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Construction and Evaluation of Post-Tensioned Pre-Stressed Concrete Pavement

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Texas Department of Transportation

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16. Abstract

The performance of cast-in-place post-tensioned concrete pavement (PCP) constructed in 1985 on IH-35 in Waco, Texas has been excellent. Encouraged by the performance of the section, the Texas Department of Transportation decided to build another PCP project on IH-35 in Hillsboro. The construction of 9-in thick PCP with mostly 300-ft long slabs started in May 2008. Issues raised during the pre-construction and construction phases were evaluated and documented in this report. Also, the detailed behavior of PCP at early ages due to environmental loading (temperature and moisture variations) and post-tensioning were evaluated with various installed gages. The strain and movement of PCP as well as temperature and relative humidity were measured. The effects of such factors as post tensioning (PT) force, friction, curling stress, creep, and shrinkage on the behavior of PCP were investigated. The stress introduced by longitudinal PT varied along the slab length, with a maximum near the armor joint and a minimum at the center of the slab. The concrete strain at mid-depth of the slab under environmental loading was also affected by friction and other restraints. The concrete thermal strain restrained by friction and other factors was larger near the slab center. The distribution of longitudinal slab movement was nonlinear along with the distance from slab center. Continued contraction of concrete slab due to creep and shrinkage was observed, one which will result in the opening of joint width. Creep and shrinkage effects should be included in the design of the initial joint width. The findings from the field evaluations of the PCP behavior were used to calibrate the theoretical analysis model PCP 3.0.

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CHAPTER 1 INTRODUCTION

1.1 BACKGROUND

Portland cement concrete (PCC) undergoes volume changes due to temperature and moisture changes. When the volume changes are not allowed and restrained, tensile stresses develop and cracks could develop. Depending on how to accommodate the volume changes, there are two types of PCC pavements: (1) plain jointed plain concrete pavement (JCP) and (2) continuously reinforced concrete pavement (CRCP). In JCP, transverse contraction joints are provided at 15-ft to 20-ft interval, so that the concrete tensile stresses resulting from temperature and moisture variations and wheel loading applications do not exceed concrete tensile strength, thus preventing transverse cracks. In other words, in JCP, provisions are made – short joint spacing and low friction between concrete and subbase – to allow concrete to undergo volume changes with minimum restraints. Short joint spacing also reduces warping stress resulting from temperature variations through the slab depth substantially. On the other hand, concrete volume changes are tightly restrained in CRCP, which results in numerous transverse cracks with short crack spacing. Cast-in-place post-tensioned concrete pavement (hereafter called PCP in this report) is a unique pavement type where concrete volume changes are fully allowed without restraint, except for the restraint imposed by concrete weight for temperature variations through the slab depth (warping). To counter-act tensile stresses in concrete due to warping and any existing frictional restraint between concrete and subbase, pre-stress is applied in the form of post-tensioning. The application of pre-stress allows the increase in slab length and the reduction in slab thickness.

In 1985, Texas Department of Transportation (TxDOT) built one mile test section of 6-in PCP on IH-35 in West in Waco District. The section provided excellent performance, and TxDOT decided to build a larger scale PCP project. Detailed information on the new PCP project is included in the research report 5-4035-1. The PCP consists of 9-in slab with 300-ft slab length on top of 4-in asphalt stabilized subbase. TxDOT initiated an implementation project, 5-4035, whose objectives included providing technical assistant to the district and the contractor, documenting activities during pre-construction and construction, and evaluating the structural behavior and short-term performance of PCP. This report documents the activities that took place during the construction of the PCP, which started on May 27, 2009. This report also discusses the structural behavior of PCP due to environmental loading (temperature and moisture variations) and post-tensioning.

1.2 OBJECTIVES

The 6-in PCP on IH-35 built in 1985 has provided excellent performance; however, there have not been in-depth evaluations of the section on structural behavior, especially the evaluations of the early-age behavior due to environmental loading and post-tensioning that could provide clues regarding how the section has provided such excellent performance. The primary objectives of this implementation project were:

1) Conduct in-depth evaluations of the early-age behavior of PCP in the field and document the findings, and
2) Provide technical assistance to TxDOT and the contractor for the design and construction of PCP in Hillsboro.

To achieve these objectives, various gages were installed in a 300-ft slab along with a semi-permanent data logger setup. Also, the research team attended meetings to address issues that needed to be resolved, provided technical opinions, and documented the progress.

1.3 REPORT ORGANIZATION

This report is organized in 5 chapters. Chapter 2 discusses the issues raised during pre-construction and construction phases and how they were resolved. Chapter 3 describes the efforts made to evaluate early-age behavior of PCP due to environmental loading and post-tensioning operations. Detailed discussions are provided on the mechanistic responses of PCP based on the analysis of the results from various gages. Chapter 4 presents the results of the efforts to calibrate the computer model developed in the 1980s to analyze mechanistic behavior of PCP. Chapter 5 summarizes the findings made in this study and provides recommendations that could improve future PCP performance if implemented along with further research.

CHAPTER 2 PRE-CONSTRUCTION AND CONSTRUCTION ASSISTANCE

The first task conducted in the implementation project was to develop design standards and special specifications that were included in the PS&E. Design standards and special specifications developed in this study are included in the Appendix A and B, respectively, in the report 5-4035-1. After the project was let and awarded, the issues that came up later were primarily related to constructability. It was partly due to the fact that the contractor did not have the experience in the construction of PCP. Prior to the construction, several meetings were held between TxDOT, the contractor, and the research team to discuss issues related to the construction of the pavement. Once the construction began, additional meetings were conducted between TxDOT, the contractor, and the research team to address issues identified during the construction process. This chapter summarizes the issues raised during the pre-construction and construction phases and how they were resolved.

2.1 ISSUES RAISED DURING PRE-CONSTRUCTON PHASE

1. Gap Slab between PCP Slabs

   After the project was awarded, it was discovered that the center portion (8-ft wide) of the PCP that divides northbound and southbound lanes could not be PCP. Numerous bridge columns or other obstructions did not allow the deployment of continuous longitudinal tendons. This issue was discussed at a meeting and it was agreed that the use of 9-in CRCP with transverse expansion joints at the locations of transverse armor joints in PCP would be the more practical solution. Design details were developed by the research team and submitted to TxDOT, which is shown in Appendix A. The reasoning for the use of CRCP with 300-ft joint spacing was that, since steel and concrete have similar coefficient of thermal expansion and polyethylene sheets were placed under CRCP, CRCP slab will move by almost the same amount due to temperature variations. It is noted that PCP was constructed first with longitudinal post-tensioning done before CRCP slab was placed. The reason for the use of CRCP of 9-in thickness was that this portion won’t be under traffic and thicker slabs were not needed. Normally, dowels were not installed at transverse expansion joints, except where the joints were under traffic due to temporary switching of the traffic. In those joints, dowels were installed.

2. Transverse Post-Tensioning

   The initial PCP width was 66-ft in each direction. As discussed above, 4-ft of 66-ft was constructed with CRCP. Since the paver cannot place 62-ft width PCP in a single placement, and also from a traffic control standpoint, the concrete had to be placed in multi-phases with shorter widths. Multi-phase concrete placements prevented the use of tendons for transverse post-tensioning, since tendons in excess of the pavement width under construction were in the way of a slip-form paver. This issue was discussed in the meeting, and it was agreed that the use of rebar, instead of tendons, will resolve this issue. The design firm hired by the contractor conducted structural analysis and came up with a design of 1-in diameter rebar at 6-ft spacing.

3. Determination of Timing for Initial Post-Tensioning
   Initial post-tensioning at early ages is applied to prevent any cracking potential while concrete is still weak. The special provision 3045 states "Longitudinal strands are stressed first. Perform post-tensioning of the concrete pavement in two stages. Apply the first post-tensioning forces of 15 kips when the concrete gains a minimum compressive strength of 1000 psi. The initial post-tensioning must occur within 8 hours after placing concrete. Perform compressive tests of concrete cylinders at the job site to determine the timing of the post-tensioning." Since the slab was 9-in thick and longitudinal tendon spacing was 15-in for 300-ft long slabs, 15-kips of initial post-tensioning was equivalent to 111 psi of compressive stress in concrete, which was considered to be sufficient to prevent any potential for transverse cracking. The issue raised was how to determine in the field when the concrete reached 1,000 psi of compressive strength. It was suggested that maturity method be utilized in accordance with testing procedure Tex-426-A, and all parties agreed. The use of the maturity method for determining when to conduct compressive strength testing was incorporated in the special provision. However, the way the special specification was written was somewhat ambiguous. For example, what if 1000 psi of compressive strength was not achieved within 8 hours? This was not a real issue during the summer placement; however, it became an issue during winter placement as will be discussed later.

2.2 ISSUES RAISED DURING CONSTRUCTON PHASE

1. Low Concrete Strength at Early Ages
   As discussed earlier, the special provision requires that the initial post-tensioning be applied when the concrete reaches 1000 psi of compressive strength and within 8 hours of concrete placement. The contractor prepared two mix designs: one for warm weather and the other for cool weather. The two mix designs are included in the Appendix A. When the project started, which was in the late spring (May 27th, 2008), ambient temperature condition was such that concrete gained adequate early strength and initial post-tensioning was feasible within 8 hours of concrete placement. However, by November, even with the mix design for cool weather, early strength was quite low that initial post-tensioning was not feasible within 8 hours. The situation was worse when the

concrete was placed in the late afternoon or late evening. In one placement, the concrete was placed at 5:30 pm and the strength after 9 hours was only 260 psi. There was a meeting called to discuss this issue and come up with a solution. Suggestions were made not to use fly ash and reduce water/cement ratio by using adequate amount or different type of water reducers. After the meeting, the contractor developed a new mix design with 6.5-sack cement and 0.27 water/cement ratio along with the use of high range water reducer. Fly ash was not used. Air temperature during the cast of the concrete was about 42 F. The strength values at 8, 10, 12, and 18 hours were 60 psi, 100 psi, 250 psi, and 1,850 psi, respectively. This information clearly indicated the effect of low temperature on reducing early age concrete strength, and a need for more practical solution to this issue. The contractor decided to use the existing mix design for cool weather for winter placement. The reason for 1,000 psi of concrete strength before the application of an initial post-tensioning is that, the initial post-tensioning application while concrete is very weak could result in large deformations in concrete due to low modulus of elasticity and large creep, which produces undesirable effect of larger joint widths. A suggestion was made during the meeting that the initial post-tensioning force could be adjusted depending on the concrete strength. The reasoning was that, if the concrete strength is low, the modulus of elasticity of concrete will also be small. For the same concrete strain, lower concrete stress will result with smaller modulus of elasticity. Therefore, the pre-stress to be applied to prevent cracking could be reduced proportionately to modulus of elasticity. This practice would result in the same pre-strain, not pre-stress. It is proposed that concrete cracking depends on ultimate tensile strain, not strength, and this approach should work. ACI 318 stipulates that concrete modulus of elasticity is proportional to the square root of compressive strength, as shown in Equation (2.1), the initial post-tensioning force was reduced proportionately to the square root of compressive strength.

$$E_c = 33\ uw^{1.5}\sqrt{f'c} \quad \text{---(2.1)}$$

where,

Ec = modulus of elasticity of concrete (psi)

uw = unit weight of concrete (lbs/cf)

f'_c = concrete compressive strength (psi)

Figure 2.1 shows the initial post-tensioning force table that field crew carries in his truck. It shows that, if the compressive strength is 600 psi or less, they have to wait for further strength gain. It shows that the initial post-tensioning force is reduced as concrete strength decreases. The decrease is proportional to the square root of concrete compressive strength.

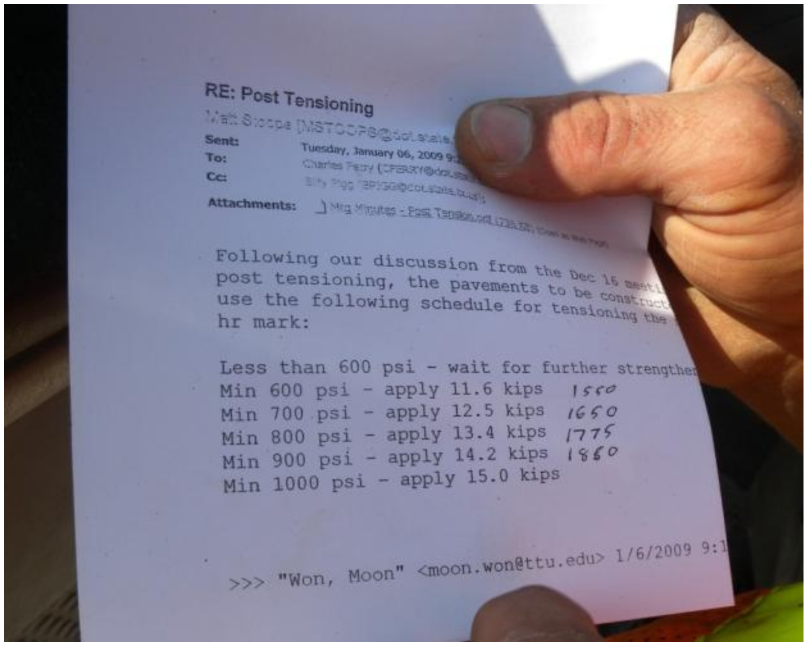


Figure 2.1 Reduction of initial post-tensioning force for low concrete strength

2. Tendons being Pushed

During the paving on the first day of PCP construction on May 27, 2008, it was noticed that longitudinal tendons were pushed by the concrete, which again pushed transverse post-tension bar and transverse steel, as shown in Figure 2.2. To prevent this, the contractor personnel had to push transverse post-tension bars and steel with his foot, as shown in Figure 2.3.

Figure 2.2 Tendons and transverse bars pushed by concrete

Figure 2.3 Contractor trying to hold transverse bars in place

The contractor shortly came up with the idea of placing #6 rebar longitudinally and tying transverse post-tension bars and transverse rebar to the longitudinal #6 bars. Figure 2.4 shows the longitudinal rebar placed. This system of placing longitudinal bars, with one bar at each side of the slabs and one bar in the middle for more than 30-ft wide placement, was incorporated for the rest of the project, as shown in Figure 2.5. This system of placing longitudinal bars helped keep the transverse post-tensing bars and transverse steel in place a little. However, it did not prevent the transverse post-tensioning bars and transverse steel from being pushed. As discussed below, the contractor started staking bars to the asphalt to keep the transverse post-tensioning bars in place during concrete placement.

Figure 2.4 Longitudinal rebar placed to correct the problems of bars being pushed by concrete

Figure 2.5 Longitudinal rebar placed at both sides and in the middle of the slab

The placements of the longitudinal rebar will reduce the pre-stress level in the concrete; however, the cross-sectional area of the rebar is quite small compared with the pavement section, and the reduction was considered to be insignificant, especially when there were no better alternatives for this constructability problem. In addition, without these longitudinal bars, tendons won't be placed straight and the resulting pre-stress loss will be more substantial.

3 Bars Driven into Subbase to Keep Reinforcement in Place

Due to the transverse reinforcement being pushed by concrete placement even after the placement of longitudinal rebar as described above, the contractor decided to install steel bars into the asphalt subbase as shown in Figure 2.6, which is called "vertical bars" in this report. These vertical bars, four bars at a location of transverse post-tensioning steel, were installed every third transverse post-tensioning steel. When the two transverse post-tensioning steel where vertical bars were not installed were not tightly fixed with longitudinal tendons, the transverse post-tensioning bars were bent as shown in Figure 2.7 due to the concrete pushing the bars.

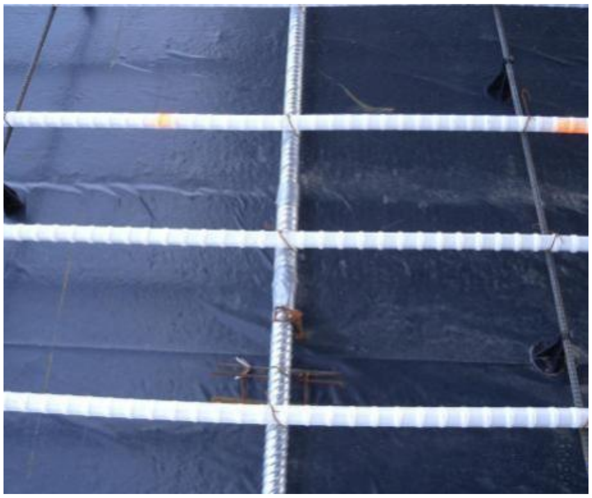


Figure 2.6 Bar installed to keep transverse post-tensioning bar in place

Figure 2.7 Transverse post-tensioning bar being bent due to concrete pressure

Driving vertical bars into the asphalt subbase in CRCP has been a topic of discussion among TxDOT engineers for a number of years. In CRCP, vertical bars are hammered into the subbase to provide rigidity to the reinforcement so that the steel mat does not collapse when the concrete is pushed forward by a slip-form paver. Currently in TxDOT, some districts require the removal of vertical bars just before the paver gets close to the bar location, while the other districts allow the contractors to leave the vertical bars. The discussion among TxDOT engineers on this topic centered around whether these vertical bars, if left in place, would restrain concrete displacements due to temperature variations and cause distresses. No formal investigations have been conducted on this subject. This issue is more significant in PCP than in CRCP, because the concrete volume changes in PCP should be allowed with minimal restraint in order to reduce pre-stress loss as much as possible. Installing vertical bars will provide restraint on concrete volume changes, resulting in less effective pre-stressing. An alternative of not installing vertical bars will allow less restraint on concrete volume changes; however, undulations in tendons would increase pre-stress loss as well. Obviously, there is a conflict between what should be done from a theoretical standpoint and constructability. During the construction of PCP in this project, constructability outweighed theoretical considerations. Investigating the effect of vertical bars on pre-stress loss was out of the scope of this study.

