying shrinkage, and to a lesser extent, temperature drop in concrete from setting temperature, will induce slab contraction at the transition area. This contraction needs to be well restrained; otherwise, shear failures could occur at the interface between the old and new concrete in the transition area and/or wide transverse cracks and resulting distress could develop. Figure 2.5-(a) illustrates the setup for the concrete displacement measurements in the longitudinal direction at the transverse construction joint in 12 in. CRCP on US287 in the Wichita Falls District. The concrete was placed on August 26, 2005, which was a Friday.
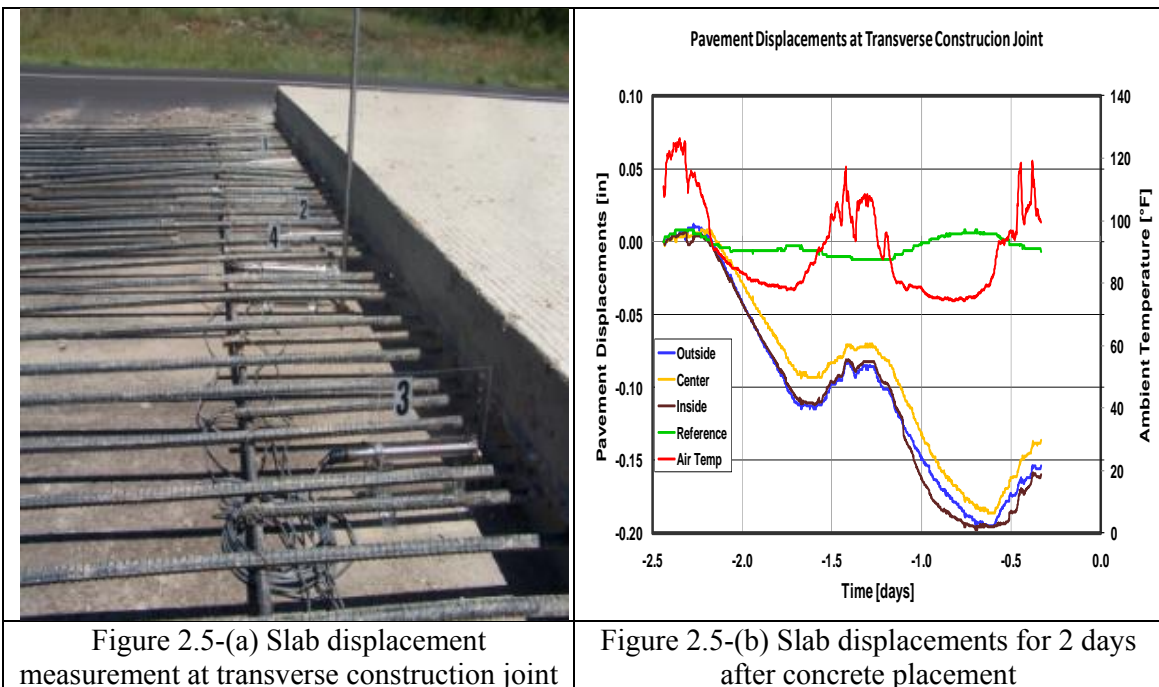
Figure 2.5-(a) Slab displacement measurement at transverse construction joint	Figure 2.5-(b) Slab displacements for 2 days after concrete placement

The construction crew had to leave in the middle of the day, and the concrete placement was completed in the late morning. When the concrete was set, the headers were removed and four LVDTs (linear variable differential transducers) were placed in the longitudinal direction as shown in Figure 2.5-(a). LVDT #4 was placed against invar as a reference. The other three LVDTs were installed directly against concrete at the mid-depth of the slab. The data gathering started at 1:28 pm on August 26, 2005 every 2 minutes until 3:58 pm on August 28, 2005 (Sunday). The construction crew had to do preparation work for Monday concrete placement and LVDTs were removed. Figure 2.5-(b) illustrates the concrete displacements along with ambient temperature. Large variations in ambient temperature are noted. Also, there are variations in displacements of LVDT against invar, even though they are not large. The variations are due to volume changes in the invar. If the invar was placed perfectly vertical, the variations could have been smaller or almost negligible. The other three lines illustrate the concrete displacements in three locations – inside, center, and outside.

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Large contractions of concrete were noted. In the x-axis, a whole number indicates midnight. Minus 2.0 indicates midnight on Friday. Minus 1.5 and -1.0 indicate Noon Friday and midnight on Saturday, respectively. By Sunday morning, there was about 0.2 in. of concrete contraction. Based on the ambient temperature history, it appears that drying shrinkage played a dominant role in concrete displacement. As will be discussed later, wet-mat curing was specified in the rehabilitation project to minimize shear stress at the interface between old and new concrete in the transition area, and to minimize potential wide transverse cracks. Concrete contraction of this magnitude could cause substantial stresses in concrete at the interface in the transition area.
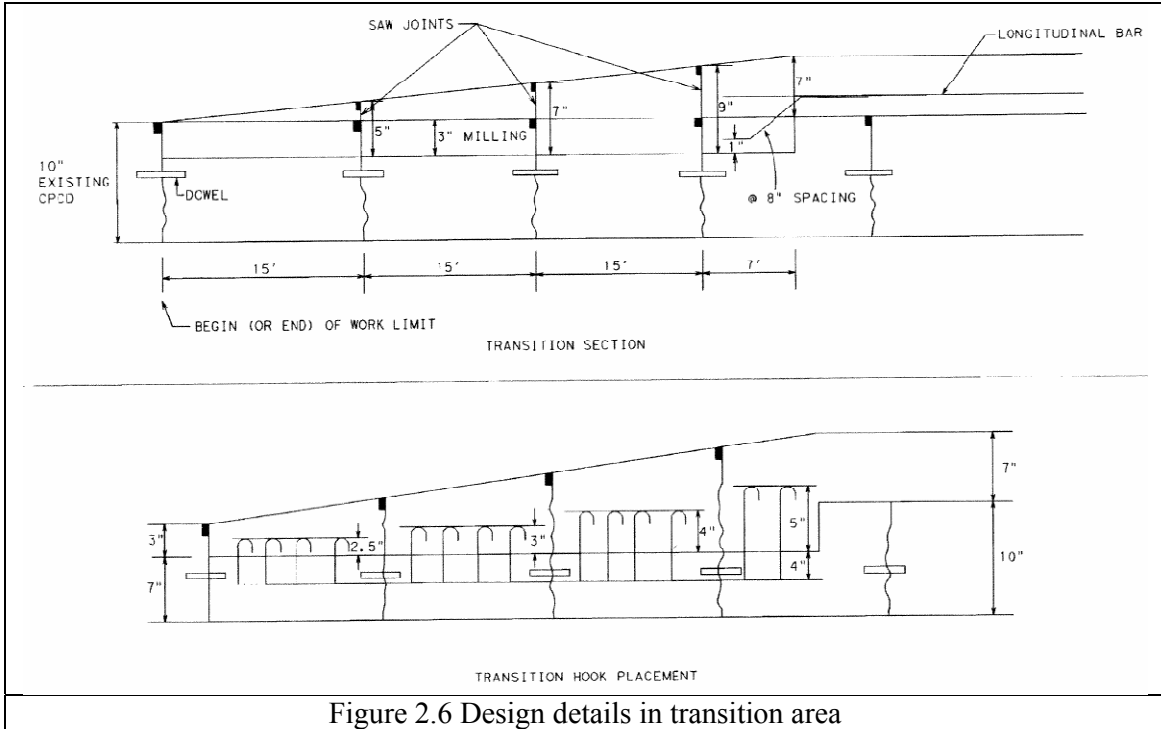
Figure 2.6 Design details in transition area

Figure 2.6 shows the design details in the transition area to minimize the potential for debonding at the interface between old and new concrete. In research project 0-4893, it was observed that most of the distresses in bonded concrete overlay were caused by debonding at the interface. Figure 2.7-(a) shows two transverse cracks at the transverse construction joint on US281 CRCP bonded overlay project in the Wichita Falls District. In this project, 4 in. of CRCP bonded overlay was placed on 6 in. of existing CRCP. Sounding testing in this area showed debonding. It is important to provide effective restraints on concrete volume changes in the transition area. It is shown that providing good bonding at the interface is of utmost importance in ensuring good performance of this rehabilitation project. Figure 2.6 shows that transition was provided in three jointed concrete pavement slabs plus 7 ft. This decision was made to provide one percent of vertical profile difference. Since the elevation difference between existing JCP and the overlaid section was 7 in., the difference in vertical profile is 7 in./((3x15+7)x12)x100 = 1.12 percent. The plan called for the milling of the top 3 in. of JCP in the transition area. During construction, a decision was made to mill the top 5 in. of JCP as shown in Figure 2.7-(b). Figure 2.7-(b) shows the beginning of the transition from JCP. The milling of 5 in. resulted in exposing dowels, since the slab thickness of the existing JCP is 10 in. Hook bars were required as shown in Figure 2.7-(b). The hook bars were installed at 4 ft

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spacing, longitudinally and transversely. The objective of the hook bars was to improve the bonding between old and new concrete.

Figure 2.7-(a) Delamination at transverse construction joint on US 281	Figure 2.7-(b) Beginning of the project with 5 in. milling of existing JCP
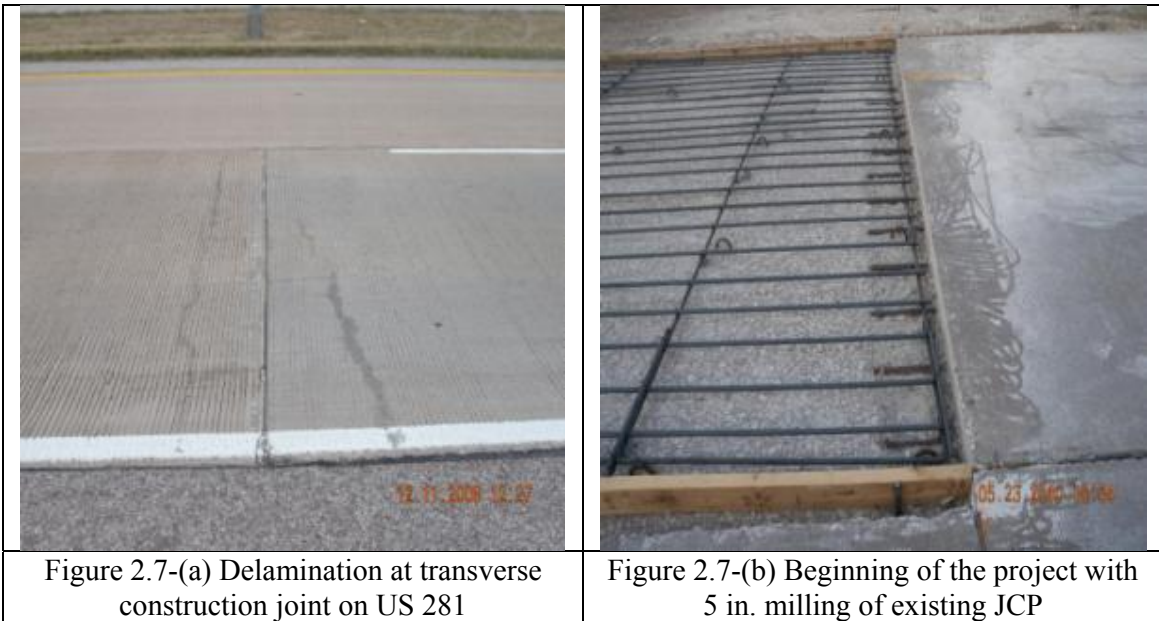
One of the most critical areas in this rehabilitation project was how to minimize the reflection cracking potential at the transverse contraction joints. As shown in Figure 2.3-(a), LTEs in some transverse joints were quite low, even though dowels were used. Joints with low LTEs could cause reflection cracking in CRCP overlay and result in distresses. Research study conducted in 1989 on SH 6 in Houston clearly indicated the potential for reflection cracking when concrete was placed on an asphalt layer with cracks.[3] To minimize the reflection cracking potential, a decision was made to place 2-ft wide non-woven fabric on top of transverse joints. Figure 2.8-(a) shows the plan requirement and Figure 2.8-(b) shows the non-woven fabric installed. The non-woven fabric used was Mirafi 1450BB from Tencate Company. The material properties of the fabric are shown in Table 2.1.

When a fabric is placed on a transverse joint, it is breaking the bond between old and new concrete, which violates the assumption of the bonded concrete overlay. However, the flexural stress at the bottom of the 7 in. overlay due to wheel loading will be maintained quite low. It was determined that the use of non-woven fabric will provide benefits of retarding or preventing reflection cracking without compromising the fatigue life of the overlay slab.
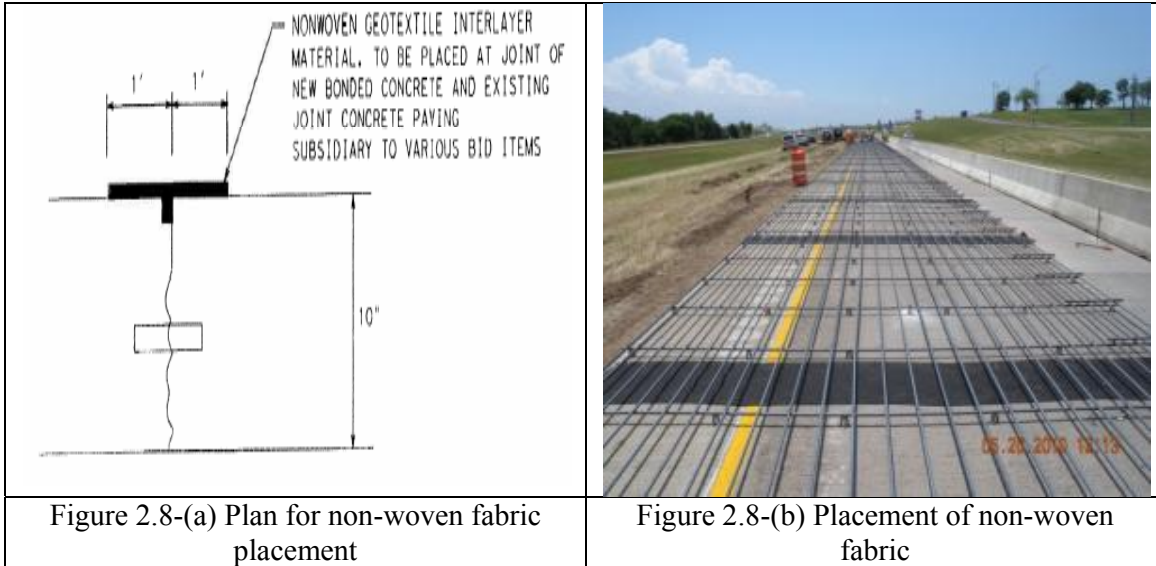
| Figure 2.8.-(a) Plan for non-woven fabric placement | Figure 2.8.-(b) Placement of non-woven fabric |

Table 2.1 Material properties of Mirafi 1450BB

Mechanical Properties	Test Method	Unit	Roll Value
Weight[1]	ASTM D 5261	g/m^2 (oz/yd^2)	495 (14.6)
Thickness Under Load	ASTM D 5199		
0.29 psi (2 kPa)[1]		mm (mils)	3.0 (118)
2.9 psi (20 kPa)[1]		mm (mils)	2.5 (98)
29 psi (200 kPa)[1]		mm (mils)	1.0 (39)
Wide Width Tensile Strength[2]	ASTM D 4595	kN/m(lbs/ft)	10 (685)
Wide Width Elongation[3]	ASTM D 4595	%	130
Water Permeability in Normal Direction under load	ASTM D 5493		
2.9 psi (20 kPa)[1]		m/s	1 x 10^{-4}
In-plane Water Permeability	ASTM D 6574		
2.9 psi (20 kPa)[2]		m/s	5 x 10^{-4}
29 psi (200 kPa)[2]		m/s	2 x 10^{-4}
Alkali Resistance[1]	EN 13249	%	96
UV Resistance (at 500 hours)[1]	ASTM D 4355	% strength retained	80

[1] Typical
[2] Minimum
[3] Maximum

2.4.2   Materials

Previous studies indicate that concrete material properties have a substantial effect on bonded concrete pavement. More specifically, concrete with low CTE and modulus of elasticity provides better performance than concrete with higher CTE or modulus of elasticity. Both properties are influenced by coarse aggregate type. In Texas, coarse aggregate types that produce high CTE also provide high modulus of elasticity, and vice versa. Field testing was

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conducted by TxDOT engineers in the early 2000s to identify the causes of good and poorly performing CRCPs. Figure 2.9 illustrates the findings. It shows the substantial effects that coarse aggregate type has on modulus of elasticity and CTE. Siliceous river gravel gives high values in modulus of elasticity and CTE. Oyster shell and lightweight aggregate provide low values for the two properties. Also, there is a loose correlation among aggregate type, modulus of elasticity and CTE – the smaller the modulus of elasticity, the smaller the CTE. From an engineering standpoint, oyster shell or lightweight aggregate concrete would be ideal for this rehabilitation project. From a practical standpoint with a fixed budget for this project, a coarse aggregate that would provide a maximum concrete CTE of 4.6 microstrain per F was specified. It was expected that by specifying a maximum CTE value, the concrete would have reasonably low modulus value as well, which is expected to lower the concrete stresses at the interface between old and new concrete in the transition area.

Figure 2.9 Various concrete properties as affected by coarse aggregate type
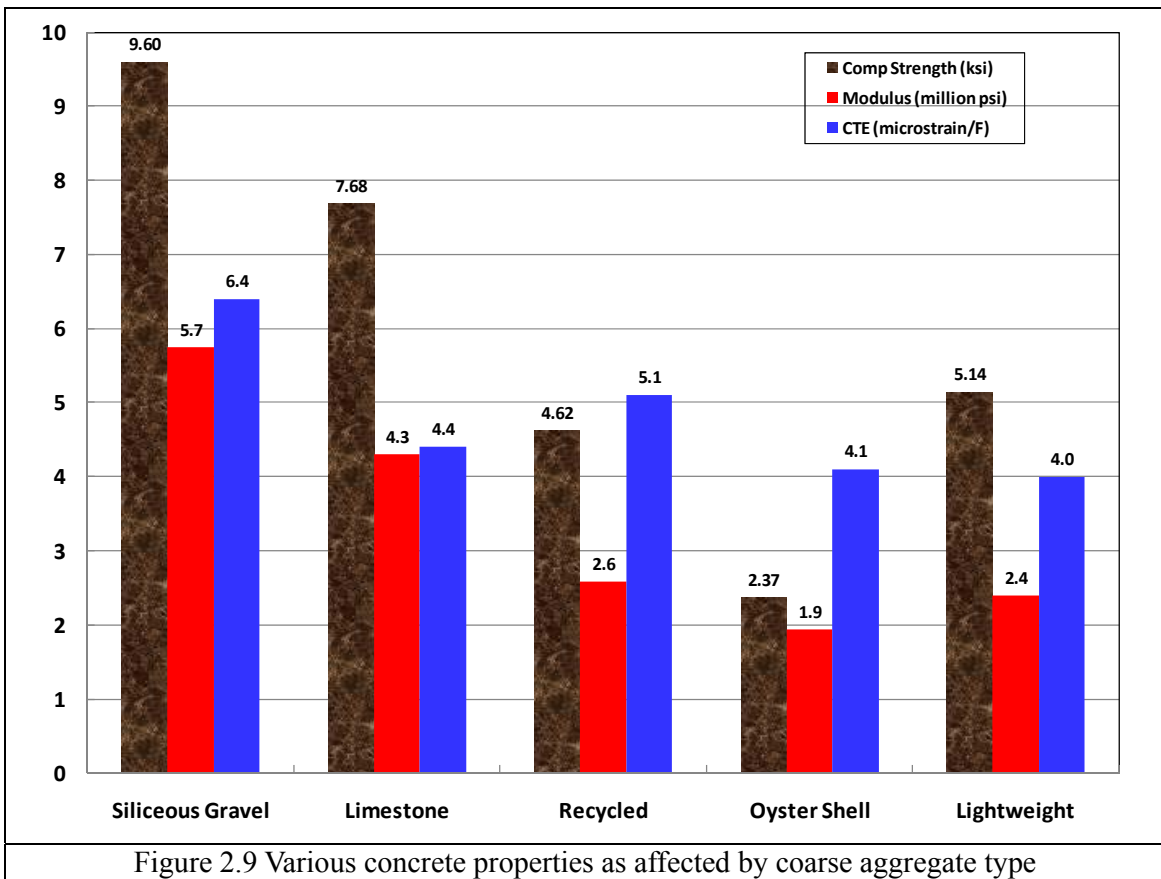
2.4.3 Construction

As discussed previously, concrete will experience volume changes due to temperature and moisture variations. To ensure good bonding between old and new concrete layers, it's important to minimize concrete volume changes at early ages when the interface bond strength is relatively low. Even though there will be delamination at transverse joints for 2-ft longitudinally, ensuring good bond between non-woven fabric will improve the pavement performance. To minimize concrete volume changes due to temperature and moisture variations at early ages, wet-mat curing was required. The wet-mat curing required was included in the Special Provision to Item 360 as shown on the following page:

Article 360.4. Construction, Section 1. Curing, 3. Curing Class HES Concrete
is supplemented by the following:

Keep all exposed surfaces of the concrete wet continuously for the required curing time. Use water curing that meets the requirements for concrete mixing water in Section 421.2.D, "Water." Do not use seawater or water that stains or leaves an unsightly residue. Keep the concrete continuously wet by maintaining wet cotton mats in direct contact with the concrete for the required curing time. If needed, place damp burlap blankets made from 9-oz. stock on the damp concrete surface for temporary protection before applying cotton mats. Then place the dry mats and wet them immediately after they are placed. Weight the mats adequately to provide continuous contact with all concrete. Cover surfaces that cannot be cured by direct contact with mats, forming an enclosure well anchored to the ground so that outside air cannot enter the enclosure. Provide sufficient moisture inside the enclosure to keep all surfaces of the concrete wet. Overlap sprays or sprinklers to keep all unformed surfaces continuously wet.

Figure 2.10-(a) shows the wet-mat curing applied in this project on the day of concrete placement (May 24, 2010). Cotton mat was placed and kept wet by water spraying. On the same day, vinyl sheet was placed on top of the cotton mat to prevent drying of cotton mat, as shown in Figure 2.10-(b). Steel rebar was placed to keep vinyl sheet in place. Cotton mat was kept in place for 7 days.

Figure 2.10-(a) Wet mat placement	Figure 2.10-(b) Vinyl sheet placement

Chapter 3: CRCP BCO Construction and Evaluation of
Material Properties and Early Age Behavior

There were two phases in the concrete placement of this project. In the first phase, concrete was placed in the inside lane and inside shoulder. In the second phase, concrete was placed in the outside lane and outside shoulder. This chapter describes the construction of the bonded concrete overlay, and discusses the results of material property evaluations. Early age behavior of the bonded concrete overlay (BCO) is briefly discussed. More detailed information on the analysis of BCO behavior will be presented in the research reports to be prepared under TxDOT Research Project 0-6274, "Project Level Performance Database for Rigid Pavements in Texas, Phase II."

3.1   Construction

There were two phases in this project. In both phases, slip-form paver was used. Concrete was supplied by ready-mix trucks. In the first phase, concrete was placed on May 24, 2010 from the north end of the project. The width of the placement was 16 ft (12 ft inside lane and 4 ft inside shoulder). Concrete supply was smooth, with no extended gap between trucks. The first concrete truck arrived at the job site about 7:30 in the morning and the concrete placement was completed at about 3:00 pm. It took about 7 hours and 30 minutes to complete the concrete placement for half a mile length, including finishing, tining and curing membrane applications.

