Technical Report Documentation Page

1. Report No.
FHWA/TX-04/0-4035-1

2. Government Accession No.

3. Recipient's Catalog No.

4. Title and Subtitle

DESIGN OF A POST-TENSIONED PRESTRESSED CONCRETE PAVEMENT, CONSTRUCTION GUIDELINES, AND MONITORING PLAN

5. Report Date
August 2003

6. Performing Organization Code

7. Author(s)
Cesar Ivan Medina Chavez, B. Frank McCullough, and David W. Fowler

8. Performing Organization Report No.
0-4035-1

9. Performing Organization Name and Address
Center for Transportation Research
The University of Texas at Austin
3208 Red River, Suite 200
Austin, TX 78705-2650

10. Work Unit No. (TRAIS)

11. Contract or Grant No.
0-4035

12. Sponsoring Agency Name and Address
Texas Department of Transportation
Research and Technology Implementation Office
P.O. Box 5080
Austin, TX 78763-5080

13. Type of Report and Period Covered
Research Report, 2000-2003

14. Sponsoring Agency Code

15. Supplementary Notes
Project conducted in cooperation with the U.S. Department of Transportation, Federal Highway Administration, and the Texas Department of Transportation.

16. Abstract

Prestressed Concrete Pavement (PCP) has been around for almost 60 years. Its application started in Europe in the 1940s, and since then it has been applied with fair success in other countries, including the United States. Domestic application of this technology has been limited for different reasons, mainly due to the lack of well-defined or standard design and construction procedures.

In the United States PCPs have been constructed in Pennsylvania, Mississippi, Arizona, Illinois, and Texas. In 1985 the Center for Transportation Research (CTR) at the University of Texas at Austin designed and constructed a one-mile PCP section that, after more than 17 years of service under heavy traffic loads, is still in very good condition. Although the overall performance of the PCP constructed in Texas has surpassed expectations, there are still design and construction flaws that need to be corrected.

In 1999 the Texas Department of Transportation (TxDOT) funded a research project that required the design and construction of a new, improved PCP section in Texas. The work presented herein attempts to provide a design methodology and preliminary construction guidelines and specifications for a generic PCP. Additionally, the study implements the design procedure for a PCP to be constructed on IH 35 in Hillsboro, Texas. It is believed that this investigation will provide valuable information and a positive step towards the standardization of the PCP application.

The results from the study show that PCP construction is very promising and provides long-term low-maintenance pavements at a competitive life-cycle cost. During the last few years there has been an increased use of this paving technology, and it is hoped that the outstanding performance of several previous projects and lessons learned from the not-so-successful projects will lead to new improvements to PCP methods of design and construction that will produce high-performance pavements.

17. Key Words
Post-tensioned prestressed concrete pavement (PCP), transverse joint, central stressing pocket, tendon, anchor, strand, PCP slab.

18. Distribution Statement
No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of report)
Unclassified

20. Security Classif. (of this page)
Unclassified

21. No. of pages
342

22. Price

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Center for
Transportation Research
The University of Texas at Austin

Design of a Post-Tensioned Prestressed Concrete Pavement, Construction Guidelines, and Monitoring Plan

Cesar Ivan Medina Chavez
B. Frank McCullough
David W. Fowler

CTR Research Report: 0-4035-1
Report Date: August 2003
Research Project: 0-4035
Research Project Title: *Further Development of Post-Tensioned Prestressed Concrete Pavements in Texas*

This research was conducted for the Texas Department of Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration by the Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin.

Center for Transportation Research
The University of Texas at Austin
3208 Red River
Austin, TX 78705

www.utexas.edu/research/ctr

Disclaimers

Engineering Disclaimer

NOT INTENDED FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES

Project Engineer: B. Frank McCullough
Professional Engineer License State and Number: Texas No. 19914
P. E. Designation: Research Supervisor

Acknowledgments

The authors would like to thank the project director, Dr. Moon Won, for his assistance in coordinating this study with TxDOT. Appreciation is also expressed to all the personnel from TxDOT's Waco District Office and the Hillsboro, Texas, Area Office, who helped throughout the field work.

Research performed in cooperation with the Texas Department of Transportation.

Table of Contents

5. DESIGN OF PCP 87

6. MATERIALS SPECIFICATIONS 151

AND CONSTRUCTION GUIDELINES 151

7. MONITORING PLAN 197

List of Tables

List of Figures

1. INTRODUCTION

1.1 Background

Prestressing is defined as the application of a predetermined force to a structural member in such a way that the combination of internal stresses in the member, resulting from this force and any other external loads, will be confined within specific limits (Ref 1). The literature suggests that prestressing principles were first introduced in 1888, but it was not until 1910 that they were better understood and patents originated. The French engineer Eugene Freyssinet is regarded as the first investigator to recognize the effect of creep in concrete and to realize that high-quality concrete and high-tensile-strength steel are necessary for adequate prestress retention.

With regard to prestressed concrete pavement (PCP) applications, it was not until the early 1940s that the technology was first used. This technology was put into practice until engineers' ingenuity provided some criteria to make use of the high compressive strength of concrete. Prestressing a concrete slab involves the application of compressive loads prior to the application of service loads or traffic loads, and by doing that, tensile stresses are significantly reduced or even eliminated. This report describes the recommended steps to be undertaken for the design and construction of a generic post-tensioned PCP. Additionally, these steps are implemented in the development of the design, construction, and monitoring plan for a PCP pavement that will be constructed on IH 35 near Hillsboro, Texas.

1.2 Objectives

The primary objective of this research study can be stated as follows:

Develop and recommend improved design and construction techniques for a cost-effective state-of-the-art pavement structure and apply those techniques in the design of a new PCP section in Texas.

To achieve this objective, an extensive literature research has been conducted to evaluate previous PCP-related work and learn from successes and failures of various projects within the United States and abroad. As a result, new applicable developments have been produced pursuing the following sub-objectives:

1. Develop a mechanistic-empirical analysis procedure for a PCP that can be used to prepare designs, specifications, and construction guidelines for a specific project.
2. Calibrate and validate the procedure.
3. Implement the procedure by developing the design, materials specifications, and construction guidelines for a new PCP project.

Additionally, it is expected that with the results obtained from the proposed PCP developments, pavement design engineers, state agencies, and contractors will further explore this paving option that will enrich the application of rigid pavements in the medium and long terms. When the application of PCPs becomes more frequent, this paving technique will be identified for being highly competitive with other conventional concrete pavements like continuously reinforced concrete pavement (CRCP) and jointed concrete pavement (JCP).

1.3 Scope

The tasks related to the research of PCPs are endless; therefore, this study will focus on the development of a mechanistic-empirical design procedure for PCP and will refine previous design and construction methods and specifications. Because the extension of the PCP section to be designed and constructed in Hillsboro, Texas, is sufficiently long, an adequate comparison of the cost-effectiveness of PCP applications with that of alternative methods can be drawn. A comprehensive monitoring plan for the short and long terms of the PCP section will be proposed using state-of-the-art equipment. Likewise, a comparison between the designed PCP and a control CRCP section regarding performance and economics will be presented. Because the PCP has not been constructed yet, the performance prediction will be validated in the future, after the PCP section is finished and used.

1.4 Methodology

This study follows an established methodology to achieve the previously described objective and sub-objectives. This methodology is graphically explained by the flow diagram displayed in Figure 1.1 and can be divided in two main components. Part I is called “Evaluation of Previous Work” and relates to the evaluation of work that was conducted for various projects built in the past, giving special attention to domestic experiences. Because improvements will be proposed for a case study in Texas, the work conducted in the past in Texas is evaluated at a deeper level. Part II in Figure 1.1 is called “New Developments” and refers to the design of a PCP section in Hillsboro and the tasks that will be developed later. Those tasks include the recommendation of materials specifications, construction guidelines, monitoring plan, comparison of PCP and CRCP pavements, discussion of new developments, and conclusions and recommendations.

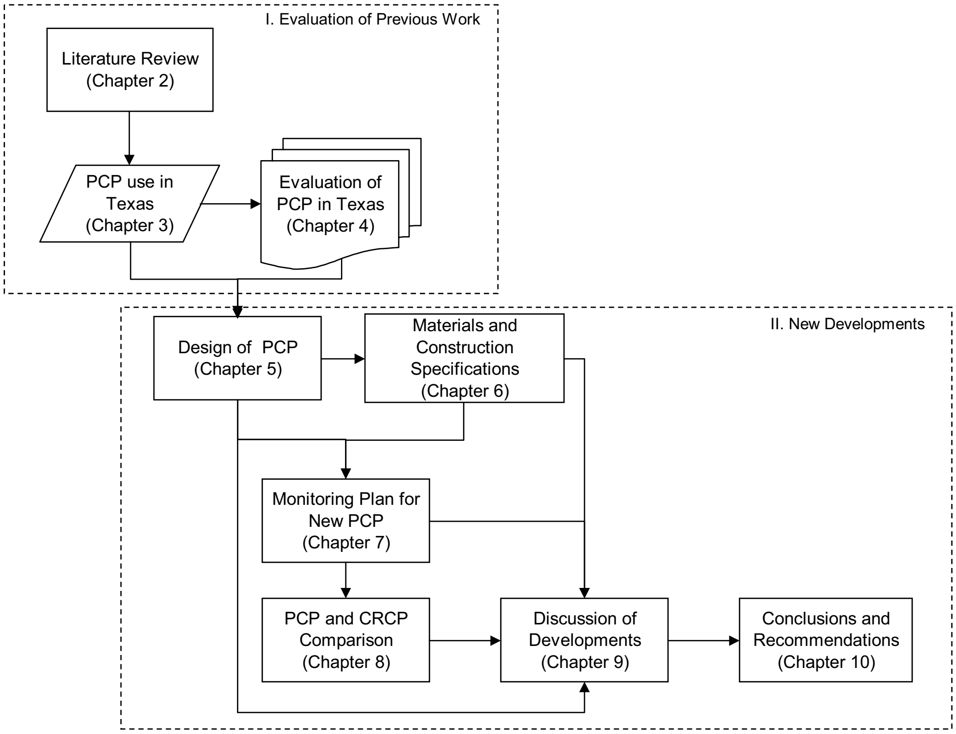


Figure 1.1 Flow diagram of the methodology process

1.4.1 EVALUATION OF PREVIOUS WORK

Chapter 2 presents the findings from the literature review and describes PCP experiences in Europe, Japan, and the United States. Domestic applications are described in more detail because more information was found for these applications. The characteristics of PCP sections constructed in Pennsylvania, Mississippi, and Arizona and their performance is summarized. Finally, improved PCP applications during the 1980s and 1990s in Illinois and Texas are briefly mentioned.

Chapter 3 focuses on the description of the application of PCP technology in Texas. A detailed description of the location of the PCP in McLennan County and its cross-section history starting with the first pavement structure built in 1952 up to the construction of the PCP is given. Additionally, the concepts developed during the design and construction of this project in the mid-1980s are summarized. Developments include introduction of transverse prestress, development of transverse joint, and conception of central stressing pockets.

Chapter 4 contains an extensive evaluation of the performance of the PCP section built in Texas in the mid-1980s near West, Texas, in McLennan County. The evaluation presents the results from condition surveys and pavement evaluations conducted from the 1980s to the present. Measured horizontal and vertical movements caused by temperature gradient effects on the PCP slab are also presented. Likewise, an evaluation of the load-deflection characteristics of the PCP slabs is presented. A statistical analysis of the deflections at the transverse joints is conducted, and their load transfer efficiency is evaluated. Finally, the elastic properties of the pavement structure are back-calculated.

1.4.2 NEW DEVELOPMENTS

Chapter 5 describes the theory behind the design of a PCP. A series of steps that might be followed for designing a generic PCP are mentioned and then implemented for the design of the new PCP in Hillsboro. The design process starts with the evaluation of the existing pavement surface from condition surveys and continues with the application of a quantitative analysis of the pavement distresses using a pavement distress index (PDI) algorithm derived by Chia-Pei-Chou (Ref 2). Afterwards, a summary of the evaluation of the structural capacity of the existing pavement structure is given. Next, using projected traffic data, the design of the new PCP structure is carried out using computer spreadsheets to facilitate the process. Finally, the most suitable PCP characteristics are recommended based on the analysis of the design.

Chapter 6 compiles a set of material specifications and construction guidelines for the PCP in Hillsboro. Recommendations are provided for the treatment of foundation soils and soils forming the embankments of the PCP structure. Materials specifications are proposed, and construction guidelines (including tasks like placing of the polyethylene sheet between the sub-base and the PCP slab, and tendon placement) are provided.

Chapter 7 presents a monitoring plan to be conducted at construction time of the PCP and continuing for the long-term. Activities that will ensure the quality of the pavement are described, and a material sampling program is recommended. Additionally, measurements of the deflection of the pavement using a falling weight deflectometer (FWD) and a rolling dynamic deflectometer equipment (RDD) are scheduled. Instrumentation tasks that will collect data, such as curling and horizontal movements of the ends of slabs, are described. Finally, it is recommended that concrete temperature and moisture are evaluated at scheduled intervals of time using state-of-the-art devices like i-Buttons, installed at predefined points in the PCP.

Chapter 8 compares the designed PCP to an equivalent continuously reinforced concrete pavement (CRCP). A CRCP constructed just south of the PCP will serve as a control section for the PCP. The control CRCP consists of a 5-mile-long 14-inch-thick pavement with double mat reinforcement steel for which condition surveys and deflection measurements have been taken every year since construction started in November 1999. The comparison of the two pavements is conducted by looking at two aspects: performance and economics. Additionally, a life-cycle cost analysis was conducted for the two pavements using some preliminary information about the costs for the PCP and current costs for the CRCP.

Chapter 9 describes the developments that resulted from this research study. The developments are described for three different categories: design, construction, and monitoring. Additionally, the most important practical considerations regarding the construction that shall be conducted are described. Finally, recommendations are made to improve the practices of the previous in-home experience (old PCP in McLennan County constructed in the mid-1980s).

Finally, Chapter 10 contains the conclusions and recommendations related to the development of the PCP in Hillsboro, Texas. Reference is made to the main objective and sub-objectives of this research study. The significance of the application of the PCP technology to a larger scale and its impact on the research and promotion of innovative pavement construction practices are presented and the potential of the PCP technology is discussed.

2. LITERATURE REVIEW

2.1 Background

The initial application of PCP goes back to the 1940s, when this technique was used in some countries in Europe. The literature suggests that the first application of a PCP was done in England in 1943. The first real field application was conducted in France at Orly International Airport, where Eugene Freyssinet pioneered the use of PCP. After its successful application in France, other countries including Austria, Belgium, England, Germany, The Netherlands, Algeria, and New Zealand started constructing pavements using this method.

Highway applications of PCP were conducted in France between 1945 and 1949, followed by some other projects in England in the early 1950s. All these projects used different prestressing applications. Later, other projects were built in Italy, Japan, and Switzerland.

2.2 Applications in Europe and Japan

Most of the pavement applications that use the PCP technology have been constructed in Europe. However, there is no record showing construction of PCP in the last ten years. In Japan few pavement sections have been constructed using precast slabs, although research studies have been conducted to analyze the possibility of widely implementing this technology. The literature suggests that at least three precast pavement sections have been built for research purposes only.

2.2.1 EXPERIENCE IN EUROPEAN COUNTRIES

During the1960s several PCPs were constructed in Europe, and they continue to be constructed today. Most of the early PCPs were built for runways and taxiways of both civilian and military airports and showed excellent performance, but with minor flaws in almost all the cases. Most of the prestressed airport pavements have been post-tensioned longitudinally and transversely with patented systems that use high-strength bars in grouted steel conduits.

For civilian airports 7 in. thick pavements have been used extensively. The projects include 830,000 yd^2 at Schiphol, Amsterdam's main airport, constructed from 1965 to 1978; and 660,000 yd^2 at Cologne-Bonn Airport, constructed from 1960 to 1968. For military airports, over one million yd^2 of 5.5 in. thick pavements, 600,000 yd^2 of 6.3 in. thick runways and taxiways, and 200,000 of 4 ¾ in. thick pavements have been built (Ref 3).

2.2.1.1 External Prestressing Methods

In general, two prestress methods were used in Europe, external prestress and internal prestress. In the external prestress method jacks or wedges are used to compress straight runways without longitudinal prestress tendons against abutments at the ends. This method was developed in France after 1945, and the first pavement built using this technique was constructed at Orly Airport in Paris, France, in 1946. This pavement was 1,378 ft long (420 m), 197 ft wide (60 m), and 7 in. thick (18 cm) (Ref 4). The slabs of the pavement were built in a triangular shape with transverse joints aligned at 45° with respect to the longitudinal axis of the runway, and both ends were confined by fixed abutments. The slab was prestressed in transverse direction, while the steel-lined joints could move against each other. Figure 2.1 shows a layout of the pavement section. Some difficulties experienced with this paving project included the placement of triangular-shaped slabs that the available paving machines could not place easily. Additionally, the tendons started to corrode in the joints and failed after some time.

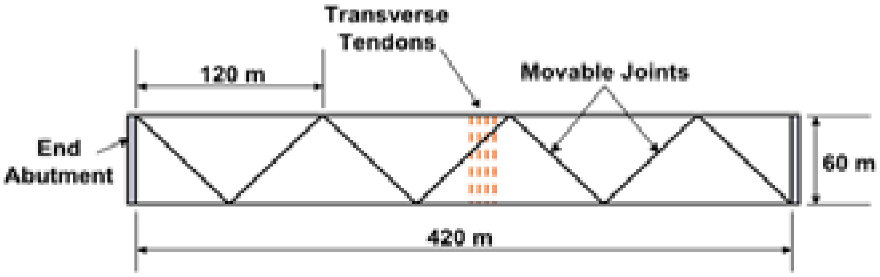


Figure 2.1 PCP test section built in 1946 at Orly Airport in Paris, France

Other variants of external prestress were used in Central Europe for a number of prestressed slabs, especially for road pavements. A very common setup was the one used in Algeria at the Maison Blanche Airport, where elastic end abutments were installed below the prestressed concrete slab for a given length and then loaded with the weight of the backfill and the slab. Figure 2.2 displays this type of setup of prestressed slabs with elastic abutments. Active joints were installed every 984 ft (300 m) with Freyssinet flat jacks that were left in the joints and reloaded from time to time after the runway had been completed. Because of this configuration, the concrete slab worked always in compression longitudinally. This type of construction was found to be effective in some areas with a fairly balanced climate, but inappropriate for areas with high ambient temperatures. Rising temperatures significantly increase the compressive stresses in the slab, and the abutments have to take a considerable amount of the force. Thus, large repetitive temperature changes are a disadvantage for this type of structure and can cause damage.

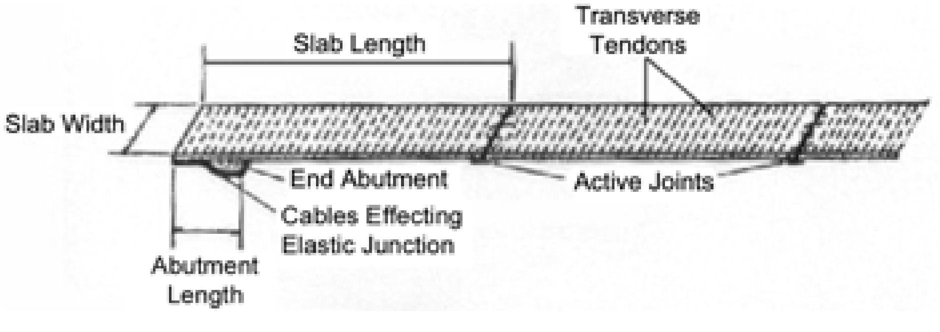


Figure 2.2 Schematic of a prestressed slab with elastic end abutments

Another example of externally prestressed pavements is a runway built at Brussels National Airport, designed by Professor Paduart in 1960. The runway was 10,827 ft long (3,300 m), 148 ft wide (45 m) and 7 in. thick (18 cm). It was compressed longitudinally against fixed abutments using flat jacks and then post-tensioned in the transverse direction. Every 1,083 ft (330 m) there was an active joint located above an access tunnel that served to operate flat jacks at the joints. Figure 2.3 shows a cross-section of the pavement through an active joint. The slab thickness was increased by 2.8 in. (7 cm) in the area of the joint, and dowel bars anchored the slab to the tunnel base to prevent buckling.

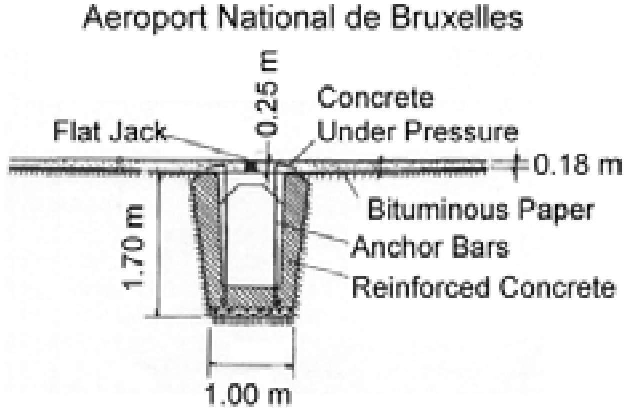


Figure 2.3 Cross-section through active joint used with the post-tensioned runway in Brussels, Belgium

Between the active joints, temporary joints were located at every third of the length of the slab to operate additional flat jacks during the first weeks after construction. Then the jacks were removed, and the temporary joints were filled with concrete.

The pavement section experienced the same symptoms as previous cases had, where concrete slabs were compressed longitudinally between abutments by means of jacks. The slabs crept under the compressive stresses, lost the necessary compression at low temperatures, and showed transverse cracks. On the other hand, large rapid increases in temperature caused longitudinal compressive stresses, leading to buckling of the slab and damage to the concrete in the area of the active joints.

2.2.1.2 Internal Prestressing Methods

The methods using internal prestress include two variants, as follows:

a) The longitudinal tendons are stressed in a stressing bed, and then the concrete is poured in longitudinal strips. The transverse tendons are placed in sheathing ducts and stressed after the concrete has hardened.
b) Sheathed longitudinal and transverse tendons are stressed and grouted after the concrete has gained sufficient strength.

In the first case, longitudinal tendons are placed without a sheathing duct, and abutments are spaced at very great distances (around 2,300 ft) and longitudinal tendons between them are prestressed. Transverse tendons are placed with sheathing ducts, and then the concrete is poured in strips 24.6 ft (7.50 m) wide. After the concrete has set, it behaves as any regular reinforced slab; transverse cracks occur when the temperature drops. The prestress load in the tendons is transferred to the concrete by a bond which becomes fully effective with the increasing strength of the concrete. A too-early transfer will cause slippage. Thus, only after the concrete of the area between the abutments has hardened sufficiently can the tendons be released to transfer the prestress load to the concrete. This is the step in which problems occurred when using this method. When tendons are released too late, transverse cracks may already have formed. On the other hand, if they are released too early, slippage occurs, and the prestress force is not effective in those slab areas. In the beginning failures were corrected, and the method was improved by providing additional strip footings with steel profiles every 328 ft (100 m) between abutments where the tendons were provided with steel straps to increase the bond and to anchor them in the steel profiles.

In the second case for internal prestress, tendons are used with sheathing ducts in both longitudinal and transverse directions, and then they are stressed against the concrete after it has hardened. In all the methods described previously, concrete slabs were not bonded to their substructure so that concrete could move freely in both directions, and temperature changes only caused stresses resulting from the minimal friction between slab and bedding.

2.2.1.3 Construction Considerations

Accuracy in leveling the substructure of the PCP slab has always been considered to be critical, and in most cases that objective was achieved by constructing a smooth bituminous layer that acted as a curing membrane sprayed onto a cement-treated base. Then an antifriction layer of highway paper or plastic foil was placed on top of the bituminous layer, and longitudinal and transverse tendons were placed on top with their sheathing ducts installed. Tendon cross points were supported at as short intervals as possible to avoid friction losses from duct deflections.

Slab thicknesses of 7 in. (18 cm) proved to be adequate for most commercial airports; military airports used 6.3 in. thick (16 cm) slabs and performed fine without damages for decades. For lighter aircraft 5.5 in. thick slabs were built in many instances and, in cases of overlay slabs, even 4.7 in. thick (12 cm) slabs were placed. Regarding bonded or unbonded tendons, tests at the Technical University of Munich indicated clearly that bonded tendons provide a higher stiffness and better load distribution than unbonded tendons do.

When placing the slab, the paving operation moved at a speed of at least 100ft/hr (per strip wide), and the concrete was vibrated at the same time. When concrete reached a compressive strength of 2,134 psi (150 kg/cm^2) after nearly 20 hours, a first partial post-tension was applied to avoid appearance of cracks due to temperature drop overnight. It was found that the temperature drop in the early morning hours of the first night represented a danger for the slowly hardening concrete and that post-tension would alleviate that. After 2 or 3 days, the prestress force was increased to 80 percent of its final value. Once all strips of pavement were placed and sufficient strength was gained, the full prestress load was applied, and at the same time transverse tendons were post-tensioned in one operation, and then all tendons were grouted.

2.2.1.4 Summary of European Experience

Several PCP sections were built in various countries in Western Europe between 1946 and 1960. Table 2.1 shows a summary of pavement sections constructed during those years. Most of the sections were built for testing and experimental purposes. Generally, some difficulties were encountered when using external prestressing methods with abutments under the road surface. The method was applicable only for straight sections; for horizontal curves and for convex and concave curves between gradients, another system had to be used. This same inconvenience was found for the first internal stressing method, where the longitudinal tendons are stressed in a stressing bed, and then the concrete is poured in longitudinal strips. In this case, only straight pavement sections were used. Only the second internal method discussed, in which sheathed longitudinal and transverse tendons are stressed and grouted after the concrete has gained sufficient strength, allowed any geometric road design without technical difficulties. In general, the use of PCP technology in Europe has resulted in a very positive and successful product, despite the difficulties and failures encountered, which according to the literature have been overcome.

Table 2.1 Summary of PCPs built in Western Europe from 1946 to 1960

Location	Function/ Application	Years of Construction	Total Area (yd^2)	Thickness (in.)	Prestressing Method
Germany: Mergelstetten, Speyer, Wofsburg, Dietersheim, Koeln-Wahn	Federal and country highways, runways, and taxiways	1953 to 1960	1,358,226	5½ to 8	Tendons
France: Luzancy, Esbly, Servas, Orly, Maison Blanche, Fontenay-Tresigny	Federal highways, runways, 1 taxiway, 1 apron	1946 to 1961	368,790	4¾ to 7	Diagonal tendons, external prestressing with jacks, tendons
Belgium: Brussels and Zwartberg	Highways and 1 runway	1947 to 1960	216,546	3 to 7	Tendons, external prestressing with jacks, external prestressing with elastic joints
Austria: Salzburg, Vienna, Wien-Schwechat	Highways, city road, 1 runway, 1 taxiway	1954 to 1959	149,439	6 to 8	External prestressing with wedges, external prestressing with bed and jacks
Netherlands: Amsterdam-Schiphol and Leiden	Highways and 1 taxiway	1950 to 1960	31,335	4¾ to 6	Tendons and prestressing bed
Switzerland: Wildegg, Naz, Moeriken, Neuchatel, Boudry	Country roads and 1 city street	1955 to 1960	31,455	4¾ to 6	External prestressing with wedges, External prestressing with jacks, External prestressing with elastic joints
England: Crawley, London, Buckinghamshire, Essex, Woolwich, Port Talbot, South Walls, and Gatwick	Roads and aprons	1950 to 1957	21,205	5 to 6½	Tendons and diagonal tendons

2.2.2 JAPANESE EXPERIENCE

Japanese studies in this field classify PCPs in three different categories, based on the probability of bending cracking occurring when concrete is under working-load conditions. This classification is as follows:

1. Class I: PCP that has a very small probability of cracking (full prestress applied)
2. Class II: PCP that has relatively small probability of cracking (partial prestress applied)
3. Class III: PCP in which cracking is allowed under working conditions with certain limits

According to the literature, studies have been conducted on the applicability of Class III PCPs, in which cracking at the bottom of the slab is permitted within certain limits. Additionally, development of a precast prestressed concrete pavement with a joint system called horn-joint has been introduced. However, this second type of pavement is out of the scope of this research study; another project conducted by Merritt et al. (Ref 6) focuses on the application of this paving technology. The present study focuses on studies conducted in Japan related to Class III PCPs; no literature was found for Classes I and II. Classes I and II differ from Class III only in the amount of steel bars used for prestressing; Class III requires considerably less steel than the other two classes and is therefore more economical. According to the results of theoretical analysis and a series of full-scale loading tests performed by Japanese pavement engineers on Class III PCPs, this paving technique can be used for highway and airport pavements, although it is better suited for heavy aircraft.

2.2.2.1 Analysis of Class III PCP

The theoretical analysis of this pavement was conducted using a model consisting of ring elements supported on a Winkler foundation, as shown in Figure 2.4.

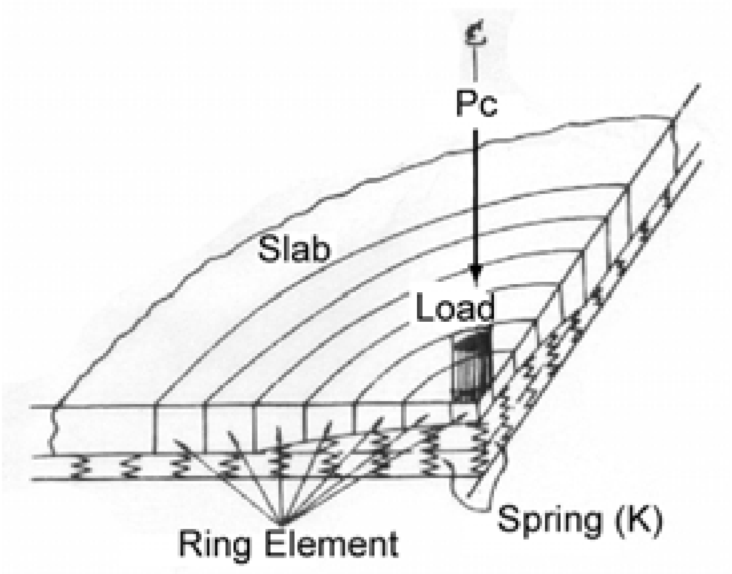


Figure 2.4 Model used in the analysis of the Class III PCP slab

In this model, as load P_c is increased and cracking occurs at the bottom of the ring elements, the effective thickness of each element decreases proportionally with the depth of the cracks. By conducting a mathematical analysis with a computer program of this model the following results were found:

1. The positive bending moment at the loading point decreases slightly after cracking.
2. The negative bending moment, related to surface cracking, increases.
3. By increasing the load, the position of the maximum negative bending moment comes closer to the loading point.

From these results, the design of the Class III PCP can be conducted as follows:

A value of the bending moment is assumed to check the section where cracks occur at the bottom of the slab. Next, a redistribution of the moment is performed, and an iteration process continues. When the bending moment M is greater on a section than the cracking moment M_c, the stresses of prestressing tendons and compressive stresses at the top of the concrete are computed assuming the distribution of stresses and strains shown in Figure 2.5, Hooke's Law, and equilibrium conditions of axial forces and moments.

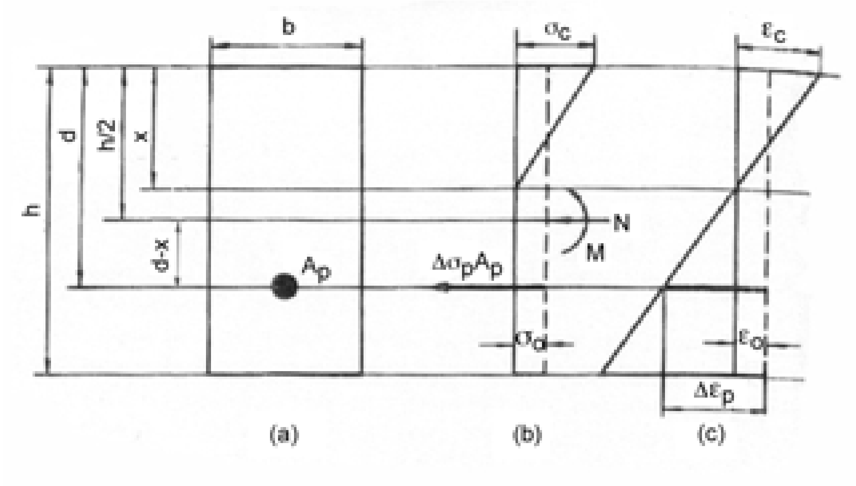


Figure 2.5 Assumed distribution of stresses and strains on cracked section

Using Hooke's Law, the following equations are obtained:

$$\frac{\sigma_c}{\frac{\Delta\sigma_p}{n}-\sigma_o} = \frac{x}{d-x} \tag{2.1}$$

$$N = \frac{\sigma_c bx}{2} - \Delta\sigma_p A_p = \sigma_o bh \tag{2.2}$$

$$M = \frac{\sigma_c bx}{2}(\frac{h}{2}-\frac{x}{3}) + \Delta\sigma_p A_p (d-\frac{h}{2}) \tag{2.3}$$

where

N = axial force of PCP slab (prestressing load)

M = bending moment

ε_o, σ_o = strain and stress caused by N

ε_c, σ_c = strain and stress at the compressive fiber of the concrete

$\Delta\varepsilon_p$, $\Delta\sigma_p$ = increased strain and stress of prestressing tendon

h = slab thickness

d = distance from upper fiber to prestressing tendon

x = distance from upper fiber to neutral axis

b = width of the section

A_p = area of prestressing tendon

n = modular ratio (E_s/E_c)

Having solved Equations 2.1, 2.2, and 2.3, one can obtain the position of the neutral axis, the increased stress of prestressing tendons, and the compressive stress of concrete using the following equations:

$$A_1x^3 + A_2x^2 + A_3x + A_4 = 0 \tag{2.4}$$

$$\Delta\sigma_p = \frac{x^2 - 2hx + 2hd}{bx^2 + 2nxA_p - 2ndA_p} nb\sigma_o \tag{2.5}$$

$$\sigma_c = x(\sigma_o - \frac{\Delta\sigma_p}{n})/(x - d) \tag{2.6}$$

where

$$\begin{aligned} A_1 &= -2\,\sigma_o\,b(bh+nA_p) \\ A_2 &= 3\,b\,\{\sigma_o(bh^2+2nd\,A_p)-2M\} \qquad (2.7) \\ A_3 &= 6\,n\,A_p\,\{\sigma_o bh(h-2d)-2M\} \\ A_4 &= -d\,A_3 \end{aligned}$$

Finally, the calculation of the load P_c, which causes surface cracking, is calculated from an analysis of the model shown in Figure 2.4 or using Equation 2.8 proposed by Meyerhof (Ref 3):

$$P_c = \{\frac{4\pi}{1-4a/(3b')} + \frac{1.8(S+S_T)}{\ell - a/2}\} * (M_r + M_r') \qquad (2.8)$$

where

S = dual wheel spacing

S_T = tandem spacing

a = contact radius of load

ℓ = radius of relative stiffness of slab

$$\ell = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)K_{75}}}$$

b′ = 3.9 ℓ

Mr = positive moment of resistance of slab

Mr′ = negative moment of resistance of slab

Mr (= Mr′) can be obtained by the sum of the flexural strength and prestress and then multiplying by the section modulus

2.2.2.2 Construction and Testing of the Class III PCP Test Section

For the construction of the PCP test section, three different types of base courses were used to investigate the influence of the K_{75} value on the performance of the slab. The slabs were designed using the theory previously explained for Class III PCP. Instrumentation of the concrete slab included installation of settlement, pressure, and strain gauges, joint meters, and thermometers. Strain of tendons was measured with some auxiliary steel bars cast near them and tensile values were measured and recorded.

The testing of the PCP section was carried out using a loading cart that simulated a similar wheel arrangement to that of a DC-8-63 aircraft. Figure 2.6 displays a picture of the loading cart on the test section. Static loading tests were made after a pre-established number of gear repetitions of 10, 500; 1,000; 3,000; 5,000; 7,000; and 10,000 were applied on the test section. During the testing, it could be seen that initially, as the load was increased, the strains also increased following a linear pattern, but after the load reached a certain value, the strains of both concrete and tendons increased rapidly out of proportion to the load. Curves were prepared plotting strain versus load for both concrete and tendons. The location where the linear trend of the curves showed the first inflection point was regarded as the minimum load that caused cracking at the bottom of the slab.

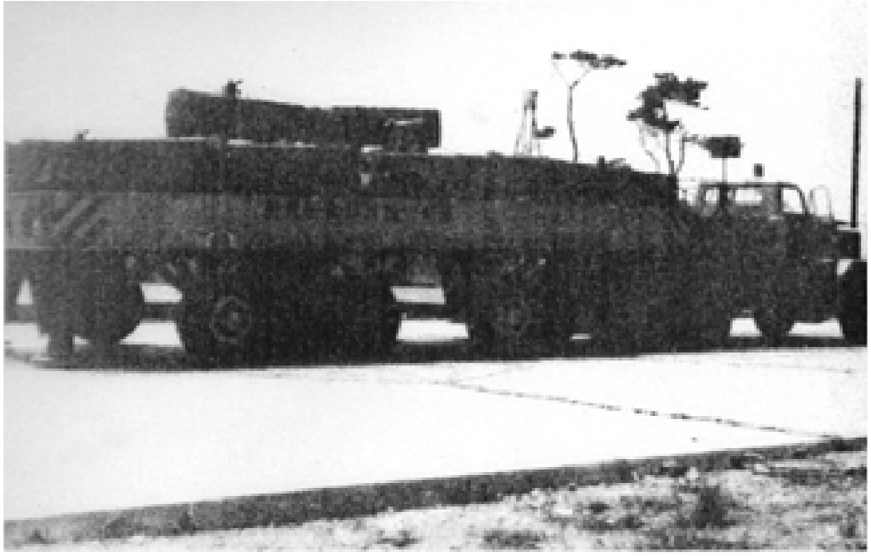


Figure 2.6 Class III PCP test section and loading cart

2.2.2.3 Summary of Japanese Experience

Following the analysis of the theory, construction, and testing of the Class III PCP test section, the proposed methodology cannot yet be considered sufficiently valid for its applicability to design PCP pavements, although the design of the section was done using the theory and formulations previously explained and the section performed well. Even though the test section did not show any problems in either structural or functional aspects, it is important to investigate the performance of Class III PCP when subjected to repetitive loads and various climatic conditions. If further research is conducted in this field with similar positive results, it is assumed that Class III PCP can be satisfactorily recommended for the construction of pavements.

2.3 Initial Domestic Experience (1970s)

After some years of successful application in Europe, the first PCP in the United States was built for an airfield in a military airport. In 1953 the United States Navy Bureau of Yards and Docks constructed an experimental 7 in. thick PCP at Patuxent River Naval Air Station, Maryland. Following that experience, two more PCPs were built in Texas. One PCP was constructed in 1955 at the San Antonio International Airport, the only nonmilitary application of PCP in the 1950s. The other pavement was constructed in 1959 in El Paso at Biggs Airforce Base (AFB). Until 1960 only two-thirds of a mile of PCP for runway applications had been constructed in the United States. It was not until 1980 that one additional runway was built at Chicago O'Hare International Airport.

The first experimental highway PCP project in the United States was built in Pittsburgh, Pennsylvania. However, the first operational highway in the United States was not constructed until 1971 near Milford, Delaware, and was only 300 ft long. In that same year, the Federal Highway Administration (FHWA) proposed the construction of a 3,200 ft long, 24 ft wide, and 6 in. thick PCP as part of the permanent airport access road network at Dulles International Airport in Virginia. This project was built using funding from the FHWA Research and Development Demonstration Projects Program to demonstrate that PCP construction was practical and economically competitive with other types of pavements.

The two main objectives of this project were, first, to explore and demonstrate the practicality and economy of using existing prestressing techniques in the construction of large-scale highway projects and, secondly, to investigate the behavior of long monolithic concrete pavement slabs with regard to movements and length changes, curling and warping at the ends, and frictional and flexural restraint stresses away from the slabs ends. Other goals of the project were to improve design criteria, determine maintenance requirements, obtain construction data, stimulate state interest, and provide information for contractors. The project was built, but the main objectives were not fully accomplished due to the inexperience with this type of construction. Unfortunately, the pavement had considerable problems: expansion joints failed, and the concrete adjacent to the joints started to spall. Furthermore, the I-beams forming the joints and the attached reinforcement were torn away from the concrete. This problem may have been due to rusting and freezing of dowels. The failures were soon repaired, and problematic joints and hardware were replaced. To overcome the problems experienced with the PCP built at Dulles International Airport, the FHWA decided to manage three additional projects built by state agencies in Pennsylvania, Mississippi, and Arizona.

The experience obtained from the projects previously described was enriched by the design and construction of three additional full-scale highway projects in different geographic zones of the United States representing various climatic zones and traffic conditions. As with the Dulles project, these projects were built under the supervision of the FHWA Demonstration Projects Program. The PCPs Pennsylvania, Mississippi, and Arizona were built with similar characteristics and comparable construction methods. All the pavements were 6 in. thick and prestress was applied only in longitudinal direction. Table 2.2 summarizes some characteristics of the three projects.

Table 2.2 Full scale PCP projects built in the United States

Project Characteristics	**Project Location**		
	Harrisburg, Pennsylvania	**Brookhaven, Mississippi**	**Tempe, Arizona**
Project Length (mi.)	1.5	2.5	1.2
Joint Spacing (ft)	600	450	400
Year Built	1973	1976	1977
Slab Thickness (in.)	6	6	6
Sub-base Thickness and Characteristics	6-in. asphalt base course	4-in. hot mix concrete	4-in. lean concrete
Post-Tensioning Application	Longitudinal direction only	Longitudinal direction only	Longitudinal direction only

2.4 Evaluation of Initial Full-Scale Projects

The PCPs built during the period from 1973 to 1977 and summarized in Table 2.2, as well as all the international projects, served as the basis for another PCP section that was built in Texas in 1985, again under the supervision of the FHWA. The following paragraphs discuss characteristics and developments achieved in the first three full-scale projects. Chapters 3 and 4 focus on discussions of the developments and evaluation of the project built in Texas. There is extensive literature describing in full detail each of the PCP projects built in Pennsylvania, Mississippi, and Arizona, and so, this section only summarizes interesting information about each project. The reader is referred to the cited literature for further information.

2.4.1 EVALUATION OF PCP SECTION IN PENNSYLVANIA

The PCP section in Harrisburg, Pennsylvania, was built in November 1973 and consists of a four-lane divided highway formed by 23 prestressed slabs 600 ft long in average, 24 ft wide, and 6 in. thick (Ref 7). The project also included a center lane for left turns that was also prestressed. Gap slabs 3 ft long, as the one show in Figure 2.7, were used.

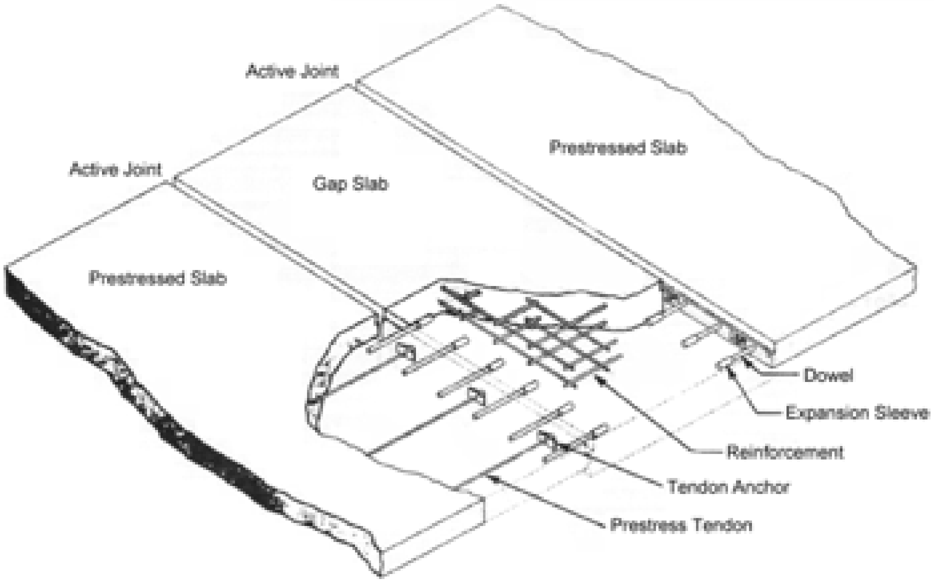


Figure 2.7 Gap slab used in the PCP section in Pennsylvania (1973)

Post-tensioning was extended into the gap slab, and terminal joints with a sleeper slab (slab placed beneath joints and ends of adjoining slabs to provide support for slab ends and joint hardware) were used at each slab end and at approaches to structures. This project used a customized steel tongue and groove assembly that facilitated the post-tensioning of the gap slab.

This pavement section has been in service for 29 years, and it is starting to be a maintenance concern. Therefore, it is expected that the pavement will be rehabilitated in the next few years. Among the failures of the pavement, minor transverse cracks developed in some of the slabs at early age, and after almost three years following construction, an intermediate narrow transverse crack was observed in 2 of the 23 PCP slabs. Cracks did not spall and were reported to be tight after seven years of service. Several longitudinal cracks also developed apparently during the harsh winters of 1976–1977 and 1977–1978 as a result of excessive frost heave under the pavement. These cracks were repaired during the fall of 1978.

Additional distresses included concrete spalling at the jacking gap area. Spalls were less than 1 in. deep and not greater in area than 15 in^2 and appeared to be caused by insufficient clearance between the top flange of the female beam and the gap concrete. In addition, during December 1975 two transverse joints were repaired after damage caused by the locking up of the tongue and groove connections. In some locations the female beam separated from the concrete slab 2 to 3 ft inward from the edge of the outside lane. During repairs it was reported that anchor pocket welds had broken. According to the last pavement instrumentation, joints movement was measured to be only about ¼ in. Finally, shoulder distress was observed along many of the slabs, and open joints with shoulder drop-offs in excess of 2 in. were observed at a number of locations.

2.4.2 EVALUATION OF PCP SECTION IN MISSISSIPPI

This project was built in 1977 and consisted of 2.5 miles (both directions) along a 4-lane section of highway US 84 at Brookhaven, Mississippi. The project consisted of 58 slabs; each slab is 450 ft long, 24 ft wide, and 6 in. thick (Ref 7). Steel bullheads were used at the end of the prestressed slabs. Gap slabs were 8 ft long and incorporated two active expansion joints, similar to those used in Pennsylvania. Styrofoam was used at the expansion joints to absorb the horizontal movements, and polysulphide joint filler was used to seal the joints. The project had a road crossing within the prestressed pavement construction, and traffic flow was maintained by constructing one-half of the slab length of the prestressed pavement at a time. A keyed construction joint was built, and the first half-slab cast was stressed from one end only. Later the tendons were extended through the other half, and the second half-slab was cast. The construction joint anchors at the first half-slab were loosened and removed, and the full slab was prestressed.

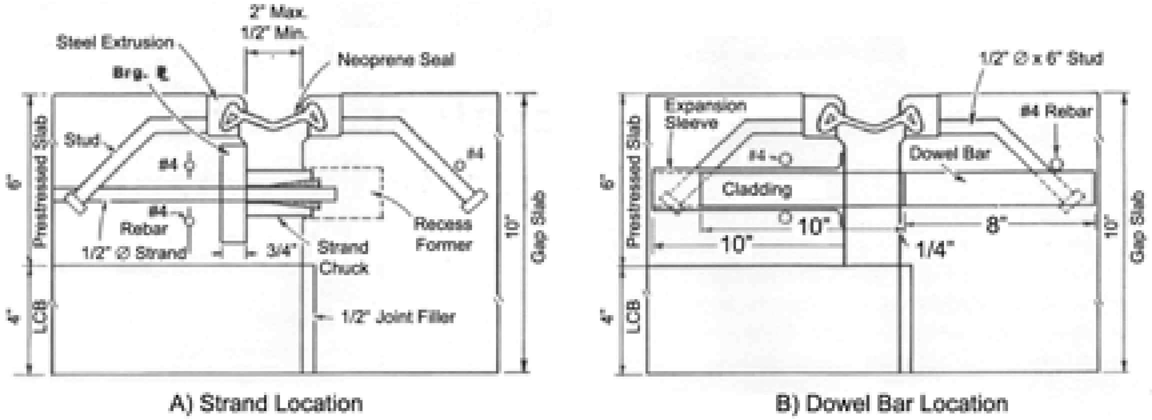
After 26 years of service, it can be reported that the project has performed reasonably well. A few transverse cracks developed soon after concrete placement and closed up after final prestressing, except for one slab placed with only one layer of polyethylene sheet. Also, a longitudinal crack developed in 15 slabs as a result of problems with the polystrip inset used to form longitudinal joints. Over the years, the joint filler material has been replaced at most joints. Seasonal joint width changes were reported to be about 1½ in.

Regarding distresses, minor spalling developed at many joints, and before 1990 the spalls were repaired using rapid-setting concrete, but since that time they have been repaired using asphalt concrete. During the summer of 2000, two full-width asphalt patches were placed at the west end of the project. As of the summer of 2000, only 2 of the 118 transverse joints have required major repair, and about 15 joints have required minor spalling repair. The pavement is now becoming a maintenance concern and is expected to be rehabilitated soon.

2.4.3 EVALUATION OF PCP SECTION IN ARIZONA

This 1.2-mile-long project was constructed during March and April 1977 along an experimental section of the Superstition Freeway (State Route 360) in Tempe, Arizona. The project consists of 30 prestressed slabs, 31.5 ft wide, 6 in. thick, and nominally 400 ft long. The pavement cross-section incorporated two 12-ft lanes and a 7.5-ft outside shoulder. Gap slabs were 8 ft long and 10 in. thick and were conventionally reinforced (Ref 9), and were installed during mid-May 1977. The joint was designed to accommodate a maximum joint opening of 2 in. and a minimum joint opening of ½ in. Figure 2.8A shows a detail of the strand location with its fixtures and the steel extrusions with its neoprene seal. Figure 2.8B displays the detail of the 1-¼ in. diameter dowel bars used for load transfer.

Traffic projections used for design in 1977 estimated about 26,000 average daily traffic (ADT) (191 million vehicles over 20 years) with 1% truck traffic. According to data collected, by 1986 the traffic count was in the order of 98,000 ADT with about 3 to 4% truck traffic. It was estimated that the pavement had already carried the 20-year design traffic within 10 years of operation.



A) Strand Location

B) Dowel Bar Location

Figure 2.8 Joint detail for the PCP section built in Arizona in 1977

Fine transverse cracking was noted near midslab soon after concrete placement. Most of these cracks closed up after the application of the final prestressing. Condition surveys conducted 9 and 11 years after construction showed that joint repairs were conducted at several locations. Joint spalling was the major problem leading to repairs, apparently resulting from poor concrete consolidation at some of these joints. Additionally, the neoprene seals were difficult to hold in place, and many of the joints were filled with sand, gravel, and other debris. A section of a prestressed slab was totally removed for a length of 16 ft and replaced with new concrete. The continuity of the prestressing steel was successfully maintained in this repair. The pavement also exhibited a substantial amount of longitudinal cracking that apparently developed over the prestressing strands in the wheel paths. Some of these cracks were 100 ft and, in some cases, longer.

Several prestressed slabs and gap slabs experienced corner breaks, possibly due to loss of support and restraint provided by misaligned load transfer devices. More problems were observed along the eastbound section of the project that was placed first. Figure 2.9 displays a picture of a failed patch at a corner break of the section.

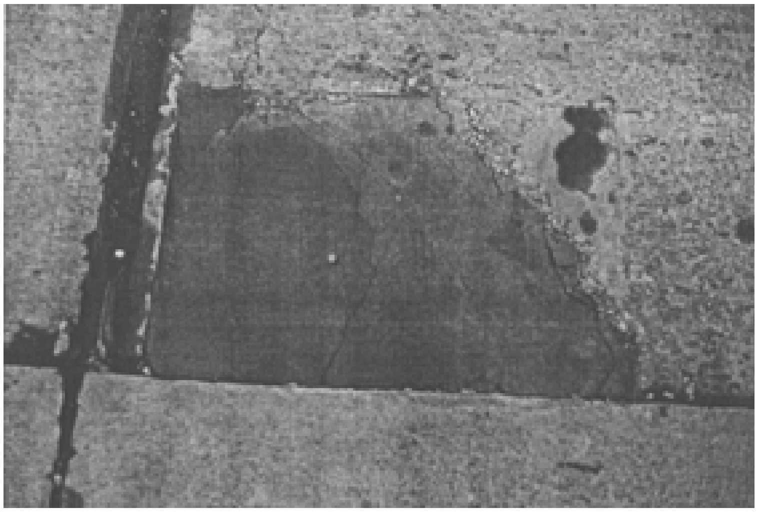


Figure 2.9 Failed patch at a corner break

Deflection testing conducted during the spring of 1988 indicated low load transfer efficiency (LTE) at joints, with values ranging from 40 to 68 percent. Deflection testing also indicated significant loss of support beneath the slab corners. The pavement started deteriorating in 1988 when the pavement exhibited rough ride as a result of undulations left in place during stop-and-go paving operations and as a result of patching at gap slabs and at joints. The PCP section was removed during 1997 as a result of the realignment of a nearby intersection (Ref 7).

2.5 Improved Domestic Applications of PCP (1980s and 1990s)

Following international experiences in Europe and Japan and the three previously described full-scale projects in Pennsylvania, Mississippi, and Arizona, some more projects were constructed in the United States. No literature was found documenting more recent PCP construction elsewhere. More recent domestic projects have taken place in Texas in 1985, where a one-mile PCP section was built on the southbound lanes of IH 35, near West, Texas. The section consists of 32 slabs with lengths of 240 and 440 ft, widths of 16 and 22 ft, and a thickness of 6 in. in main lanes and shoulders. No gap slabs were used; instead central stressing pockets were developed and implemented. This PCP has been instrumented several times in 1988, 1989, and 2001. A full description of this project is contained in Chapters 3 and 4.

Another project was constructed on highway US220 (also IH99) in Blair County, Pennsylvania. The section was built during the fall and winter of 1988 on the two northbound lanes and consists of 60 slabs 400 ft long, 24 ft wide, and 7 in. thick. Gap slabs were used and post-tensioned as part of the main slabs. A survey conducted in 1990 showed a significant amount of transverse cracking attributed to ineffective prestressing or loss of prestress. Although there is excessive cracking, the riding quality of the pavement is still acceptable, and no major repairs have been done.

One more application of PCP took place in Illinois at the O'Hare International Airport. In 1980 two segments of post-tensioned pavement were placed on the east end on Runway 27L. Each segment was 400 ft long, 150 ft wide, and 8 to 9 in. thick. Post-tension was applied in both longitudinal and transverse directions. Individual lanes 24 ft wide and 400 ft long were cast, and post-tensioned longitudinally as soon as they reached specified strengths. Once all six lanes were cast and fully prestressed longitudinally, transverse post-tension was done across all six lanes. Post-tensioning was done in three stages, and then tendons were bonded using a pressured nonshrink grout. Transverse tendons were placed first ½ in. below the neutral axis of the slab, and the longitudinal tendons rested directly on the transverse tendons.

Underneath the PCP slab a 2 in. thick sand leveling course was placed over the existing CRCP, and then two polyethylene sheets were placed between the leveling course and PCP slab to reduce the friction between layers. The two different slab thicknesses were used for experimental purposes and to observe differences on performance. After 22 years of service under stringent conditions (50,000 to 60,000 departures annually over the life of the pavement), the PCP is reported to be in outstanding condition. Except for a problem caused by snowplows in a joint, there has been no maintenance at all on the slabs. A recent condition survey conducted in January 2001 indicated no existence of cracks or other distresses.

The most recent documented construction of a PCP was done at the Greater Rockford Airport in Illinois. The pavement was built in 1993, is formed of one single segment 1,200 ft long, 75 ft wide, and 7 in. thick and is part of the primary taxiway serving the united parcel service (UPS) air-to-ground terminal at the airport, so the pavement supports heavy aircraft in a daily basis. The post-tensioned slab was cast using fibrous concrete with flexural strength of 1,000 psi, and the concrete was placed in a single paving lane of 75 ft using a bridge paver. The post-tensioned slab was placed on top of a 6 in. thick econocrete subbase and a 6 in. thick granular subbase under the econocrete. A double layer of polyethylene was used to reduce friction between the slab and the subbase.

Only longitudinal post-tensioned slab was applied to this PCP, and tendons were stressed to an 80 percent of their ultimate capacity. Again, the post-tension was done in three stages starting approximately 12 hours after casting the concrete and then continuing as the concrete reached prescribed strengths. From the time the PCP was first constructed in 1993 through 1998, it was monitored four times a year. Hairline cracks were observed shortly after construction, but those did not propagate during the first five years when monitored. During the last report in 2000, the pavement was found to be performing as designed with no significant distresses and has not required any maintenance.

2.6 Summary of Domestic Experience

From the review of all the previous PCP projects built in the United States, it can be concluded that a properly designed and constructed PCP might be able to provide a maintenance-free pavement with a service life of 20 or more years. The primary concerns with PCP projects have been joint-related problems and effective cracking control. Ideally PCP slabs should not crack at all. Even though PCP has significantly fewer joints, any joint repair requiring corrective work to the joint hardware results in a major repair effort. Experiences to date also indicate that PCP can be designed to incorporate considerably thinner slabs than conventional construction procedures, and this is probably one of its main advantages. It has been demonstrated that when construction problems related to the prestressing are encountered, they can be adequately corrected.

Regarding the characteristics of PCPs, a wide range of slab lengths has been used in domestic projects. Slab lengths ranging from 400 to 600 ft appear to perform well provided proper prestressing levels are used for the longer slabs. Even the 1,200 ft long PCP single segment built at the Greater Rockford Airport in Illinois has shown adequate performance with only longitudinal post-tension. However, it is recommended that shorter slab lengths be used until better joint hardware is developed. Some specific observations regarding the design, construction, and maintenance of PCPs are as follows:

1. If concrete shoulders are used, they should be constructed monolithically with the traffic lanes and prestressed together.
2. Computed long-term minimum midslab prestress levels should be at least 50 psi and preferably about 100 psi.
3. Joint hardware design needs to be simplified and, if possible, standardized for different applications for highways and airport facilities.
4. PCPs can be repaired without much difficulty, from tendon replacement to joint hardware replacement.