2.3 OTHER ISSUES UNIQUE TO PCP CONSTRUCTION

Since PCP is quite different from the other types of PCC pavement in the way concrete volume changes are addressed, there were a number of issues unique to PCP construction. They include (1) complexity of installing tendons and armor joints and resulting low productivity, (2) extensive efforts needed to install and finish concrete for stressing pockets, (3) back-pressure of concrete on paver due to the existence of armor joint, (4) issues related to providing space for the connection of transverse post-tensioning bars at longitudinal construction joint, and (5) exposing grouting hose at armor joints.

1. Complexity of Tendon and Armor Joint Installation

   The length of the majority of the slabs was 300-ft. The contractor usually placed 3 slabs a day, which is equal to 900-ft a day. This production rate is quite low, compared with JCP or CRCP, where normal production rate is about 2,000-ft to 2,500-ft a day. In PCP construction, the speed of the paver was actually comparable to that for the placement of JCP or CRCP. What was limiting the production rate of PCP was the placement of tendons and other reinforcement, joint installation, and the need for subsequent operations such as post-tensioning. For CRCP, a crew of 15 to 20 people place steel for a mile a day, whereas in PCP, the same number of people or even more place just 300-ft a day.

2. Extensive Efforts Needed for Stressing Pocket Installations

   Since the spacing for longitudinal tendons is 15-in for 300-ft long slab, there are 27 tendons required for 35-ft wide PCP slab. Accordingly, there were 27 stressing pockets in each 300-ft slab. The efforts required for the installation of the stressing pockets was quite substantial, from installing forms to removing concrete on their top while the concrete is still fresh, as can be seen in Figure 2.8. It shows that 8 workers are removing fresh concrete to expose the top cover of the stressing pockets.

3. Backpressure of Concrete on Paver

   In the construction of JCP or CRCP, there is little resistance from concrete in front of the paver on the slip-form paver, even though it has been speculated that transverse steel in CRCP could provide resistance to the paver, resulting so-called “chattering.” It is because in JCP and CRCP construction, no obstacles exist in front of the paver, whereas in PCP construction, transverse armor joints pose a point of substantial resistance on the paver through the backpressure of concrete. The contractor stated that the paver operator feel the pressure, which normally reduces the paving speed and production rate. Due to the unique nature of the PCP construction, it appears that this issue cannot be resolved in any other way than the current practice of reducing the paver speed.

4. Space for Transverse Joint at Longitudinal Construction Joint

Since 1-in bar is used for transverse pre-stressing, there was a need for providing enough space for the connection of the bars at the longitudinal construction joint. This is quite similar to locating and exposing the female portion of the tie bars when the multi-piece tie bar system is used. The difference is that, in PCP construction, the slab thickness is only 9-in while the female piece is much larger. As a result, there were edge slumps as shown in Figure 2.9. This problem was more prevalent at the early stage of PCP construction. The contractor improved their practice of installing the female pieces closer to the edge and clearly identifying their locations, and this issue was resolved.

Figure 2.8 Removing concrete from the top of stressing pocket cover

Figure 2.9 Edge slump due to the removal of concrete for the space for the connection of transverse bars

5. Exposing Grouting Hose at Armor Joints

Grouting hoses for transverse post-tensioning bars or those at the stressing pockets for longitudinal tendons did not interfer with the concrete placement as can be seen in Figure 2.10. On the other hand, those at the armor joints did interfere with concrete placement if they were not embedded below the concrete surface. Due to the constructability issue, the grouting hoses at the armor joints were positioned below the slab surface and later they were exposed by chipping the concrete manually. Identifying the exact location of the grouting hoses and exposing them by chipping concrete was time consuming and labor intensive, as can be seen in Figure 2.11. Improved method could enhance the productivity of the PCP construction.

Figure 2.10 Grouting hose at longitudinal construction joint not interfering with placement

Figure 2.11 Chipping concrete to expose grouting hoses at armor joints

CHAPTER 3 EARLY-AGE BEHAVIOR OF PCP

This chapter describes efforts made to evaluate early-age behavior of PCP due to temperature and moisture variations and the application of post-tensioning. Various gages, including concrete temperature and moisture sensors, vibrating wire strain gages, linear variable differential transducers (LVDTs), and in-situ coefficient of thermal expansion (CTE) and drying shrinkage of concrete sensors, were installed in 300-ft long slab. Semi-permanent data logger was installed within the right of way with the support of Hillsboro Area Office of the Waco District, and data has been gathered from the concrete placement. Data will be periodically downloaded, analyzed, and stored in TxDOT's rigid pavement database for years to come. The importance of continued monitoring of the behavior of PCP is that it could provide clues to why the 6-in PCP section built in 1985 on IH-35 in the Waco District has been performing so well. Also, the information from this field testing will be used to calibrate the mechanistic model developed to analyze the behavior of PCP slabs.

3.1 FIELD TESTING PROGRAM

3.1.1 DESCRIPTION OF THE TEST SECTION

As discussed earlier, the concrete placement for the PCP on IH-35 in Hillsboro, Texas by the Texas Department of Transportation (TxDOT) started on May 27, 2008. This new PCP consists of 9-in. thick concrete slab over 4-in. asphalt stabilized subbase. Most of the slabs were 300-ft long, except for few with 100-ft long to accommodate specific site needs. The slab selected for field instrumentation was 300-ft long and 35-ft wide. The typical seven-wire monostrand tendons with 0.6 in. diameter were placed at 15-in. spacing in the longitudinal direction. The 1.0-in. diameter post-tensioning bar was placed at 6-ft spacing in the transverse direction. To minimize subbase friction, a single layer of polyethylene plastic sheet was placed over the asphalt stabilized subbase. The concrete placement of the test slab began at 9:30 a.m. and was completed at 11:00 a.m. on February 23, 2009. It took 2 and a half hours to complete concrete placement for a slab. The concrete mix used for this slab was one of the two mix designs (one for cold and one for warm weather) used in this project. Since it was February when the concrete was placed, the mix design for cold weather was used. The mix design is included in the Appendix A. It was a 6-sack mix with 20 % replacement of cement with Class F fly ash. The water cement ratio was the maximum allowed for Class P concrete, which was 0.45. For comparison, the mix design for warm weather had 5.5-sack mix with 20 % replacement of cement with Class F fly ash, and the water cement ratio was 0.45.

The longitudinal post-tensioning (PT) was introduced with a tendon coupler at stressing pockets located in the middle portion of the 300-ft long slab between two transverse armor joints (see Figure 3.1). Steel plates with 0.4-in. thickness, which were placed near the armor joints, were used to apply compressive force to the concrete slab when PT was applied to tendons. The number in the stressing pockets in Figure 3.1 represents the sequence of PT application. Table 3.1 summarizes the procedure of PT application in the PCP test section. Longitudinal PT was applied twice with different levels of forces in the tendon. In order to accurately estimate concrete compressive stress as induced by PT, the compressive strength and elastic modulus of the cylinders under standard curing conditions were measured in accordance with ASTM C 469-02 (*1*). The elastic modulus of the slab was correlated with that of the concrete cylinder through the maturity based on measured temperature (*2*). The sectional average temperature was used to calculate the equivalent age of the concrete slab.

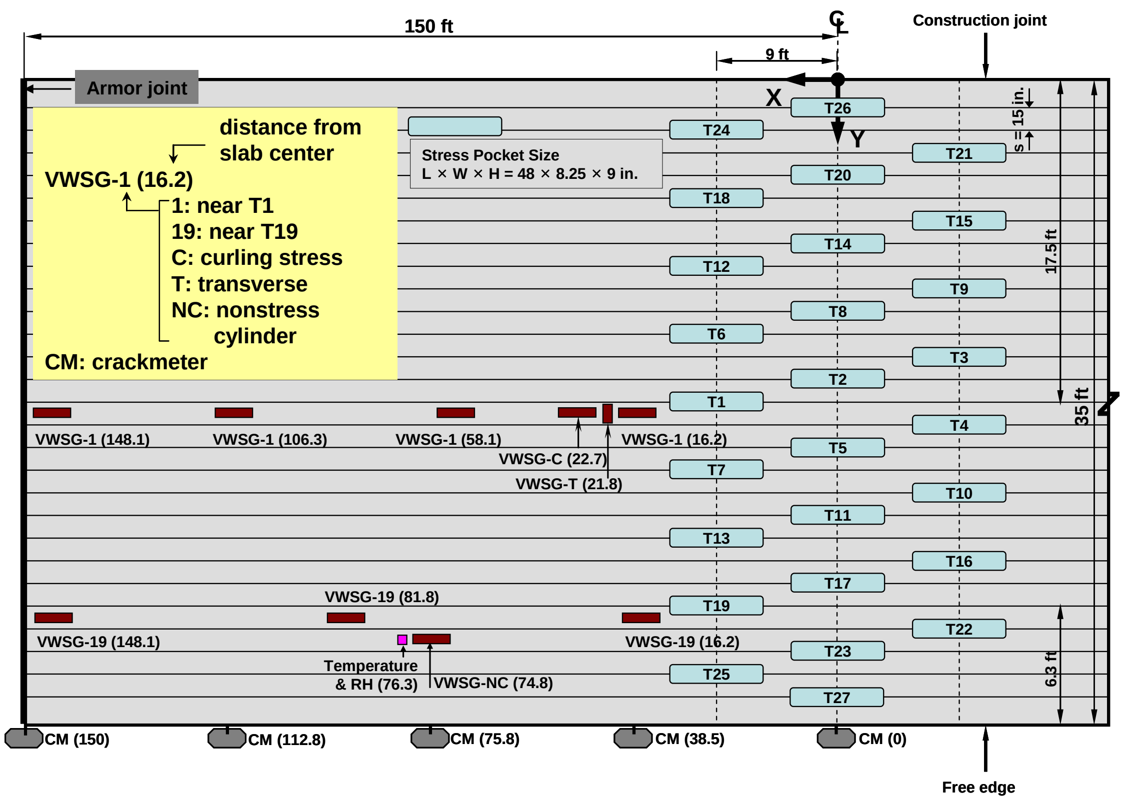


Figure 3.1 Test section geometry and field instrumentation of various sensors

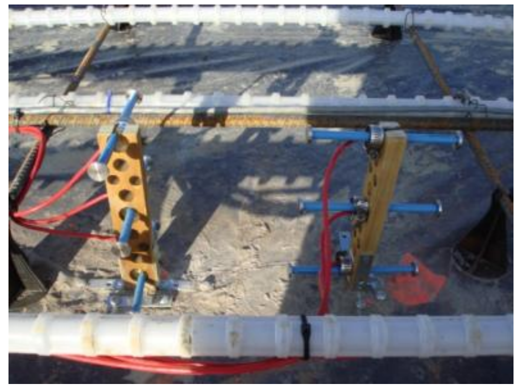
Table 3.1 Application of post-tensioning and relevant material properties

PT Application	Elapsed time [day]	Equivalent concrete age [day]	Force in tendon [kips]	Compressive strength [psi]	Elastic modulus [10^6 psi]
1st longitudinal	0.56	0.58	10	875	1.81
2nd longitudinal	3.06	3.08	35	3962	5.15
Transverse	7.10	6.51	96	4584	5.31

3.1.2 FIELD INSTRUMENTATION

In order to investigate the behavior of PCP due to environmental loading and post-tensioning, various gages mentioned above were installed at different locations as shown in Figure 3.1. The figure shows that there are 27 longitudinal tendons. To investigate the variability of concrete strain in the longitudinal direction, two sets of vibrating wire strain gages were installed longitudinally at different distances from slab center: the first set (VWSG-1) are near tendon T1 and the second set (VWSG-19) are near tendon T19. The number in parenthesis in Figure 3.1 denotes the distance from the slab center in feet. These seven gages were placed at mid-depth (4.5-in). Slab movements under environmental loading, which consisted of temperature and moisture variations, are made up of axial and bending components (*3*). At the centroid of the slab, which is mid-depth in PCP, the free thermal movement will be mainly affected by the average temperature in the section (*3*). This axial movement will be restrained by the subbase friction. The bending components can also be restrained by self-weight of slab, which will cause the curling stress (*4*). In order to accurately assess environmental loading in the slab, eight sensors for temperature and relative humidity (RH) of concrete were installed at different depths as shown in Figure 3.2-(a). Three VWSGs (VWSG-C) were also longitudinally installed at different depths, i.e., 1.0-in., 4.5-in., and 8.0-in., to measure curling stress (see Figures 3.1 and 3.2-(b)). Three VWSGs (VWSG-T) were installed at the same depth as the VWSG-C in a transverse direction in order to evaluate the effect of transverse PT on the behavior of the PCP slab. Figure 3.2-(c) shows the non-stress cylinder (NC) installed to measure concrete strain without stress-induced strain, i.e., thermal strain and drying shrinkage (*5, 6*). The NC design is such that it isolates concrete in the NC from the surrounding concrete and thus prevents the stress transfer from the surrounding concrete to the concrete in the NC. The concrete in the NC without holes is sealed and thus the VWSG in the NC without holes measures the thermal strain of concrete under stress-free conditions. However, the VWSG in the NC with holes measures the strain due to changes in temperature and moisture of concrete because the holes allow the moisture exchange between concrete inside and outside of the NC without interference. Therefore, the free thermal strain due to temperature changes, i.e., the coefficient of thermal expansion, can be measured with VWSG in the NC without holes. Furthermore, the difference of measured strains between VWSGs in NCs with holes and without holes corresponds to strain changes due to moisture variations only, i.e., drying shrinkage. At the free edge, five sets of crackmeters (CM) and LVDTs that measure slab movement were placed at different distances from the slab center before the first longitudinal PT application (see Figure 3.1). As shown in Figure 3.2-(d), each crackmeter set can measure slab movement in longitudinal, transverse, and vertical directions.

(a) Sensor for temperature and RH



(b) VWSG at different depths

(c) Non-stress cylinders

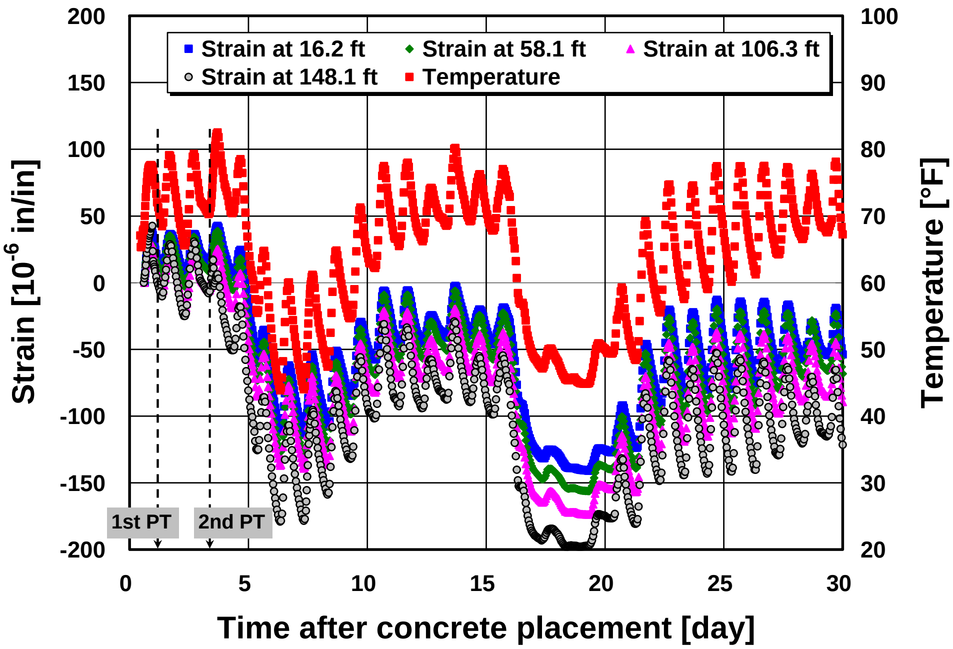
(d) Crackmeter at armor joint

Figure 3.2 Various gages installed

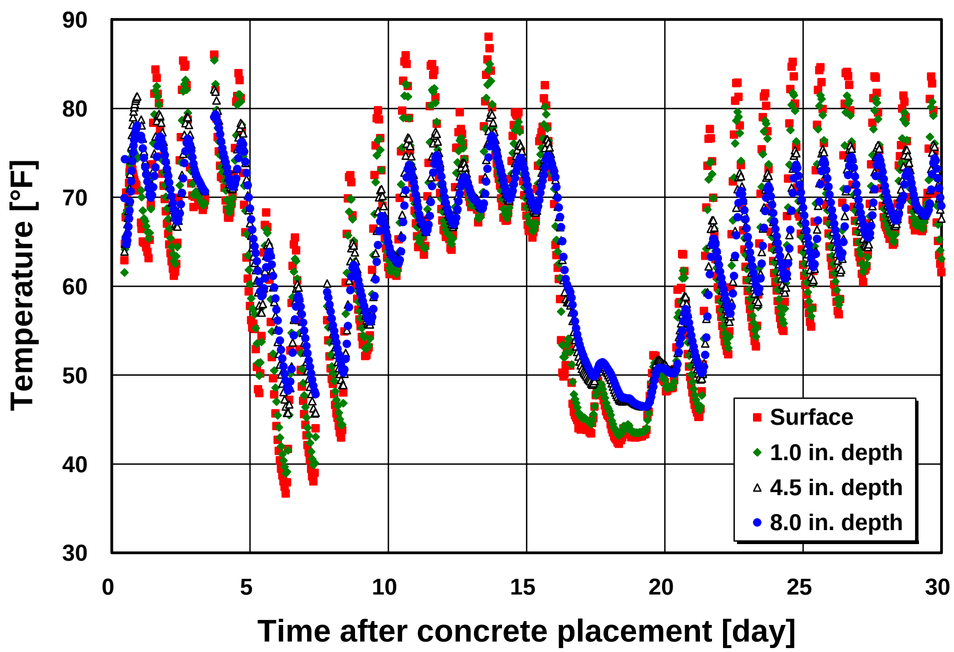
3.2 PRESENTATION OF FIELD INSTRUMENTATION RESULTS