Figure 3.1 shows the sequence of the construction utilized in the Phase 1 construction.

| 1.Cold milling in transition section | 2.Hook bar installation | 3.Fabric installation |

4. Reinforcement installed	5.Reinforcement in transition	6.Surface cleaning
7. Concrete placement	8.Tining and curing operation	9.Joint saw cutting
10. Cotton mat for curing	11. Vinyl sheet for wet curing	12. Open to traffic

Figure 3.1 Construction sequence of CRCP BCO

Transition sections in the north and south ends consisted of three slabs plus half a slab each, for a total of 52.5 ft long. The top 5 in. of concrete was removed by cold milling. Hook bars were installed every 4 ft in both directions in the transition sections, and longitudinal steel was placed. Non-woven fabric, 2 ft wide and 16 ft long, was installed at transverse contraction joints with nails and epoxy. Longitudinal and transverse steel was placed with

chairs. Female multi-piece tie bars were installed at transverse steel location. At the same time, steel bars were placed in the transition areas. Before concrete was placed, the surface of JCP was cleaned with pressurized water jets. Lane marking was not completely removed. While the surface of JCP was still in saturated, surface dry (SSD) condition, concrete was placed. Once concrete attained enough stiffness, the tining operation was done, followed by curing membrane applications. Joint saw cutting was done at longitudinal contraction joint and transverse contraction joints in the transition areas. After the saw cutting was completed, cotton mat was placed and water sprayed. Afterwards, vinyl sheet was placed to keep the cotton mat wet. Cotton mat and vinyl sheet were placed on the same day (May 24, 2010). The wet mat curing system continued until June 1.

Phase 2 construction followed the same sequence, except that cold milling was done on the JCP surface to produce a rough texture, and longitudinal steel was not placed in the transition areas. Concrete was placed on July 14, 2010. Unlike Phase 1 construction, there was a problem for the concrete placement. The section was about 600 ft long and the following problems were encountered:

1. Excessive water was observed on the concrete surface after curing operations were completed.
2. Slip form paver was not able to spread concrete throughout the whole width. Forms had to be placed to finish edges.
3. Slip form paver could not push the concrete and had to back up and slip the concrete a second time.
4. Paving was done for approximately 600 linear ft, but had to install three headers.
5. The concrete placed was too dry and in some cases, water was added into the ready mix truck that was over the allowable limit.
6. Spreader and slip form paver did not follow a consistent grade.

The primary cause of this problem was that the ready mix trucks had to wait at the jobsite for more than 1.5 hours before the concrete was placed. This section was removed and replaced.

3.2   Concrete Material Evaluations

The concrete mix used in this project was Class HES concrete with a minimum compressive strength of 2,600 psi in 24 hours. The coarse aggregate used was limestone with 1.5 in. maximum nominal size and fine aggregate was siliceous river sand. The mix design is shown in Table 3.1.

Table 3.1 Concrete Mixture Proportion

	Unit	Amount
Cement	lb/cy	397
Fly Ash	lb/cy	132
Water	lb/cy	238
Coarse aggregate	lb/cy	2,009
Fine aggregate	lb/cy	1,148
Air Entraining Agent	oz/cy	5.6
Water reducing retarder	oz/cy	7.2

Concrete compressive strength values were 2,800 psi at 1 day, 3,030 psi at 3 days and 3,880 psi at 8 days, which met the requirement of a minimum of 2,600 psi at 24 hours. Setting characteristics of concrete was evaluated as shown in Figure 3.2-(a) and the results are illustrated in Figure 3.2-(b). The initial setting was obtained in 2 hours and 10 minutes and the final setting was at about 3 hours. The setting information indicates this concrete set a little bit quickly compared with normal paving concrete, considering the concrete delivery time was less than 20 minutes.
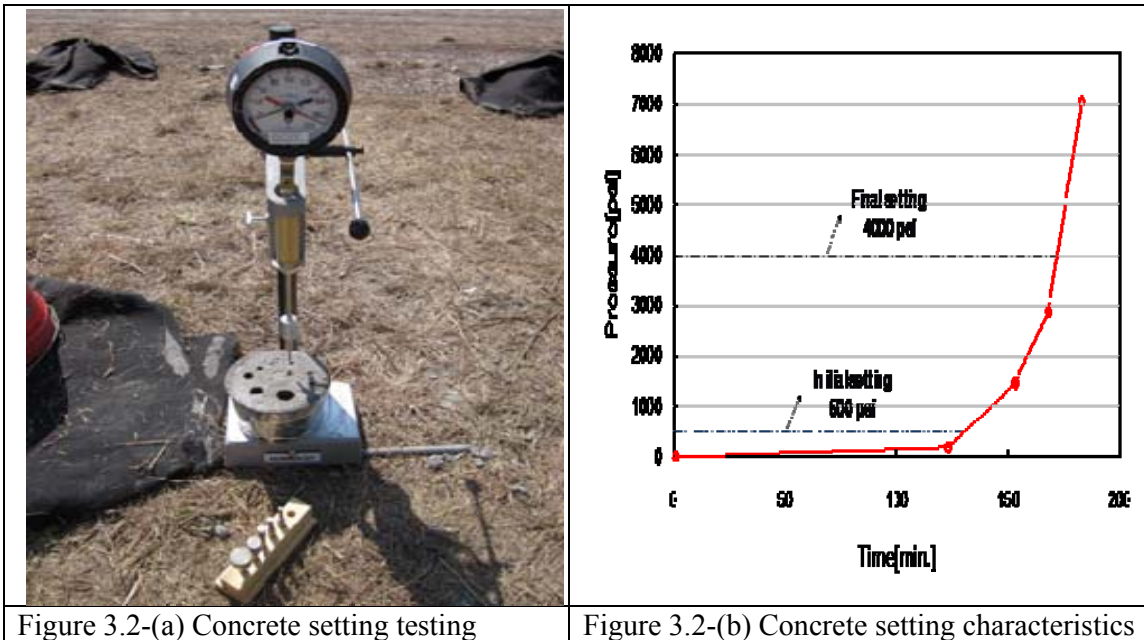
Figure 3.2-(a) Concrete setting testing	Figure 3.2-(b) Concrete setting characteristics

3.3   Early Age BCO Behavior Evaluation

Early age BCO behavior was investigated by various gages installed at the job site. The gages installed included steel strain gages (SSGs), vibrating wire strain gages (VWSGs), thermo-couple wires and concrete relative humidity (RH) gages.

SSGs were installed to evaluate steel stresses in the transition area as well as in the overlaid area. Steel stresses in the transition area could provide information on how much force is exerted from concrete volume contractions due to drying shrinkage and temperature

drop from setting temperature. Information from VWSGs could provide valuable information on in-situ CTE and drying shrinkage as well as concrete stresses. Temperature information from thermo-couples and RH in concrete will be used to isolate concrete strains due to stresses from total strains. These gages were installed in several places and the discussions below will be in accordance with gage types – steel strain data first, VWSG data and followed by RH data.

3.3.1. Steel Strain Gages

Steel strain gages were installed in two different areas – in the transition and overlaid areas.

3.3.1.1. Transition section

Figure 3.3 illustrates SSGs installed in the transition area at the north limit of the project. A total of 13 SSGs were installed – 10 of them in the bent bars and three in the longitudinal steel.
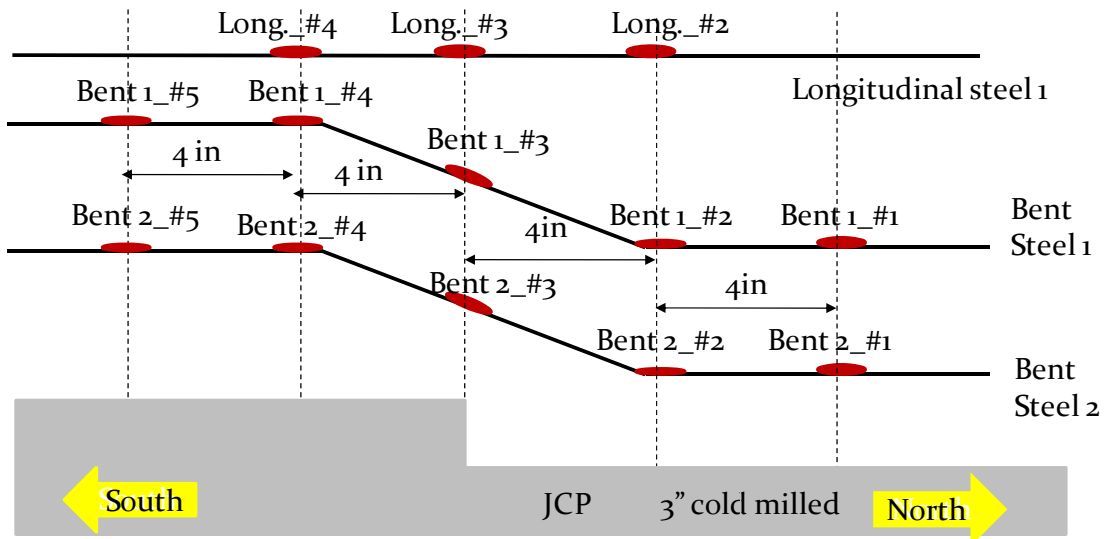
Figure 3.3 Steel strain gages installed in north transition section

Figure 3.4-(a) illustrates steel configuration at the intersection between normal overlay and transition area. Bent bars were installed to strengthen the rigidity of the concrete in this area, because of the differences in structural rigidity of concrete slabs (7 in. vs 12 in. thick slabs). The comparison of steel strains in bent bars 1 and 2 are shown in Figure 3.4-(b). There is little variation in steel strains in the bent bars at location #3, where concrete thickness varies from 7 in. to 12 in. It should be noted that the direction of the steel at this location is not horizontal, and that the steel strains measured are in the inclined direction.

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Figure 3.4-(a) Steel reinforcement in the transition area	Figure 3.4-(b) Steel strains in the bent bars in the transition area

Figure 3.5-(a) compares steel strains at longitudinal steel and bent bar at location 2, which is 4 in. north from the intersection between normal overlay and transition area. Very little variation wa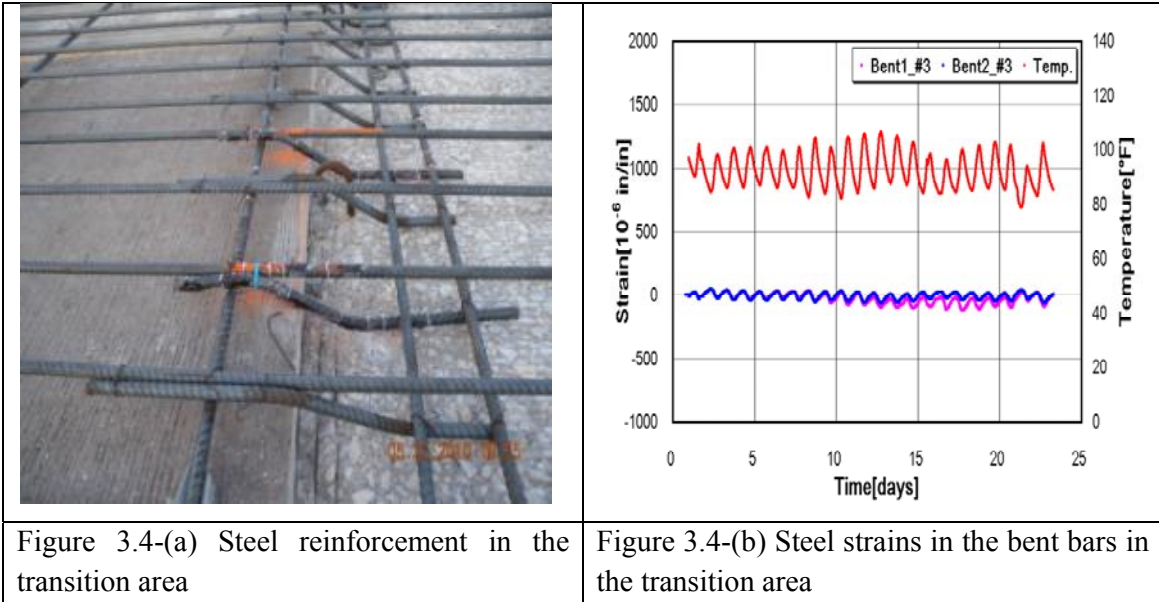s observed in the steel strain in the bent bar, whereas quite large variations in steel strain, up to 1,300 $\mu\varepsilon$ a day, were observed in the longitudinal steel.

Figure 3.5-(a) Variations in steel strain at transition area	Figure 3.5-(b) Steel strain gages installation plan
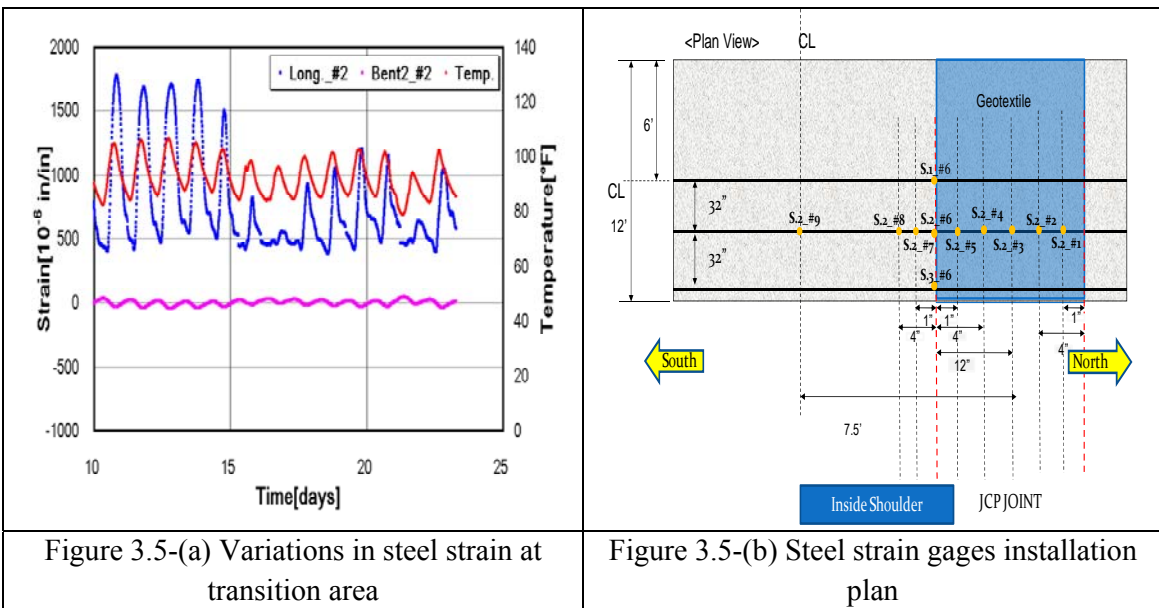
Large strain variations in the longitudinal steel could have been caused by transverse micro-cracks in concrete in this area, even though no cracks were visually observed in this area at that time. Temperature and moisture variations, and resulting concrete strains and stresses, were maximum in the concrete near the surface. Once micro-cracks develop, the stresses in the concrete are relieved and cracks do not propagate. Low steel strains in the bent bar, which is below the mid-depth of 7 in. concrete slab, indicate the micro-cracks did not

propagate below the mid-depth of the slab. In CRCP, most of the transverse cracks on the surface propagate up to 2 to 3 in. below the surface and stresses are dissipated. No cracks are observed at the mid-depth of the steel. In this section, the bond between old and new concrete in the transition area is enhanced by rough texture from cold milling and hook bars. Enhanced bond will further restrain concrete volume changes and micro-cracks on the surface won't propagate into the bottom of the slab as well as they might in concrete with low bond strength.

As discussed earlier, 2 ft wide non-woven fabric was placed on transverse contraction joints to minimize the potential for reflection cracking. The behavior of concrete and steel in the area with fabric is of interest, since there is a large difference in bond characteristics – full bond achieved assumed in the area without fabric and almost no bond in the area with fabric. To investigate the steel strain variations in this area, steel strain gages were installed as shown in Figure 3.5-(b). Strain gages were installed in three longitudinal bars.

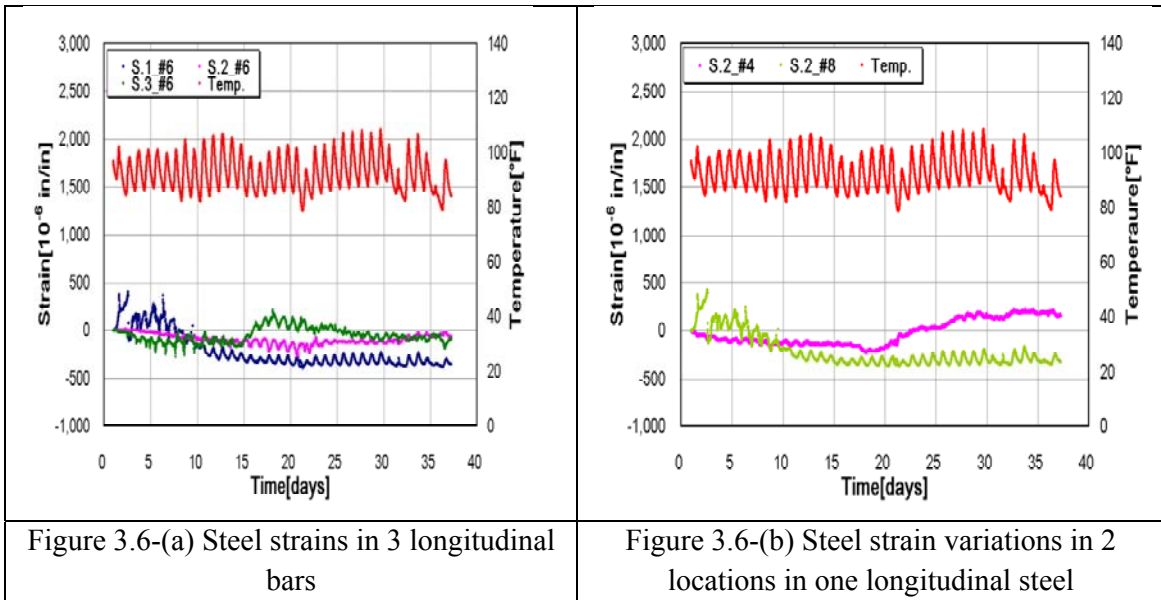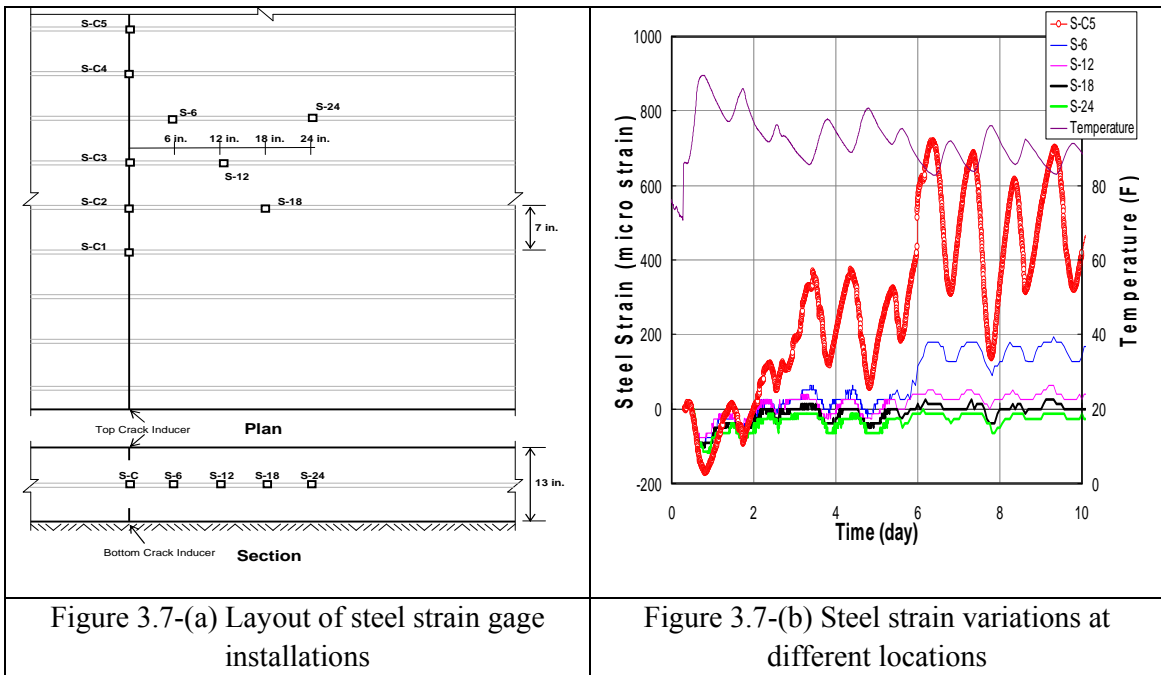|	
Figure 3.6-(a) Steel strains in 3 longitudinal bars	Figure 3.6-(b) Steel strain variations in 2 locations in one longitudinal steel

Figure 3.6-(a) presents the steel strains in three longitudinal bars at the end of the fabric. Daily steel strain variations were quite small, which indicates no transverse cracks in this area and illustrates that the bond between new CRCP and JCP with no fabric was quite good. It also indicates that almost no bonding between new CRCP and JCP due to the use of fabric does not necessarily alter the steel and concrete stress distributions in this area. This finding is supported by the transverse cracking in this project. About 80% of transverse cracks were developed at transverse steel locations, which indicates that the concrete stresses due to environmental loading were much more affected by the interactions between steel and concrete, not by the bond strength at the interface between old and new concrete. This finding should be interpreted with a caution. Stresses in longitudinal steel are affected by transverse cracks. Steel stresses could be quite large at transverse cracks, and diminish quite rapidly as the distance from the crack increases. In 2004, steel strains were measured on CRCP in US 59 in Cleveland, Houston District.[4] Figure 3.7-(a) illustrates the steel strain gage

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installation layout. Steel strain gages were installed at the same transverse location in five bars. A transverse crack was induced at this location. Also, additional strain gages were installed at 6 in. spacing longitudinally up to 24 in. from the induced transverse crack. Figure 3.7-(b) shows that steel strain becomes negligible once the distance from a transverse crack is about 12 in. and further.