2.7 Advantages and Shortcomings of Using PCP in Lieu of Conventional Paving Methods

The advantages of a PCP when compared to a conventional paving method like CRCP or JCP reside mainly in a better utilization of the construction materials. Among the advantages of using PCP instead of conventional pavements are the following:

1. Substantially thinner slabs are used in PCPs, which means lower concrete mass and fewer associated problems (e.g., high temperature differential between top and bottom of the slab and delamination)
2. Prestress of the slabs implicitly makes use of the outstanding compressive strength of concrete, and at the same time it minimizes or even reduces the effects of tensile stresses in the pavement
3. PCP technology allows greater contiguous concrete surfaces with fewer joints, which results in smoother pavements
4. A PCP can be temporarily overloaded without causing damage, provided that stresses in the prestressing steel remain within the elastic range
5. If design and construction of PCP are successful within some limits, the pavement can be considered as maintenance-free during its entire life

Because PCP construction implies the use of thin concrete slabs, deflections are a major concern, since a secondary failure may occur in other layers, and thus should be checked and monitored to avoid excessive deformations. Besides excessive deflection, other difficulties limiting the widespread use of PCP include the following:

1. The initial higher construction cost, due primarily to the cost of the transverse joints
2. The absence of a standard design methodology
3. The unfamiliarity of pavement engineers with methods of design and construction
4. The repair of cracking in PCP is more laborious than in conventional concrete pavements

2.8 Summary

At the present time wider use of PCP is being explored in the United States and overseas. The literature review presented here has demonstrated that PCP construction is very promising and provides long-term, low-maintenance pavements for a competitive life-cycle cost. During the last few years there has been an embracing trend for this paving technology. Variants of PCP, like precast prestressed concrete pavements, have been and will be constructed in different states in the United States, including Texas, California, Illinois, and New York. It is hoped that the outstanding performance of several previous projects and lessons learned from the not-so-successful projects will lead to new improvements to PCP methods of design and construction, which will produce high-performance pavements.

3. APPLICATION OF PCP TECHNOLOGY IN TEXAS

3.1 Introduction

Chapter 2 of this report discussed the findings from the application of PCP technology in Europe and Japan and afterwards in the United States. This chapter focuses on the description of the experience acquired in Texas based on all the prior studies. Three PCP sections were constructed in Texas in the past: the first section was built in 1955 on a taxiway at San Antonio International Airport; the second section was a runway construction project built in 1959 at Biggs Air Force Base (AFB) in El Paso. Unfortunately, very little information was found about these projects. A fourth project was built in 1985 on IH 35 near West, Texas. The construction of this project resulted from a request from the FHWA to continue "Research Project 5E," which had as a primary objective the development of design and construction techniques for "zero-maintenance pavements." Hence, in 1983 the former Texas State Department of Highways and Public Transportation (SDHPT), now the Texas Department of Transportation (TxDOT), requested that Dr. B. Frank McCullough and Dr. Ned Burns of the Center for Transportation Research (CTR) oversee the design and construction of two new PCP experimental sections in Texas. One project was planned and designed for IH35 in Cooke County and the other for IH35 in McLennan County. The one in McLennan County was constructed under TxDOT Research Project 556. The Cooke County section was not constructed due to funding constraints.

3.2 Location of the PCP Section Built in McLennan County

Before any design was devised for the project in McLennan County, the FHWA requested TxDOT's input in selecting the appropriate sites for construction. Therefore, after searching for candidate locations and analyzing rehabilitation needs, TxDOT selected two segments on IH 35 with poor soils, a range in climatic conditions, and high truck traffic. The site selections were made in McLennan and Cooke Counties, the PCP sections were designed, and the McLennan County project was constructed on IH 35, 15 miles north of Waco, near West, Texas. The section was built in the southbound roadway starting at milepost 351.3 and extending for almost one mile south. Road FM 1858 is located just north of the section, and Exit 351 (Wiggins Road) is located to its south. Figure 3.1 is a map displaying the location of the PCP section (Ref 10).

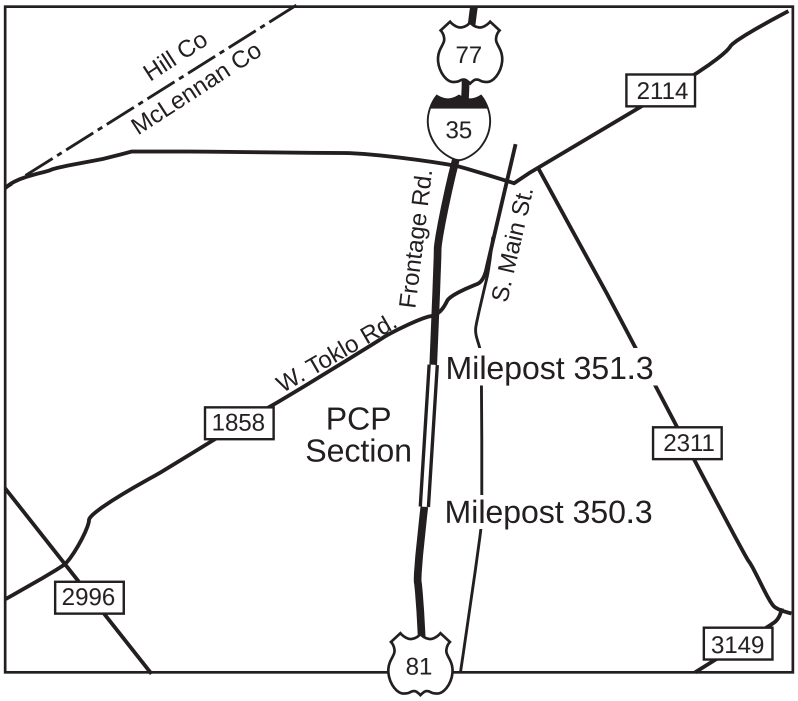
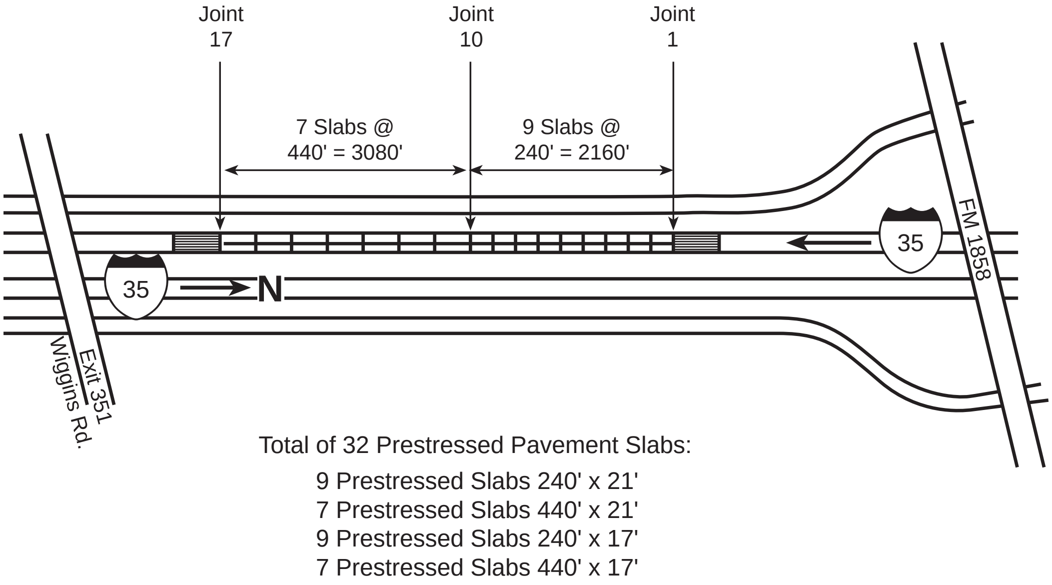


Figure 3.1 Location of PCP section constructed in McLennan County

3.3 Layout of PCP Section

The pavement section was designed and built to accommodate two traffic lanes (24 ft), plus inside (4 ft) and outside (10 ft) shoulders. Thus, the total width of the pavement is 38ft, the inside lane and shoulder, forming the inside pavement placement strip, are 17ft wide. The outside lane and shoulder form the outside placement pavement strip totaling a width of 21 ft. The pavement could have been placed full-width since all traffic was detoured to the northbound roadway, but two placement strips were used instead. The intent was to establish the feasibility of pulling two strips together and to mitigate longitudinal construction joint problems, since this would be typical of most rehabilitation projects.

Starting at the north end of the PCP section, Joint 1 marks the beginning of the pavement utilizing slabs 240 ft long with this configuration of slabs from Joint 1 to Joint 9. Joint 10 has a slab 240 ft long on the north side and a slab 440 ft long on the south side, thus, Joint 10 is designated as a transition joint. Joints 11 to 17 have 440 ft long slabs, with Joint 17 being the last joint of the PCP section at its south end, and the south side of Joint 17 is an overlaid JCP pavement. Figure 3.2 displays the layout of the PCP section (Ref 11).



Total of 32 Prestressed Pavement Slabs:
9 Prestressed Slabs 240' x 21'
7 Prestressed Slabs 440' x 21'
9 Prestressed Slabs 240' x 17'
7 Prestressed Slabs 440' x 17'

Figure 3.2 Layout of PCP section showing slab setup

3.4 History of Cross-Section

The section where the PCP is located was initially constructed in 1952, and rehabilitation tasks were performed as needed between that time and the construction of the PCP. Modifications were made to maintain an adequate riding quality of the road. According to the literature and TxDOT’s archives, three different stages or cross-sections were identified for this pavement section. The cross-section described in the following subsections evolved in this order:

1. Initial construction of JCP section (1952)
2. Construction of asphalt concrete overlay (1975)
3. Construction of PCP overlay (1985)

3.4.1 INITIAL CONSTRUCTION OF JCP SECTION (1952)

The first pavement section constructed consisted of a JCP pavement, built as an improvement project to provide a divided highway on US81 since the northbound roadway was built earlier. When the construction of the interstate highway system began the highway was designated Interstate Highway 35. The pavement structure from top to bottom consisted of a 12 in. thick plain concrete slab with expansion joints spaced at 120-ft intervals and curling and warping joints spaced at 15 ft. Underneath the concrete slabs, a 5 in. thick granular base rested on a 6 in. thick lime-stabilized subbase. Finally, natural compacted soil supported the whole structure. Figure 3.3 is a sketch of the cross-section of the pavement as it was originally built.

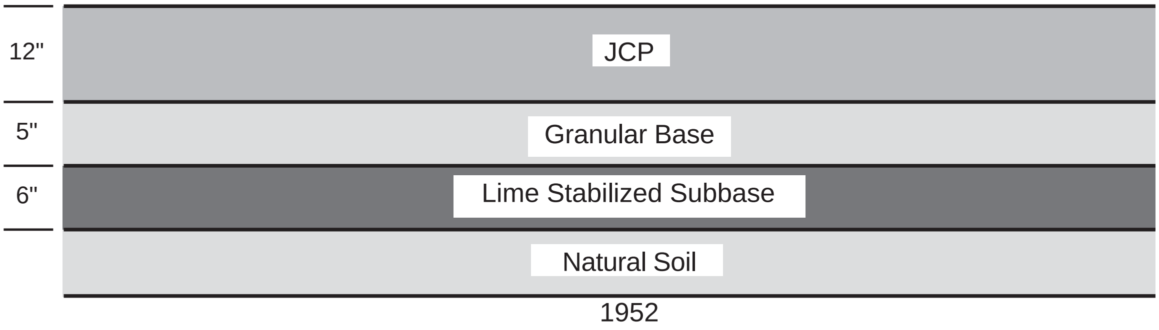


Figure 3.3 Initial cross-section of pavement on IH 35 southbound near West, Texas

The JCP pavement had the same geometry as it currently has. It consisted of a 38 ft wide section with two traffic lanes 12 ft wide each, plus shoulders of 10ft and 4ft on the outside and inside, respectively. The flexible shoulders with an asphalt concrete surface were built in the same time frame as the JCP main lanes. Figure 3.4 depicts a plan view of the constructed section.

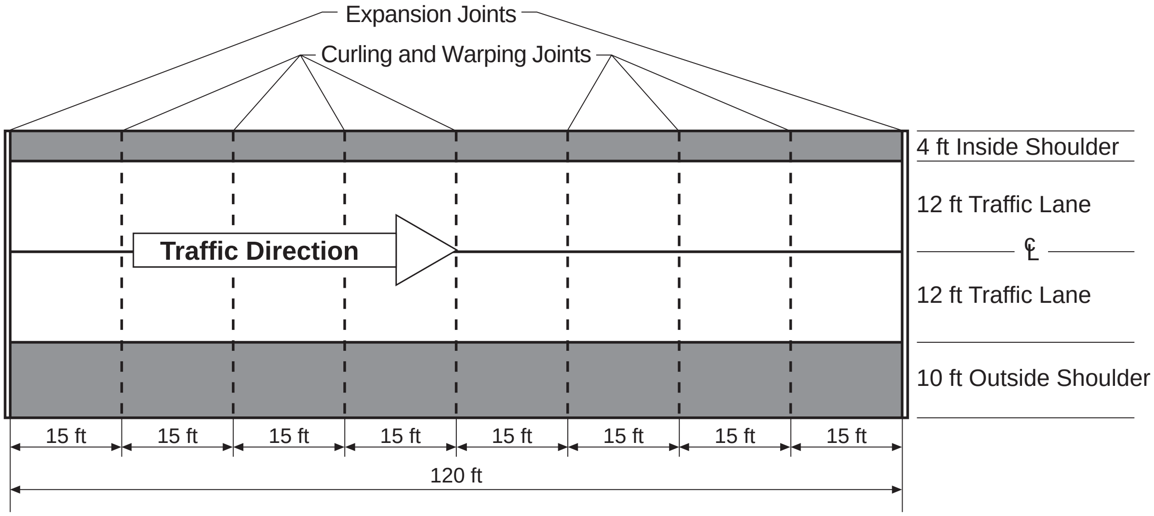


Figure 3.4 Plan view of pavement section on IH35 southbound near West, Texas

3.4.2 CONSTRUCTION OF ASPHALT CONCRETE OVERLAY (1975)

After 23 years of service, the JCP pavement required rehabilitation to restore an adequate riding surface. Thus, activities including patching and sealing of the JCP were performed, and a 4 in. thick asphalt concrete overlay was constructed on top of the structure. The width of the section stayed as it was originally constructed. Figure 3.5 is a cross-section of the pavement after being rehabilitated.

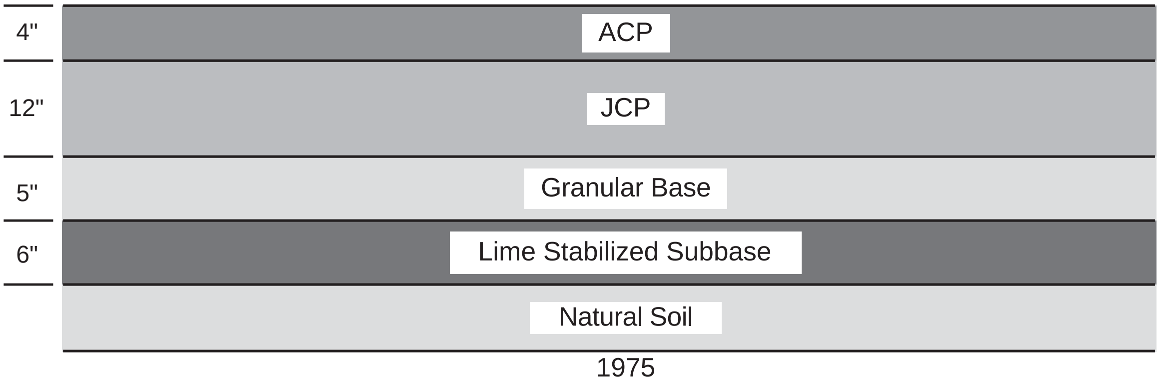


Figure 3.5 Cross-section of pavement section after being rehabilitated in 1975

3.4.3 CONSTRUCTION OF PCP OVERLAY (1985)

In 1985, after 10 years of service and after routine maintenance on the asphalt overlay had not significantly improved the riding quality, another rehabilitation job was required for this pavement section. At the same time, engineers from the SDHPT (now TxDOT) selected this candidate section as the location for building the PCP pavement. The construction of the section took place from September 17 to November 20 in 1985.

For the construction of the PCP, the existing asphalt concrete pavement (ACP) surface had to be treated. First, the oxidized 4 in. thick 10-year-old asphalt concrete layer was removed, the JCP concrete layer was patched, and cracks and joints were sealed. Next, a new 2 in. thick asphalt concrete layer was placed as a leveling course, and on top of that, the current 6 in. thick PCP concrete slabs were cast. The section was opened to traffic in December 1985. Figure 3.6 shows a cross-section of the PCP.

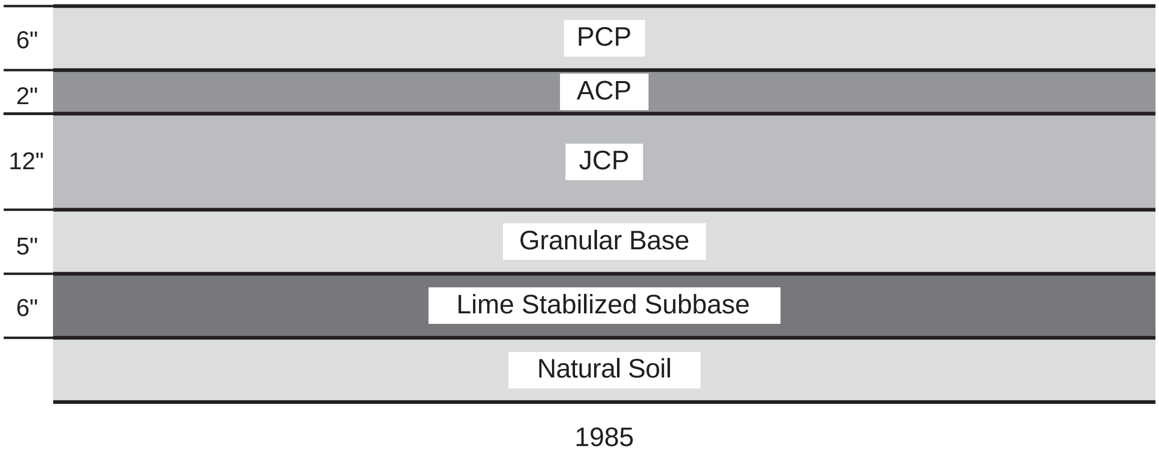


Figure 3.6 Cross-section of pavement section as it is since 1985

3.5 Improvements to PCP Technology in Texas

In order to design and construct the PCP section in McLennan County, CTR (Ref 5) developed a series of improvements to PCP pavements to correct some of the failures observed on previous projects built in the United States and overseas. The innovations in PCP technology introduced in Texas included three critical aspects:

1. Construction of central stressing pockets instead of gap slabs
2. Introduction of transverse prestress
3. Design of an efficient transverse joint

3.5.1 CENTRAL STRESSING POCKETS

Gap slabs and their joints have presented problems on various projects, as described in Chapter 2, so central stressing pockets were used instead of gap slabs. With this technique longitudinal tendons are stressed at internal blockouts or central stressing pockets. This stressing method was and is still used in building construction where end stressing is not feasible. Additionally, it was used with success to perform repairs on the Pennsylvania PCP project built in 1977. Central stressing pockets offer some advantages over gap slabs. Gap slabs require two joints, one at each side on the longitudinal direction. Central stressing pockets require only one joint per slab, which means that less construction time is required and more rapid opening to traffic is achieved; having only one joint reduces the possibility of failure, the need for maintenance activities, and construction costs. Figure 3.7 shows a detail of the central stressing pocket designed for the PCP section in McLennan County. Here, longitudinal tendons extending from both ends of the slabs are coupled in the pocket once they have been stressed with a loading ram.

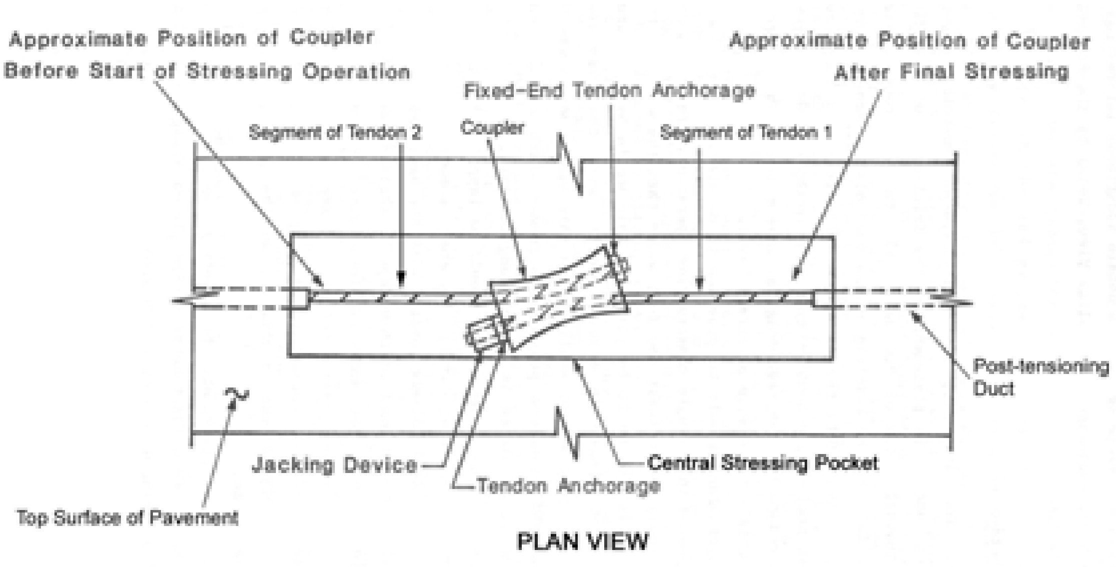


Figure 3.7 Detail of central stressing pocket used in Texas

Although central stressing pockets offer great advantages over the construction of gap slabs, several issues required careful management during construction, including the following:

1. Holding the stressing pocket forms in position during concrete placement
2. The bottom part of the screed of the passing slip-form paver hitting the stressing pocket forms
3. If concrete vibrators are not well-positioned onto slip-forms, poor concrete compaction might result near the stressing pocket blockouts

If a good construction plan is developed ahead of time, these problems can be overcome, thus making the paving process much easier and significantly reducing construction time.

3.5.2 TRANSVERSE PRESTRESS

Transverse prestress has been found to be essential for the good performance of PCP. Investigation of all the projects built in the United States reveals that, in addition to the PCP sections built at Biggs Air Force Base and McLennan County, the only other PCP section constructed using transverse prestress was that built in 1980 in Illinois at the O'Hare International Airport. According to the literature review described in Chapter 2 of this report, this project has performed much better than its predecessors, due largely to the benefits obtained from transverse prestress. It has been observed that a lack of transverse prestress might result in extensive longitudinal cracking in the pavement (Ref 6).

Having learned from previous experiences and their failures, CTR recommended that transverse prestress be used in the construction of the PCP section in McLennan County. Different configurations of the tendons were analyzed by experts at CTR, and finally the transverse tendons for the PCP section were laid out as shown in Figure 3.8. The looped tendon configuration shown here was adopted because it efficiently resists wheel loads, prevents longitudinal cracking, and prevents possible separation of the separately placed pavement strips.

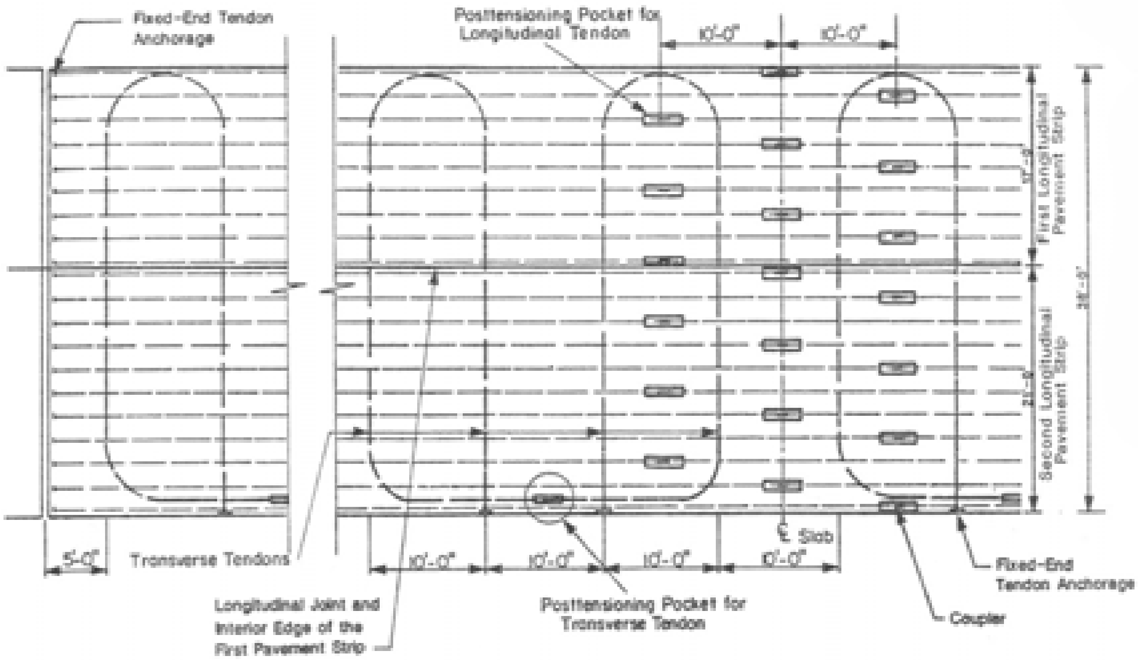


Figure 3.8 Plan view of transverse tendons used for PCP in Texas

The looped tendon configuration used in the PCP section had the following advantages:

1. By looping each strand, a transverse stressing length of 40ft was covered. This operation required less time than would have been expended if straight tendons had been used
2. Looped tendons permitted the exterior edges of the pavement strips to be slip-formed without the need for formwork and without interference from protruding tendons

Among the inconveniences associated with the use of looped tendons, the following were found during construction:

1. It was laborious to lay out the tendons in the field and hold them in the desired curvature

2. Transverse tendons that protruded from the first pavement strip interfered with slip-forming operations for the second strip
3. Looped tendons experience higher prestress losses than straight tendons; hence, more tendons were required to reach the desired level of prestress

Although these inconveniences were encountered during construction, the use of transverse prestress has contributed greatly to the good performance of the PCP section after 17 years in service.

3.5.3 TRANSVERSE JOINT

The literature research suggested that in almost all the previous projects, expansion joints or transverse joints caused most of the major problems in the PCP. In some cases, and specifically for the section built in Arizona in 1977, faulting and spalling of the concrete was found adjacent to the joints. Therefore, the design of the joint used in Texas was refined and failures were corrected resulting in a more successful expansion joint. The joint hardware is shown in Figure 3.9 and consists of a steel support structure with Nelson deformed bars used to secure the joint to the pavement. Stainless steel dowel bars were used for load transfer, and dowel expansion sleeves allow free movement of the opposite slab. The neoprene seal fits between the two sides of the joint and impedes the intrusion of undesired materials deep into the joint.

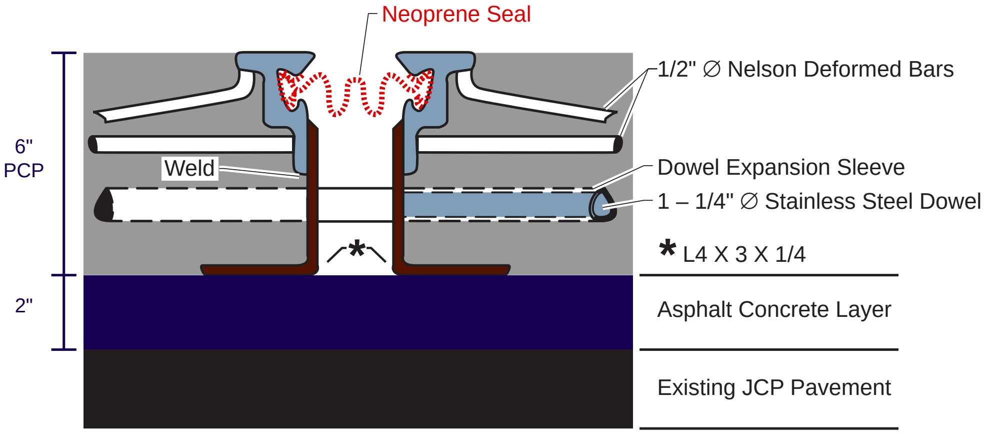


Figure 3.9 Transverse joint developed for the project in McLennan County

The transverse joint used in the section in McLennan County has performed without major problems. Chapter 4 of this report contains an evaluation of the joints in terms of their load transfer efficiency. Some time-consuming activities while handling the joints and the concrete near the joints during construction include the following:

1. The transverse joint was held stationary during the application of the pretension to longitudinal tendons before pouring the concrete. It was necessary to temporarily secure the joints to the ground to avoid their displacement while the concrete paver passed over them

2. The concrete adjacent to the joint had to be carefully vibrated using a hand-held vibrator to avoid poorly compacted material. This was a crucial process during paving that required special attention

Other than these minor inconveniences that were successfully managed during construction, transverse joints have performed as designed.

3.6 Synopsis of the Construction of the PCP Section

This section summarizes the procedure followed during placement of the PCP overlay in McLennan County. Construction steps and problems encountered during each step are stated as they appeared.

3.6.1 LAYING DOWN OF THE REDUCING FRICTION MEMBRANE

After preparing the old structure, the first task involved the placement of a single layer of polyethylene sheeting that was placed on top of the leveling layer of asphalt. A single layer of polyethylene proved to be efficient in reducing the friction between the supporting asphalt layer and the PCP slab. When the polyethylene sheet was rolled out longitudinally, the edges were tacked down to prevent the wind from blowing the sheeting out. Various weights were placed at different points on the sheeting until the tendons were placed on top. Holes in the sheeting were repaired before casting the concrete layer. This operation required 4 to 5 workers and was conducted with no problems or delays.

3.6.2 PLACEMENT OF SIDE FORMS

When side forms were used, they were placed on top of the polyethylene sheeting and held down using spikes buried in the underlying pavement. After a slab was poured, side forms were removed and placed for the next slab to be poured. Initially, side forms were used on both edges of the first pavement strip, but because paving operation was slowed down the forms were used on only one edge of the pavement, while the other edge was slip-formed. The edge of the slab with transverse tendons extending out required the use of side forms. Slip forming only one edge speeded up the paving operation. When the second pavement strip was placed, the edge of the previously placed strip served as a side form, and the other edge was slip-formed.

3.6.3 TENDON PLACEMENT AND FORMING OF CENTRAL STRESSING POCKETS

The longitudinal tendons were laid out first, and then transverse tendons rested on top of them. The transverse tendons were tied to the longitudinal tendons after they were both placed on chairs located at various predefined points painted on the asphalt layer. The points could be seen through the polyethylene sheet and helped in positioning the tendons in the appropriate locations. Next, the longitudinal tendons were anchored to the transverse joints, and the transverse tendons were guided through the side forms.

Central stressing pocket forms were also placed at this time. The forms were made of steel boxes with no bottom and a removable top that could be slipped over the tendons after the tendons had been placed. Figure 3.10 displays a set of forms temporarily placed on the ground showing where the longitudinal tendons will reach.

Figure 3.10 Placement of central stressing pocket forms

3.6.4 PLACEMENT OF TRANSVERSE JOINTS

Transverse joints were placed at almost the same time that the tendons were placed. Initially and for all the joints, no gap was provided, and the two sides of each joint were held together by tack welding short metal straps across the top of both sides of the joint. The joint was anchored to the ground (as shown in Figure 3.11) to prevent its rotation when tension was applied to the tendons on one side. The anchoring of the joint was done using steel bars that were driven into the ground on both sides of the joint.

Figure 3.11 Placement and anchorage of transverse joints

Immediately before a slab was poured these anchors were removed from the joint, leaving just the metal straps holding the joint together. At this point, the joint was unable to rotate due to the presence of the concrete. When a new slab was poured on the other side of the joint, the metal straps were removed, enabling the joint to open and close freely with any slab movements.

Some joints arrived warped to the site, causing problems with tack welding the joint together with no initial opening, and some other joints arrived from the manufacturer with no holes for the dowels. Those problems had to be solved on the site as they appeared.

3.6.5 CONCRETE PLACEMENT, FINISHING, AND REMOVAL OF STRESSING POCKET FORMS

The concrete was delivered to the site by trucks and was placed directly in front of the paver, while a slip-form paver was used to pave both lanes of the overlay. The concrete was partially spread by hand before the paver moved over it. Hand placement of the concrete was necessary around the chairs holding up the prestressing tendons to prevent the paver from displacing the tendons as it went by. Figure 3.12 shows the configuration of longitudinal and transverse tendons and the lifting chairs and also shows the forms for the central stressing pockets, a concrete truck, and the paver.

Figure 3.12 Paving operations for a PCP slab in McLennan County

There were some problems with the concrete when delivered by trucks. The quality was not consistent, and in some cases it had to be rejected. On one occasion, a set retarder was used for hot-weather day paving, and mistakenly it was still used during the evening placements, causing very slow setting of the concrete and low early concrete strengths.

The finishing of the concrete surface was done by hand, using the appropriate tools to provide surface texture. After the final finishing of the concrete, the stressing pocket forms were removed, and a curing compound was sprayed on with a hand-held sprayer. Once the concrete gained sufficient strength, 8 to 10 hours after placement, a small post-tensioning force was applied to prevent transverse cracking.

3.6.6 POST-TENSIONING OF LONGITUDINAL TENDONS

Post-tensioning operations were conducted in two stages. The initial stressing of the longitudinal tendons was performed after the concrete gained enough strength to withstand the prestress forces. Early strength of the concrete was determined by testing cylinders sampled from the same concrete mix. In general, the initial stressing was done between 8 and 10 hours after the slab had been poured. The tendons were post-tensioned in a second stage, approximately 48 hours after the slab was poured. Both initial and final post-tension were performed with a portable hydraulic ram like the one shown in Figure 3.13.

Figure 3.13 Hydraulic ram used in post-tensioning operations

Initially, the stressing pockets were only 30 in. long and were too small to accommodate the stressing ram. Therefore, the stressing pockets were enlarged for the following slabs from 30 to 48 inches. Only minor problems were encountered during post-tensioning operations, when grips slipped or broke during stressing. The tendons at the center of the slab were always stressed first. Next the tendons were stressed by alternating out towards the tendons at the edges of the slab.

3.6.7 POST-TENSIONING OF TRANSVERSE TENDONS

Transverse tendons were post-tensioned after all paving operations were complete and prior to opening to traffic. Each tendon was stressed in only one stage, instead of the two stages used for the longitudinal tendons, since the placement widths were insufficient to develop longitudinal cracking. The transverse tendons were stressed after allowing the first pavement strip to move independently of the second pavement strip, to match up transverse joint openings. Once transverse tendons were stressed, very little differential movement occurred between strips. Initial transverse joint openings and the time allowed before stressing had to be empirically and visually determined so that the final joint openings were equal for the two slabs. Similarly, as with longitudinal tendons, transverse tendons were stressed starting at the middle of the slab and then alternating out towards the two ends of the slab.

3.6.8 LONGITUDINAL JOINT

The longitudinal joint between the two pavement strips required preparation to prevent bonding between the two structures. After the first pavement strip was paved, the edges of the slabs were coated with asphalt, which served as a bond breaker between strips. A compressible material was then placed around the transverse tendons at the longitudinal joint. Extra compressible material was placed on those tendons located near the ends of the slabs, where large differential longitudinal movements between the two pavement strips were expected.

3.6.9 PATCHING OF STRESSING POCKETS

The stressing pockets were filled with concrete after the stressing operations were completed for each slab. The pockets were cleaned of all debris and then filled, and the concrete was finished and textured. Wet mats were placed over the concrete in the pockets to promote curing. The large number of pockets increased the need for hand-finishing for the project significantly, and it was difficult to get a good match between the concrete of the slab and the concrete of the pockets, basically due to the different concrete ages.

3.7 Summary

Although Texas' experience in the application of PCP technology dates back from the 1950s with the construction of a Taxiway at the International Airport in San Antonio, it was not until 1985 that major improvements were made based upon other experiences around the world and in the United States, allowing for the design and construction of the section in McLennan County. This project, directed by CTR, incorporated new developments that included the use of central stressing pockets instead of gap slabs, implementation of transverse prestress with a looped pattern, connecting two longitudinal placements, and use of an efficient hassle-free transverse joint. The PCP section was designed to last for 20 years, and after 17 years of service under critical traffic conditions, it is still in very good condition.

4. EVALUATION OF THE EXISTING PCP IN TEXAS

4.1 Introduction

The existing PCP section near West, Texas, was constructed in 1985, and since then it has been continuously monitored by means of instrumentations provided at different times. The first data were collected right after construction, and after that the section was instrumented on several occasions during 1988 and 1989. The latest data collection was conducted in April and June 2001. This chapter focuses on the evaluation of data collected in all the instrumentations that have been done since the section was constructed. This step represents a preliminary step that is valuable for the design of the new PCP section to be constructed near Hillsboro, Texas. The evaluation procedure covers the analysis of data that affect the performance of the PCP, including a visual inspection of the pavement surface, joint movement, joint load transfer capability, and deflection measurement. Based on this evaluation, the new design presented in Chapter 5 was developed to correct potential failures in the new PCP section.

4.2 Evaluation of PCP Section

Evaluations of the pavement have been done at various times in two different ways. The first type of evaluation consists of condition surveys that collect detailed information for each slab of the PCP, including information about distresses commonly found in conventional portland cement concrete pavement (PCCP) such as cracking, spalling, punchouts, etc. Additionally, the general condition of transverse joints and presence of debris in the joints are reported.

The second type of evaluation is related to the inspection of the behavior of the pavement from a structural standpoint and includes deflection measurement at different points along the section using loading devices such as the falling weight deflectometer

(FWD) and rolling dynamic deflectometer (RDD). Although both FWD and RDD devices were used to measure deflections, only the FWD data are presented herein. Likewise, measurement of horizontal and vertical movement at the ends of slabs and estimation of load transfer efficiency at transverse joints are part of this type of evaluation.

4.2.1 CONDITION SURVEY

Condition surveys have been performed at different times since the PCP section was constructed in 1985. The characteristics of the PCP require the surveys to be conducted by walking, first on one side of the pavement to rate one lane and then walking on the other side to rate the other lane. By conducting the surveys in this manner, the rater ensures that observations are reliable for each slab and joint. Extent and severity of distresses are recorded in the same way as for conventional concrete pavements, although instead of recording data in condition survey forms, it is done schematically by drawing slabs and joints directly to paper. Schematics of condition surveys conducted in the 1980s can be found in the literature related to the performance evaluation of the PCP section (Ref 12). Schematics of the latest condition survey conducted in April 2001 are contained in Appendix A.

4.2.1.1 Evaluation of PCP Slabs

In general, the latest condition survey showed that the section is in very good condition, and only few localized distresses were found in the slabs. From the first observation conducted in April 1988 to the second one conducted in February 1989, only small differences were reported (Ref 12). The survey conducted in April 2001 showed more longitudinal cracking on the outside lane and a few more distresses compared to the data collected in 1989.

The distresses found in April 2001 included four asphalt concrete patches (ACP) located in the outside lane of Slabs 2, 11, and 16, the one in Slab 2 being the largest one, starting at a central stressing pocket and extending for 34ft, and covering a longitudinal crack. Figure 4.1 depicts a layout of the PCP section and arrangement of the slabs. Of the other three patches one is located in Slab 11 and two are in Slab 16, and all are smaller than one square foot. The literature reports that these patches were small potholes that originated from the unintentional incorporation of debris or clay balls in the concrete during mixing. Figure 4.2 shows the ACP located in Slab 2, which is covering a longitudinal crack.

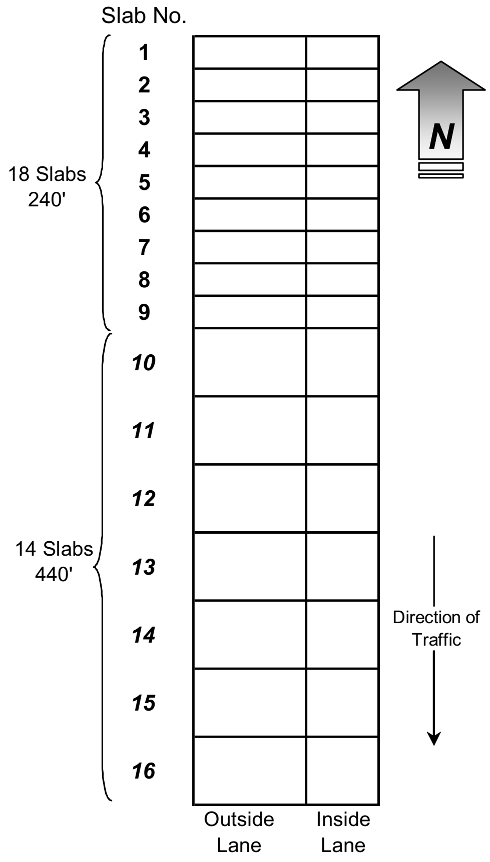


Figure 4.1 Layout of the PCP section in McLennan County

Figure 4.2   ACP found in Slab 2 of PCP section

Another type of distress found during the latest condition survey was minor surface raveling in Slabs 4, 5, 6, 12, 13, and 15. In almost all the cases, the raveling in the slab was located in the concrete adjacent to the joints. Four slabs presented raveling in the central zone of the slab.

The most frequent distress found in the slabs was longitudinal cracking. This cracking was found at an early stage, not long after construction of the pavement, and, according to observations, it originated at the corners of the central stressing pockets, where longitudinal tendons were stressed during construction. Longitudinal cracking is more frequent in the outside lane, near the outside wheel path. It is believed that longitudinal cracks extended from the central stressing pockets and propagated outwards in either one or both sides of the pocket. Cracks were not found across the concrete poured inside the pockets; they were always present at the edges. Figure 4.3 shows a longitudinal crack in Slab 11. This image shows one of the worst cases of this type of distress.

Figure 4.3 Longitudinal crack located in Slab 11 of the PCP section

Finally, two transverse cracks were found in two slabs during the 1988 survey. One crack is located in the inside lane of Slab 5, and the second crack is located in the outside lane of Slab 15. Both cracks are very tight and were almost imperceptible to the naked eye when doing the survey in 2001.

4.2.1.2 Evaluation of Transverse Joints

From the first condition survey conducted in 1988 to the last one conducted in 2001, the same general conditions have been reported for the joints. No spalling or transverse cracking has been found near the joints in either survey. In some cases, longitudinal cracks in the slabs extend until they intersect the joint in a perpendicular direction. All condition surveys have reported the presence of debris inside the joints, but fortunately horizontal movement of the slabs has not been restrained. In a few cases, the neoprene seal attached to the joint has been cut by debris. Figure 4.4 displays an image of a joint filled with debris, especially in the area of the outside shoulder.

Figure 4.4 Joint of PCP section showing presence of debris

4.2.1.3 Overall Visual Evaluation

The results from all the condition surveys conducted so far show that from the time the PCP section was constructed in 1985 up to the present time, few localized distresses have developed. The distresses include longitudinal cracks and four ACPs that have been present for almost the entire life of the section without reducing its serviceability level. All the outside lane slabs have longitudinal cracks near the outside wheel path. Almost no cracking was observed in the inside lane. Although all the joints contain debris, no major problems have developed at the joints or the concrete near the joints.

4.2.2 JOINT INSTRUMENTATION: HORIZONTAL AND VERTICAL MOVEMENT

Horizontal and vertical movement of the slabs has been evaluated at different times during the life of the PCP. The first evaluation was done right after construction in 1985, and more evaluations were conducted during 1988 and 1989, and lately, in April and June 2001. The evaluations measured horizontal and vertical movements of the PCP slabs at the ends of the slabs near the joints, where they are critical.

4.2.2.1 Horizontal Movement

The measurements made during 1988 and 1989 (Ref 12) included measurements of ambient and slab temperatures and vertical and horizontal displacements at the ends of the PCP slabs. The results obtained from those measurements conducted in the 1980s are compared herein to the results obtained from the latest measurements conducted in April and June 2001. Because the PCP slabs were anchored at the center, horizontal movement of slabs of the same length was expected to be approximately equal. Additionally, long slabs were expected to experience more movement than short slabs.

The devices used to measure the horizontal movement included a caliper and a strain measuring device or demec bar. Unfortunately, due to the magnitude of the movement of the slabs, the demec bar had a limited application because the PCP joint widths sometimes reached values that exceeded the range of measuring displacement of the demec bar. However, both gears were used whenever possible, and data were collected accordingly. Figure 4.5 displays the caliper used for the measurements and the reference points for the readings.

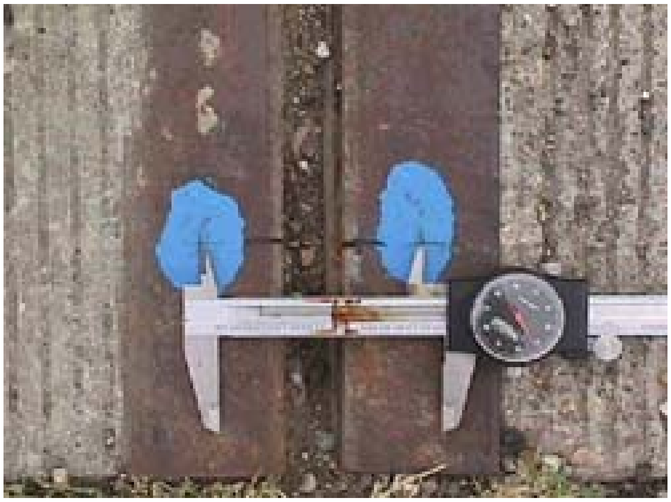


Figure 4.5 Caliper used to measure horizontal movement of the joints

The results obtained from the measurements were processed and plotted as shown in Figure 4.6. Here the curves represent the movements recorded during the instrumentation conducted in April 2001 for Joints 8, 10, 11, and 12 of the PCP section. The continuous lines represent the data recorded with the caliper, and the dashed lines represent the data collected with the demec bar. The discontinuity in the curves showing the results from the demec bar in Joints 11 and 12 is due to the lack of data, that is, when the magnitude of the movement of the slabs exceeded the allowable measuring range of the demec bar. Although the demec bar could not provide complete information, it can be seen from the curves of Joints 8 and 10 that the readings provided by the caliper are comparable to the readings obtained with the demec bar. For Joints 11 and 12 the slope of the lines recorded with the demec bar show a very similar slope to the corresponding segment of the curves recorded with the caliper. The horizontal movements shown in Figure 4.6 are congruent. It can be seen that Joint 8 experienced less movement than the other joints because it has short slabs at both sides, and its maximum relative movement reached a value of 0.07 in. Joints 11 and 12 experienced the maximum relative movements of all the joints because they have long slabs at both sides; their maximum relative movements were 0.30 and 0.35 in., respectively. Finally, Joint 10 (the transition joint) experienced a maximum relative movement of 0.16 in., which is an intermediate value compared to the other three cases. This is due to the fact that Joint 10 has one short slab on its north side and one long slab on its south side.

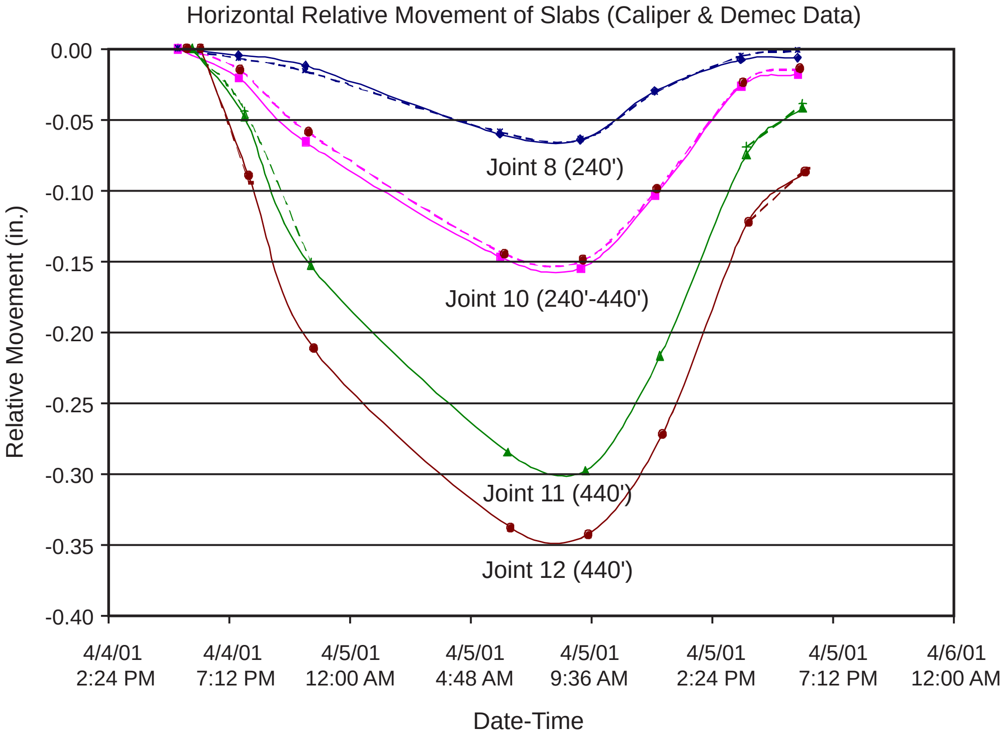


Figure 4.6 Relative horizontal movement of Joints 8, 10, 11, and 12

A comparison of joint movement using measurements recorded in the 1980s and in 2001 was conducted using concrete temperatures near the joints at a depth of 1 in. This analysis compared the variation of relative movement of the joints due to the relative change in concrete temperature. The concrete temperature measured at a 1 in. depth was used because it has a greater impact on the movement of the slab than temperatures recorded at any other depth.

Figure 4.7 shows the variation of relative temperature and the variation of relative horizontal movement for all of the short slabs. It can be seen that the slope of the regression line for all the instrumentations conducted during 1988 and 1989 has a value of 0.004, and the slope of the regression line for the measurements made in 2001 is 0.0031. Although there is a small difference between these two slope values, it could be stated that in general the short slabs have experienced the same movement throughout their life, and this explains why the two regression lines almost lie on top of each other.

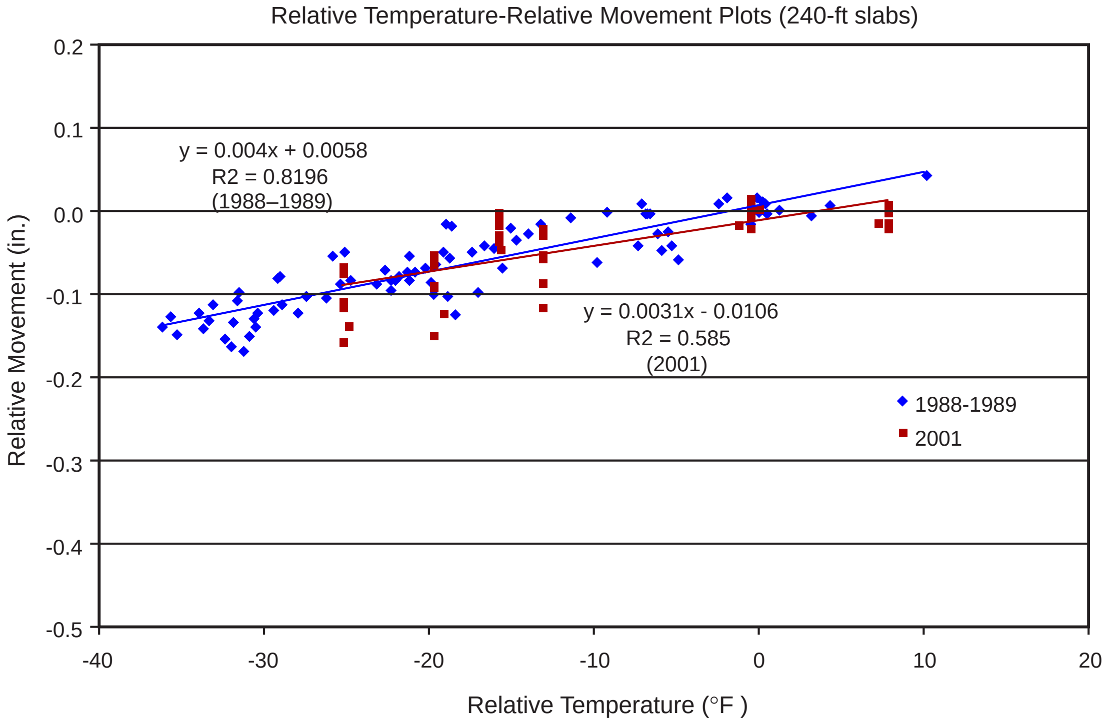


Figure 4.7 Temperature-movement relationship for short slabs

Similarly, Figure 4.8 shows a plot containing the data collected for the long PCP slabs. The joint movements recorded during the 1980s are compared to the ones observed in 2001. Unlike short slabs, long slabs exhibit a different behavior, and the regression lines are dissimilar. The regression line representing the movements recorded in the 1980s has a slope of 0.0077, and the regression line corresponding to the movements registered in 2001 has a slope of 0.0151. This means that the movements of the slabs recorded in 2001 were twice the movements recorded in the 1980s. Although horizontal movement of the slabs has increased through time, visual inspection of the joints and slabs demonstrates that both the short and long slabs are in good condition. The difference between movements for long slabs could be explained by the inevitable loss of friction between the concrete slab and the thin asphalt leveling layer underneath the slab, caused by accumulated expansion and contraction of the concrete slabs.

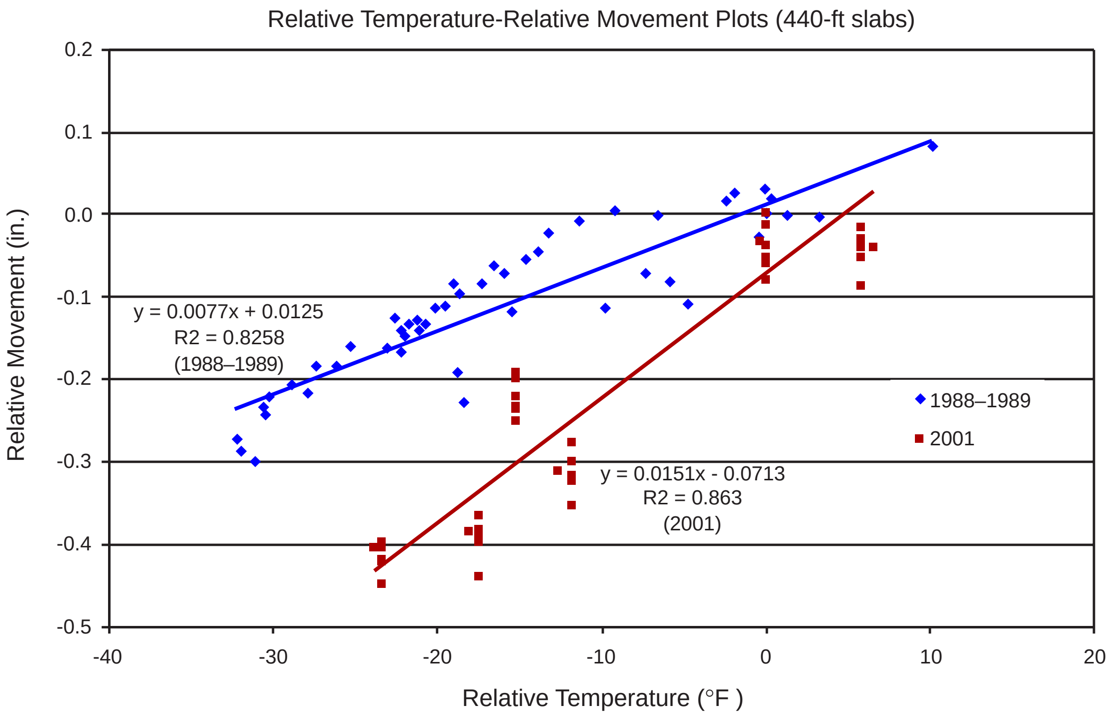


Figure 4.8 Temperature-movement relationship for long slabs

4.2.2.2 Vertical Movement

Vertical movement or curling of slabs is associated with various factors, including the temperature differential between the top and bottom planes of the slab, concrete compressive strength, modulus of elasticity, slab thickness, and joint spacing (Ref 13). Although curling is inevitable in concrete slabs, it has to be minimized in pavements to avoid excessive internal tensile stresses, which in combination with external wheel loads have a detrimental effect on the performance of the slabs.

Curling in all the PCP slabs was measured during the instrumentations conducted in 1988 and 1989 using dial indicators and linear variable differential transformers (LVDTs). For the instrumentation conducted in April 2001, dial indicators like the one shown in Figure 4.9 were installed at Joints 8, 10, 11, and 12. The gauges were installed at both sides of each joint (north side and south side) to compare the movement of the two slabs reaching the same joint and were placed at a distance of 9 in. from the center of the joint to both sides. This arrangement was adopted because it would provide comparable vertical movements and also because that was the minimum distance that allowed a comfortable position and space within which to measure horizontal displacements using the demec bar and the caliper. Figure 4.10 displays the set of dial indicators used at Joint 12. The same configuration was used for the other three joints.

Figure 4.9 Dial indicator used to measure curling at ends of PCP slabs

Figure 4.10 Setup of dial indicators used to measure vertical movement

The gauges were placed on top of the measuring points through conduit pipes that extended from outside of the pavement. The pipes were insulated with foam to reduce the change in temperature of the metal, and the foam was painted white to reflect the sun rays.