3.2.1 EARLY-AGE SLAB BEHAVIOR DUE TO ENVIRONMENTAL LOADING

Figure 3.3-(a) illustrates the variations of longitudinal strain and temperature measured with VWSG-1 gages from the time of concrete placement (see Figure 3.1). The whole number in the x-axis denotes midnight of the day after concrete placement. For example, "5" means midnight of the fifth day after concrete placement. It shows that temperature played an important role in the development of longitudinal concrete strain in PCP. The measured strains responded positively with temperature variations. The figure also illustrates that compressive strains increased moving away from the center of the slab towards the armor joint. It is postulated that the friction between the concrete slab and the subbase caused these variations, which will be discussed in next section. Also, the practice of leaving bars driven into the subbase to keep the transverse post-tensioning steel appears to have contributed to this. Figures 3.3-(b) and (c) show measured temperature and RH along the depth of slab, respectively. As expected, the maximum temperature variations occurred near the top of the slab, with the smallest variations near the bottom of the slab. For the monitoring period, the average temperature in the section was very similar to that at mid-depth (4.5-in.) which will be further discussed later. The variation of RH was limited to only the surface region, with negligible variation at inner regions as shown in Figure 3.3-(c). It appears that the curing operations were done properly in accordance with TxDOT specification requirements of two applications with each application of not to exceed 180 ft^2/gal of curing compound. Figure 3.3-(d) shows the longitudinal strain and temperature of PCP slab at different depths measured with VWSG-C in Figure 3.1. At early ages, the strain at 4.5-in. depth was higher than that at 1.0-in. depth. At older ages, however, the variation of strains was almost similar to each other even though significant differences of temperature still existed. It suggests that the curling stress was present in the measured concrete element. Figure 3.3-(e) shows the transverse, longitudinal, and unrestrained strains in the slab depending on temperature variation. Because RH variations were limited to the surface region for the monitoring period, measured strain with VWSG in non-stress cylinder with holes was very similar to that in non-stress cylinder without holes. Compared with the unrestrained concrete strains obtained in the non-stress cylinder with holes (see Figure 3.2-(c)), the transverse and longitudinal strains were smaller. This indicates that the concrete in the slab was in a compression stress state in the longitudinal as well as the transverse direction. In the non-stress cylinder, the concrete is in a stress-free condition. Figure 3.3-(f) illustrates that the slab movement in the longitudinal direction was mainly caused by temperature variations. The movement was correlated quite well with the temperature variations in Figure 3.3-(a). The longitudinal slab displacements became larger moving away longitudinally from the center of the slab. The details on each measured data will be thoroughly discussed in the next section.

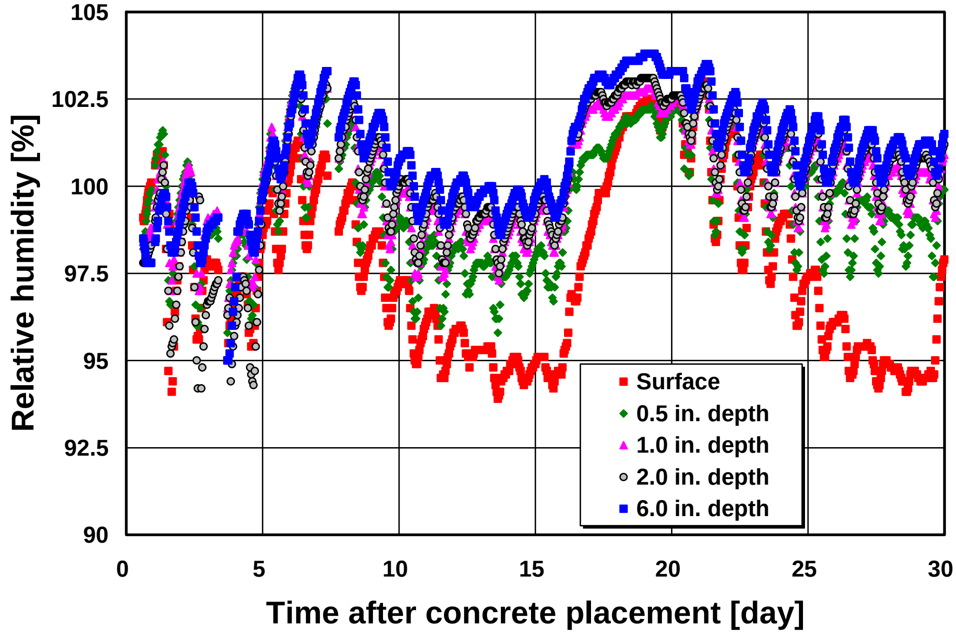


(a) Temperatures and longitudinal strains of VWSG-1 gages

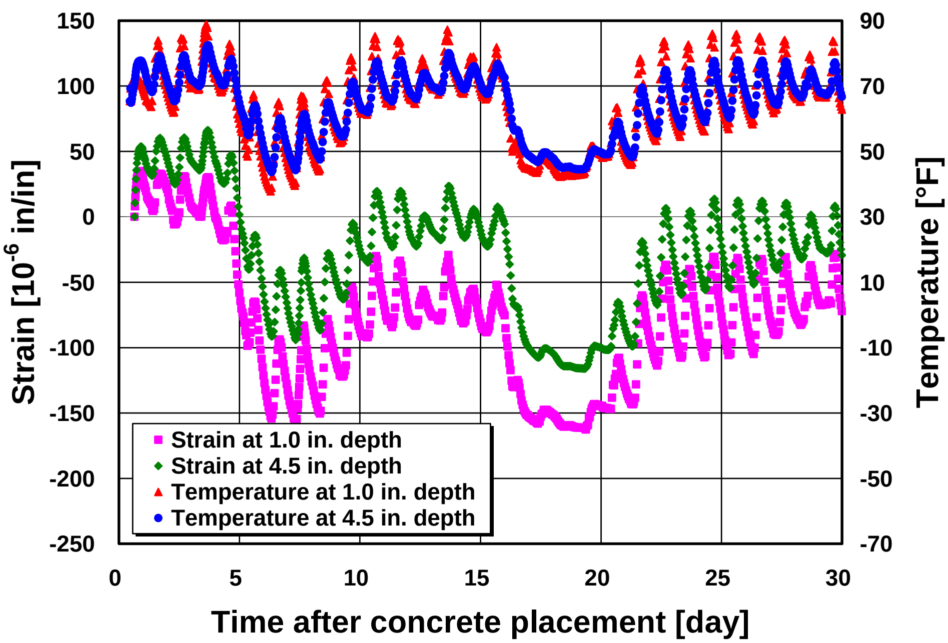


(b) Temperature at different depths

Figure 3.3 (a) and (b) Measured data for one month in field instrumentation

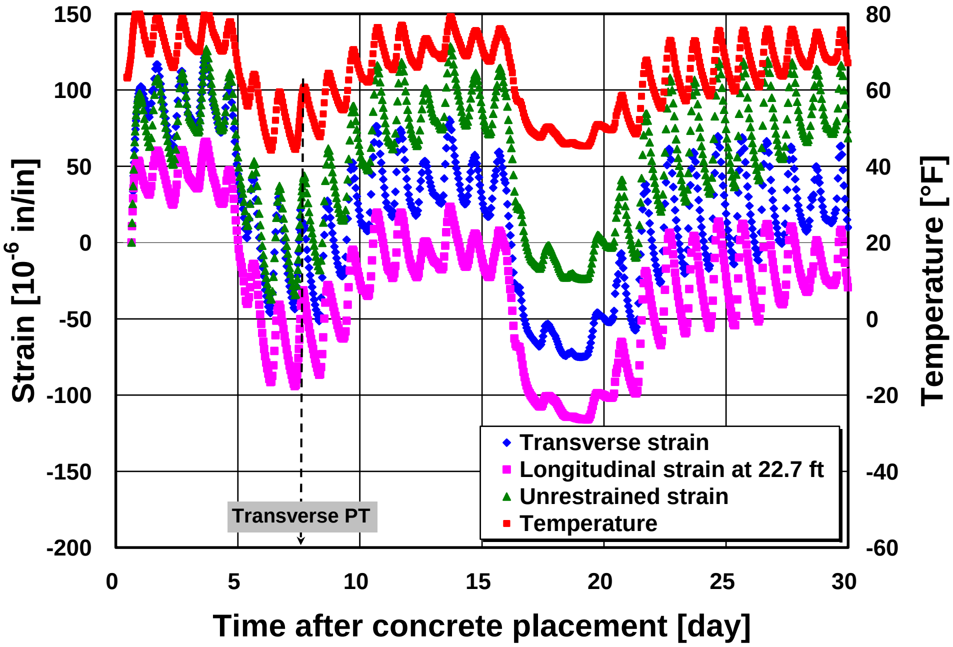


(c) RH at different depths

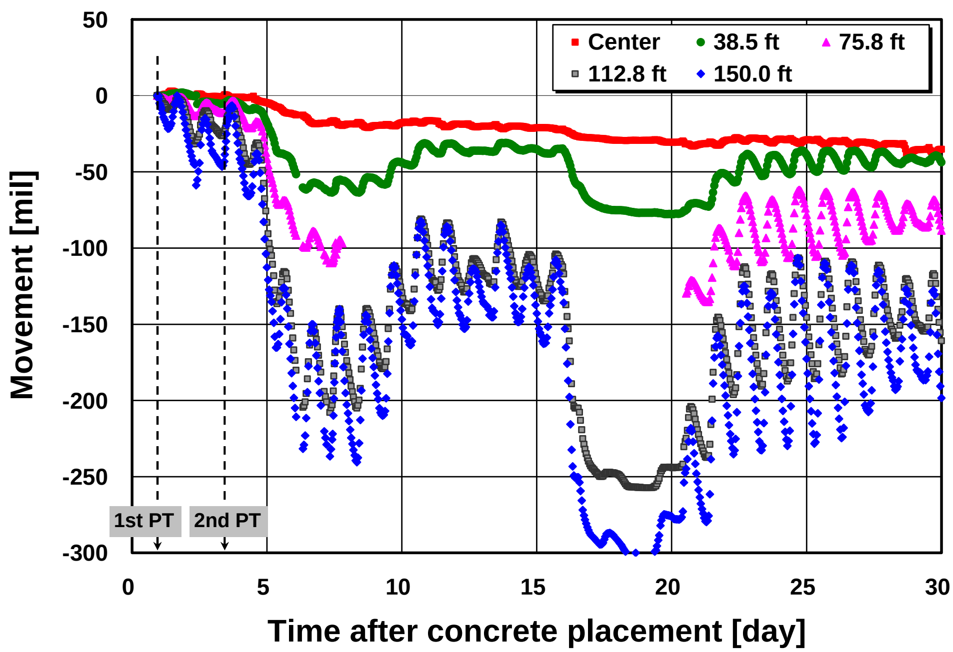


(d) Temperature and longitudinal strain of VWSG-C gages

Figure 3.3 (c) and (d) Measured data for one month in field instrumentation



(e) Measured transverse, longitudinal and unrestrained strains



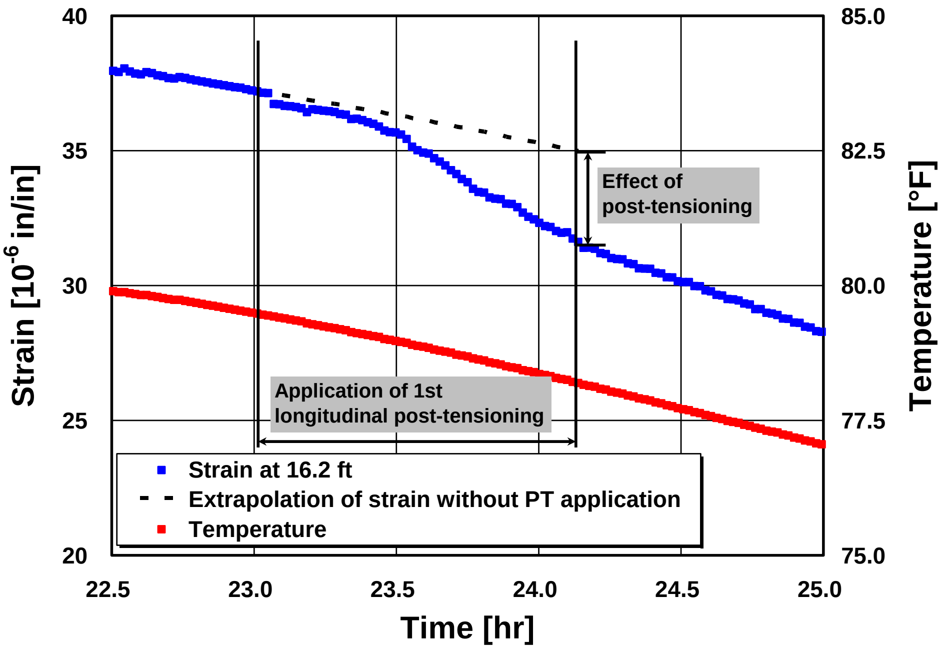
(f) Longitudinal movement of slab at different locations from slab center

Figure 3.3 (e) and (f) Measured data for one month in field instrumentation

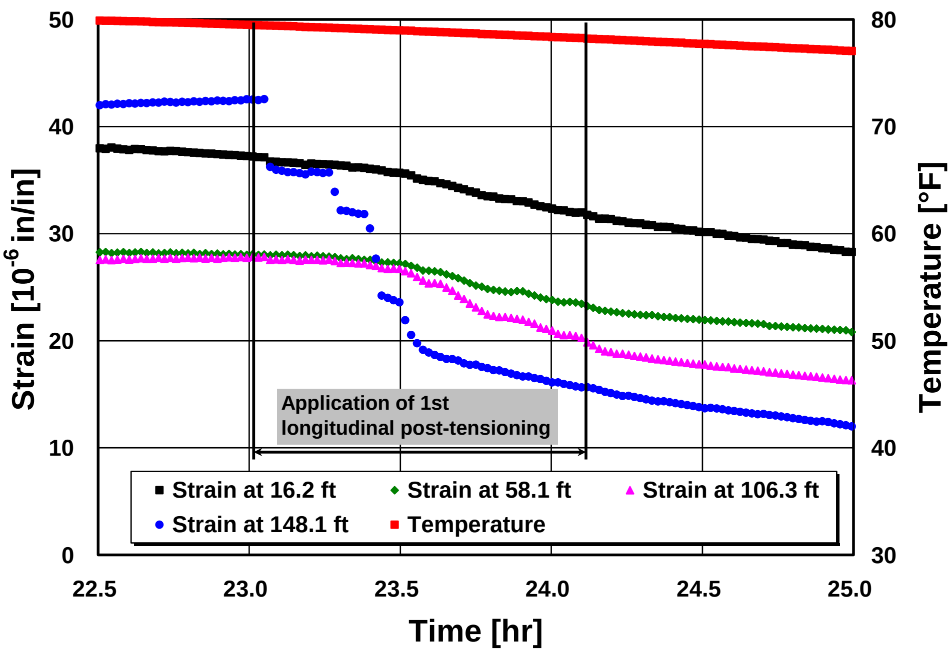
3.2.2 BEHAVIOR OF PCP DUE TO PT APPLICATION

Figure 3.4-(a) explains how to estimate the effective variation of concrete strain due to PT application. Because the slab temperature changed continuously, the measured strain during PT application may

include strain changes due to temperature variation even though the PT was applied for a short period. This may overestimate or underestimate the effect of PT application on strain changes in a concrete slab. In the case of the first longitudinal PT application, a simple difference between before and after PT applications would overestimate strain change because temperature was decreasing during the first PT application. Therefore, the net variation of strain due only to PT was calculated by subtracting strain that would have been obtained strictly with temperature effects without PT from strain after PT, as shown in Figure 3.4-(a). The same approach was used to estimate transverse strain and slab movement due to PT application. Figure 3.4-(b) represents the strain changes of concrete elements at various longitudinal locations near tendon T1 measured with VWSG-1 gages due to the first longitudinal PT application. The maximum variation of strain occurred near the armor joint, while the strain variation near the slab center was smallest. It is believed that both the subbase friction and bars driven into the subbase to support transverse post-tensioning steel (called vertical bar) caused this difference. The strain variations of concrete elements near tendon T19 measured with VWSG-19 gages, as shown in Figure 3.4-(c), were similar to strains of VWSG-1 gages. Information in Figures 3.4-(b) and 3.4-(c) indicates the concrete element was compressed more near the armor joint and less near the slab center by longitudinal PT. It is interesting to note that the strain variation near the armor joint (strain at 148.1 ft) in Figure 3.4-(c) started to be compressed later compared to the other elements. It suggests that the concrete element near the armor joint was compressed primarily by T19 or tendons near T19, while elements in the other locations along T19 were compressed by applications of other tendons as well. The effective region of PT force was localized near the armor joint even though the dead end anchor placed near the armor joint was 6.0-in. wide and 3.3-in. high. Figures 3.4-(a), (b), and (c) also show the effect of creep after PT application. Because the compressive stress was introduced by PT application, variations of strain due to temperature changes became greater after PT application than before PT application. Figure 3.4-(d) shows the distribution of longitudinal concrete strain from the slab center to the armor joint due to the first PT application. It manifests the effect of friction on the development of concrete strain in a PCP slab. The PT force applied to the concrete element was decreased by frictional resistance approaching the slab center. Therefore, the effective concrete strain became decreased as the distance from the armor joint increased. Figures 3.4-(e) and (f) show the longitudinal movement and its distribution along the slab due to the first longitudinal PT application, respectively. When the first longitudinal PT was applied, the slab moved toward its center, which means the slab was compressed. The magnitude of slab movement was significantly decreased at the slab center. This result also indicates the effect of frictional resistance on the slab movement due to PT application. The effective post-tensioning force to the concrete slab was reduced by the frictional resistance and vertical bar as the concrete element moved away from the armor joint. The results in Figure 3.4-(e) are consistent with measured strain in Figures 3.4-(b), (c), and (d). Figure 3.4-(e) also indicates the effect of creep on the movement of slab. After PT application, the slab still moved quickly toward slab center compared to that before PT application. The similar distribution of concrete strain changes and slab movement were recorded when the second longitudinal PT was applied to slab as well.