Figure 3.6-(b) presents steel strains in a longitudinal steel bar at two different locations – one at 4 in. from the end of fabric into the fabric area and the other at 4 in. from the end of fabric into the no fabric area. It is shown that steel strain variations are quite small in both locations, even though those outside the fabric area have larger daily variations than those inside the fabric area.
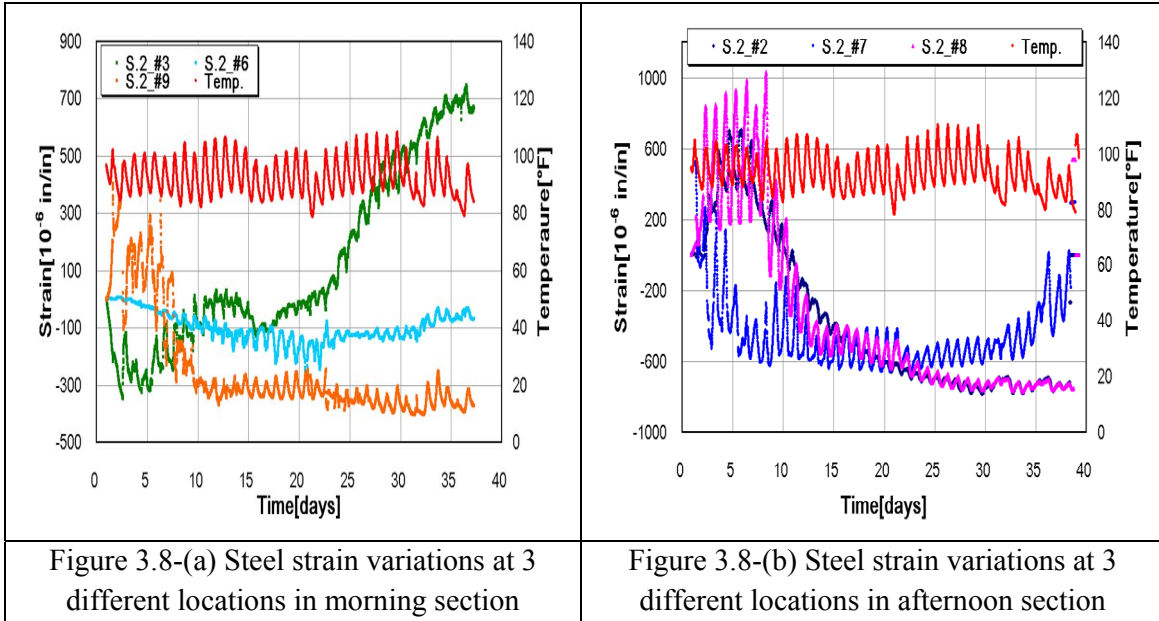

Figure 3.7-(a) Layout of steel strain gage installations	Figure 3.7-(b) Steel strain variations at different locations

Figure 3.8-(a) Steel strain variations at 3 different locations in morning section	Figure 3.8-(b) Steel strain variations at 3 different locations in afternoon section

Figure 3.8-(a) presents the steel strain variations at three different locations in one longitudinal steel bar – in the center of the fabric, at the edge of the fabric and 7.5 ft away from the center of the fabric (or contraction joint). It shows that the steel strains at the center of the fabric were in compression at the beginning probably due to the drying shrinkage of concrete. Also, at the early ages, when the temperature goes up, steel in the center of fabric undergoes tensional direction while the steel strain at 7.5 ft away from the contraction joint undergoes compression direction. The volume changes of concrete above the fabric are not restrained as much as the concrete above the non-fabric area, since the interface friction at the fabric area is quite low. With time, as concrete develops strength including bond strength, the concrete outside the fabric area develops larger structural rigidity, since the slab thickness is now 17 in. compared with 7 in. in the fabric area. When the concrete temperature decreases, the concrete volume contracts, and with larger slab rigidity of the bonded area pulling the concrete in fabric area, steel and concrete in the center of the fabric undergoes tensile strain.

One of the objectives of the steel strain, as well as concrete strain, measurements was to evaluate the effects of debonding at the interface by the use of non-woven fabric on the pavement responses. To achieve this, the same set of steel strain gages shown in Figure 3.4-(b) was installed in the afternoon section where fabric was not installed. To eliminate the boundary effects, fabric was removed from 20 joints. Figure 3.8-(b) illustrates that steel strains are in compression.

3.3.2. *Vibrating Wire Strain Gages (VWSGs) in Concrete*

To evaluate the behavior of CRCP bonded overlay on JCP, VWSGs were installed in two locations – one in the slab at 1,845 ft south from the north limit of this project (called morning section in this report), and the other at 2287.5 ft south from the north limit of this project (called afternoon section in this report). Figure 3.9-(a) presents the VWSG setup in

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the morning section. The afternoon section has the same VWSG setup, except that fabric was removed from 20 transverse joints and gages were installed in the middle slab among 21 slabs.

3.3.2.1 CRCP Behavior at the Corner

To investigate the concrete behavior at the corner location, three VWSGs were installed at the bottom of the slab in three different directions – transverse, diagonal, and longitudinal. In Figure 3.9-(a), they are shown as CN_Bot_Trans, CN_Bot_Dig and CN_Bot, respectively.

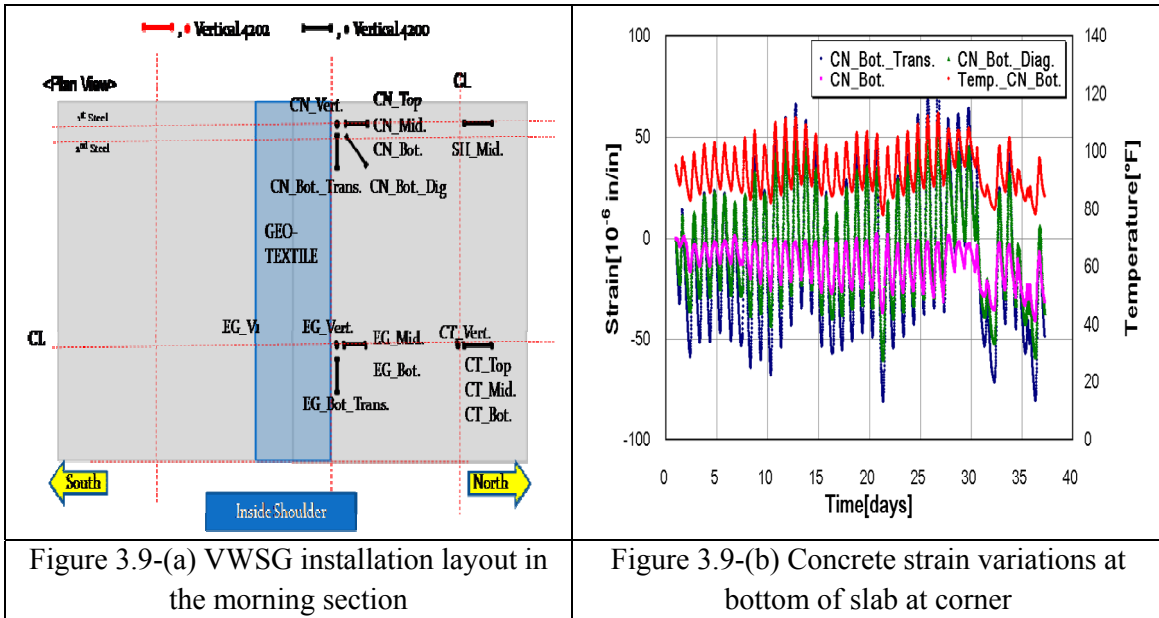| Figure 3.9-(a) VWSG installation layout in the morning section	Figure 3.9-(b) Concrete strain variations at bottom of slab at corner

Figure 3.9-(b) illustrates the concrete strain at the bottom of the slab at the corner in three directions. It is noted that the concrete strains move in compression direction in the morning and in tension direction in the afternoon. In other words, in the graph, concrete temperatures and concrete strains are in the same direction. The temperature shown here is the concrete temperature at the bottom of the 7 in. CRCP slab. This appears to be contrary to what would be expected considering curling effect. In the morning, concrete temperature at the top of the slab is normally lower than that at the bottom, resulting in curling up of the slab. If the curling effect is dominant and the bond between old and new concrete is not perfect due to fabric, it is expected that the concrete at the bottom of the 7 in. CRCP slab will be in tension in the morning and compression in the afternoon. Figure 3.10-(a) shows the temperature variations through the depth of 7 in. CRCP slab, which indicates quite small temperature variations through the depth compared with daily temperature variations. In normal concrete pavement slabs on regular subbase, the temperature variations through the slab depth are larger than those shown here. One of the reasons could be that the 7 in. CRCP slab is resting on 10 in. JCP slabs, which reduces the temperature variations through the slab depth. Small temperature variations through the slab depth explain why the concrete strains in Figure 3.10-(b) move in the compression direction in the morning and in the tension direction

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in the afternoon. The axial component of the concrete displacement is much larger than the curling component. It should be also noted that the subbase under the 10 in. JCP in this project is aggregate base, which has a small frictional coefficient. The order of concrete strains in three directions – transverse, diagonal followed by longitudinal – also supports the dominant effect of axial component of concrete displacement. Large concrete strains in the transverse direction indicate relative free movement in that direction, and accordingly quite small concrete stresses result in that direction, with a low probability of longitudinal cracking. On the other hand, small concrete strains in the longitudinal direction indicate large concrete stress in that direction, and a high potential for transverse cracking.

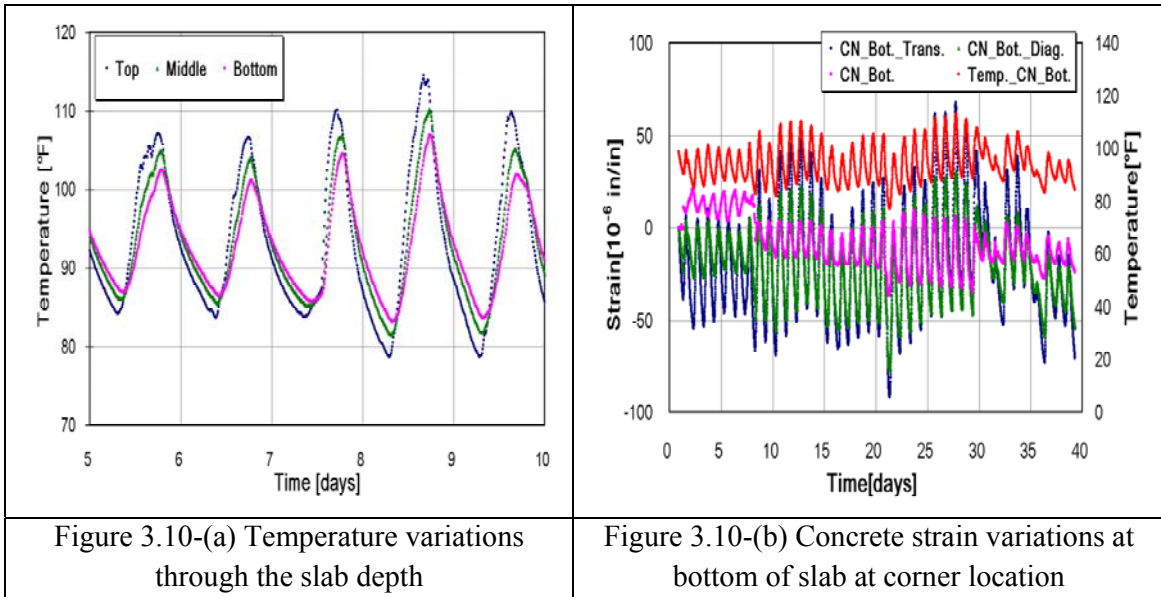| Figure 3.10-(a) Temperature variations through the slab depth	Figure 3.10-(b) Concrete strain variations at bottom of slab at corner location

Figure 3.10-(b) shows the concrete strains at the bottom of the 7 in. CRCP at the corner in three directions in the afternoon section where fabric was removed. Compared with Figure 3.9-(b), the same trend is observed and it appears that the interface bond condition doesn't affect the bottom concrete strains at the corner.

3.3.2.2 CRCP Behavior at the Center
The concrete behavior of 7 in. CRCP on top of the center of JCP due to environmental loading was investigated. Three VWSGs were installed in the longitudinal direction at three different depths – top (1 in. from the slab surface), middle, and bottom (1 in. from the bottom of 7 in. CRCP). In Figure 3.11-(a), they are shown as CT_Top, CT_Mid and CT_Bot, respectively.
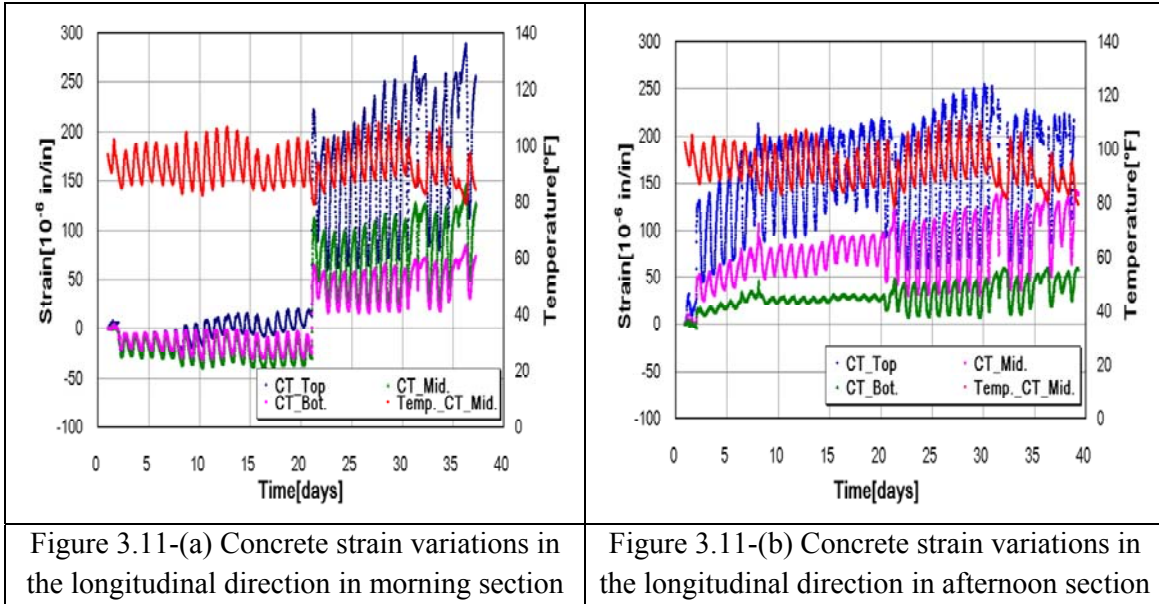
| Figure 3.11-(a) Concrete strain variations in the longitudinal direction in morning section | Figure 3.11-(b) Concrete strain variations in the longitudinal direction in afternoon section |

Figures 3.11-(a) and 3.11-(b) illustrate the concrete strains in the longitudinal direction in the morning and afternoon section, respectively. Comparison of the morning and afternoon sections indicates that concrete strain variations in the afternoon section are much larger than those in the morning section, up to day 21. In CRCP, concrete strains are maintained quite small, unless transverse cracks occur within the gage length of VWSG. In Figure 3.11-(a), large variations of concrete strains are observed at day 21. This indicates a transverse crack occurred at day 21. On the other hand, Figure 3.11-(b) shows large concrete variations at day 2, which indicates a transverse crack occurred at that time. Also noted in the figures is that concrete strains are largest at the top and smallest at the bottom. The increase in the concrete strains at the bottom after the crack occurrence indicates that debonding at the interface occurred in the vicinity of the transverse crack.

Figures 3.12-(a) and 3.12-(b) show the crack width variations for 2.5 days after cracks occurred in the morning and afternoon sections, respectively. It is observed that cracks occurred while concrete temperature was going down, not at the lowest concrete temperature. The concrete temperature here is at the mid-depth of the 7 in. CRCP slab. The widths of cracks were quite small, even at the top of the slab. Cracks were less than 0.0015 in. in the morning section and less than 0.0008 in. in the afternoon section, even though with the further drying shrinkage and temperature drop, these crack widths might increase. Also interesting are the crack width variations through the slab depth. It is noted that, in both sections, crack widths at all three depths moved in the same direction. In other words, crack widths at all three depths followed the same variations over time. This is quite different from crack width variations through a slab depth observed in normal CRCP.

Figure 3.13-(a) illustrates crack width variations through the slab depth observed in normal CRCP. This 13 in. CRCP section is on US 59 in Cleveland, Houston District.

Figure 3.12-(a) Crack width variations along slab depth in morning section	Figure 3.12-(b) Crack width variations along slab depth in afternoon section
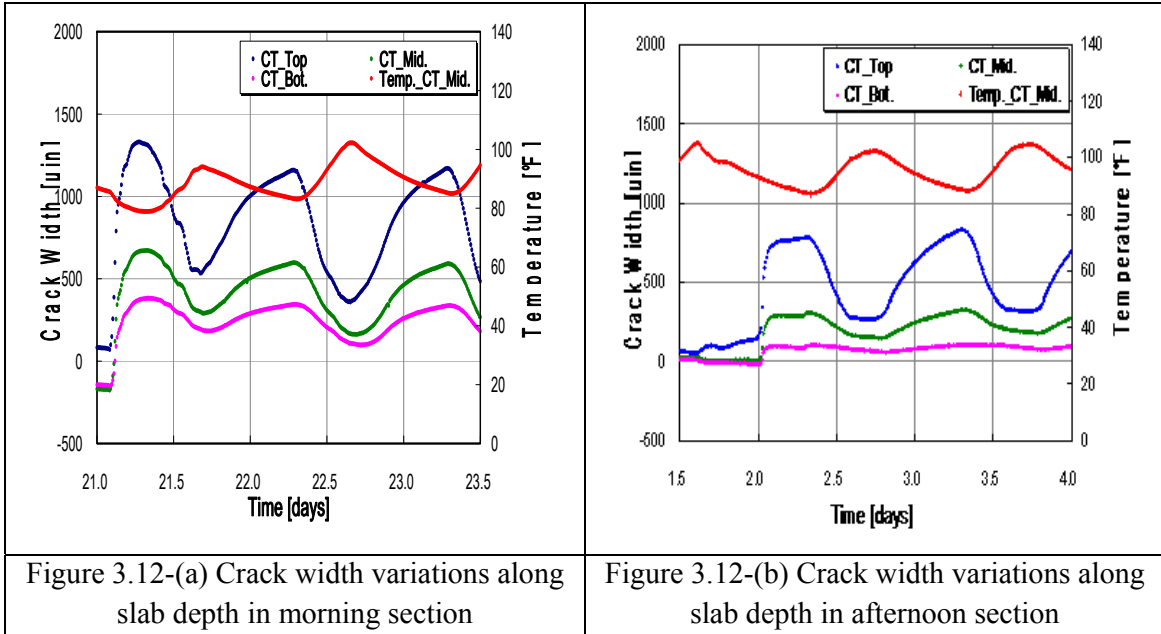
Concrete was placed on July 20, 2004.[5] It shows that the crack width at the bottom of the slab (1 in. from the slab bottom) varies in the opposite direction with the crack widths at the top (1 in. from the slab surface) and middle of the slab. The crack width variations through the depth of the slab in normal CRCP indicate bending action with a neutral axis a little below the depth of the steel (mid-depth of the slab). On the other hand, those in CRCP BCO indicate bending action with a neutral axis below the bottom of the slab. The difference in crack width variations through the slab depth between BCO and normal CRCP appears to be due to the interface bonding condition at the bottom of CRCP and the layer below.

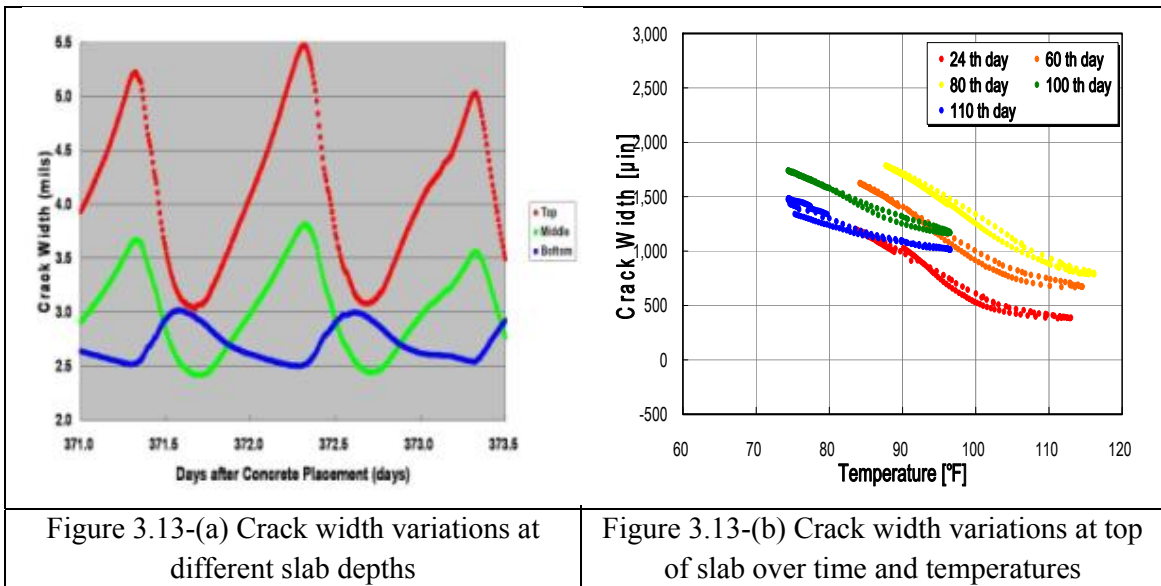| Figure 3.13-(a) Crack width variations at different slab depths	Figure 3.13-(b) Crack width variations at top of slab over time and temperatures

Figure 3.13-(b) shows the crack width variations at the top of the slab over time of the crack in the morning section. Crack width increased from day 24 to day 60 to day 80. On the other hand, crack width actually decreased on day 100 and further on day 110, even though concrete temperature decreased in that time period. This finding is consistent with the observations made in other CRCP sections, where crack widths decreased over time.[1] The difference is that the observations made in Reference 1 were at an induced transverse crack, while in this project, the crack occurred naturally. The prevailing current theory on CRCP crack behavior is that crack width increases over time, and load transfer decreases with the increase in crack width, ultimately resulting in punchouts. The findings in this study and reference (5) indicate otherwise. Long-term monitoring will be made on the behavior of this crack and included in the report under TxDOT Project 0-6274.

3.3.2.3 Non-Stress Cylinder Result Analysis

Non-stress cylinders (NSC) were installed in the morning and afternoon concrete sections. Two NSCs were installed in each location – one with impervious NSC (INC) and the other with pervious NSC (PNC), as shown in Figure 3.14. The center of the NSCs was 4 in. from the surface of the CRCP. In Figure 3.14, the cylinder on the left is INC and the one on the right is PNC. The concrete inside of the INC remains almost saturated, since it does not lose any moisture and initially sufficient moisture is provided on the inside wall of the INC. The only shrinkage in the concrete in INC could be chemical and autogenous shrinkage. Since excess water was provided in the INC using cotton, both shrinkages are assumed to be negligible, and the concrete strains in INC are due to temperature variations only. The slope of a line between concrete temperatures and concrete strains in INC, therefore, will be in-situ CTE. On the other hand, the concrete in PNC will lose or gain moisture through holes by moisture exchange with the surrounding concrete and it is assumed that there is moisture equilibrium in concretes inside and outside of the PNC. Accordingly, the concrete strains in the PNC will be due to both thermal and moisture variations. The difference in concrete strains in INC and PNC will be drying shrinkage.
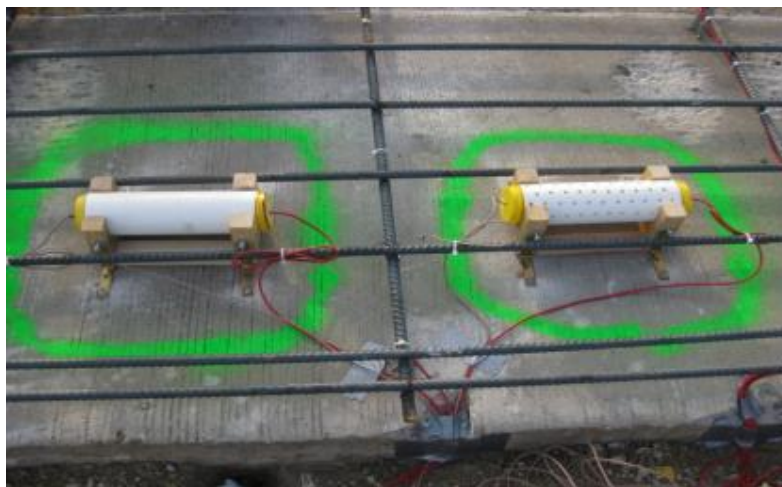

Figure 3.14 Impervious (left) and pervious (right) NSC

Figures 3.15-(a) and 3.15-(b) show the INC and PNC data from the morning and afternoon sections, respectively. They show that concrete strains vary along with concrete temperature. Also, the difference in concrete strains between INC and PNC in the morning section is much smaller than that in the afternoon section. This indicates that the drying shrinkage in the morning section would be smaller than that in the afternoon section.