Vertical movements recorded for each joint are displayed in Figure 4.11, in which the relative vertical displacement is plotted against time for a cycle of 24 hours. In all of the curves, the total movement for the cycle reached values varying from 0.02 in. to 0.10 in., except for two cases in which, due to unknown causes, the dial indicators used in the north sides of Slabs 10 and 11 were displaced from their original recording positions to other locations. This happened overnight and it explains the inconsistent trend of the two curves beginning at their fourth reading point. From that point and beyond, the curves adopted unexpected shapes.

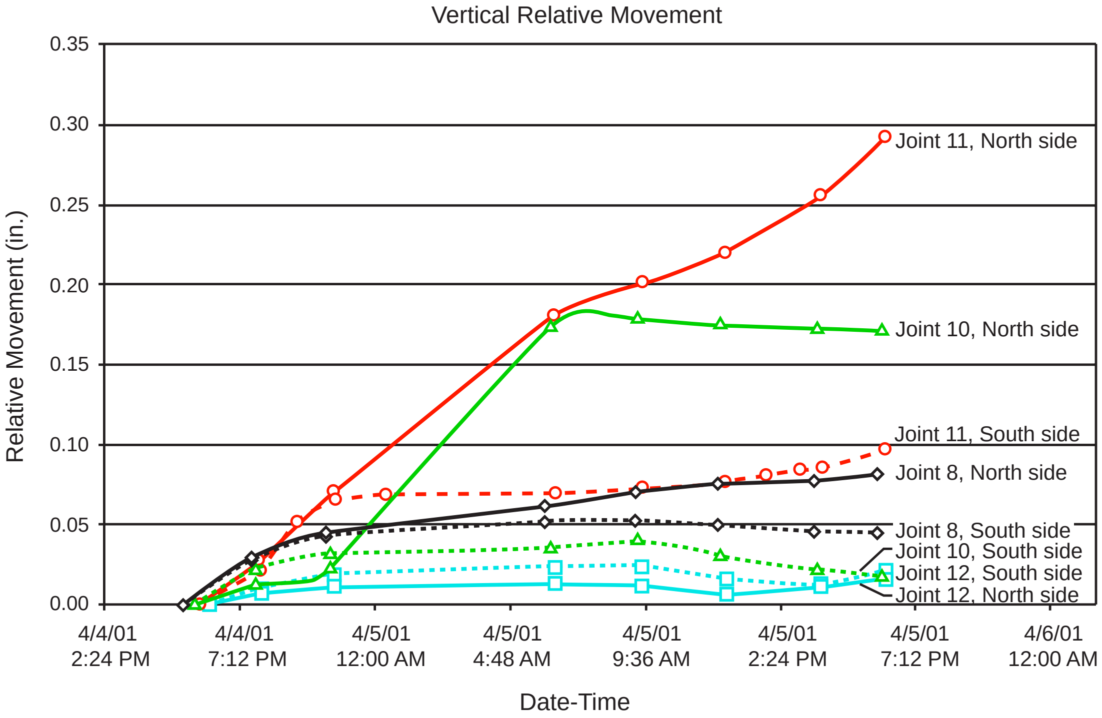


Figure 4.11 Vertical movement recorded for Joints 8, 10, 11, and 12 (April 2001)

Despite the problems with the two gauges in Joints 10 and 11, the rest of the curves followed a similar pattern. In general, Figure 4.11 shows that curling at the ends of the slabs is not dependent on their length, as the case for horizontal movement.

As a checking process and to verify that the values recorded in April 2001 were in agreement with previously recorded data, the information from the 1980s was retrieved from the literature (Ref 12), summarized, and plotted as shown in Figure 4.12. The data collected from the 1980s were correlated using regression analysis tools. A second order polynomial curve was adjusted to curling values recorded at different times and seasons during 1988 and 1989. From the regression curve, it can be seen that the minimum vertical movement is usually registered during cold winter conditions and that the maximum movement takes place during hot summer conditions. Having generated the regression curve and entering on the abscissa with the month of April, it is observed that the corresponding vertical movement of the slabs would be around 0.11 in. This value is very similar to the values recorded in April 2001, which were around 0.10 in.

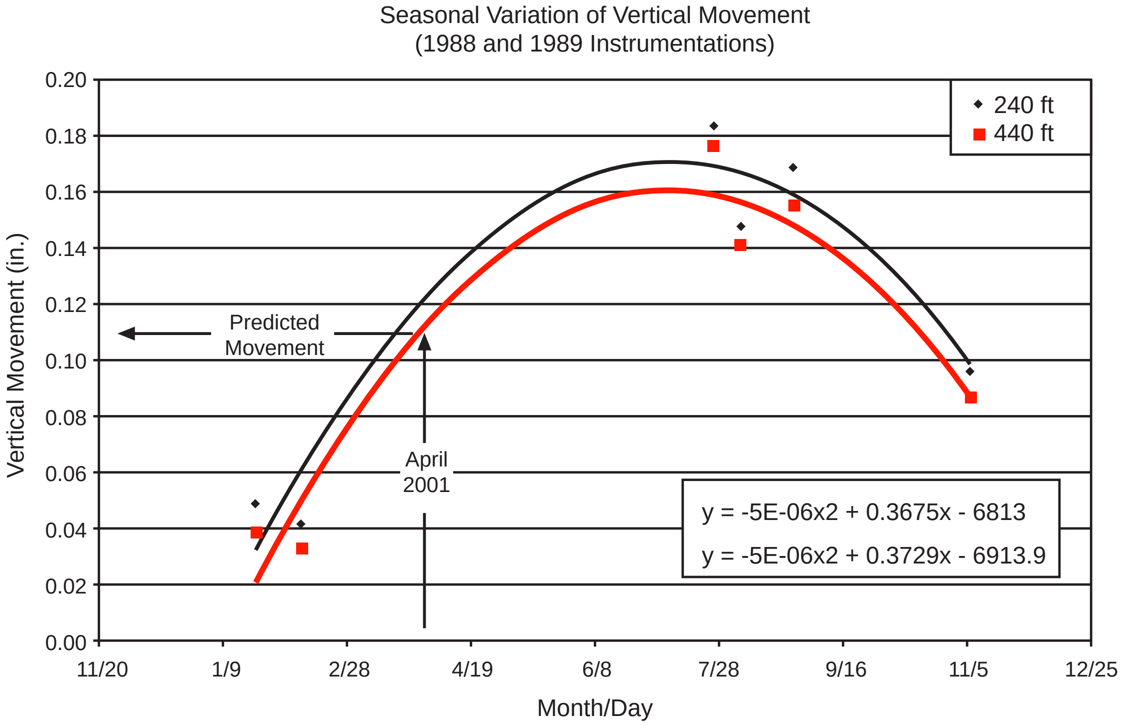


Figure 4.12 Vertical movement recorded in the 1980s

4.2.2.3 Summary of Joint Instrumentations

Measurements of the PCP section have been conducted on several occasions during the 1980s, and more recently, in April and June 2001. Horizontal and vertical movements recorded at all the measurements have been compared, and from the results it may be concluded that the PCP section is behaving as expected. Horizontal movement of the slabs was found to be within reasonable ranges, and thus the joints are still behaving as designed. Movement of the short slabs (240ft) was observed to be very similar throughout the life of the pavement. In contrast, the movement of the long slabs (440ft) was found to be greater in 2001 than in the 1980s. Despite the difference among movements, in all cases they fall within acceptable limits.

Curling of the PCP slabs was found to be reasonable, and no relation was found between length of slab and rate of vertical movement. Independent of the length of the slab, the overall vertical displacement was found to be no greater than 0.2 inches. This maximum rate of movement was registered during hot summer conditions. For cold winter conditions, curling of the slabs is not expected to be greater than 0.06 inches. In the last measurement conducted in April 2001, the vertical movement of the slabs varied from 0.02 in. to 0.10 in.

4.3 Structural Evaluation of Transverse Joints

This section describes the results of a series of deflection measurements that were taken using a falling weight deflectometer (FWD) to evaluate the structural condition of the PCP section, especially at the transverse joints. The transverse joints are designed to accommodate horizontal and vertical movements of the slabs and are expected to perform as designed throughout the entire life of the pavement. If they do not behave as expected, it means that the design did not meet expectations and thus could lead to failure of the pavement. Given the relatively wide joints of PCPs, load transfer ability might be a concern when designing the pavement.

The objective of this evaluation was to determine whether the PCP joints were still working appropriately by analyzing their load transfer efficiency, and to do that, a statistical analysis was performed using the deflection values provided by the FWD equipment. Although the RDD was also used for deflection measurement, its results were not used for any calculation in this report. Instead, the continuous deflection spectrum obtained from the RDD was used to find possible localized weak areas or points along the PCP, though no problems were found in this regard.

4.3.1 FWD DATA COLLECTION

FWD testing equipment is widely used to estimate deflections in pavements. The deflection values obtained from the FWD are usually post-processed to estimate other characteristics of the pavement. The FWD equipment used for the measurements in McLennan County is shown in Figure 4.13. The image illustrates the trailer hitched to the back of a van and displays the loading cell and sensors that measure deflections at various predefined points.

Figure 4.13 FWD equipment used for deflection measurement

Deflection sensors in the FWD can be arranged in different positions before and after the load. Figure 4.14 displays the configuration of sensors used for the measurements. Sensor 1 is located at the loading plate, and the rest of the sensors are located 12 in. apart from each other. The distance between the outermost sensors is 72 in. (6 ft).

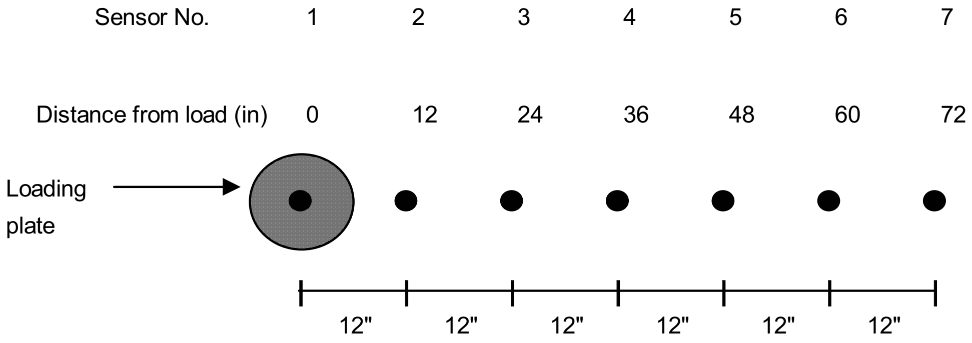


Figure 4.14 Arrangement of sensors in the FWD equipment

The PCP section is approximately one mile long and was built using a total of 32 slabs. Of these 32 slabs, 16 slabs are 240 ft long and 16 slabs are 440 ft long, and they all have a constant thickness of 6 in. To conduct the experiment, FWD deflection measurements were taken at various predefined locations on the 32 concrete slabs. Figure 4.15 displays a layout of the PCP section with slab divisions and locations where deflections were measured.

As shown in Figure 4.15, three testing points were selected on each slab: one point at the north end (start), one point at the center of the slab (middle span), and one point at the south end of the slab (end). This testing plan had two goals: first, to compare deflection values at both sides of each joint and evaluate the joint transfer efficiency; and second, to compare deflections near the joints to deflections at the mid-span of the slabs to determine if any difference was found in the load carrying capability of those two locations. The raw deflection data obtained from the FWD including load and deflection values measured for the outside lane of the PCP, are contained in Appendix B.

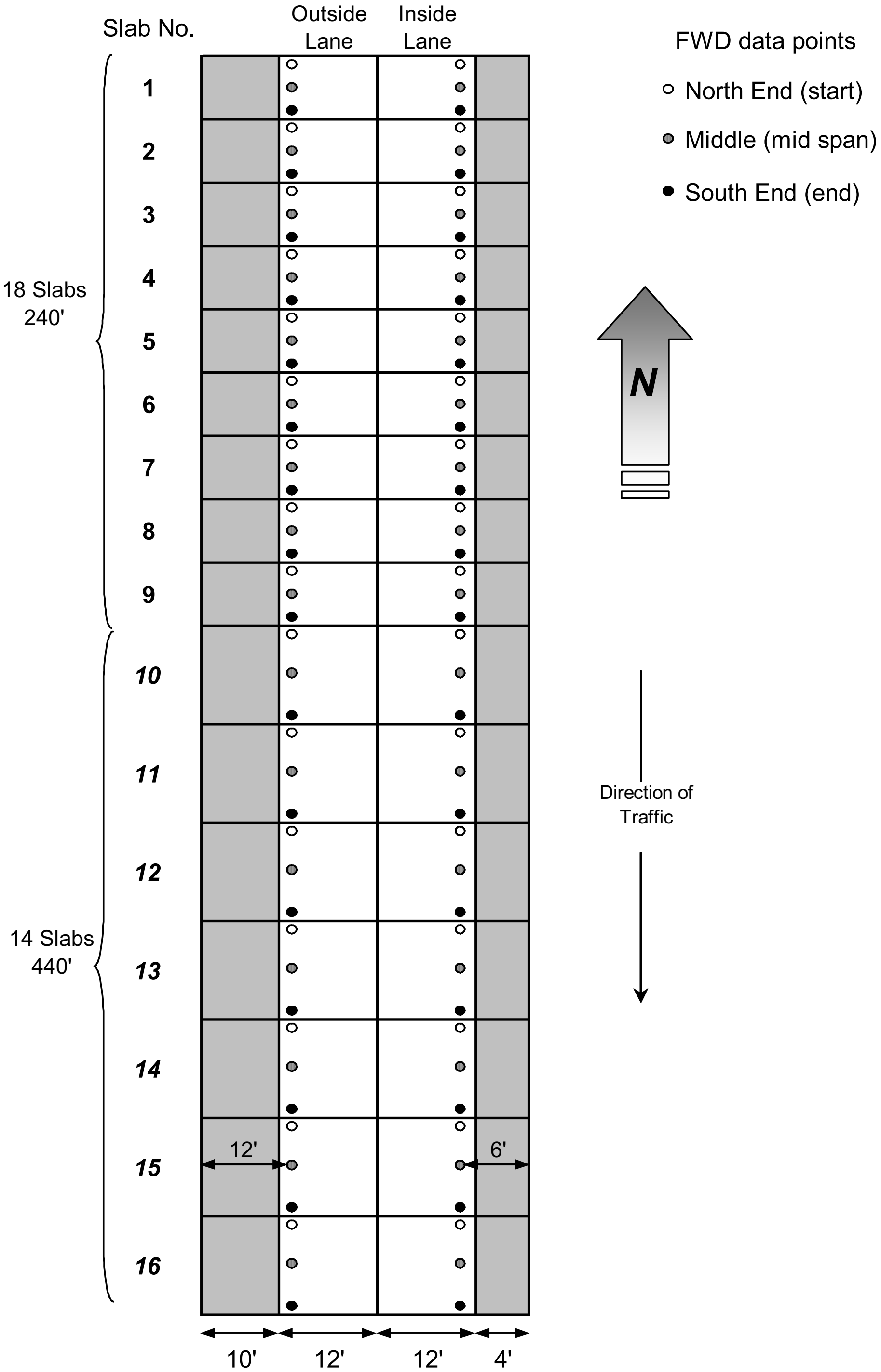


Figure 4.15 Points where FWD data were collected

4.3.2 ANALYSIS OF FWD DATA

The first step in the analysis of the deflection data was a visual examination of the deflection bowls generated by the FWD readings for all the testing points. This is an essential step in the analysis process because erroneous data can be discarded before they are included in a further step in the analysis. To easily observe the data, the information was post-processed using Excel spreadsheets for two different groups having different slab lengths, one group for 240 ft long slabs and another group for 440 ft long slabs.

The FWD dropped four loads at each point, increasing the load magnitude by approximately 2 kips each time. Next, the load used for the analysis was the normalized average of the four loads; the resultant was approximately 11.5 kips. Using this load magnitude, deflection bowls were plotted for the seven sensors of the FWD and were visually inspected to define whether any data did not meet test of logic and thus should be removed. FWD data were removed when the following conditions were met: first, when a point could not be compared to the rest of the points in the PCP section because of a free end condition; second, when the deflection obtained from the FWD was erroneous or inconsistent; and third, when a point was located in a transition zone (e.g., located near Joint 10 of the PCP). The list of points removed and not considered for further deflection analysis and the reason for removal are shown in Table 4.1.

Table 4.1 FWD measuring points removed for deflection analysis

Slab No.	Point Removed	Reason for Removal
1	North End (start)	Free end condition
1	Middle (mid-span)	Defective value point
4	Middle (mid-span)	Defective value point
9	Middle (mid-span)	Defective value point
9	South End (end)	240~440ft transition joint
10	North End (start)	240~440ft transition joint
13	North End (start)	Defective value point
16	South End (end)	Free end condition

4.3.3 STATISTICAL ANALYSIS OF DEFLECTIONS

Once the deflection data were cleaned and biased or erroneous points were removed, the second step in the analysis procedure was to conduct an appropriate statistical analysis to determine whether the load transfer capability of the joints was diminished by the use of the facility. The most suitable statistical technique used in this case was a two-tail t-test, which compares two samples of data at a time.

Using a t-test analysis technique allows the researcher to determine whether there is not a significant difference between the two means of the samples (null hypothesis or H_0). In this case, it was necessary to determine whether the difference between deflection means of two samples of data was not significantly different. In other words, the means of two samples of deflections were compared to each other using the t-test technique, and the difference of those means was judged to be significant or not. A detailed description of the steps followed in the t-test analysis is contained in Appendix C.

Deflection samples were organized so that four cases were analyzed. Deflections were compared at both sides of the joints to check joint load transfer, and then deflections near the joints were compared to deflections at mid-spans of the slabs to determine if load-carrying capacity was different between those two locations of the slabs. Table 4.2 shows how the samples were organized for the analysis: Cases 1 and 2 compare short slabs, and Cases 3 and 4 compare long slabs.

Table 4.2 Deflection cases analyzed using the t-test technique

Case No.	Slab Length (ft)	Sample 1	Sample 2
1	240	North End (start)	South End (end)
2	240	North End (start)	Mid-Span
3	440	North End (start)	South End (end)
4	440	North End (start)	Mid-Span

4.3.3.1 Results of Statistical Analysis and Interpretation

The results from the t-test are summarized in Table 4.3. Likewise, the detailed analysis of the four cases is contained in Appendix C.

Table 4.3 Summarized results of the t-test for Cases 1 to 4

Case No.	Computed t-value	Table t-value	Reject/Do Not Reject Null Hypothesis (H_O)
1	-0.510	2.140	Do not reject
2	1.260	2.140	Do not reject
3	-0.004	2.262	Do not reject
4	-0.090	2.228	Do not reject

According to the results from the four cases shown in Table 4.3, none of the null hypotheses (H_0) was rejected. This means, according to the fundamentals of this statistical technique, that there was not a significant difference between sample means in each case. In other words, the means of the paired samples in the four cases are not significantly different from each other, which means that for Cases 1 and 3 the load transfer capability of the joints is very good and is nearly 100% efficient. For Cases 2 and 4 it means that the load carrying capacity at the joints is not significantly different from the capacity at the mid-span of the slabs, which in turn indicates that the structural capacity of the PCP slabs is the same along the slab, including the joints. For practical purposes, deflections at both sides of the joints are expected to be the same. Therefore, the mean deflection for each joint was calculated using the deflection values obtained at both sides of that joint. Final deflection bowls are shown in Figures 4.16 to 4.19. In Figures 4.16 and 4.17 the prefix J stands for joint; for instance, J12 is the deflection value at Joint 12 and J16 is the deflection at Joint 16. In Figures 4.18 and 4.19 the prefix S stands for mid-span of slab, S2 being the deflection at the mid-span of Slab 2 and S16 the deflection at the mid-span of Slab 16.

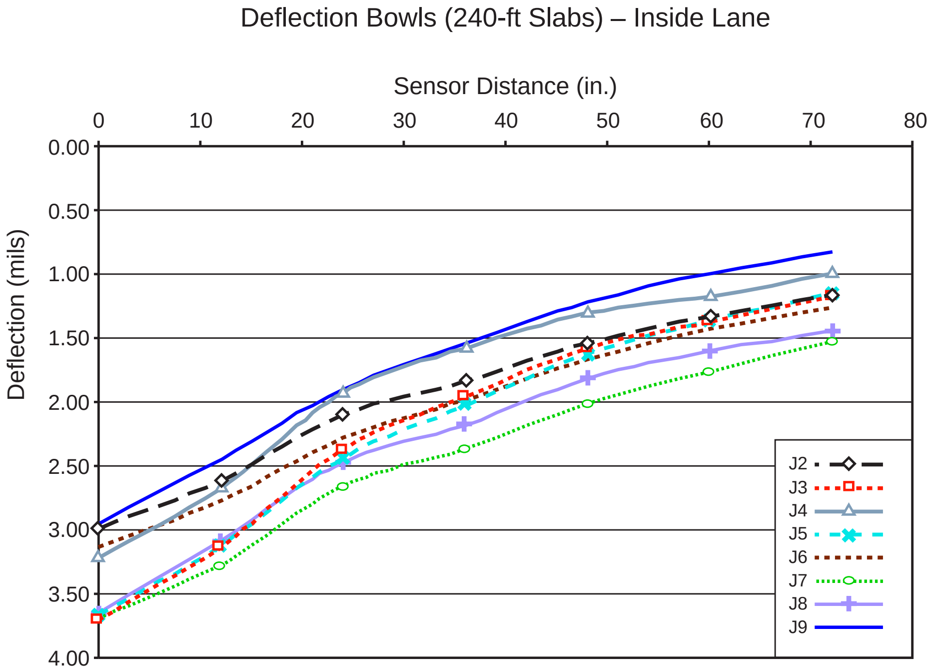


Figure 4.16 Deflection bowls at the joints of short slabs (240ft)

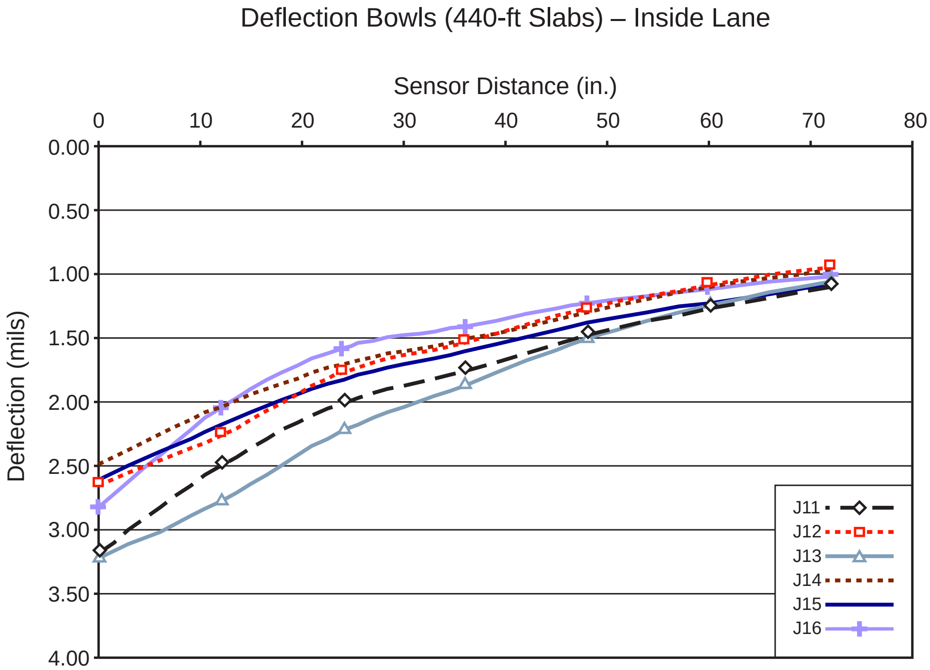


Figure 4.17 Deflection bowls at the joints of long slabs (440ft)

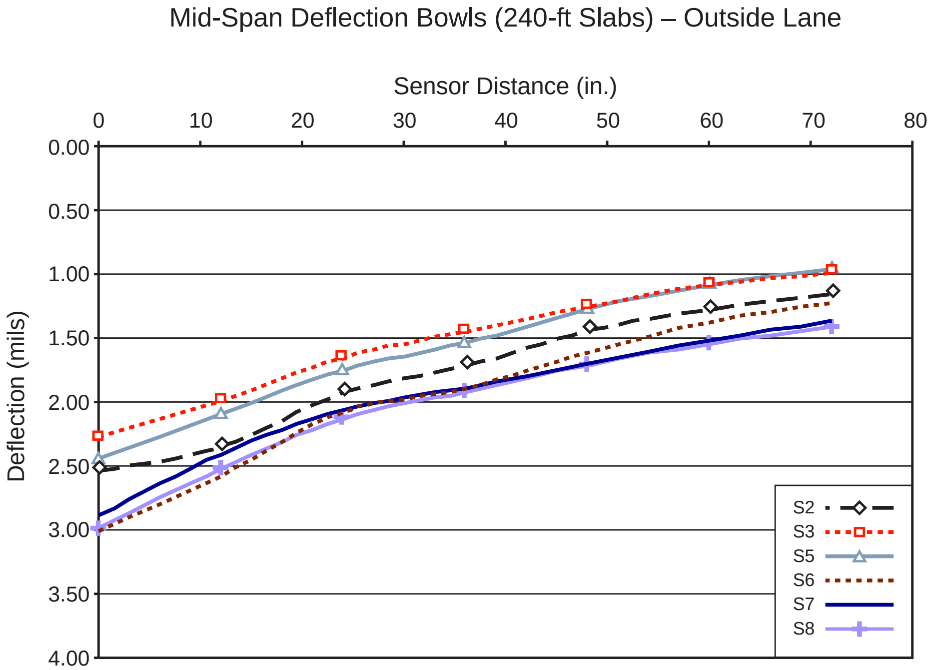


Figure 4.18 Deflection bowls at the mid-span of short slabs (240ft)

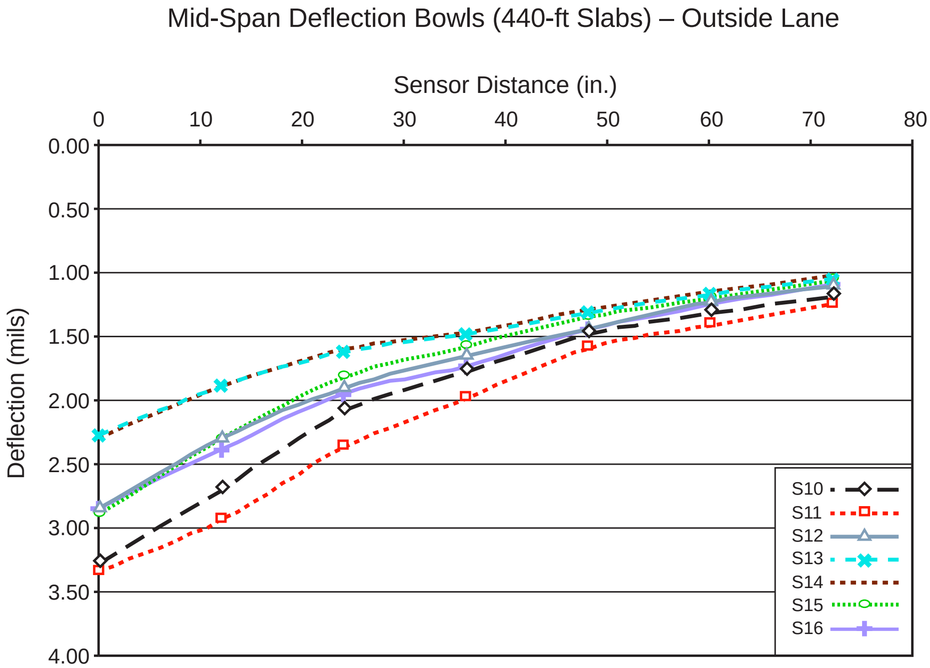


Figure 4.19 Deflection bowls at the mid-span of long slabs (440ft)

4.3.3.2 Load Transfer Efficiency at the Joints

Nondestructive testing (NDT) equipment such as FWD is commonly used to derive the load carrying capacity and load transfer ability of joints or cracks in pavements. Although deflection data are easily available from the electronic files generated by the FWD, it is always necessary to visually inspect the data before going further in any analysis that uses these data. Given the nature of the FWD test and the various factors that have an influence on its results, it is not uncommon to find erroneous deflection values that have to be manually removed to avoid biased analysis results. Fortunately, for the data collected for the PCP section, only a few deflection points were considered defective and thus were removed to perform the statistical analysis.

The t-test analysis performed showed that there is no significant difference between deflections measured at either side of the joints. This implies two things: first, that the load transfer ability of the joints is very good, and second, that all the joints are still performing as they were originally designed and built 17 years ago. In general, it can be stated that the PCP section is in very good structural condition.

4.4 Back-Calculation of Elastic Properties of PCP

There are different methods to determine the properties of the layers of a pavement structure. These methods are broadly divided in two groups: destructive testing (e.g., trenches and extraction of cores) and nondestructive testing (NDT) (e.g., falling weight deflectometer, ground penetrating radar, rolling dynamic deflectometer, etc.). Usually, NDT is preferred over destructive testing, as it is the FWD equipment most commonly used due to its practicality and because of the availability of software to back-calculate the properties of the layers of the pavement.

Load-deflection relationships obtained from the FWD are input variables used for the computation of the modulus of elasticity of each layer of the pavement. The software used for back-calculation of the PCP properties was EVERCALC Version 5.11 from the

Washington State Department of Transportation (WSDOT) (Ref 14), released in September 1999. It runs in Microsoft Windows and is user-friendly. This program is used by various highway agencies and has been cataloged as the best-performing program for back-calculation (Ref 15). The program assists with an optimum back-calculation of the modulus of elasticity of the pavement layers using FWD data to characterize the stiffness of the pavement. The input deflection values and their corresponding loads are used by EVERCALC, which uses an iterative linear-elastic approach that finds the set of pavement properties that minimizes the error between the measured and the calculated deflection bowls and provides the best fit.

4.4.1 ORGANIZATION OF INPUT DATA

In addition to the evaluation of the load transfer efficiency of the PCP joints and comparison of deflections at the joints and mid-span of slabs, the FWD deflection values were used for the estimation of the elastic properties of the PCP slabs and the pavement layers underneath the slabs. The estimation of the elastic properties of the PCP structure was done using EVERCALC, inputting the FWD raw data available and the cross-section of the structure, among other input variables. As previously done for the estimation of the load transfer efficiency of the joints, first the FWD data were visually analyzed, and erroneous data were removed for the analysis. The back-calculation of the PCP structure was done separately for each traffic lane. For the outside lane three points were removed, and thirteen points were used in the analysis. For the inside lane only one point was removed, and fifteen points were valid for the analysis. One point represents the deflection data at the mid-span of each slab.

For back-calculation purposes, only deflection values measured at the mid-span of the slabs were included in the analysis to comply with the assumptions of the theory of multilayer analysis and the software, which was designed to estimate elastic properties of a given pavement structure for interior loading conditions only. Corner or edge loading conditions are not valid for analysis in back-calculation procedures with EVERCALC or any other pavement back-calculation software that uses multilayer analysis models.

4.4.1.1 Generation of Files in EVERCALC

The software consists of three main sections. The first section, General Data, requires information about the setup of the FWD equipment. It asks for the number of sensors used for the analysis and the distances between them and the loading point. This section also requires the pavement structure characteristics to be input, such as the number of layers in the pavement (limited to five), hypothetical upper and lower limits of the moduli of the pavement layers, and their Poisson ratios. After inputting all the data, the program performs a number of calculation iterations and finally provides modulus values for all the layers in the pavement.

The second section of EVERCALC aids in the conversion of raw FWD data (*.fwd) to a new set of deflection data (*.DEF), which is a format that is used by the software to do the back-calculation. In this section of the program, the pavement layer thicknesses have to be entered. Then a decision has to be made about the desired number of drops per load (independent or averaged) to be considered in the calculation by the program. In this case, for the analysis of the PCP, the average of the four FWD drops was considered.

The third section of the program takes care of the back-calculation process. The user only needs to select the two previously generated files and name two more files, a summary file and an output file. The summary file contains the input data and computed values of modulus of elasticity for the pavement structure. The output file contains the input data, a detailed calculation of the modulus of elasticity of all the layers of the pavement, and the state of stresses under the applied load at predefined locations in the structure.

4.4.1.2 Input of Pavement Data

The input of the pavement parameters to the program is very simple, and the execution of the program until produces an output takes only a few seconds. The laborious part of the back-calculation is the process of selecting reasonable final modulus values for each layer. To accomplish that, input parameters have to be varied and run a number of times until the optimum combination of values is found. This phase is more complicated than just executing EVERCALC.

Each lane of the PCP section was analyzed separately, and input parameters were varied in order to obtain reasonable output results for each case. At least fifteen runs were made for each lane, but only the ones considered good approximations were considered feasible and are contained in Appendix D. The appendix presents the input variables for the best five runs in each case, inside and outside lanes. As an example of the input parameters that have to be varied with runs, Table 4.4 shows the set of inputs for Run No. 1 included in Appendix D. These inputs were used in both inside and outside lane back-calculations.

Table 4.4 Input parameters used for back-calculation of PCP (Run No. 1)

Layer No.	Description	Thickness (in.)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)
1	PCP layer	6	0.15	4,500	500	10,000
2	ACP layer	2	0.35	750	300	1,000
3	JCP layer	12	0.15	4,500	500	10,000
4	Granular base	5	0.35	100	10	500
5	Natural soil	N/A	0.40	30	10	250

4.4.1.3 Summary of Back-Calculated Modulus

The final modulus values as computed by EVERCALC are presented in Table 4.5. These values were the ones that better represented the characteristics of the PCP pavement.

Table 4.5 Final back-calculated modulus values for the PCP section

Layer No.	Description	Computed Modulus Outside Lane (ksi)	Computed Modulus Inside Lane (ksi)
1	PCP layer	6,276	5,395
2	ACP layer	263	207
3	JCP layer	2,668	1,282
4	Granular base	163	155
5	Natural soil	41	44

It can be observed from Table 4.5 that the back-calculated modulus for all the layers of the outside lane are higher than those computed for the inside lane. This output was expected because for the outside lane the distance from the loading point to the edge of the concrete slab was 12ft, and for the inside lane the loading distance was only 6 ft. This means that the loading conditions for the inside lane were more critical than those for the outside lane because as the distance between the edge of the pavement and the loading point decreases, the more critical the loading condition becomes, causing higher deflection and stress values. Ideally, if the distances from the edges of the lanes to the loading points were the same, the computed modulus values for both lanes would be very similar. Probably the outside lane would show lower modulus values than the inside lane because traffic loads are more critical in the outside lane, causing more deterioration of the pavement.

4.5 Summary

This chapter focused on the evaluation of the PCP section in different aspects. First, an analysis of the results of various condition surveys conducted during the 1980s and one conducted in 2001 showed that the PCP section is in outstanding condition, and only few localized distresses were found in the concrete slabs. It was observed that transverse joints contain debris that has prevented slabs to move freely, although this has not caused a negative impact on the performance of the joints. The restriction that debris imposes on the movement of the joints has not been critical.

Secondly, an evaluation of joints' performance was performed by analyzing the horizontal and vertical movements measured at the ends of the slabs. In both cases, the measured movements were within reasonable values, and thus, it was concluded that the transverse joints are still behaving as they were designed. Additionally, to complement this evaluation, a statistical analysis of the load transfer efficiency of the joints demonstrated that they are working in an excellent manner and transmit loads at nearly 100% efficiency.

Finally, the back-calculation of the pavement provided the strength of each layer in the pavement. The back-calculation showed that the PCP concrete slab, which is the most important layer of the pavement structure, is still in very good structural condition. The back-calculated modulus of elasticity of the concrete ranged between 5,000 and 6,000 ksi for the outside lane and its shoulder and the inside lane and its shoulder, respectively. All the results obtained in this evaluation of the PCP, from condition surveys to back-calculation of the pavement, represent a valuable tool in designing the new PCP section presented in Chapter 5, where discussions are presented for each design aspect, and decisions are made based upon the successes and failures of the PCP section already constructed in McLennan County.

5. DESIGN OF PCP

5.1 Introduction

The design of PCP can be conducted following the guidelines available in the literature, although a standardized design methodology is not available yet. To overcome this situation, pavement designers follow the existing guidelines that in the end provide valid and applicable results. This chapter describes a process for the design of any PCP, and to accomplish that, previous design experiences and steps, especially from the PCP section in McLennan County, are considered. Likewise, the described steps are applied to the design a of PCP section that will be constructed in Hillsboro, Texas. The design of this case study intends to serve as an example for designing any other PCP.

5.2 Design Considerations

The design of a PCP section requires a full understanding of a series of concepts that affect the quality of the pavement and its performance. The PCP design considerations will affect the pavement during its entire service life and therefore are described herein. The two basic design considerations are the factors affecting the design and design variables.

5.2.1 FACTORS AFFECTING DESIGN

In general, the factors that have the greatest influence on the design of a PCP are the same factors that affect the design conventional pavements. These factors include the effect of traffic loads, environmental conditions (e.g., ambient temperature and moisture), concrete temperature and temperature gradient, and friction resistance caused by the slab supporting layer. In addition to these factors, the design of PCP has to account for the effect of prestress and slab movement. All these factors must be considered in the design of PCP to ensure that the final product will meet the expectations of a high-performance concrete pavement.

5.2.1.1 Traffic Loads

Traffic loads generate compressive and tensile stresses at the top and bottom fibers of pavement slabs, respectively, as shown in Figure 5.1. Due to the relatively high resistance of concrete to compressive stresses, the critical stresses in a pavement slab are tensile stresses.

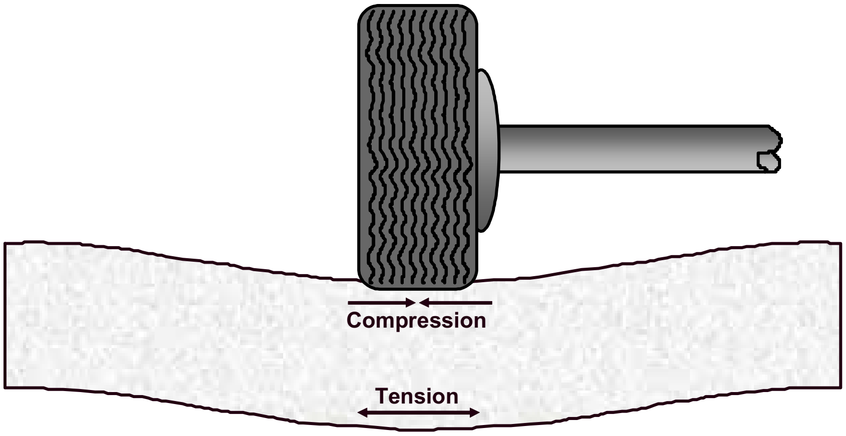


Figure 5.1 Slab stresses at top and bottom of slabs due to wheel loads

The magnitude of these tensile stresses depends on the magnitude of the applied wheel loads and the strength of the supporting layer underneath the slab or subbase. The determination of these stresses can be accomplished using different techniques. The finite element method (FEM) is the most sophisticated procedure that offers flexibility, permitting detailed modeling of material response and consideration of discontinuities such as cracks and joints. The method that is most widely used is the elastic layered analysis, which assumes that the pavement layers are homogeneous, isotropic linear elastic, and can be described by the two elastic constants, Young's modulus or modulus of elasticity (E) and Poisson's ratio (μ) (Ref 15).

The stresses computed using the elastic layered theory are interior stresses, away from the edges of the pavement. However, for PCP the most critical wheel load stresses occur at the edge of slabs due to the lack of support from contiguous concrete. To overcome this situation, stresses determined from the elastic layer theory have to be adjusted using a critical stress factor (CSF) (Ref 16). In the case of PCP with shoulders paved monolithically, the CSF applies only at the end of the slabs in the longitudinal direction, at expansion joints. Previous experience with the PCP in McLennan County shows that a reasonable CSF value of 1.3 might be used for PCP stress analysis near the expansion joints.

Determination of loading effects requires the computation of traffic loads, which are commonly converted to equivalent standard axle loads. Findings from the AASHO Road Test showed that the damage due to an axle of any mass (load) can be represented by a number of 18-kip equivalent single axle loads or ESALs (Ref 17). There is a variety of methods for forecasting ESALs, and all of them require detailed data of annual average daily traffic (AADT), percentage of trucks, and traffic growth rate.

5.2.1.2 Temperature Effects

The effects of temperature on concrete pavements are of particular interest for designers, especially in the case of PCP, where long slabs are constructed. As demonstrated in Chapter 4 in the evaluation of the PCP in McLennan County, temperature changes in the concrete mass cause horizontal movement and also curling of the slabs. Horizontal movement of the slabs is resisted by the slab itself and the friction that is created between the bottom of the slab and the subbase. This friction resistance between layers has to be considered in the design because it affects the magnitude of stresses developed in the bottom of the slab, prestress losses, and joint widths between PCP slabs.

Curling of the slabs is caused by several factors, such as the temperature differential between the top and bottom fibers of the slab, concrete compressive strength, modulus of elasticity, slab thickness, and joint spacing (Ref 13). The factor that has the greatest impact on curling at any point in time is the temperature gradient across the depth of the slab. In general, when the top fiber of the slab is warm, it tends to curl upward, but the ends of the slab tend to curl downward, simulating a simply supported beam at the ends and carrying an evenly distributed load. This load is in fact the weight of the slab, which tends to counteract the upward movement of the slab, creating tensile stresses to form in the bottom of the slab. Conversely, when a warming effect is generated on the bottom of the slab, such as the effect of a warm supporting layer underneath the slab, it tends to curl upward to let the central span of the slab support the entire weight of the slab. This in turn, counteracts the curling movement and causes the development of tensile stresses, but this time in the top of the slab.

Figure 5.2 illustrates these effects. The first condition is met during the warmest period of the day, usually in the afternoon. The second condition is met during the cooling period or cycle, which takes part during the night and extends to the early morning hours. During that time the supporting layer underneath the slab is warm due to the heat absorbed during the day, and the air on top of the slab is much cooler.

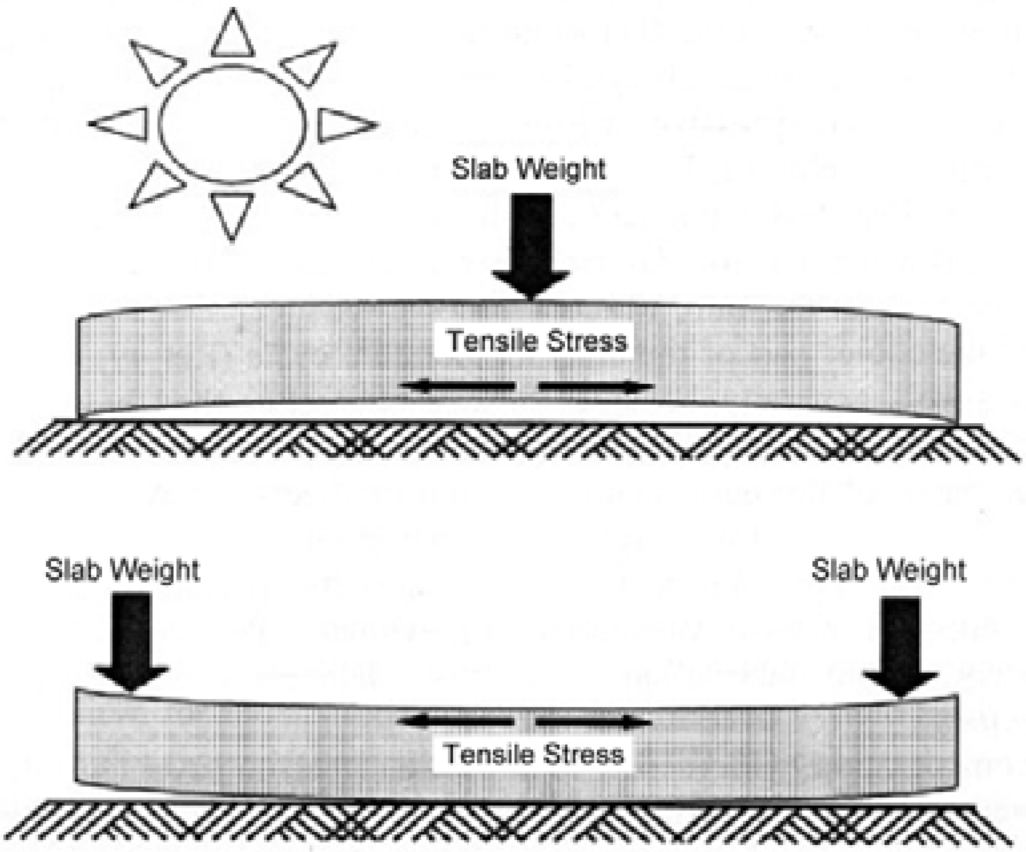


Figure 5.2 Effects of temperature on the curling movements of pavement slabs

Although curling is inevitable in concrete slabs, it has to be minimized to avoid the development of excessive tensile stresses, which in combination with wheel loads have a detrimental effect in the performance of the slabs. Therefore, an analysis of stresses resulting from seasonal and daily temperature cycles is required to ensure that their magnitude does not exceed limiting values. The most critical factors affecting the temperature gradient in the slab are ambient temperature at placement time, heat of hydration of the cement, and thermal conductivity of the pavement. These factors are of particular interest during hot weather concreting (summer placement) because these conditions cause more curling of the concrete slabs.

5.2.1.3 Moisture Effects

Similar to temperature gradients in pavements, moisture gradients cause warping stresses in pavements. During nonrainy conditions, moisture content in pavements increases gradually from the bottom to the top because moisture escapes from the top surface of the pavement. This difference in moisture across the depth of the pavement causes curling of the slabs. When this happens, tensile stresses develop in the top of the slab, and compressive stresses take place in the bottom fiber. This effect is beneficial at mid-depth of the PCP slab because compressive stresses in the bottom of the pavement tend to counteract tensile stresses due to friction between the slab and its subbase. However, this effect can be detrimental for the top fiber of the slab near the ends, where additional tensile stresses from thermal curling are present during the cooler part of the daily temperature cycle (Ref 18).

Another factor that has to be considered in the design and construction of PCP is shrinkage due to rapid moisture loss at an early age. The moisture loss causes shrinkage cracking, which can lead to the appearance of distresses and premature pavement failure. Using standard curing techniques will prevent the concrete from experiencing this negative impact.

5.2.1.4 Friction Resistance

As previously mentioned, daily and seasonal temperature changes cause expansion and contraction of pavements. These horizontal movements create a frictional resistance between the bottom of the slab and the top surface of the subbase. In concrete pavements this resistance is very significant and has to be quantified for design purposes. The frictional resistance depends on the coefficient of friction, the length of the slab, and the modulus of elasticity of the concrete. In long prestressed slabs this resistance is considered to be evenly distributed on the cross-section of the slab and can be broken into three components: bearing, adhesion, and shear; as shown in Figure 5.3. Bearing force is represented by the weight of the slab on the layer underneath. Its direction depends on the surface texture, moisture condition, and temperature. Adhesion is the component that results from the attraction of the slab to the support layer, and its magnitude depends also on the moisture and temperature conditions of the supporting layer. The shear constituent depends on the surface characteristics of the two layers in contact when movement begins. Its magnitude is proportional to the magnitude of the bearing component. If the combination of these forces is too high, so that it exceeds the strength of the supporting layer, this last one will fail (Ref 19).

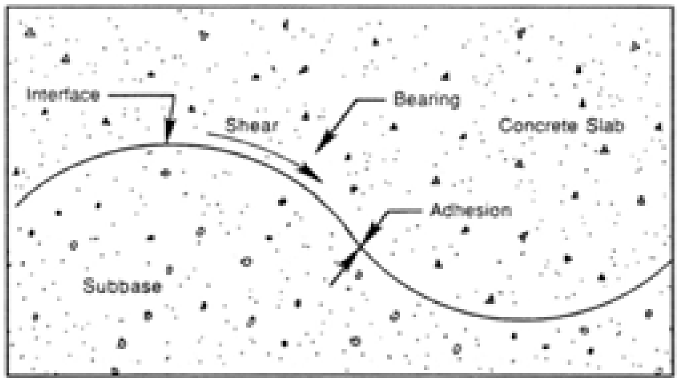


Figure 5.3 Components of friction resistance: bearing, adhesion, and shear

The relationship between slab movement and support layer resistance can be categorized in three ways (Ref 20):

1. Movements partially restrained by the support layer resistance: movements produced by daily temperature changes
2. Movements unrestrained by the support layer resistance: concrete swelling, shrinkage, and creep
3. Movements temporarily restrained by the support layer resistance: elastic shortening, which is diminished by the friction when the prestress force is applied, but which affects the full slab length shortly after prestressing

Because long-term movements from seasonal temperature changes occur at minute daily rates, as compared to daily temperature movements, they therefore take place without significant frictional resistance. However, frictional resistance to movements from daily temperature changes produces stresses in the slab. For instance, compressive stresses develop when the slab expands, and tensile stresses develop when the slab contracts. The latter situation is more critical, as these tensile stresses may be in addition to those tensile stresses caused by wheel loads and curling to such an extent that the slab may crack (Ref 20).

Movement of conventional concrete pavement slabs (e.g., JCP) caused by temperature variation has its maximum value at the end of slabs and decreases to zero movement at the center of the slab. Similarly, frictional resistance decreases from a maximum at the ends to zero at the center. Inversely and as a result of these stresses, tensile stresses (for slab contraction) increase from zero at the ends to a maximum at the center of the slab. This relationship is depicted in Figure 5.4(a).

In a PCP frictional resistance has another effect and causes a decrease in the amount of compressive stress transferred to the concrete from post-tensioning. This effect is illustrated in Figure 5.4(b). The reduction of post-tensioning force along the slab requires that a higher post-tensioning force be applied at the ends of the slab. To reduce the effect of frictional resistance, which causes tensile stresses in the pavement and reduces the amount of prestress transferred to the concrete during post-tensioning, a friction-reducing membrane is placed beneath the PCP slab to lower the coefficient of friction between the pavement slab and supporting base. The main considerations in selecting a friction-reducing medium are dependence on the efficiency in reducing restraint, practicability for road construction, and economics (Ref 21). Previous research studies and experience, including the PCP section built in McLennan County, indicate that a single layer of polyethylene sheeting is the most practical friction-reducing medium for meeting these requirements.

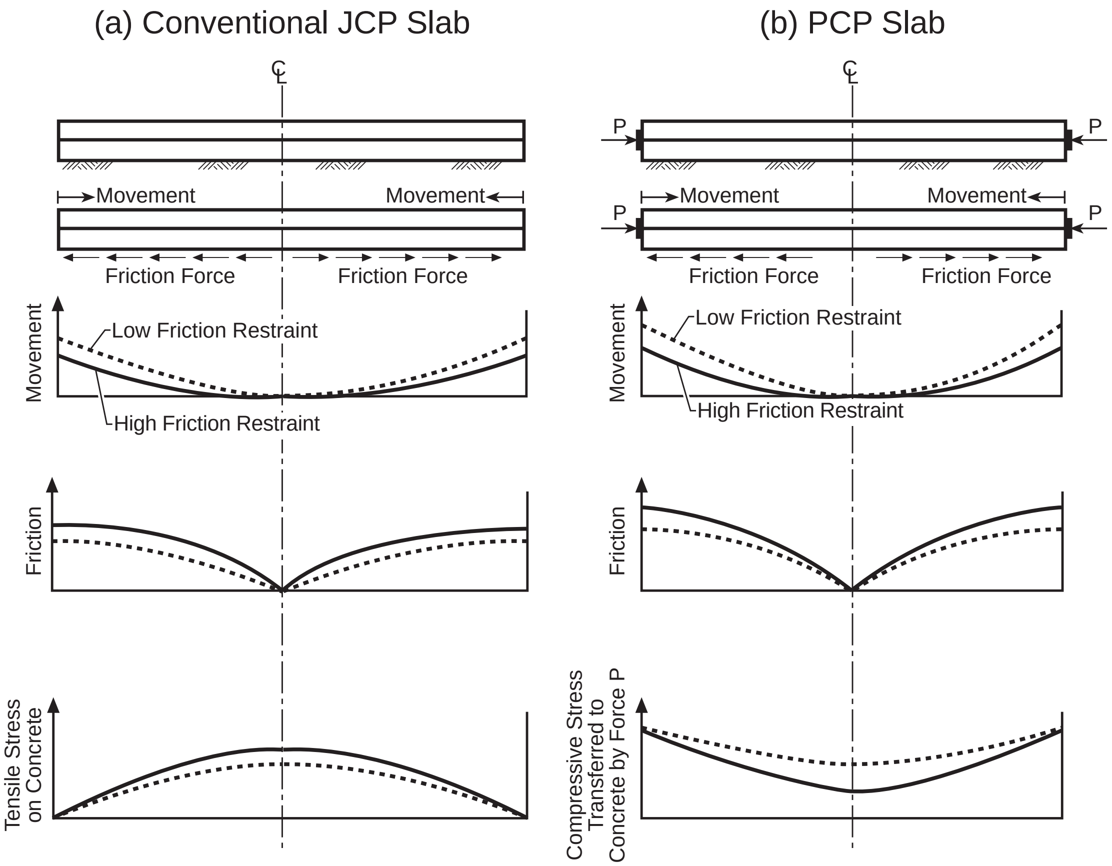


Figure 5.4 Effects of frictional restraint on (a) JCP slab and (b) PCP slab

5.2.1.5 Prestress Losses

Prestress losses are very critical for the design of PCP. Therefore they have to be estimated. Essentially, the strength of a PCP is dependent on the precompression applied to the concrete from prestressing. These losses have to be calculated to ensure that the required prestress level is maintained over the length of the slab and, most important, over the design life of the pavement. Estimates from previous experiences have shown that prestress losses can be from 15 to 20% of the applied prestress force in a PCP that is well-constructed and post-tensioned (Ref 5). The factors that contribute to these losses include:

- Elastic shortening of the concrete
- Shrinkage of the concrete
- Creep of the concrete
- Relaxation of the stressing tendons
- Slippage of the stressing tendons in the anchorage
- Friction between the stressing tendons and ducts
- Frictional resistance between the slab and base material

Extensive testing and experience in the PCP practice have produced methods to reliably quantify all these factors. Details of these methods can be found in the literature (Ref 5).

5.2.1.6 Transverse Prestress

The literature research presented in Chapter 2 showed that extensive investigations of the PCPs constructed in the United States, prior to the development of the PCP constructed in McLennan County, demonstrated that transverse prestress is an essential component of PCP. The absence of transverse prestress in this type of pavement resulted in extensive longitudinal cracking after exposure to traffic in most of the cases. The transverse prestress resists the applied wheel loads, preventing longitudinal pavement cracking and possible disconnection of separately placed pavement lanes or pavement strips. Therefore, it is critical that transverse prestress be incorporated in any PCP.

5.2.1.7 Joint Movement

Horizontal movement of the slabs is caused by expansion and contraction of the concrete due to daily and seasonal temperature cycles. This movement was clearly observed during the instrumentations of the PCP section in McLennan County, described in Chapter 4. It was concluded from the instrumentations that horizontal movements can reach significant values if concrete slabs are very long. The literature findings showed that the length of PCP slabs should not exceed 600ft to avoid excessive horizontal movement at the expansion joints. In general, the expansion joint width requirements usually govern the permissible slab length.

One critical aspect in the design and construction of PCPs is related to the expansion joint. It should be designed so that full closure never happens. Otherwise, it will cause crushing of the concrete near the joint. During construction, an initial joint opening must be provided to avoid this situation and also to allow seals to be accommodated. This initial joint opening varies, depending on the slab length and the season in which the pavement is constructed. Joint opening has to be evaluated for construction during the critical conditions of summer and winter. If the pavement is constructed during the spring or the fall, the expected movements will fall between the ones calculated for the other two critical conditions. In general, an expansion joint should never open more than 4 in.; this will ensure good riding quality and will prevent an excess of debris from getting inside the joint.

Horizontal movements are well-understood, and experience and research have provided accurate formulations for their prediction given various conditions. The conditions that influence the expansion and contraction of the concrete include the amount of prestress in the pavement, type of friction-reducing medium beneath the slab, coefficient of thermal expansion of the concrete, and the length of the slab. Calculation of these movements must be done prior to slab construction, assuming various conditions, so that the limiting joint width requirements are met when construction takes place.

5.2.2 DESIGN VARIABLES

Having revised the factors that affect the design of a PCP, it is now possible to describe the variables that must be considered in any PCP design. These variables include analysis of foundation strength and embankment properties, pavement thickness, magnitude of prestress, slab length, and slab width.

5.2.2.1 Foundation Strength and Embankment Properties

Experience has shown that foundation strength has a very significant effect on the performance of pavements. Although the relationship between the foundation strength and the performance of conventional concrete pavements has been thoroughly explored, PCPs still need to be researched in this field. The design of PCP assumes the same relationships associated with a conventional concrete pavement. Two of these relationships are as follows (Ref 5):

1. For a given load, the stress in a pavement is inversely proportional to the strength of the supporting foundation
2. The ability of the pavement to withstand repetitive loads is directly proportional to the strength of the supporting foundation

The first relationship means that, as the supporting foundation becomes weaker, the stresses generated in the pavement by wheel loads increase. In other words, if the supporting layer is weak, cracking and failure of a pavement will occur rapidly; in contrast, if the supporting layer is strong, the chances of cracking and failure of the pavement are much less likely to happen. The second relationship suggest that a pavement with a weaker supporting foundation will fatigue and eventually fail faster than a pavement with a stronger supporting foundation.

With regard to embankments, it is very important to verify the characteristics of the soils used for their construction. Problems have been found in embankments constructed with plastic soils with swelling potential. Expansion and contraction of the soil in the embankment will cause a rapid deterioration of the riding quality of the pavement and possible loss of support, which will ultimately affect the performance of the pavement. Therefore, it is essential to know the plasticity characteristics of the soils to be used in embankments and to understand their ability to swell and shrink at a given density, moisture, and loading condition when exposed to traffic loads.