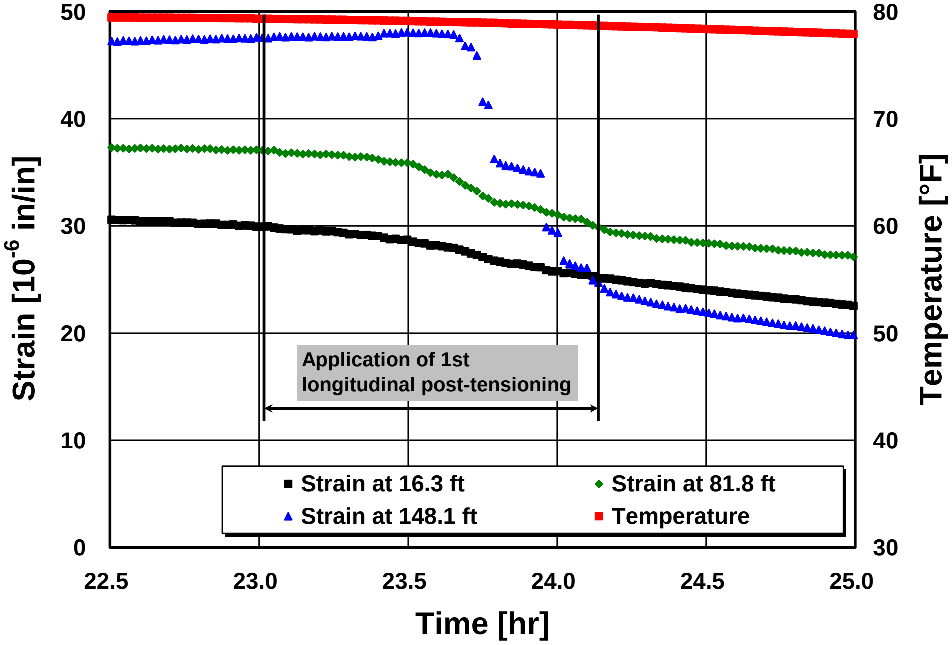


(a) Estimation of effective strain change due to first longitudinal PT

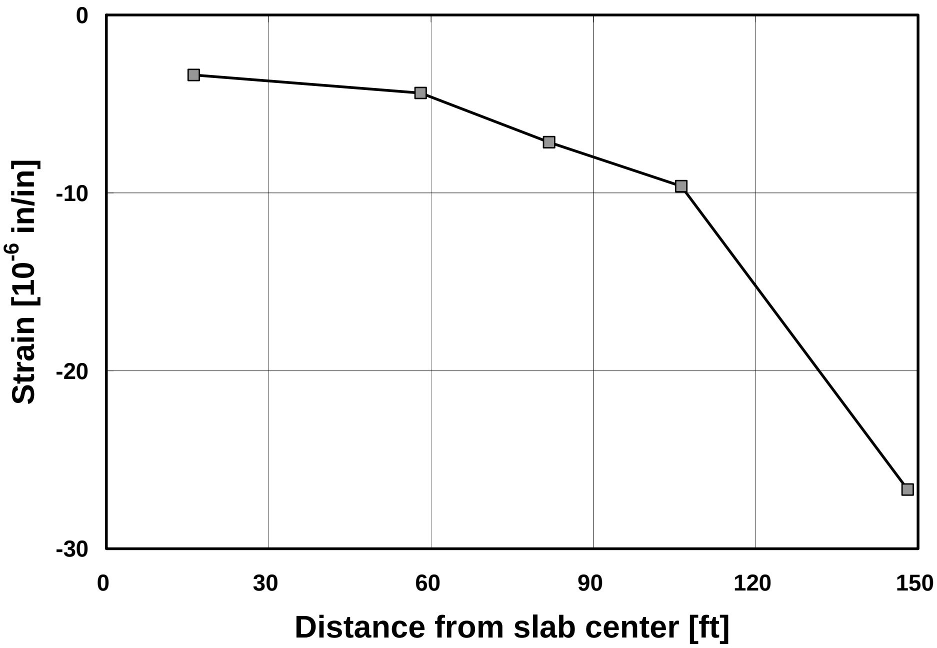


(b) Strain changes of VWSG-1 gages due to first longitudinal PT

Figure 3.4 (a) and (b) Concrete strain and slab movement due to first longitudinal PT application

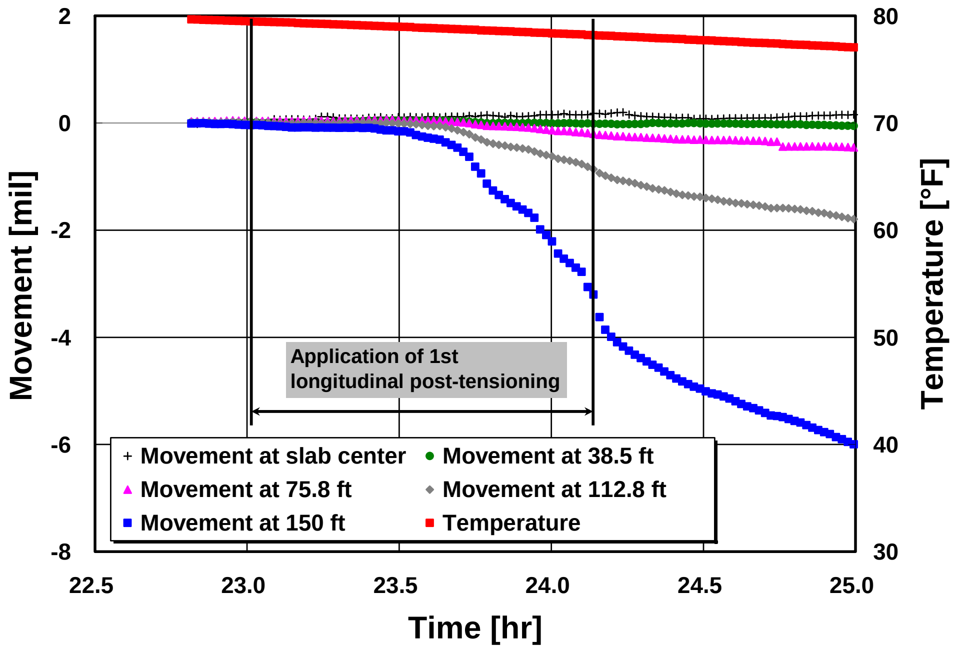


(c) Strain changes of VWSG-19 gages due to first longitudinal PT

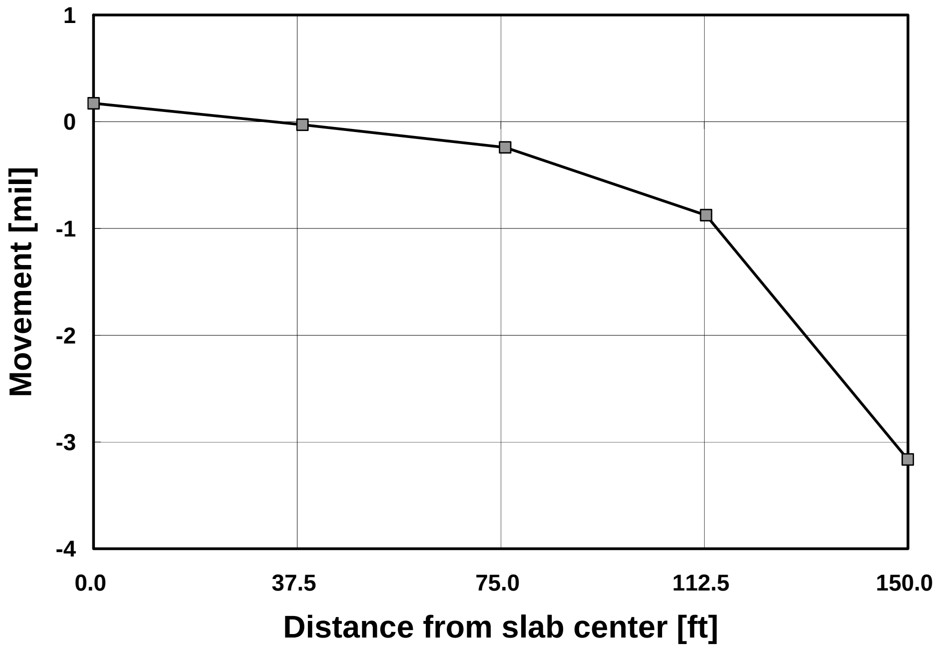


(d) Distribution of longitudinal strain change due to first longitudinal PT

Figure 3.4 (c) and (d) Concrete strain and slab movement due to first longitudinal PT application



(e) Longitudinal slab movement at different locations due to first longitudinal



(f) Distribution of longitudinal slab movement due to first longitudinal PT

Figure 3.4 (e) and (f) Concrete strain and slab movement due to first longitudinal PT application

Table 3.2 summarizes the variation of compressive strain and corresponding compressive stress due to longitudinal PT application. The elastic modulus in Table 3.2 was used to calculate compressive stress. It indicates that the compressive stress induced by longitudinal PT was smaller in the region near the slab center than in the region near the armor joint. The friction and vertical bars were considered to cause this difference. Since the 2nd longitudinal post-tensioning of 46.6 kips per strand is specified with 15-in spacing between longitudinal tendons, the equivalent average concrete compressive pre-stress is 345 psi. The applied pre-stress in concrete in longitudinal direction is much smaller than the value that would have been obtained if all the assumptions made in the design of PCP were realistic and the specified amount of post-tensioning force was applied.

Table 3.2 Variation of strain and corresponding stress due to longitudinal PT

Distance from slab center [ft]	1st longitudinal PT		2nd longitudinal PT	
	Compressive strain change [10^{-6} in./in.]	Compressive stress [psi]	Compressive strain change [10^{-6} in./in.]	Compressive stress [psi]
16.2	3.4	6.1	9.1	46.8
58.1	4.4	7.9	11.6	59.6
81.8	7.2	12.9	17.1	88.1
106.3	9.6	17.4	19.3	99.2
148.1	26.7	48.3	36.1	186.1

Figure 3.5 illustrates the variations of concrete strain in transverse and longitudinal directions when the transverse PT was introduced. It indicates that the transverse PT efficiently induced the compressive concrete stress in the transverse direction. As expected, the transverse strain decreased rather quickly due to transverse PT. The longitudinal strain, however, was slightly increased due to the Poisson effect. The amounts of transverse and longitudinal strains were -21.5×10^{-6} in./in. and 3.1×10^{-6} in./in., respectively. The corresponding compressive stress—114.2 *psi*—was introduced in the concrete slab in the transverse direction. Since the transverse post-tensioning of 46.6 kips per bar is specified with 6-ft spacing between transverse post-tensioning steel, the equivalent average concrete compressive pre-stress is 72 psi in the transverse direction. The applied pre-stress in concrete is much larger than the specified value.

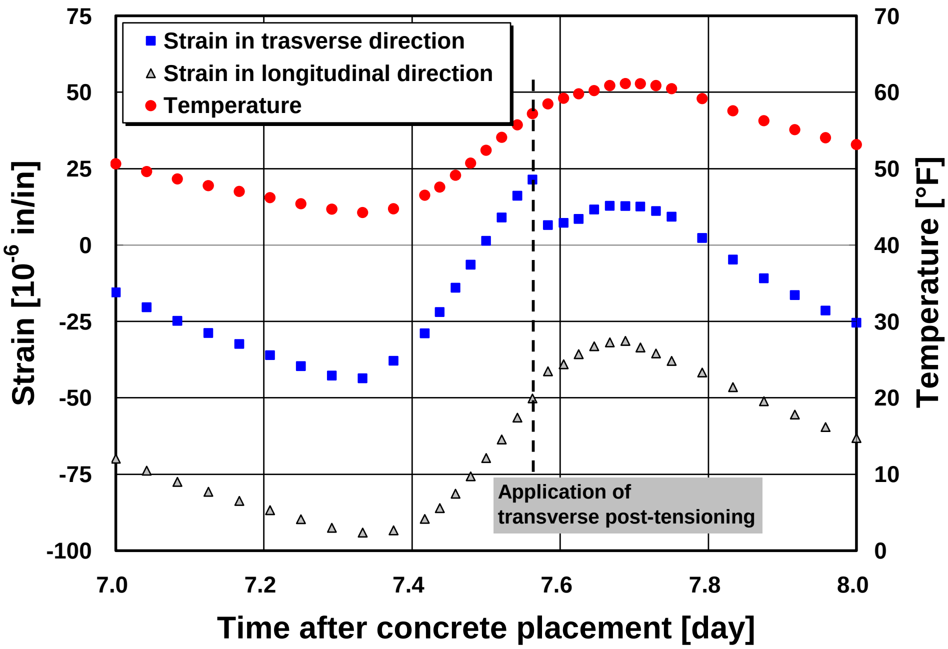


Figure 3.5 Variation of transverse and longitudinal strains in concrete slab due to transverse PT application

3.2.3 BEHAVIOR OF PCP DUE TO AVERAGE SLAB TEMPERATURE VARIATIONS

As mentioned earlier, the movement of the concrete slab consisted of axial and bending components. A particular period (26^{th} day 5:00 p.m. to 27^{th} day 4:00 a.m.), in which the variation of temperature was relatively high for the hardened concrete, was selected to thoroughly investigate the slab's behavior under environmental loading. The data shown in Figure 3.3 were analyzed for the above specified period.