Figure 3.15-(a) Concrete strain variations in NSCs in the morning section	Figure 3.15-(b) Concrete strain variations in NSCs in the afternoon section
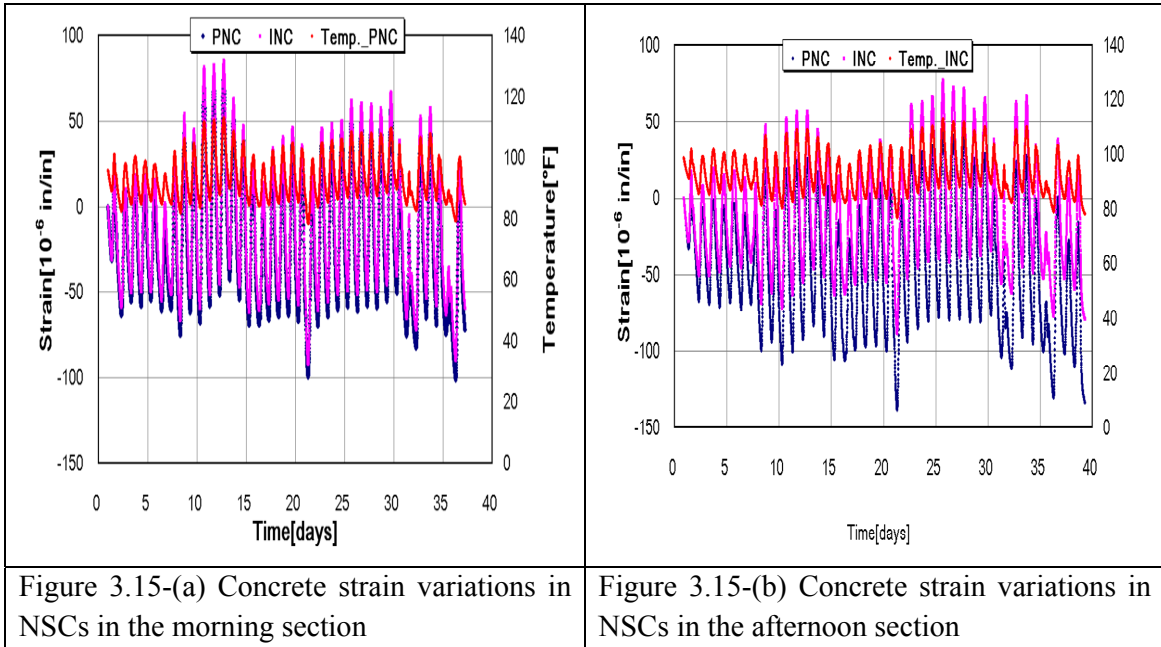
3.3.2.3.1 In-situ CTE Evaluation: In-situ CTE was estimated from the data in INC in the morning and afternoon sections, as shown in Figure 3.16-(a) and 3.16-(b), respectively. CTE values of 5.0 and 4.5 microstrain/ °F were obtained in the morning and afternoon sections, respectively. A difference of 0.5 microstrain/ °F was observed between morning and afternoon sections. It is well known that concrete paste has higher CTE than coarse aggregate. [6,7]. It could be that, when concrete was put into INC in the morning section, the proportion of cement paste was larger than that in the mix design. The CTE value of concrete with this coarse aggregate as evaluated by CSTMP was less than 4.6 microstrain/ °F. For the accurate evaluation of in-situ CTE, efforts should be made to ensure that the proportions of the concrete mixture in the non-stress cylinders are similar to those of the concrete supplied. This finding also illustrates the requirement of a fixed mix design for the evaluation of CTE of coarse aggregate.

| Figure 3.16-(a) CTE from INC in morning section | Figure 3.16-(b) CTE from INC in afternoon section |
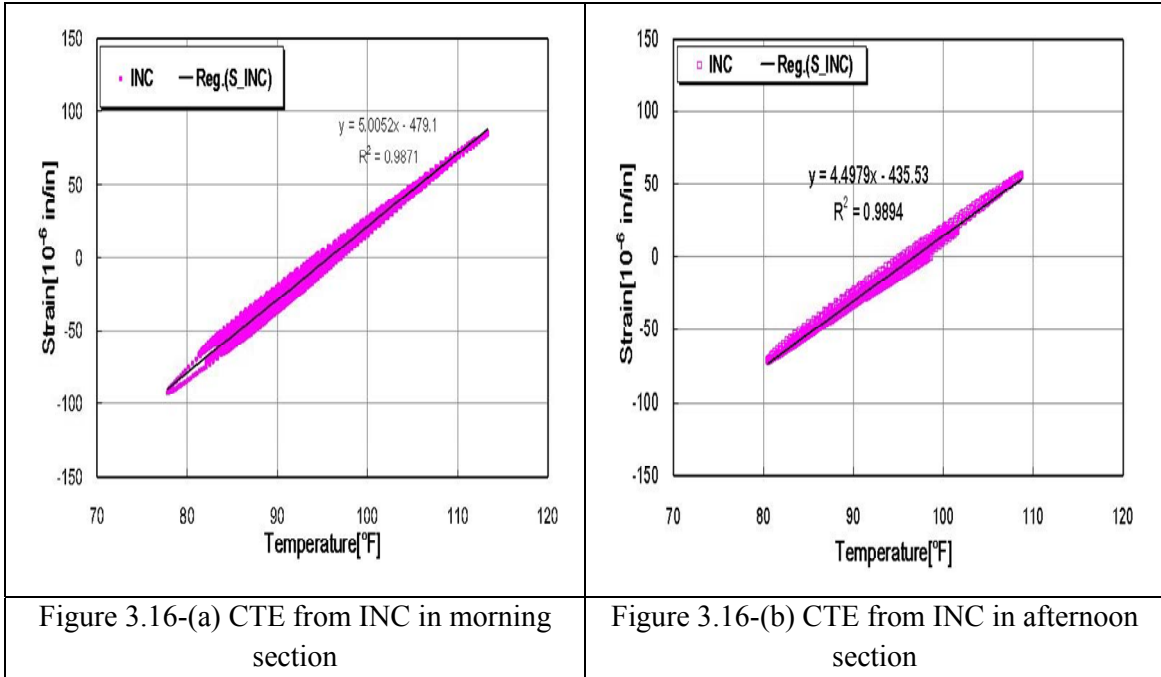
3.3.2.3.2 In-situ drying shrinkage evaluation: In-situ drying shrinkage was estimated from the data in INC and PNC in the morning and afternoon sections, as shown in Figure 3.17-(a) and 3.17-(b), respectively. Since the NSCs were installed in the lower part of the CRCP overlay (the center of NSCs was 4 in. from the surface of the slab), it is expected that the drying shrinkage will not be high. Figures 3.17-(a) and 3.17-(b) show that, overall drying shrinkage is quite small, less than 15 and 60 microstrains in the morning and afternoon sections, respectively. These values are much smaller than drying shrinkage of concrete exposed to normal ambient condition. Wet-mat curing of the concrete surface might have contributed to the small drying shrinkage. The smaller drying shrinkage in the morning section could be due to the larger CTE value in that section compared with that in the afternoon section.

| Figure 3.17-(a) Drying shrinkage in morning section | Figure 3.17-(b) Drying shrinkage in afternoon section |

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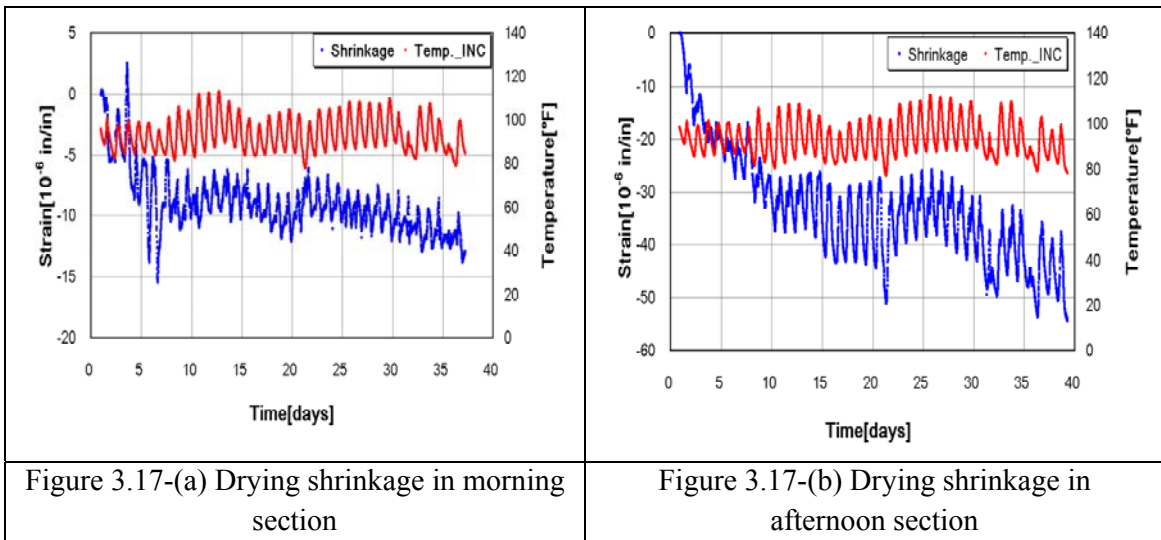
3.3.3 Difference in Phase 1 and Phase 2

In Phase 1, longitudinal steel in the main overlaid area continued to the end of the project limit, which means the longitudinal steel was placed in the transition areas. On the other hand, in Phase 2, longitudinal steel was not placed in the transition areas. Long-term performance of the transition areas in inside (phase 1) and outside (phase 2) lanes will be monitored.

3.4 Structural Evaluation of Early Age BCO

The primary benefit of bonded concrete overlay (BCO) is to enhance the structural capacity of the existing pavement system. The structural capacity of the JCP was evaluated using FWD (falling weight deflectometer) along the project before and after the overlay. Figure 3.18-(a) shows the testing results in the FWD deflections at the mid-slab conducted in the outside lane. The distance is from the beginning of the project in the north limit. Before overlay, overall deflections were large, and deflections larger than 10 mils were observed in several locations. After overlay, the deflections were reduced substantially. The average deflection at 9,000 lb loading after overlay was 1.02 mils. Figure 3.18-(b) illustrates the deflections in CRCP with various slab thicknesses in Texas. This information was obtained from the testing in Research Project 0-6274. The average deflection of 1.02 mils after overlay in this project is comparable to the deflections of 14 in. or 15 in. CRCP. The performance of 14 in. and 15 in. CRCP in Texas has been excellent and it is expected that this rehabilitation section will provide good performance for years to come. This section will be included in 0-6274 as an experimental section and deflection testing will be conducted on a regular basis.

Figure 3.18-(a) FWD deflections before and after overlay	Figure 3.18-(b) Statewide CRCP slab thickness vs deflections
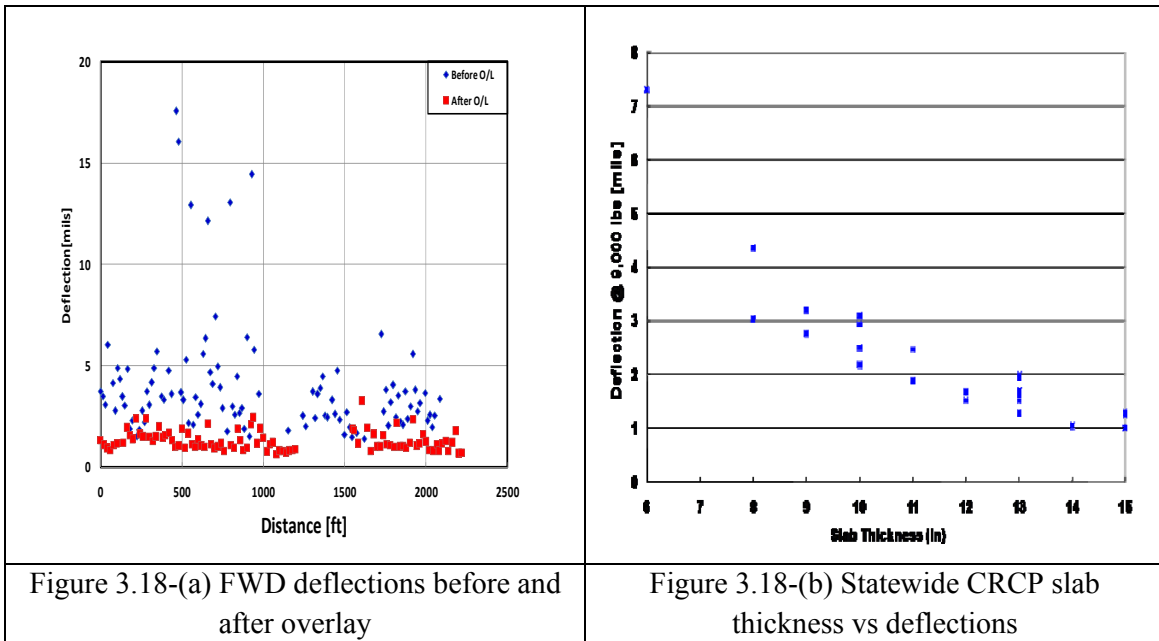
Chapter 4: Summary and Conclusions

There have been distresses and performance issues in jointed plain concrete pavement (JCP) in Sherman, Paris District. The pavement, 10 in. slab on 6 in. flexible base and lime treated subgrade, was built in 1984. Major distresses include transverse cracking and faulting in the longitudinal joint between the outside lane and outside shoulder. Forensic investigations revealed that the subbase support was neither uniform nor adequate. Substantial repair work has been conducted over the years; however, the performance problem still exists. There are no ideal rehabilitation solutions to such problems as in US 75 in Sherman. Asphalt overlay is the most cost-effective rehabilitation method for deteriorated JCP; however, reflection cracking usually occurs within a few years, especially when the truck traffic is quite heavy as in this project, which will require additional repairs or rehabilitations. In FY2009, the Texas Department of Transportation (TxDOT) had about 4,000 lane miles of JCP, most of which were approaching or had already passed the end of their intended design lives. There is a strong need for an optimum rehabilitation strategy for these deteriorating JCP.

In Texas, continuously reinforced concrete pavement (CRCP) has provided excellent performance, except for localized distresses due to materials and construction quality issues. One of the characteristics of CRCP that JCP doesn't have is its flexibility thanks to the reinforcement in the mid-depth of the slab. The combination of CRCP on deteriorated JCP through bonded overlay could provide a good rehabilitation option for deteriorated JCP. CRCP bonded overlay was constructed on half a mile section of JCP on US 75 southbound from Exit 64 in Sherman. Slab thickness and longitudinal steel reinforcement were 7 in. and 0.7 %, respectively. To optimize the performance of this rehabilitation method, knowledge and experience gained over the years by the research team on CRCP through numerous research studies under the TxDOT research program were utilized. Some of the unique requirements in the specifications for this project were:

1) Thermal coefficient of concrete was limited to less than 4.6 microstrain/°F. This is based on the fact that one of the key findings regarding CRCP performance is that the smaller the volume change potential and modulus of elasticity of the concrete, the better the performance.

2) Wet mat curing was required for a minimum of 7 days. In bonded concrete overlay, it is reported that good interface bond between old and new concrete is an important factor. Concrete undergoes volume changes at early ages due to drying shrinkage. If drying shrinkage is not controlled, debonding might take place at the interface due to concrete volume changes at early ages when the bond strength is not sufficiently developed.

3) Longitudinal steel was placed 0.5 in. below the mid-depth of CRCP slab. In CRCP, longitudinal steel is normally placed at the mid-depth of the slab. In the bonded overlay, concrete volume changes at the bottom of the CRCP need to be minimized at early ages to prevent debonding while concrete bond strengthens. Placing longitudinal steel closer to the interface could restrain concrete volume changes better, thus

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enhancing the bond between old and new concrete.

4) Non-woven fabric, 2-ft wide, was placed on transverse contraction joints throughout the lane width. Its objective was to minimize the potential for reflection cracking. The use of this fabric will break the bond between JCP and CRCP. Considering the deleterious effects of reflection cracking and debonding, a decision was made to use non-woven fabric. This decision was partly based on the finding that delaminated concrete at the depth of the reinforcing steel on IH 35 provided satisfactory performance for 10 years without distress, even though there was not reinforcement anymore in the top concrete layer (total CRCP thickness of 14 in.). In this project, 0.7 % of reinforcement was provided in the 7 in. CRCP, and it was postulated that this pavement should provide better performance.

5) Vertical hook bars were installed in the transition area to enhance the structural rigidity and bonding between old and new concrete. This decision was based on the distresses observed on the US 281 bonded overlay project, where distresses were observed in the transition area, even though its overall performance has been excellent.

Various gages were installed in the first phase of the project (inside lane and inside shoulder) to evaluate the behavior of CRCP overlay on JCP system due to environmental loading (temperature and moisture variations). No field experiment was conducted in the second phase of the construction (outside lane and outside shoulder) due to limitations in funding. Gages installed included steel strain gages, vibrating wire strain gages, thermo-couples and concrete relative humidity gages. One of the primary objectives of this field testing was to understand detailed behavior of this overlay system and improve the design and construction practices further, if needed. It will take at least one seasonal cycle to fully understand the behavior of this pavement system. Also, deflection testing was conducted before and after the overlay with falling weight deflectometer (FWD). Findings made so far can be summarized as follows:

1) Load transfer efficiency (LTE) at transverse contraction joints varied substantially, from very low to high, even though dowels were used. However, faulting was not noticed.

2) 7 in. CRCP overlay reduced the deflections substantially. The average deflection in the overlaid system was 1.02 mils at 9,000 lb loading, which is comparable to 14 in. or 15 in. thick CRCP.

3) In-situ drying shrinkage was quite small. It appears that the use of wet mat curing helped reduce the drying shrinkage. In-situ coefficient of thermal expansion (CTE) was within the specification requirement, even though one test data shows a little higher than the requirement.

4) Concrete temperature variations through the slab depth within 7 in. CRCP appear to be smaller than those in normal CRCP slab. The smaller temperature variations through slab depth resulted in axial concrete displacement in CRCP slab, rather than curling type displacement. These concrete temperature variations will help improve

34

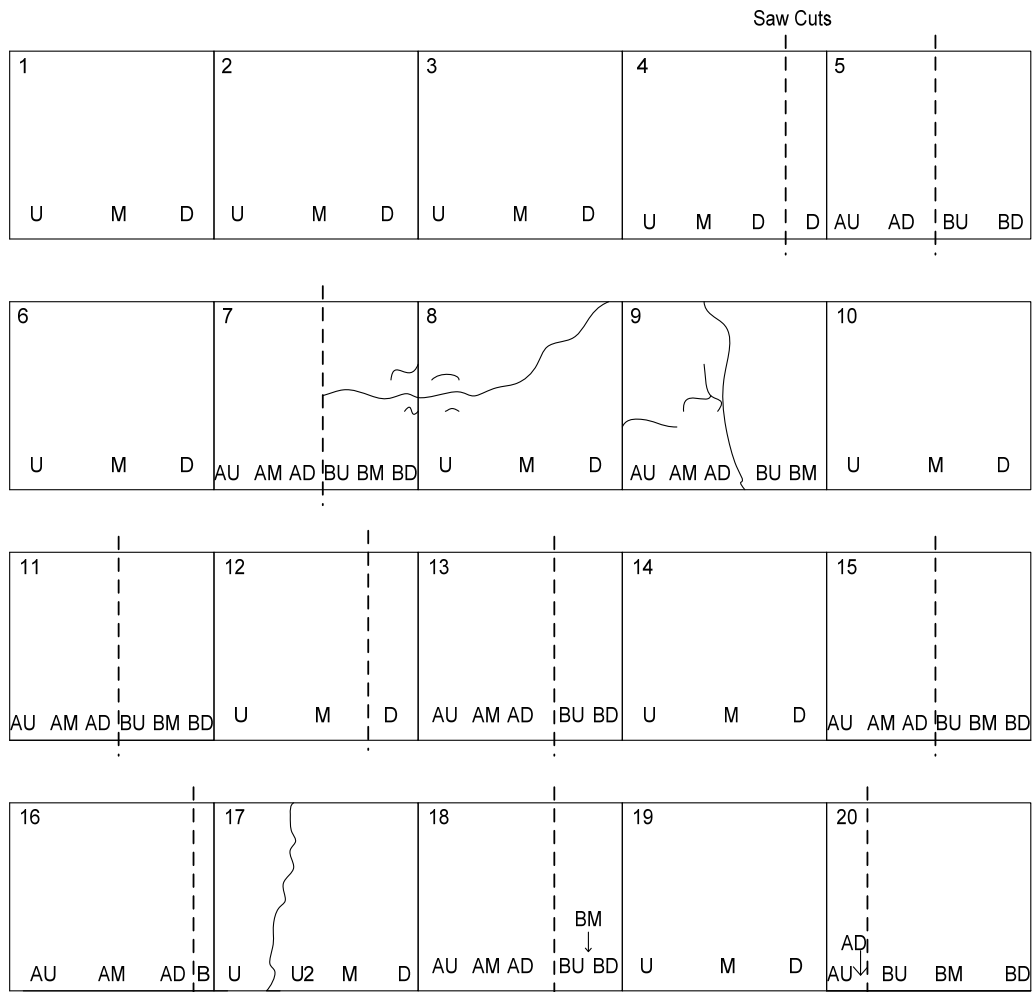bonded concrete overlay performance by minimizing curling-induced slab deformations.

5) The behavior of transverse cracks in CRCP overlay is different from that of transverse cracks in normal CRCP. The crack width variations through the depth of the slab in normal CRCP indicate bending action with a neutral axis a little below the depth of the steel (mid-depth of the slab). On the other hand, those in CRCP BCO indicate bending action with a neutral axis below the bottom of the CRCP slab. The difference in crack width variations through the slab depth between BCO and normal CRCP appears to be due to the interface bonding condition at the bottom of CRCP and the layer below.

If TxDOT considers the use of this rehabilitation method for deteriorated JCP, it is recommended that the use of light weight aggregate (LWA) be considered as coarse aggregate or as intermediate aggregate. The use of LWA in concrete will reduce CTE, modulus of elasticity and drying shrinkage of concrete, thus potentially improving the performance of this rehabilitation method.