5.2.2.2 Pavement Thickness

The design thickness for conventional concrete pavements is generally dependent on foundation strength, concrete strength, and the number and magnitude of wheel load repetitions. However, for PCPs there is greater flexibility with the pavement thickness selection. For every design there is a range of thicknesses that might be used, and in most cases it is possible to simply select a desired pavement thickness and adjust the amount of prestress in the pavement to meet the design criteria.

Prestressed slabs have considerably greater load-carrying capacity than indications based on flexural strength by the amount of prestress (Ref 3). The failure load is much higher than the load that produces the first crack at the bottom of the pavement. Furthermore, if the load that caused the bottom crack is removed, the prestress closes the crack, and the pavement regains most of its rigidity. This is the characteristic that gives PCPs a potential structural advantage over conventional concrete pavements, since they can carry traffic loading beyond the purely elastic range of normal concrete. The use of this concept can result in slab thicknesses considerably thinner than conventional pavements carrying the same loads.

The selection of the final thickness of the PCP slab is influenced by different factors, such as location of the pavement (urban street or highway), traffic loads (percentage of trucks), expected vertical movement (curling), and economics. The most important decision factor is the location of the pavement and expected traffic loads. A practical limit for PCP thickness seems to be a value not less than 50 to 60% of the thickness that would be used for urban street applications and not less than 60 to 65% of the thickness that would be used in a CRCP in major highways. Likewise, sufficient concrete cover has to be provided for all of the reinforcement and other hardware at the joint in the pavement. As the thickness is reduced, stresses should be evaluated in the lower layers of the pavement structure to ensure that they are at acceptable levels.

Thicknesses of less than 6 in. are generally not recommended, except for light traffic conditions.

5.2.2.3 Magnitude of Prestress

This variable refers to the prestress force applied to the pavement from pretensioning or post-tensioning of the concrete. The magnitude of prestress varies along the length of the pavement owing to prestress losses, as previously described in Section 5.2.1.5. In general, the magnitude of prestress must be such that the compressive stress at all points along the length and width of the pavement is greater than or equal to the minimum compressive stress required to meet the fatigue requirements over the life of the pavement. The fatigue requirements are a function of the number of load repetitions and foundation strength. These requirements will be discussed later in this chapter.

The compressive stress at any point along the length of the pavement can be expressed as a critical stress combination of the magnitude of the applied prestress, stress generated by applied wheel loads, curling stress resulting from temperature differential over the depth the slab, and friction stress caused by the supporting layer resistance. This combination of stresses is given by Equation 5.1 below (Ref 6):

$$\sigma_{CR} = \sigma_P + \sigma_W + \sigma_C + \sigma_F \tag{5.1}$$

where

σ_{CR} = critical stress combination, (+) = Tension, (-) = Compression

σ_P = effective prestress at the critical location

σ_W = stress generated by applied wheel load

σ_C = curling stress caused by temperature differential through the slab

σ_F = friction stress caused by slab-base interaction

Stresses are different in the top and bottom of the slab, requiring both to be analyzed. Curling stresses are assumed to be equal and opposite (tensile [+] versus compressive [-]) in the top and bottom of the slab. Stresses caused by wheel loads are assumed to be tensile (+) in the bottom of the slab and zero in the top. Friction between slab and subbase causes both tensile (+) and compressive (-) stresses, depending on the movement of the slab, and is assumed to be uniform over the pavement depth. Although these stresses vary along the length of the pavement, the only two points at which the stresses must be evaluated are at the ends of the slab and at midslab.

5.2.2.4 Slab Length

The length of the slab is governed by the expansion joint width requirements, as previously discussed in Section 5.2.1.7. As the length of the slab increases, the amount of expansion and contraction movement caused by temperature changes also increases, resulting in wider (or narrower) expansion joint widths. Among the factors to consider with regard to the length of slabs, economics and riding quality are probably the most important. As for economics, it is clearly understood that the longer the slabs, the fewer expansion joints will be required. Expansion joints are a significant cost component of PCPs. Likewise, as the slab length is increased, the more prestress is needed, and consequently, the prestressing cost is increased. Furthermore, as the slab length is increased, the expansion joint widths also increase, thereby affecting the riding quality of the pavement. Conversely, as the slab length is decreased, the number of expansion joints increases, again affecting both riding quality and economics. Therefore, a good balance has to be found between the economics and riding quality of the PCP by selecting an optimal slab length. If the project requires more than one slab length, stipulations should be made accordingly.

5.2.2.5 Slab Width

The slab width (or pavement strip width) is delimited by the exterior edges of the finished pavement, in transverse direction. Although slab width does not affect substantially the structural design of the PCP, it has a great impact on the constructability of the project. This slab width is controlled by various factors, such as location of the pavement, equipment limitations, and traffic management. The location of the pavement is important because it is necessary to know the number of pavement strips that will accommodate the total width of the pavement, which depends on the number of lanes and pavement shoulders. It is important to plan ahead the order in which strips will be constructed and then post-tensioned together.

The equipment limitation factor refers to the width of pavement strips that can be placed by the concrete paver. The same type of paver is used for the construction of conventional concrete pavements and for the construction of PCPs. One aspect that has to be considered is the fact that for wider pavement strips, fewer longitudinal joints are needed, although the number of strips is governed by traffic volume and available working space.

Traffic management deals with the necessary arrangements to accommodate traffic while construction takes place. For a pavement placed on a roadway that is near or over its capacity, it may only be possible to divert traffic off of one lane at a time for construction. In this case, the PCP would consist of multiple pavement strips.

Finally, special consideration must be given to the pavement shoulders. It is always desirable to construct a PCP in which the shoulders are built monolithically with traffic lanes. If it is constructed in this way, wheel loads will always be on the interior of the pavement, and the critical edge loading condition will not be an issue.

5.3 Implementation of Design Considerations

The previous sections in this chapter described a methodology that might be followed for designing a PCP. Design considerations and variables were explained in general terms so that they could be applied for the design of any PCP. Although this methodology provides the basic steps to be followed for the design, variations in the design procedure are indeed possible at designer's discretion because particular considerations have to be made for specific applications. With an understanding of this general procedure for the design of a PCP and all the variables involved, the subsequent sections of this chapter apply this methodology to the design of a new PCP section in Hillsboro, Texas, in Hill County.

5.3.1 DESIGN OF A NEW PCP IN HILLSBORO, TEXAS

Owing to the outstanding performance of the 17-year-old PCP section in McLennan County and based on a feasibility study, TxDOT officials decided to launch Research Project 0-4035, "Further Development of Post-Tensioned Prestressed Concrete Pavements in Texas." In general terms this consists of evaluating the existing PCP section in McLennan County and, based on the results, designing and constructing a new PCP that will achieve the following objectives:

1. Correct design and construction failures of the previous project
2. Use past and recent experiences in the field of PCP and apply them in the new design and construction
3. Improve design and construction practices of PCP
4. Evaluate the cost-effectiveness of PCP versus other paving options

These four objectives are fulfilled throughout this report. Objectives 1, 2, and 3 are covered in Chapters 5, 6, and 9; and Objective 4 is covered in Chapter 8.

5.3.1.1 Location of the New PCP Section

With regard to the selection of the site for construction, it was necessary to select an ideal location in Texas, so that the PCP section could be compared to a conventional PCCP control section. After looking at different options, TxDOT officials decided to develop the project on IH 35, in Hillsboro, Texas. This site was selected because, besides being a major interstate highway, other concrete pavement sections were recently built in the same area. Actually, the control section for the PCP is a 14 in. thick CRCP that was built just south of where the PCP will be located. Once the PCP section is constructed and finished, it will be under the same conditions of climate and traffic loading of the

CRCP, which is ideal for the achievement of Objective 4 of Research Project 0-4035, that is, to compare performances of the CRCP control section and the PCP.

The PCP section in Hillsboro will be 7.4 miles long in total, divided in two equally long roadbeds, northbound and southbound. Each roadbed will accommodate four traffic lanes plus inside and outside monolithically paved shoulders. The southern limit of the PCP is mile 364.4, right where the 5-mile 14 in. thick CRCP control section ends at its northern limit. The PCP extends 3.7 miles north on each roadbed, ending at mile 368.1. Another CRCP section is located just north of the northern limit of the PCP section. Figure 5.5 displays a map showing the limits and location of the PCP on IH 35.

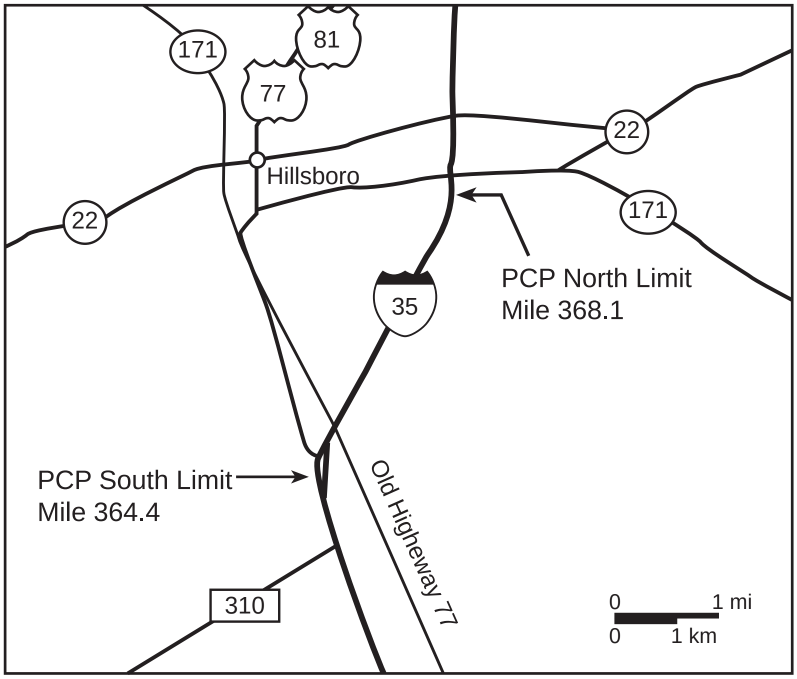


Figure 5.5 Location of new PCP section to be constructed in Hillsboro, Texas

5.3.2 DESIGN STEPS

The design procedure for the PCP section in Hillsboro, Texas, was conducted according to the steps below:

1. Embankment issues: This section gives a brief description of the materials used for construction of embankments. Chapter 6 contains a broad discussion of the required properties of soils forming these structures and gives guidelines of required plasticity limits, moisture content, compaction equipment, and possible lime stabilization.
2. Condition survey: The results from a condition survey for the existing pavement are presented herein. Using the data collected from the survey, an objective analysis of the distresses is performed by calculating a pavement distress index (PDI) (Ref 2), which assigns weight and severity factors to different distresses and combines them in an algorithm to obtain a number that indicates the degree of deterioration of the pavement surface.
3. Deflection measurement: Using the information collected during the condition survey and based on calculated PDI values, deflection measurements were conducted using FWD and RDD equipments, and the results are presented herein.
4. Back-calculation procedure: This step is fundamental for the design of the new PCP. As done with the PCP in McLennan County, described in Chapter 4, the program EVERCALC was used to calculate the elastic properties of the layers of the pavement and the natural soil.
5. Traffic data analysis: This is a very important step that provides traffic information for the thickness design of the PCP. Traffic data in form of annual average daily traffic (AADT) are converted into ESALs and projected for the design period of the PCP.
6. Thickness design: Having analyzed traffic expectations for the design period and using the back-calculated elastic properties of the existing pavement, the thickness design of the PCP is performed using the multilayer analysis theory. Possible slab thicknesses and corresponding required prestresses are presented. Two different designs are presented:

one design applies to the overlay of the existing pavement structure, and the other design is for the PCP to be constructed in the adjacent median or right-of-way of the existing roadbeds, where new lanes will be constructed.

7. Slab length: This section presents various options with regard to the length of the PCP slabs. The selection of the final slab length depends primarily on two factors, riding quality and economics, as previously described in Section 5.2.2.4.

5.4 Application of Design Procedure

To design the PCP, the steps previously described are followed herein. Given the somewhat lengthy nature of the design process delineated here, the final results of the design are summarized at the end of this chapter.

5.4.1 EMBANKMENT ISSUES

An embankment is a volume of earthen material that is placed and compacted for the purpose of raising the grade of a roadway (or railway) above the level of the existing surrounding ground surface. Embankments are constructed using different types of materials, soils and coarse materials including aggregates, rock, crushed pavement materials, etc. Normally, the coarser the fill material, the closer it is placed to the base of the embankment to provide a firm foundation and also to facilitate drainage and prevent saturation. As the embankment rises from the ground, the quality of the embankment materials improves.

Because embankment properties influence pavements performance, Chapter 6 focuses on the analysis of soil properties required for highway application and provides recommendations that might be adopted to treat materials to build the embankments for the PCP section in Hillsboro, Texas. Issues related to soil properties, like plasticity index

(PI) values and how to minimize them to prevent excessive embankment movements, are discussed. The effects of compaction methodology and equipment used are also discussed. Likewise, the feasibility of using lime for soil stabilization purposes to improve the engineering properties of the existing clayey soils is proposed.

5.4.2 EVALUATION OF EXISTING PAVEMENT

At the present time the pavement on IH35 in Hillsboro, Texas, where the new PCP will be constructed, is an asphalt concrete pavement. The original pavement section was built in 1962 as part of the emerging interstate highway system. The evolution of the cross-section from initial construction to the present is described later in this chapter. The two highway roadbeds are divided by a variable width median along the 3.7 miles of pavement section.

5.4.2.1 Condition Surveys

Condition surveys were conducted for the two roadbeds using procedures recommended in the literature (Ref 22), dividing the total length of the section in various subsections of 0.2 miles each in most of the cases, or a fraction of that if necessary. Subsections for the northbound roadbed were numbered from 1 to 21, and for the southbound roadbed from 22 to 43. Surveys were conducted at a project level, meaning that the 43 subsections were observed along the shoulder, and distresses were recorded in detail in each case.

The results from the survey showed that the existing pavement was, depending on the location, in fair or good condition. Observed distresses include transverse cracking, longitudinal cracking, rutting, patching, and alligator cracking. Figure 5.6 displays the typical longitudinal crack that was found between the outside lane and its shoulder. Longitudinal cracks were present in most of the length of the pavement, on both roadbeds. Figure 5.7 shows a failed patched area that developed severe alligator cracking on the outside lane of the northbound roadbed.

To quantify the cumulative damage of the pavement due to all the distresses, it was necessary to rate the pavement in a distinctive manner that would aid in the location of potentially weak zones in the pavement. The tool that provided a reliable way to interpret the results obtained from the condition survey and assign comparable grades or degrees of deterioration was a pavement distress index (PDI).

Figure 5.6 Longitudinal crack found between the outside lane and shoulder

Figure 5.7 Failed patched area presenting alligator cracking

5.4.2.2 Pavement Distress Index (PDI)

A PDI is the mathematical combination of various distresses in pavements. Before the calculation of the PDI, each distress type must be assigned a weight factor and a severity factor. Next, an analytical algorithm converts the distresses from the condition survey to a single rational value that provides an indication of the pavement's damage and riding quality. Broad application of the PDI has found that potholes have the highest impact on the riding quality of a pavement, and longitudinal cracking has the lowest impact (Ref 2). This concept of the PDI has been adopted for pavement management purposes by many highway agencies. The general form of the PDI is given by Equation 5.2:

$$\mathrm{PDI} = 100 - \sum_{i=1}^{n}\sum_{j=1}^{m} D_i x S_{ij} x E_{ij} \qquad (5.2)$$

where

D_i = deducted points of the i^{th} type of distress

S_{ij} = weight of the j^{th} severity class of the i^{th} type of distress

E_{ij} = the extent (percentage of area or the occurrence frequency) of the j^{th} severity class of the i^{th} type of distress

n = number of distress types

m = number of severity classes

In Equation 5.2, the value of PDI ranges from 0 to 100, where an index value of 100 represents a pavement with no distress at all. If PDI is less than 0, a value of 0 is assigned. Using this PDI criterion, the 43 pavement subsections were analyzed and clustered in 6 different groups, depending on the level of pavement surface deterioration. For instance, the northbound section was divided in two groups, NB_1 (subsections 1 to 15) and NB_2 (subsections 16 to 21), and the southbound section was divided in four groups, SB_1 (subsections 22 to 25), SB_2 (subsections 26 to 33), and SB_3 (subsections 34 to 40), and SB_4 (subsections 41 to 43). Each group received a PDI score computed using Equation 5.2. Table 5.1 shows the computed PDI scores for each group.

Table 5.1 Computed PDI scores for the existing pavement in Hillsboro, Texas

Roadbed	**Group**	**Subsections**	**PDI**
Northbound	NB_1	1 to 15	77
	NB_2	16 to 21	97
Southbound	SB_1	22 to 25	91
	SB_2	26 to 33	86
	SB_3	33 to 40	62
	SB_4	41 to 43	98

As previously mentioned, the maximum PDI score is 100, meaning a flawless pavement. From Table 5.1 it can be seen that the most deteriorated area of the pavement was located in the southbound roadbed, specifically the area included in Group SB_3. The best-preserved part of the pavement was found also in the southbound roadbed, corresponding to the area included in Group SB_4, just south of Group SB_3.

Although the raw data collected from the condition surveys were very useful, they did not provide a distinctive way to differentiate between the conditions of two or more areas along the pavement. The PDI is an algorithm that allows pavements to be rated in an objective manner by assigning scores and permits differentiating between bad, fair, or good conditions of pavements. Additionally, the scores assigned with the PDI algorithm serve as a guide for pavement managers and designers in deciding if either NDT or any other destructive testing should be conducted in specific sections of the pavement. The score values limiting different conditions of the pavement might vary among agencies. For the particular case analyzed in Hillsboro, all the groups in Table 5.1 are in good condition, except for Group SB_3, which had a PDI score of 62, ranking this section of the pavement somewhere between poor to fair condition.

5.4.3 STRUCTURAL EVALUATION OF PAVEMENT

The visual evaluation of the pavement, conducted by condition surveys and by estimation of the PDI for various sections of the pavement, determined which areas were in fair and good conditions. Because the PDI only evaluates the pavement from a functional standpoint that reflects the riding quality and safety of a pavement, a supplemental structural evaluation of the pavement was necessary to determine its bearing capacity. This evaluation was conducted using equipment that provided load-deflection characteristics of the pavement. FWD and RDD are the most commonly used equipment that evaluate the structural soundness of a pavement and therefore were used in this investigation.

5.4.3.1 Deflection Data Collection Using the RDD

The RDD is a dynamic loading truck that provides continuous deflection data along the pavement. Although the information obtained from the RDD has considerable potential in pavement research, its main objective at the present time is to locate weak points or zones along the pavement. The locations of weak zones are identified by simply locating areas with high deflection values in a deflection profile. For the pavement in Hillsboro, Texas, the RDD was used in both roadbeds; only dangerous zones where traffic prevented the testing tasks were skipped. Figure 5.8 displays a rear view of the RDD truck.

Figure 5.8 Back view of the RDD truck

Figure 5.9 depicts the mechanical components of the RDD that apply the dynamic loading and measures the deflections of the pavement at various points. The RDD has two cells that measure the continuous load and four geophones that measure the deflections at four different locations.

Figure 5.9 RDD mechanical components: loading cells and geophones

Figure 5.10 displays the deflection profile for the southbound roadbed obtained from the RDD. The abscissa axis shows the number of points continuously measured, which in this case is almost 6,800. The ordinate axis displays the deflection measured at each point in thousandths of inch (mils). It can be seen that deflection values follow a regular pattern, except for the zone in which the peaks reach values above 12 to 15 mils. This zone corresponds to the area delimited by Group SB_3 in Table 5.1. In the zone for which low PDI scores were estimated, high deflections were measured with the RDD as well. This means that in some way, the condition of the surface of the pavement is directly related to high deflection values recorded with the RDD. For the northbound roadbed, deflection values were very regular, ranging between 3 and 8 mils, with no significant peak zones in the profile.

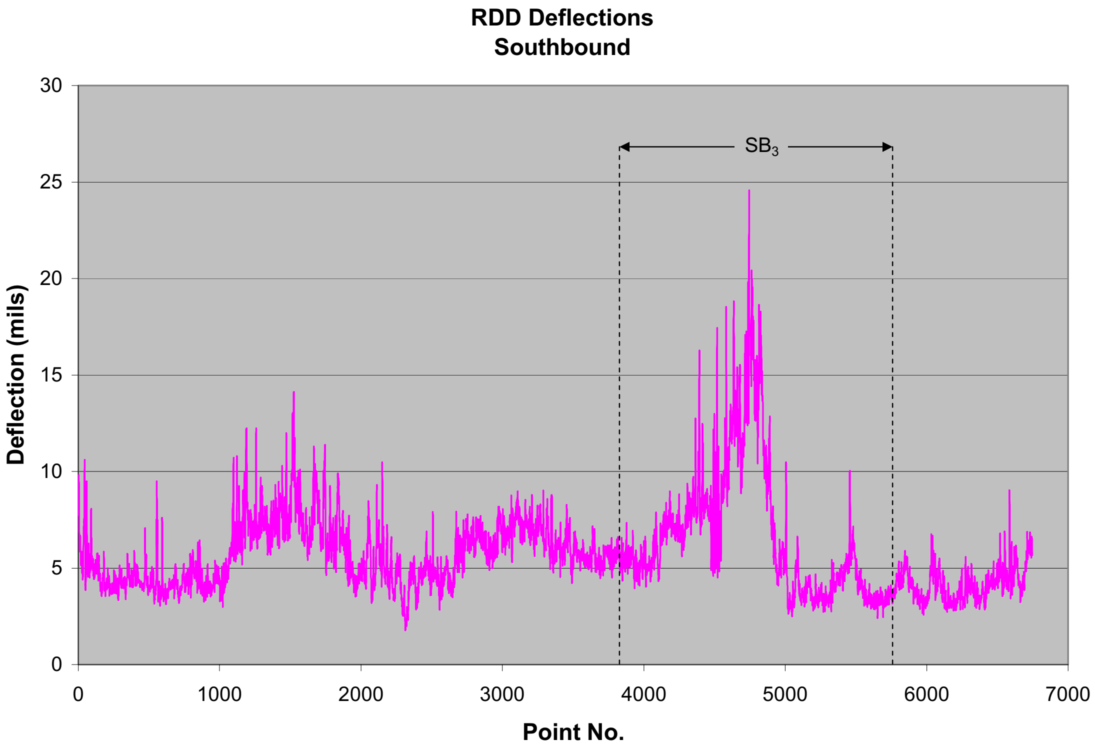


Figure 5.10 Deflection profile obtained with the RDD equipment

5.4.3.2 Deflection Data Collection Using the FWD

The equipment most widely used for deflection measurement is the FWD. This equipment has been used worldwide to evaluate the structural condition of pavements at both network and project levels. Unlike RDD, the data provided by the FWD are discrete and thus have a different interpretation. Both equipment types complement each other, but for the most part the information provided by the FWD has been considered more valuable for design purposes.

Figure 5.11 depicts an image of the FWD used for deflection measurements in Hillsboro, Texas. A full-size van equipped with a computer and a distance measuring instrument (DMI), among other gadgets, hauls the FWD equipment. For the testing conducted in Hillsboro, FWD deflections were measured at the same time RDD deflections were measured. A train of testing equipment, headed by the RDD and followed by the FWD and a crash truck, was used during measurements.

Figure 5.11 FWD equipment and hauling van

Although the FWD was used in most of the 3.7 miles of each roadbed, dangerous zones from a traffic sight distance standpoint were skipped. The deflection data obtained from the FWD were used for the estimation of the elastic properties of the existing pavement.

5.4.4 BACK-CALCULATION OF THE ELASTIC PROPERTIES OF THE EXISTING PAVEMENT

One of the most important steps in the design of an overlay is the estimation of the structural condition of the existing pavement. This estimation can be done by two ways. One method is by destructive testing, which consists of sampling the layers of the pavement and evaluating their strength characteristics. The other, and usually preferred, way to do it is by nondestructive testing (NDT). In this case, an evaluation of the load-deflection characteristics of the pavement is performed by analyzing the raw data provided by the FWD.

The first step in the back-calculation consists in characterizing the cross-section of the pavement. It is necessary to investigate the characteristics of each layer in the pavement system. Usually, highway agencies keep construction and rehabilitation records for the major highway networks. These records are very useful since they contain information about the layers of the pavement, such as materials and thicknesses.

5.4.4.1 Cross-Section Investigation

According to the information provided by the TxDOT Hillsboro Area Office, the pavement under study was originally constructed in January 1962. The traffic lanes' surface was an 8 in. thick CRCP pavement with a 2 in. thick ACP over 6 in. thick flexible base shoulders stabilized with 6% of portland cement. The CRCP was laid down on top of a 4 in. thick subbase course stabilized with 2.5% type A lime. Underneath the subbase a 6 in. thick stabilized subgrade was constructed on top of the natural soil. After 17 years of service, the CRCP had to be overlaid because roughness of the pavement became an issue due to highly active swelling clay in the area. Therefore, in 1977 a 6 in. thick hot mix asphalt concrete overlay was placed on top of the CRCP and shoulders to mitigate the roughness problem and to improve the structural capacity of the CRCP. It was not until 1990 that the pavement was again overlaid with a thin asphalt concrete layer. This time it was only 1 in. thick and was placed to cover reflection cracks and provide better skid resistance. Finally, since the placement of the last thin asphalt overlay, the pavement has been maintained by conducting several preventative maintenance activities, including routine crack sealing and patching. Although it was not documented, it is quite possible that an overlay of nearly 3 to 5 in. was placed between 1990 and 2001 on the north end of the section of the northbound roadbed. Figure 5.12 illustrates the evolution of the present pavement cross-section.

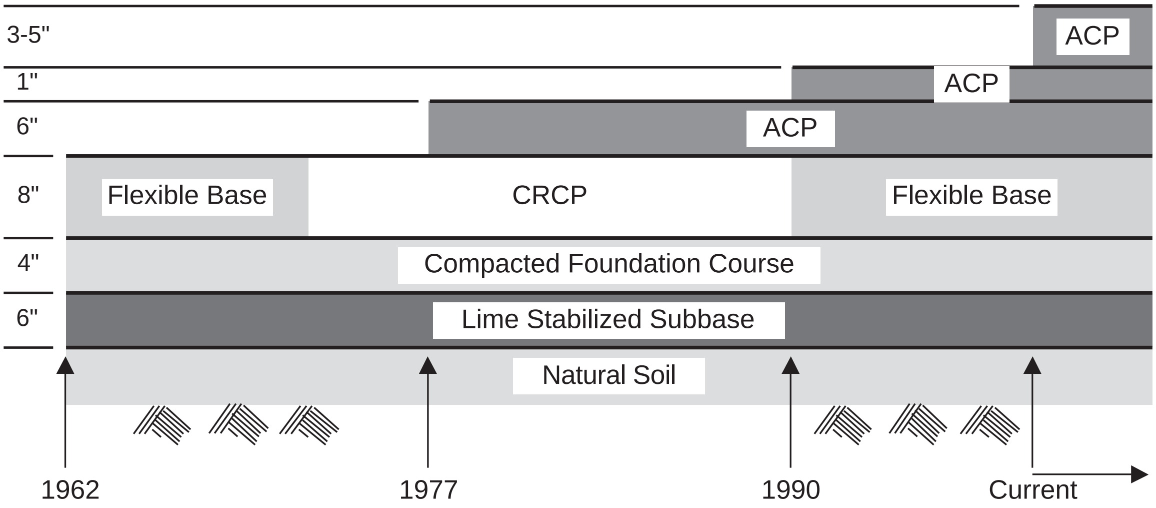


Figure 5.12 Cross-section showing the evolution of the existing pavement in Hillsboro, Texas

5.4.4.2 Elastic Properties of the Pavement

The elastic properties of the asphalt pavement were estimated using a back-calculation procedure, as was done for the existing PCP in McLennan County. The cross-section of the pavement and the raw load-deflection data obtained with the FWD equipment were used for this purpose. The deflection data were first visually examined before proceeding with in the back-calculation process, and erroneous data were discarded.

Having the information about the cross-section and the raw data from the FWD, the back-calculation of the pavement was done using the software EVERCALC. The same process followed for the PCP section was applied here, and the back-calculation was performed separately for the two roadbeds. For the northbound roadbed, the raw data from the FWD were input for the analysis, producing one set of representative values for the entire roadbed. For the southbound roadbed, two analyses were conducted: one for the FWD data for Group SB_3, for which the PDI score was the lowest and RDD deflections where the highest, and another analysis for the rest of the deflection data in the roadbed. Table 5.2 shows the back-calculated values for each layer of the pavement systems.

Table 5.2 Back-calculated values of modulus of elasticity

Layer Identification	**Northbound Back-Calculated Modulus (ksi)**	**Southbound Back-Calculated Modulus (ksi)**
Asphalt Concrete	760	681
CRCP	3,075	4,351
Subbase	268	194
Subgrade	134	109
Natural Soil	37	37

Table 5.2 shows the estimated values of the modulus of elasticity of each layer in the pavements. The values for the northbound section are representative of the whole roadbed. The values for the southbound correspond to the load-deflection relationships recorded for Group SB_3, which are more critical than those for the rest of the roadbed. These two sets of values of modulus of elasticity were considered for further analysis and design of the PCP.

5.4.5 TRAFFIC DATA ANALYSIS

In recent years traffic on IH 35 has grown at a drastic rate, especially in the last 8 to 10 years. This situation demanded an in-depth investigation of the traffic variable for the new PCP section design. Traffic information was obtained from TxDOT's Department for Traffic Planning and Programming (DTPP). The information provided by TxDOT included average daily traffic (ADT) from 1985 to 2000, percentage of truck traffic, and some other useful parameters. Table 5.3 summarizes the ADT values provided by TxDOT. The values shown are for the northbound roadbed only. The values for the southbound roadbed were similar to the ones shown here.

Table 5.3 ADT values from 1985 to 2000 on IH35 in Hillsboro, Texas

Fiscal Year	ADT Northbound
1985	11,500
1986	12,000
1987	13,000
1988	12,500
1989	13,355
1990	13,355
1991	13,355
1992	13,780
1993	15,825
1994	15,825
1995	15,825
1996	20,185
1997	20,185
1998	19,820
1999	21,120
2000	22,860

5.4.5.1 Traffic Projection

To estimate the average annual growth of traffic, the information contained in Table 5.3 was processed in a spreadsheet, and the results are shown in Figure 5.13. It can be seen in Figure 5.13 that the traffic from 1985 to 2000 increased at a rate of 4.6% annually, which seems to be a reasonable value. Traffic on IH 35 is estimated to grow between 4 and 8% yearly (Ref 23). Following the same growth rate, the projected ADT for a period of 30 years is 91,024, and for a period of 40 years it reaches a value of 142,716.

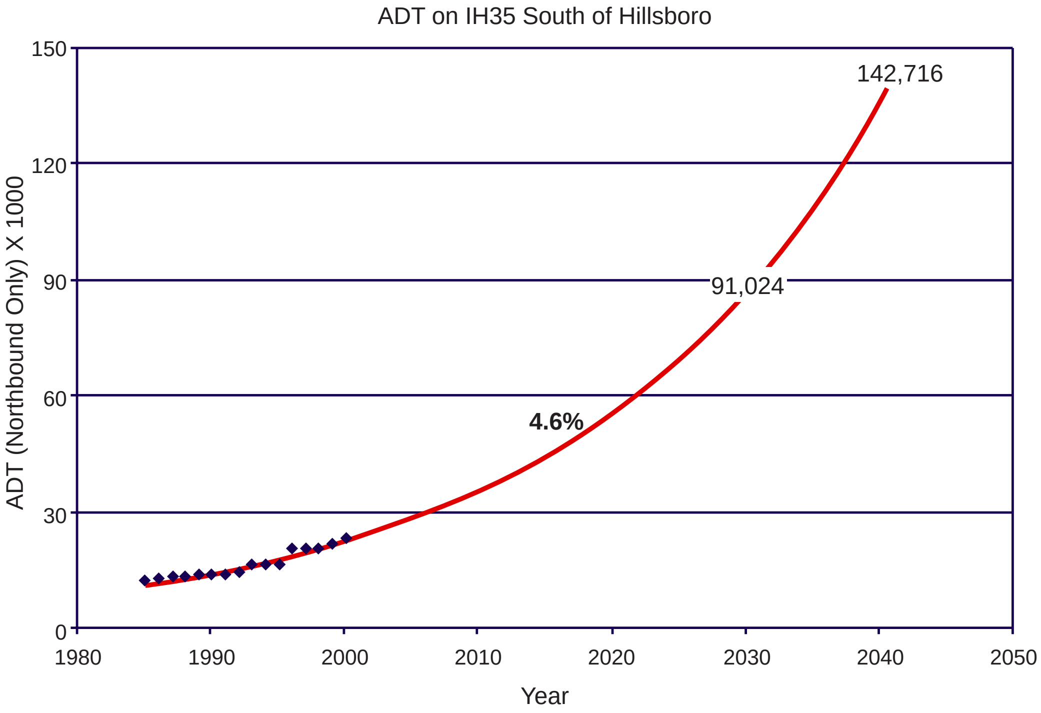


Figure 5.13 Observed and projected traffic on IH 35 in Hillsboro, Texas

5.4.5.2 Estimation of Equivalent Single Axle Loads (ESALs)

To estimate the number of 18 kip equivalent single axle loads (ESALs) for the design lane, four regression models developed in the 1980s at CTR under Research Project 472 were used (Ref 24). These four models allow for some flexibility, depending on the detail of the information available. The regression models were developed using traffic data from weight in motion (WIM) stations distributed across Texas.

The selection of the model to be used is basically governed by the amount of traffic information available. Model 1 requires considerable input information compared to Model 4, which only requires little input. The variables used in all the models are described in Table 5.4.

Table 5.4 Variables used for ESAL prediction models

Variable	Description
ADT	Average daily traffic
PTRUCK	Percentage trucks
YR	Year data collected (last two digits, e.g., 1987=87 and 2012=112)
HT	Highway type (IH or US)
PTAND	Percentage tandem axles
DIST	TxDOT[*] district
ATHWL	Average 10 heaviest wheel loads (lbs.)
ESAL2	Equivalent single axle loads
ADT85	1985 ADT
G	ADT growth rate

[*] Former State Department of Highways and Public Transportation (SDHPT)

For the computation of ESALs, the following input values were used:

- Percentage of single axle trucks = 5.5%
- Percentage of all trucks = 25.4%
- Average 10 heaviest wheel loads = 13,500 lbs.
- Percentage of tandem axle trucks in ATHWL = 50%
- Lane distribution factor = 0.65

The mathematical expressions derived for each model are as follows:

Model 1

This model provides the best fit of all the models, but it uses detailed truck information that is not always readily available.

ESAL2 = H1 + 225.02 (ATHWL) + H2(ADT) + 4.153(ADT) (PTRUCK) + H3 (YR) + H4 ln (ADT) + 2202.66 (PTAND) + 957460

where

H1 = -4986000 for Interstate Highway or 0 for US Highway
H2 = 7.0396 for Interstate Highway or 69.78 for US Highway
H3 = -24245.1 for Interstate Highway or 11072.2 for US Highway
H4 = -62238 for Interstate Highway or -579338 for US Highway

Model 2

This model provides a good fit, using the TxDOT district number as a surrogate predictor in place of the detailed truck variables used in Model 1.

ESAL2 = H1 + H2 + 12037(YR) + H3(ADT)+ H4+ H5(YR) + H6(YR) - 433658

where

H1 = -3499293 for Interstate Highway or 0 for US Highway
H5 = 44119 for Interstate Highway or 0 for US Highway
H2, H3, H4, H6 = functions of the TxDOT district number and road type

Model 3

This model provides an average fit and uses the 1985 ADT and G (linear growth rate).

ESAL2 = 46056(YR) + H1 + 1198183(G) + 477(ADT85) + H2(YR) - 9084(YR)(G) -1.3895(YR)(ADT85) + H3(G)+H4(ADT85) - 136(G)(ADT85) + (YR)(G)(ADT85) + H5(G)(ADT85) - 5966144

where

H1 = -1083536 for Interstate Highway or 0 for US Highway
H2 = 59554 for Interstate Highway or 0 for US Highway
H3 = -471910 for Interstate Highway or 0 for US Highway
H4 = -350 for Interstate Highway or 0 for US Highway
H5 = -61.16 for Interstate Highway or 0 for US Highway

Model 4

This model provides only a modest fit and utilizes the ADT of 1985 as predictor.

ESAL2 = 15640(YR) - H1 - 205.19(ADT85) + H2(YR) + 3.108(YR)(ADT85) -650.498

where

H1 = -2907059 for Interstate Highway or 0 for US Highway
H2 = 44723 for Interstate Highway or 0 for US Highway

The four models are used in the current study for the computation of projected ESALs for a design period of 30 years, and the obtained results are plotted in Figure 5.14.

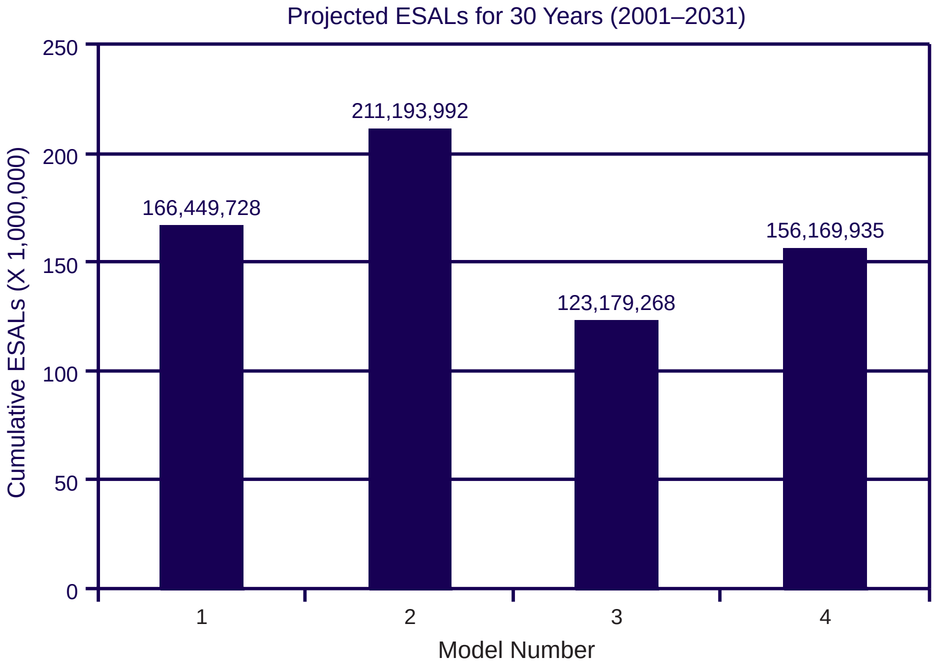


Figure 5.14 Projected ESALs for a design period of 30 years

Because Model 1 requires considerably more input data than the other 3 models, it predicts more reliable values. The projected values presented in Figure 5.14 are the cumulative ESALs for the 30-year design period and for the design lane.

According to an analysis conducted separately by TxDOT's Waco District Office, the projected number of ESALs for a period of 30 years (1999 to 2029) is 162,110,000 (Ref 25). However, the approach followed by TxDOT for the computation was not provided. As can be seen, this number is just below the predicted number of ESALs obtained from Model 1. Therefore, the number of ESALs for the design lane will be as follows:

30-year design period – 166 million 18 kip ESALs

5.4.6 ELASTIC DESIGN FOR FATIGUE LOADING

The thickness design for fatigue loading is a crucial part of the PCP thickness design process, where all the data already available are used with a single purpose, that is, to determine an appropriate PCP thickness that will withstand traffic conditions. Therefore, the cross-section of the existing pavement, the back-calculated elastic properties of the pavement, and projected traffic are used in combination with assumed design parameters for the estimation of the thickness of the PCP slab and its required prestress.

5.4.6.1 Equivalent Pavement

The basis for the design of the PCP relies on the design of an equivalent conventional pavement analyzed using the theory of elasticity for fatigue loading. This fatigue design accounts for the effect of accumulated load applications on the pavement over its design life. Following this criterion, a CRCP was selected as the control section because this type of pavement is commonly constructed on major interstate highways, and the section constructed just south of the PCP limits is a CRCP that ultimately will be compared to the PCP. Chapter 8 describes in greater detail the comparison of the PCP designed herein and the CRCP already in service on IH 35 in Hill County. The parameters used for the fatigue loading design of the equivalent CRCP were the following:

- Design life: 30 years
- ESAL applications: 166 million
- Concrete flexural strength: 700 psi
- Concrete modulus of elasticity: 4,000 ksi

The design life of 30 years was selected because that is a typical value used for applications in major highways and is the design period for which the CRCP located south of the PCP was designed. As previously estimated in Section 5.4.5.2, the projected ESAL repetitions for a period of 30 years are 166 million, considering a 4.6% annual growth rate. The concrete flexural design strength and modulus of elasticity correspond to typically observed 28-day values for concrete pavement design.

As part of the improvement plan for construction of the PCP section on IH 35, the current pavement section will be widened from two to four traffic lanes in each direction. This means that the existing pavement structures or roadbeds will be widened to either one side or both sides. At the time of writing this document, no definite plan has been defined by TxDOT. To cover this widening plan requirement, the fatigue loading design was made for two conditions: one design for the PCP overlay to be constructed over the existing structure and another design for the new PCP to be constructed either on the median or the right-of-way of the existing pavement or on both sides. For practical purposes, this last case will be called “the new pavement on the median” later in this report.

5.4.6.2 Fatigue Design Procedure

The first step for the fatigue design was to determine the thickness of a CRCP that will meet the requirements for each of the two conditions, one for the existing pavement and another for the median. In both cases the design was conducted using the elastic layered theory. For the design of the overlay of the existing pavement, first the back-calculated cross-sections of the two roadbeds were tested using identical hypothetical loading conditions. Next, a comparison of the deflections at mid-depths of the existing asphalt layer provided the weakest of the two structures from a fatigue standpoint. Finally, the weakest of the two structures was used for the design of the overlay for the two roadbeds. For the design of the pavement on the median, a pavement structure underneath the CRCP was proposed, and the thickness design was calculated accordingly. For both designs the program BISAR (Ref 26) and a spreadsheet that calculates the pavement thickness using the American Association of State Highway and Transportation Officials (AASHTO) procedure (Ref 27) were used very effectively.

5.4.6.3 Fatigue Design of PCP Overlay for Existing Pavement

The first step in the design was to determine which of the two roadbeds governed the design. To achieve this step, a multilayered elastic analysis was conducted using BISAR, inputting the same hypothetical loading conditions and the pavement elastic properties obtained from back-calculation as shown in Table 5.5. From the table it can be seen that the deflection values under the same loading conditions are very close to each other. The northbound roadbed experienced the most critical deflection, with a value of 0.00517 in. Although the asphalt layer is weaker in this roadbed, the diminished stiffness of the CRCP layer causes the upper asphalt layer to deflect more than the corresponding asphalt layer of the southbound roadbed. This basically means that the stiffness of the CRCP layer defines the most critical condition. Based upon this estimation of the deflection, the conditions of the northbound roadbed were used for the design of the overlay.

Table 5.5 Loading conditions and calculated deflection of existing pavement

Input Loading Conditions:	**Load (lbs.): 18,000** **Radius of Loaded Area (in.): 6**	
Layer Identification	**Northbound Back-Calculated Modulus (ksi)**	**Southbound Back-Calculated Modulus (ksi)**
Asphalt Concrete	760	681
CRCP	3,075	4,351
Subbase	268	194
Subgrade	134	109
Natural Soil	37	37
Deflection at Mid-Depth of Asphalt Concrete Layer	**0.00517 in.**	**0.00512 in.**

Following the AASHTO procedure (Ref 17) and using the spreadsheet for pavement design (Ref 27), the thickness of the CRCP overlay to withstand a 30-year design life with the given traffic conditions, existing pavement structure, and a reliability of 95%, was calculated. The result was a 13.5 in. thick CRCP.

5.4.6.4 Fatigue Design of New PCP on Median

The thickness design of the pavement to be constructed on the median was done in a similar way to the design of the overlay of the existing pavement. The AASHTO spreadsheet was used as an auxiliary tool. This time, the CRCP was assumed to be constructed over a 7 in. thick asphalt stabilized subbase with a modulus of elasticity of 700 ksi. The subgrade characteristics were assumed to be the same as those of the existing pavement, with an elasticity modulus of 37 ksi. This time the result of the analysis required a 14 in. thick CRCP.

5.4.6.5 Estimation of Slab Thickness and Required Prestress

Having estimated the thicknesses of the equivalent CRCP for the two described conditions, the next step was to obtain a range of PCP thicknesses equivalent to the CRCPs. This equivalency was obtained assuming that tensile stresses generated by loads in the bottom of the CRCP slab increase as the thickness of the CRCP slab decreases. To counteract this effect, compressive stresses must be induced in the concrete in the form of prestress. Applying this concept, as the thickness of the pavement decreases the tensile stresses increase, and consequently the compensating prestress has to be increased. Accordingly, the calculation of the PCP thickness was estimated on the principle that the tensile stresses developed at the bottom fiber of the CRCP must equal the tensile stresses developed at the bottom fiber of the PCP. In this manner, for the existing pavement the tensile stresses developed at the bottom of the 13.5 in. thick CRCP must be equal to the tensile stresses developed at the bottom of the PCP slab. For the pavement on the median, the same concept applies, but in this case the equivalent CRCP is 14 in. thick.

The calculation of the tensile stresses at the bottom of the pavement due to wheel loading was determined through the elastic layered theory, which considers the contribution of each layer of the pavement structure. The application this theory allows determining the stress, strain, and deflection for any point inside the pavement. For the computation of stresses, the theory assumes a loading condition and uses the elastic modulus (E) and Poisson's ratio (μ) of each layer and its thickness (D); additionally, the friction between layers is considered.

Figure 5.15 shows the characteristics of the pavement structure analyzed for the overlay of the existing asphalt concrete pavement. As previously mentioned, the AASHTO design required a 13.5 in. thick CRCP to be placed on top of the existing 10 in. thick asphalt concrete (ACP) layer. The pavement was loaded by a 20-kip single axle with effective tire pressure of 125 psi. The layers underneath the CRCP layer correspond to the existing cross-section of the actual pavement, and their characteristics are the ones estimated by back-calculation.

Similarly, Figure 5.16 shows the characteristics of the pavement structure analyzed for the pavement to be constructed on the median. In this case, the AASHTO design required a 14 in. thick CRCP placed on top of a 7 in. thick asphalt stabilized subbase. The subgrade characteristics were assumed to be similar to the subgrade of the existing pavement structure with regard to strength (modulus of elasticity) and deformability (Poisson's ratio).

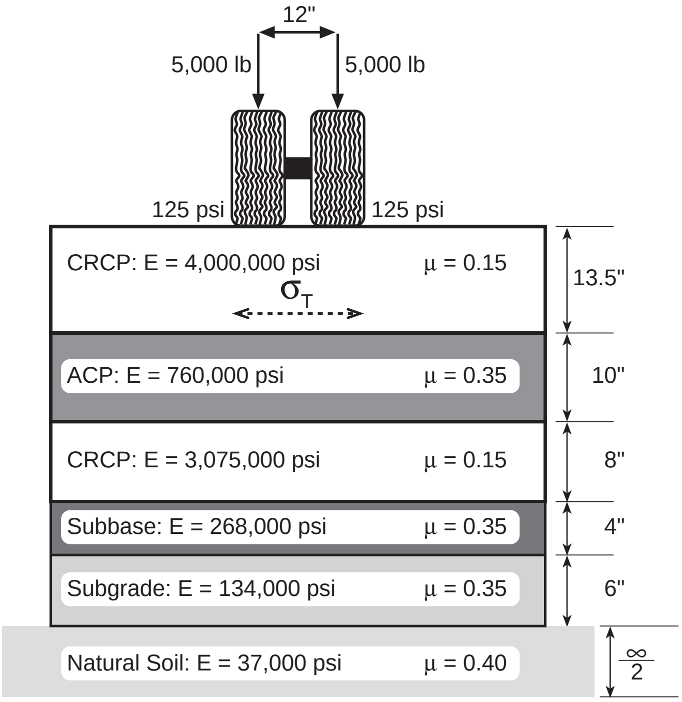


Figure 5.15 Structure analyzed for the design of the overlay of the existing pavement

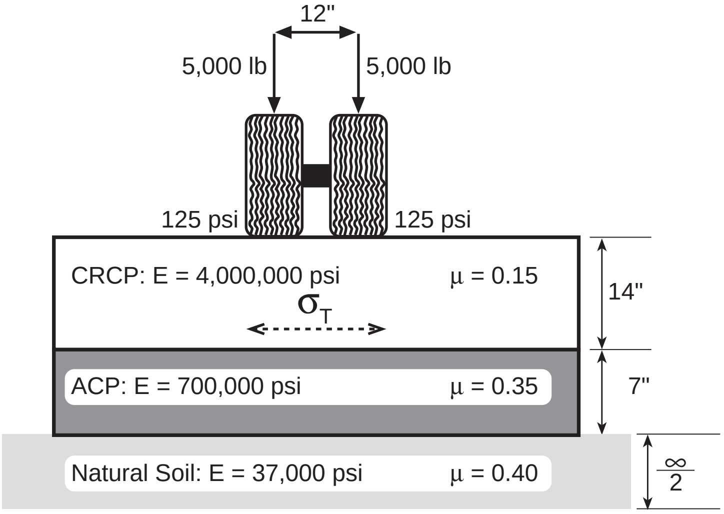
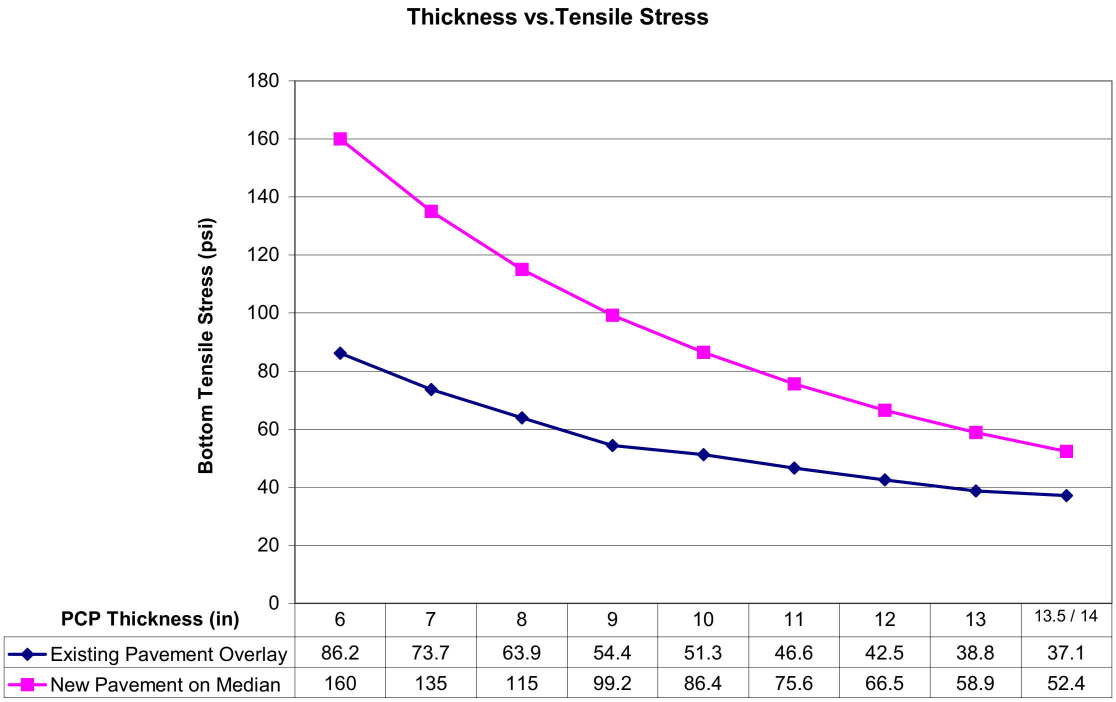


Figure 5.16 Structure analyzed for the design of the pavement on the median

The tensile stresses at the bottom of the equivalent CRCP slabs of the two pavement structures were determined using BISAR. Stresses were computed at three critical points: at the bottom fiber of the slab underneath each wheel load and at the midpoint between the two loads. In both analyses a frictionless condition was considered between the concrete slab and the supporting layer because that is a more critical condition than any other possible, where at least some friction occurs in the interface. For the layers underneath the concrete, a full friction condition was assumed.

BISAR was run a number of times for each analysis case, the overlay and the new pavement on the median. The CRCP thicknesses in Figures 5.15 and 5.16 were varied from 13.5- and 14 in. thick to 6 in. thick for a one-inch decrement at a time. As the thickness was reduced, the tensile stresses at the bottom of the concrete layer increased proportionally, as previously described. Figure 5.17 shows the variation of thickness and tensile stress at the bottom of the concrete layer for the two cases analyzed.



PCP Thickness (in)	6	7	8	9	10	11	12	13	13.5 / 14
Existing Pavement Overlay	86.2	73.7	63.9	54.4	51.3	46.6	42.5	38.8	37.1
New Pavement on Median	160	135	115	99.2	86.4	75.6	66.5	58.9	52.4

Figure 5.17 Variation of PCP slab thickness and tensile stresses at bottom fiber

The stresses shown in Figure 5.17 were used to determine the prestress requirements due to wheel loading. The estimation of this required prestress (σ_{PR}) was determined by subtracting the tensile stress of the thickest pavement from the tensile stress of a given thickness. Mathematically this can be expressed with Equations 5.3 and 5.4:

$$\sigma_{PR\ i} = \sigma_{T\ i} - \sigma_{T\ 13.5} \quad \text{for the existing pavement} \qquad (5.3)$$

$$\sigma_{PR\ i} = \sigma_{T\ i} - \sigma_{T\ 14} \quad \text{for the new pavement on the median} \qquad (5.4)$$

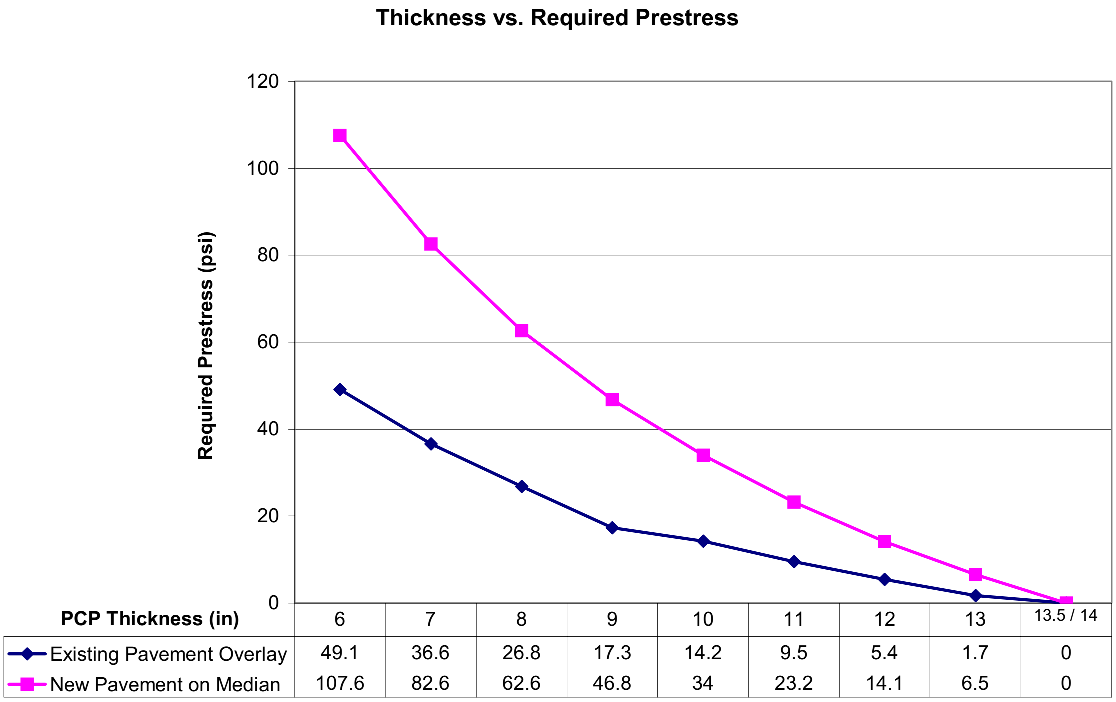
where

$\sigma_{PR\ i}$ = required prestress for the i thickness

$\sigma_{T\ i}$ = bottom tensile stress for the i thickness

$\sigma_{T\ 13.5}$ and $\sigma_{T\ 14}$ = bottom tensile stress for the thickest pavement i = pavement thickness, 6 ≤ i ≤13.5, 14

Following this approach, the required prestress (σ_{PR}) was estimated for the same analyzed thicknesses in Figure 5.17. The resulting prestress was plotted as shown in Figure 5.18.



PCP Thickness (in)	6	7	8	9	10	11	12	13	13.5 / 14
◆ Existing Pavement Overlay	49.1	36.6	26.8	17.3	14.2	9.5	5.4	1.7	0
■ New Pavement on Median	107.6	82.6	62.6	46.8	34	23.2	14.1	6.5	0

Figure 5.18 Required prestress (σ_{PR}) for various PCP thicknesses

The required prestress (σ_{PR}) shown in Figure 5.18 is the minimum compressive stress required at every point along the pavement to produce a PCP equivalent to that of the CRCP 13.5- and 14 in. thick for the overlay of the existing pavement structure and the new pavement on the median, respectively. The effective post-tensioning force to be applied to the PCP will be considerably higher than the calculated required prestress because prestress losses occur within the pavement, as previously described in Section 5.2.2.3 in this chapter.