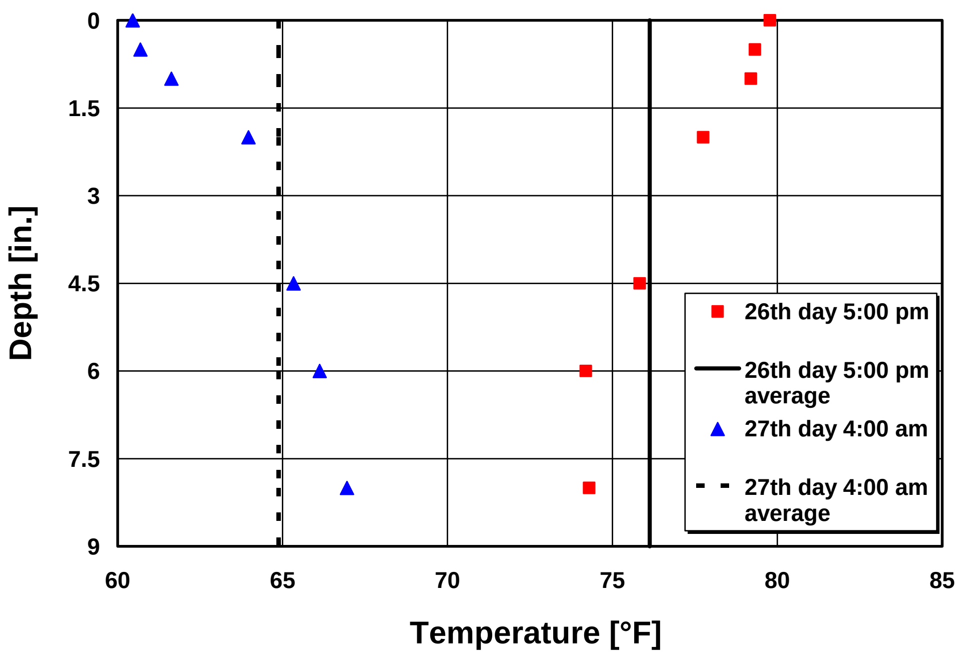
Figure 3.6-(a) illustrates the measured temperature profile of the PCP slab for the specified period (see Figure 3.3-(b)). It indicates that the slab had nonlinear temperature variations through the slab depth and the average temperature in the section was almost similar to temperature at mid-depth. Because the axial component of slab movement is dependent on the sectional average of the temperature profile (*3*), the temperature at mid-depth can be correlated with axial deformation. Figure 3.6-(b) shows the longitudinal concrete strain and temperature measured with VWSG-1 gages for the 25^{th} to 27^{th} day from the concrete placement. The variation of concrete strain near the armor joint was higher than that near slab center even though all measured concrete elements were subject to very similar temperature variations. It indicates that the slab's axial thermal deformation was restrained more near the slab center and less near the armor joint. Figure 3.6-(c) illustrates concrete strains of VWSG-1 gages along with temperature changes for the above specified period. The coefficient of the linear regression equation is related to the degree of restraint on concrete volume changes. If there is no restraint on concrete volume changes, the slope of the line should be close to the slope of unrestrained concrete,

which is equal to the coefficient of thermal expansion (*6*). The degree of restraint can be defined as follows (*7*):

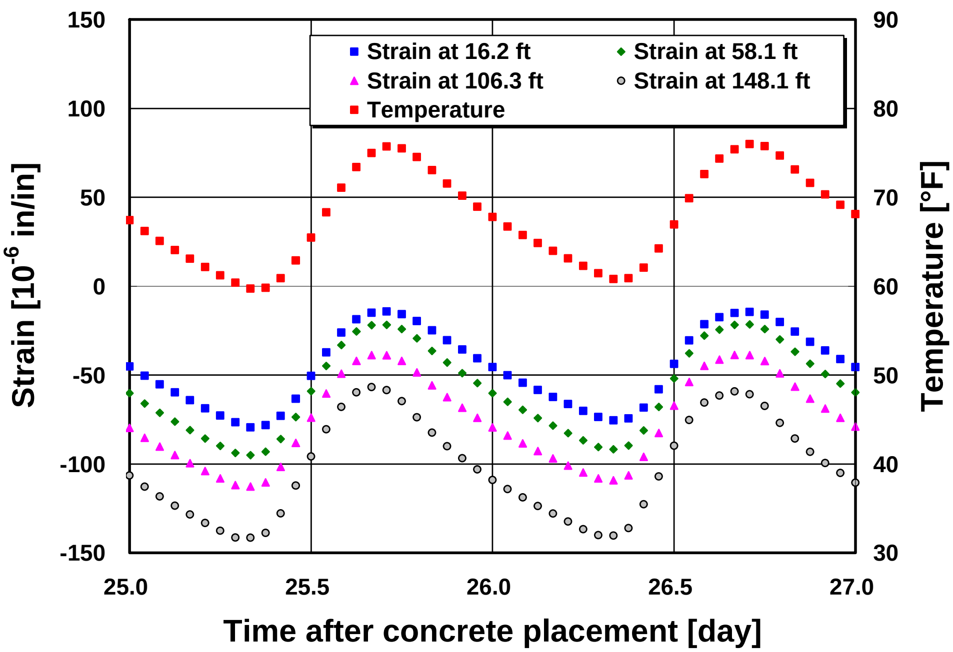
$$R = (s_f - s_a) / s_f \qquad (3.1)$$

where R = degree of restraint [-]; s_f = slope of unrestrained concrete [10^{-6} in./in./˚F]; and s_a = slope of restrained concrete [10^{-6} in./in./˚F].

The degree of restraint of the concrete element at different locations from PCP slab center is shown in Figure 3.6-(d). Figures 3.6-(c) and (d) illustrate that more restraint existed as the concrete element became closer to slab center. It suggests that the slab's center region was subjected to more restraint due to friction and the vertical bars, which would result in the development of higher stresses in concrete due to environmental and wheel loading. In this project, one layer of polyethylene sheet was placed to reduce the subbase friction and pre-stress loss. An option of two layers of polyethylene sheet was considered but abandoned due to construction difficulty. Because two layers of polyethylene sheet provide less subbase friction, their use would be more desirable in addressing pre-stress loss.

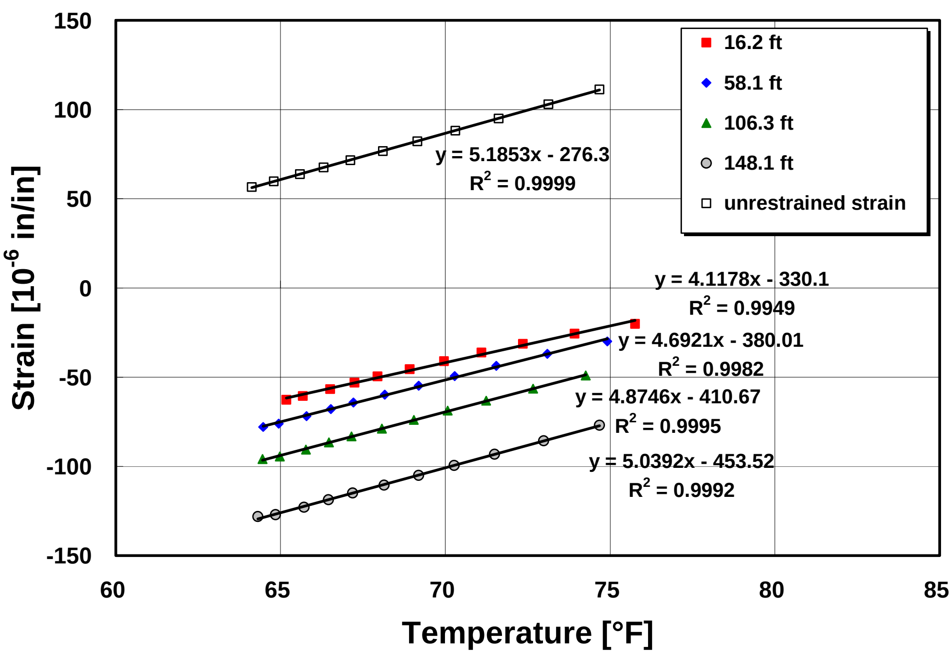


(a) Measured temperature profile

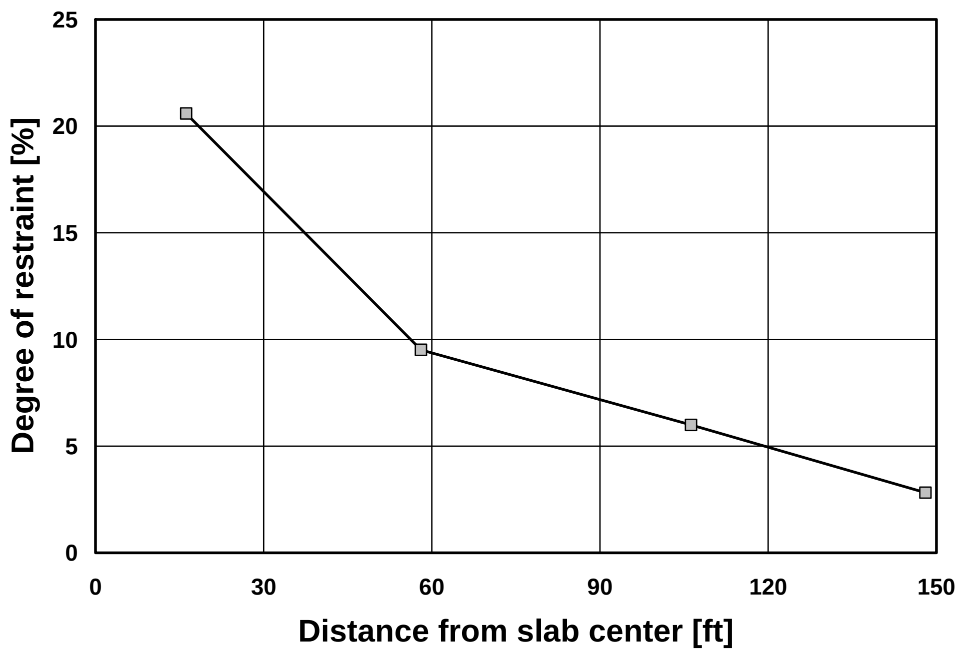


(b) Concrete strain and temperature measured with VWSG-1 gages for 25th to 27th day

Figure 3.6 (a) and (b) Distribution of longitudinal strain and degree of restraint of concrete element at different locations in PCP slab for specified period



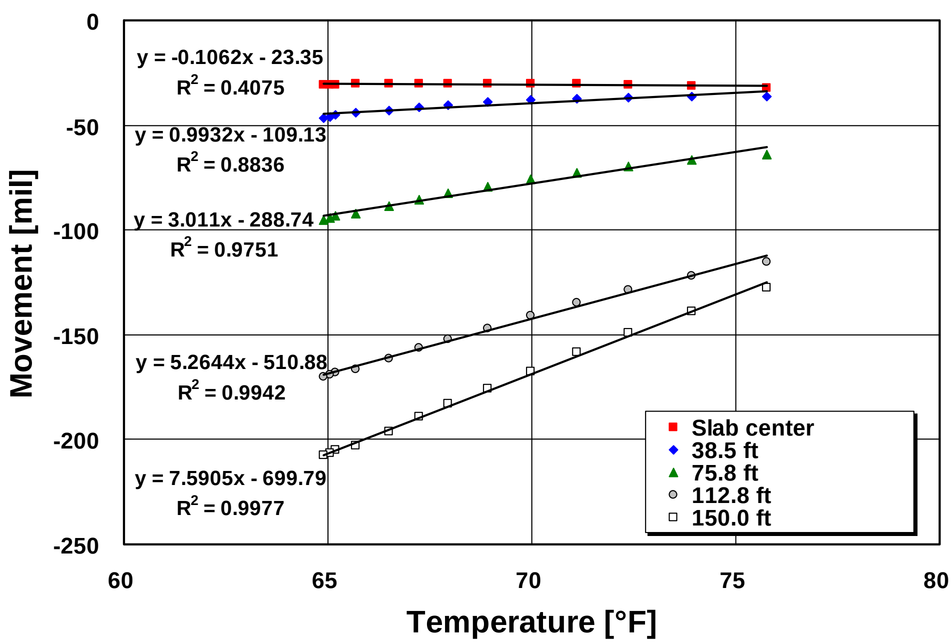
(c) Strain versus temperature at mid-depth of slab



(d) Distribution of degree of restraint of concrete slab

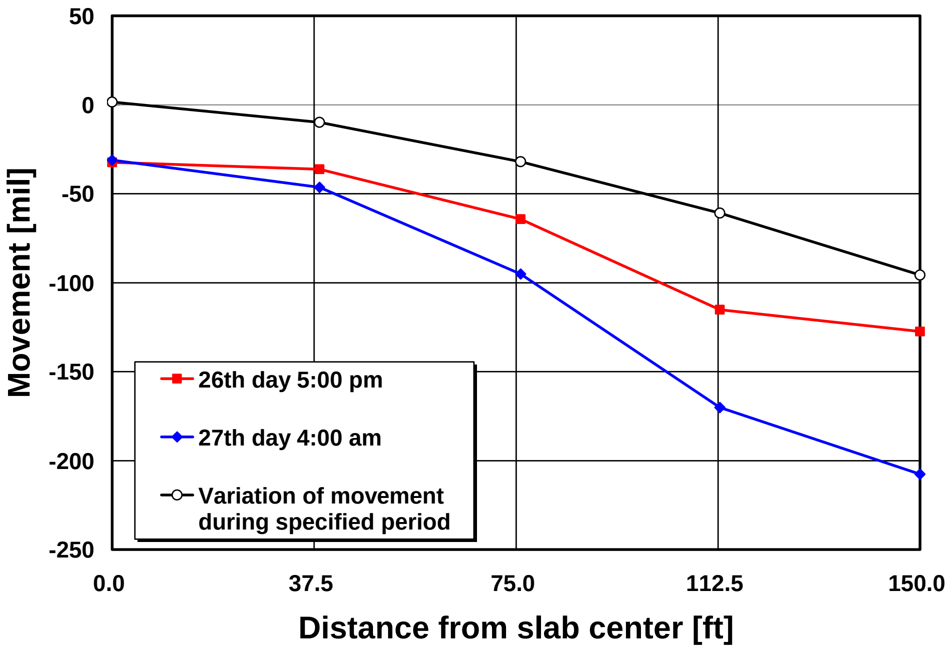
Figure 3.6 (c) and (d) Distribution of longitudinal strain and degree of restraint of concrete element at different locations in PCP slab for specified period

Figure 3.7-(a) illustrates the relationship between longitudinal slab movement at different locations along the slab edge and temperature changes for the specified period (26th day 5:00 p.m. to 27th day 4:00 a.m.). As expected, the slope was highest at the armor joint and smallest at slab center. The degree of restraint for slab movement can be also evaluated based on the equation (3.1). Because the slope of unrestrained strain in Figure 3.6-(c) is equal to the coefficient of thermal expansion (*8, 9*), free thermal movement of PCP without restraint can be numerically estimated and the degree of restraint for slab movement can be also calculated with equation (3.1). From the slope of the armor joint movement in Figure 3.7-(a), the calculated degree of restraint at the slab's armor joint was 19%, which means that 19% of free slab movement was restrained at the armor joint. Figure 3.7-(b) shows the distribution of slab movement in the longitudinal direction for the specified period. The variation of movement during the specified period displayed nonlinear distribution along the distance from slab center. This result is consistent with the slopes of measured strains as shown in Figure 3.6-(c) because the concrete strain, which is a spatial derivative of movement, increased nonlinearly as the measured element approached the armor joint.



(a) Longitudinal movement of slab versus temperature at mid-depth

Figure 3.7 (a) Distribution of longitudinal slab movement under environmental loading for the specified

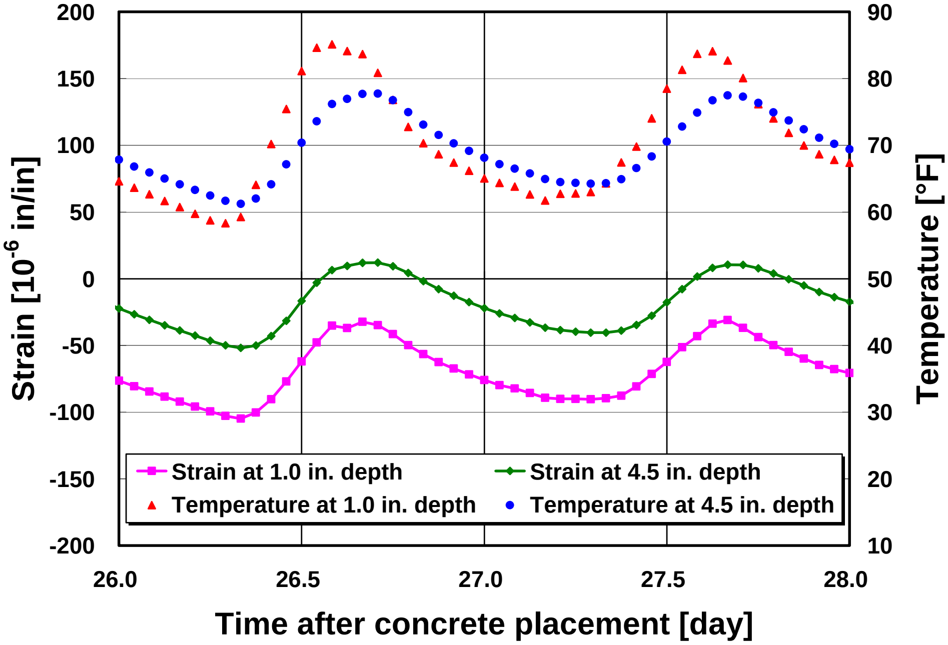


(b) Distribution of longitudinal movement

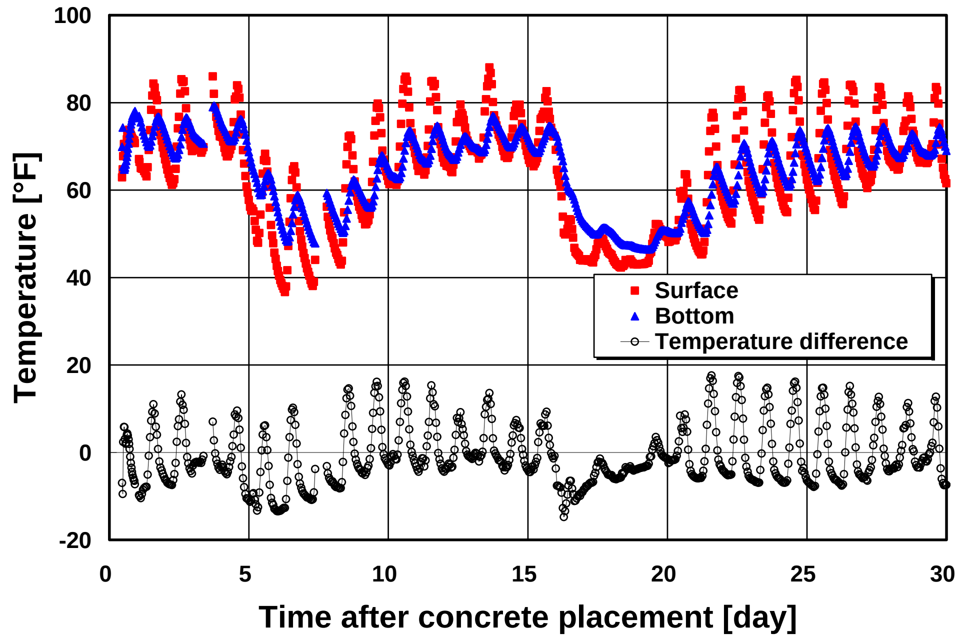
Figure 3.7 (b) Distribution of longitudinal slab movement under environmental loading for the specified period

3.2.4 BEHAVIOR OF PCP SLAB DUE TO TEMPERATURE DIFFERENTIAL

Figure 3.8-(a) shows concrete temperature and strains at different depths measured by VWSG-C in Figure 3.1. The variation of concrete strain at 1.0-in depth was comparable to that at 4.5-in. depth even though the variation of temperature was higher at 1.0-in. depth than at 4.5-in. This implies that the curling stress exists in the measured concrete element. The measured element was located near the slab center (see Figure 3.1) and thus the bending component of slab movement was restrained by the slab's self-weight. Therefore, the variation of strain at 1.0-in. depth in Figure 3.8-(a) mainly included the concrete slab's axial movement. Figure 3.8-(b) illustrates temperatures at the surface and bottom of the slab and corresponding temperature difference. The maximum difference—18 °F—occurred at the 21st day. If the temperature gradient can be assumed to be linear, the curling stress—±320 psi—can occur in a PCP slab. In the curling stress calculation, the slope of unrestrained strain in Figure 3.6-(c) was used as the coefficient of thermal expansion and 28-day elastic modulus was assumed. As shown in Figure 3.6-(a), however, the PCP had a nonlinear temperature gradient and thus the curling stress will be affected by not only temperature difference between the top and bottom of slab, but also the degree of nonlinearity of temperature gradient (*3*).