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Saw Cuts

1			2			3			4			5			
U	M	D	U	M	D	U	M	D	U	M	D	D	AU	AD	BU	BD

| 6 | | | 7 | | | | | | 8 | | | 9 | | | | | 10 | | |
| U | M | D | AU | AM | AD | BU | BM | BD | U | M | D | AU | AM | AD | BU | BM | U | M | D |

| 11 | | | | | | 12 | | | 13 | | | | 14 | | | 15 | | | | |
| AU | AM | AD | BU | BM | BD | U | M | D | AU | AM | AD | BU | BD | U | M | D | AU | AM | AD | BU | BM | BD |

| 16 | | | | 17 | | | | 18 | | | | | 19 | | | 20 | | | | |
| AU | AM | AD | B | U | U2 | M | D | AU | AM | AD | BU | BD | U | M | D | AD | AU | BU | BM | BD |

BM

37

21	22	23	24	25
U　　M　　D	AU AM AD　BU BD	U　　M　　D	U　　M　　D	AU　AD　BU　BD

26	27	28	29	30
U　　M　　D				U　　M　　D

31	32	33	34	35
U　　M　　D	U　　M　　D	U　　M　　D	U　　M　　D	U　　M　　D

36	37	38	39	40
U　　M　　D	U　　M　　D	U　　M　　D	U　　M　　D	U　　M　　D

41	42	43	44	45
U M D	U M D	U M D	U M D	U M D

46	47	48	49	50
U M D	U M D	U M D	U M D	U M D

51	52	53	54	55
U M D	U M D	U M D	U M D	U M D

56	57	58	59	60
U M D	U M D	U M D	M D	U M D

| 61 | 62 | 63 | 64 | 65 |
| U   M   D | U   M   D | U   M   D | U   M   D | AU   AD   D |

| 66 | 67 | 68 | 69 | 70 |
| U   M   D U   D | U   M   D | U   U   D | U   D U   D | U   D   B |

| 71 | 72 | 73 | 74 | 75 |
| A   BU   BD   C | U   D | AD   BU   BD | AU   AD   B | A   B   C   D |

| 76 | 77 | 78 | 79 | 80 |
| A   B   BD | U   M   D | A   BU   BM   BD | A   B   C   D | A   B   C   D |

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111				112				113				114			115				
A	B	C	D	A	B	C	D	A	B	C	D	A		B	C	A	B	B	C	C

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42
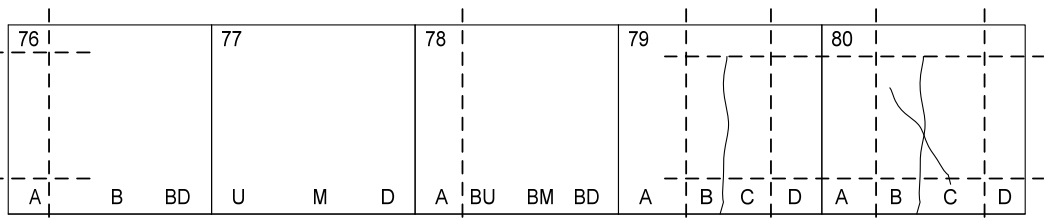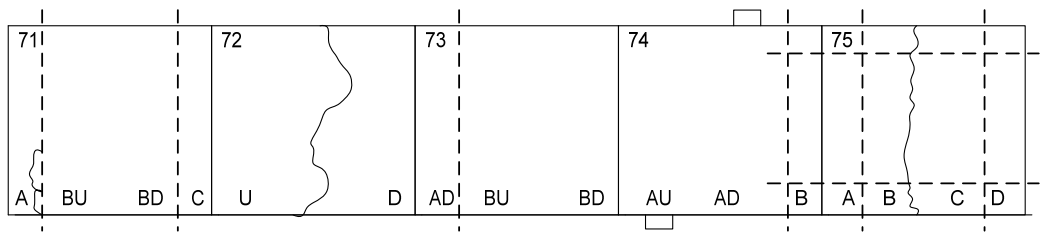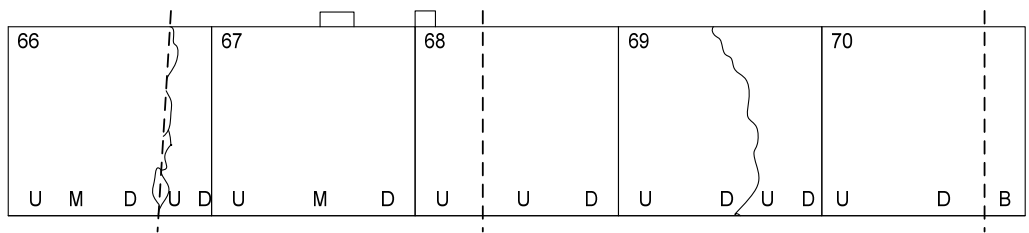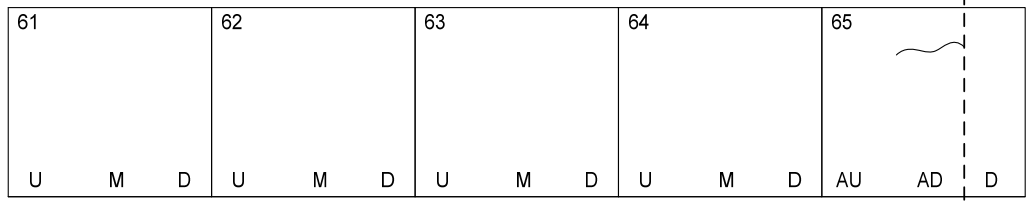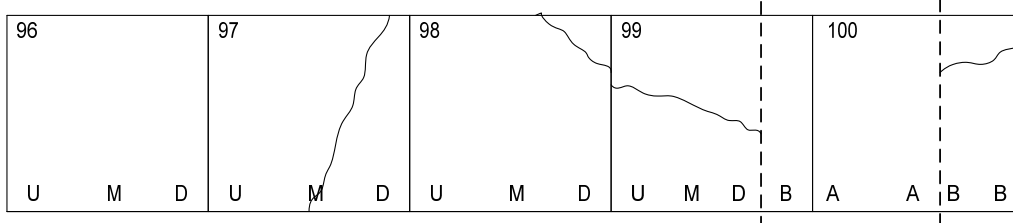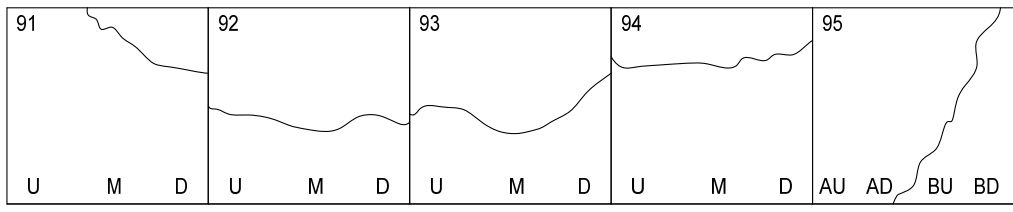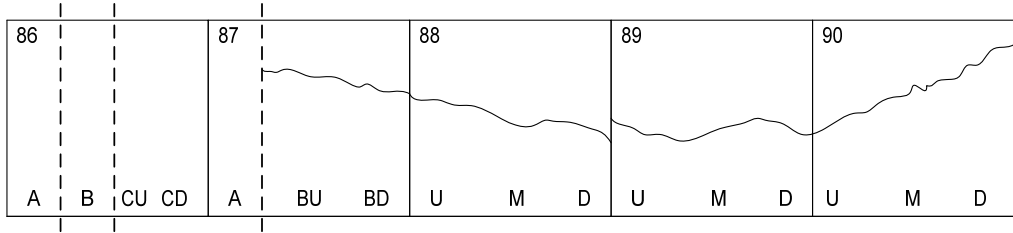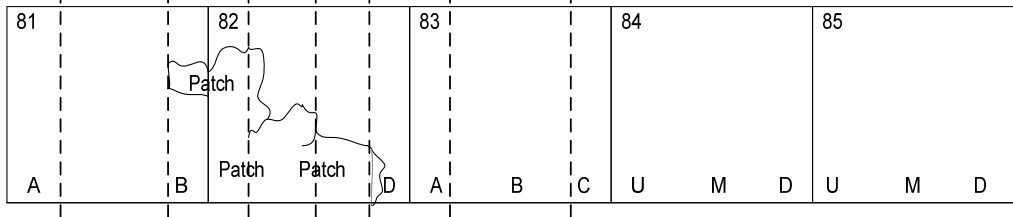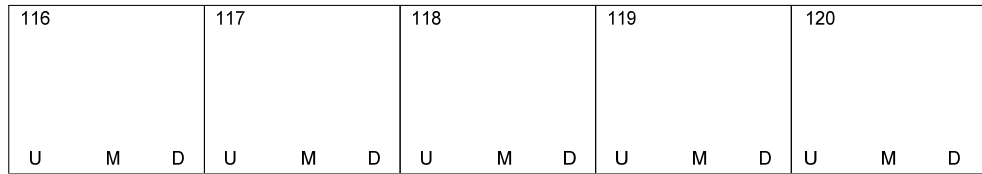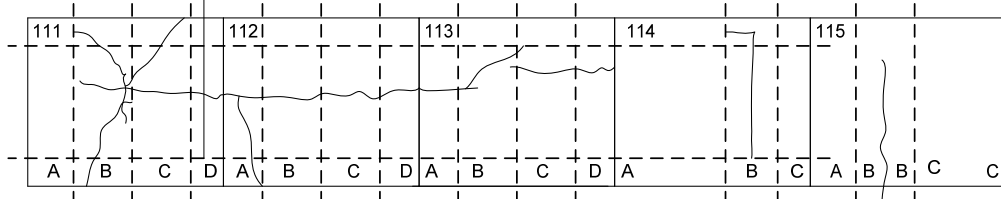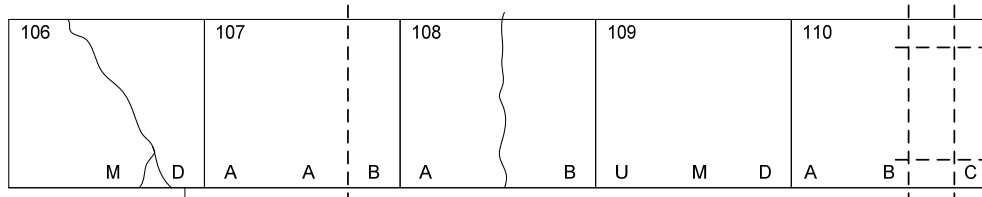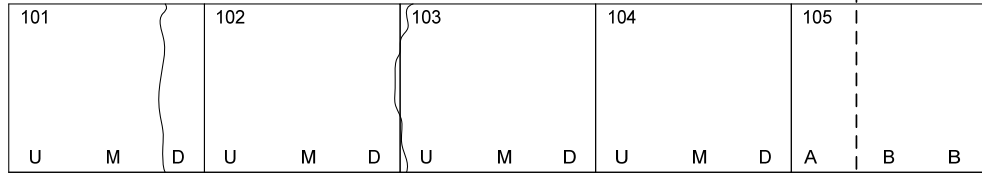
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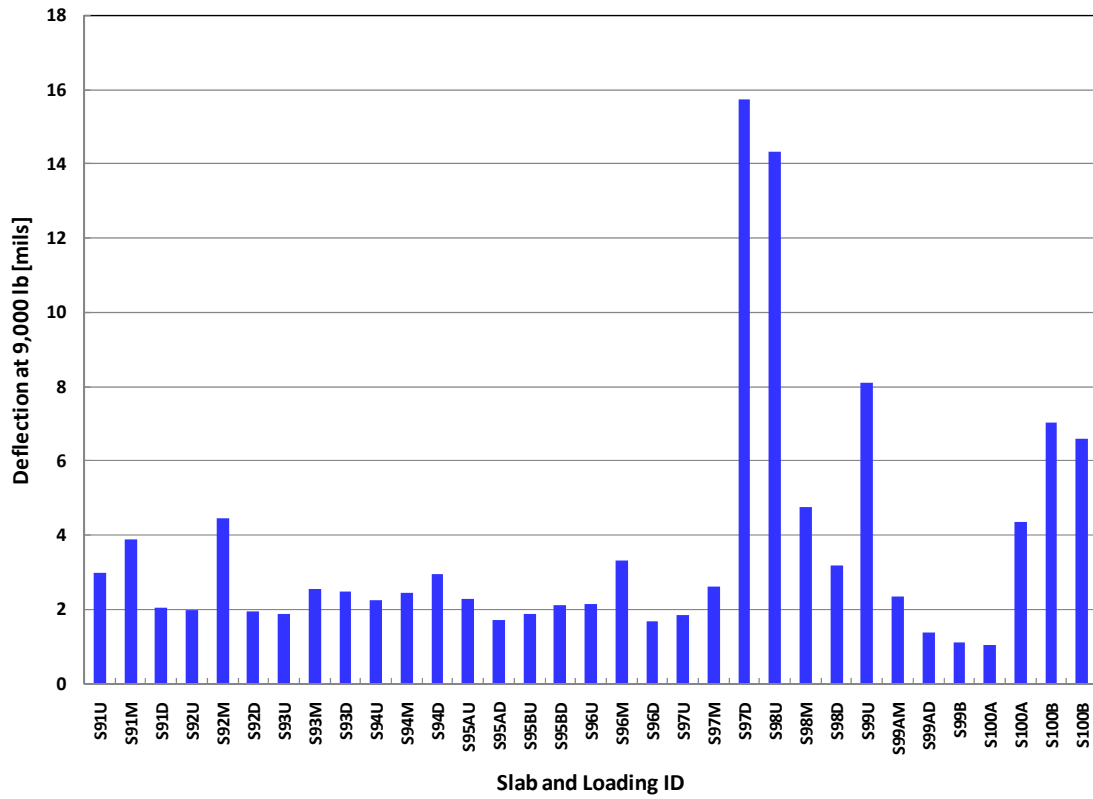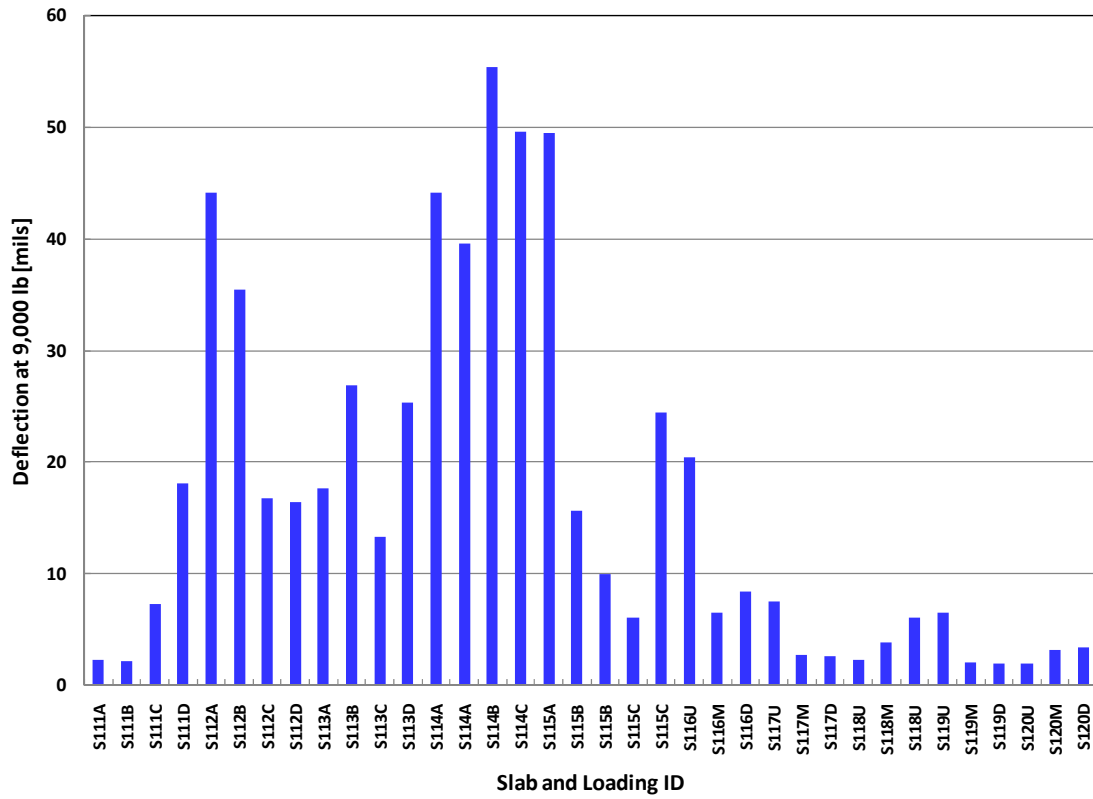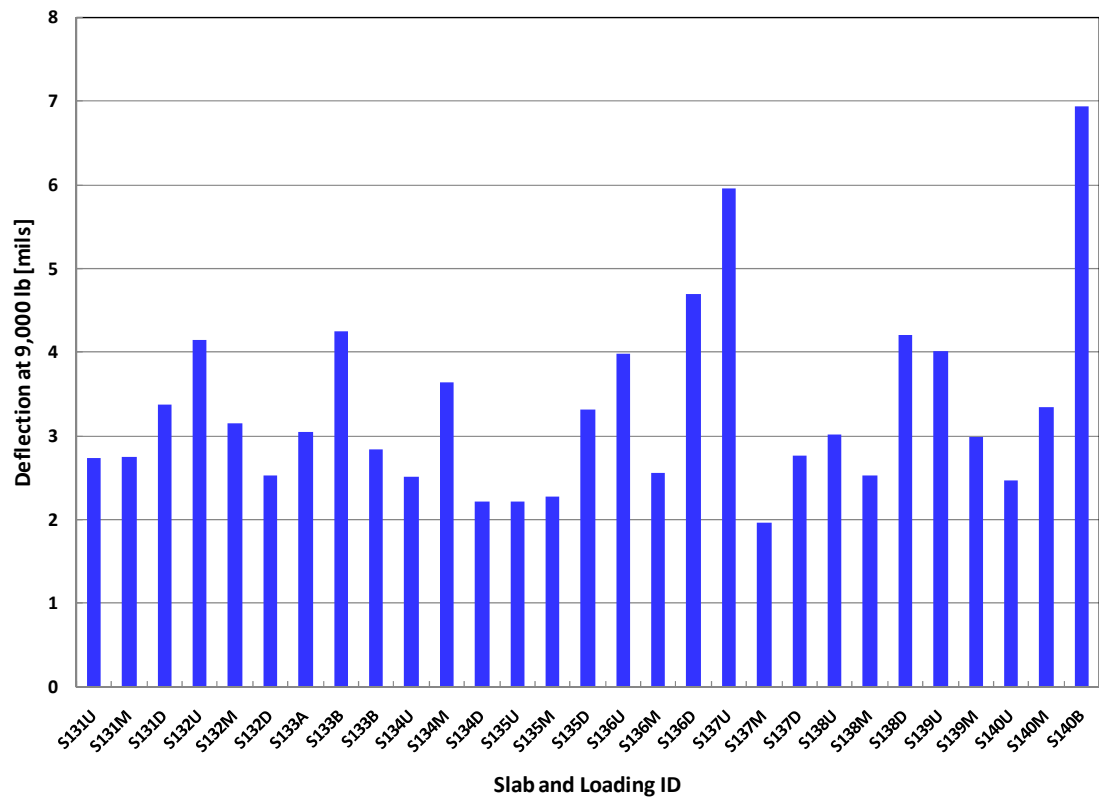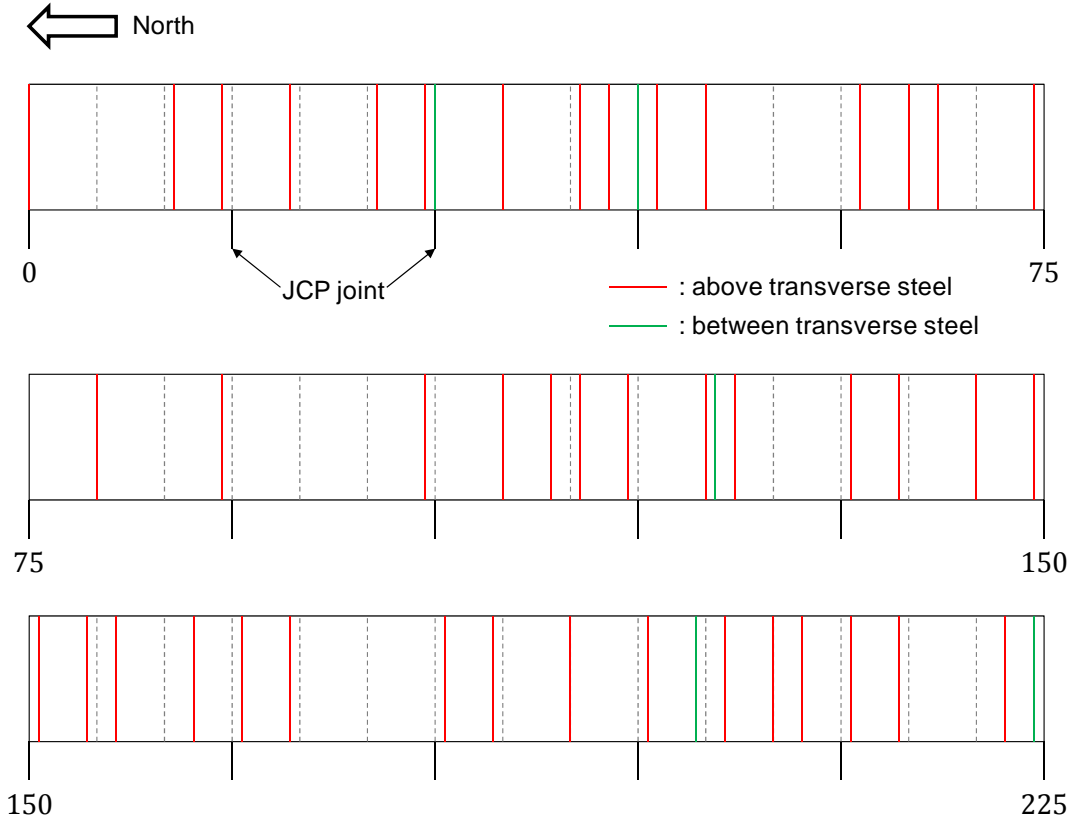| 136			137			138			139			140		
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APPENDIX B
FWD Deflections at Various Locations

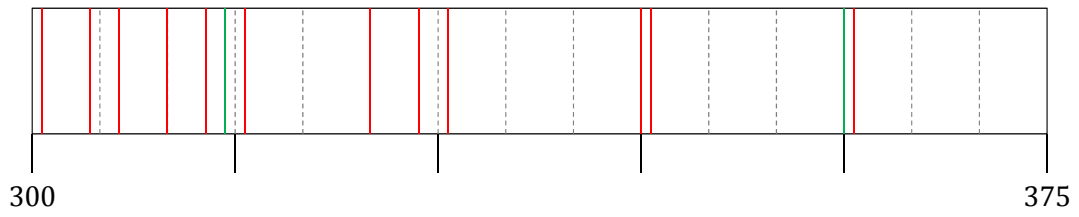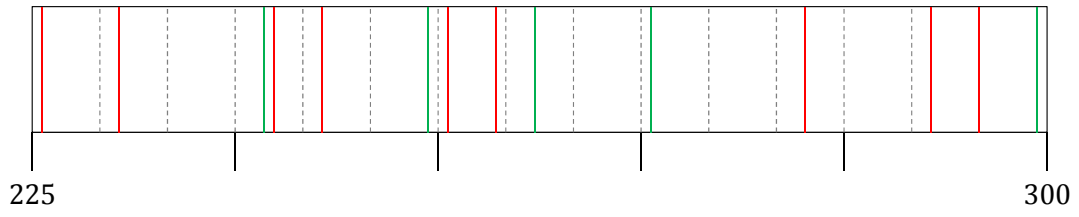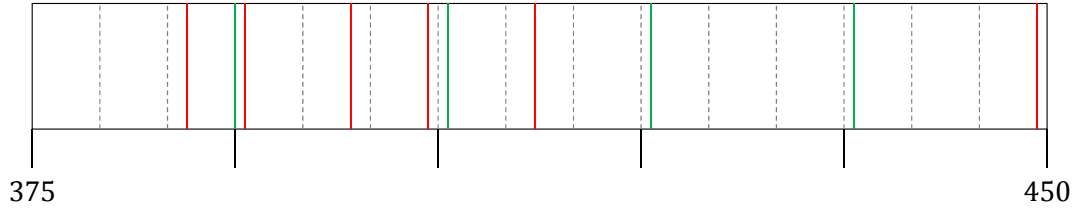Deflection at 9,000 lb [mils] vs Slab and Loading ID