5.4.7 ELASTIC DESIGN FOR ENVIRONMENTAL STRESSES AND WHEEL LOADS

The design of PCP requires the compliance with two criteria:

1. Critical tensile stresses at the critical location (bottom fiber of the PCP) must not cause a fatigue failure of the prestressed slab
2. The combination of wheel loads, temperature differential across the depth of the slab, and moisture stresses should never be greater than the combined concrete flexural strength and residual prestress to avoid cracking of the concrete

The first criterion is known as the fatigue loading design and deals with the estimation of the PCP pavement and its required prestress. This part of the design was just covered in Section 5.4.6. The second criterion is known as the elastic design for environmental stresses and wheel loads and will be covered in this section. The final or effective prestress applied to the PCP should meet both criteria. The design for environmental stresses and loading of the PCP can be mathematically expressed by Equation 5.5:

$$f + \sigma_p \geq \sigma_t + \sigma_c + \sigma_F \qquad (5.5)$$

where

f = allowable flexural stress in the concrete

σ_p = effective prestress at the critical location

σ_t = stress caused by applied wheel load

σ_c = curling stress due to temperature differential through the slab

σ_F = friction loss between slab and supporting layer

Equation 5.5 is a variation of Equation 5.1. The PCP slabs of the existing pavement (overlay) and the new pavement on the median will include prestressed concrete in their full width. This means that the shoulder areas will also be considered in the design as an integral part of the PCP slab. Therefore, the critical condition to be analyzed in the design is the bottom fiber of the PCP slab, where tensile stresses due to traffic, curling, and friction are additive.

5.4.7.1 Environmental and Wheel Load Stresses Design Equation Components

Each term in Equation 5.5 was calculated for various conditions, but a brief explanation of each term is given here. The allowable flexural stress in the concrete (f) is the design concrete flexural strength used in the estimation of the required prestress shown in Figure 5.18 affected by a safety factor (SF). The American Concrete Institute (ACI) Committee 325 Concrete Pavements (Ref 28) suggests a safety factor of two (SF = 2) for use in the design of primary highways. According to the ACI the 28-day design flexural strength of 700 psi assumed for design becomes an allowable flexural strength of 350 psi.

$$f = 350 \text{ psi}$$

The wheel load stresses at the bottom of the PCP slab for the two cases shown in Figures 5.15 and 5.16 were calculated using BISAR. The obtained values are shown in Figure 5.17 and correspond to the term σ_t in Equation 5.5.

The curling stresses predicted for the mid-length of the slab were computed using Equation 5.6 (Ref 29):

$$\sigma_C = \frac{E_c \cdot \alpha \cdot \Delta T_D}{2(1-\mu)} \qquad (5.6)$$

where

E_c = modulus of elasticity of the concrete

α = coefficient of thermal expansion (CTE) of the concrete

ΔT_D = temperature differential through the PCP slab depth

μ = Poisson’s ratio of concrete

Prestressing losses (σ_F) due to friction during post-tensioning of the slab were estimated using Equation 5.7 below (Ref 11):

$$\sigma_F = \frac{\mu_{\max} \cdot \gamma \cdot L}{288} \qquad (5.7)$$

where

$\mu_{\max}$ = maximum coefficient of friction

γ = concrete unit weight

L = slab length

5.4.7.2 Prestress Losses, Final Prestress, and Tendon Spacing

The longitudinally required prestress at the end of the slabs (f_{pe}) will be the governing or maximum value of the three following conditions:

1. Calculated prestress level (σ_P) from elastic design using Equation 5.5
2. The combination of the calculated fatigue prestress and friction stress ($\sigma_{PR} + \sigma_F$)
3. The value of the friction stress (σ_F) increased by 100 psi, to satisfy the frictional resistance criterion, which states that the required prestress level should exceed the maximum friction at any section of the pavement by at least 100 psi (Ref 28)

To obtain the final prestress requirements at the midslab (f_p), prestress losses were calculated for different conditions including the friction between post-tensioning tendons and their ducts, shrinkage and creep in the concrete, and steel relaxation (Ref 30). Friction restraint between the PCP slab and the supporting layer was already included in the design for environmental stresses and wheel loads.

The friction between the tendon and conduit (f_t) results from curvature and wooble. Curvature is due to intentional tendon profile variations and wooble to unintentional tendon profile variations. Tendon friction might be computed using ACI's recommendations with Equation 5.8:

$$f_t = f_{pe}(1 - \exp\{-[ux + (\frac{KL}{2})]\}) \qquad (5.8)$$

where

f_{pe} = governing end prestress, psi
u = curvature friction coefficient
x = angular change of tendon from jacking end to midslab, radians
K = wooble friction coefficient, per foot
L= slab length,ft.

For straight portions of pavements, as is the case for the PCP in Hillsboro, Texas, intentional angular changes are negligible. Therefore, Equation 5.8 is reduced to Equation 5.9:

$$f_t = f_{pe}\{1 - \exp[-(\frac{KL}{2})]\} \qquad (5.9)$$

Prestress losses due to concrete shrinkage (f_s) are given by Equation 5.10:

$$f_s = \varepsilon_s \cdot E_s(\frac{A_s}{A_c}) \qquad (5.10)$$

where

ε_s = concrete shrinkage strain
E_s = modulus of elasticity of tendon steel, psi
A_s = area of tendon per unit width of slab, in^2
A_c = area of slab per unit width of slab, in^2

Prestress losses due to creep (f_{cr}) are given by Equation (5.11):

$$f_{cr} = C_u (\frac{E_s}{E_c}) \cdot f_{pe} (\frac{A_s}{A_c}) \qquad (5.11)$$

where

C_u = ultimate concrete creep coefficient

E_c = modulus of elasticity of concrete, psi

Steel relaxation (f_r) losses are given by Equation 5.12:

$$f_r = \rho \cdot f_{pe} \qquad (5.12)$$

where

ρ = relaxation coefficient for the appropriate stress level

The final prestress at the midslab (f_p) is obtained by discounting all the previously described prestress losses from the governing prestress at the end of the slabs (f_{pe}). Mathematically, this is calculated using Equation 5.13:

$$f_p = f_{pe} - f_t - f_s - f_{cr} - f_r \qquad (5.13)$$

Finally, the spacing of tendons may be obtained by using Equation 5.14:

$$SS = \frac{f_p \cdot A_{tendon}}{f_{pe} \cdot D} \qquad (5.14)$$

where

SS = spacing of tendons, in.

A_{tendon} = cross-sectional area of tendon, in^2

D = thickness of slab, in.

5.4.7.3 Slab Length and Joint Movement

The length of the PCP slabs depends on different factors ultimately related to the riding quality of the pavement and economics. In addition to those two considerations for the selection of the length of the PCP slabs, some variables, including expected joint width, prestress force applied at the end of slabs, friction restraint of supporting layer, and required minimum prestress at midslab, influence the selection of the length of the PCP slabs. Nevertheless, the governing criterion for selecting the length of the slabs is the maximum expected joint movement. In any PCP the transverse joint should not open more than 4 in. under extreme conditions, usually during low winter temperatures. If transverse joints open more than 4 in., the riding quality of the pavement is affected, and the safety of users might be jeopardized.

For all these reasons, slab end movement is a critical aspect in the design of the PCP. Although slab movement is influenced by a series of variables, such as coefficient of friction between the slab and the supporting layer, concrete CTE, concrete shrinkage and creep, and daily and seasonal temperature variations, estimation can be done with reasonable accuracy. To estimate this slab end movement previous experiences with PCP in the United States and abroad have provided sufficient data, and a series of equations has been developed to estimate partial movements for various conditions.

There are a number of factors that have to be considered in calculating movement of the PCP slabs. Among these factors are the seasonal variation in length due to average seasonal temperature change, the daily variation in length, and the permanent shortening of the slabs. These factors are illustrated in Figure 5.19 (Ref 11):

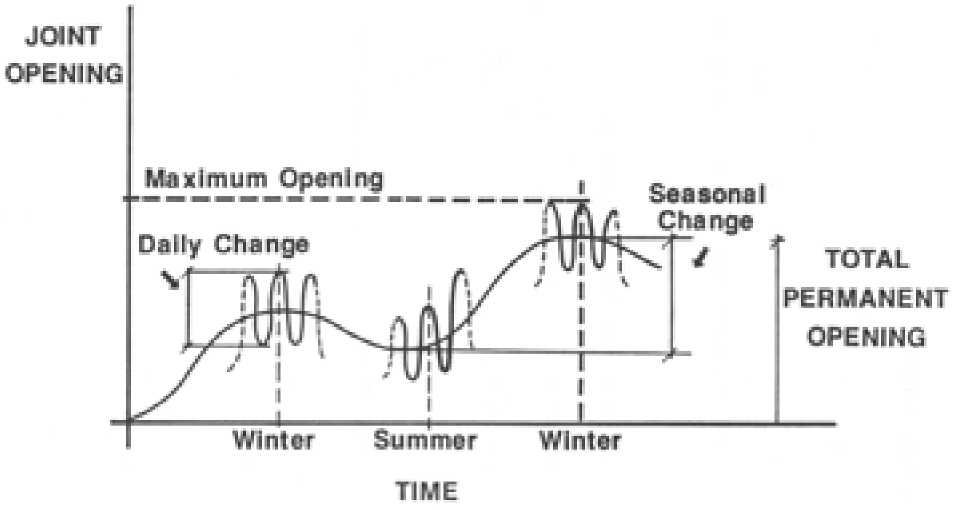


Figure 5.19 Joint opening due to seasonal, daily, and permanent length variation

The overall movement of the slab might be calculated by adding up the three effects shown in Figure 5.19. Temperature-associated movements are dependent on the concrete CTE and local temperature variations. These movements are affected by seasonal effects that develop over a long period of time and daily temperature effects that influence the friction underneath the PCP slab. The equations developed to estimate the movements are described below (Ref 31):

Seasonal movement (d_1) is given by Equation 5.15:

$$d_1 = \alpha \cdot \Delta_t \cdot L \tag{5.15}$$

where

α = concrete CTE, $1x10^{-6}$/°F

Δ_t = seasonal variation in average concrete temperature, °F

L = slab length,ft.

Similarly, the summer daily variation or slab movement associated with daily temperature variation during the summer season (d_2) is given by Equation 5.16:

$$d_2 = (\alpha \cdot \Delta_{ts} \cdot L) - d_f \qquad (5.16)$$

where

Δ_{ts} = summer maximum temperature minus summer average temperature, °F

d_f = slab movement restrained by friction, in.

The slab movement restrained by friction (d_f) is given by Equation 5.17:

$$d_f = (\frac{\sigma_f}{2E_c}) \cdot L \qquad (5.17)$$

where

σ_f = maximum friction stress underneath the PCP slab, psi

E_c = modulus of elasticity of the concrete, psi

The winter daily variation or maximum daily slab movement during the winter months (d_3) is given by Equation 5.18:

$$d_3 = (0.85\alpha \cdot \Delta_{tw} \cdot L) - d_f \qquad (5.18)$$

where

Δ_{tw} = winter average temperature minus winter minimum temperature, °F

Concrete shrinkage is caused by the amount of water in the concrete mix, the water-cement ratio (w/c), aggregate type, and curing conditions of the slab. For concrete prisms drying from all faces, long-term shrinkage strain varies from 100 to 500 millionths; for slabs drying only from the top, a value of 250 millionths is assumed. Approximately 40% of this strain takes place during the first month after construction; the remaining 60% is usually considered for future slab shortening. The slab shortening is given by Equation 5.19:

$$d_4 = \varepsilon_s \cdot L \qquad (5.19)$$

where

ε_s = concrete shrinkage strain, in./in.

Concrete creep affects the long-term shortening of the PCP slab subjected to sustained stress and is dependent on concrete aggregate gradation, aggregate type and shape, cement content, water-cement ratio, concrete density, curling, and element size. This slab shortening due to creep (d_5) is given by Equation 5.20:

$$d_5 = \varepsilon_k \cdot L \qquad (5.20)$$

where

ε_k = concrete creep strain, in./in.

The concrete creep strain can be calculated using Equation 5.21:

$$\varepsilon_k = C_u (\frac{f_{av}}{E_c}) \qquad (5.21)$$

where

C_u = ultimate creep coefficient, its value ranges from 0.5 to 3.0; a value of 2.5 is suggested for PCP creep associated shortening (Ref 31)

f_{av} = average prestress along the slab length, psi

The elastic shortening of the slab (d_6) is analogous to the shortening due to creep, but that assumes the length of the slab as given by Equation 5.22:

$$d_6 = (\frac{f_{av}}{E_c}) \cdot L \qquad (5.22)$$

Finally, the total movement of the slab is obtained by adding up movements d_1 to d_6. This movement is the one expected to occur for the total length of the slab. The movement at each end is about one-half of the total calculated movement.

5.4.7.4 Computation of Strand Spacing and Joint Movement Using a Spreadsheet

Once the prestress at the end of the slabs is computed, and prestress losses are estimated and discounted, the next step is to calculate the strand spacing for both longitudinal and transverse directions. Because there is not a unique strand spacing value, its estimation is done by performing an iterative process, where different variables are considered, and various strand spacings are possible. These variables that influence the strand spacing include geometric properties of the PCP slab, concrete properties such as aggregate type, CTE, Poisson's ratio, creep and shrinkage strains, flexural strength, modulus of elasticity, etc. Additionally, steel properties including strand sectional area, ultimate and yield strengths, and modulus of elasticity are variables that have a great impact on strand spacing.

Uncontrolled climate variables were also considered for the estimation of strand spacing and movement of the PCP slabs. Those variables, as previously discussed, include seasonal and daily temperature variations. Seasonal temperature variations account for the variability of hot temperatures during summer conditions and low temperatures during the winter. Daily temperature variation considers separately the effect of extreme high and low temperature gradients in the PCP slab. For daily summer conditions it considers the difference between the highest and average temperatures recorded for the season; for daily winter conditions it considers the difference between the average and the minimum temperatures recorded for the season.

In order to manage all these and other variables, the procedure followed herein was to input the variables in various feasible combinations in a series of computer spreadsheets. The variables that were combined for the analysis were placement season including summer and winter; and aggregate type, including siliceous river gravel (SRG) and limestone (LS). The slab lengths analyzed were 250, 300, and 350ft. For the transverse direction, slab widths of 25, 49, and 71ft were evaluated for two traffic lanes, four traffic lanes, and four traffic lanes plus shoulders conditions, respectively. For all those lengths and widths, PCP slab thicknesses of 8, 9, and 10 in. were proposed, and prestress at the end of the slabs, final prestress force at midslab, strand spacing, and slab movement were computed by the spreadsheet. A summary of all the results from the spreadsheet analysis is included in Tables 5.6 through 5.9, which present the obtained critical conditions for the final design. As can be seen from these tables, for the PCP overlay for the existing pavement and also for the new pavement on the median, the most critical conditions for both strand spacing and slab or joint movement occur during the summer season and when SRG is used. Additionally, from the spreadsheet analysis it can be seen that the friction between the PCP slab and its supporting layer is significant in the transverse direction, and thus this had to be considered in the calculations. Printouts of all the spreadsheets obtained from the analysis are included in Appendix E.

Table 5.6 Summary of results: longitudinal direction, PCP overlay of the existing pavement, summer season, SRG aggregate

Slab Length (ft)	Thickness (in.)	Prestress at End of Slab (psi)	Final Prestress Force at Midslab (lb.)	Strand Spacing (in.)	Total Slab Movement (in.)
250	8 9 10	229 265 308	36,992	20 16 12	2.503
300	8 9 10	252 288 331	35,975	18 14 11	3.003
350	8 9 10	275 311 354	34,984	16 13 10	3.504

Table 5.7 Summary results: longitudinal direction, new pavement on the median, summer season, SRG aggregate

Slab Length (ft)	Thickness (in.)	Prestress at End of Slab (psi)	Final Prestress Force at Midslab (lb.)	Strand Spacing (in.)	Total Slab Movement (in.)
250	8 9 10	296 323 354	36,992	16 13 10	2.547
300	8 9 10	319 346 377	35,975	14 12 10	3.056
350	8 9 10	342 369 400	34,984	13 11 9	3.566

Table 5.8 Summary of results: transverse direction, PCP overlay of the existing pavement, summer season, SRG aggregate

Slab Length (ft)	Thickness (in.)	Prestress at End of Slab (psi)	Final Prestress Force at Midslab (lb.)	Strand Spacing (ft)	Total Slab Movement (in.)
25	8 9 10	39 29 26	41,895	11 14 14	0.244
49	8 9 10	50 40 37	41,345	9 10 10	0.479
71	8 9 10	60 50 47	40,847	7 8 8	0.694

Table 5.9 Summary of results: transverse direction, new pavement on the median, summer season, SRG aggregate

Slab Length (ft)	Thickness (in.)	Prestress at End of Slab (psi)	Final Prestress Force at Midslab (lb.)	Strand Spacing (ft)	Total Slab Movement (in.)
25	8 9 10	75 59 46	41,895	6 7 8	0.247
49	8 9 10	86 70 57	41,345	5 6 6	0.484
71	8 9 10	96 80 67	40,847	4 5 5	0.701

Based on the results shown by the summary tables above, the combination of a 9 in. thick PCP slab with a 300-ft length seems to be a good selection for both the overlay of the existing pavement and the new pavement structure on the median. This combination allows reasonable strand spacings for the longitudinal and transverse directions for both paving conditions. Additionally, slab movements also fall within reasonable values. If 250 ft long slabs were constructed, then 20% more transverse joints would be required for the construction of the pavement, which has a direct negative impact on the construction cost, maintenance activities, and riding quality of the pavement. Alternatively, if 350 ft long slabs were built, expected horizontal movements would be around 3.5 in., which is half an inch greater than those expected for 300 ft long slabs. Additionally, strand spacing will be narrower for 350 ft long slabs than for 300 ft long slabs, which means that more tendons would be required to achieve the desired prestress level.

5.4.7.5 Prestressing Stages

As has been experienced in several other PCP projects in the United States and abroad, the post-tensioning of the concrete slab has to be done in at least two stages. Initial prestress is usually applied a few hours after the pavement has been placed to prevent cracking of the concrete that might result from its lack of plasticity to absorb movements. During this initial post-tensioning it is not necessary to overcome 100% of the tensile stresses acting on the slab; it is only necessary to apply a certain amount of prestress to maintain tensile stress below the tensile strength gained by the concrete at that time (Ref 11). The final prestress is applied once the concrete has gained sufficient strength. With regard to the time of initial post-tensioning and prestressing force, caution must be exerted not to cause failure of the anchor zone. Previous experience with the PCP in McLennan County showed that concrete placement time is a variable that has a significant influence on defining the time and force to be applied to the concrete. When placement of the concrete is done early in the day, slabs experience a temperature drop that results from the difference of higher day temperatures and lower night temperatures. Conversely, when concrete placement takes place late in the afternoon, slabs usually do not experience that abrupt afternoon-night temperature drop. The more the temperature drops, the more tensile stresses develop in the concrete slab, and thus early-morning placement is more critical than late-afternoon placement.

Based on the effect of placement time on the development of tensile stresses in the concrete, if slabs are placed early in the morning, initial post-tension should not be conducted more than 8 hours later, once the concrete has gained sufficient strength. Studies by O'Brien with early post-tensioning of concrete slabs (Ref 31) indicate that 10 kips might be applied safely at the anchor zone after 8 hours. For other cases, such as late-afternoon placement, initial prestress may be applied no earlier than 8 and no later than 12 hours after placement. The allowable post-tensioning force should be determined by compressive tests of concrete cylinders at the job site. It is recommended that this force should not exceed 15 kips to minimize the level of creep taking place after initial prestressing (Ref 31).

As for final post-tensioning, depending on the type of concrete mix used for construction, the maximum tendon prestress force of 46.6 kips might be applied to the PCP slabs once they have gained sufficient strength. Usually, for concrete mixes using no additives and prepared with Type I portland cement, final post-tensioning might be done 48 hours after placement.

5.5 Summary

This chapter described a procedure that can be followed for the design of any PCP. General design considerations and factors affecting the design, such as traffic loads, temperature and moisture, and friction resistance between the PCP concrete slab and its supporting layer, were covered. Design variables were described for general cases and applied to a case study in Hillsboro, Texas. The design of the PCP for the case study is presented in a step-by-step manner that will serve as a guide for other PCP designs and is summarized in the flowchart shown in Figure 5.20.

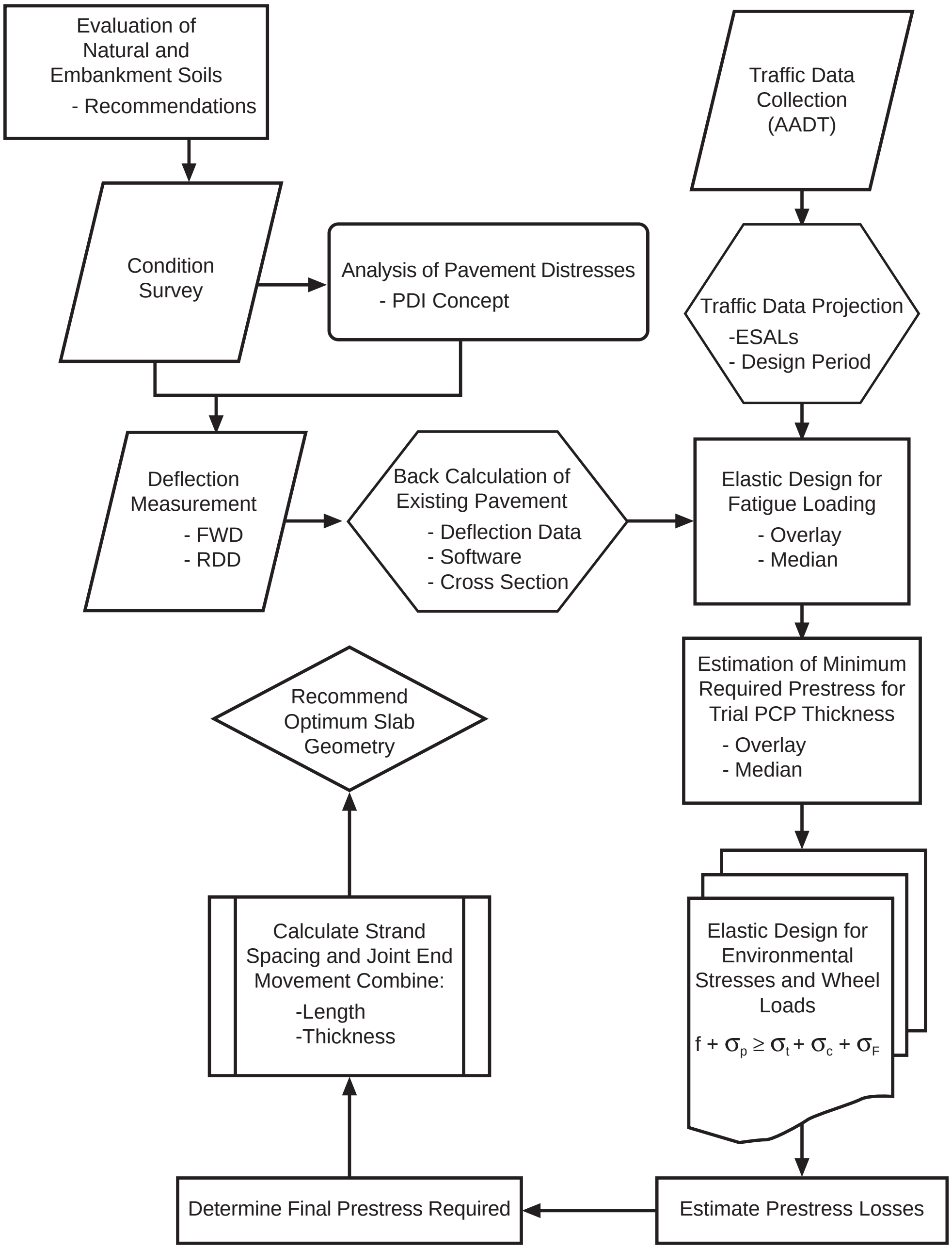


Figure 5.20 Flowchart used for designing the PCP

Because the final definite design characteristics of any PCP are flexible for selecting PCP slab thickness and length, engineering judgment has to be applied according to experience and expectations. Therefore, the recommended characteristics of the PCP to be constructed in Hillsboro, Texas, are summarized in Table 5.10.

Table 5.10 Design highlights for the PCP in Hillsboro, Texas

Design Feature	Recommendation
Design life	30 years
Projected traffic for design life	166 million ESALs
Concrete design flexural strength	700 psi
Concrete modulus of elasticity	4000 ksi
Thickness of PCP slab	9 in.
Length of PCP slab	300ft
Longitudinal strand spacing	Existing pavement overlay: 14 in. New pavement on median: 12 in.
Transverse strand spacing	Existing pavement overlay: 8ft New pavement on median: 5ft
Expected longitudinal slab movement	3.0 in. (for 300ft)
Expected transverse slab movement	0.7 in. (for 71ft)

6. MATERIALS SPECIFICATIONS
AND CONSTRUCTION GUIDELINES

6.1 Introduction

This chapter contains the steps to be followed for the construction of the PCP in Hillsboro, Texas. The methodology presented here follows upon the design performed in Chapter 5. As a result, this section of the report focuses on the presentation of a series of materials requirements and construction steps that, although tailored for the construction of the PCP section in Hillsboro, Texas, might be adapted for use in any other PCP constructed elsewhere. Although the recommendations presented here are considered to be comprehensive and ready for implementation, they need to be reviewed by TxDOT officials to be considered final.

The first subject discussed is the results obtained from extensive research conducted for soils used in highway embankments. Recommendations are given for the construction of embankments, and the required soil properties are highlighted. Later in this chapter, high-performance concrete pavement properties are discussed and recommended for the long-term performance PCP. Furthermore, specifications for other material including post-tensioning steel strands, strand chairs, anchors, joint hardware, friction-reducing membrane, etc., are described.

Finally, a series of construction steps based on previous experiences in the United States and abroad is intended to serve as a guideline for the construction of the PCP in Hillsboro, Texas. Although a broad quality control/quality assurance (QC/QA) plan to monitor the performance of the PCP is contained in Chapter 7, this chapter emphasizes the relevance of conducting such activities.

6.2 Materials Specifications

Although similar pavement construction projects have comparable material specifications, it is common practice that every project has specific or particular specifications that apply only to that project. Given the infrequent construction of PCP projects, the availability of construction specifications is scarce. However, the general requirements of basic materials for PCPs (e.g., subbase material, concrete mix, etc.), are similar to the requirements for conventional concrete pavements.

This report specifies material requirements for the pavement's substructure and superstructure separately. The substructure of the pavement is formed by the layers constructed beneath the subbase down to the foundation soil. Similarly, the superstructure of the pavement is formed by the material layers placed above the subgrade up to the riding surface layer, in this case, the concrete PCP slab.

6.2.1 SUBSTRUCTURE REQUIREMENTS

In all construction projects, including pavement projects, an evaluation of the characteristics of the foundation soil has to be performed by either surveying the soil or by consulting the available literature. For the PCP to be constructed in Hillsboro, Texas, a vast literature was found describing the characteristics of the soil in the area where the pavement will be located (Ref 32). Additionally, and because the PCP will be constructed over the same area where the existing IH 35 is located, information about foundation soils and embankments was also available through the TxDOT Bridge and Design Divisions.

6.2.1.1 Foundation Soil

According to the literature, the foundation soil where the PCP will be constructed in Hillsboro, Texas, has varying characteristics depending on the specific location along the highway. Various soils including Ferris, Heiden, Houston Black, and Lamar were found during soil surveys conducted in the past (Ref 32). All the soils, except for Lamar, are very clayey soils with low strengths and severe shrinking-swelling characteristics. The ASTM (Ref 33) classifies these soils as CH, containing fines passing sieve number 200 between 75 and 99%. The value of the LL varies from 51 to 90, and the PI ranges from 34 to 65. These characteristics classify them as fat clays, and fat clays with sands in the best case scenario. Lamar soil has much better engineering properties; the ASTM (Ref 33) classifies it as CL and CL-ML. The fine content passing sieve number 200 is between 60 to 80%. The LL ranges between 20 and 40% and the PI varies between 5 and 18. These characteristics classify the soil as lean clay and silty clay. Unfortunately, and according to conducted surveys, Lamar soil was found only in a length of half a mile under the existing pavement on IH 35, just north of Hillsboro.

Although it is beyond the scope of this report to provide specifications for the foundation soil, its handling and treatment might be conducted as is usually done for conventional pavements. The material specifications drafted by TxDOT (Ref 34) should be applied with discretion, specifically Part II Division I, which deals with earthwork tasks. Special attention should be given to Items 110, “Excavation,” and 112, “Subgrade Widening.” Additionally, the following section, which deals with soil compaction and requirements for embankments, is applicable to the foundation soil.

6.2.1.2 Embankments: Soil Compaction Overview and Density Requirements

The materials used for highway embankments have a great influence on the ultimate performance of pavements. The construction of embankments goes beyond the simple placement and compaction of earthen material that will support the pavement structure. Important issues related to soil properties, like PI values and how to minimize them to prevent excessive embankment movements, have to be discussed first. Additionally, the feasibility of using lime for soil stabilization purposes to improve the engineering properties of soils is discussed. Among the objectives of this section of the report are the following:

1. To emphasize that soils with high PI values used in embankments represent a problem that will affect the performance of the pavement if they are not treated adequately
2. To identify the plasticity characteristics of the soils that will be used for the construction of the embankments of the PCP section
3. To provide guidelines to estimate the potential vertical rise (PVR) of the embankments that will support the PCP
4. To evaluate different compaction techniques and soil moisture characteristics and combinations that might reduce the PVR or settlement of the embankments
5. To find out from previous research studies the benefits of using lime treated soils
6. To evaluate the feasibility of using lime for stabilization of the soil used in the construction of the embankments of the PCP section

In the concrete pavement construction industry, there is almost no factor that has greater influence on the long-term performance of a finished facility than the adequate compaction of embankments. A combination of experience and engineering knowledge has demonstrated that a reliable method of compaction is essential. A comprehensive understanding of the behavior of soils, their interaction with other materials and structures, and the presence of water will prevent future problems.

In the 1930s Mr. R. R. Proctor of the Bureau of Waterworks and Supply for the City of Los Angeles, California, developed testing equipment and methods for determining the moisture and density characteristics of various soils that provided the greatest stability and water tightness (Ref 35). The soil laboratory tests developed by Mr. Proctor for defining a degree of compaction obtained by the energy developed from a specific number of impacts of a rammer on a confined molded soil sample were identified by his name. Current tests (e.g., ASTM D 1557, “Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000ft-lbf/ft^3 (2,700 kNm/m^3)),” are based on Proctor’s original work and are used worldwide. Today, standards such as this have considerable influence in the design and application of compaction equipment. Compaction equipment is designed with the objective of providing specified densities for highway embankments and surface courses with the fewest number of passes.

In all highway projects at the present time, transportation agencies require minimum densities for embankments compacted by controlled methods. State agencies usually specify required densities as a percentage of the values obtained using either ASTM D2922-01, “Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth,)” or ASTM D2937-00, “Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method E (2002).” Depending on the soil characteristics and classification, and its location in the embankment, transportation agencies specify a minimum of 85 to 90% of compaction and a maximum of 95 to 100% of compaction relative to the ASTM standards above. There is no universal range of values used everywhere.

6.2.1.3 Plasticity of Soils Used in Highway Embankments

Highway engineers consider clayey soils to be expensive soils because they usually need to be stabilized to improve their engineering properties. Highway embankments are built using rock, soil, aggregates, rubblized pavement, or a combination of materials. When soils with a high PI and swelling potential are used, expansion and contraction of those masses of soil will cause a rapid deterioration of the riding quality of the facility and possible loss of support. Due to this concern, the American Society for Testing and Materials (ASTM) has issued some recommendations for classification and use of soils and soil-aggregate mixtures based on their properties. ASTM D3282-93, “Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes,” describes a procedure for classifying soils into seven groups based on laboratory determination of particle-size distribution, liquid limit, and plasticity index. This standard should be used when precise engineering classification is required, especially for highway embankment construction purposes. Evaluation of soils is made by means of a group index, which is a value calculated from an empirical formula (Ref 36). As a recommendation, ASTM D3282-93 considers soils having a liquid limit (LL) above 40 and plasticity index above 10 to be unsuitable for use in highway embankments.

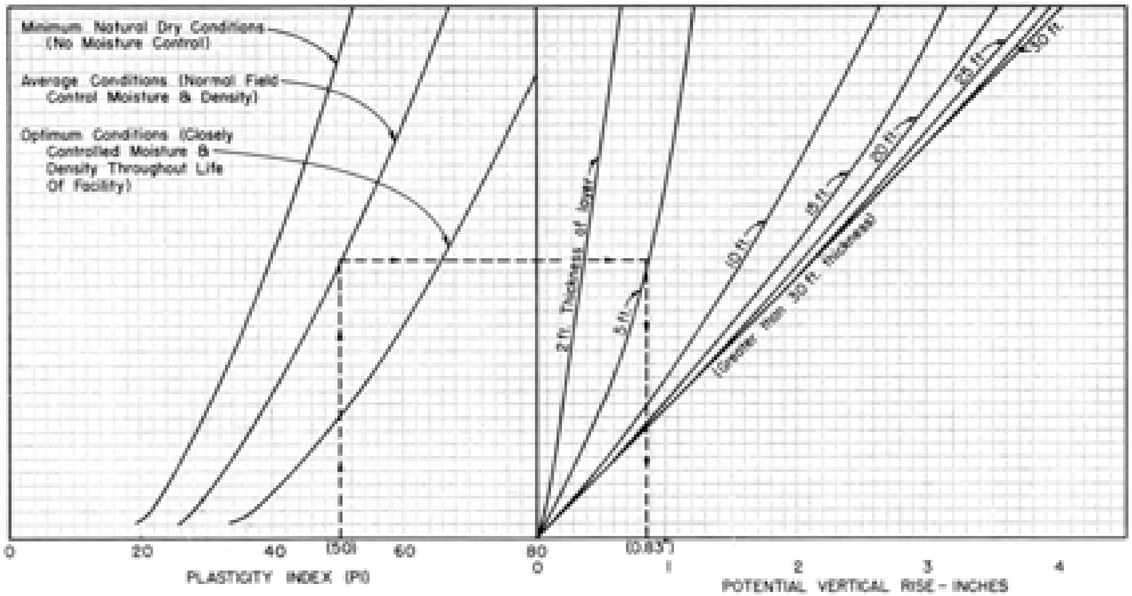
6.2.1.4 Potential Vertical Rise (PVR) of Embankments

The PCP section in Hillsboro will be constructed in an area that has a known history of soils with high swelling characteristics, as previously mentioned. Those swelling conditions of the natural soil have significantly reduced the performance of some pavements built along this section of IH 35. Figure 6.1 displays an embankment located on the northbound roadway of IH 35 where the future PCP section will be constructed.

Figure 6.1 Existing embankment where PCP section will be constructed

As shown in Figure 6.1, the embankment material underneath the existing pavement structure has experienced significant movement due to the high plasticity of the clayey soil in the embankment. These types of movements must be controlled, especially for embankment at bridge approaches or other structures where movement of the embankment is problematic. Excessive swelling or shrinkage of the embankment near bridges becomes particularly critical as the depth of the embankment is large and can result in very substantial movement. This can cause a loss of support beneath the pavement which can lead to differential settlement of the bridge and approach slab.

It is essential to know the plasticity characteristics of the soils to be used in embankments and to understand their ability to swell and shrink at a given density, moisture, and loading condition, when exposed to traffic loads. This information can be used to determine the PVR and the drainage properties of the soils used in embankments. McCullough et al. (Ref 37) developed a chart that correlates PI and the PVR of a given embankment for different thicknesses ranging from 2ft to more than 30ft, which is shown in Figure 6.2. To determine the PVR of a soil stratum, the Atterberg Limits or limits of consistency should be obtained as indicated by the ASTM D4318-00, “Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils,” or a similar standard. Additionally, natural moisture content of the soil layers at various depths must be known. The PVR can be computed following the procedure described in Test Method Tex-124-E (Ref 38). Figure 6.2 shows a group of curves that infer the approximate PVR for natural soils with different moisture levels given the PI of the soil and the height or thickness of the embankment.



Notes:

(1) This chart is predicated upon the following assumptions:
   (a) The subgrade soils for the thickness shown all are passing the No. 40 mesh sieve.
   (b) The subgrade soil has a uniform moisture content and plasticity index throughout the layer thickness for the conditions shown.
   (c) A surcharge pressure from 20 inches of overburden (±10 inches will have no material effect).
(2) Calculations are required to determine the PVR for other surcharge pressures.

Figure 6.2 Chart for determining the approximate PVR for natural soils

From Figure 6.2 it can be seen that for a given embankment thickness, the PVR varies depending on the moisture level of the soil. For embankments placed at minimum natural dry conditions, the swelling potential is at its highest predicted value. At optimum conditions, with closely controlled moisture levels, the swelling potential is reduced to a minimum level, and at average conditions in the field the swelling potential is somewhere between the two other cases.

6.2.1.5 Effect of Water Content and Compaction Method

As stated previously, the soils forming the embankments for the new PCP are clayey soils. Clays are formed by very small plate-like particles with a large surface area so that surface forces dominate body forces, allowing a clay soil to form several different types of structures. These structures may be broadly categorized as dispersed and flocculated. A dispersed structure consists of particles that are arranged in a parallel array with primarily face-to-face contacts. In contrast, a flocculated structure consists of a more random orientation of the clay particles having edge-to-face contacts. There are different conditions that affect the type of structure, but in general, anything that increases the repulsive forces between particles will result in a more dispersed structure, while anything that decreases the repulsive forces will result in a more flocculated type of structure. If a clay soil is compacted at a water content considerably below the optimum, a flocculated structure will develop. As the water content is increased, the structure will become more dispersed so that at water contents near saturation, the structure may be totally dispersed. Figure 6.3 shows the change in density and particle orientation for varying water content of a typical clayey soil.

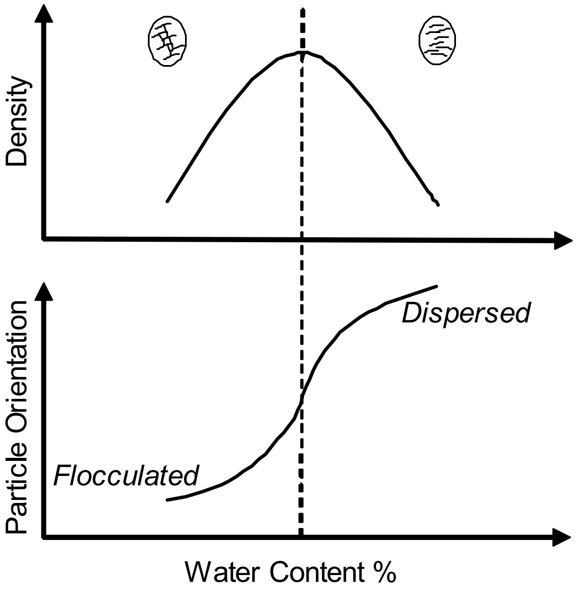


Figure 6.3 Variation of clayey soil properties with water content

The actual degree of orientation over the range of water content will be a function of the compactive effort. However, methods of compaction also influence the structure obtained. High shear strains imposed during compaction produce dispersed structures. Thus, dynamic compaction will produce higher strains than static compaction and consequently a more dispersed structure arrangement. Experience and laboratory research studies have shown that clay samples compacted dry of optimum experience considerably less shrinkage than samples of the same soil compacted wet of optimum. On the other hand, there is evidence to indicate that samples compacted dry of optimum show higher swelling characteristics and swell to higher water contents than do samples of the same density compacted wet of optimum (Ref 39).

As for the effect of compactive effort and type of compaction on the structure of the soil, it has been observed that different results can be found for different soils, but in general soils compacted by kneading compaction show a different structure when compacted wet and dry of optimum water content. Those changes are due to the increase in water and to the large shear strains during wet compaction. In contrast, for most soils compacted by static methods there will be little change in structure regardless of the compacted water content and density.

Additionally, for soils compacted dry of optimum moisture, both static and kneading compaction produce no appreciable shear deformation in the soil and consequently result in similar structures (relatively flocculated or with random orientations of clay particles). However, for soils compacted wet of optimum kneading compaction produces appreciable shear deformations resulting in a more dispersed structure, and static compaction produces a relatively flocculated structure.

With regard to soil strength, when compacted dry of optimum, method of compaction has little effect, and relative strength of soils is very similar. When compacted wet of optimum, relative strengths at low strains vary widely for different methods of compaction, but at high strains only small differences are noticed (Ref 39).

Figure 6.4 graphically summarizes the variation of axial shrinkage and swelling for a sandy clay compacted wet and dry of optimum moisture content and using static and kneading compaction methods (Ref 39). It can be seen that when compacted dry of optimum, the soil swelled more than when compacted wet of optimum. Additionally, static compaction produced more swelling with both low and high moisture contents. Relating to axial shrinkage, a great difference was observed in the soil with the two moisture contents, where wet of optimum conditions accentuate shrinkage. As for the effect of compaction equipment, it was observed that kneading compaction caused greater shrinkage of the soil than static compaction.

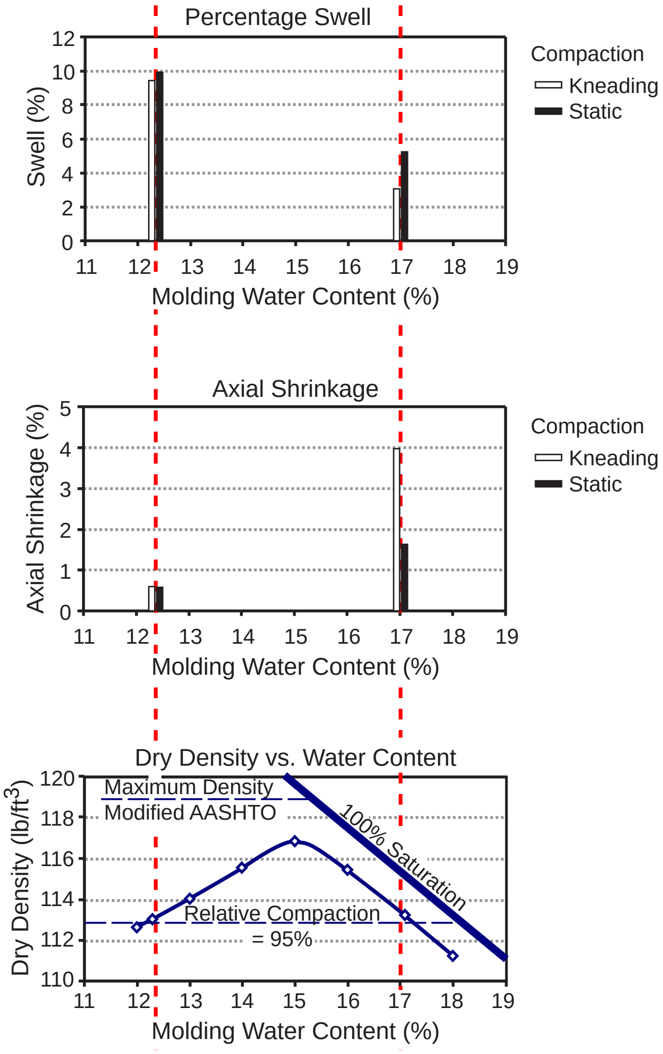


Figure 6.4 Variation of soil properties with water content

Figure 6.5 shows the variation of swelling pressure, which can be interpreted as the tendency of soil to heave, on dry density for a sandy clay compacted with static and kneading compaction methods. The swell pressures exerted by static compaction exceed those of specimens of equal densities and water contents prepared by kneading compaction, indicating a greater degree of flocculation in the samples prepared by static compaction.

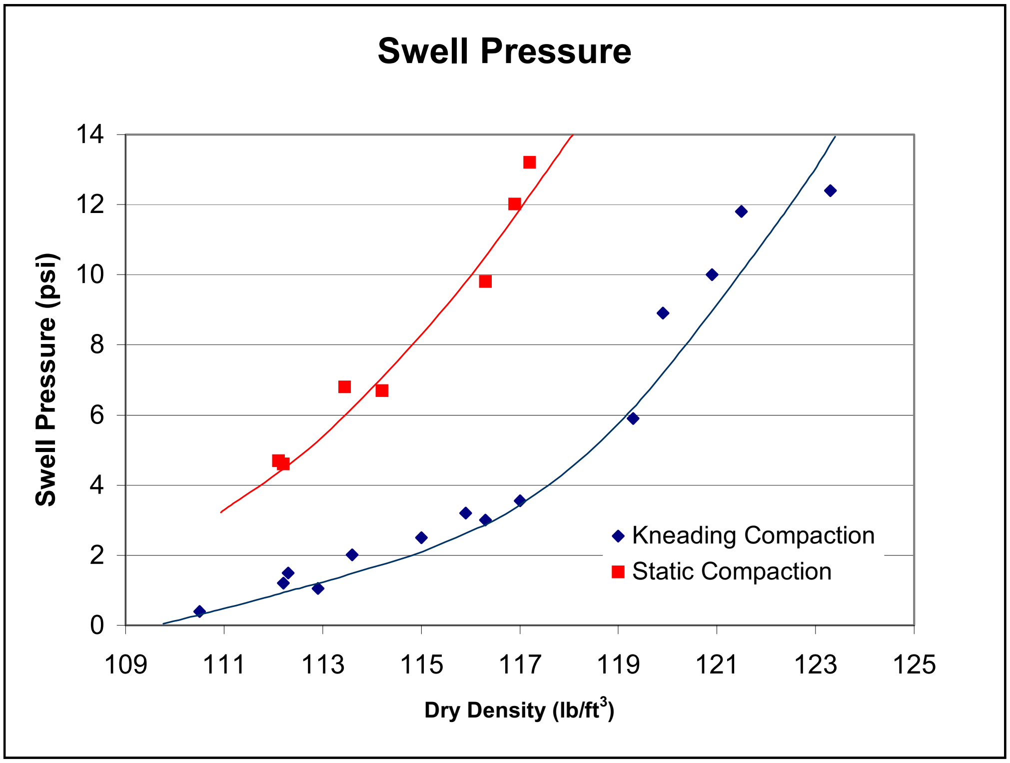


Figure 6.5 Effect of compaction methods on swell pressure for a sandy clay

6.2.1.6 Benefits of Soil Stabilization

Soil stabilization consists of mixing a natural soil with chemical products to improve its engineering properties. Stabilization of soils can also be accomplished by mechanical methods, but in general chemical treatment is preferred due to the improvements made to the original soil (strength, plasticity, permeability, swelling characteristics, etc.). Soil stabilization has been used for several years. Various investigations and research projects have gone into the development of materials evaluation and mixing techniques to determine the best combinations of soils and stabilizing agents.

Materials used for soil stabilization include asphalt products, cement, lime, lime by-products, and fly ashes. This section focuses only on the use of lime because it is the material most commonly used by TxDOT, and it is well-documented in the construction specifications (Ref 34). Lime has been found to be an inexpensive material compared to other products like portland cement, and it provides greater benefit to the soil. In a research study conducted by Kennedy et al. (Ref 40), different Texas soils including clays and sandy clays were stabilized using portland cement and lime, and were tested before and after stabilization. The tests included Atterberg Limits and unconfined compressive strength. Atterberg Limits were determined for the original soils and then for the treated soils three days after the mixing process. Unconfined compressive strength was determined at various ranges of time over the curing period for both dry and wet moisture conditions. From the research study the following conclusions were emphasized:

1. Lime treatment produced higher strengths than portland cement treatment did for high-plasticity clays
2. Cement treatment produced higher strengths than lime treatment for low-plasticity sandy clays
3. Lime treatment provided higher retention of strength than cement treatment did when samples were exposed to moisture
4. Dry-conditioned compressive strengths increased for both lime and cement treatments when clays were compacted using the modified AASHTO compactive effort in lieu of the standard AASHTO compactive effort. Figure 6.6 shows how both lime and cement treatments boost the unconfined compressive strength of soil in dry conditions

5. Wet-conditioned strengths decreased for cement-treated clays when increasing from standard to modified compactive effort. Lime-treated clays tested under the same conditions showed an increase in strength similar to the one observed in dry conditions. Figure 6.7 shows how only lime treatment boosts the unconfined compressive strength of the soil in wet conditions
6. Lime treatment seemed to reduce the swelling tendencies of the expansive clays, while the cement treatment did not

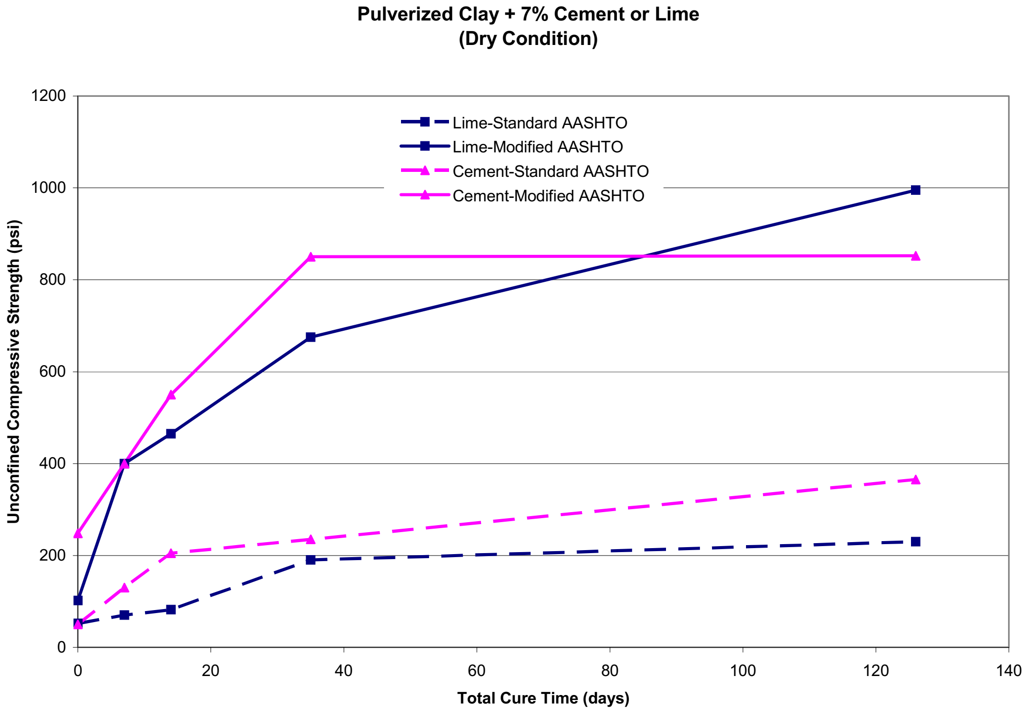


Figure 6.6 Effect of lime and portland cement on strength of soil (dry condition)

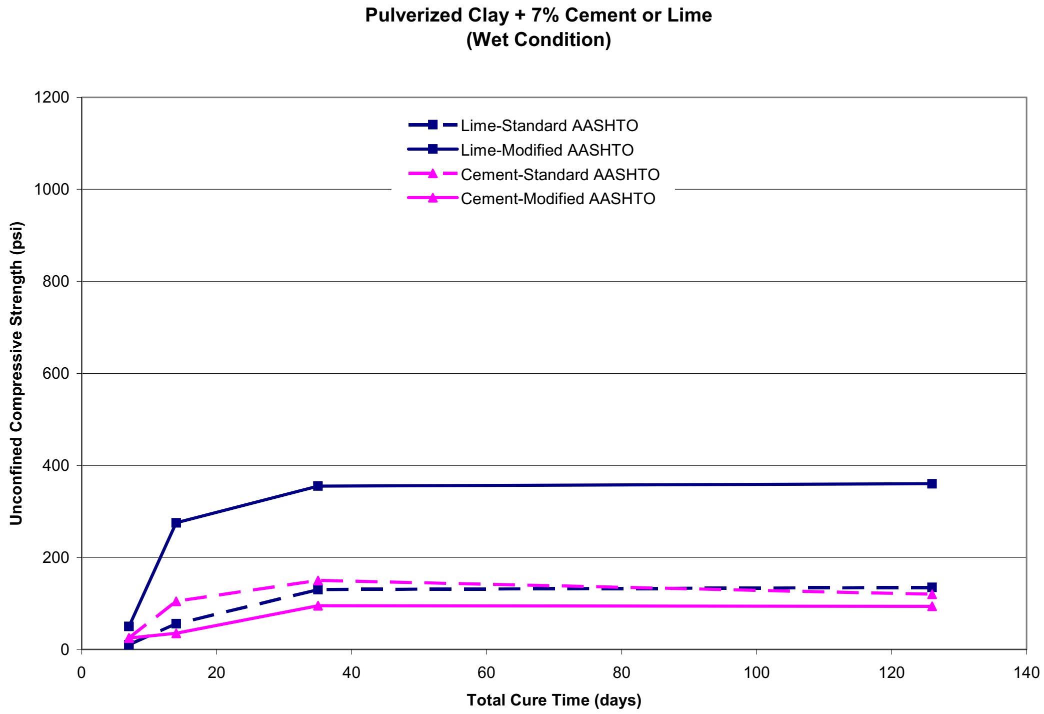


Figure 6.7 Effect of lime and portland cement on strength of soil (wet condition)

This research study showed the benefits of stabilizing soils using lime treatment. Based on the literature findings, the following section describes the properties recommended for the soils used for the embankments of the PCP section in Hillsboro, Texas.

6.2.1.7 Recommended Properties of Soils Used in Embankments

Although construction of embankments is a common task for highway projects, experience has shown that soil and materials used for embankments may require special attention. Plasticity characteristics and engineering properties of the soils should be carefully examined prior to construction. Adequate compaction should be ensured, and chemical treatment should be considered if the soils do not comply with the minimum requirements established by the construction and materials specifications. TxDOT's construction specifications (Ref 34) recommend the procedures for placement and compaction of materials used for the construction of roadway embankments in Item 132 “Embankment.” It describes the construction process in terms of materials to be used, compaction method, and density requirements.

For materials, Item 132 states that if granular (Type A) material is to be used in a retaining-wall-backfill area, it shall meet the requirements for backfill material. This requires it to be free from vegetation or other objectionable matter and reasonably free from lumps of earth. This material must be stable enough to form an embankment and shall meet the following plasticity requirements:

The liquid limit shall not exceed 45
The plasticity index shall not exceed 15
The bar linear shrinkage shall not be less than 2

ASTM considers soils having liquid limit (LL) of 40 and above and plasticity index (PI) of 10 and above to be unsuitable for use in highway embankments. With regard to the compaction and specified density for the embankment, Item 132 recommends the values specified in Table 6.1, unless otherwise shown on the plans.

Table 6.1 Recommended compaction and moisture values for embankment soils

Material Description	**Density, Percent**	**Moisture**
Non-swelling soils with plasticity index PI = 10	Not less than 98	
Swelling soils with plasticity index of 10 < PI = 35	Not less than 98 nor more than 102	Not less than optimum
Swelling soils with plasticity index over 35	Not less than 95 nor more than 100	Not less than optimum

The density determination is made in accordance with Test Method Tex-114-E (Ref 38). Field density determination is made in accordance with Test Method Tex-115-E (Ref 38).

Item 260, "Lime Treatment for Materials Used as Subgrade (Road Mixed)," (Ref 34) recommends types of lime to be used, mixing methods, compaction equipment, construction process, and density requirements.

6.2.1.8 Summary and Recommendations for Embankments

This section described some potential problems with roadway embankments that may affect the long-term performance of the PCP pavement. It presents some guidelines and references for required soil characteristics, compaction methods, moisture conditions, and soil stabilization. Experience has demonstrated that combined input of contractors, highway agencies, and researchers yields the best approach in any construction task and minimizes undesired results.

The area where the PCP section will be constructed in Hillsboro, Texas, contains swelling soils with high PI values that must be carefully considered to minimize excessive vertical rise or settlement of the embankments in the future. Based upon the ASTM recommendations presented herein, embankment material with a PI greater than 10 and a LL greater than 40 should be stabilized with lime or another accepted material.

Compacting using a kneading compactor at a moisture content above the optimum will prevent the material from high swelling pressures, although excessive shrinkage should be monitored. As with any construction project, quality control during construction is essential to ensure adequate compaction and proper treatment of the soil used for the embankment.

6.2.2 SUPERSTRUCTURE REQUIREMENTS

As stated in Section 6.2, the superstructure of the pavement is formed by all the material layers placed above the subgrade, from the subbase up to the riding surface layer, in this case, the PCP concrete slab. It is common practice that materials used in the superstructure of a pavement are of higher and better controlled quality; thus, the required characteristics of the materials forming the superstructure of the PCP should be similar to the ones used for the construction of any other type of pavement. In Texas the quality of base and subbase materials and their specifications have been very well studied, with good end results. Item 247, "Flexible Base," in TxDOT's standard specifications (Ref 34) details the physical requirements for flexible base materials, which will correspond to the requirements for the subbase of the PCP.

6.2.2.1 Special Considerations for Subbase Materials

The construction of the subbase layer for all primary highway pavements in Texas is done with a good-quality crushed material that is usually treated with asphalt, portland cement, or lime, this last one being the most commonly used material. For the PCP to be built in Hillsboro, Texas, different subbase materials might be used. The only requirement will be that the materials comply with the minimum requirements for the construction of conventional pavement structures. The subbase could possibly include recycled asphalt concrete mixed with new subbase quality material, if TxDOT's officials consider this to be a feasible option. The thickness of the existing asphalt concrete layer on IH 35, where the PCP will be constructed, varies between 10 in. and 12 in., which could provide sufficient recycled material for the conformation of a new high-quality reclaimed asphalt pavement (RAP) subbase layer. RAP is defined as a salvaged, milled, and pulverized asphaltic pavement that is broken or crushed so that 100% will pass through the 2-in. sieve. Item 340, "Hot Mix Asphaltic Concrete Pavement," in TxDOT's construction standard specifications (Ref 34) details the requirements for the salvaged material.