(a) Variation of strains and temperature at different depths of concrete slab



(b) Variation of temperatures at top and bottom of slab and corresponding temperature difference

Figure 3.8 Curling strain and stress in PCP slab

3.2.5 LONG-TERM BEHAVIOR OF PCP SLAB DUE TO CREEP AND SHRINKAGE

The measured concrete strain by VWSG is total strain, which includes strains due to temperature and moisture variations as well as strains due to stress and creep (*7*). Because the longitudinal and transverse strains existed in the range of smaller values than unrestrained strain as shown in Figure 3.3-

(e), the concrete element at mid-depth of PCP may be in a compression stress state. This compressive stress will induce such additional strain as creep, which was shown in Figures 3.4-(a), (b), (c), and (e). The variations of strain and movement along with temperature changes were different before/after the longitudinal PT application. Moisture differential would also affect long-term behavior of PCP. Figure 3.9 shows the variation of longitudinal slab movement at different locations when the temperature of concrete at mid-depth was 70 °F. It indicates that the slab contracted with time after placement. It is postulated that creep and shrinkage induced this gradual slab contraction. Therefore, creep and shrinkage effects should be included in the design of initial joint width.

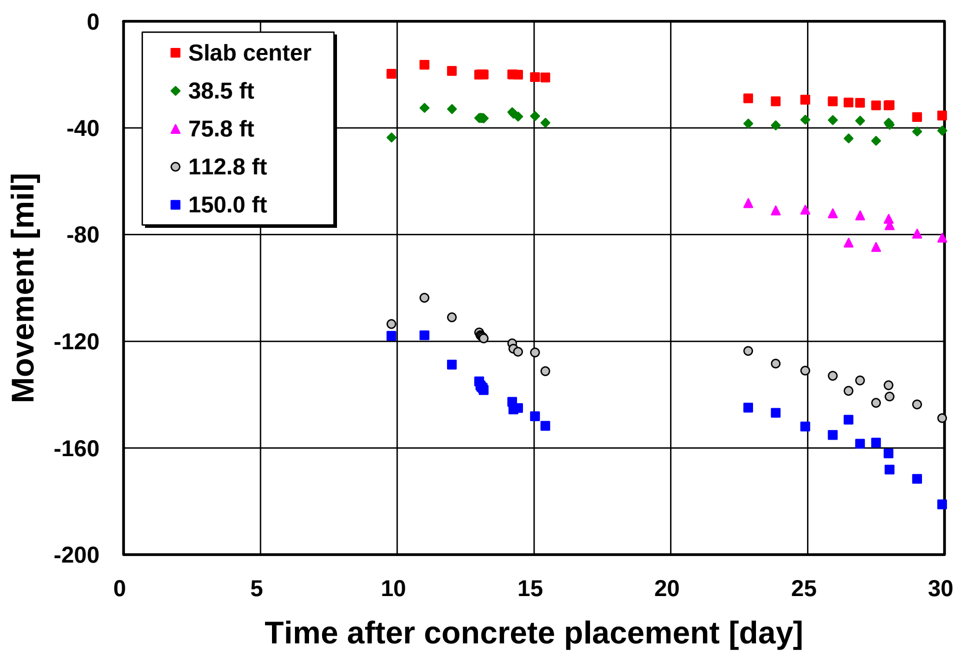


Figure 3.9 Variations of slab movement at different locations due to creep and shrinkage

CHAPTER 4 CALIBRATION OF THE MECHANISTIC MODEL PCP 3.0

4.1 BACKGROUND

PSCP 3.0 is a mechanistic model developed for the analysis of stresses and displacements in cast-in-place post-tensioned concrete pavement caused by climatic variables, wheel loads, and changes in concrete material properties. This program can be used for the design of PCP, namely slab thickness and tendon size and spacing for given subbase structural and frictional condition and climatic conditions. The objective of the calibration efforts made in this project was that, once the program is calibrated, PCP design can be developed on a routine bases at TxDOT using PCP 3.0. This program is an analysis program, not a design program. The user has to try a number of combinations of the inputs and determine the design that will provide optimum structural responses and accordingly long-term performance. The model, developed originally in the 1986, has been revised over the years, and the most current one is PCP version 3.0. The PCP 3.0 program consists of a source code written in Fortran 90 programming language and a user interface developed with Microsoft® Visual Basic 6.0. This setup is thought to be reliable and, most importantly, user friendly. The basic assumptions made in the development of the PCP model are as follows:

1. Concrete is homogeneous, isotropic, and linear elastic.
2. The slab behaves elastically under all loading conditions, and total stresses are obtained by superposition of stresses attributed to wheel loading, temperature curling, pre-stress, and concrete creep and shrinkage.
3. The program incorporates the inelastic nature of slab base friction forces.

4.2 BRIEF DESCRIPTION OF THE MODEL PCP 3.0

The description of the original model is fully described in the report by Mendoza, et al. and not repeated in this report. Rather, a brief description of inputs and outputs of the program is provided. The input values shown in the screens in this chapter were the actual values evaluated in the field for the calibration of the program, unless default values were used when the testing was not conducted for the specific variables.

4.2.1 INPUT REQUIREMENTS FOR PCP 3.0

The input data to PCP 3.0 is classified in ten groups, described as follows:

1) Geometry

This is the first input screen as shown in Figure 4.1. After the description of the analysis task, it requests three basic geometric information of the PCP slab: slab length, slab width, and slab thickness. As described earlier, this program is for the analysis of the PCP system, not a design program. For the optimum use of this program for PCP design, a user develops tentative PCP designs, primarily slab length and slab thickness, and conducts analyses for those tentative designs. The behavior of the PCP with each design is evaluated and the optimum design is selected that provides the best potential performance.

2) Concrete Properties

The next input screen, shown in Figure 4.2, asks for various properties of the concrete mix that will be used for the construction of PCP. The default values are provided by the program in case the user does not have information. In this screen, five concrete properties are required; coefficient of thermal expansion (CTE), ultimate drying shrinkage strain, unit weight of concrete, Poisson's ratio, and creep coefficient. Unit weight of concrete and Poisson's ratio do not have substantial effects on PCP behavior, so the value between 140 and 150 pounds per cubic ft for unit weight and 0.15 for Poisson's ratio will be adequate. However, the other three variables, especially CTE, have significant effects on PCP behavior and performance. TxDOT has the capability to evaluate CTE of concrete fairly accurately, and the user is encouraged to have the CTE of the concrete evaluated by CSTMP in Austin. Ultimate drying shrinkage strain is somewhat difficult to evaluate. Also, it is dependent on not only concrete material properties, but curing condition as well, which cannot be predicted with a reasonable accuracy during the design phase of PCP. The research team of this implementation project evaluated drying shrinkage of concrete in several CRCP projects and this PCP project, and found that the use of default value provided in the program (0.0003 in/in) is conservative. Accordingly, with good estimates of the ultimate drying shrinkage, it may be acceptable to use the default value provided in the program. The last input of concrete property on this screen is the concrete creep coefficient. The creep coefficient is defined as the final asymptotic amount of additional creep strain that occurs over time divided by the initial elastic strain that occurs when load is first applied. Creep of the concrete is quite difficult to evaluate. Fortunately, its effects are not as significant as CTE or ultimate drying shrinkage, and it is recommended that the default value provided in the program (2.10 in/in) be used.

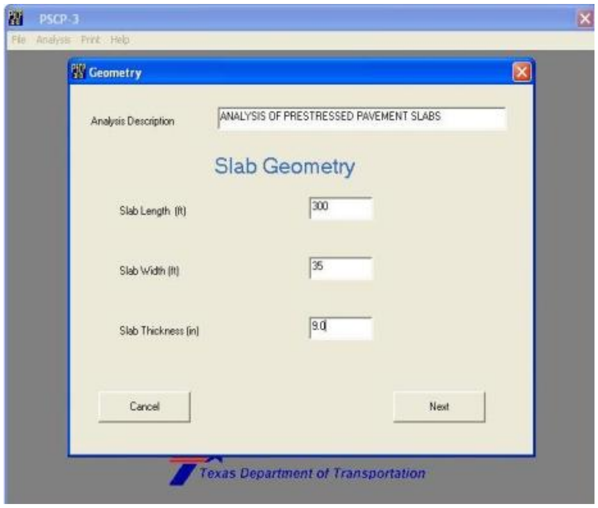


Figure 4.1 Input screen for slab geometry

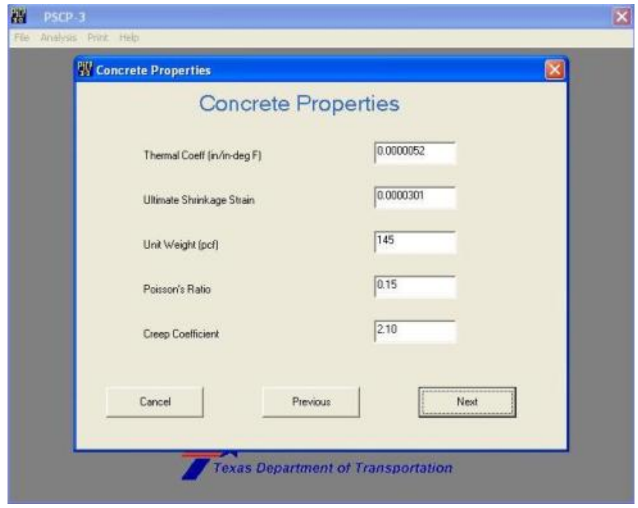


Figure 4.2 Input screen for concrete properties

3) Coarse Aggregate Type

In addition to the general concrete properties required in the second screen shown in Figure 4.2, more specific information is required for to describe the concrete coarse aggregate type. Coarse aggregate type has substantial effects on modulus of elasticity of concrete and coefficient of thermal expansion (CTE). Since CTE value is specified in the second input screen, the information on the coarse aggregate type provided on this screen is apparently used to estimate compressive strength and modulus of elasticity of concrete. There is a comment on this screen, "If "Aggregate Type" is chosen, strength development curve is developed from the recommendations given on Project CTR-422: Thu Univ. of Texas at Austin. If you wish to input Age-Compressive Strength relation for concrete, do not choose an aggregate type." This comment implies that if a coarse aggregate type is chosen, then the program will estimate compressive strength. However, as will be explained in the next screen, that's not the case. Regardless whether the user specifies the coarse aggregate type or not, the program requires that the age-strength relationship be provided in the next screen. It appears that the aggregate selection on this screen is used for estimating modulus of elasticity of concrete, but not concrete strength. It is not known if the comment on this screen is inaccurate or if there is a minor problem in this portion of the input algorithm in the program. Based on the experience with PCP 3.0, it appears that the selection of coarse aggregate type is used for the estimation of concrete modulus of elasticity only, not the strength. The default values of modulus of elasticity for the eight coarse aggregate types in this program were obtained from the findings from a TxDOT research project 422 conducted in the 1980s. In that study, various coarse aggregates were collected that were used for concrete paving in Texas at that time and the effects of coarse aggregates on strength and modulus of elasticity were evaluated. Based on the testing results, regression equations were developed, which were included in this program. It might be that during the design phase of PCP, the coarse aggregate to be used in the project might not be known.

In that case, the input for coarse aggregate type can be skipped and the 28-day concrete modulus of elasticity can be entered in the screen. If the modulus value is not known during the design phase of PCP, it could be estimated from the ACI equation (2.1) using 28-day compressive strength of concrete.

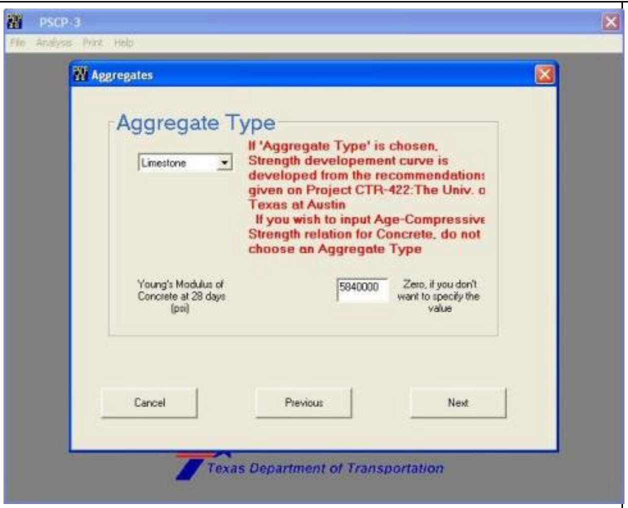


Figure 4.3 Input screen for coarse aggregate type

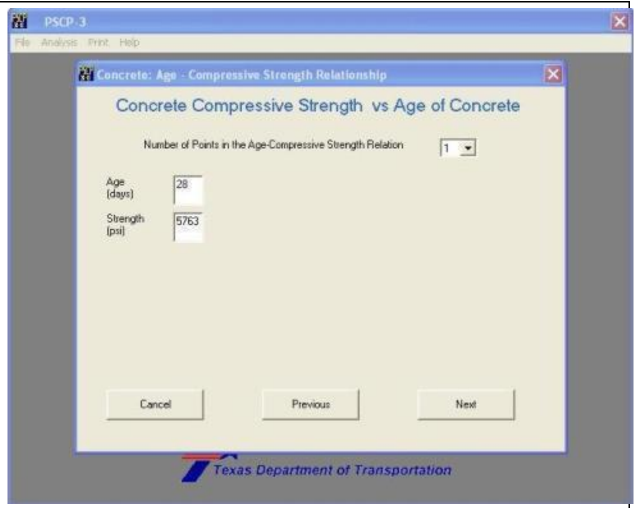


Figure 4.4 Input screen for age-strength of concrete

4) Concrete Compressive Strength vs Age

In the next screen (Figure 4.4), the user specifies the number of points in the age-compressive strength relation, and provides corresponding sets of age-strength values. As stated earlier, the selection of coarse aggregate type does not provide this relationship. In other words, whether the user chooses coarse aggregate type or not in the previous screen, age-strength relation should be provided in this screen. From a practical standpoint, the age-strength relationship might not be known during the design phase of PCP. However, the minimum concrete strength at 28 days must be included in the specifications. The user can use this 28-day strength value for the input in this screen. In Figure 4.4, 28-day strength of 5,763 psi was used, since it was the actual strength of the concrete used in the slab where gages were installed for the calibration of the program.

5) Coefficient of Friction–Displacement Relationship

The next screen requires input for modulus of subgrade reaction and coefficient of friction at various displacements. Since frictional resistance at the slab-subbase interface causes the loss in the applied pre-stress, it is important to provide accurate information on the characteristics of slab-subbase friction. There are three options available in the program.

- Linear: The friction is assumed to behave linearly until it reaches the point at which the slab moves freely. In reality, the frictional relationship between slab and subbase is essentially nonlinear; however, this simplification is commonly used for pavements design.
- Exponential: Frictional characteristics can be modeled using an exponential function if two sets of displacement vs frictional stress are known.
- Multi-linear: When sufficient information is available for slab movements vs corresponding frictional stresses, this option can be selected. The program develops a multi-linear model based on the provided information.

For the calibration of the program, multi-linear model was chosen as shown in Figure 4.5. The values in this screen were selected from a study by Wesevich et al.