Deflection at 9,000 lb [mils] vs Slab and Loading ID
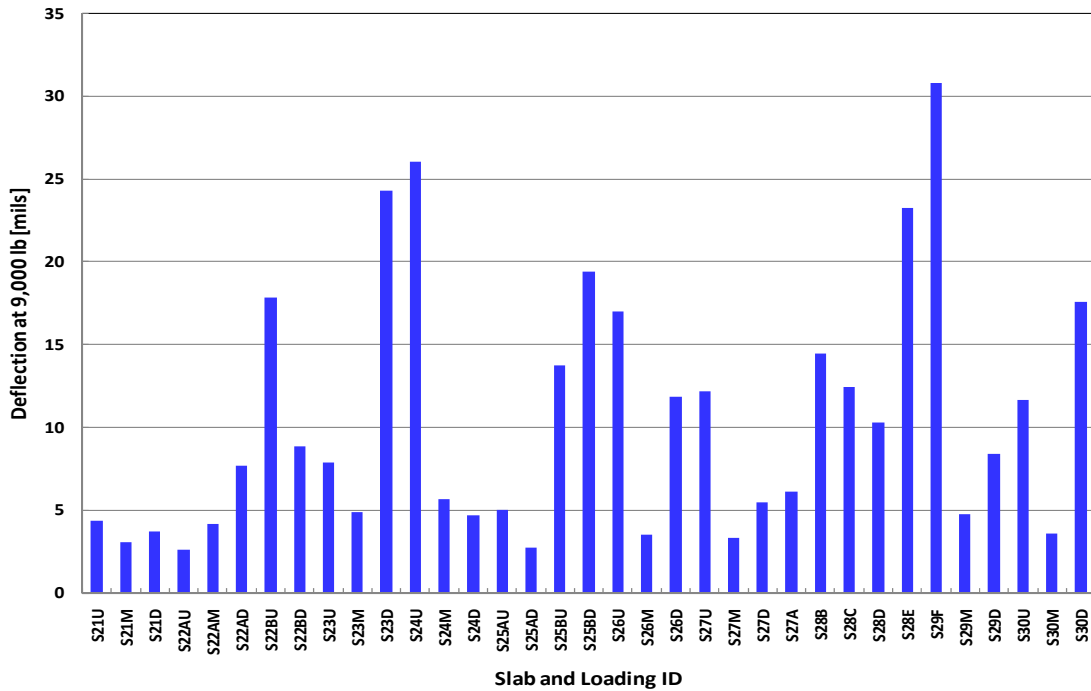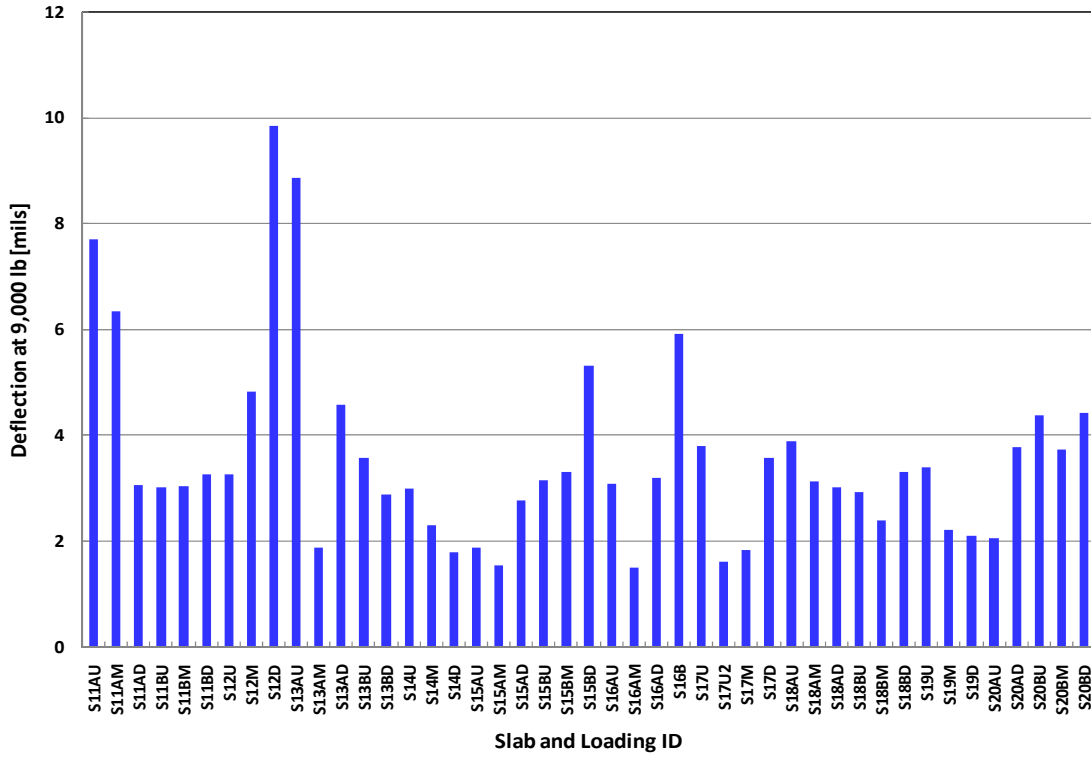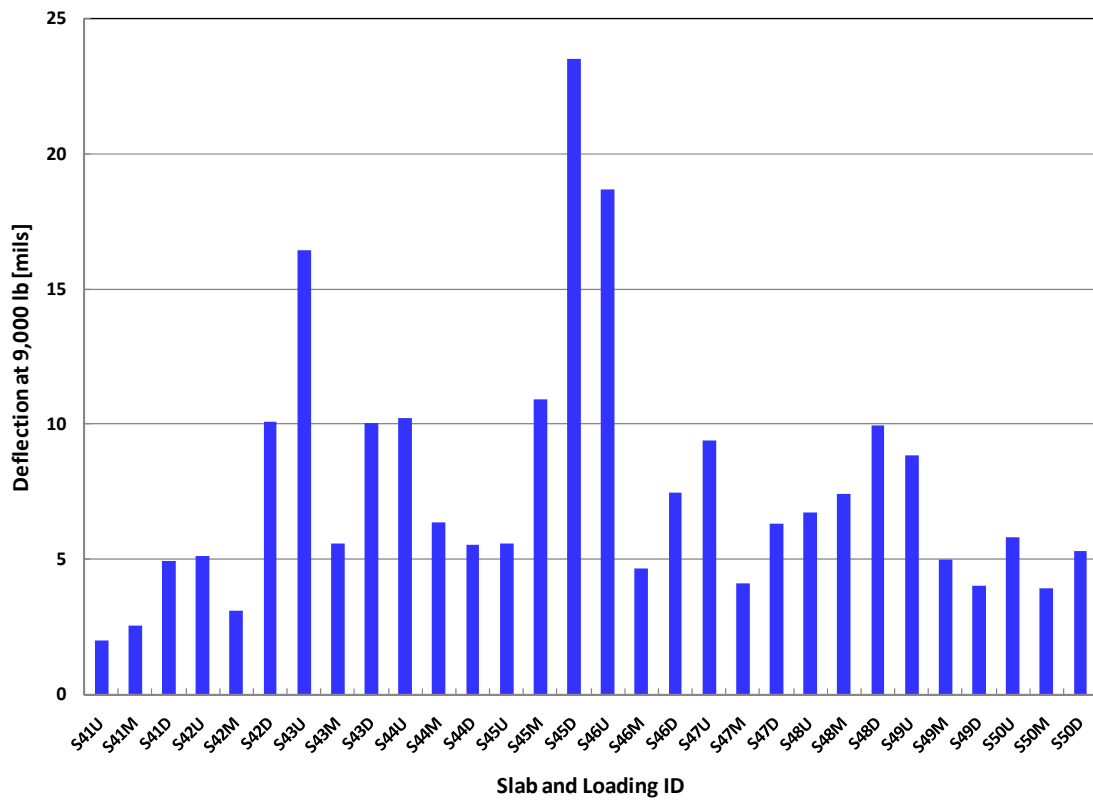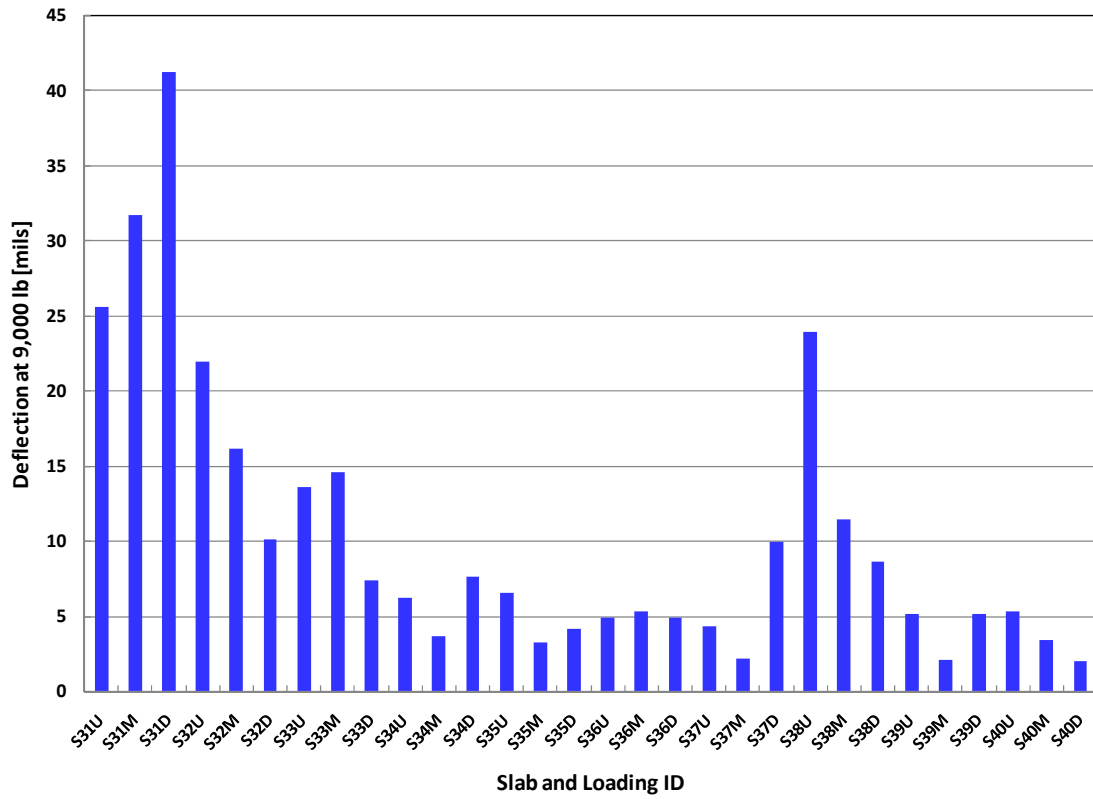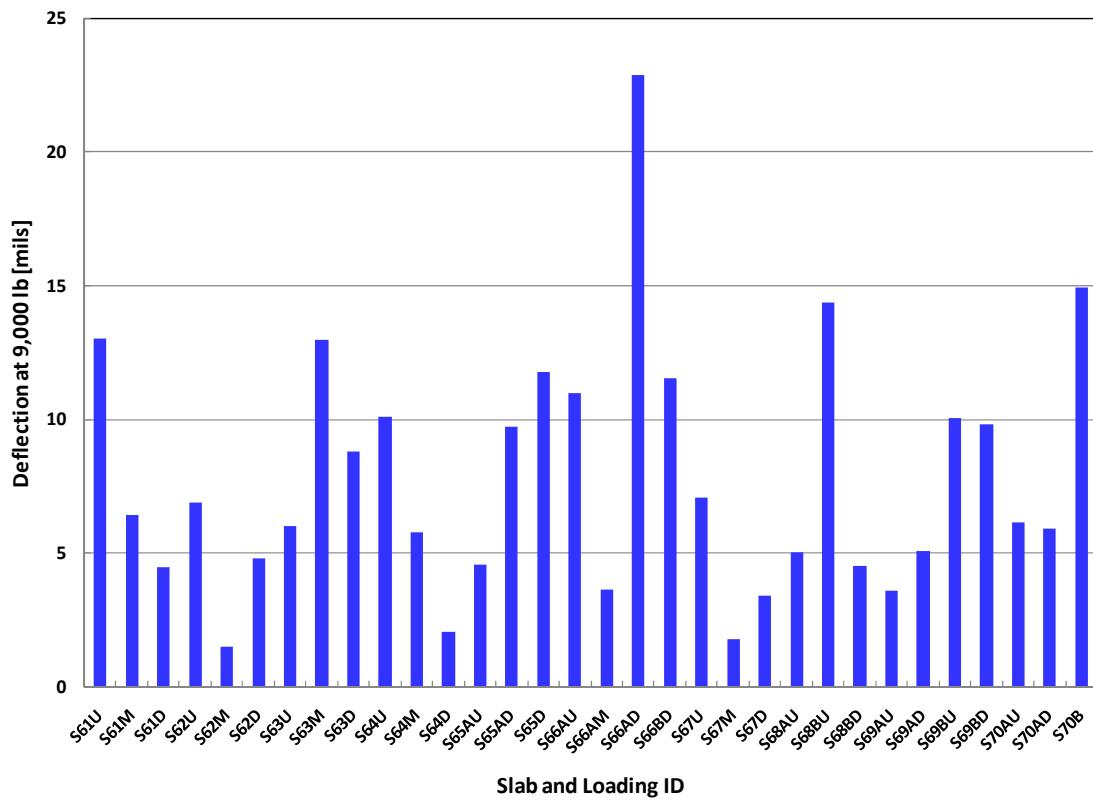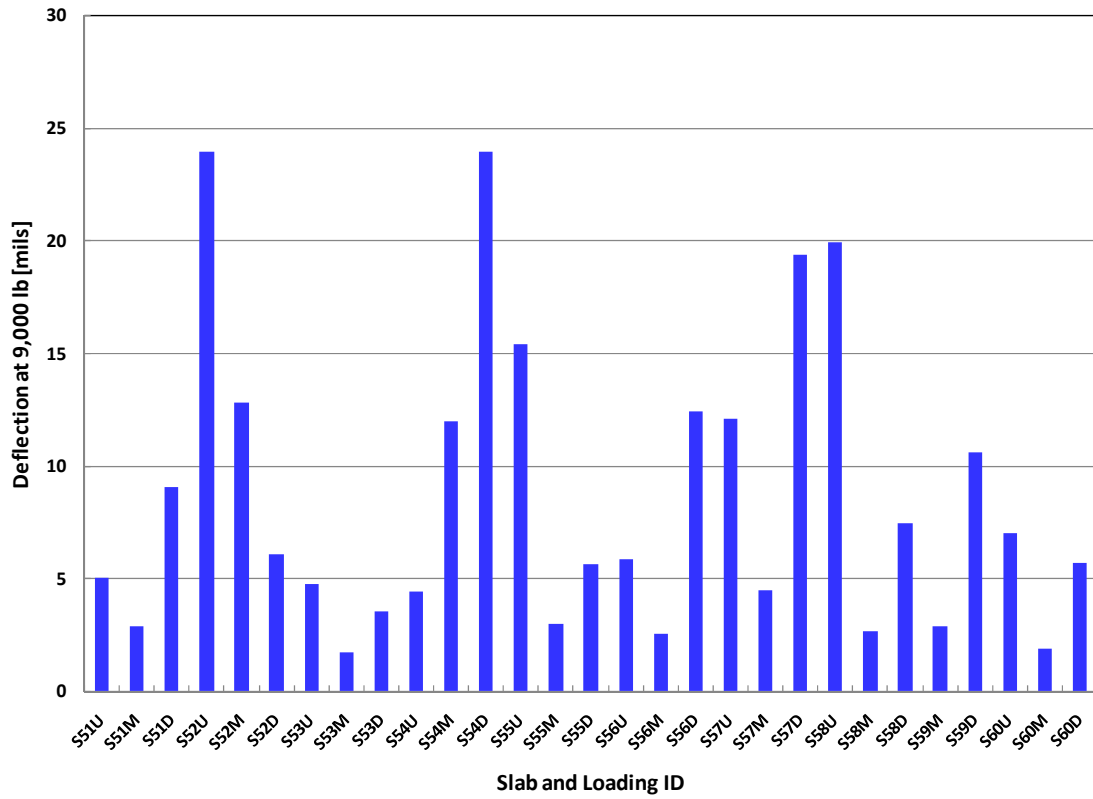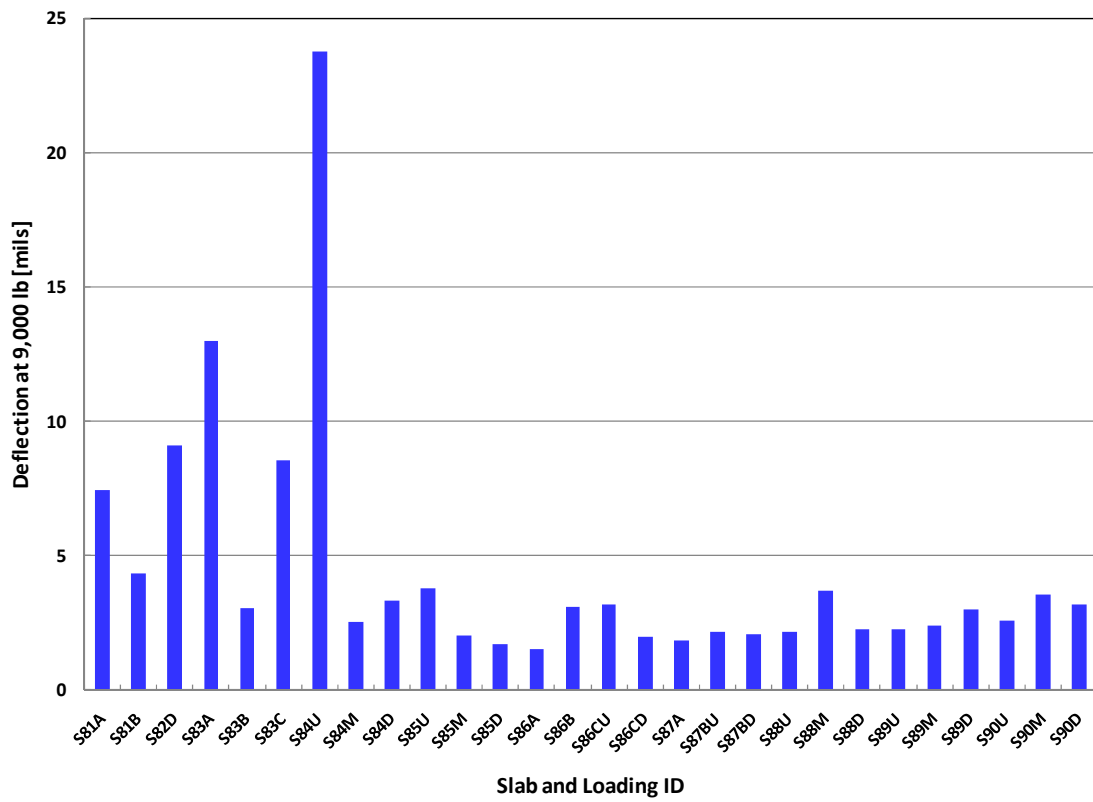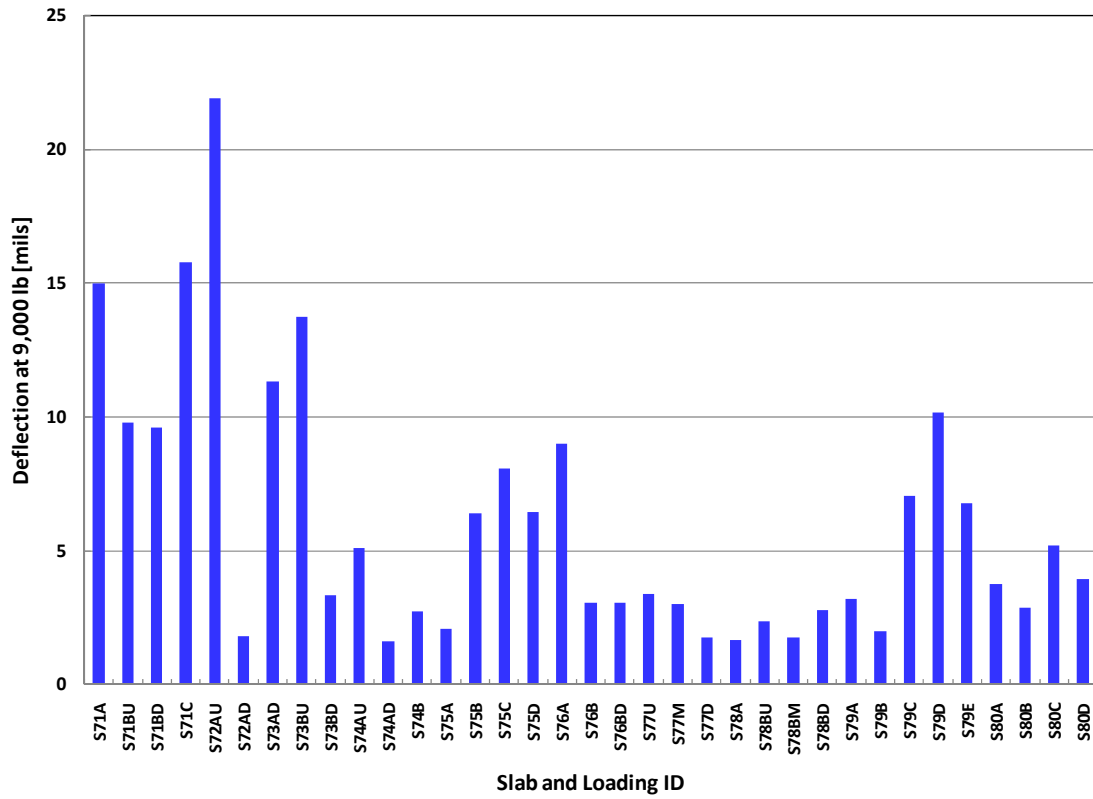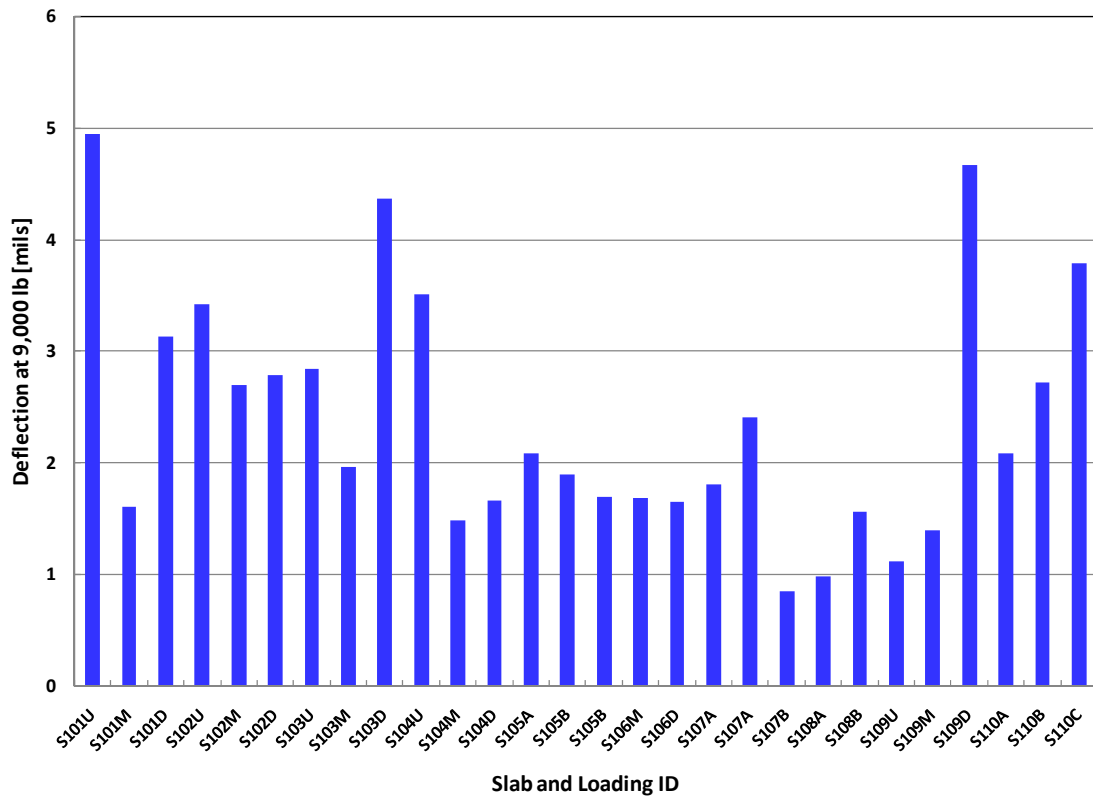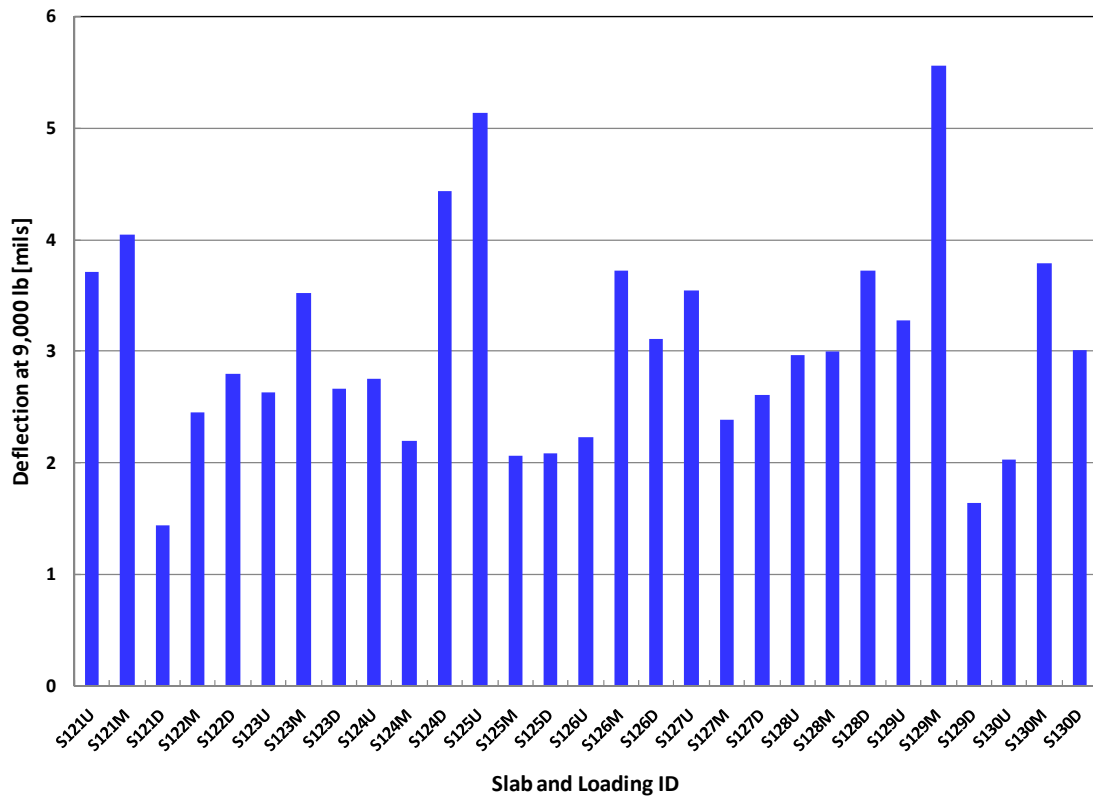
APPENDIX C
Cracking in BCO