In addition to incorporation into bases and subbases, RAP has been successfully used in recycled asphalt pavements. In some cases, when the material is available in large quantities, it is used for embankment construction and as a fill material. When used as an embankment or fill material, the undersize portion of crushed and screened RAP, typically less than 2 in., may be blended with soil and/or finely graded aggregate. Uncrushed or more coarsely graded RAP has been widely used as the embankment base. The use of RAP in Europe dates from the 1970s, and in the United States various states including Connecticut, Indiana, Kansas, Montana, New York, Tennessee, California, Illinois, and Louisiana have used the material to some extent. In all the cases the performance of RAP in embankments has ranged from satisfactory to good.

The performance of RAP in base and subbase materials has been reported to be from satisfactory to excellent, depending on how it is used, in what proportion, and, most importantly, on the properties of the constituent materials and asphalt concrete type used in the old pavement. Positive features of RAP that has been properly incorporated into granular base and subbase applications include adequate bearing capacity, good drainage characteristics, and very good durability. However, RAP that is not properly processed or blended to design specification requirements may result in poor pavement performance. For conventional pavements, increasing the RAP content results in a decrease in the bearing capacity of the granular base, but for PCP application this might not be a concern, because one of the major advantages of this paving technology is the fact that the much higher bearing capacity of the PCP slab compensates for the possible lower capacity of the layers placed underneath. Finally, some of the engineering properties of RAP that are of particular interest when it is used in granular base or subbase applications include gradation, bearing strength, compacted density, moisture content, permeability, and durability (Ref 41).

6.3 Material Specifications for PCP Slabs

The construction of the PCP slabs involves the use of a great number of materials, including portland cement concrete, post-tensioning steel strands, steel anchors, steel chairs, transverse joints, and miscellaneous materials. This section describes and recommends the characteristics of the materials that might be used for the construction of the slabs. However, the final decision and selection of all the materials has to be approved by TxDOT's officials. In all cases the utilized material must meet required specifications by TxDOT or the ASTM.

6.3.1 PORTLAND CEMENT CONCRETE SPECIFICATIONS

Due to the relevance of the PCP project, which will be constructed on a highly trafficked highway, it is recommended that a high-performance concrete (HPC) is used for the construction of the PCP slabs. An HCP is generally defined as a concrete that meets certain criteria proposed to overcome limitations of conventional concrete. It may include concrete that provides either substantially improved resistance to environmental influences (durability in service) or substantially increased structural capacity while maintaining adequate durability. At the same time, an HPC should significantly reduce construction time to allow rapid opening to traffic, without compromising long-term serviceability.

Although the properties of the concrete used in the paving industry is well understood and specified by state agencies, it is important to observe and monitor different variables that will determine the performance of the PCP. Among the controllable variables influencing the performance of the concrete slabs are type of cement used, coarse aggregate type, use of admixtures, construction process, curing procedure, etc. Additionally, uncontrollable variables related to climatic conditions during and after construction have a great impact on the ultimate performance of the concrete.

Item 360, "Concrete Pavement," in TxDOT's construction standard specifications (Ref 34) contains information about the concrete type that should be used for the construction of pavements. According to TxDOT's specifications, Class "P" portland cement concrete should be used, unless otherwise shown on the plans. As for the quality of the concrete, it should be in accordance with Item 421, "Portland Cement Concrete," as well as any additional requirements shown on the plans.

In engineering terms, and with regard to the meaning of HPC, Aitcin and Neville report that "in practical application of this type of concrete, the emphasis has in many cases gradually shifted from the compressive strength to other properties of the material, such as a high modulus of elasticity, high density, low permeability, and resistance to some forms of attack" (Ref 42). Following this criterion and given the PCP design detailed in Chapter 5, in which the concrete was assumed to have certain performance values (e.g., flexural strength and modulus of elasticity), Table 6.2 recommends some performance characteristics that should be monitored during the production of the concrete for the PCP slabs. Provisions should be made to meet these requirements.

Table 6.2 High-performance characteristics required for PCP slabs

Performance Characteristic	Test Method
Compressive Strength	ASTM C 39
Flexural Strength	ASTM C 78, ASTM C 293
Modulus of Elasticity	ASTM C 469
Shrinkage	ASTM C 157
Creep	ASTM C 512
Petrographic Analysis	ASTM C 295, ASTM C 856

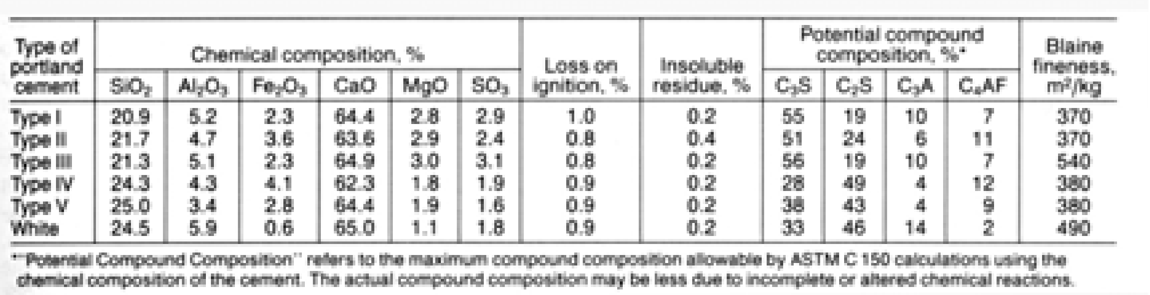
6.3.1.1 Portland Cement Types

Item 421, "Portland Cement Concrete," in TxDOT's construction standard specifications (Ref 34) states that Class "P" concrete should be composed of any portland cement type including I, IP, II, and III with some exceptions. For instance, Type III cement should not be used when the anticipated air temperature for the succeeding 12 hours will exceed 60ºF. This means that if the pavement is built during the summer, Type III cement would not be allowed for construction because high early strength would be accelerated, causing negative thermal shrinkage effects. Additionally, it is recommended that the same cement type is used in monolithic placements.

Type I cement is used for general purposes. It is used when the concrete will not be exposed to aggressive environments, such as sulfate attack or dramatic temperature change during production and placement of concrete pavements. TxDOT specifications also state that Type IP cement, which is a mixture of portland cement and fine pozzolan, might be used in lieu of Type I or Type II cement when high early strengths are not required. The effect of pozzolan and other mineral admixtures is described later in this chapter. When moderate sulfate resistance or moderate heat of hydration is desired, Type II cement would be indicated. Type III cement is generally used when high strengths are required at early ages. This type of cement is similar to Type I cement, except that contains finer particles. Cold weather concreting and a required quick opening to traffic are common reasons for using this type of cement. Type IV cement is used where the heat generated from hydration must be minimized. It slows down the strength-gaining process of the concrete and is not usually specified for production of concrete pavement. Type V cement is rarely used in pavement applications; it is available in particular parts of the United States where high sulfate-resistance is necessary. Since it is more difficult to find this type of cement in the market, occasionally Type II is used instead (Ref 44).

ASTM C150-02A, "Standard Specification for Portland Cement," (Ref 45), shows the characteristics of portland cements Types I through V and other types that could be used for the concrete for PCP pavements. The use of any type of cement depends on various factors, the environmental conditions to which the concrete will be exposed being the most significant. Whereas for the PCP section to be constructed in Hillsboro, Texas, either Type I or Type III could be used, other PCPs exposed to high-sulfate environments, such as that of Houston or Galveston, should probably use Type II or Type V cement. Additionally, the use of mineral admixtures in the concrete enhances the short- and long-term performance of the concrete. Table 6.3 (Ref 44) summarizes the chemical composition of cement Types I through V. An understanding of chemical composition is essential to choosing the cement type that is best for a particular pavement project.

Table 6.3 Chemical composition and compound compositions of typical cements



Type of portland cement	Chemical composition, %						Loss on ignition, %	Insoluble residue, %	Potential compound composition, %*				Blaine fineness, m²/kg
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3			C_3S	C_2S	C_3A	C_4AF	
Type I	20.9	5.2	2.3	64.4	2.8	2.9	1.0	0.2	55	19	10	7	370
Type II	21.7	4.7	3.6	63.6	2.9	2.4	0.8	0.4	51	24	6	11	370
Type III	21.3	5.1	2.3	64.9	3.0	3.1	0.8	0.2	56	19	10	7	540
Type IV	24.3	4.3	4.1	62.3	1.8	1.9	0.9	0.2	28	49	4	12	380
Type V	25.0	3.4	2.8	64.4	1.9	1.6	0.9	0.2	38	43	4	9	380
White	24.5	5.9	0.6	65.0	1.1	1.8	0.9	0.2	33	46	14	2	490

*"Potential Compound Composition" refers to the maximum compound composition allowable by ASTM C 150 calculations using the chemical composition of the cement. The actual compound composition may be less due to incomplete or altered chemical reactions.

6.3.1.2 Admixtures

Experience and research have shown that concrete properties can easily be modified by adding certain materials to concrete mixtures. There is a great variety of products that can be used to improve concrete mechanical and chemical properties. In Europe around 70 to 80% of the concrete produced contains one or more admixtures. The purpose of these materials is to improve characteristics of the concrete, including workability, acceleration or retardation of setting time, control of strength development, and enhancement of resistance to frost action, thermal cracking, alkali-aggregate expansion, and acidic and sulfates solution (Ref 46).

ASTM C125-02, “Definitions of Terms Relating to Concrete and Concrete Aggregates,” defines an admixture as a material other than water, aggregates, hydraulic cements, and fiber reinforcement which is added to the concrete or mortar immediately before or during the mixing process. Although the physical effect of admixtures on concrete mixtures can be noticed right away, it might take several days or even months for the chemical reactions to be evident, depending on the admixture used and its proportion. Due to the diversity of admixtures and a lack of a single classification, they are classified into three different categories: surface-active chemicals, set-controlling chemicals, and mineral admixtures (Ref 46). Surface-active chemicals, also known as surfactants, include admixtures that are commonly used for air entrainment or water reduction in concrete mixtures. Set-controlling chemicals are used as either retarding or accelerating admixtures. In fact, some chemicals act as retarders when used in small amounts, but when used in large dosages they behave as accelerators. Mineral admixtures are fine siliceous materials which are added to the fresh concrete in relatively large amounts, usually between 20 and 100% by weight of portland cement. This report focuses only on the application of mineral admixtures for the production of the concrete for the PCP.

Mineral admixtures could be pozzolanic (e.g., low-calcium fly ash), or cementitious (e.g., granulated iron blast-furnace slag), or both (e.g., high-calcium fly ash). Likewise, mineral admixtures are divided in two main groups:

1. Natural materials: the materials that have been processed only to produce a pozzolan. The processing usually includes crushing, grinding, and size separation. Among the natural materials are all natural pozzolans derived from volcanic rocks and minerals composed of aluminosilicates (cooled magma). Based on the principal reactive constituent, natural pozzolans can be classified into volcanic glasses, volcanic tuffs, calcined clays or shales, and diatomaceous earths.

2. By-product materials: the materials that are not the primary product of an industrial production and may or may not require any processing (e.g., drying and pulverization) before being used. Examples of these admixtures are ashes from the combustion of coal and some crop residues, volatilized silica from metallurgical operations, and granulated slag from ferrous and nonferrous metal industries. Industrialized countries are large producers of fly ash, volatilized silica, and granulated blast-furnace slag.

It is well known that the addition of mineral admixtures to the fresh concrete has a great positive effect on various concrete properties for both the short- and the long-terms. These effects on concrete properties are visible in the following aspects:

1. Workability
2. Durability when exposed to thermal cracking
3. Durability when exposed to chemical attack
4. Strength

The improvement in workability is really noticeable for fresh concrete mixtures that show a tendency to bleed or segregate. Incorporating finely divided particles improves the workability by reducing the size and volume of voids. The small size of fly ashes and slags allows for a reduction of water content for a given consistency. Research conducted using fly ash in concrete showed that by substituting 30% of the cement with the fly ash required 7%less water than the control concrete mixture with equal slump (Ref 47). Research has also shown that even though all mineral admixtures tend to improve the cohesiveness and workability of the fresh concrete, many do not have the same water-reducing capability of the fly ashes and slags.

With regard to the durability when exposed to thermal cracking, various studies show that by adding natural pozzolan, fly ash, or slag to a mass of concrete; the temperature declines almost in a direct proportion to the amount of the portland cement replaced by the admixture. Replacement of portland cement with fly ash has been conducted in the United States for several decades, and in mass concrete production proportions ranging from 60 to 100% by weight of the cement have been employed with successful results. Additionally, concrete exposed to high temperatures (>90ºF) commonly undergoes a strength loss due to microcracking on cooling. In contrast, concrete containing mineral admixtures frequently shows a strength gain.

The durability of concrete to chemical attack is a property that is directly related to the permeability of the concrete. The more permeable the concrete is, the weaker it is against chemical destructive actions like the alkali-aggregate expansion and attack by acidic and sulfate solutions. Laboratory tests have shown the improvement in the chemical durability of concretes containing mineral admixtures. Additionally, investigations have shown that the reduction of the permeability of concrete increases with time, and in some cases dramatically decreased permeability was observed even after 6 months of age of the concrete. Some work (Ref 48) confirmed that cement pastes containing 10 to 30% of a low-calcium fly ash significantly reduces concrete permeability during the first 90 days of age. In the case of cement pastes with 30% of silica fume or 70% of granulated blast-furnace slag, even at 28 days after hydration, the system was found to be almost impermeable. Depending on the properties of the mineral admixture, combinations of 40 to 65% of granulated blast-furnace slag, or 30 to 40% low calcium fly ash, or 20 to 30% natural pozzolans have been found to be very effective in limiting the alkali-aggregate expansion to acceptable levels. Condensed silica fumes in amounts as low as 10% are also very effective in reducing concrete permeability.

The impact of mineral admixtures on the strength of concrete is very important, especially for economic and durability reasons. Low-calcium fly ashes and natural pozzolans tend to reduce the early strength of concrete up to 28 days, but improve the ultimate strength. When used as partial replacement for fine aggregates, all mineral admixtures are able to increase the strengths of concrete at both early and late ages. Several studies have shown that the strength of concrete can be dramatically increased by adding mineral admixtures. The Aalborg Cement Company in Denmark, one of the largest cement firms in Europe, documented that using condensed silica fume, special aggregates of controlled particle size, and a superplasticizer, specimens with a ratio between water and cementitious material lower than 0.2 gave over 29,000 psi compressive strength.

6.3.1.3 Coarse Aggregate Type

The type of aggregate used in the production of concrete mixtures has a great impact on its ultimate performance and durability. Various research studies in Texas (Ref 22) have shown that the two most commonly used coarse aggregates in concrete, limestone and silica-based aggregate, produce very different performances in concrete pavements. This difference has to be evaluated before construction of the PCP takes place. The design of the PCP in Hillsboro, Texas, showed that there is a very significant difference between the movements computed for the PCP slabs when different coarse aggregates are used. The PCP design detailed in Chapter 5 showed that the high CTE of an aggregate causes high undesirable joint end movements of the PCP slabs, especially if the concrete is placed during the summer season. According to the results from the design of the PCP, it is highly recommended that limestone coarse aggregate be used for the production of concrete. Using silica-based aggregate will increase joint end movements that might be excessive, causing a penetration effect between the edges of the slabs.

With regard to the quality and cleanliness of the coarse aggregate, Item 421, "Portland Cement Concrete," of the construction standard specifications (Ref 34) states that the coarse aggregate used in the concrete should be washed and free from material with injurious amounts of salt, alkali, vegetable matter, and other objectionable material. The aggregate will be subjected to five cycles of both the sodium sulfate and the magnesium sulfate soundness tests in accordance with Test Method Tex-411-A (Ref 38). Material loss will be evaluated according to the specifications and will be accepted or rejected based on the tests results. Although concrete mix plants have a reliable quality control for their materials, caution should be exerted at the time the aggregate is used for concrete production.

6.3.1.4 Fiber-Reinforced Concrete

Fiber-reinforced concrete is conventional concrete to which discontinuous discrete fibers are added during mixing operations (Ref 44). The fibers could be made from steel, plastic, glass, and other materials. They are available in different sizes and shapes; typical lengths are from 0.25 to 3 in., and thicknesses range from 0.0002 to 0.030 in. Steel fibers are the most commonly used due to the significant improvement achieved in concrete flexural strength, impact strength, toughness, fatigue strength, and resistance to cracking. Other types of fibers have shown varied results. Although it is well known that the addition of fibers to the concrete reduces its workability, fiber contents up to 4 to 5% by volume of concrete can be used; however, 1 to 2% is the practical limit for field placement of most fibers. In pavement applications, steel fibers are the most commonly used, with contents varying from 0.6 to 1.0%. Tests for fiber-reinforced concrete are described in ASTM C995-01, "Test Method for Time of Flow of Fiber-Reinforced Concrete through Inverted Slump Cone," and ASTM C1018-97, "Test Method for Flexural Toughness and First Crack Strength of Fiber-Reinforced Concrete (Using Beam with Third Point Loading)."

Given that flexural strength rather than compressive strength is specified for pavements and considering that the most important contribution of fiber -reinforcement in concrete is not to strength but to flexural toughness of the material, it may be reasonable to consider using fibers in the concrete mixture for the PCP in Hillsboro, Texas. An experimental study by Shah and Rangan (Ref 49) shows that increasing the fiber content increases strength as well as toughness; however, while the increase in strength is only mild, the increase in toughness is very dramatic. Figure 6.8 displays graphically this behavior (Ref 46).

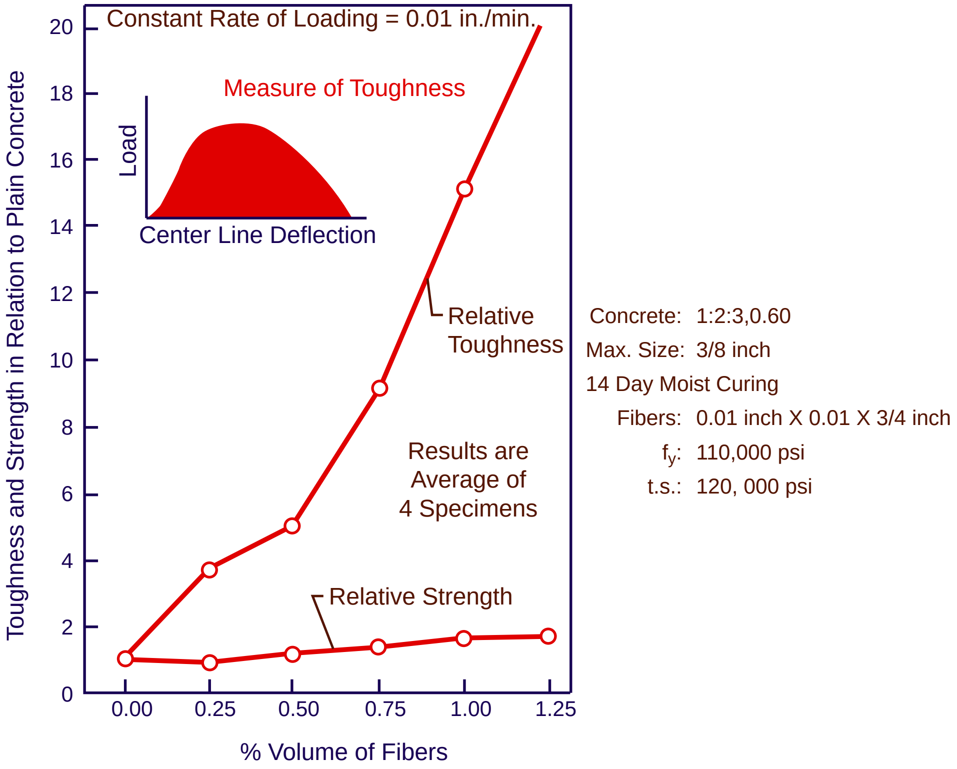


Figure 6.8 Increase in strength and toughness of fiber-reinforced concrete in relation to plain concrete for different volumes of fiber

6.3.2 POST-TENSIONING STEEL STRANDS

Steel strands are made of a group of twisted wires that usually contain a center wire surrounded by other six twisted wires. Seven-wire strands are typically used as a prestressing element in the bridge industry. In various PCP applications in the United States and abroad, the most commonly used diameters for steel strands are 3/8 in., 1/2 in., and 6/10 in., with tensile strength ranges from 250 to 270 ksi. Chapter 5 in this report assumed a strand 0.6 in. in diameter for the design of the PCP in Hillsboro, Texas. This is a commercial strand diameter that was previously used in the PCP in McLennan County and also for a more recent precast prestressed concrete pavement that was built in Georgetown, Texas, during the spring of 2002.

According to the literature, strand materials for prestressing should consist of either of the following two groups:

A. Uncoated, low-relaxation wire strand, Grade 270 (1860 MPa)
B. Uncoated, stress-relieved (normal-relaxation) strand, Grade 270 (1860 MPa)

With regard to the specifications of the strands, both groups should conform to AASHTO M203 (ASTM A416), "Uncoated Seven-Wire Steel Strand for Concrete Reinforcement." Additionally, Group B strands should conform to AASHTO M204 (ASTM A421), "Uncoated Stress-Relieved Steel Wire for Prestressed Concrete." Additionally, the capability of the strand to properly develop bond should be certified from the strand supplier. A light bond coating of tight surface rust on prestressing tendons is permissible, provided the strand surface shows no pits visible to the unaided eye after rust is removed with a nonmetallic pad (Ref 50).

Investigations have shown that the presence of light rust on strands has proven to be an enhancement to bond over bright clean strands and therefore should not be a deterrent to the use of the strands. However, if after an evaluation of rusting a pit is visible to the unaided eye, the material must be rejected (Ref 51). A visible pit greatly increases the stress in the steel and reduces the capacity of the strand to withstand repeated or fatigue loading. In many cases, although heavily rusted strand with relatively large pits will still test to an ultimate strength greater than specification requirements, it will not meet the fatigue test requirements.

6.3.3 ANCHORS

The steel strands will be anchored to the end of the slabs using anchorage hardware, which will ensure that the full prestress force is applied to the joint panel over the length or width of the slab. There are various types of anchoring systems, and their application depends on the type of pavement to be constructed. For the PCP in Hillsboro, Texas, the anchoring system will consist of a standard post-tensioning anchor, commonly used in the construction of prestressed concrete beams. Figure 6.9 displays a standard (fixed end) anchorage system that might be used in the PCP.

Figure 6.9 Standard anchor (fixed end) recommended for PCP construction

Using this standard anchor will allow the wedges to be set after the strands are inserted into the ducts. The anchor system shall meet the following requirements (Ref 50):

1. For bonded tendons:
    a. An anchorage for bonded tendons tested in an unbonded state will develop 95% of the actual ultimate strength of the prestressing steel, without exceeding anticipated set at time of anchorage. An anchorage that develops less than 100% of the minimum specified ultimate strength shall be used only where the bond length provided is equal to or greater than the bond length required to develop 100% of the minimum specified ultimate strength of the tendon.
    b. The required bond length between the anchorage and the zone where the full prestressing force is required under service and ultimate loads will be sufficient to develop the specified ultimate

strength of the prestressing steel. The bond length is determined by testing a full-sized tendon.

    c. If in the unbonded state the anchorage develops 100% of the minimum specified strength, it need not be tested in the bonded state.

2. For unbonded tendons:
    a. An anchorage for unbonded tendons will develop 95% of the minimum specified ultimate strength of the prestressing steel, with an amount of permanent deformation that will not decrease the expected ultimate strength of the assembly.
    b. The minimum elongation of a strand under load in an anchorage assembly tested in the unbonded state will be not less than 2% when measured in a gauge length of 10 ft

In general, anchorage castings will be nonporous and free of sand, blow-holes, voids, and other defects. For a wedge-type anchorage, the wedge grippers should be designed to prevent premature failure of the prestressing steel due to notch or pinching effects under static test load conditions to determine yield strength, ultimate strength, and elongation of the tendon. All these basic requirements should be demonstrated by an acceptable testing program.

6.3.4 TRANSVERSE JOINT HARDWARE

One of the most important aspects of the design and construction of a PCP involves a reliable transverse joint. In most of the PCP projects constructed worldwide the transverse joint diminished the performance of the pavements, primarily due to inefficient hardware. However, all the previous experiences with PCPs constructed in the United States and elsewhere served as a guideline for the conception of the transverse joint used in the PCP in McLennan County in Texas. The transverse joint used for the pavement in McLennan County refined the developments made in other projects, and thus successful joint hardware was specified.

The layout of the joint is shown in Figure 6.10 and consists of a steel support structure with Nelson deformed bars used to secure the joint to the concrete pavement slab. Additionally, stainless steel dowel bars are used to allow a good load transfer between slabs. Dowel expansion sleeves allow free movement between contiguous slabs. A neoprene seal is inserted between slabs and serves as a barrier to intrusion of undesired material into the joint.

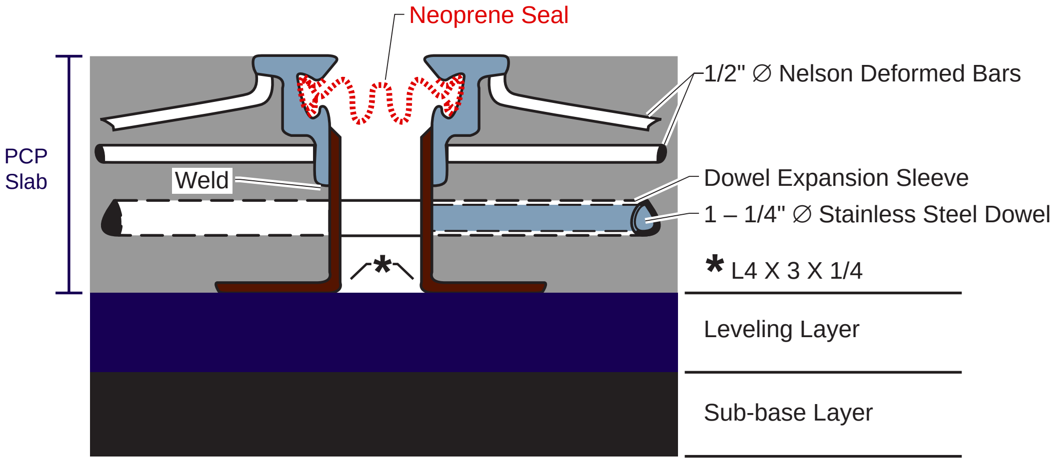


Figure 6.10 Transverse joint hardware

The transverse joint shown in Figure 6.10 is believed to be adequate for use in the PCP section to be constructed in Hillsboro, Texas. No radical modifications are needed for this joint, in contrast to the one used in McLennan County. An alternative neoprene seal type may possibly be sought in the market, in case the one that was used previously is no longer available.

6.3.5 STRAND DUCTS AND GROUT

Since bonded post-tensioned tendons will be recommended for the PCP in Hillsboro, Texas, it is very important that the ducts are strong enough to retain their shape during construction operations. The sheathing should prevent the entrance of cement paste or water from the concrete, and it should not cause harmful electrolytic action or deterioration. The inside diameter will be at least 1/4 in. larger than the nominal diameter of a single strand. The inside cross-sectional area of the sheath will be at least twice the net area of the prestressing steel (Ref 50). Sheaths should have vents at each end of the slab and near the central stressing pockets to allow injection of the grout.

With regard to the grout, there are various options. The simplest grout consists of a mixture of cement and water. Fly ash and pozzolanic mineral admixtures may be added at a ratio not to exceed 30% by weight of cement. Mineral admixtures will conform to ASTM C618-02, “Standard Specification for Coal Fly Ash and Raw or Calcined Natural Pozzolan for Use in Concrete.” Admixtures containing more than trace amounts of chlorides, fluorides, zinc, or nitrates will not be used. This grout should achieve a minimum compressive strength of 2,500 psi at 7 days and 5,000 psi at 28 days when tested in accordance with ASTM C109/C109M-02, “Standard Test Method for Compressive Strength of Hydraulic Cement Mortars,” and should have a consistency that will facilitate placement. Water content should be the minimum necessary for proper placement, and the water-cementitious materials ratio shall not exceed 45% by weight. Another option is to use grout already available in the market. Although this might be a more expensive alternative, errors during the mixing procedure will be avoided, resulting in a more reliable material.

6.4 Briefs on Construction Materials and Construction Guidelines

The previous sections in this chapter described the materials that might be used for the construction of the PCP in Hillsboro, Texas. The current section summarizes the specifications for the materials and the steps that are recommended for the construction process. Prestressing of the PCP slabs will be made in accordance with the plans and the following specifications as closely as possible, and they will govern the furnishing, storing, and handling of prestressing materials. For all the requirements, the indicated item number corresponds to the item described in TxDOT's "Standard Specifications for Construction of Highways, Streets and Bridges" (Ref 34). Additionally, if indicated, other national standard requirements (e.g., ASTM, AASHTO, etc.) should apply.

6.4.1 MATERIALS

6.4.1.1 Concrete Pavement

As previously stated, the materials and proportions used for production of concrete shall conform to Item 421, "Portland Cement Concrete." Coarse aggregates should be tested according to Test Method Tex-411-A. If air is entrained into the concrete mixture, it should not exceed 6%, unless TxDOT indicates otherwise. The concrete should have a flexural strength of 700 psi at the age of 28 days. If compressive strength is used as a surrogate test, trial tests should be performed to obtain a reliable correlation between the two tests and for the different sources of aggregates.

6.4.1.2 Reinforcing Steel

Reinforcing steel should conform to the requirements as specified in Item 360, "Concrete Pavement." Periodic tensile strength tests of the reinforcing steel should be done in accordance with Item 440, "Reinforcing Steel."

6.4.1.3 Expansion Joints

The expansion joints should conform to the requirements as specified in the plans. The joint hardware should comply with specifications as follows:

A. Armor Angles. The armor angles will conform to the requirements as specified in Item 441, "Steel Structures" and Item 442, "Metal for Structures."

B. Joint Extrusion. The joint extrusion will conform to the requirements of ASTM A606-01, "Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved

Atmospheric Corrosion Resistance," and to the configuration shown on the plans.

C. Neoprene Seal. The neoprene seal or diaphragm used will conform to the requirements as specified in Item 435, "Elastomeric Materials."

D. Dowel Bars. Dowel bars will conform to the requirements as specified in Item 360, "Concrete Pavement," and as shown on the plans. Additionally, the stainless steel will conform to the requirements of ASTM 176-99, "Standard Specification for Stainless and Heat-Resisting Chromium Steel Plate, Sheet, and Strip." The encasement should not be less than 0.01 in. in thickness.

E. Dowel Bar Expansion Sleeves. Expansion sleeves will be made of stainless steel. They will be properly welded to the armor angle at the locations shown on the plans. The free end of the sleeve will be capped to prevent entry of mortar or grout.

6.4.1.4 Prestressing Strands

Prestressing strands should be 0.6 in. diameter seven-wire prestressing strands and should belong to any of the following two groups:

A. Uncoated, low-relaxation wire strand, Grade 270 (1860 MPa)

B. Uncoated, stress-relieved (normal-relaxation) strand, Grade 270 (1860 MPa)

In any case, the reinforcing steel will conform to ASTM A416/A416 M-99, "Standard Specification for Steel Strand Uncoated Seven-Wire for Concrete Reinforcement." Furthermore, if steel from Group B is selected for construction, it will conform to ASTM A421/A421 M-98A, "Standard Specification for Uncoated Stress-Relieved Steel Wire for Prestressed Concrete." The capability of the strand to properly develop bond will be certified by the strand supplier.

The encasing material or ducts will be of sufficiently strong polyethylene, conforming to the requirements of ASTM D1248-02, “Standard Specification for Polyethylene Plastics Extrusion Materials for Wire and Cable.” The sheathing should have a minimum thickness of 0.036 in.

6.4.1.5 Strand Anchors

Since bonded tendons are recommended over of unbonded tendons, a standard fixed-end anchorage system might be used because this type of anchor will allow setting the wedges after the strands are inserted into the ducts. The anchor system will conform to the following requirements:

A. An anchorage for bonded tendons tested in an unbonded state will develop 95% of the actual ultimate strength of the prestressing steel, without exceeding anticipated set at time of anchorage. An anchorage which develops less than 100% of the minimum specified ultimate strength will be used only where the bond length provided is equal to or greater than the bond length required to develop 100% of the minimum specified ultimate strength of the tendon.
B. The required bond length between the anchorage and the zone where the full prestressing force is required under service and ultimate loads will be sufficient to develop the specified ultimate strength of the prestressing steel. The bond length is determined by testing a full-sized tendon.
C. If in the unbonded state the anchorage develops 100% of the minimum specified strength, it need not be tested in the bonded state; otherwise, anchorage should be tested accordingly.

The load from the anchorage device will be distributed to the concrete by means of approved devices that will effectively distribute the load to the concrete. These devices shall conform to the following requirements:

A. The average bearing stresses (S_b)on the unconfined concrete created by the device shall not exceed either of the following values:

At service load:

$$f'c \rangle \ S_b = 0.6 \times f'c \times \sqrt{\frac{A'B}{AB}}$$

At transfer load:

$$S_b = 0.8 \times f'c \times \sqrt{\frac{A'B}{AB}} - 0.2$$

where

AB = bearing area of the device,

A′B = maximum area of the device bearing surface that is similar to and concentric with the bearing area of the device

f'c = compressive strength of the concrete at the time of initial prestress

B. Bending stresses in the plate or assembles induced by the pull of the stressing will not cause visible distortion when 85% of the ultimate load is applied as determined by the engineer. Plastic flexural strength of the plates or assembles will be adequate for 125% of the ultimate load. Design will not be based on a yield stress in the plates or assembles greater than 50 ksi.

6.4.1.6 Tendons, Anchors, and Tendon Couplers

Tendon, anchors, and couplers will develop at least 100% of the required ultimate strength of the tendon, with a minimum elongation of 2%. In addition they will withstand 500,000 cycles from 60 to 70% of the required ultimate strength of the tendon without failure or slippage.

All tendons will be identified by reel number and tagged for identification. Anchors will be identified, and the contractor will furnish specimens for testing purposes in accordance with Test Method Tex-710-I. Likewise, the contractor will furnish prestressing tendons including couplers with end fitting attached, to be tested for ultimate strength. These tendons will be 5ft of net length, measured between ends of fittings. If additional testing is required, specimens will be furnished by the contractor without cost.

6.4.1.7 Strand Stressing Equipment

Hydraulic jacks or rams used to stress strands will be equipped with either a pressure gauge or a load cell for determining the applied stress. Gauges will be accurate and must be calibrated and certified by an authorized entity. If a load cell is used, it will be calibrated and will be provided with an indicator showing the equivalent prestressing force applied to the strand. The range of the load cell will be such that the lower 10% of its capacity will not be used in determining the jacking stress. Safety measures will be taken by the contractor and TxDOT to prevent accidents due to possible breaking or slippage of the prestressing tendons during post-tensioning activities.

6.4.1.8 Friction-Reducing Membrane

An effective friction-reducing medium is required for two main reasons: first, to minimize the prestress losses due to subgrade restraint and achieve the desired prestress level with the minimum required post-tensioning strand; and second, to allow the pavement to respond to hygrothermal changes during its life without causing excessive tensile stresses in the pavement. Therefore, the basic requirements for friction-reducing mediums are: (1) to reduce subgrade restraint, (2) to be practical for construction purposes, and (3) to be economical.

Tests have been conducted to determine the efficiency of various friction reducing materials including sand, polyethylene sheeting, paraffin wax, oil, asphalt, and combinations of these materials. Mendoza (Ref 20) conducted friction tests using asphalt and polyethylene with single and double film applications. The results showed that the coefficient of friction obtained with the application of asphalt was twice as high as the one obtained with a single polyethylene sheath. Additionally, it was found that the reduction of friction that resulted from using two sheaths of polyethylene instead of one was not significant and complicated construction tasks instead. Therefore, using a single polyethylene sheath will suffice for the construction of the PCP near Hillsboro, Texas.

The friction-reducing membrane will consist of polyethylene sheeting conforming to the requirements of ASTM D2103-97, “Standard Specification for Polyethylene Film and Sheeting.” A 6-mil-thick membrane, Type 4, should be used. The sheeting will always exceed by at least two feet the width or the length of the concrete strip being poured.

6.4.2 CONSTRUCTION GUIDELINES AND SPECIFICATIONS

The following tasks will be performed during the construction of the PCP. Although the tasks are listed in consecutive order, their sequence may be varied as necessary with prior authorization from TxDOT.

6.4.2.1 Placement of Friction-Reducing Membrane

Once the subbase and leveling layers of the PCP are placed and compacted to specifications, a polyethylene membrane will be placed on the ground and extended longitudinally across the entire length of the PCP slab or slabs to be poured. Longitudinal and transverse overlaps will be of at least 2ft and the sheeting should be at least 2ft wider at each side of the concrete strip being poured. The sheeting will be tacked in place and secured before continuing with other activities.

6.4.2.2 Placement of Transverse Joints

The transverse joints should be placed at predefined locations, considering that the design length of the slabs is 300 ft The joint will be completely fabricated and assembled at the provider’s facilities, including the insertion of the neoprene seal, dowels, deformed Nelson bars, and all the necessary hardware for the anchors. Small steel jumper plates will be welded across the top of the joint to keep the assembly closed with the neoprene seal inside. The joint should be carried to the site ready for installation. After setting it in place, it will be secured to the ground to avoid being displaced when performing other tasks.

6.4.2.3 Prestressing Strand Placement

The longitudinal and transverse post-tensioning strands will be placed at the locations defined by the design and additional recommendations from the plans. The strands will be supported by quality approved chairs. Chairs should be placed carefully so as not to damage the strand ducts or the friction-reducing polyethylene sheet. The distances between tendons will be accurate, and a tolerance of ±1 in. with respect to the specified tendon spacing will be accepted. The tolerance for vertical positioning of strands will be ±¼ in. At intersections of longitudinal and transverse tendons, appropriate chairs will be used.

6.4.2.4 Central Stressing Pocket Form Placement

The material used as form for the central stressing pockets should not react with the concrete; it should be a non-absorptive, strong material that will withstand the imposed forces of placement, vibration, buoyancy, and weight of the plastic concrete during placement. The forms will be properly anchored to prevent movement or misalignment during concrete placement. The sides of the forms will be treated with form oil or other bond breaking coating before concrete reaches the form. Materials that might stain or react with the concrete should not be allowed. After removing the forms and once post-tensioning activities are finished, the pockets should be filled with the same type of concrete mixture used in the pavement, and the concrete should be textured accordingly.

6.4.2.5 Concrete Placement

Paving activities may start after transverse joints, prestressing strands, and central stressing pocket forms are in place for a number of slabs. The paving process is similar to conventional CRCP paving. Special attention should be exercised during placing, vibrating, and finishing of the concrete near transverse joints and forms. Additionally, correct positions of the chairs and tendons will be continuously checked as the paver passes through. Concrete or aggregate trapped in transverse joints should be removed.

Placement of the concrete pavement will be conducted in a predefined sequence to minimize traffic disruptions. TxDOT will provide a detailed plan for paving activities, and the PCP slabs will be paved as needed. Although at press time a tentative paving plan has been drafted, no final decisions have been made. Therefore, the paving sequences are not included in this report. Once the first pavement strip is poured, consecutive strips should be constructed using one of the edges of the previously poured strip as side form. Those edges of hardened pavement strips that will serve as side forms should be prepared to prevent bonding between strips; asphalt might be used as a bond-breaking interface between concrete slabs. The finishing of the concrete will be conducted using the carpet drag method, which is known to provide an adequate final texture to pavements.

6.4.2.6 Concrete Curing

The effect of curing conditions on the performance of the concrete pavement is critical. Strict control of curing methods should be exerted when paving in extremely cold or hot weather conditions. The purpose of the curing membrane is to minimize the evaporation of water from the fresh concrete. The necessary arrangements should be made so that the evaporation rate does not reach a value of 0.20 lb./ft^2/hr. Curing tasks will be conducted according to what is specified in TxDOT's construction specifications, Item 526 (Ref 34). The curing compound shall be applied using an approved and calibrated mechanical-powered pressure sprayer, either air or airless. The curing membrane will be applied immediately after the concrete is finished and will seal the exposed surface with a single coat. Delaying this process may cause a negative impact on the concrete that might result in the development of cracking, spalling, and delamination. If another coat is required, it should be applied instantly. The rate of coverage of the curing compound will be the one recommended by the manufacturer, but will never be less than 1 gallon per 180 ft^2 of concrete area.

6.4.2.7 Post-Tensioning

Post-tensioning of the concrete pavement should be done in at least two stages. The first post-tensioning operation can take place once the concrete has gained sufficient compressive strength. For instance, if the slabs are placed early in the morning, initial post-tensioning should not be conducted more than 8 hours later. A force of 10 kips can be safely applied at anchor zones. If the slabs are placed late in the afternoon, initial post-tensioning may be applied no earlier than 8 and no later than 12 hours after placement. The actual allowable post-tensioning force will be determined by conducting compressive tests of concrete cylinders at the job site. It is recommended that this force should not exceed 15 kips to minimize the level of creep taking place after initial prestressing. Final post-tensioning will be applied once concrete has gained sufficient strength, usually after 48 hours for a concrete mixture prepared with Type I portland cement. The maximum tendon prestress force to be applied will be 46.6 kips.

Each strand will be stressed by jacking it at the central stressing pockets as indicated on the plans. Loading will start at the center strands of each pavement strip and will progress toward the edges by alternatively loading strands on each side of the center strands. Spare jacking equipment must be available at the job site in case malfunction of the equipment occurs. If equipment breaks down and paving operations are stopped, the contractor may be required to set a temporary header and to temporarily post-tension the portion of the slab being poured.

Transverse strands will be stressed in a similar manner used with the longitudinal strands. The prestressing force should be no less than 25 kips and no more than 30 kips if additional pavement strips are to be constructed to the sides. The final prestressing force to be applied once all the strips are placed will be 46.6 kips.

6.4.2.8 Miscellaneous Construction Tasks

Additional activities that need to be performed during construction time include the instrumentation of the PCP slabs to monitor early and long-term performances of the PCP, fresh and hardened concrete testing, measurement of paved concrete, and payment issues. Concrete slabs will be instrumented to monitor the behavior of the pavement at different times throughout its service life. All the instrumentation hardware will be placed where needed on the subbase of the pavement or inside the concrete slabs at various depths. The contractor will be required to use care and make provisions while paving the areas where instrumentations will be set up.

Construction progress will be measured by the completed square yardage of pavement, based upon the dimension stipulated on the plans. The payment for the executed work will be calculated using the unit price, according to bid results. This payment will include full compensation for furnishing, loading and unloading, storing, hauling and handling all concrete materials needed, including the freight involved; for placing and adjusting forms; for mixing, placing, finishing, and curing concrete; for furnishing and installing all materials, including reinforcing steel, prestressing strands, strand ducts, dowels, anchors, couplers, and friction-reducing polyethylene; for all post-tensioning operations; for furnishing and installing all devices for placing and supporting the reinforcement steel, prestressing strands and dowels; and for all the handling, labor, equipment, appliances, tools, and incidentals necessary to complete the work. Expansion joints will be measured and paid conforming to TxDOT's special specification item 4017, "Elastomeric Sealed Expansion Joints for Bridges."

6.4.3 MAINTENANCE AND REPAIR OF PCP

The maintenance of the PCP includes similar activities to the ones performed for conventional concrete pavements, except that for PCP no cracks are supposed to appear in the concrete, given that prestress losses have been within the estimated limits. If prestress losses are greater than the ones calculated, then the minimized effective prestress in the slab will cause cracking of the concrete. In the eventuality of cracks, a good quality epoxy should be used to seal them and minimize the intrusion of contaminated water deep in the concrete.

With regard to the maintenance of the joints, routine maintenance tasks should be performed at regular intervals of time. It is highly recommended that debris removal from the joint is performed every year. The best time to conduct this task is during the cold months of the year because the joints are wide open, and removal of debris is more effective. Additionally, as part of the maintenance tasks, the joint seals have to be visually examined, and any badly deteriorated or broken seal must be promptly replaced to prevent debris from getting deep into the joint.

Finally, because pavements are commonly cored to evaluate the in situ strength of the concrete, it is highly recommended that preventative measures are taken to eliminate the possibility of cutting the post-tensioning tendons. Although the hazard associated with cutting a bonded tendon is greatly reduced compared to an unbonded tendon, it represents a risk for the maintenance crew and for the structural soundness of the PCP. It is recommended that warning signs be placed along the pavement section as a safety measure that will reduce the possibility of doing any inadequate drilling or cutting of the concrete.

6.5 Summary

This chapter covered issues related to material specifications and construction guidelines that will be used for the construction of the PCP in Hillsboro, Texas.

However, these guidelines can be adapted for any PCP located elsewhere. The requirements for the materials forming the substructure and superstructure of the pavement were proposed. Embankment materials were discussed, and procedures for soil selection, compaction methods, and lime stabilization for expansive soils were recommended. Additionally, the possibility of using RAP as an alternative material in the construction of the PCP subbase was discussed based on previous experiences with this type of material in the highway construction industry in the United States and abroad.

The requirements for the production of high-performance concrete were described, and the types of portland cement that might be used were discussed. Additionally, the effect of the coarse aggregate used in the concrete mixture, use of mineral admixtures, and incorporation of fibers in the concrete were discussed. Requirements for post-tensioning steel strands, anchors, transverse joints, strand ducts, and grout material were specified.

Finally, although Chapter 7 covers in detail a monitoring plan for the PCP, a brief description of the activities to be conducted was discussed. It is proposed that the PCP should be instrumented in such a way that variables like horizontal movement, vertical movement, and concrete temperature variation with depth and time are recorded at different occasions during and after construction. By conducting periodic evaluations of these parameters, two main objectives will be achieved: first, predicted design values of concrete stress and slab movements will be verified and calibrated; and second, the data collected from the instrumentations will allow a rational rating of the condition of the PCP and its performance. The results obtained from all the evaluations will enrich the design and construction of new PCP sections and will promote this type of paving technology.

7. MONITORING PLAN

7.1 Introduction

Current design of prestressed pavements is conducted using mechanistic approaches that allow effective comparison of predicted values of concrete stresses and slab movements to real measured values. In PCP design these two parameters are estimated using models that consider the effect of seasonal and daily temperature cycles, long-term effect of shrinkage, creep, and traffic loads. Therefore, after design and construction are finished, it is very important to have a monitoring plan for the pavement that will start at construction time and will last for the entire service life of the facility.

The following pages in this chapter describe a monitoring plan for the PCP section that will be constructed in Hillsboro, Texas. Various activities to be performed, including quality control and quality assurance (QC/QA) of materials, routine visual inspections, and evaluation of the structural condition of the pavement, are described. Likewise, recommendations are provided for both short-term and long-term instrumentations of the pavement.

7.2 Objective of the Monitoring Plan

In any construction project where research is applied and implemented, it is advisable to develop a monitoring plan to check the conditions of the project periodically over time. Therefore, a monitoring plan for the PCP section to be constructed in Hillsboro is presented here. The plan attempts to serve basic research purposes including the following:

1. Verify and calibrate predicted design values of concrete stress and slab movement.

2. Check the performance of the PCP in a rational way that will allow an objective evaluation of the evolution of the project.
3. Enrich the design, construction, and monitoring phases of PCPs so that this paving technique is promoted.

The monitoring plan includes the development of QC/QA activities during the construction stage of the PCP, recommended instrumentation of PCP slabs, planning of periodic condition surveys, and the evaluation of the structural condition of the pavement.

7.3 Quality Control/Quality Assurance (QC/QA) Tasks

Although the design stage is the spinal cord of any engineering project, the construction phase is fundamental to achieving the desired quality of the project. Likewise, the quality of the materials produced or used in the project has a great influence on the ultimate performance of any structure. With regard to the PCP to be constructed in Hillsboro, the construction steps should closely follow the construction specifications provided in Chapter 6 of this report. Additionally, care should be exerted during the construction of the PCP, as commonly done for conventional paving projects such as CRCPs. Paving tasks are very similar for these two types of pavements, with the difference that the PCP has long-spaced steel transverse joints, and special care should be taken when paving or vibrating the concrete near the joints.

Before pursuing this QC/QA concept any further, it is important to differentiate between the two terms, which sometimes are mistakenly used interchangeably. Quality control (QC) is represented by all the actions conducted by a manufacturer or contractor to provide control of activities being done or goods being provided, so that the specified minimum standards of good practice for the work or product are followed and met. On the other hand, quality assurance (QA) is associated with the procedures undertaken by an owner or client to assure that what is being done or is being provided is in agreement with the standards of good practice for the work.

As can be seen, although the QC and QA concepts seem similar, they have a basic difference. This difference resides in who is in charge of performing the quality testing procedures and carrying out tests. QC is performed by the contractor, to make sure that his product or service satisfies the quality standards, whereas QA is the done by the owner to verify that the contractor is working to achieve the quality standards. Therefore, it is common to hear that QC/QA activities are performed during the execution of a construction process and until it is finalized. In pavement construction QC/QA tasks involve sampling and testing of the materials used for the construction of the structure to measure their compliance with standards that assure an adequate performance. For the construction of the PCP, the standards of quality are expressed in terms of construction specifications and are contained in Chapter 6 in this report.

7.3.1 PCP FEATURES SUBJECT TO QC/QA

There are various quality measures of the PCP that have to be controlled to guarantee that construction will be performed as designed and specified. These characteristics serve as reliable predictors of the performance of the pavement. Examples of these quality measures are construction materials, concrete strength, and PCP thickness. The next step in the QC/QA process is defining a sampling and testing program for the quality measures of the PCP subject to verification.

7.3.2 SAMPLING PROGRAM

The concreting of a PCP is conducted in a similar way to that of a CRCP; both techniques require paving to be conducted continuously, only for the PCP, transverse joints delimit contiguous slabs. The PCP slabs are constituted of many components: cement, aggregates, steel strands, strand ducts, transverse joints and hardware, etc. Therefore, the overall quality of the pavement relies on testing adequate samples of each of those components. Finished products or materials, including polyethylene sheeting, steel strands, strand anchors, transverse joint components and hardware, strand ducts, and grout, will be sampled as production lots become available during construction time. Additionally, providers of these products should provide quality statements that certify that the products comply with the required ASTM, AASHTO, and TxDOT specifications before any construction activity takes place. This will serve as a preliminary QC that will assure the good quality of the products that will be used for the construction of the PCP.

With regard to the sampling and testing of the concrete, the provider(s) shall furnish reports of the quality of the various components of the concrete mixture that will be used, including cement, aggregates, admixtures, and water. These reports will be rendered to TxDOT before any concrete is supplied, and periodic testing will be done at regular intervals at the request of the state agency.

7.3.2.1 Concrete Stratified-Random Sampling

The concrete mixture delivered to the site will be sampled and tested comprehensively to ensure its quality. As done during construction of conventional concrete pavements, samples will be independent and stratified-random, instead of simply random. Using stratified random sampling provides more refined samples and ensures that they will not concentrate in a single area. The stratified random sampling has been successfully used in various pavement projects in Texas, where the concepts of lot and sublot were introduced to facilitate the understanding of the concept.

7.3.2.2 Lots and Sublots

Sampling of pavements is done by lots. A lot is a determined quantity, usually an area, of placed pavement to which an acceptance quality procedure is applied. For conventional concrete pavements, a lot is generally represented by one day's production. Sometimes, a lot can be defined by stop-and-go operations during construction or by changes in equipment, materials or personnel (Ref 52).

A sublot is a fraction of a lot, and its size should be consistent so that a lot is formed by various sublots. Usually, the number of sublots per lot varies from three to five, but four or more sublots per lot generally assure a good approximation to a normal sampling distribution about the sample means (Ref 52). In stratified random sampling, at least one random sample should be taken for testing from each identified sublot, with each member of the population having an equal probability of being sampled, which makes the procedure statistically valid (Ref 53).

The recommended sample size for QA criteria is at least four stratified random samples from a lot size of a single production day (Ref 52). QC sample sizes depend on economic considerations. The concept of stratified random sampling in highway construction is shown in Figure 7.1, in which at least one sample is taken from each sublot at a random location. This concept can be applied to any type of concrete sampling, including cylinders, beams, cores, concrete slab thickness measurement, etc.

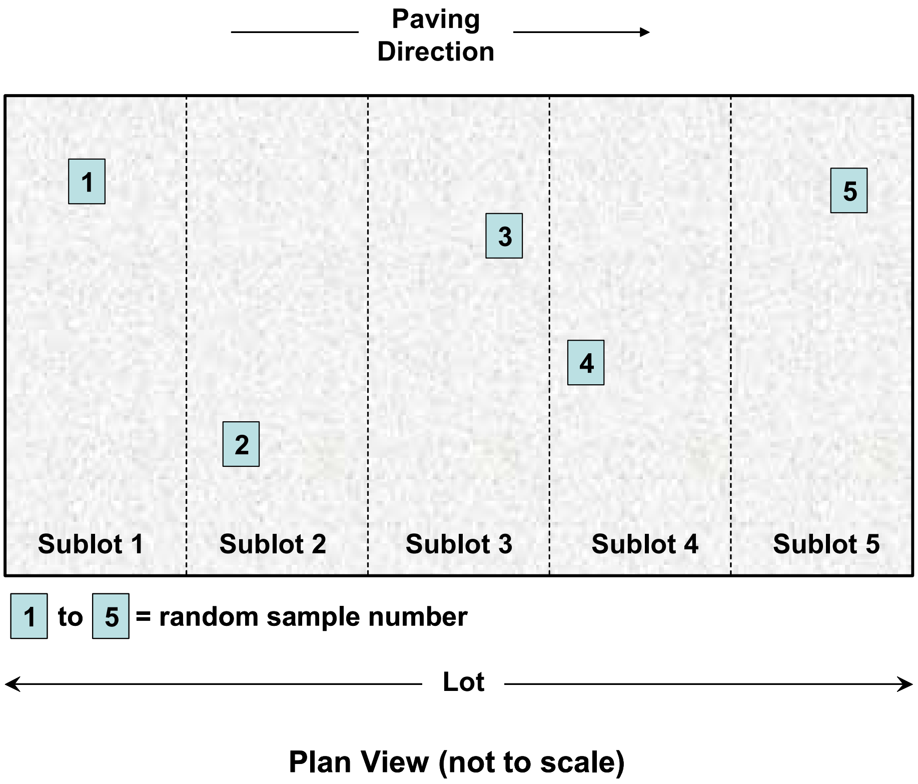


Figure 7.1 Example of stratified random sampling in pavement construction

For the PCP to be constructed in Hillsboro, concrete slabs will vary in width from 22 to 25ft and will all have a length of 300 ft Thus, the areas of the slabs will vary from 6,600 ft^2 (733 yd^2) to 7,500 ft^2 (833 yd^2), which are reasonable sizes that could represent a sublot. Figure 7.2 illustrates how a PCP slab could be, for practical sampling purposes, considered as one sublot, and thus a lot (approximately one day's paving work) will be formed by a group of five sublots or slabs. Applying the criterion previously described for conventional concrete pavement sampling to the sampling of the PCP, it will be necessary to sample at least three out of five slabs.

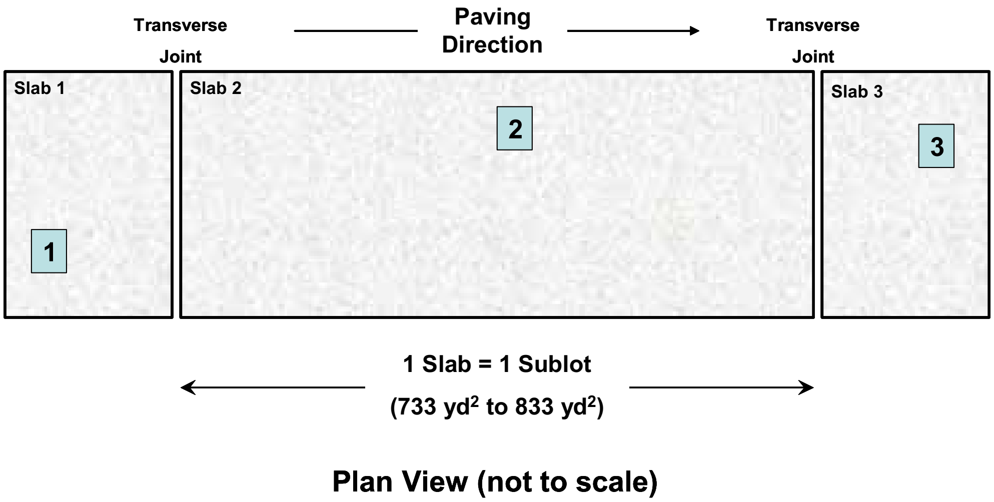


Figure 7.2   Representation of a sublot for sampling of the PCP

7.3.3 AS-DESIGNED TARGET VALUES AND VARIABILITY

It is well understood that for any project compliance with the specifications cannot be based only on a single design value or quality measure. The existence of variability has to be considered in various aspects of the design and construction stages of a project. Variability occurs in construction materials, in construction equipment, in construction procedures, in human operation, etc. Therefore, if variability of the construction is to be judged by testing, it has to be acknowledged that variability is an inherent characteristic of the tests as well (Ref 54). The variability of the tests is implicit in testing procedures, sampling methods, testing equipment, and testing personnel. Variability is commonly taken into account in the specifications, and when the specifications are rigorous, they only allow a small amount of variability around the target value.

Data collected in testing procedures are described by their statistics. The target value for the mean of the samples tested should be the design value for that quality measure. The quality measure is a variable, X, for which central tendency and dispersion measures are calculated and used to determine acceptance or rejection in accordance with the quality standards established in the project specifications. Those statistics are inferences about the properties of the entire population. Equations 7.1 and 7.2 represent the estimation of the sample mean, $\overline{X}$, and variance, S_X^2, of the population mean, μ_X, and variance, σ_X^2, respectively.

$$\overline{X} = \frac{\sum_{i=1}^{n} X_i}{n} \qquad (7.1)$$

$$s_x^2 = \sum_{i=1}^{n} \frac{\left(X_i - \overline{X}\right)}{n-1} \qquad (7.2)$$

Another useful measure of the dispersion of any random variable is the coefficient of variance, CV, represented by Equation 7.3.

$$CV = \frac{\sqrt{s_x{}^2}}{\overline{X}} \qquad (7.3)$$

Quality measures are random variables and are assumed to be normally distributed. In order to compare test results to design target values, it is necessary to test the statistical hypothesis that the mean and variance obtained from the tests correspond to the population. The population of each of the quality measures is described by its mean and variance target values established in the project specifications. The statistical hypothesis testing is commonly accomplished by the t-test and the F-test. These tests provide a basis for acceptance or rejection of the contractor's tests results. The target values established in the project specifications define the quality levels for which the owner or agency is willing to pay 100% of the bid price (Ref 54).