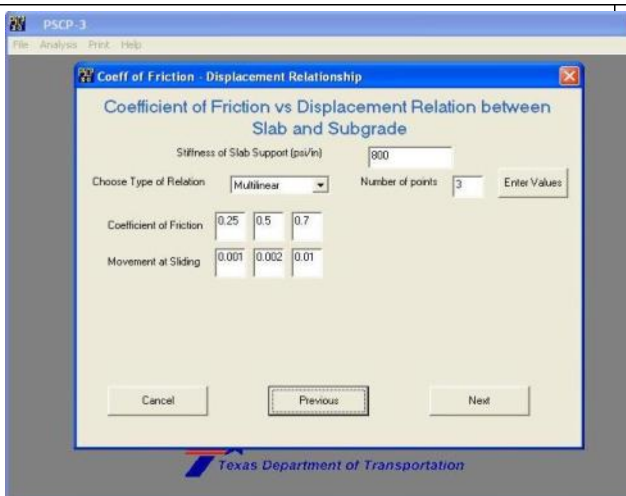


Figure 4.5 Input screen for subbase friction vs displacement

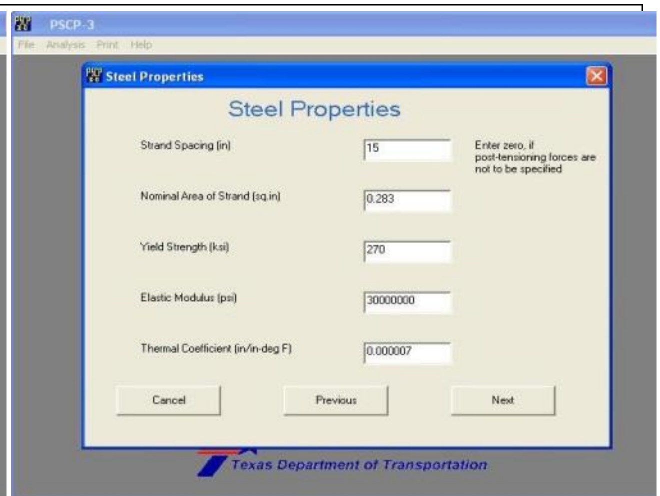


Figure 4.6 Input screen for subbase friction vs displacement

6) Steel Properties

Figure 4.6 shows that there are five variables required for reinforcement design. The first variable, which is one of the primary design variables, is longitudinal strand spacing. Since the concrete stress in PCP slab depends on three variables – environmental condition, wheel loading, slab thickness, and pre-stress level, for given concrete material properties and slab support condition – PCP design is to select an optimum combination of longitudinal strand spacing, strand force, and slab thickness. In this screen, five variables – longitudinal strand spacing, nominal area of strand, yield strength, modulus of elasticity, and thermal coefficient of strand.

7) Wheel Loading

PCP 3.0 considers static wheel−load application only, and the information required for the calculation of wheel load stress is as follows as shown in Figure 4.7:

- Age of concrete when the wheel load was applied (days),
- Design wheel load (lbs), and
- Wheel base radius (usually 6 inches)

Wheel load stress is computed using Westergaard's interior loading condition.

8) Temperature Data for Initial Period

Temperature history at early ages of PCP, called initial period in the program, is quite important in evaluating subsequent behavior of PCP slabs in terms of concrete stresses and displacements. The required inputs for concrete temperatures in the initial period are (1) time of setting, (2) setting temperature, (3) number of temperature data points for initial period, (4) number of subsequent periods for the analysis, (5) mid-depth temperature and top-down temperature differentials for the number of data points specified in (4). The input screen is shown in Figure 4.8. As stated earlier, the input data shown in this screen was actual data obtained in the slab that was heavily instrumented for the evaluation of PCP behavior for the calibration of this program.

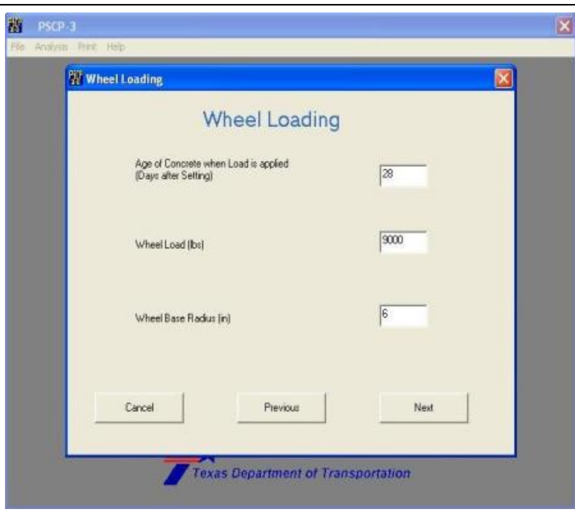


Figure 4.7 Input screen for wheel loading

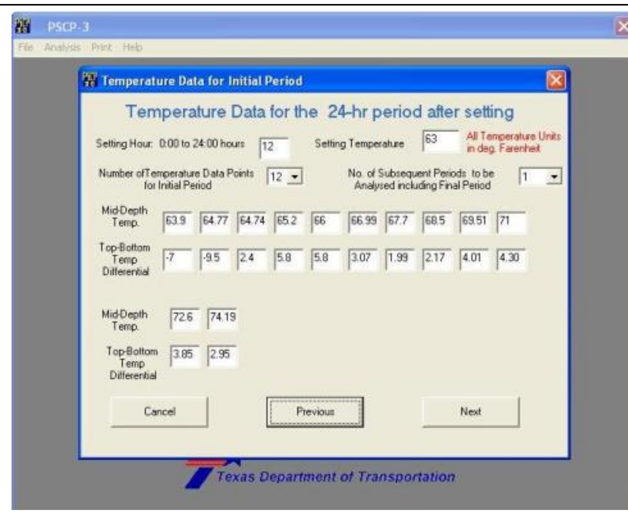


Figure 4.8 Input screen for initial period concrete temperature

9) Temperature Data for Subsequent Period

The program allows the analysis of the long-term PCP behavior due to temperature variations. For the long-term behavior analysis, the user is required to provide the time of analysis since setting (days), and slab's mid-depth temperature, top to bottom temperature differentials, and hour of day for 12 time periods, as shown in Figure 4.9.

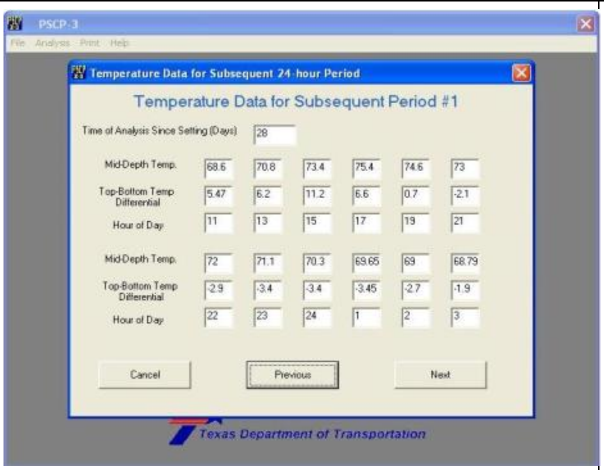


Figure 4.9 Input screen for concrete temperature for subsequent period

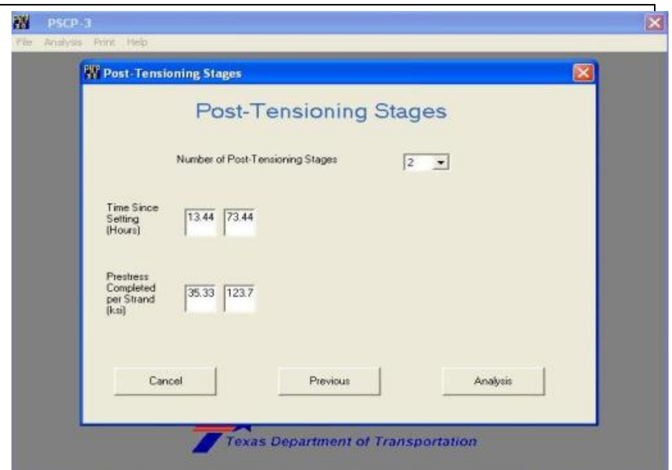


Figure 4.10 Input screen for information on post-tensing

10) Post-Tensioning Stages

The final input group is related to post-tensioning. Figure 4.10 shows the input screen. The number of post-tensioning stages needs to be specified. Normally, two stages are needed – first for initial post-tensioning to prevent early-age transverse cracking, and the final one to apply design tensioning force. For each stage, time of post-tensioning application after the concrete setting and applied force per strand are to be provided.

4.2.2 PRESENTATION OF OUTPUT

The outputs are provided in two formats – plots and test files. Figure 4.11 shows the first screen when output files are requested. For plots, there are three sections: initial period, final period, and comparisons. For initial period, there are four categories of outputs; (1) end movement, (2) slab curling movement, (3) total stresses, and (4) stress contributions. There are four locations where total concrete stresses are estimated; (1) top at mid-slab, (2) bottom at mid-slab, (3) top at the slab end, and (4) bottom at slab end. Stress contributions are also computed at the same four locations. For final period, end slab movement, curling movement along the slag in the longitudinal direction, and total concrete stresses are presented. For "Comparisons," the initial and final slab movements, top and bottom concrete stresses, and concrete stresses at the mid-slab and slab end (both at the top and at the bottom of the slab) are presented. For "Comparisons," the results are presented for the first 24 hours after the concrete placement.

This section briefly presents the outputs from the program for the input values shown in the Figures 4.1 through 4.10 above. In PCP 3.0, output information can be presented in two formats – plots and texts. Figure 4.12 presents the slab displacements at the armor joint. It shows that, for the first 24 hours, there was very little slab movement. It might be due to the small temperature variations within the time frame. Initial curling movement at the armor joint is shown in Figure 4.13. It shows that, at the beginning, there was upward curling, and with time, it became downward curling. Figure 4.14 illustrates the concrete stress at the top of the mid-slab for the first 24 hours. It shows that, initially there was tensile stress, and after initial post-tensioning, the concrete was in compression state.

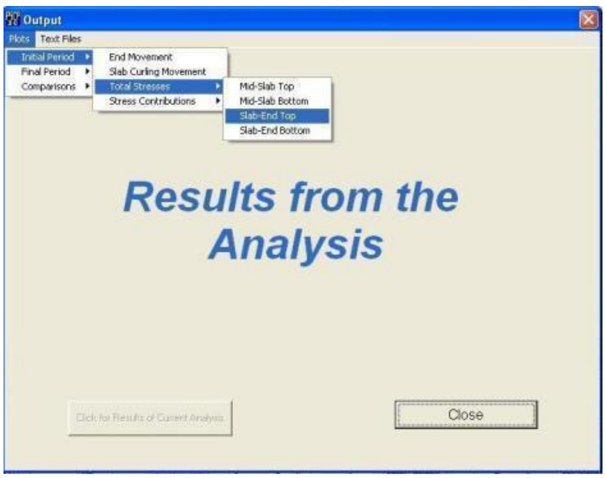


Figure 4.11 Output screen from PCP 3.0

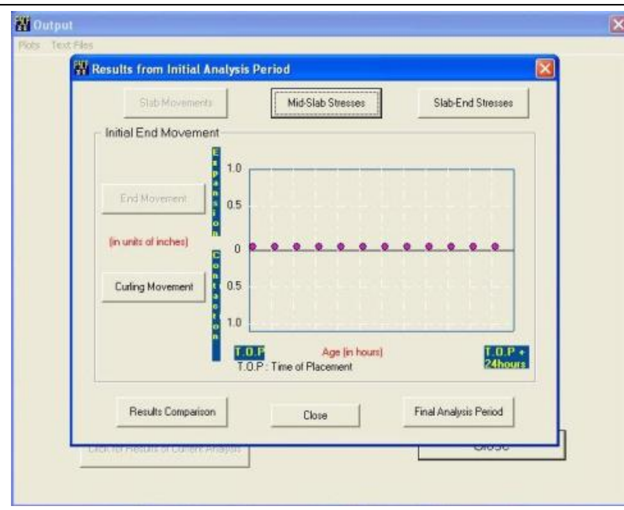


Figure 4.12 Initial slab end movement

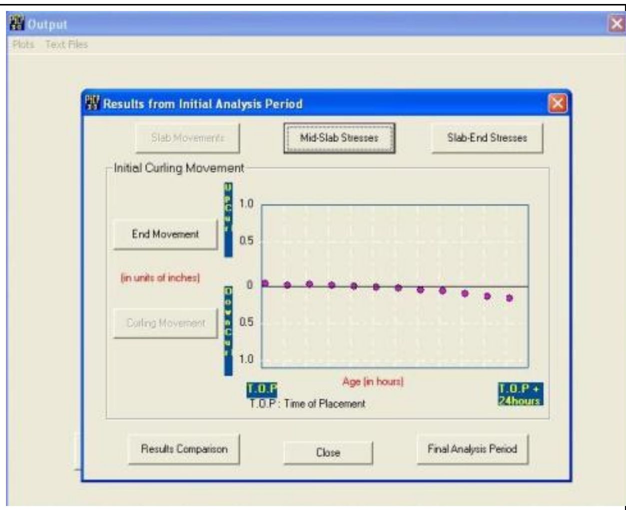


Figure 4.13 Initial curling movements

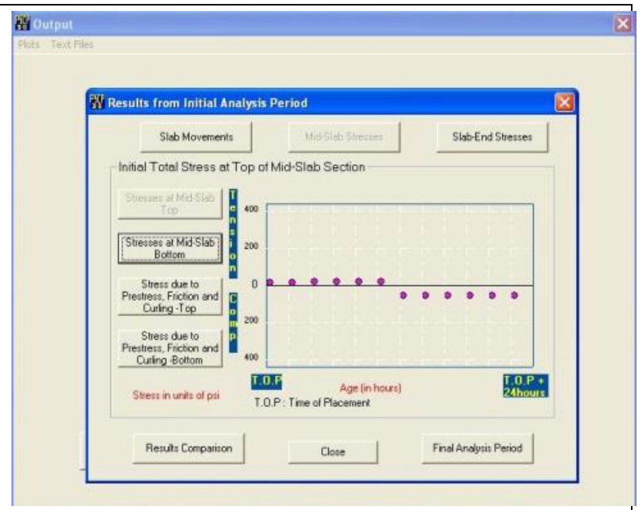


Figure 4.14 Initial stress at mid-slab top

4.3 EFFORTS MADE TO CALIBRATE THE PROGRAM PCP 3.0

In this section, efforts made to calibrate the theoretical model are presented with discussions. The test slab was placed on February 23, 2009 and at the time of the analysis for the calibration of the model; the slab was not old enough to provide meaningful comparisons of field measurements with results from the output for final period in the program. Accordingly, calibration efforts were limited to the comparisons of field testing results with predicted values from PCP 3.0 up to the latest field testing period available at the time of the analysis for calibration.

First, the input data used for the analysis was presented in the Figures 4.1 through 4.10. Table 4.1 summarizes the input values used.

TABLE 4.1 Input values used for calibration of the model

• **Geometry**		
Slab Length	300 ft	
Slab Width	35 ft	
Slab thickness	9 in	
• **Concrete**		
Thermal Coefficient	0.0000052 in/in /^{o}F	
Ultimate Shrinkage Strain	0.0000301 in/in	
Unit Weight	145 pcf	
Poisson's Ratio	0.15	
Creep coefficient	2.10	
• **Aggregates**		
Aggregate Type	Limestone	
Young's Modulus of concrete @ 28 days	5.84 x 10^{6} psi	
• **Concrete compressive strength vs age of concrete**		
Compressive strength of concrete @ 28 days	5,763 psi	
• **Coefficient of friction–displacement relationship**		
Stiffness of slab support	800 psi/in	
Values of coefficient of friction and their corresponding slab movement	Movement	Coefficient of friction
	0.001	0.25
	0.002	0.50
	0.010	0.70

• **Steel properties**												
Strand Spacing	15 in											
Nominal area of strand	0.283 in^2											
Yield Strength	270 ksi											
Elastic modulus	30 x 10^6 psi											
Thermal Coefficient	7 x 10^{-6} psi											
• **Wheel loading**												
Age of concrete when load is applies (days after setting)	28 days											
Wheel load	9000 lbs											
Wheel base Radius	6 in											
• **Temperature data for initial period**												
Setting Time	12 hrs											
Setting Temperature	63 ^oF											
Mid- depth Temperature (^oF)	63.9	64.74	64.77	65.2	66	66.9	67.7	68.5	69.51	71	72.6	74.19
Top- bottom temperature differential	-7	-9.5	2.4	5.8	5.8	3.07	1.99	2.17	4.01	4.3	3.85	2.95
• **Temperature data for subsequent period after setting**												
Time of analysis since setting	28 days											
Mid depth Temperature	68.6	70.8	73.4	75.4	74.6	73	72	71.1	70.3	69.6	69	68.8
Top- bottom temperature differential and the hour of day	5.5	6.2	11.2	6.6	0.7	-2.1	-2.9	-3.4	-3.4	-3.5	-2.7	-1.9
	11:00	13:00	15:00	17:00	19:00	21:00	22:00	23:00	24:00	1:00	02:00	03:00
• **Post tensioning stages**												
Number of post tensioning stages	2											
Time since setting (hrs) and the corresponding prestress	13.44 hrs	35.33 ksi										
	73.44 hrs	123.7 ksi										

4.3.1 COMPARISON OF SLAB DISPLACEMENTS

For the first effort to calibrate PCP 3.0, slab displacements were compared. The actual slab displacements were measured in three different directions (x-, y-, and z-direction) at five locations along the slab edge, as described in the previous chapter. Figure 4.15 shows the instrument installed to measure slab displacements. Figure 4.16 illustrates the slab displacements in the longitudinal direction at 5 different locations along the slab edge. The distance shown in Figure 4.16 is from the center of the slab, which is 150-ft from a transverse armor joint. It shows that the displacements are the minimum at the center of the slab, and the maximum at the arm

or joint. It also illustrates that the displacements are approximately proportional to the distance from the center of the slab.