North

0   75

JCP joint

―――― : above transverse steel
―――― : between transverse steel

75   150

150   225

51

225   300

300   375

Morning Section   Hook Bar

375   450

Hook Bar

450   525

525   600

600   675

675   750

750   825

825   900

900   975

975   1050

1050   1125

1125   1200

1200   1275

1275   1350

1350   1425

1425   1500

1500   1575

54

2025   2100

2100   2175

2175   2205

South
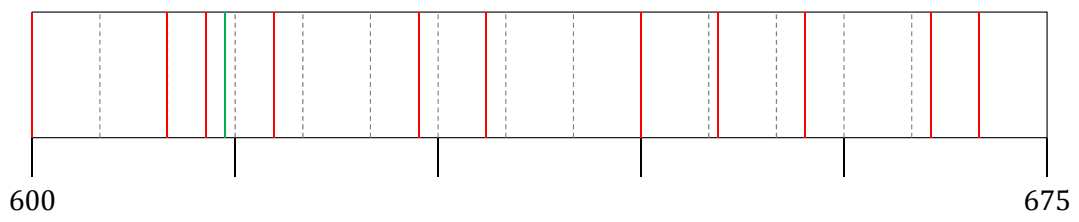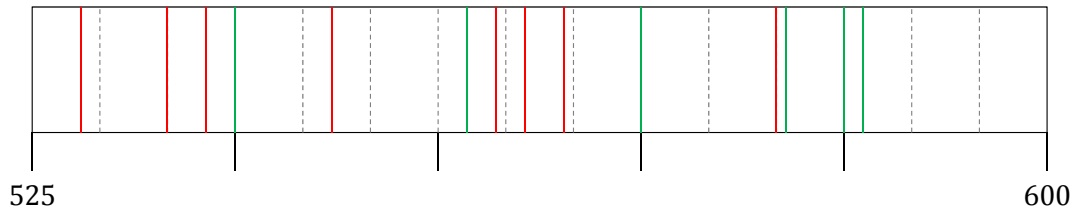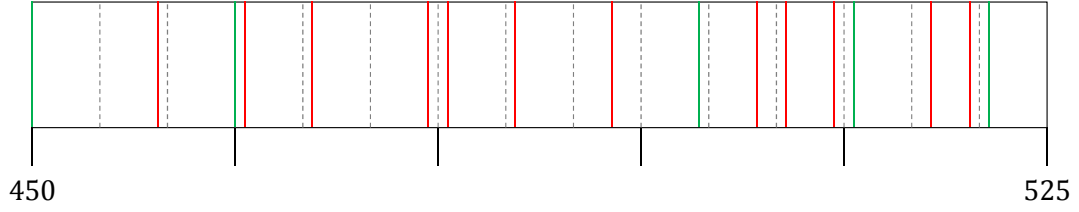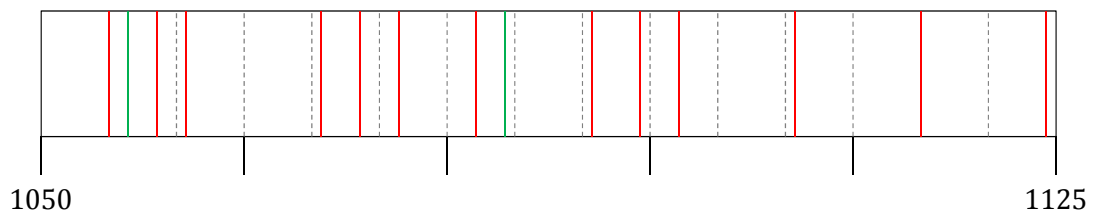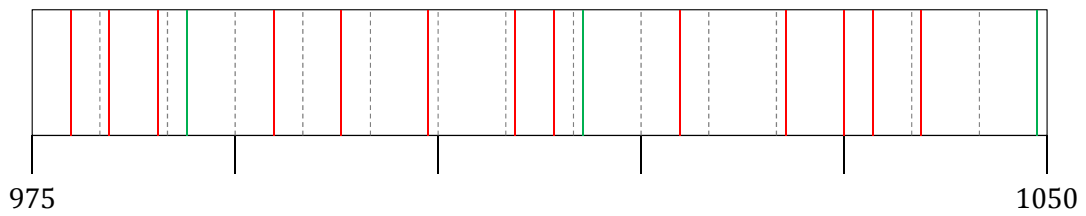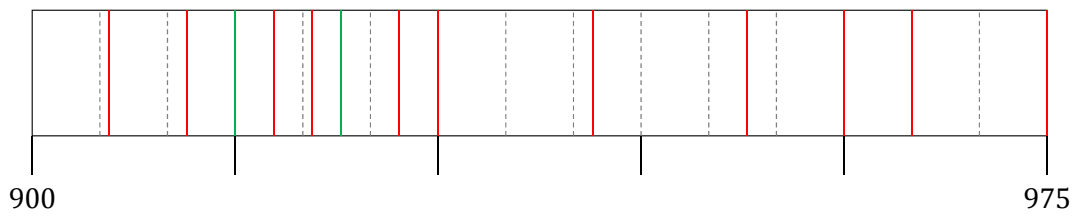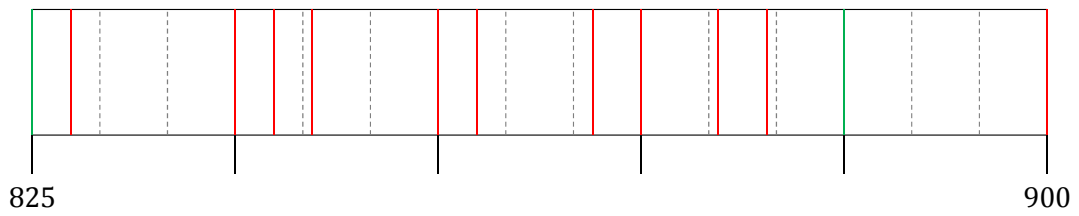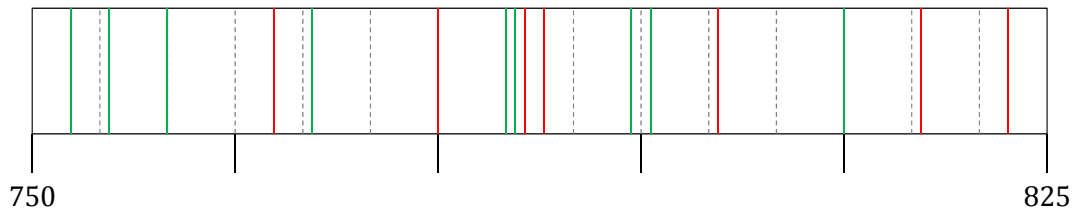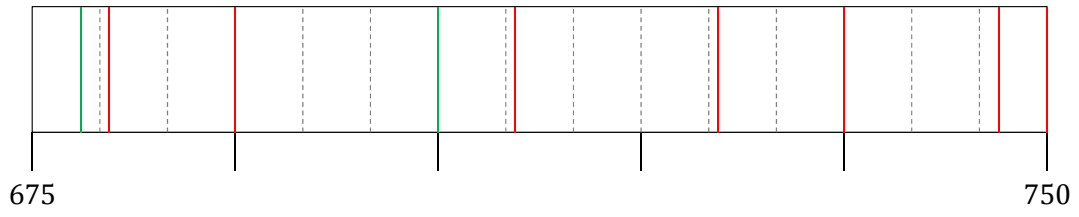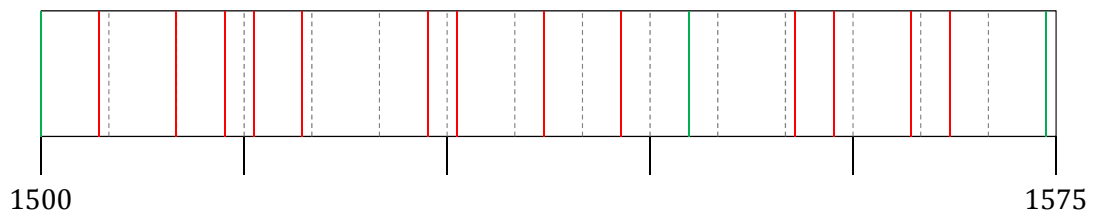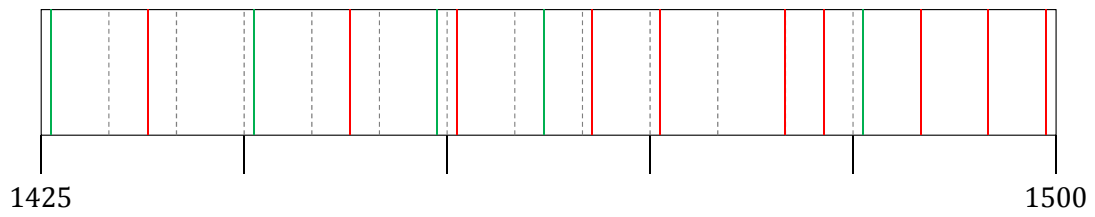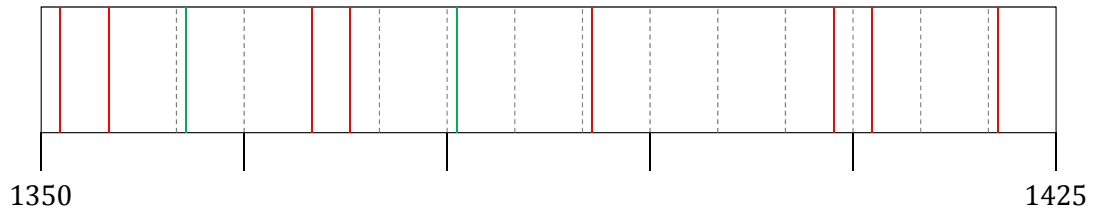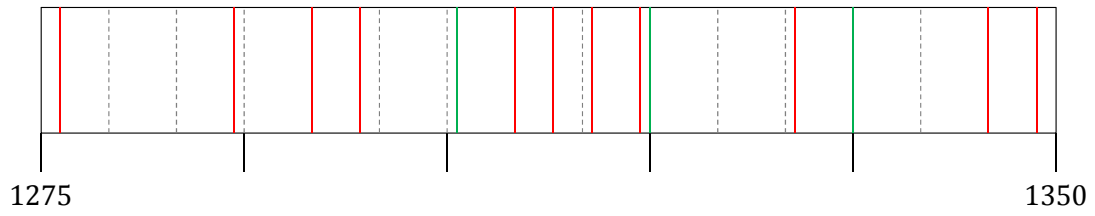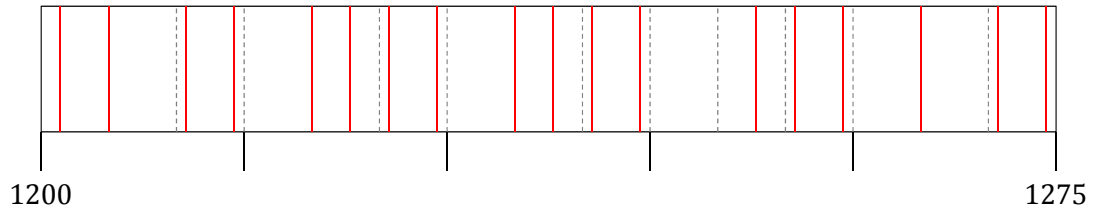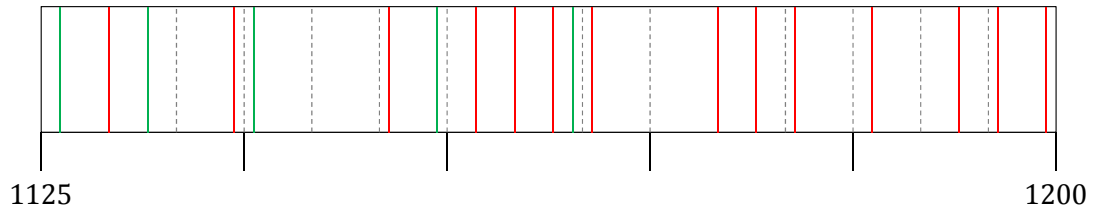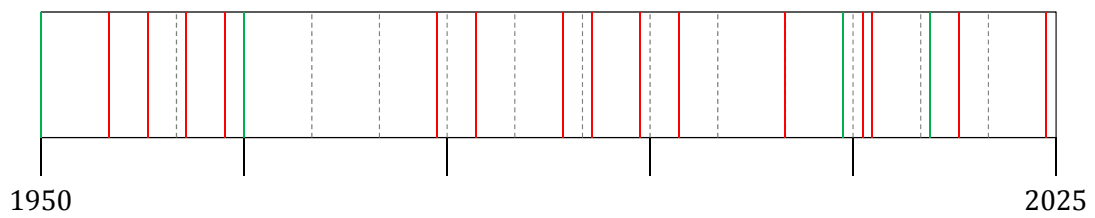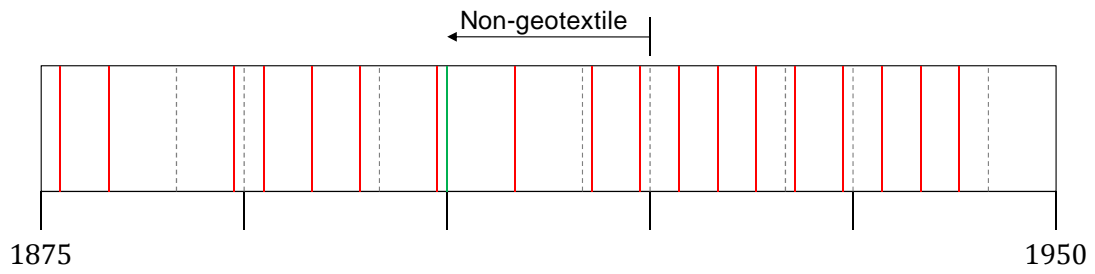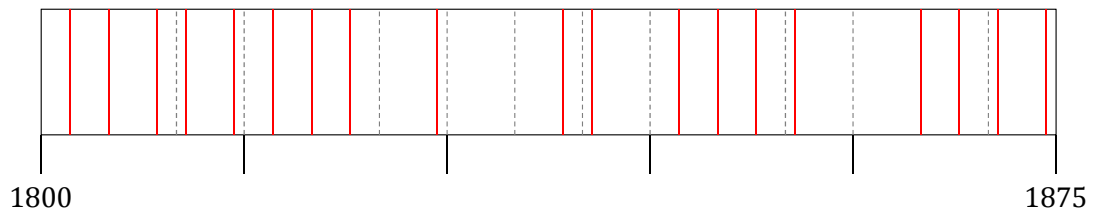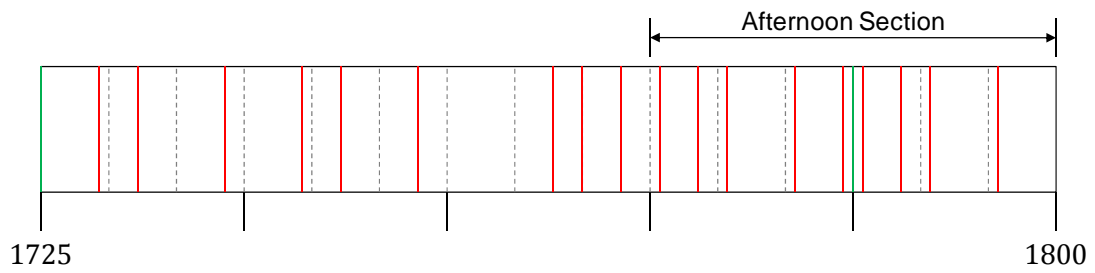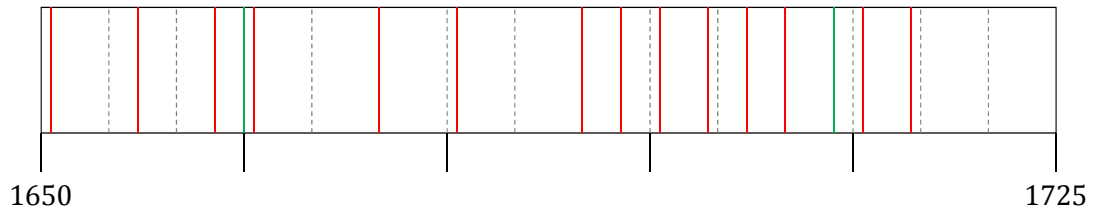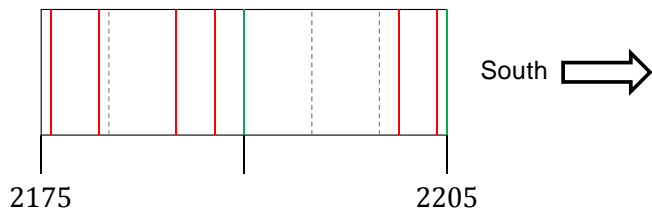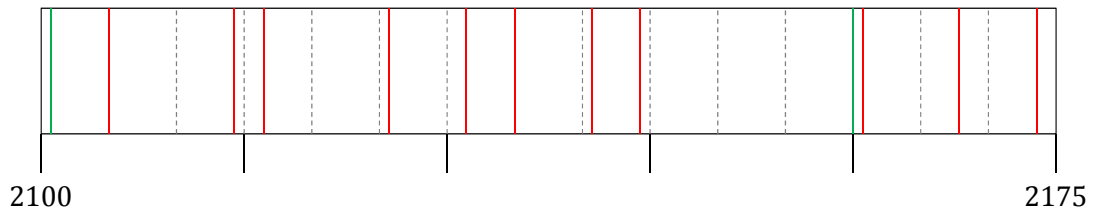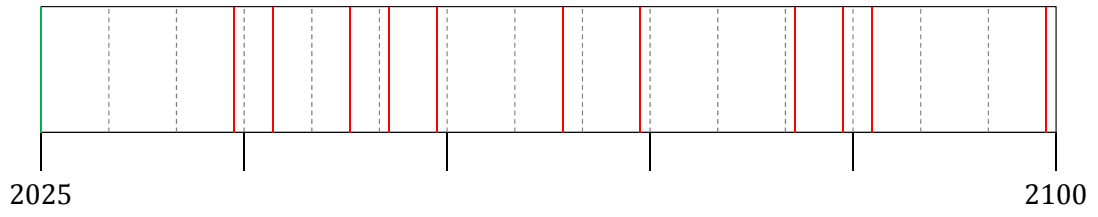
Appendix D

Special Provisions to Items 360 and 421

SPECIAL PROVISION
360---009
Concrete Pavement

For this project, Item 360, "Concrete Pavement," 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 360.1. Description is voided and replaced by the following:

360.1. Description. Construct continuously reinforced concrete pavement overlay over jointed concrete pavement in accordance with the typical sections shown on the plans.

Article 360.2. Materials, Section A. Hydraulic Cement Concrete is voided and replaced by the following:

A. Hydraulic Cement Concrete. Provide Class HES concrete in accordance with Item 421, "Hydraulic Cement Concrete." Design Class HES to meet the requirements of Class P concrete and a minimum average compressive strength of 2,600 psi in 24 hr., unless other early strength and time requirements are shown on the plans or allowed. No strength overdesign is required. Type III cement is allowed for Class HES concrete. Use coarse aggregate to produce concrete with a coefficient of thermal expansion (CoTE) less than 4.6 x 10^{-6} in/in/ F. Prior to construction, submit test specimens to TxDOT, Construction Division, for aggregate acceptance. Provide samples or test specimens as directed. TxDOT Construction Division will perform the testing and the test results are final.