Table 7.1 presents a list of some common concrete pavement test parameters with their allowed coefficients of variance. These values have been used in actual paving projects and are published in the literature (Ref 53). These values are an indication of what has been achieved in the past in terms of variability and could serve as a guide for their application in the PCP project in Hillsboro.

Table 7.1 Coefficients of variance for some quality measures for conventional concrete pavements

Quality Measure	Coefficient of Variance
Thickness	4%
Concrete Modulus of Rupture	10%
Concrete Elastic Modulus	10%

7.3.4 CONCRETE TESTING

There are different tests that should be performed to evaluate the quality of the concrete mixture delivered to the site and again later when concrete has hardened. These tests are usually conducted for QC/QA purposes of conventional concrete pavements and include the following:

1. For fresh concrete: consistency (slump), thickness and temperature, air content and unit weight, and chloride content
2. For hardened concrete: concrete strength, concrete modulus of elasticity, concrete thermal expansion, petrographic analysis, and durability

Additionally, once the concrete is hardened nondestructive testing is usually conducted including deflection and vibration tests (pulse velocity). The following pages briefly describe these tests and their relevance in concrete pavement applications.

7.3.4.1 Concrete Consistency

This test is conducted right after the concrete arrives at the construction site. This is a very common test that is performed to check the consistency and workability of the concrete. Different types of structures require different slump values; TxDOT's specifications (Ref 34) state that the slump for concrete pavement should have a minimum value of 1 in. and a maximum of 3 in. For slipform paving applications, the desired slump value is 1½ in.; this means that concrete applied with this technique should have a low slump for adequate forming.

The slump test, ASTM C143, "Test Method for Slump of Portland Cement Concrete," should be started within 5 minutes after the sample has been obtained and should be completed within 2 ½ minutes. Additional concrete consistency tests include the British compacting factor test; Powers remolding test; German flow table test (DIN 1048); Vebe test; ball penetration test; ASTM C360, "Test Method for Ball Penetration in Fresh Portland Cement Concrete"; and the inverted slump cone test, ASTM C995, "Test Method for Time of Flow of Fiber-Reinforced Concrete through Inverted Slump Cone." Although the Vebe test is especially applicable to stiff and extremely dry mixes, it has not been widely used in Texas.

7.3.4.2 Pavement Thickness and Temperature

The measurement of pavement thickness might be performed by destructive and nondestructive methods. Destructive testing consists of measuring the pavement thickness directly from core samples. Destructive testing is not recommended for PCP pavements. Thickness estimation from nondestructive techniques involves seismic methods (wave propagation) for hardened concrete and simple concrete penetration or probing for fresh concrete.

Temperature measurement is very important when related to concrete. Most specifications limit the temperature for fresh concrete. PavePro, developed by Schindler (Ref 55), is a computer program that predicts the time history of concrete temperatures and zero-stress temperatures from the ambient air temperatures. The program is currently being calibrated and validated with field data from various locations in Texas (Ref 56). This program will be useful for controlling concrete temperatures during construction of the PCP.

To measure concrete temperatures in the field, glass or armored thermometers are used. The thermometer should be accurate to ±1°F and should remain in the concrete for at least two minutes or until the reading stabilizes. Electronic temperature meters or thermocouples could also be used for very good and precise results. The temperature measurement should be completed within five minutes and should conform to ASTM C1064, "Test Method for Temperature of Freshly Mixed Portland Cement Concrete."

7.3.4.3 Air Content and Unit Weight

Concrete air content can be measured using various methods, including the pressure method ASTM C 231, "Test Method for Air Content of Freshly Mixed Concrete by the Pressure Method"; the volumetric method ASTM C173, "Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method"; and the gravimetric method ASTM C138, "Test Method for Unit Weight, Yield, and Air Content (Gravimetric) of Concrete."

The pressure method is based on Boyle's Law, which relates pressure to volume. Using this method requires the use of correction factors to obtain the correct entrainment of air. Therefore, the instrument has to be calibrated for a given elevation above the sea level. This test is the one that takes the least amount of time of all the methods. The volumetric method requires removal of air from a known volume of concrete by agitating the concrete in an excess of water. This method is not affected by atmospheric pressure, but care must be taken to agitate the sample sufficiently to remove all air.

The gravimetric method is often used when laboratory control is exercised because mixture proportions and specific gravities of the ingredients must be accurately known. The equipment used is the same for the measurement of concrete unit weight. Significant changes in unit weight can be a convenient way to detect variability in air content. Concrete unit weight can also be measured using nuclear methods, as indicated in ASTM C1040, "Test Methods for Density of Unhardened and Hardened Concrete in Place by Nuclear Methods."

7.3.4.4 Chloride Content

The chloride content of concrete and its ingredients should be checked to assure it is below the limit necessary to prevent corrosion of reinforcement (or post-tensioning) steel. An approximation of the water-soluble chloride content of fresh concrete can be obtained using a procedure initiated by the National Ready Mix Concrete Association (NRMCA)(Ref 57). The procedure gives a fair and quick approximation, but it should not be used for compliance purposes.

7.3.4.5 Concrete Strength

Verification of concrete strength can be done by testing cylinders, beams, and core samples. Although for conventional pavements core samples are trustworthy for faithfully representing the properties and conditions of the in-place pavement, for PCP pavements this sampling method offers some hazards, mainly because the drilling process might cut the post-tensioned steel strands if the precise location of the tendons is not known.

To test concrete strength, compressive, flexural, and splitting tensile strength tests are commonly used. For pavements, where critical stresses are of tensile nature, the most suitable tests are flexural strength or splitting tensile strength. Generally, concrete cylinders and extracted cores are used for compressive and splitting tensile strength testing, whereas beams are tested for flexural strength. Additionally, the maturity of concrete, which is a nondestructive testing (NDT) method that infers the strength of concrete based on age and temperature, may be used to estimate the compressive strength of concrete.

The procedures for preparation of specimens of fresh concrete are described in ASTM C 31, “Practice for Making and Curing Concrete Test Specimens in the Field.” Cylindrical specimens have a length equal to twice the diameter. Standard dimensions are 6 by 12 in., but 4 by 8 in. specimens are sometimes used. Beams for determination of flexural strength are specimens cast and hardened in horizontal position. The length will be at least 2 in. greater than three times the depth as tested. The ratio of width to depth as molded will not exceed 1.5. The standard beam will be 6 in. by 6 in. Cores or beams extracted in-situ are obtained from hardened concrete, as specified in ASTM C 42, “Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete.”

In concrete pavements, although compressive strength tests are done extensively, tensile strength is definitely the most important characteristic to be monitored. Although there is no standard procedure for a direct determination of the tensile strength of concrete, splitting tensile strength and flexural strength are used as good indicators of this parameter.

The splitting tensile strength (STS) test is a standardized procedure found in ASTM 496, “Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens.” This test method was developed in Brazil, and it is sometimes called the Brazilian Test. To test a specimen, the cylinder is placed on its side, and a diametrical compressive force along its length is applied. The specimen is loaded until failure, which occurs along a vertical plane running through its axis. The tensile strength is calculated as shown in Equation 7.4.

$$\mathrm{STS} = \frac{2\mathrm{P}}{\pi\,\mathrm{Ld}} \qquad (7.4)$$

where

STS = splitting tensile strength, psi
P = maximum applied load, lb
L = length of the specimen, in.
d = diameter of specimen, in.

The flexural strength of concrete is obtained from testing beams by either ASTM C 78, “Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading),” or ASTM C 293, “Test for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading).” The difference between the two procedures is the location of the load applied on the beam.

At times, when it is not possible to test specimens using either STS or flexural strength, compressive strength is used as a surrogate test to indirectly estimate the tensile strength of the concrete. The ratio of STS to compressive strength varies from 0.08 to 0.14, and the ratio of flexural strength obtained by third-point loading to compressive strength varies from 0.11 and 0.23. For center-point loading these ratios are even higher for flexural strength to compressive strength (Ref 58).

An alternate test for the estimation of concrete strength through NDT is the maturity method, which relates to the development of strength of a cementitious mixture depending on cement hydration and pozzolanic reactions. Concrete maturity depends on the curing history of concrete, which is given by its temperature and moisture content through time. The procedure for estimating concrete strength using the maturity method is described in ASTM C 1074, "Practice for Estimating Concrete Strength by the Maturity Method." The maturity function is a mathematical expression that accounts for the combined effects of time and temperature on strength gain of concrete. The maturity function converts the temperature history to a maturity index that indicates the strength evolution. This function is specific for each concrete mixture. The primary assumption of this method is that specimens of a given concrete mixture will achieve equal strength values if their maturity indices are equivalent. A shortcoming of the method is that it has to be supplemented by other indicators of the potential strength of the concrete mixture, which in most of the cases is destructive testing.

7.3.4.6 Concrete Modulus of Elasticity

ASTM C 469, "Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression," is the standard test to estimate this parameter. It is a compressive procedure applied to concrete cylinders and cores that consists in determining the stress-to-strain ratio (chord modulus of elasticity) and the lateral-to-longitudinal-strain ratio (Poisson's ratio). The chord modulus is given by the slope of a line drawn between two points on the stress-strain curve. The points correspond, one to a strain of 50 µin./in., and the second, to 40% of the ultimate load. The obtained modulus of elasticity corresponds to the approximate average concrete modulus throughout a customary working stress range. The chord modulus is calculated using Equation 7.5 (Ref 46):

$$E = \frac{S_2 - S_1}{\varepsilon_2 - 0.000050} \qquad (7.5)$$

where

E = chord modulus of elasticity, psi

S_2 = stress corresponding to 40% of ultimate load, psi

S_1 = stress corresponding to a longitudinal strain, ε_1, of 50 µin./in., psi

ε_2 = longitudinal strain produced by stress S_2

7.3.4.7 Concrete Thermal Expansion

Concrete expands as temperature rises and contracts as temperature falls, although it can expand to some extent as free water in the concrete freezes. Temperature changes in concrete are mainly due to climatic conditions and cement hydration. An average value for the coefficient of thermal expansion of concrete is about 5.5 millionths per degree Fahrenheit (5.5 µin./in./°F); however, values ranging from 3.2 to 7.0 have been observed. This amounts to a length change of 2/3 in. for 100ft of concrete subjected to a rise or fall of 100°F.

Thermal expansion and contraction of concretes varies with factors such as coarse aggregate type, cement content, water-cementitious materials ratio, temperature range, concrete age, and relative humidity. However, as it was observed of the design of the PCP in Chapter 5 and the instrumentations of the PCP in McLennan County in Chapter 4, aggregate type is the factor that has the greatest influence.

Due to the influence of all the variables on the thermal coefficient of concrete and their interactions, there is no standard test procedure purposely developed for its estimation. However, for practical purposes, tests are conducted preparing cores according to ASTM C 341, "Test Method for Length Change for Drilled or Sawed Specimens of Cement Mortar and Concrete."

7.3.4.8 Petrographic Analysis

This type of analysis is done in forensic studies when poor performance of the concrete has occurred unexpectedly or when a post-service coarse aggregate analysis needs to be done. The tests commonly used for petrographic analysis include ASTM C295, “Practice for Petrographic Examination of Aggregates for Concrete,” which can be helpful in identifying potentially reactive aggregates. ASTM C856, “Practice for Petrographic Examination of Hardened Concrete,” aids in determining the constituents of concrete, concrete quality, cause of inferior performance, distress, or deterioration. The results of the test facilitate the estimation of the future behavior and structural safety of concrete elements. Components that can be analyzed using petrographic analysis include paste, aggregate, mineral admixture, and air content; frost and sulfate attack; alkali-aggregate reactivity; degree of hydration and carbonation; water-cement ratio; bleeding characteristics; fire damage; scaling; poputs; effect of admixture; and several other aspects (Ref 44).

7.3.4.9 Durability

Although there is still controversy about the durability concept of concrete, it can be said that it refers to the ability of the material to resist deterioration caused by environmental conditions and service loads. Properly designed and constructed concrete should last for a very long time without significant distress throughout its service life (Ref 44). There are various tests that allow a determination of the durability of concrete. ASTM C666, “Test Method for Resistance of Concrete to Rapid Freezing and Thawing,” ASTM C671, “Test Method for Critical Dilation of Concrete Specimens Subjected to Freezing,” and ASTM C682, “Practice for Evaluation of Frost Resistance of Coarse Aggregates in Air-Entrained Concrete by Critical Dilation Procedures”; are examples of these tests. Additionally, ASTM provides testing procedures related to the effects of deicing salts, corrosion, alkali-aggregate reaction, sulfate resistance, abrasion resistance, etc. on concrete and reinforcement steel.

7.3.5 NONDESTRUCTIVE TESTING

There are a number of nondestructive testing procedures that are applicable to pavements. These tests have been widely used to evaluate the relative strength of the concrete and in some cases the layers underneath the pavement. Among the most widely used tests are the rebound, penetration, pullout, vibration tests, and deflection tests. Other techniques make use of X-rays, gamma radiography, microwave absorption, and acoustic methods. However, they are not commonly used in the paving industry. Although all methods have their limitations, each one can be well correlated to different concrete strength parameters.

The most common testing methods of those previously mentioned are the rebound method and the vibration test (pulse velocity). The rebound method is conducted using a Schmidt rebound hammer like the one shown in Figure 7.3. This hammer is a surface-hardness tester that gives quick inexpensive characteristics of the concrete in terms of uniformity and provides only the relative strength of the concrete. Therefore, if this device is used for concrete pavement evaluation, it must be calibrated using real strength values obtained from concrete samples, such as cylinders or cores.

Figure 7.3 Schmidt hammer used for the concrete rebound test

The testing device consists of a plunger that is hammered onto the concrete by a calibrated spring. A number read from the hammer's scale gives an indirect value of the compressive strength of the concrete. The results obtained with the hammer have to be corrected by various factors, including concrete smoothness; element size, shape, age, moisture, and type of aggregate. ASTM C805, "Test Method for Rebound Number of Hardened Concrete," provides guidelines for the development of the test and for correction of the concrete hardness number.

The vibration test method or pulse velocity test consists of the determination of the strength of the concrete by measuring the travel speed of sound in the structure. Resonant frequency induced to the concrete and the travel time of short pulses of vibration aid in determining the quality of the concrete. High velocities are indicative of good concrete, while low velocities pinpoint poor concrete. This technique is used to locate discontinuities in concrete elements. Additionally, this technique allows to infer the modulus of elasticity of the concrete and to locate the presence of cracks. ASTM C215, "Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens," provides guidelines for the determination of discontinuities in a concrete element.

7.4 Evaluation of the Structural Capacity of the Pavement

The evaluation of the structural capacity of pavements can be performed either by destructive testing (e.g., trenching, coring, sampling) of the pavement structure or by conducting nondestructive testing. This section describes only the nondestructive testing devices that are regularly used and how the results obtained from each device are interpreted and judged.

7.4.1 FALLING WEIGHT DEFLECTOMETER (FWD)

The first falling weight deflectometer (FWD) was developed in Europe in the 1970s and was later used in Canada and the United States in the mid- to late- 1980s (Ref 59). Its domestic use increased during the long-term pavement performance (LTPP) study conducted in the Strategic Highway Research Program (SHRP). The FWD is an impulse load deflection measurement device that provides discrete load-deflection data that are used in structural evaluation of pavements. A falling mass dropped onto a rubber load plate causes micro deflections that are measured by geophones and then recorded in electronic files. The load applied by the FWD can be varied by simply adjusting the falling mass and the height of the drop, so that a spectrum of load-deflection relationships can be obtained for a given pavement structure. Vertical peak deflections are measured by the FWD in the center of the loading plate and at varying distances away from the plate. These deflections can be plotted as deflection basins or bowls that are useful in the interpretation of the absorption of impact by the pavement. Figure 7.4 displays an image of an FWD trailer, showing its loading plate and geophones or sensors that measure the deflection caused by the load.

Figure 7.4 FWD loading plate and arrangement of geophones

The principle of the FWD is based on the assumption that the transient force impulse created by the device in the pavement closely approximates the pulse created by a moving wheel load. It has been observed that FWDs produce more realistic load-deflection results than either static or steady-state vibratory loading devices, and thus it has been widely used for testing the structural adequacy of pavements. The electronic information generated by recent FWD models allows pavement engineers to easily interpret deflection basins generated by a specific load. Additionally, these electronic files can be used to back-calculate the elastic properties of a known pavement structure, as described in Chapters 4 and 5 in this report.

7.4.2 ROLLING DYNAMIC DEFLECTOMETER (RDD)

The rolling dynamic deflectometer (RDD) is a truck-mounted device that measures continuous load-deflection profiles of pavements. This equipment has been widely used in Texas for the evaluation of highway and airport pavements. Although the RDD provides continuous load-deflection data, it cannot give the data provided by the FWD. Both devices are supplementary to each other. While the RDD allows for the efficient location of weak localized areas along a pavement, the FWD follows up and helps to determine the real cause of the problem. The stiffness of a layer in the pavement or the texture of the riding surface might cause noisy signals in the RDD signature profile. Figure 7.5 displays an image of the RDD and its main components.

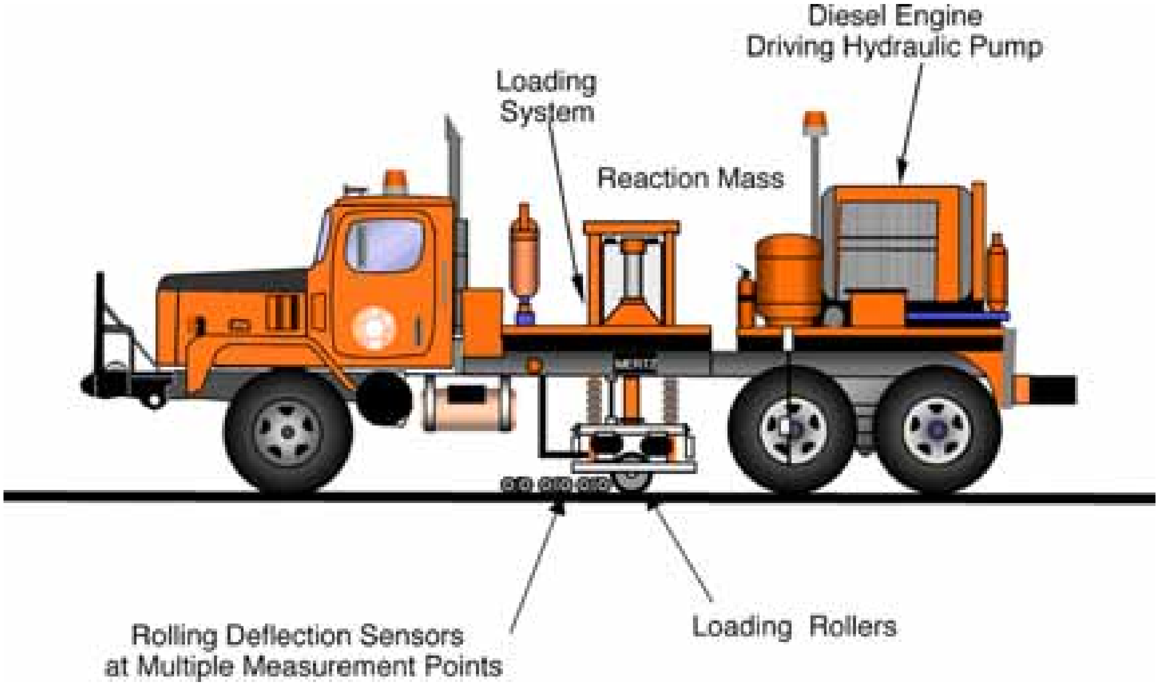


Figure 7.5 Image of the RDD device used for pavement evaluation

The RDD uses the same principle used in seismic analyses, where induced dynamic loads in the pavement generate waves that are propagated at various speeds and frequencies that help determine the continuous load-deflection behavior of the structure. The RDD has a servo-hydraulic vibrator with a 7,500-lb. reaction mass that is energized hydraulically to generate vertical dynamic forces as large as 70,000 lb. peak-to-peak over a frequency range of about 5 to 100 Hz.

7.5 Instrumentation Tasks

The instrumentation of the PCP section will allow monitoring of the behavior of the pavement at defined periods of time. The data collected from the instrumentations are really valuable and in combination with QC/QA tasks will give a comprehensive understanding of the performance of the section. The activities that could be performed for the instrumentation are countless. However, this section describes the most important activities that shall be conducted in order to achieve a good monitoring plan starting at construction time and continuing throughout the life of the pavement. To accomplish an effective instrumentation plan, activities performed for previous instrumentations are discussed and then improved by recommending the use of reliable state-of-the-art equipment.

7.5.1 BACKGROUND ON PREVIOUS INSTRUMENTATIONS

The instrumentation of pavement projects originated with a need to investigate how new materials and construction techniques performed over time compared to pavements constructed using old customary procedures. Today, almost every single pavement project that is constructed is instrumented to some degree. As more innovations are introduced in the paving industry, more sophisticated instrumentations are conducted so that a better understanding of pavement behavior is achieved.

For PCPs constructed over the world, instrumentations have included simple visual inspections or condition surveys, and also more rationalized tasks. For instance, the PCP projects constructed in the United States in Virginia, Pennsylvania, Mississippi, and Arizona, and later in Texas in McLennan County, were effectively instrumented, and the procedures conducted at those times have served as guidelines for new instrumentations. In all these domestic PCP projects, items monitored included ambient temperature, concrete temperature, concrete strain, horizontal movement, vertical movement (curling), and joint width.

All the projects except the one in Texas reported troubles in recording concrete strain using embedded gauges. An explanation for this failure relies on the fact that in all four cases the gauges were installed using the same approach. However, the reports from the Virginia and Mississippi projects stated that the gauges were “not sufficiently rugged” for field applications.

7.5.1.1 Prestressed Pavement in Virginia

This pavement was constructed in 1971. It consisted of 6 prestressed slabs varying in length from 400ft to 700ft In this project, ambient temperature was recorded continuously during construction. Concrete temperature was measured using thermocouples installed at four depths at four locations in three slabs. Although vibrating wire gauges were installed at various depths, they did not produce usable data. Therefore, measurements were manually taken using a multi-position strain gauge or demec bar. A total of 10 locations were measured in four slabs.

Horizontal movement was measured using a pipe-wire-scale with an accuracy of 20 mils, and this was done at seventeen locations in all slabs. Vertical movement was measured with a profiler beam at ten locations in four slabs. Likewise, a clinometer measured the slope change along profile lines (Ref 11). Finally, joint widths were measured across gap slabs using calipers.

7.5.1.2 Prestressed Pavement in Mississippi

The project in Mississippi was constructed in 1976. It consisted of 58 slabs with a length of 450 ft Ambient temperature was continuously recorded, and concrete temperature was measured for two slabs using type ETH-50D gauges. Concrete strains were obtained with embedded gauges at three depths in four slabs. Unfortunately, no usable data were generated, and thus strain gauged bars were used instead at twelve locations in four slabs.

Horizontal movement was monitored using a pipe-wire-scale at both ends of three slabs only. Vertical movement was measured using “survey equipment,” and elevation change was obtained for 3-ft intervals along slab ends and at 5-ft intervals in the interior of two slabs. Joint width was measured using dial calipers accurate to 1 mil in most of the slabs. Other testing procedures included estimation of tendon elongation, modulus of elasticity of the concrete, dynaflect deflection, PCA road meter, and skid resistance.

7.5.1.3 Prestressed Pavement in Arizona

The prestressed pavement in Arizona was built in 1977 and was formed by 30 slabs, 400ft in length. For this project ambient temperature was recorded using "thermistors" with an accuracy of 1°F. Thermistors were installed in three slabs near the slab ends and in one slab at the midspan. Concrete strains were measured by embedded Ailtech CG129 gauges at three depths in three slabs at the ends and one slab at midspan. Data recorded by the gauges were described as questionable.

Horizontal movements were monitored in one slab using only "distance measuring equipment" that allowed to record data with an accuracy of 1 mil. Vertical movements were measured for two slabs with what was called "iron pin," which was a device driven all the way through the subgrade that gave readings with an accuracy of 1 mil. Joint widths were measured for five slabs with an extensometer bar accurate to 1 mil. Visual crack surveys were done for all the slabs.

7.5.1.4 Prestressed Pavement in McLennan County in Texas

The instrumentation conducted in Texas for the project in McLennan County was divided into two categories, short-term and long-term instrumentation. The short-term instrumentation consisted of recording continuous electronic information, including ambient and concrete temperatures, concrete strain, and horizontal and vertical movements, and of mechanical instrumentation to back up electronic information. The instrumentation was done on various pavement slabs. Recorded data included installation of thermocouples in the concrete at three depths. Strain gauges measured concrete strain at four depths and at eighteen locations. Horizontal and vertical movements were recorded using displacement transducers at five locations. Likewise, ambient temperature was recorded at one location using a thermocouple. The data from the instrumentation were recorded over several daily temperature cycles. This process required using an automated data acquisition system.

The long-term instrumentation consisted in taking measurements by mechanical methods that would compare to those backing up the electronic measurements during the short-term instrumentation. Joint widths were measured at all joints using an extensometer. Curling of the slab corners was measured using survey equipment. Load transfer capabilities of the transverse joints were estimated using a Dynaflect apparatus. Deflection readings were taken on two or more pavement slabs to record the deflection characteristics at several locations throughout the slabs. The latest instrumentation activities were conducted in April and June 2001. These instrumentations are detailed in Chapter 4 in this report.

7.5.2 PROPOSED INSTRUMENTATION FOR THE PCP IN HILLSBORO, TEXAS

The instrumentation for the PCP to be constructed in Hillsboro, Texas, includes a number of tasks that will allow monitoring the section during construction time (short-term) and over the entire life of the pavement (long-term). Additionally, before any construction takes place, it will be necessary to do a broad monitoring plan of climatic conditions for the expected time when construction will be initiated. To accomplish this, a helpful tool that can be used is program PavePro (Ref 55). Among the most critical conditions that have to be evaluated before any concrete is poured are the following:

1. Predicted water evaporation rate greater than 0.2 lb./sq.ft/hr.
2. Predicted substrate temperature of 125 °F or higher
3. Forecasted daily temperature differentials of more than 25°F anticipated for the 24-hr. period after concrete placement

PavePro is an effective tool that aids in predicting these critical situations and helps contractors and state agencies to take special precautions if any of these situations arise.

Before getting into the details of the instrumentation devices to be used and recommending a monitoring scheme, it is important to enlist some of the objectives of the short-term and long-term instrumentations.

7.5.2.1 Short-Term Instrumentation

Objectives of the short-term instrumentation include, but are not limited to, the following:

1. Record ambient temperatures, which are known to have a critical impact on the development of temperature gradients through the depth of the slab. Temperatures at top, mid-depth, and bottom should be continuously recorded and correlated to slab horizontal and vertical movements.
2. Record longitudinal movements at the time of post-tensioning, which will allow for estimation of the actual coefficient of friction between the slab and the subbase.
3. Measure tendon elongation to verify that the required prestress force is applied on the concrete.
4. Detect the appearance of any cracks. Condition surveys have to be continuously done during the first 72 hours after concrete is poured, and early detection of cracks, especially in the longitudinal direction, will determine if an increment in post-tension force is necessary during the first 24 hours.
5. Concrete compressive strength will be evaluated with cylindrical specimens to define the maximum prestressing force that can be applied to the concrete at the anchor zone. The concrete compressive strength will be evaluated at different ages to monitor its adequate evolution.

7.5.2.2 Long-Term Instrumentation

The main objectives of the long-term instrumentation are related to the evaluation and validation of the performance of the pavement throughout its life. Periodic instrumentations will require at least the following activities:

1. Detailed condition surveys will show if any distresses develop and to what extent. Evaluation of the condition of the transverse joints and the neoprene seals is a critical factor that affects the overall performance of the pavement.
2. Measurement of horizontal and vertical slab movements and joint widths. Measuring horizontal movements will help determine if friction conditions between the slab and the subbase are changing with time. As for the vertical movements, it will be possible to determine if recorded values are within acceptable limits so that no excessive stresses are developed at slab ends and also to determine if the riding quality of the pavement in maintained over time.
3. Estimation of the load transfer efficiency at transverse joints will help corroborate that the joints are behaving as they were designed.
4. Measurement of the structural capacity of the pavement using both discrete (FWD) and continuous (RDD) methods to obtain the load-deflection basins at different times is fundamental to determine if the soundness of the pavement varies within acceptable limits.

7.5.3 INSTRUMENTATION DEVICES AND PARAMETERS TO BE MONITORED

The instrumentation of the PCP section has to focus on obtaining useful information that could be realistically measured by manual methods, by automated devices, or by both. The most critical parameters that have to be measured are the following:

For the short-term:

1. Concrete temperature
2. Moisture loss of fresh concrete or evaporation rate
3. Concrete and post-tensioning steel strains
4. Slab horizontal and vertical movements
5. Load-deflection profile

For the long-term:

1. Slab horizontal and vertical movements
2. Possible cracking
3. Load-deflection profile

The next step is to identify the devices and procedures that will facilitate the data collection for both the short-term and long-term instrumentations.

7.5.3 1 Concrete Temperature

Concrete temperature at early ages is a very critical factor that has to be carefully handled. High temperatures promote the quick hydration of the cement paste and the development of outstanding tensile stresses in the concrete, which favors the appearance of cracks. The temperature of the concrete will be monitored using two devices: maturity meters and i-Buttons.

Standard maturity meters are used to measure concrete temperatures at any depth using embedded wires in the concrete at one end and a thermocouple at the other end. The end with the thermocouple is plugged into a digital thermometer, and the concrete temperature is easily obtained. On the other hand, i-Buttons will also be installed in sets of three buttons per location. Figure 7.6 shows how the i-Buttons will be installed at the top, mid-depth, and bottom of the PCP slab in assemblies of three (Ref 56). Plastic rules will be used to build i-Buttons assemblies, and the buttons will be attached to the plastic using some epoxy material. Figure 7.7 shows how the assembly will be driven into the concrete (Ref 56).

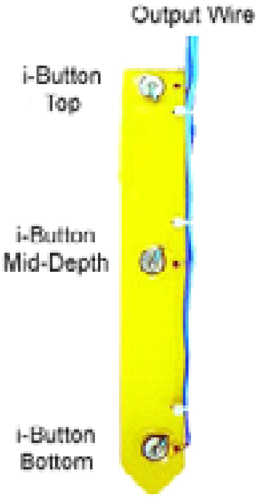


Figure 7.6   Assembly of i-Buttons to be installed in PCP slabs

Figure 7.7   Assembly of i-Buttons being driven into fresh concrete

Installing i-Buttons will allow continuous concrete temperature measurements for long periods of time. These devices can be programmed in such a way that they record the time-temperature evolution of the body in which they are embedded for months. Once in a while, the data are retrieved using a laptop, and the memory of the i-Button is reset so that new data can be stored. Figure 7.8 displays an i-Button, a female attachment, and a probe which can be used for retrieving data (Ref 60).

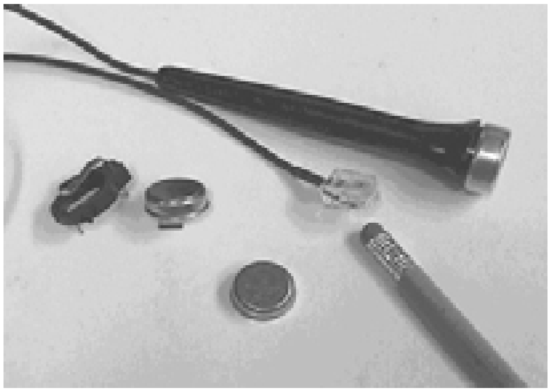


Figure 7.8 Image of i-Button, female attachment, and probe

7.5.3 2 Concrete Moisture

When concrete is initially placed, the loss of moisture should not be above 0.2 lb./sq.ft/hr., as it was previously explained in this chapter. When the evaporation of water in fresh concrete exceeds that threshold, it is very likely that drying-shrinkage cracks will appear, especially in pavements. The moisture of fresh concrete depends on variables like ambient temperature, concrete temperature, relative humidity, and wind speed.

Two methods will be used to measure the moisture content of the concrete and the evaporation rate. To accomplish this, various measurements have to be taken. The first method will consist in using a weather station that will record ambient temperature, relative humidity, and wind speed. Additionally, to calculate evaporation rates, the concrete temperatures can be obtained with either i-Button measurements or by maturity meters. All these variables will enable computing the evaporation rate of the concrete using a set of mathematical expressions (Ref 52).

Additionally, and to supplement the information collected using the previously explained method, hygro-Buttons will be installed at the same locations of i-Buttons (Ref 56). In fact, hygro-Buttons can be linked to the same network as the i-Buttons so that both devices collect data simultaneously. This reduces the wiring network to one inside the concrete slab which supports both types of buttons. Hygro-Buttons are similar in size to i-Buttons, they have the size of a dime. Figure 7.9 displays an image of the hygro-Button.

Figure 7.9 Hygro-Button used for measurement for moisture in concrete

The hygro-Button is a device manufactured by Dallas Semiconductor (Ref 61). The device was recently released to the market and sells for nearly $20 a piece. Although the price is on the high side, the data that it provides might justify its use. Like an i-Button, the hygro-Button can be easily connected to a laptop computer for data retrieval and resetting.

To measure the moisture content in the concrete, researchers at the Center for Transportation Research (CTR) of The University of Texas at Austin designed a simple containment device which isolates the hygro-Button from the wet concrete, but allows the humidity and evaporation rate to be measured (Ref 56). Figure 7.10 displays the PVC containment developed at CTR. This hygro-Button containment will be installed in the PCP section to measure concrete moisture and evaporation rate in real time. The data obtained from this device will be correlated with the data obtained by indirect measurements of data collected using a weather station. Implementing both methods will assure that sufficient moisture profile data are recorded in the event of any device failure.

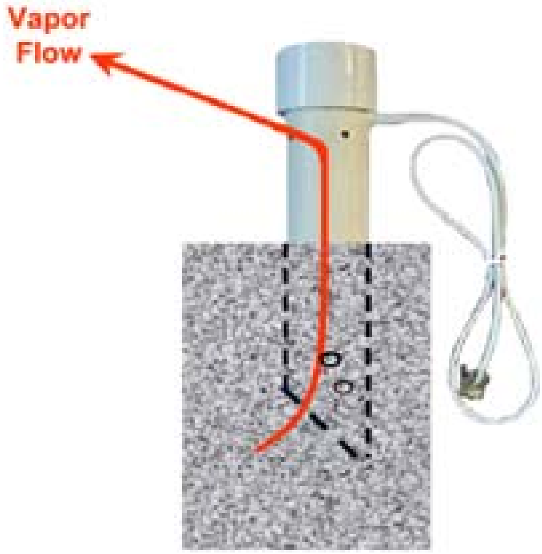


Figure 7.10 Hygro-Button containment installed in the concrete

7.5.3 3 Concrete and Post-Tensioning Steel Strains

The objective of measuring steel and concrete strains is to obtain the stresses generated in both materials. Although instrumentation of the concrete does not represent a problem, it is not the case for the post-tensioning steel. Because the steel strands will be protected by ducts and grouted, the instrumentation tasks are not easily conducted. However, the strain of the steel can be obtained by simply using strength of materials theory. Elongations of the post-tensioned steel will be known, and given those, the induced stresses and strains can be calculated.

The instrumentation of the concrete will be done using embedded strain gauges available in the market, such as the vibrating wire (VW) gauge Type EM-5, or similar. This device is used to measure strains in reinforced concrete and mass concrete. It has also been used in prestressed concrete applications. Figure 7.11 displays an image of the VWG Type EM-5 (Ref 62).

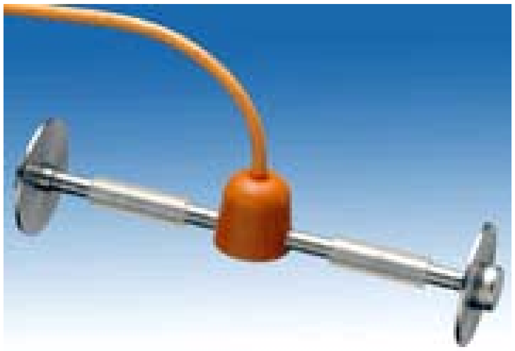


Figure 7.11 Vibrating wire (VW) gauge used for measurement of concrete strain

According to the results reported in the literature, these strain gauges are reliable and can be used in concrete applications without a problem. This means that they are rugged enough to withstand the maneuvers related to concrete construction. The installation of the gauges is considered easy. A pair of strong wood or plastic sticks are attached to the steel using plastic ties, and then a VW gauge is placed on top of the sticks and secured with plastic ties. The output cable has to be secured in the same way. Figure 7.12 shows how the VW is usually installed when used in reinforced concrete applications (Ref 62). This setup is also suitable for use with prestressed concrete.

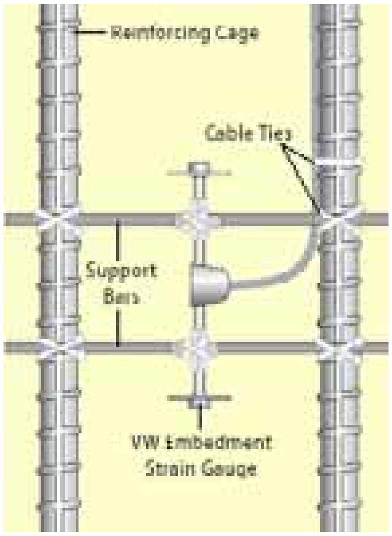


Figure 7.12 Installation of the VW strain gauge in the concrete

The data collected by the VW gauge can be easily retrieved using a laptop computer, by simply plugging in the gauge's output cable to a port in the computer. The data are post-processed by computer software.

7.5.3 4 Slab Horizontal and Vertical Movements

The evaluation of the horizontal and vertical movements of the slabs, especially at the ends, will be measured using mechanical gauges, as it was done in the latest instrumentation of the PCP in McLennan County, where calipers and analog displacement strain gauges were used. Additionally, for the measurement of the horizontal movement another device might be used for continuous electronic data collection. The device that can be installed at transverse joints is a joint meter. Figure 7.13 displays an image of the device, which is anchored to the concrete slabs using four threaded anchors (Ref 63). The anchors attach the device's mount and the target so that a displacement transducer could be attached to the mount, and readings could be taken using the target as the referencing point. An output cable is used to hook it up to a port in a laptop, so that data could be retrieved. The disadvantage of the device is that it has to be removed from its position if construction traffic and operations are constrained. Additionally, the data collected have to be recorded in real time, and thus the laptop has to be continuously connected to the output cable.

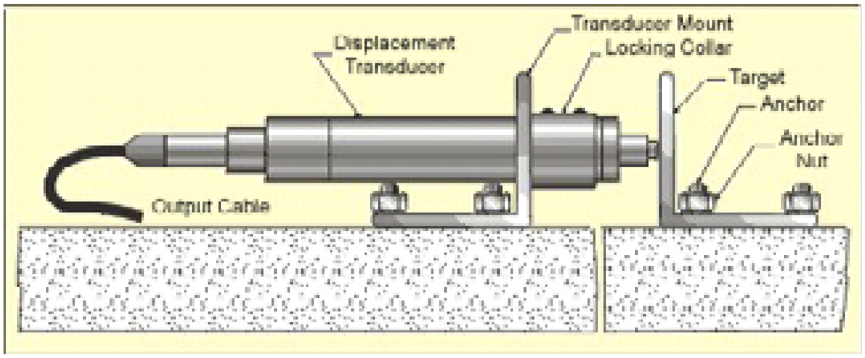


Figure 7.13 Image of a joint meter used for horizontal movement evaluation

Vertical movement should be measured using analog displacement strain gauges. Figure 7.14 shows a photo of instrumentation to measure vertical displacements at the ends of two contiguous slabs in McLennan County. This approach has been used many times with good results. Therefore, it is recommended for the instrumentation of the PCP in Hillsboro, Texas.

Figure 7.14   Measurement of vertical movement at slab ends

7.5.3 5 Load-Deflection Profile

The measurement of the load-deflection profile is done once the pavement is ready to carry traffic loads. The first time is usually before the facility is opened to traffic. It is very convenient to perform at least one complete evaluation of the pavement once it is finished and ready to be opened. Later, evaluations of the pavement will be performed on a regular basis to compare the evolution of the load-deflection profiles. The equipment that will be used to conduct this activity includes the FWD and the RDD. Due to the value and quantity of data that will be generated by these equipments, the information will be saved in compact discs (CDs) and stored in a file cabinet containing all the information related to the PCP section. The functioning of the FWD and RDD has already been explained in sections 7.4.1 and 7.4.2 in this chapter.

7.6 Integral Monitoring and Instrumentation Plan

The previous sections in this chapter have described tasks that will serve as guidelines to monitor the PCP during construction time and throughout its service life.

The present section presents an integral monitoring and instrumentation plan, which summarizes the tasks previously described and recommends the frequency of sampling and testing tasks. Table 7.2 summarizes a master plan containing five major tasks, including sampling of finished products or materials, sampling and testing of fresh concrete, testing of hardened concrete, nondestructive pavement testing, and instrumentation.

Conducting the tasks in the master plan will ensure that the quality of the materials used for the construction of the pavement meets the required specifications. Likewise, the proposed testing plan for fresh and hardened concrete and the instrumentation of the pavement for the short and long terms will ensure the desired high performance of the pavement and will validate the design procedure, respectively. In Table 7.2, Task No. 5 describes the required instrumentation “per instrumented slab.” It is recommended that the number of instrumented slabs is not lower than 10% of the total number of slabs in the project. For instance, if the project requires the construction of 130 slabs, at least 13 slabs will be instrumented.

Table 7.2 Master plan for monitoring and instrumentation tasks

Task No.	Task Description	Work Plan
1	**Sampling of Finished Products or Materials**	
	Polyethylene Sheeting, Steel Strands, Strand Anchors, Transverse Joint Components and Hardware, Strand Ducts, and Grout	As production lots become available during construction
2	**Sampling and Testing of Fresh Concrete**	
	Lots and Sublots	1 Slab is equal to 1 Sublot, 5 sublots equal 1 lot
	Concrete Testing	Samples for 1,3,7,14, and 28 days
	Concrete Consistency	3 samples evenly distributed per sublot
	Pavement Thickness and Temperature	3 tests evenly distributed per sublot
	Air Content and Unit Weight	1 test per sublot or more at engineer's discretion
3	**Testing of Hardened Concrete**	
	Cylinders	
	Compressive Strength	Test at 1,3,7,14, and 28 days
	Splitting Tensile Strength	Test at 3,7, and 28 days
	Concrete Modulus of Elasticity	Test at 3,7, and 28 days
	Concrete Thermal Expansion	Test cores as indicated by ASTM C341
	Beams	
	Flexural Strength	Test at 3,7, and 28 days
4	**Nondestructive Pavement Testing**	
	Rebound Test and Vibration Test (Pulse Velocity)	Test at 1,3,7,14, and 28 days
	Evaluation of Structural Capacity of the Pavement	
	Falling Weight Deflectometer (FWD)	Perform test before opening to traffic
	Rolling Dynamic Deflectometer (RDD)	Perform test before opening to traffic
5	**Instrumentation** ***Short-Term Instrumentation***	
	Concrete Temperature	Use maturity meter and 1 i-Button assembly per instrumented slab
	Moisture Loss of Fresh Concrete or Evaporation Rate	Setup weather station and use 1 hygro-Button per instrumented slab
	Concrete Strain	Install 3 vibrating wire (VW) gauges per instrumented slab, longitudinally
	Horizontal Movement	Install 2 joint meters per instrumented slab, at both ends. Compare to caliper
	Vertical Movement	Use 4 analog dial gauges per instrumented slab, 2 at each end of slab
	Load-Deflection Profile	Test using FWD and RDD before opening to traffic
	Condition Survey	Perform daily inspections during and after construction
	Long-Term Instrumentation	
	Horizontal Movement	Measure monthly using calipers during first 6 months. Once a year thereafter. Alternate summer and winter conditions
	Vertical Movement	Measure monthly using analog dial gauges during first 6 months. Once a year thereafter. Alternate summer and winter conditions
	Condition Survey	Survey monthly during first 6 months. Once a year thereafter. Alternate summer and winter conditions
	Load-Deflection Profile	Perform test every 1 or 2 years after construction, as possible. Alternate summer and winter conditions

7.7 Summary

This chapter described the details of a number of activities that have to be performed to monitor the PCP section to be constructed in Hillsboro, Texas. The main objectives of the monitoring plan are to verify and calibrate the design of the pavement, inspect the performance of the section, and use the results to enrich the design and construction methods of the PCP technology. Likewise, this chapter specifies required QC/QA tasks that should be conducted to ensure the quality of the end-product. These tasks include a sampling program and testing procedures for the materials used for the construction of the pavement and instrumentation tasks for the short and long terms. A description of the required instrumentation devices is also included. Finally, a master plan recommends materials sampling and testing frequencies and instrumentation details for instrumented slabs.

8. COMPARISON OF PAVEMENTS: PCP VS. CRCP

8.1 Introduction

The North American Free Trade Agreement (NAFTA) launched in October 1992 had a great impact on the vehicular traffic on IH 35. Added to the traffic generated by NAFTA, the regional traffic in Texas also increased, so that in the last ten years traffic growth has followed an exponential increase in major highway routes. According to research conducted at CTR at The University of Texas at Austin, IH 35 moves nearly 75% of the overland trade between Mexico, the United States, and Canada (Ref 64). This hasty increase in traffic has resulted in a boost for the structural design criterion of the main highway network. For instance, the thickness of a CRCP design has increased from 8 inches in 1958 to 15 inches in 2003 for interstate highways.

As was already mentioned in previous chapters in this report, a PCP section was designed and constructed in 1985 in McLennan County in Texas. The performance of the PCP has been evaluated and reported in Chapter 4. The history of the evaluations of this 17-year-old PCP section shows that the overall performance of the pavement is remarkable, especially considering that the concrete slab constructed for the PCP is only 6 in. thick and that traffic loads have grown at a dramatic rate in volume and weight.

As a result of the excellent performance of the PCP in McLennan County and realizing that it has almost been a maintenance-free pavement, TxDOT's Waco District Office contacted CTR at The University of Texas at Austin to pursue the design and construction of a new pavement section that will use the PCP technology. Again, the section was to be built on IH 35, in Hillsboro, Texas.

8.2 Objective

The design and construction of a new PCP project in the Waco District was catalyzed by two main conditions: the rapid traffic growth in the area and the desire to implement this paving technique so that it could be compared to a CRCP control section. To accomplish this ultimate goal it was necessary to construct a new equivalent high-performance PCP section that could be compared to a CRCP in terms of performance and economics. Although the one-mile PCP section constructed in McLennan County in 1985 showed excellent performance, the economic viability of the PCP was still an issue, and therefore, the PCP technology could not be endorsed for use on large projects. The limited length of the PCP test section in McLennan did not address the economic aspects of the PCP technique. In 1985 the construction of the PCP focused more on providing an alternative pavement type that will compare only in performance to a CRCP.

In order to be able to compare both the performance and the economic feasibility of the PCP to those of an equivalent CRCP, TxDOT selected Hillsboro as the ideal location for construction of the PCP. This site was selected because a 5-mile CRCP section, which will serve as a control section for the PCP, was recently constructed in the same area. It is fortunate for research comparison purposes that the CRCP and the PCP will be located next to each other. The CRCP section was built just south of where the PCP section will be constructed, on IH 35.

The main objective of this chapter is to describe how the two paving techniques can be compared objectively in terms of performance and cost. To achieve this goal, first, some background on the control section is provided to show that design considerations for the two pavement projects are alike. Next, a comparison scheme is recommended. The fact that the pavements will be located contiguous to one another makes the comparison easier and more realistic because the structures will be exposed to the same traffic, environmental, and foundation soil conditions.

8.3 Background on Control Section

The control CRCP section to be compared to the PCP is located on IH 35. It starts at the southern mile marker 359.3 and ends at mile marker 364.4, which corresponds to the starting mile marker of the PCP. According to the cross-section history of the pavement (Ref 64), the original construction of the structure was conducted in September 1941 and consisted of a jointed pavement with curling and warping joints spaced at 15 ft Later, the pavement required various maintenance and rehabilitation tasks that were carried out from January 1951 to May 1987. In 1987 a 4.5 in. thick layer of asphalt concrete was placed on top of the structure to improve its riding quality, which was affected mostly by reflected cracks and joints from the underlying jointed pavement. Finally, in November 1999, after the design was completed and the bid was awarded, the construction of the CRCP was initiated. It was not until December 2000 that both southbound and northbound roadbeds were totally finished.

8.3.1 DESIGN PROCEDURE

The design of the CRCP section was conducted using the methodology proposed in the Guide for the Design of Pavement Structures by AASHTO (Ref 17). The pavement thickness design was determined for three different support conditions because foundation characteristics were different for the northbound and southbound roadbeds. Input design parameters were divided into six different categories: design constraints, subgrade parameters, subbase parameters, traffic parameters, concrete pavement properties, and drainage parameters.

8.3.1.1 Design Considerations

The design of any pavement is a process that requires vast documentation of all the parameters involved in the design. Since showing the entire design process of the CRCP is beyond the scope of this study, only key defining considerations and parameters used for the design are described here and only those that are comparable to the ones used for the design of the PCP. The detailed design of the control section is contained in the literature (Ref 64).

One of the most important steps in the design was the estimation of the traffic variable. As with any pavement design, this variable was estimated using traffic prediction models that provide a range of design values. Traffic demand was estimated using the same approach used for the design of the PCP. Four traffic prediction models developed at CTR at The University of Texas at Austin (Ref 24) were used, and ESALs were computed for given average daily traffic (ADT) values, percentage of trucks, traffic distribution, etc. The calculation of ESALs was done for periods of 30 and 40 years, and the estimated values were 127 million and 222 million ESALs, respectively. For further thickness design, TxDOT decided that 30 years would be the design period for which the section would be constructed. Thus, the calculated ESALs number for the 30-year period was increased by directional distribution and lane distribution factors. Considering those factors, the final value of ESALs for the design lane was 160 million over the 30 years.

In addition to the estimation of traffic demand, the design of the CRCP required the recommendation of other design parameters mostly related to the strength of the concrete. Therefore, the AASHTO design procedure required the input of the mean modulus of rupture (S′x) determined at 28 days using a third-point loading testing procedure. Based on previous design experiences, a value of 700 psi was selected for this particular project. Likewise, the design modulus of elasticity value of the concrete was assumed using typical values found in research studies. For this project a design value of the modulus of elasticity of 4,000 ksi was assumed.

The thickness design for the CRCP was conducted using the AASHTO procedure (Ref 17), taking into consideration previously described and other parameters. The AASHTO design spreadsheet (Ref 27) described in Chapter 5 was also used to facilitate the computations. The spreadsheet requires the input of a number of design variables, including subgrade and foundation soil parameters, base and subbase parameters, traffic parameters, concrete strength parameters, and drainage parameters. Additionally, the spreadsheet requires inputting a confidence level for the design (reliability number). After entering all the input values, a macro in the spreadsheet generates the output thickness design rounded up to the nearest half of an inch. For a 30-year design period the spreadsheet calculated thickness values ranging from 12.5 in. to 14.5 in., depending on the location of the pavement (northbound or southbound). Finally, to avoid having different thickness values, an overall thickness of 14 in. was selected as the design value for both roadbeds. Table 8.1 summarizes some key design input parameters used for the design of the control CRCP section

Table 8.1 Summary of design input values for the CRCP control section

Parameter	**Design Value**
Design period	30 years
Design traffic	160 million ESALs
Concrete modulus of rupture	700 psi
Concrete Modulus of Elasticity	4,000 ksi
Design thickness	14 in.

8.4 Comparison Scheme

As it was previously mentioned, the comparison of the PCP and the CRCP control section has to be done in light of two factors: performance and economics. An effective way to achieve a good performance comparison of the two sections is by conducting periodic evaluations of the conditions of the pavements. Those evaluations have to include measurable data so that the real deterioration of the structures is reflected numerically. Fortunately, the existing CRCP control section has been evaluated periodically. Data collection has included visual condition surveys with detailed crack spacing, crack width measurement, and evaluation of the structural capacity of the pavement using FWD and RDD.

Evaluations of the CRCP have been done at different occasions during construction in 1999 and 2000, and up to the present time. Data collected have been analyzed, and the evolution of the pavement has been continuously recorded in files saved at CTR at The University of Texas at Austin. In the same way, the PCP section to be constructed will be evaluated according to the monitoring plan proposed in Chapter 7. The evolution of the pavement will be documented, and deterioration will be measured and recorded accordingly.

In order for the comparison to be as scientific and realistic as possible, both pavements were designed for a 30-year life period. Since the sections are contiguous, they will function under very similar climatic and traffic conditions. Although the two pavements deteriorate in different ways and develop different distresses, there are some performance parameters that can easily be compared. Furthermore, the evaluations to be conducted from time to time for each pavement will help identify which pavement deteriorates faster and the reasons involved.

With regard to the comparison of the pavements from the economics perspective, it is necessary to estimate the costs involved with the construction and maintenance of the pavements and, ultimately, the pay-back period of the investment. Although for CRCPs the construction costs are well studied, this is not the case for PCPs, mainly because this paving technique is not as common. However, a preliminary estimation of the costs involved with the construction of the PCP in Hillsboro is presented later in this chapter. Likewise, the results of a life-cycle cost analysis of the two paving options are summarized at the end of the chapter.

8.4.1 PERFORMANCE COMPARISON

The evaluation of pavement performance requires studying the functional behavior of the pavements (Ref 59). To conduct a performance analysis of a pavement, it is necessary to know the history of its riding quality for the period for which the performance is analyzed. During the AASHO road test conducted from 1958 to 1960, Carey and Irick developed the pavement serviceability-performance concept (Ref 65). A serviceability index refers to the evaluation of the condition of a pavement at the time of measurement. The following subsections describe different ways in which the performance of the pavements under study can be evaluated.

8.4.1.1 Condition Surveys

Conducting visual inspections of the pavements at different times is probably the easiest and least expensive procedure to build a history of the condition of the structures. However, condition surveys do not provide information about the deformation of the pavement profile or deterioration of its structural adequacy. Usually, visual inspections help to identify key features of the pavement that need to be further evaluated using other, more sophisticated methods and equipment. Chapter 7 in this report recommended conducting periodic condition surveys according to a predefined schedule. The information collected from the surveys will complement other performance data, such as roughness and structural capacity.

8.4.1.2 Roughness

In general, the performance of a pavement is determined by summarizing the serviceability record over a period of time. The serviceability concept has been used for over 40 years to evaluate pavements at the network and project levels. Studies from the AASHO road test showed that in almost 95% of the cases, the serviceability level of a pavement is indicated by the roughness of the surface profile. Roughness can be defined as the distortion of the pavement surface that contributes to an uncomfortable ride (Ref

59). This roughness varies with speed and characteristics of the testing vehicle or equipment.

There are different devices that measure the roughness of a pavement. Among the most common equipment are profile devices, profilographs, and response type devices. The equipment evaluates either the profile of the pavement or the response of the vehicle to the roughness, and depending on the equipment that is used, an algorithm is used to generate roughness summary statistics that are useful for objective evaluations. A summary statistic that is used worldwide is the international roughness index (IRI).

TxDOT uses a profilometer to determine the longitudinal profile along wheel paths of pavement lanes. The data are collected in electronic format and provide depth measurements in millimeters for each wheel path. Next, to generate summary statistics, the program KIRIPSI is used to estimate the IRI and the present serviceability index (PSI). These two parameters are stored in TxDOT's pavement management information system (PMIS) database and are available for some main highways in Texas. Given the usefulness of the information provided by the profilometer, it is recommended that roughness data be collected to estimate the IRI and the PSI of the CRCP and PCP pavement sections under study. The history of these data will provide objective results about the rate of deterioration of each section.

8.4.1.3 Structural Capacity

Another indicator of the performance of the pavement sections will be the evolution of the capacity of the structures to withstand imposed loads. Using the FWD and RDD devices will be a fundamental part of this task, which will provide the most valuable data in terms of the evolution of the pavement sections. Theoretically, as time goes by and pavement structures are in service, their structural adequacy tends to deteriorate. This could first be observed during condition surveys and later by looking at the load-deflection profiles generated by the RDD and FWD, where deflections at a given location increase with time. The monitoring plan in Chapter 7 recommends the frequency for the evaluation of the structural capacity of the PCP. Likewise, the CRCP testing plan that was initiated right after construction of the section has to be continued so that sufficient history data are generated to conduct the comparison of the performances of the CRCP and the PCP.

8.4.2 ECONOMICS COMPARISON

A comparison of the costs involved with the construction of the two paving technologies is not an easy task because no current information is available for PCP construction. Construction costs are well tabulated for CRCPs, but for PCPs this is not the case. Due to the uncommon practice of the construction of PCP sections, construction unit prices are unclear. However, this section of the report provides a preliminary analysis of the construction costs for the PCP in Hillsboro. This cost of the PCP was estimated by combining updated costs of the PCP constructed in McLennan County in 1985 and available cost information for prestressed concrete beams. The unit prices and costs of the PCP in McLennan County were affected by the inflation rates observed for every year from 1985 to the present. At the end of this chapter, when the costs of the PCP and CRCP sections are compared, a life-cycle cost analysis is used for a more comprehensive comparison of the overall costs.

8.4.2.1 CRCP Cost

Although the total cost of the control CRCP section under study is not well documented, the common application of this paving technique makes it easy to collect plenty of information about paving costs for various thicknesses and construction districts in Texas (Ref 66). In general, construction costs of CRCPs vary depending on the following factors:

1. Construction district: Bid prices are different from one region or district to another.

2. Paving area: It is common that the larger the paving area, the more the costs are reduced for bidding purposes.
3. Thickness: Because thick pavements require more material than thin pavements, they result in increased costs; however, paving area is a factor that, in combination with thickness, could result in lower or higher unit prices and costs.