Figure 4.15 Instruments for slab movements

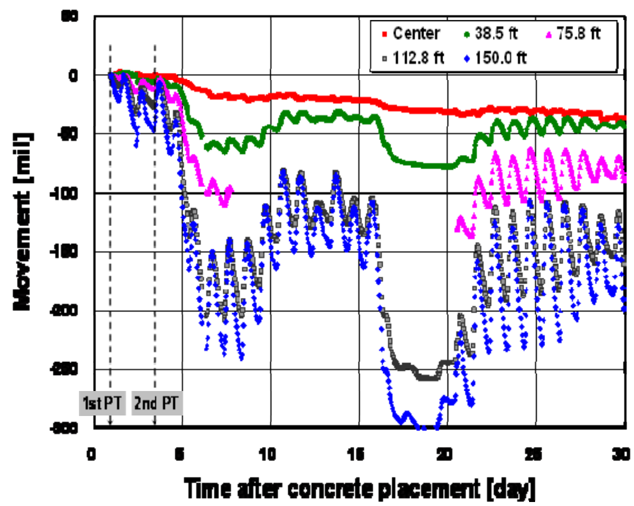


Figure 4.16 Slab displacements in longitudinal direction

The predicted slab movements in the longitudinal direction from the PCP 3.0 for the first 24 hours are presented in Figure 4.17. It is observed that the slab displacements at 2 hours after concrete setting are much larger than the measured ones. Also, it indicates that the slab actually expanding over 24-hr period except when the initial post-tensioning was applied. This expansion of the slab is supposedly due to the increase in concrete temperature for the 24-hr period, which was less than 10 F from the setting temperature. What is noted is the rather large expansion due to the temperature increase of less than 10 F. Also, the contraction of the slab during the initial post-tensioning is about 25 mils from the program. Figure 4.18 illustrates the measured slab displacements at various locations during the initial post-tensioning. At the slab end, or at the armor joint, the contraction was less than 4 mils. There are more than 6 times difference in slab displacements between PCP 3.0 predictions and actual measurements. When there is a substantial difference between values predicted from a theoretical model and actually measured, other efforts should be preceded to investigate the reason for the large difference before any effort to calibrate the model is made. The evaluation of the model for its reasonableness and validity was out of the scope of this implementation project, and no substantial efforts were made in this regard. However, the practice of leaving the bars hammered into the subbase to support transverse post-tensioning bars, as discussed in Chapter 2, could have resulted in this large difference. Also, the use of single polyethylene sheet might have not provided as frictionless a system as considered. The real reason for this discrepancy has not been identified in this study. Further evaluations are recommended to resolve this issue.

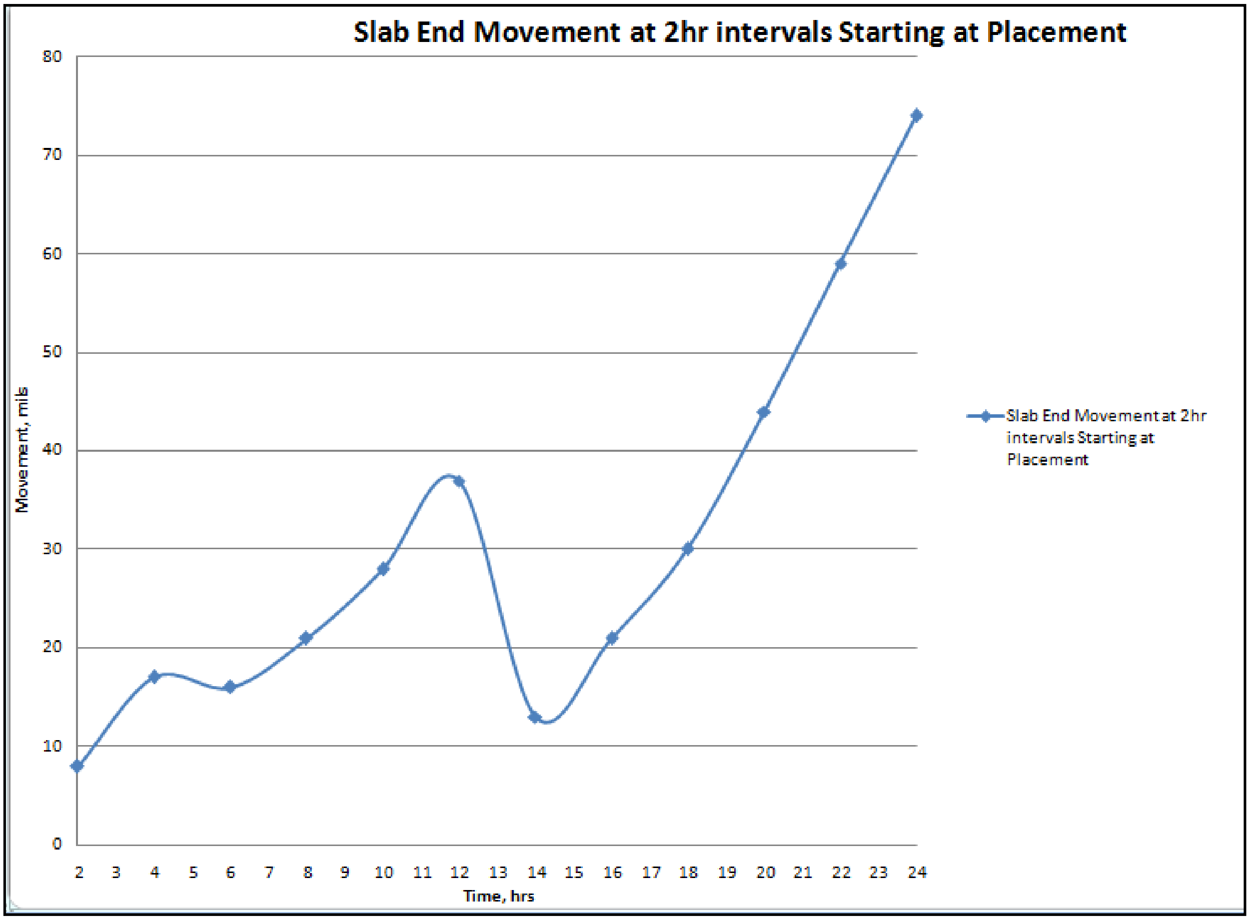


Figure 4.17 Longitudinal slab movements predicted from PCP 3.0

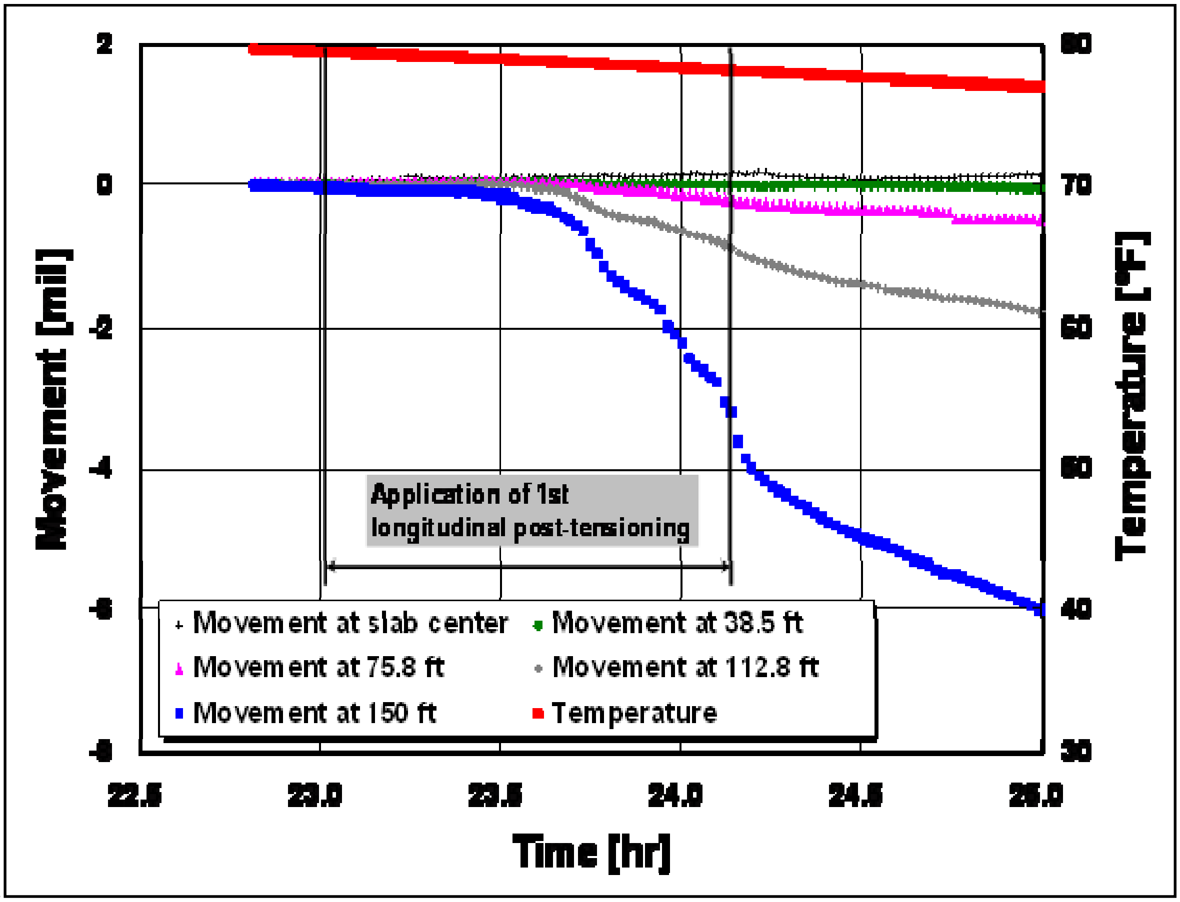


Figure 4.18 Measured slab displacements during post-tensioning

4.3.2 SUMMARY

Efforts were made to calibrate the theoretical model PCP 3.0 using actual slab behavior measured in the field. As a first step, the predicted slab movements from PCP 3.0 were compared with actual measurements. A large discrepancy was observed. When there is such a large difference in slab displacements between predicted and actual values, efforts should be made to identify the cause(s). However, those efforts were out of the scope of this implementation project. Further calibration efforts were not made. It appears that there was a conflict between the need for constructability and what's desirable from a theoretical standpoint. The practice of installing and leaving bars driven into the subbase to support the transverse post-tensioning steel is a good example. Similar practice is currently exercised in the construction of CRCP, and no efforts were made to evaluate the consequences of that practice on CRCP performance. Further research effort could provide answers to the technical validity of the practice.

CHAPTER 5 FINDINGS AND RECOMMENDATIONS

The construction of 9-in thick cast-in-place post-tensioned concrete pavement (PCP) with mostly 300-ft long slabs on IH-35 in Hillsboro started in May 2008. Issues raised during the pre-construction and construction phases were evaluated. Also, the detailed behavior of PCP at early ages due to environmental loading (temperature and moisture variations) and post-tensioning were evaluated with various gages installed. The findings from the field evaluations of the PCP behavior were used to calibrate the theoretical analysis model PCP 3.0.
Based on the efforts in this study, the following conclusions and recommendations are made:

FINDINGS

1. The construction of PCP is quite different from that of other Portland cement concrete (PCC) pavement. The differences can be summarized as follows:
    a. The slab thickness of PCP can be substantially reduced compared with conventional PCC pavement types. This reduction of concrete materials needed will save material cost and reduce carbon dioxide produced during cement production.
    b. Installing reinforcement (transverse and longitudinal tendons) and armor joints were time-consuming and labor intensive.
    c. Installing central stressing pockets required substantial amount of labor.
    d. There was backpressure from concrete on a slip-form paver when it was close to armor joint.
    e. The production rate of PCP was limited due to (1) time required to install reinforcements, and (2) high man-hour needed for post-concrete placement, such as post-tensioning.
    f. At armor joints, grouting hoses were exposed by chipping concrete, which was labor intensive.

2. The issues raised during pre-construction and construction phases are as follows:
    a. Gap slab between PCP slabs: This issue is not unique to the PCP. Rather, the use of gap slabs was necessary due to the site condition. Due to the obstructions in the center portion of the PCP width, longitudinal tendons could not be installed continuously. The 8-ft wide center portion was placed with continuously reinforced concrete pavement (CRCP) with expansion joints where armor joints are located in PCP.

b. Transverse post-tensioning: Due to the multi-phase construction of PCP slabs, transverse post-tensioning had to be continuous at longitudinal construction joints. Initially, tendons were designed for transverse post-tensioning. Due to the use of a slip-form paver, the use of tendons was not feasible. An alternative of using 1-in diameter post-tensioning bar was implemented.
c. Timing for initial post-tensioning: When the concrete was placed at cold temperature, strength gain was slow and 1,000 psi compressive strength was not achieved within 8 hours, which was the latest timing for initial post-tensioning application. Decision was made to reduce the post-tensioning force proportionately to the strength at 8 hours, as long as strength was greater than 600 psi.
d. Tendons being pushed: On the first day of concrete paving, longitudinal tendons and transverse post-tensioning bars were pushed by concrete. #6 bars were placed longitudinally, which were tied to transverse bars and the problem was alleviated.
e. Bars installed to provide rigidity of reinforcement: The installation of longitudinal steel did not completely solve the issue of reinforcement being pushed. Bars were driven into the subbase to support transverse post-tensioning steel.

3. The behavior of PCP slabs at early ages were evaluated in the field by installing various gages, including concrete temperature and moisture sensors, vibrating wire strain gages, linear variable differential transducers (LVDTs), and in-situ coefficient of thermal expansion (CTE) and drying shrinkage sensors. Data were periodically downloaded and analyzed. The findings are as follows:
    a. The compressive stress introduced by longitudinal post-tensioning varied along the slab length, with a maximum near the armor joint and a minimum at the center of the slab.
    b. The concrete strain at mid-depth of the slab under environmental loading was also affected by the subbase friction.
    c. The distribution of longitudinal slab movement was nonlinear along with the distance from the slab center. This is consistent with variations of concrete strain at different locations, which had different degree of restraint. Furthermore, 19% of free slab movement was restrained at the armor joint.

d. The effect of concrete creep and shrinkage was observed. Continued contraction of the concrete slab due to creep and shrinkage will result in the opening of joint width. Creep and shrinkage effects should be included in the design of initial joint width.
e. Temperature distribution along the slab depth was non-linear with the maximum gradient near the surface. RH in the concrete varied little except near the surface. Concrete with a low water-cement ratio and good curing appear to be responsible for the little variations in RH.

4. The efforts made to calibrate the computer program PCP 3.0. There was a substantial difference (more than 6 times) in concrete displacement for initial post-tensioning between actual measurements and predicted values from PCP 3.0. It was postulated that this difference was due to subbase friction or the use of vertical bars to provide stability to the reinforcements.

RECOMMENDATIONS

In this implementation project, the construction of the PCP project was closely monitored and efforts were made to document the unique aspects or issues of the PCP construction. In general, PCP construction was more labor intensive than normal type concrete pavement construction. Since much less cement and aggregates are used in PCP compared with other types of concrete pavement, if material cost becomes quite high, PCP will become cost-effective. It is recommended that TxDOT improve the special provision used in this project incorporating the findings made in this study. Another item of further research is whether leaving vertical bars in place to support reinforcement will have adverse effects on PCP or CRCP performance. This topic may not be large enough to be a topic for an independent research study; however, its impact could be substantial considering how much CRCP TxDOT is constructing and if TxDOT decides to build PCP in the future.

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APPENDIX A

CONCRETE MIX DESIGNS FOR COLD AND WARM WEATHER

A. Mix Proportions for Cold Weather

	Unit	Amount
Cement	lb/yd^3	455
Fly ash	lb/yd^3	114
Water	lb/yd^3	256
Coarse Aggregate	lb/yd^3	1986
Fine Aggregate	lb/yd^3	1086
Air entraining agent	$fl\ oz/yd^3$	9
Water reducer	$fl\ oz/yd^3$	40
w/c ratio	-	0.45
Air content	%	4.5
Slump	in.	1.5

B. Mix Proportions for Warm Weather

	Unit	Amount
Cement	lb/yd^3	419
Fly ash	lb/yd^3	105
Water	lb/yd^3	236
Coarse Aggregate	lb/yd^3	1986
Fine Aggregate	lb/yd^3	1086
Air entraining agent	fl oz/yd^3	9
Water reducer	fl oz/yd^3	40
w/c ratio	-	0.45
Air content	%	4.5
Slump	in.	1.5

Unit Amount