Article 360.2. Materials, Section C. Curing Materials is voided and replaced by the following:

C. Curing Materials. Provide Type 2 membrane curing compound conforming to DMS-4650, "Hydraulic Cement Concrete Curing Materials and Evaporation Retardants." Provide cotton mats that consist of a filling material of cotton "bat" or "bats" (at least 12 oz. per square yard) completely covered with unsized cloth (at least 6 oz. per square yard) stitched longitudinally with continuous parallel rows of stitching spaced at less than 4 in., or tuft both longitudinally and transversely at intervals less than 3 in. Provide cotton mats that are free from tears and in good general condition. Provide a flap at least 6 in. wide consisting of 2 thicknesses of the covering and extending along 1 side of the mat.

Article 360.2. Materials is supplemented by the following:

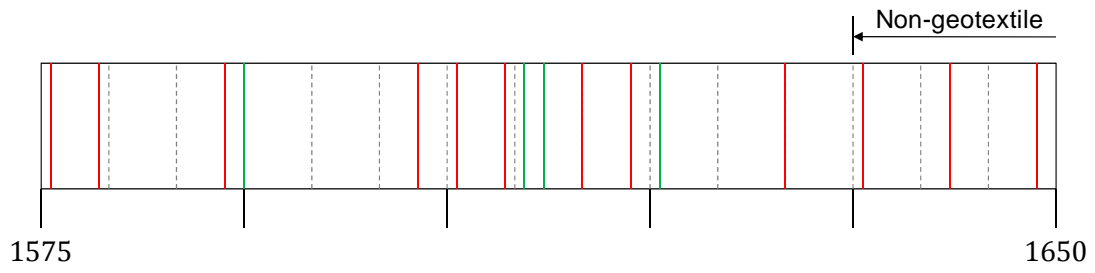**G. Nonwoven Geotextile Interlayer Material.** Provide nonwoven geotextile interlayer material that meets the following properties:

1) Mass per unit area: greater than 13.3 oz/sy and less than 16.2 oz/sy per ASTM D5261

2) Thickness under pressure: greater than 0.04 in. at 29 psi per ASTM D 5199

3) Wide-width tensile strength: greater than 685 lb/ft per ASTM D 4595

4) Wide-width maximum elongation: less than 130 % per ASTM D 4595

Article 360.3. Equipment, Section E. Curing Equipment. The third sentence is voided and replaced by the following:

Provide curing equipment that is independent of all other equipment when required to meet the requirements of Article 360.4.I, "Curing."

Article 360.3. Equipment is supplemented by the following:

K. Existing Concrete Pavement Surface Preparation Equipment. Provide power-operated water blasting or shot blasting equipment capable of removing dirt, oil, paint, membrane curing compound, and other foreign material, as well as any laitance or loose concrete from the surface receiving the new concrete. Provide power-operated equipment capable of milling concrete surface without damaging the concrete. Use only approved equipment for this operation. Dispose of waste material, including water, caused by this operation using a self contained, portable vacuum unit. Do not push, shove, or drag left over slush (waste material, etc.) over the surface of the travel lanes.

To minimize contamination, ensure the bonded concrete paving operation follows the hydro-cleaning operation unless otherwise directed. If the cleaned surface becomes contaminated, re-clean it at no additional cost to the Department.

Article 360.4. Construction, Section A. Paving and Quality Control Plan is voided and replaced by the following :

A. **Paving and Quality Control Plan and Surface Preparation.** Submit a paving and quality control plan for approval before beginning pavement construction operations. Include details of all operations in the concrete paving process, including longitudinal construction joint layout, sequencing, curing, lighting, early opening, leave-outs, sawing, inspection, testing, construction methods, other details and description of all equipment. List certified personnel performing the testing. Submit revisions to the paving and quality control plan for approval. Remove all dirt, oil, paint, laitance and loose concrete by shotblasting or hydrocleaning the entire concrete pavement surface to be overlaid with bonded concrete overlay. Begin the concrete placement within 8 hours following the shotblasting or hydrocleaning operation unless otherwise directed by the Engineer. Blast or re-clean the surface if it becomes contaminated. Ensure that the surface of existing concrete pavement is in damp condition but that there is no free water on the surface of the existing concrete when placing the concrete for the bonded concrete overlay.

Article 360.4. Construction, Section G. Concrete Placement, 4. Temperature Restrictions. The first paragraph is voided and replaced by the following:

Place concrete that is between 40°F and 90°F when measured in accordance with Tex-422-A at the time of discharge. Do not allow the temperature of the paving concrete to exceed 90°F when placing. Once the concrete temperature reaches 90°F at the time of discharge at the job site, take immediate corrective action to reduce and control the concrete temperature or stop concrete production. The Contractor is allowed to transport and place concrete produced up to the time of

notification of the high concrete temperature. Sample the concrete temperature after it is discharged on grade.

Take special measures when the difference in the ambient temperature following the time of placement versus the expected low temperature in a 24 hour period is expected to exceed 25°F. Place the bonded concrete overlay a minimum of 18 hours prior to the time the maximum temperature difference is expected.

The Contractor may perform paving at night.

Article 360.4. Construction, Section H. Spreading and Finishing, Section 3. Surface Texturing is voided and replaced by the following:

Complete final texturing before the concrete has attained its initial set. Drag the carpet longitudinally along the pavement surface with the carpet contact surface area adjusted to provide a satisfactory coarsely textured surface. Prevent the carpet from getting plugged with grout. Do not perform carpet dragging operations while there is excessive bleed water.

A metal-tine texture finish is required unless otherwise shown on the plans. Immediately following the carpet drag, apply a single coat of evaporation retardant at a rate recommended by the manufacturer. Provide the metal-tine finish immediately after the concrete surface has set enough for consistent tining. Operate the metal-tine device to obtain grooves spaced at 1/2 in., approximately 3/16 in. deep, with a minimum depth of 1/8 in., and approximately 1/12 in. wide. Do not overlap a previously tined area. For surfaces that do not have adequate texture, the Engineer may require corrective action, including diamond grinding or shot blasting. Use manual methods for achieving similar results on ramps and other irregular sections of pavements. Repair damage to the edge of the slab and joints immediately after texturing.

Article 360.4. Construction, Section I. Curing, 3. Curing Class HES Concrete is supplemented by the following:

Keep all exposed surfaces of the concrete wet continuously for the required curing time. Use water curing that meets the requirements for concrete mixing water in Section 421.2.D, "Water." Do not use seawater or water that stains or leaves an unsightly residue. Keep the concrete continuously wet by maintaining wet cotton mats in direct contact with the concrete for the required curing time. If needed, place damp burlap blankets made from 9-oz. stock on the damp concrete surface for temporary protection before applying cotton mats. Then place the dry mats and wet them immediately after they are placed. Weight the mats adequately to provide continuous contact with all concrete. Cover surfaces that cannot be cured by direct contact with mats, forming an enclosure well anchored to the ground so that outside air cannot enter the enclosure. Provide sufficient moisture inside the enclosure to keep all surfaces of the concrete wet. Overlap sprays or sprinklers to keep all unformed surfaces continuously wet.

Article 360.4. Construction. Section K. Protection of Pavement and Opening to Traffic is voided and replaced by the following:

K.   Protection of Pavement and Opening to Traffic. Testing for early opening is the responsibility of the Contractor regardless of job-control testing responsibilities unless otherwise shown in the plans or directed. Testing result interpretation for opening to traffic is subject to the approval of the Engineer.

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05-09</div>

1. Protection of Pavement. Erect and maintain barricades and other standard and approved devices that will exclude all vehicles and equipment from newly placed pavement for the periods specified. Before opening to traffic, protect the pavement from damage due to crossings using approved methods. Where a detour is not readily available or economically feasible, an occasional crossing of the roadway with overweight equipment may be permitted for relocating the equipment only but not for hauling material. When an occasional crossing of overweight equipment is permitted, temporary matting or other approved methods may be required.

Maintain an adequate supply of sheeting or other material to cover and protect fresh concrete surface from weather damage. Apply as needed to protect the pavement surface from weather.

2. Opening Pavement to All Traffic. Do not open the pavement to traffic, including vehicles of the Contractor, until the last concrete placed is at least 12 hours old and meets a minimum compressive strength of 2,800 psi.

At the end of this period, the pavement may open for use by vehicles of the Contractor or the public. Such opening, however, in no manner relieves the Contractor of his/her responsibility for the work in accordance with Item 7, "Legal Relations and Responsibilities". Before opening sections of the pavement to traffic, seal the joints and clean the pavement.

Article 360.5. Measurement is supplemented by the following:

C. Bonded Concrete Overlay. Bonded concrete overlay will be measured by the square yard of surface area of the depth specified, completed and accepted work.

Article 360.6. Payment is voided and replaced by the following:

The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid as provided under Article 360.5, "Bonded Concrete Overlay" of the depth specified. This price is full compensation for furnishing, loading, unloading, storing, hauling, and handling concrete ingredients, including freight and royalty involved; for curing; placing and adjusting forms, including supporting material or preparing track grade; for water blasting to prepare concrete surfaces prior to placing the bonded concrete overlay; for furnishing and installing steel fibers; furnishing and installing reinforcing steel; furnishing materials for sealing joints; for mixing, placing, finishing, curing, and sawing concrete; for cleaning and sealing concrete joints; and for manipulations, labor, tools, equipment, and incidentals necessary to complete the work.

Remove and replace concrete failing to meet the 7 day minimum strength requirements at no expense to the Department, unless otherwise directed.

When surface Test Type B, as specified in Item 585, "Ride Quality for Pavement Surfaces", is used, a bonus or deduction for each 0.10 mile section of each travel lane will be calculated and applied in dollars and cents.

SPECIAL PROVISION

421---031

Hydraulic Cement Concrete

For this project, Item 421, "Hydraulic Cement Concrete," 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 421.2.D. Water, Table 1. Chemical Limits for Mix Water is voided and replaced by the following:

Table 1
Chemical Limits for Mix Water

Contaminant	Test Method	Maximum Concentration (ppm)
Chloride (Cl)	ASTM C 114	
Prestressed concrete		500
Bridge decks and superstructure		500
All other concrete		1,000
Sulfate (SO_4)	ASTM C 114	2,000
Alkalies ($N_{a2}O + 0.658K_2O$)	ASTM C 114	600
Total Solids	ASTM C 1603	50,000

Article 421.2.B. Supplementary Cementing Materials (SCM) is supplemented with the following:

6. **Modified Class F Fly Ash (MFFA).** Furnish MFFA conforming to DMS-4610, "Fly Ash."

Article 421.2.D. Water, Table 2. Acceptance Criteria for Questionable Water Supplies is voided and replaced by the following:

Table 2
Acceptance Criteria for Questionable Water Supplies

Property	Test Method	Limits
Compressive strength, min. % control at 7 days	ASTM C 31, ASTM C 39[1,2]	90
Time of set, deviation from control, h:min.	ASTM C 403[1]	From 1:00 early to 1:30 later

[1] Base comparisons on fixed proportions and the same volume of test water compared to the control mix using 100% potable water or distilled water.
[2] Base comparisons on sets consisting of at least two standard specimens made from a composite sample.

Article 421.2.E.2 Fine Aggregate. The fifth paragraph is voided and replaced by the following:

Acid insoluble (%) = {(A1)(P1)+(A2)(P2)}/100
where:
  $A1$ = acid insoluble (%) of aggregate 1
  $A2$ = acid insoluble (%) of aggregate 2

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$P1$ – percent by weight of aggregate 1 of the fine aggregate blend
$P2$ = percent by weight of aggregate 2 of the fine aggregate blend

Article 421.2.E.2. Fine Aggregate. The final paragraph is voided and replaced by the following:

For all classes of concrete, provide fine aggregate with a fineness modulus between 2.3 and 3.1 as determined by Tex-402-A.

Article 421.2.E. Aggregate is supplemented by the following:

4. **Intermediate Aggregate.** When necessary to complete the concrete mix design, provide intermediate aggregate consisting of clean, hard, durable particles of natural or light-weight aggregate or a combination thereof. Provide intermediate aggregate free from frozen material and from injurious amounts of salt, alkali, vegetable matter, or other objectionable material, and containing no more than 0.5% clay lumps by weight in accordance with Tex-413-A.

   If more than 30% of the intermediate aggregate is retained on the No. 4 sieve, the amount retained on the No. 4 sieve must either meet the requirements of or come from a source that meets the requirements of a wear not more than 40% when tested in accordance with Tex-410-A and a 5-cycle magnesium sulfate soundness of not more than 18% when tested in accordance with Tex-411-A.

   If more than 30% of the intermediate aggregate passes the 3/8" sieve, the portion passing the 3/8" sieve must not show a color darker than standard when subjected to the color test for organic impurities in accordance with Tex-408-A and must have an acid insoluble residue, unless otherwise shown on the plans, for concrete subject to direct traffic equal to or greater than the value calculated with the following equation:

$$\text{Alia} \geq \frac{60 - (\text{Alfa})(\text{Pfa})}{(\text{Pia})}$$

   where:

   Alfa = acid insoluble (%) of fine aggregate or fine aggregate blend

   Pfa = percent by weight of the fine aggregate or fine aggregate blend as a percentage of the total weight of the aggregate passing the 3/8" sieve in the concrete mix design

   Pia = percent by weight of the intermediate aggregate as a percentage of the total weight of the aggregate passing the 3/8" sieve in the concrete mix design

Article 421.2.F. Mortar and Grout is supplemented by the following:

Section 421.4.A.6, "Mix Design Options," does not apply for mortar and grout.

Article 421.3.A. Concrete Plants and Mixing Equipment is supplemented by the following:

When allowed by the plans or the Engineer, for concrete classes not identified as structural concrete in Table 5 or for Class C concrete not used for bridge-class structures, the Engineer may inspect and approve all plants and trucks in lieu of the NRMCA or non-Department engineer sealed certifications. The criteria and frequency of Engineer approval of plants and trucks is the same used for NRMCA certification.

Article 421.3.A.2. Volumetric Mixers is supplemented by the following:

Unless allowed by the plans or the Engineer, volumetric mixers may not supply classes of concrete identified as structural concrete in Table 5.

Article 421.4.A Classification and Mix Design. The first paragraph is voided and replaced by the following:

Unless a design method is indicated on the plans, furnish mix designs using ACI 211, "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete," Tex-470-A, or other approved procedures for the classes of concrete required in accordance with Table 5. Perform mix design and cement replacement using the design by weight method unless otherwise approved. Do not exceed the maximum water-to-cementitious-material ratio.

Article 421.4.A. Classification and Mix Design, Table 5 Concrete Classes voided and replaced by the following:

Table 5
Concrete Classes

Class of Concrete	Design Strength, Min. 28-day f'c (psi)	Maximum W/C Ratio[1]	Coarse Aggregate Grades[2,3]	General Usage[4]
A	3,000	0.60	1–4, 8	Inlets, manholes, curb, gutter, curb & gutter, conc. retards, sidewalks, driveways, backup walls, anchors
B	2,000	0.60	2–7	Riprap, small roadside signs, and anchors
C[5]	3,600	0.45	1–6	Drilled shafts, bridge substructure, bridge railing, culverts except top slab of direct traffic culverts, headwalls, wing walls, approach slabs, concrete traffic barrier (cast-in-place)
C(HPC)[5]	3,600	0.45	1–6	As shown on the plans
D	1,500	0.60	2–7	Riprap
E	3,000	0.50	2–5	Seal concrete
F[5]	Note 6	0.45	2–5	Railroad structures; occasionally for bridge piers, columns, or bents
F(HPC)[5]	Note 6	0.45	2–5	As shown on the plans
H[5]	Note 6	0.45	3–6	Prestressed concrete beams, boxes, piling, and concrete traffic barrier (precast)
H(HPC)[5]	Note 6	0.45	3–6	As shown on the plans
S[5]	4,000	0.45	2–5	Bridge slabs, top slabs of direct traffic culverts
S(HPC)[5]	4,000	0.45	2-5	As shown on the plans
P	See Item 360	0.45	2–3	Concrete pavement
DC[5]	5,500	0.40	6	Dense conc. overlay
CO[5]	4,600	0.40	6	Conc. overlay
LMC[5]	4,000	0.40	6–8	Latex-modified concrete overlay
SS[5]	3,600[7]	0.45	4–6	Slurry displacement shafts, underwater drilled shafts
K[5]	Note 6	0.45	Note 6	Note 6
HES	Note 6	0.45	Note 6	Note 6

[1]. Maximum water-cement or water-cementitious ratio by weight.
[2]. Unless otherwise permitted, do not use Grade 1 coarse aggregate except in massive foundations with 4-in. minimum clear spacing between reinforcing steel bars. Do not use Grade 1 aggregate in drilled shafts.

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Class of Concrete	Design Strength, Min. 28-day f'_c (psi)	Maximum W/C Ratio[1]	Coarse Aggregate Grades[2,3]	General Usage[4]

[3] Unless otherwise approved, use Grade 8 aggregate in extruded curbs.

[4] For information only.

[5] Structural concrete classes.

[6] As shown on the plans or specified.

[7] Use a minimum cementitious material content of 650 lb/cy of concrete. Do not apply Table 6 over design requirements to Class SS concrete.

Article 421.4.A. Classification and Mix Design, Table 6 Over Design to Meet Compressive Strength Requirements. Footnote 3 is supplemented by the following:

For Class K and concrete classes not identified as structural concrete in Table 5 or for Class C concrete not used for bridge-class structures, the Engineer may designate on the plans an alternative over-design requirement up to and including 1,000 psi for specified strengths less than 3,000 psi and up to and including 1,200 psi for specified strengths from 3,000 to 5,000 psi.

Article 421.4.A.1. Cementitious Materials is supplemented by the following:

The upper limit of 35% replacement of cement with Class F fly ash specified by mix design options 1 and 3 may be increased to a maximum of 45% for mass placements, high performance concrete, and precast members when approved.

Article 421.4.A.3. Chemical Admixtures is supplemented by the following:

When a corrosion-inhibiting admixture is required, use a 30% calcium nitrite solution. The corrosion inhibiting admixture must be set neutral unless otherwise approved. Dose the admixture at the rate of gallons of admixture per cubic yard of concrete shown on the plans.

Article 421.4.A.4 Air Entrainment is voided and replaced by the following:

Air entrain all concrete except for Class B and concrete used in drilled shafts unless otherwise shown on the plans. Unless otherwise shown on the plans, target an entrained air content of 4.0 % for concrete pavement and 5.5 % for all other concrete requiring air entrainment. To meet the air-entraining requirements, use an approved air-entraining admixture. If the air content is more than 1-1/2% below or 3% above the required air, the load of concrete will be rejected. If the air content is more than 1-1/2 but less than 3% above the required air, the concrete may be accepted based on strength tests. For specified concrete strengths above 5,000 psi, a reduction of 1% is permitted.

Article 421.4.A Table 7 Air Entrainment is voided.

Article 421.4.A.6. Mix Design Options. The first paragraph is voided and replaced by the following:

For structural concrete identified in Table 5 and any other class of concrete designed using more than 520 lb. of cementitious material per cu. yd., use one of the mix design Options 1-8 shown below, unless otherwise shown on the plans.

Article 421.4.A.6. Mix Design Options. The second paragraph is voided and replaced by the following:

For concrete classes not identified as structural concrete in Table 5 and designed using less than 520 lb. of cementitious material per cu. yd., use one of the mix design Options 1–8 shown below, except that Class C fly ash may be used instead of Class F fly ash for Options 1, 3, and 4 unless sulfate-resistant concrete is shown on the plans.

Do not use mix design options 6 or 7 when High Performance Concrete (HPC) is required. Option 8 may be used when HPC is required provided: a minimum of 20% of the cement is replaced with a Class C fly ash; test method Tex-440-A, "Initial Time of Set of Fresh Concrete" is performed during mix design verification; the additional requirements for permeability are met; and the concrete is not required to be sulfate-resistant.

Article 421.4.A.6.b. Option 2 is voided and replaced by the following:

b. Option 2. Replace 35 to 50% of the cement with GGBFS or MFFA.

Article 421.4.A.6.c. Option 3 is voided and replaced by the following:

c. Option 3. Replace 35 to 50% of the cement with a combination of Class F fly ash, GGBFS, MFFA, UFFA, metakaolin, or silica fume. However, no more than 35% may be fly ash, and no more than 10% may be silica fume.

Article 421.4.A.6.g. Option 7 is voided and replaced by the following:

g. Option 7. When using hydraulic cement only, ensure that the total alkali contribution from the cement in the concrete does not exceed 4.00 lb. per cubic yard. of concrete when calculated as follows:

$$\text{lb. alkali per cu. yd.} = \frac{(\text{lb. cement per cu. yd.}) \times (\%\ Na_2O\ \text{equivalent in cement})}{100}$$

In the above calculation, use the maximum cement alkali content reported on the cement mill certificate.

Do not use Option 7 when any of the aggregates in the concrete are listed on the Department's List of Aggregate Sources Excluded from Option 7 ASR Mitigation.

Article 421.4.A.6.h. Option 8 is voided and replaced by the following:

h. Option 8. For any deviations from Options 1–5, perform testing on both coarse and fine aggregate separately in accordance with ASTM C 1567. Before use of the mix, provide a certified test report signed and sealed by a licensed professional engineer, from a laboratory on the Department's List of Approved ASTM C 1260 Laboratories, demonstrating that the ASTM C 1567 test result for each aggregate does not exceed 0.10% expansion.

Do not use Option 8 when any of the aggregates in the concrete are listed on the Department's List of Aggregate Sources Excluded from Option 8 ASR Mitigation. When HPC is required, provide a certified test report signed and sealed by a licensed professional engineer demonstrating that AASHTO T 277 test results indicate the permeability of the concrete is less than 1,500 coulombs tested immediately after either of the following curing schedules:

- Moist cure specimens 56 days at 73°F.

- Moist cure specimens 7 days at 73°F followed by 21 days at 100°F.

Article 421.4.B. Trial Batches is supplemented by the following:

Once a trial batch substantiates the mix design, the proportions and mixing methods used in the trial batch become the mix design of record.

Article 421.4.B. Trial Batches. The fourth sentence of the second paragraph is voided and replaced by the following:

Test at least 1 set of design strength specimens, consisting of 2 specimens per set, at 7-day, 28-day, and at least one additional age.

Article 421.4.D. Measurement of Materials, Table 9 is voided and replaced by the following:

Table 9
Measurement Tolerances – Non-Volumetric Mixers

Material	Tolerance (%)
Cement, wt.	-1 to +3
SCM wt.	-1 to +3
Cement + SCM (cumulative weighing), wt.	-1 to +3
Water, wt. or volume	±3
Fine aggregate, wt.	±2
Coarse aggregate, wt.	±2
Fine + coarse aggregate (cumulative weighing), wt.	±1
Chemical admixtures, wt. or volume	±3

Article 421.4.E. Mixing and Delivering Concrete. The first paragraph is supplemented with the following:

Do not top-load new concrete onto returned concrete.

Article 421.4.E.3. Truck-Mixed Concrete. The first paragraph is voided and replaced by the following:

Mix the concrete in a truck mixer from 70 to 100 revolutions at the mixing speed designated by the manufacturer to produce a uniform concrete mix. Deliver the concrete to the project in a thoroughly mixed and uniform mass and discharge the concrete with a satisfactory degree of uniformity. Additional mixing at the job site at the mixing speed designated by the manufacturer is allowed as long as the requirements of Section 421.4.A.5, "Slump" and Section 421.4.E, Mixing and Delivering Concrete" are met.