According to the construction and bidding cost research performed for CRCPs, it was found that unit prices vary from $23.50 to $50.60 per paved and finished square yard, depending on the three factors mentioned above, especially thickness and paving area. These unit prices were effective in December 2002 and current in March 2003. Therefore, the average unit price per square yard of a finished 14 in. thick CRCP is $45.00. This value is used to compare costs of the two paving methods in the life-cycle cost analysis later in this chapter.

8.4.2.2 PCP Cost

As previously mentioned, there is no full certainty about the current unit prices and costs for this paving technique, at least in Texas. However, to obtain reasonable construction costs the following approach was used:

1. Unit prices for the PCP constructed in McLennan County in 1985 were projected using an engineering economic analysis technique based on present worth analysis that accounted for yearly inflation rates from 1985 to 2003.
2. Current unit prices of construction of prestressed beams used in the highway industry served as a guideline for the estimation of the unit price for the PCP slabs.
3. The unit prices obtained from steps 1 and 2 were analyzed and judged so that a final unit price was assigned for construction of the PCP.

Table 8.2 shows the inflation rates observed during the period 1985−2002 in the United States (Ref 67). The inflation rates shown measure the annual rate of change of the cost of goods and services representative for a year. These inflation rates were computed using the consumer price index (CPI), which is the index used in the United States (Ref 67). The inflation rate for a particular year is computed by taking the price index of that year minus the price index of the previous year, divided by the price index of the previous year, all multiplied by 100.

Table 8.2 Inflation rates in the United States observed from 1985 to 2002

Year	Inflation Rate (%)	Unit Price
1985	3.54	$ 25.41
1986	1.86	$ 26.31
1987	3.66	$ 26.80
1988	4.12	$ 27.78
1989	4.81	$ 28.92
1990	5.39	$ 30.32
1991	4.22	$ 31.95
1992	3.01	$ 33.30
1993	2.98	$ 34.30
1994	2.60	$ 35.32
1995	2.76	$ 36.24
1996	2.96	$ 37.24
1997	2.35	$ 38.34
1998	1.51	$ 39.24
1999	2.21	$ 39.84
2000	3.38	$ 40.72
2001	2.86	$ 42.09
2002	3.00	$ 43.30

Table 8.2 also shows that the unit price of a square yard of the PCP in McLennan County was $25.41 in 1985 (Ref 11). The unit price of that same pavement is shown for the following years up to 2002, when according to the computed values in the table, the unit price of the item would be $43.30. In addition, considering that the current unit price per square yard of an 8 in. to 10 in. thick CRCP varies from $24.00 to $40.00 (Ref 66), the estimated value of $43.30 seems reasonable. That would be the unit price of the same 6 in. thick PCP constructed in 1985 if it was constructed in March 2003. Since the PCP designed for Hillsboro is thicker than the one shown in Table 8.2 its unit price per square yard should be higher. Based on this analysis and on unit prices of prestressed beams used in bridges, the preliminary unit price per square yard of the 9 in. thick PCP will be

$48.00. This value is the one that is used in the estimation of the life-cycle cost analysis of the PCP section.

8.4.2.3 PCP Cost vs. CRCP Cost

Having estimated the unit prices per square yard of the CRCP and the PCP, the next step is to compare the costs of the two pavements. First, it should be recalled that the design of the PCP detailed in Chapter 5 was based on the design of an equivalent CRCP section of 14 inches of thickness. This thickness of the equivalent CRCP was obtained from the design using the AASHTO procedure (Refs 17 and 27), and that is why the cost of the 6 in. thick PCP is compared to the cost of a 14 in. thick CRCP.

Table 8.3 summarizes the unit prices of the two paving techniques and the area of pavement that needs to be constructed for the new PCP in Hillsboro. If a CRCP was to be constructed instead, the equivalent thickness would be 14 in., and the paving area for the 7.4 miles (northbound plus southbound) is shown in columns two and three in square feet and square yards, respectively. Therefore, considering a unit price of $45.00 per square foot, the total cost of a CRCP would be $13.87 million. Notice that for this paving technique the cost of the joints is zero because the CRCP does not need transverse expansion joints.

Table 8.3 Summary of construction costs for CRCP and PCP

Pavement Type and Thickness	Paving Area (ft^2)	Paving Area (yd^2)	Unit Price (yd^2)	Joint Cost (USD)	Total Cost (USD)
CRCP - 14 in.	2,774,112	308,235	$ 45.00	$ -	$ 13,870,575.00
PCP - 9 in.	2,774,112	308,235	$ 48.00	$700,000.00	$ 15,495,280.00

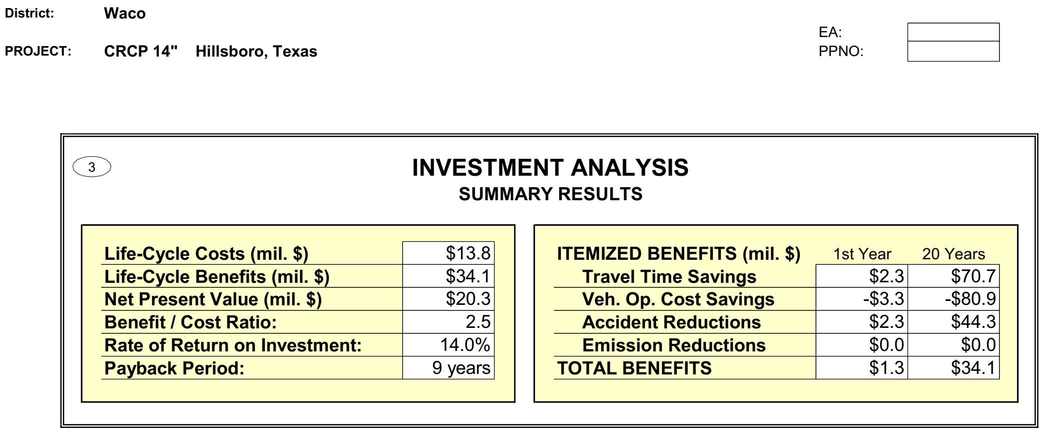
Difference = $ 1,624,705.00

Alternatively, if the 9 in. thick PCP is constructed as designed over the same area, but with a unit price of $48.00 per square yard, the total construction cost would be $15.49 million. Notice than for this paving technique the cost of the transverse joints is $700,000.00, which is substantial and is equivalent to 4.5% of the total cost of the PCP. Additionally, the unit price of the 9 in. thick PCP is greater than the unit price of the 14 in. thick CRCP. The reason for the difference between unit prices is the cost added to the PCP for post-tensioning tasks. Thus, from the analysis of the initial construction costs it can be seen that construction of the PCP requires an investment of $1.62 million more than an equivalent CRCP.

Fortunately for engineering projects, where large amounts of money are usually involved, investment decisions are not made based only on initial costs. Rather, decisions are made based on further engineering economic analyses that consider other complex variables that affect final decisions. Obeying these trends, a life-cycle cost analysis was performed for the two different paving methods.

There is a great variety of methods and programs available in the industry that perform this type of analysis. To analyze the two paving options in Hillsboro, a program used by the California Department of Transportation (CalTrans) was used. The program is the California Life-Cycle Benefit/Cost Analysis Model [Cal-B/C] (Ref 68) and consists of a set of spreadsheets that analyze both highway and transit projects. The user inputs data defining the type, scope, and cost of a project. The model then calculates its life-cycle cost, net present value, benefit/cost ratio, internal rate of return, payback period, and itemizes the first year and life-cycle benefits.

The inputs for the PCP and the CRCP are almost the same, except for construction costs, project support costs, mitigation costs, and costs after construction is completed. Usually, the costs after construction (e.g., maintenance and rehabilitations costs) are greater for CRCPs than for PCPs, which is why PCPs are often called free-maintenance pavements. Figures 8.1 and 8.2 display the summary results obtained from the life-cycle cost analyses for the CRCP and the PCP, respectively.



District: **Waco**

PROJECT: **CRCP 14" Hillsboro, Texas**

EA:

PPNO:

3

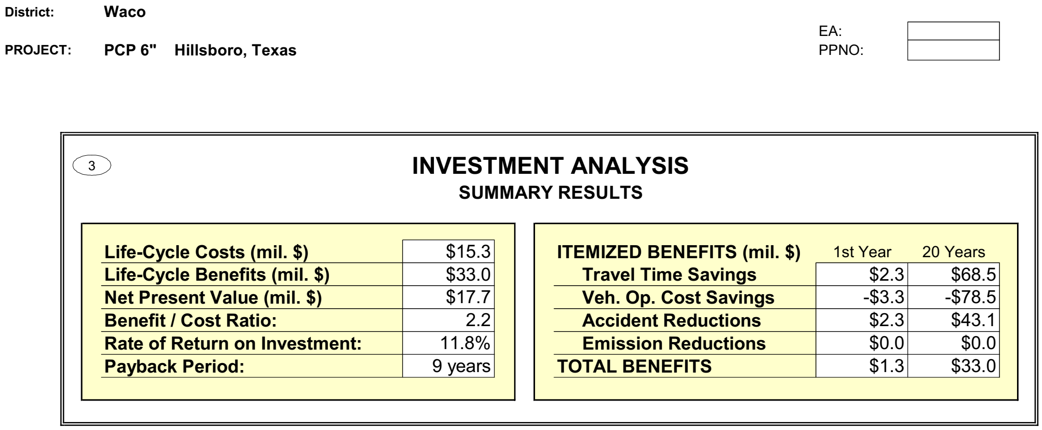
INVESTMENT ANALYSIS

SUMMARY RESULTS

Life-Cycle Costs (mil. $)	$13.8
Life-Cycle Benefits (mil. $)	$34.1
Net Present Value (mil. $)	$20.3
Benefit / Cost Ratio:	2.5
Rate of Return on Investment:	14.0%
Payback Period:	9 years

ITEMIZED BENEFITS (mil. $)	1st Year	20 Years
Travel Time Savings	$2.3	$70.7
Veh. Op. Cost Savings	-$3.3	-$80.9
Accident Reductions	$2.3	$44.3
Emission Reductions	$0.0	$0.0
TOTAL BENEFITS	$1.3	$34.1

Figure 8.1 Summary of life-cycle cost analysis for CRCP



District: **Waco**

PROJECT: **PCP 6" Hillsboro, Texas**

EA:

PPNO:

3

INVESTMENT ANALYSIS

SUMMARY RESULTS

Life-Cycle Costs (mil. $)	$15.3
Life-Cycle Benefits (mil. $)	$33.0
Net Present Value (mil. $)	$17.7
Benefit / Cost Ratio:	2.2
Rate of Return on Investment:	11.8%
Payback Period:	9 years

ITEMIZED BENEFITS (mil. $)	1st Year	20 Years
Travel Time Savings	$2.3	$68.5
Veh. Op. Cost Savings	-$3.3	-$78.5
Accident Reductions	$2.3	$43.1
Emission Reductions	$0.0	$0.0
TOTAL BENEFITS	$1.3	$33.0

Figure 8.2 Summary of life-cycle cost analysis for PCP

Comparing the results obtained from the analyses, two parameters are significant and deserve further discussion. According to CalTrans's program, the benefit/cost ratio for the CRCP is 2.5, whereas for the PCP it is 2.2. This means that the ultimate benefit obtained from either of the projects is almost the same. Likewise, the payback period for both paving techniques is 9 years with rates of return of 14.0% for the CRCP and 11.8% for the PCP. For both analyses, the assumed real discount rate is 5%. Although the rate of return for the PCP is just a little lower than the one for the CRCP, the pay-back period for the two cases is the same, meaning that in general terms, from an economic analysis standpoint, both paving techniques offer the same benefits.

The spreadsheets prepared for the CalTrans's program only perform calculations for 20-year periods. This is an inconvenience that would require total modification of the spreadsheets, if an analysis were required for a longer period of time. That is why Figures 8.1 and 8.2 only show the itemized benefits for the analyses for 20 years, instead of 30, which is the period for which the CRCP and PCP sections were designed structurally.

8.5 Summary

The main objective of this chapter was to describe a methodology to compare CRCP and PCP paving techniques from performance and economics standpoints. These comparisons are not simple, especially the one related to economics. To achieve a full comparison of the performances of the pavements it is necessary to have a detailed history of the behavior of both sections, the PCP and the control CRCP section. That information is not available yet because the PCP has not been constructed. However, this chapter recommended a methodology so that the performances of the pavements can be compared in the future by collecting data at regular intervals of time, including condition surveys, roughness measurements, and load-carrying capacity estimations using FWD and RRD devices.

The comparison of the sections from an economic standpoint was conducted by investigating current costs for construction of CRCPs in Texas and comparing those costs to predicted costs of PCPs construction. Since unit prices for PCPs are not precisely known, unit prices from the section built in McLennan County were converted to net present values using national inflation rates. Additionally, costs of prestressed concrete beams served as guidelines for the estimation of preliminary PCP construction costs. A comparison of the construction costs using both paving techniques was conducted, and the analysis showed that constructing the PCP will initially cost 11.7% more than an equivalent CRCP. It was observed that the transverse joints cost represents 4.5% of the total cost of the PCP. To finalize the economic comparison of the two paving techniques, a life-cycle cost analysis was conducted using a program utilized by CalTrans. This analysis showed that the benefit/cost ratio and the pay-back period for both pavements are very similar, meaning that both paving methods are very comparable. However, based on the previous experience with the PCP in McLennan County, it is expected that the PCP to be constructed in Hillsboro will technically outperform the control CRCP section placed next to it.

9. DISCUSSION OF DEVELOPMENTS

9.1 Introduction

The literature review and the continuous monitoring of the existing PCP in McLennan County in Texas described in previous chapters in this report showed that the pavement, which was constructed in 1985 and was designed for a 20-year life period, is still in great condition. Although it has almost reached the end of its design life and has withstood much more vehicular traffic than what it was designed for, its riding quality is very good. However, there are some conceptually designed and constructed features of the PCP that did not work as expected. Although those features have not caused major structural problems to the pavement itself, they still need to be improved in future PCP applications.

9.2 Objective

Chapter 4 in this report evaluated the overall performance of the existing PCP in McLennan County, and Chapter 5 described the recommended steps for designing the PCP section in Hillsboro, Texas. The objective of this chapter is to highlight the developments proposed in the new design and construction procedures. These developments aim to correct problems experienced with the previous PCP design and construction in Texas. In those cases in which the applied procedures have been demonstrated to be successful, no major changes are recommended for the new PCP section.

9.3 Nature of Developments

The new design developments proposed for construction are broad and include the use of better and more controlled materials, construction procedures, and monitoring tasks. The developments discussed here are divided in three categories: design, construction, and monitoring. Additionally, after a brief description of the developments, some of the most relevant practical construction considerations that aim to correct failures of construction practices observed in the PCP in McLennan County are discussed, so that those problems will be prevented in the new PCP in Hillsboro.

9.3.1 DESIGN

One of the major developments in the design area is represented by the design methodology recommended in this study, described in Chapter 5. Although the method described for design does not pretend to be a standard, it serves as a practical guide for other designs. The design method is a mechanistic-empirical approach that modeled the pavement as a multilayered visco-elastic structure supported by a visco-elastic foundation. By modeling the pavement in this way, it is possible to calculate the stress, strain, and deflection at any point within the pavement. Additionally, in order to predict the performance of the pavement influenced by factors not precisely modeled by mechanistic models, field observations and empirical correlations were used. For instance, the behavior of the pavement was estimated as a function of traffic and environmental conditions. In other words, the behavior of the slabs was estimated in terms of slab movements. Although mechanistic-empirical design procedures have limited application for flexible pavements, it is accepted by pavement and materials researchers in the asphalt industry that these methods will improve design practices, as has been the case for rigid pavements.

The steps for the design of a generic PCP pavement were described in Chapter 5, and then that methodology was implemented for designing the PCP in Hillsboro. The design of the new PCP used the information provided by the long-term documentation of the performance of the old PCP section in McLennan County. Thus, the field observations and empirical correlations required by mechanistic-empirical design methods were used in the PCP design for Hillsboro.

9.3.1.1 PCP Design Computer Program

Another ongoing development in this study, related to the design of PCPs, is the improvement of an already existent PCP design program. The program PSCP4 was developed as a performance prediction tool. PSCP4 is an updated user-friendly version of the old PSCP2 program, developed at CTR of The University of Texas at Austin (Ref 69) that incorporates a Windows®−based interface for inputting data. The program has a graphic interface that was created using Microsoft Visual Basic 6.0. By incorporating Visual Basic, the user has the flexibility of going back to any input screen to change input parameters. This is helpful when performing many runs and comparing different case scenarios. PSCP4 input data include PCP slab geometry, concrete strength properties and coarse aggregate type, steel properties, wheel load characteristics, concrete temperature data, and a number of post-tensioning stages. The output data generated by the program include information about slab end stresses and movements, curling stresses and movements, and total slab stresses and movements. Output files can be retrieved and displayed in either text or graphic formats.

Although at press time the development and completion of PSCP4 was in its final stage, it was not used for the design of the PCP in Hillsboro because time-consuming logistic problems arose. Therefore, the design of the pavement was conducted using the methodology detailed in Chapter 5 and a set of spreadsheets to expedite design tasks, which usually require analyzing various combinations of input design parameters. It is expected that PSCP4 will be ready for use in the short-term.

9.3.1.2 Design Spreadsheets

Because the design of a PCP requires analyzing various combinations of input data, the use of computer spreadsheets is a basic tool. Input data should combine different alternatives of slab length and thickness with properties such as strength of concrete, traffic, and environmental conditions. Using computer spreadsheets expedites the design process, allowing the combination of different input values in one run. Results are obtained quickly, and the selection of an optimum solution is much easier.

The spreadsheets prepared for the design of the PCP in Hillsboro combined input variables for three slab lengths and three slab thicknesses. In every run a total of nine slabs can be analyzed using the same parameters for concrete strength and coarse aggregate type, post-tensioning steel characteristics, and climatic conditions.

After running a series of length and thickness combinations for the PCP, the designer evaluates the outputs and rationalizes the results to a few feasible options. Although the selection of the final PCP slab dimensions is flexible, the decision has to be made by finding a balance between the riding quality of the pavement and economic considerations. A set of printouts of the design conducted for the PCP in Hillsboro can be found in Appendix E in this report. The results obtained from the spreadsheets show the required strand spacing and estimated movement at the end of the analyzed slab for the given conditions.

9.3.1.3 Other Developments

The design of the new PCP required the evaluation of the structural condition of the existing pavement structure. This evaluation was conducted by NDT using FWD and RDD devices. However, before analyzing the structural capacity of the structure, an evaluation of the pavement surface was conducted with condition surveys. The data collected during the latest condition survey were used in the objective estimation of the condition of the pavement. The implementation and utilization of the PDI rating criterion served as a predesign tool that allowed locating potentially weak zones along the existing asphalt pavement. Chapter 5 described how the PDI was successfully used to divide the total pavement length into subsections and groups that rated the degree of surface deterioration. It has been determined that the PDI rating criterion should be used as a predesign tool for any type of pavement and should include more distresses in its computation.

9.3.2 CONSTRUCTION

Developments related to the construction stage of the PCP were described in Chapter 6. A comprehensive description of the natural foundation soils that will support the PCP was given. Findings showed that for almost the entire length of the project the natural soils are clayey with high plasticity values. In addition, the fact that the soils forming the embankments of the current pavement structure also have shrinking and swelling problems required comprehensive recommendations for embankment soils treatment. It has been observed that the weakness of those soils in the presence of water has at times been underestimated, causing problems in the pavement structure.

Additional developments included drafting a list of the required construction materials and their applicable specifications. ASTM or TxDOT material testing procedures and specification standards were described for each material type. Finally, recommendations were made with regard to construction steps. It was not possible to describe the construction process in great detail because modifications are likely to be made at construction time. However, some useful guidelines were drafted.

9.3.3 MONITORING

The recommendations and developments related to the monitoring plan for the PCP section covered the description of QC/QA tasks to control the quality of the materials that will be used for the construction of the pavement. Concrete sampling and testing activities will play a major role in the overall performance of the concrete mixture. The monitoring plan will be pursued according to what was specified in Chapter 7 of this report. Additionally, the advancements proposed in terms of instrumentation of the PCP will provide valuable information that will enable the most effective calibrations between predicted and measured PCP slab stress, deflection, and movement. It was recommended that the state-of-the-art instrumentation should be backed up by conventional or manual instrumentation tasks.

9.4 Practical Construction Considerations

In 1985 the construction of the PCP in McLennan County required the implementation of new paving procedures that had no precedents at that time. The combination of design and construction innovation uncertainties resulted in a pavement that has performed outstandingly, although it has some flaws. This section describes some construction characteristics and practices that will need to be modified when constructing the new PCP in Hillsboro, Texas. Adopting these recommendations will ensure the correction of previous design and construction oversights, which in turn will result in a better pavement.

9.4.1 PRESTRESSING STRANDS

One of the most important features of the construction of the new PCP that will need modification is the arrangement of the prestressing strands. This is a critical aspect that affects the performance of the pavement, and it will require modifications to the configuration adopted in the previous project constructed in Texas in McLennan County. Next, recommendations for the new layout of transverse and longitudinal prestressing strands are discussed.

9.4.1.1 Transverse Strands

The PCP in McLennan County implemented a looped tendon configuration as shown in Figure 9.1. This layout has a number advantages allowing prestressing a greater length of pavement in a single operation than with a straight tendon pattern. Another advantage is that the looped pattern allows the paving of concrete strips without using formworks.

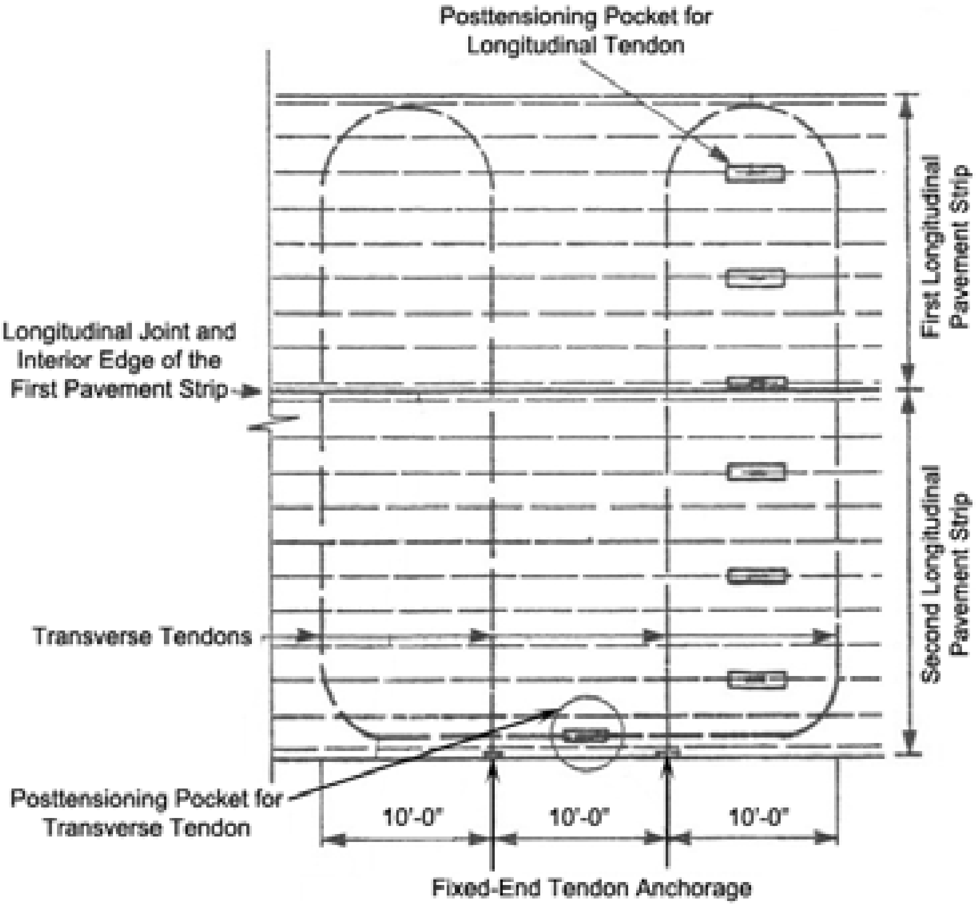


Figure 9.1 Transverse tendon configuration used in PCP in McLennan County

The prestressing operations of the looped tendons used in McLennan County's PCP were performed at post-tensioning pockets located between the fixed-end tendon anchors, as shown in Figure 9.1. The first problem with this layout was experienced during construction. It was difficult to place and hold the tendons in the desired bent position. This was a task that required some extra time to be performed adequately before pouring concrete. Additionally, the greatest inconvenience in constructing the pavement was the prestress loss due at the tendon loops. It is believed that the longitudinal cracks that appeared on the PCP at an early age were caused in part to the lack of prestress in the transverse direction. The prestress loss was probably underestimated during the design stage at the curves of the tendons, causing the concrete tensile stresses to be greater than the prestress induced by the tendons

For the new PCP in Hillsboro it will be necessary to modify the transverse prestressing criterion. Instead of using a looped pattern configuration, simple straight tendons will efficiently transmit the post-tensioning force through the width of the pavement, as shown in Figure 9.2. The new PCP in Hillsboro will consist of three concrete strips that will be paved separately for each roadbed according to the sequence that will be provided by TxDOT. The three concrete strips will hold four traffic lanes, plus inside and outside shoulders. Bell-shaped strand receivers will be cast at the edges of the pavement in the longitudinal direction to guide the tips of the strands from one pavement strip to the other when post-tensioning the three strips altogether. Figure 9.2 shows the sequence of paving strips, but in reality the order in which the actual strips will be placed might vary at construction time, depending on TxDOT's traffic control plan and the desired time to open the PCP to traffic.

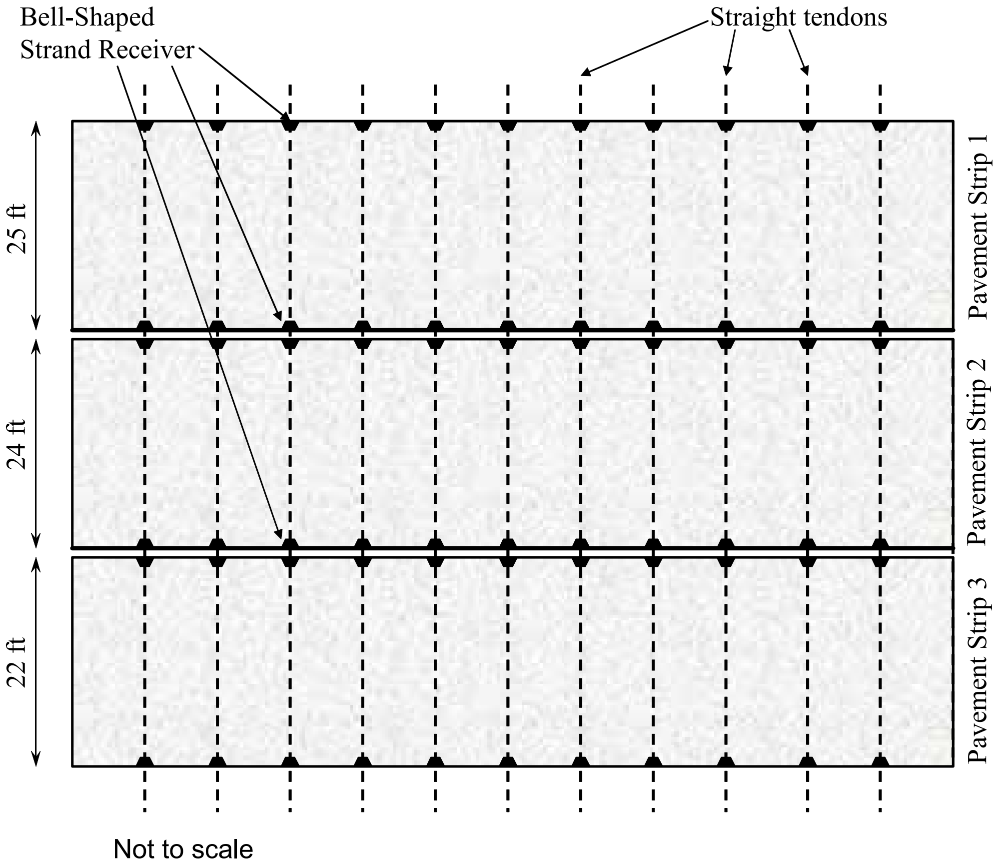


Figure 9.2 Recommended transverse tendon configuration for PCP in Hillsboro

9.4.1.2 Longitudinal Strands

The PCP section constructed in McLennan County implemented an effective arrangement of the longitudinal prestressing tendons. After approximately 6 to 7 hours of paving the first set of pavement strips, some transverse cracks started to appear on the surface of the concrete. To counteract the effect of those tensile stresses in the concrete that caused the cracks, the initial post-tensioning was applied 8 to 10 hours after concrete placement, with very good results. The full post-tensioning force was applied once the concrete gained sufficient strength, approximately 48 hours after placement. Early condition surveys of the PCP showed that the early transverse cracks were no longer visible because the induced prestress eliminated them.

Despite the effectiveness of the longitudinal post-tensioning tasks, for the new PCP to be constructed in Hillsboro it is recommended that another construction approach is followed. The only change that is suggested is that instead of using one central stressing pocket per longitudinal tendon, it should be used to hold two contiguous tendons, as shown in Figure 9.3.

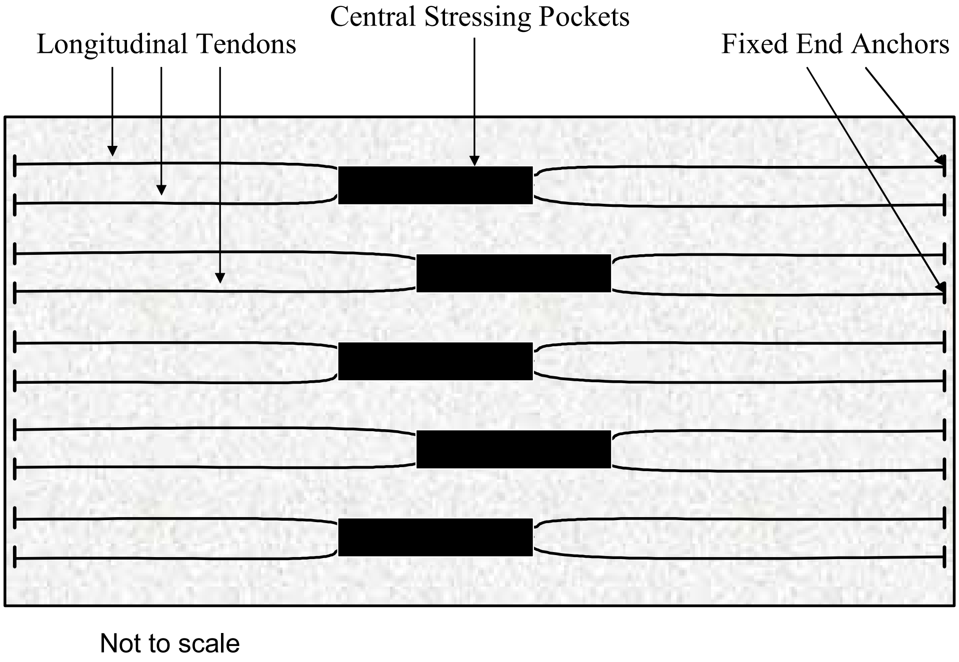


Figure 9.3 Proposed longitudinal tendon layout for PCP in Hillsboro, Texas

Accommodating two tendons in each stressing pocket will cut their number in half, but the most important benefit will be a reduction in stress concentrations at mid-slabs, where the central stressing pockets are located. A reduction of the tensile stresses greatly reduces the chances of cracks appearing at the central stressing pockets, as explained in section 9.4.2 below. Finally, as it was stated in Chapter 6, both longitudinal and transverse tendons should be bonded using a good-quality grout available in the market or a cement and water paste. Using bonded tendons will reduce the loss of prestress applied to the strands due to a potential failure of the anchoring system.

9.4.2 CENTRAL STRESSING POCKET

The shape of the central stressing pockets used for the PCP in McLennan was probably the feature that most affected the overall quality of the pavement. Early after the pavement was constructed, the appearance of cracks around the central stressing pockets required special investigation. The typical cracking pattern observed around the stressing pockets is shown in Figure 9.4.

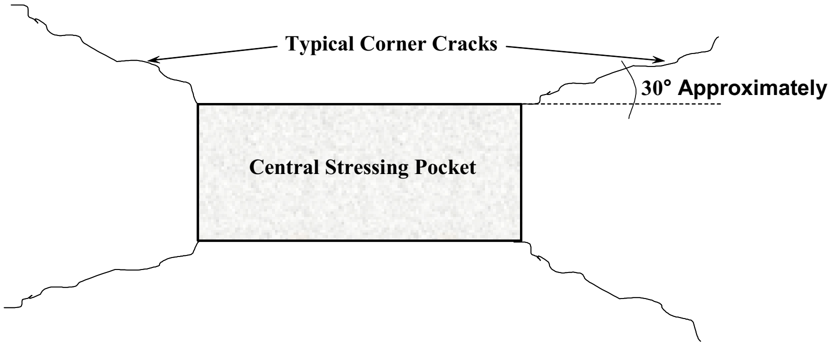


Figure 9.4 Cracking pattern observed around central stressing pockets

The results from the investigation of the cracking indicated that stress concentrations at the zone of central stressing pockets and other factors might have contributed to their development. Probably the factor that contributed the most was the effect of wheel load stresses combined with stress concentrations. Because pockets are closely spaced in the central region of the slabs, stress concentrations started at the corners and propagated from one pocket to another, and in the worst case, along the slab. Appendix A shows the cracking found in all the PCP slabs during the latest condition survey conducted in April 2001. The results from the investigation also confirm what was already mentioned in Chapter 4: that the cracks are not structurally significant and have not caused any further distress affecting the serviceability of the pavement.

To avoid this inconvenience in the new PCP to be constructed in Hillsboro, it is recommended that the shape of the central stressing pocket be changed from rectangular with sharp corners to rectangular with rounded corners, as shown in Figure 9.5. Changing the shape of the stressing pockets will reduce the stress concentrations because there will not be an abrupt change in the geometry of the pocket that will cause the development and propagation of cracks. A precast prestressed concrete pavement built in Georgetown, Texas, used this type of stressing pocket with good results (Ref 6). After one year of construction, no cracks have developed around the pockets.

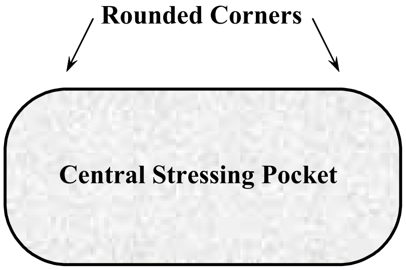


Figure 9.5 Recommended shape of stressing pockets for new PCP in Hillsboro

9.4.3 LENGTH OF PCP SLAB

The evaluation of the performance of the PCP slabs described in Chapter 4 showed that the length of the slab has a significant influence on its behavior. The continuous monitoring tasks performed for the evaluation of the horizontal and vertical movements of the slabs have yielded two conclusions as follows:

1. The length of the PCP slab does not significantly influence the vertical (curling) movement at the end of the slab.
2. The longer the PCP slab, the greater its horizontal displacement estimated by theory and measured in the field.

The history of 17 years of instrumentations of the PCP in McLennan County shows that horizontal movements measured for long slabs have almost doubled. However, for short slabs movements have been almost steady. A possible explanation for this is the assumption that the friction underneath long slabs decreases due to relatively large horizontal movements. For short slabs, the movement does not cause a significant depletion of the original friction between the slab and its supporting layer.

Based on the slab movements observed in the PCP in McLennan it was decided to limit the length of the slabs from 250ft to 350 ft The results of the PCP design performed in Chapter 5 recommend using 300 ft long slabs. Using this length of slab will optimize two conditions: the riding quality and the economics of the pavement. Additionally, the risk of experiencing increasing horizontal movements along time will be diminished.

9.5 Summary

This chapter discussed some of the developments that resulted from the new design of the PCP to be constructed in Hillsboro, Texas. A series of improvements in design, construction, and monitoring tasks were summarized. Additionally, the most important construction practices to be implemented were explained and include different layouts of prestressing strands, both transverse and longitudinal; a change in the shape of the central stressing pocket from rectangular with sharp corners to rectangular with rounded corners; and the use of shorter PCP slabs.

Finally, Table 9.1 summarizes various construction tasks that were conducted for the construction of the old PCP in McLennan County. These tasks were evaluated, and based on the results, either the same construction procedure is recommended when design and construction procedures were successful, or else modifications are suggested in those cases where the end results of the task were not as desired or optimum.

Table 9.1 Evaluation of construction tasks and recommended modifications

Construction Task	Evaluation	Modification
Mid-slab stressing	Worked fine in general	Shape of stressing pocket and tendons layout
Two stage stressing	Provided desired results	Apply accordingly
Transverse stressing	Insufficient prestress	Change procedure
Transverse expansion joint	Worked fine	Use similar joint hardware
Multiple strip placement	Adequate	Apply as needed for constructing three strips
Unbonded tendons	Although worked fine are not recommended	Grout tendons
Mid-panel anchoring	Worked fine	Use similar technique

10. CONCLUSIONS AND RECOMMENDATIONS

10.1 Introduction

This chapter summarizes the most relevant results of this research study. First, general conclusions that respond to the primary objective and subobjectives of the conducted research, stated in Chapter 1, are presented. Next, specific conclusions that refer to particular developments that resulted from the investigation are presented. Finally, there is a series of recommendations pertaining to the application of the PCP technology, followed by a summary statement.

10.2 General Conclusions

The highway pavement industry requires constant innovations because the rapidly growing numbers of traffic volume and load weight demand the establishment and application of more reliable design and construction practices. This investigation has provided evidence of how the PCP technology is for the most part reliable and economically feasible. Previous experiences with this technology show that the performance of almost all the PCPs constructed in the United States has surpassed expectations with the exception of those with only longitudinal post-tensioning constructed as part of a FHWA program.

A review of the application of this paving technology has shown that construction of high-performance PCP projects in the United States, in general, offers the following advantages:

1. Better utilization of construction materials: Thinner slabs require use of less concrete, which in turn means a reduction of the problems associated

with temperature differentials common in conventional concrete pavements.

2. Constructing thinner slabs also permits an effective "thick slab" due to prestressing to replace a thinner slab without impact to overhead clearances during reconstruction.
3. Application of prestressing forces to the PCP slab makes use of the vast compressive strength of the concrete, causing a reduction or even elimination of undesired concrete tensile stresses that cause cracking of the pavement.
4. Construction of long-spaced transverse joints in the PCP provides a smooth riding surface and thus fewer joint-related problems.
5. If overloads are temporarily imposed on the PCP, they can be resisted without causing permanent damage to the structure.
6. If the design and construction phases are successfully conducted, the good performance of the PCP will be ensured to a degree that it might turn into a zero-maintenance pavement.

On the other hand, as with all the existing paving methods, the PCP technique is not faultless. However, its shortcomings are related more to unawareness about its application. The most commonly mentioned inconveniences of PCPs, according to pavement engineers, include the following:

1. Engineers are not familiar with the procedure, or they simply are not aware of the existence of the technique.
2. There are no current standards for the design and construction of PCPs. Unlike conventional concrete pavements, the design of PCPs requires the application of uncommon procedures and algorithms.
3. Initially the cost of PCPs is higher due to the cost of transverse joints and hardware, which represents between 5 and 10% of the total construction cost of the PCP.

To overcome these inconveniences, the work conducted and described in this study has attempted to take a step forward in the promotion of PCP technology. The knowledge acquired from the successes and failures of previously constructed PCP sections all over the world has served to increase our understanding of the areas in which this technology needs to be reinforced. It is strongly believed that as the application of PCPs becomes more common, the technique will compete with conventional paving methods in every aspect.

In Texas there is ample documentation of the research on this type of paving technology. The PCP test section that was constructed in McLennan County more than 17 years ago, which was designed for a 20-year-life period, is still working efficiently with no signs of significant deterioration. Early after construction, this pavement showed only mild longitudinal cracking problems that did not diminish the structural capacity of the PCP in the long-term. According to the investigations, those cracks originated at the corners of the rectangular central stressing pockets located at the center of the slabs and then propagated longitudinally to different extents.

As an attempt to improve the overall quality of the PCP technique, the work conducted under this study developed an innovative approach for designing and constructing a cost-effective state-of-the-art pavement structure. Likewise, this approach was applied to designing a new PCP in Texas, and materials specifications and construction guidelines have been recommended. The design approach proposed in this study is a mechanistic-empirical procedure, which bases its success on the combination of theoretical analysis and measured observations. In this way, materials specifications and construction guidelines were prepared for the given project and could be implemented in other case studies with some modifications. The PCP that was designed herein will be constructed on IH 35 in Hillsboro, Texas.

10.3 Specific Conclusions

The following conclusions are made regarding the stated subobjectives and the various procedures conducted throughout this study:

1. A mechanistic-empirical design procedure has been developed and can be used as a guide to prepare designs, specifications, and construction guidelines for a particular project.
2. The procedure proposed herein has been calibrated and validated using both field observations and empirical correlations. The behavior of the new pavement was predicted as a function of traffic and environmental conditions.
3. The procedure suggested in this study has been implemented, and design, specifications, and construction guidelines have been prepared for a particular project.

The flexibility of the design process of a PCP allows pavement engineers to provide solutions based on necessities. There is no an absolute solution to PCP design. However; there is always an optimum solution that balances the best combination of riding quality and the economics of the project.

In addition to the development and implementation of the procedure recommended in this study, improvements have been made in other aspects related to PCPs, including the following:

Recommendations for treatment of foundation and embankment soils have been drafted, and they are applicable to any type of pavement construction. It has been noticed that the plastic characteristics of the natural soils or the soils forming some of the embankments on IH 35 and other routes in Texas sometimes have been underestimated. The recommendations given in Chapter 6 promote the evaluation and treatment of those soils because they have a great impact on the performance of the pavement structure.

Additionally, Chapter 6 provides the required specifications for fresh and hardened concrete, post-tensioning steel strands, anchors, transverse joints, ducts, etc., and explains the basic tasks to be conducted for the construction of the designed PCP.

A monitoring plan for the new PCP that involves the development of tasks to insure the quality of the pavement is detailed in Chapter 7, and recommendations to conduct a reliable state-of-the-art instrumentation for the long and short terms are provided. A master plan for periodic monitoring and instrumentation is proposed along with testing and instrumentation frequencies.

One of the reasons behind the development of this study was the desire to compare the newly designed PCP to a control CRCP section that is already in service. Because of time constraints, it was not feasible to conduct this task because the PCP has not been constructed. Chapter 8 presents a preliminary methodology to achieve that objective. The comparison considers two aspects of the pavements to be compared: performance and life-cycle cost. The results of the evaluation suggest that PCPs are initially more costly than CRCPs; however, past experiences have shown that a PCP could outperform a CRCP in the long-term when working under the same conditions.

A description of the major developments of this study is given in Chapter 9. Improvements over the previous in-home experience (old PCP in McLennan County constructed in 1985) are sought. In those cases, specific features of either design or construction procedures were demonstrated to be so successful in the previous experience that no major changes were suggested for the new PCP section.

10.4 Recommendations

Due to the extensive number of tasks associated with the development of this study, the recommendations that can be outlined for improvement are also abundant. However, the following paragraphs condense the most important recommendations for ensuring the success of the PCP technology.

As with any type of product or service, the dissemination of information plays a major role in the success of the PCP technology. It is very important that the advantages of this technique, especially in the long-term, are communicated to material producers, government agencies, and contractors. This will provide the pavement industry with a better understanding of the PCP capability and applicability. Although this construction technique is not a panacea, it offers a very reliable option, especially for highly trafficked highways with overhead clearance problems where high-performance pavements are required.

Probably the most significant reason for the lack of widespread use of the PCP technique is the absence of a standard design method. It is recommended that pavement engineers work towards the creation of a standardized design method. The developments in this study have contributed in some way to achieve that goal. Likewise, the application of computer-aided software serves two purposes: the attractiveness of the PCP technology and the rapid solution of specific designs. The PSCP4 program developed during this study should be completed and distributed to pavement engineers for their evaluation and improvement.

With regard to the construction of PCPs, the major difference between this paving technique and conventional methods resides in the use of transverse expansion joints and application of post-tension forces to the concrete slabs. For the production of concrete, the same construction practices for conventional concrete pavements apply to PCPs. For instance, concrete should be placed when cool ambient temperature prevails. Loss of moisture in the concrete should be minimized by conducting paving activities late in the afternoon, and by applying a proven quality curing compound on top of the concrete slab right after finishing the pavement texture. Paving activities should not be conducted if extreme environmental conditions occur or are expected for a period of 24 to 36 hours following concrete placement.

It is highly recommended that the monitoring plan described in Chapter 7 be pursued as closely as possible. Experience shows that data collected during monitoring activities always contribute in the development of better design and construction practices. In fact, the FHWA recognizes that pavement research saves more than $50 million in highway construction costs each year.

10.5 Summary

It is hoped that with the results obtained from the proposed PCP developments herein and those from forthcoming projects, pavement design engineers, state agencies, and contractors will promote the widespread use of this paving option, which will improve the application of rigid pavements in the medium and long terms. As the construction of PCPs becomes more frequent, this paving technique will be recognized for being effective and for providing durable, almost maintenance-free pavements.

This study has attempted to advance the development of a design procedure, specifications, and construction guidelines for PCP towards achieving more reliable portland cement concrete pavements. It is hoped that it has provided the design for a feasible paving technique that introduces innovations believed to be applicable for the construction of high-performance pavements required for today’s highly trafficked highways.

REFERENCES

1. Libby, James. “Modern Prestressed Concrete-Design Principles and Construction Methods.” 4th Edition. New York: Van Nostrand Reinhold, 1990.
2. Chia-Pei, Chou, and Chang Chien-Yen. “Algorithm for Determining Weight Factors of Pavement Distress Index.” *Transportation Research Record 1592*. Department of Civil Engineering, National Taiwan University, Taipei, Taiwan, Republic of China, 1997.
3. ACI Committee 325. “Recommendations for Designing Prestressed Concrete Pavements.” Title No. 85-S44 (ACI325.7R-88). *ACI Structural Journal,* July/August 1988.
4. Klunker, Fritz. “Status and Developments in the Construction of Prestressed Concrete Runways in European Airports.” *Proceedings of the 2nd International Conference on Concrete Pavement Design*, held at Purdue University, April 14-16, 1981. Session 6: Airport Pavements. Purdue University, West Lafayette, Indiana.
5. Cable, Neil, Ned Burns, and B. Frank McCullough. “New Concepts in Prestressed Concrete Pavements.” Research Report 401-2. Center for Transportation Research. The University of Texas at Austin, December 1985.
6. Merritt, Dave, B. Frank McCullough, Ned Burns, and Anton Schindler. “The Feasibility of Using Precast Concrete Panels to Expedite Highway Pavement Construction.” Research Report 9-1517-1. Center for Transportation Research. The University of Texas at Austin, February 2000.
7. Tayabji, Shiraz, Ernest Barenberg, Wade Gramling, and Paul Teng. “Prestressed Concrete Pavement Technology Update.” 7th International Conference on Concrete Pavements. Orlando, Florida, September 2001.
8. Federal Highway Administration (FHWA). “Technological Review of Prestressed Pavements.” Report No. FHWA-RD-77-8. Washington, D. C., December 1976.
9. Powers, Richard, and John Zaniewski. “Nine Year Performance Evaluation of Arizona’s Prestressed Concrete Pavement.” *Transportation Research Record 1136.* Washington, D. C., January 1987.
10. Internet Yahoo Maps. Website http://www.yahoo.com.
11. Mendoza-Diaz, Alberto, B. Frank McCullough, and Ned Burns. “Design of the Texas Prestressed Concrete Pavement Overlays in Cooke and McLennan Counties and Construction of the McLennan County Project.” Research Report 555/556-1. Center for Transportation Research. The University of Texas at Austin, February 1986.

12. Mandel, Elliot, Jose Tena-Colunga, and Kenneth Hankins. "Performance Tests on a Prestressed Concrete Pavement-Presentation of Data." Research Report 556-1. Center for Transportation Research. The University of Texas at Austin, November 1989.

13. Suprenant, Bruce. "Why Slabs Curl?" Part II: Factors affecting the amount of curling. *Concrete International Magazine.* April 2002.

14. Joe Mahoney et al. *"EVERCALC User's Manual."* Washington State Department of Transportation (WSDOT). Pavement layer back-calculation program, February 1995.

15. Stock, F. "The Design of Prestressed Concrete Pavements." *The Structural Engineer.* Volume 66. No. 11/7. June 1988.

16. Harvey, Treybig, B. Frank McCullough, Phil Smith, and Harold Von-Quintus. "Overlay Design and Reflection Cracking Analysis for Rigid Pavements." Volume 1−Development of New Design Criteria. Research Report FHWA-RD-77-66. Federal Highway Administration (FHWA), August 1977.

17. American Association of State Highway and Transportation Officials (AASHTO). *"AASHTO Guide for Design of Pavement Structures 1993."* Washington, D.C., 1993.

18. American Concrete Institute (ACI). "Recommendations for Designing Prestressed Concrete Pavements." *ACI Structural Journal.* Volume 85. No. 4, July−August 1988.

19. Rozycki, Dan, and Robert Rasmussen. "Assessment of Slab-Base Interaction in PCC Airfield Pavements." Research Paper. Transtec, Inc. Austin, Texas, 1998.

20. Mendoza-Diaz, Alberto; B. Frank McCullough, and Ned Burns. "Behavior of Long Prestressed Pavement Slabs and Design Methodology." Research Report 401-3. Center for Transportation Research. The University of Texas at Austin, September 1986.

21. Wesevich, James, B. Frank McCullough, and Ned Burns. "Stabilized Sub-base Friction Study for Concrete Pavements." Research Report 459-1. Center for Transportation Research. The University of Texas at Austin, April 1987.

22. Medina, Cesar, and B. Frank McCullough. "Current Status of the Rigid Pavement Database in the State of Texas." Research Report 1778-1. Center for Transportation Research. The University of Texas at Austin, August 1999.

23. Koontz, James C. "Historic Trends and Future Consequences of Projected Traffic Flows along Rural Interstate 35." Research Report 1326-1. Center for Transportation Research. The University of Texas at Austin, August 1996.

24. Dossey, Terry, and Angela Weissmann. "A Continuously Reinforced Concrete Pavement Database." Research Report 472-6. Center for Transportation Research. The University of Texas at Austin, November 1989.

25. Texas Department of Transportation (TxDOT). Interstate Project Office-Waco District. Facsimile Information, July 2000.

26. Shell Oil Company and Koninklijke/Shell Laboratorium (KSLA). "*Input Guide for the BISAR Computer Program.*" Amsterdam, The Netherlands, 1978.

27. Rasmussen, Robert. "1993 AASHTO Pavement Design Spreadsheet Version 1.01." Transtec Incorporated and Saguaro Engineering and Technology Corporation. Austin, Texas, 1996.

28. ACI Committee 325 Concrete Pavements. "Recommendations for Designing Concrete Pavement Slabs with Posttensioned Steel Tendons." *ACI Journal.* Proceedings, Volume 121.

29. Westergaard, H. M. "Analysis of Stresses in Concrete Roads Caused by Variations of Temperatures." *Public Roads*, May 1927: 54−60.

30. Tayabji, S. D., B. E. Colley, and P. J. Nussbaum. "Prestressed Pavement Thickness Design." *Transportation Research Record 888.* Washington, D. C., 1982.

31. Nussbaum, P. J., S. D. Tayabji, and A. T. Ciolko. "Prestressed Pavement Joint Designs." *Transportation Research Record 888.* Washington, D. C., 1982.

32. United States Department of Agriculture. Soil Conservation Service in Cooperation with Texas Agricultural Experiment Station. "Soil Survey of Hill County, Texas." Austin, Texas, 1991.

33. American Society for Testing and Materials (ASTM). *ASTM Standards*. ASTM D2487-00, "Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)." March 2000.

34. Texas Department of Transportation (TxDOT). "Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges." Austin, Texas, 1993.

35. Radzikowski, H. A. and John J. Laing. "Compaction of Highway Embankments and Surfaces." Division of Development, Office of Operations, Bureau of Public Roads. Washington, D. C., September 1959.

36. American Society for Testing and Materials (ASTM). *ASTM Standards*. ASTM D3282-93, (Reapproved 1997) "Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes."

37. McCullough, B. F., et al. "Determination of Potential Vertical Rise for Highway Embankments." Preliminary Research Report 221. Center for Transportation Research. The University of Texas at Austin, Texas, 1981.

38. Texas Department of Transportation (TxDOT). *"Manual of Testing Procedures."* Volume I. Soils Section. Austin, Texas, September 1995.

39. Kennedy, Thomas et al. "Notes on Compaction." Modern Pavement Materials Class Handouts. Spring 2000. The University of Texas at Austin, March 2000.

40. Kennedy, Thomas, Robert Smith, Richard Holmgreen, and Maghsoud Tahmooressi. “An Evaluation of Lime and Cement Stabilization.” Presentation prepared for the 66th Annual Meeting of the Transportation Research Board (TRB), January 1987.

41. Senior, S. A., S. I. Szoke; and C. A. Rogers. “Ontario's Experience with Reclaimed Materials for Use in Aggregates.” Presented at the International Road Federation Conference. Calgary, Alberta, 1994.

42. Aitcin, P.C., and A. Neville. “High-Performance Concrete Demystified.” *Concrete International*. January Volume 15. No. 1 (1993): 21−26.

43. Goodspeed, C. H., S. Vanikar, and R. A. Cook. “High-Performance Concrete Defined for Highway Structures.” *Concrete International*. February Volume 18. No. 2 (1996): 62−67

44. Portland Cement Association (PCA). “Design and Control of Concrete Mixtures.” Thirteen Edition, by Steven H. Kosmatka and William C. Panarese. *Engineering Bulletin.* Skokie, Illinois, 1988.

45. American Society for Testing and Materials (ASTM). ASTM C 150, “Standard Specification for Portland Cement.” 1986.

46. Mehta, Kumar P., and Paulo J. M. Monteiro. “Concrete: Microstructure, Properties, and Materials.” College Custom Series. McGraw Hill Companies, Inc. Second Edition, 1993.

47. Berry, E. E., and V. M. Malhotra. *ACI Proceedings.* Volume 77. No. 2. 1980: 59−73.

48. Manmohan, D., and P. K. Mehta. “Cement-Concrete-Aggregates.” Volume 3. No. 1. 1981: 63−67.

49. Shah, S. P., and B. V. Rangan. *ACI Proceedings.* Volume 68. No. 2. 1971: 126−135.

50. Eastern Resource Center−Federal Highway Administration (FHWA). PCEF QA/QC Sub-Committee. “Specification Guidelines for Precast/Prestressed Concrete Bridge Products.” Internet website:

    http://www.fhwa.dot.gov/resourcecenters/eastern/infrastr/pcefgdsp.htm

51. Sason, Augusto. “An Evaluation of Degree of Rusting on Prestressed Concrete Strands.” *PCI Journal.* Volume 37. No. 3. May-June 1992: 25−30.

52. Grater, Stefan. “An Investigation toward Performance-Oriented Specifications for Portland Cement Concrete Pavement.” Doctoral Dissertation. The University of Texas at Austin, December 1996.

53. American Association of State Highway and Transportation Officials (AASHTO). *“AASHTO Guide for Design of Pavement Structures.”* Volume 2. 1986.

54. Trevino-Frias, Manuel. “Techniques and Procedures for Bonded Concrete Overlays.” Doctoral Dissertation. The University of Texas at Austin, December 2002.

55. Schindler, Anton K., and B. Frank McCullough. “Temperature Control During Construction to Improve the Long-Term Performance of Portland Cement Concrete Pavements.” Research Report Number 1700-2. Center for Transportation Research. The University of Texas at Austin, May 2002.

56. Project 1700 Researchers. “Instrumentation Plan for Verifying/Calibrating and Implementing Project 1700 Developments.” Techmemo IAC 00-1700-6.1. Center for Transportation Research. The University of Texas at Austin, February 2003.

57. National Ready Mix Concrete Association (NRMCA). “Standard Practice for Rapid Determination of Water Soluble Chloride in Freshly Mixed Concrete, Aggregate, and Liquid Admixtures.” *NRMCA Technical Information Letter No. 437.* March 1986.

58. Carrasquillo, P. M. “Concrete Strength Testing.” *Significance of Tests and Properties of Concrete and Concrete Making Materials.* ASTM STP 169C. Philadelphia, PA. 1994.

59. Haas, Ralph; Ronald Hudson, and John Zaniewski. “Modern Pavement Management.” Krieger Publishing Company. Malabar, Florida. Original edition, 1994.

60. Ramaiah, Shantala V. “Estimating in situ Strength of Concrete Pavements under Various Field Conditions.” Masters Thesis. The University of Texas at Austin, May 2001.

61. Dallas Semiconductor. i-Buttons and Accessories. Internet Website http://www.maxim-ic.com/index.cfm.

62. CEP. Vibrating Wire Embedment Strain Gauges Used to Measure Strain in Reinforced Concrete and Mass Concrete. Internet Website http://www.cep.com.sg/instruments/sg-embedment.html.

63. VISHAY Manufacturers. Semiconductors and Passive Components. Internet Website http://www.vishay.com.

64. Schindler, Anton K. “The Design of a High Performance Concrete Pavement in the Waco District.” Masters Thesis. The University of Texas at Austin, May 1999.

65. Carey, W. N., and P. E. Irick. “The Pavement Serviceability-Performance Concept.” *Highway Research Bulletin 250.* 1960.

66. Texas Department of Transportation, TxDOT. “Average Low Bid Unit Price-Construction.” Internet Website http://www.dot.state.tx.us/business/avgd.htm.

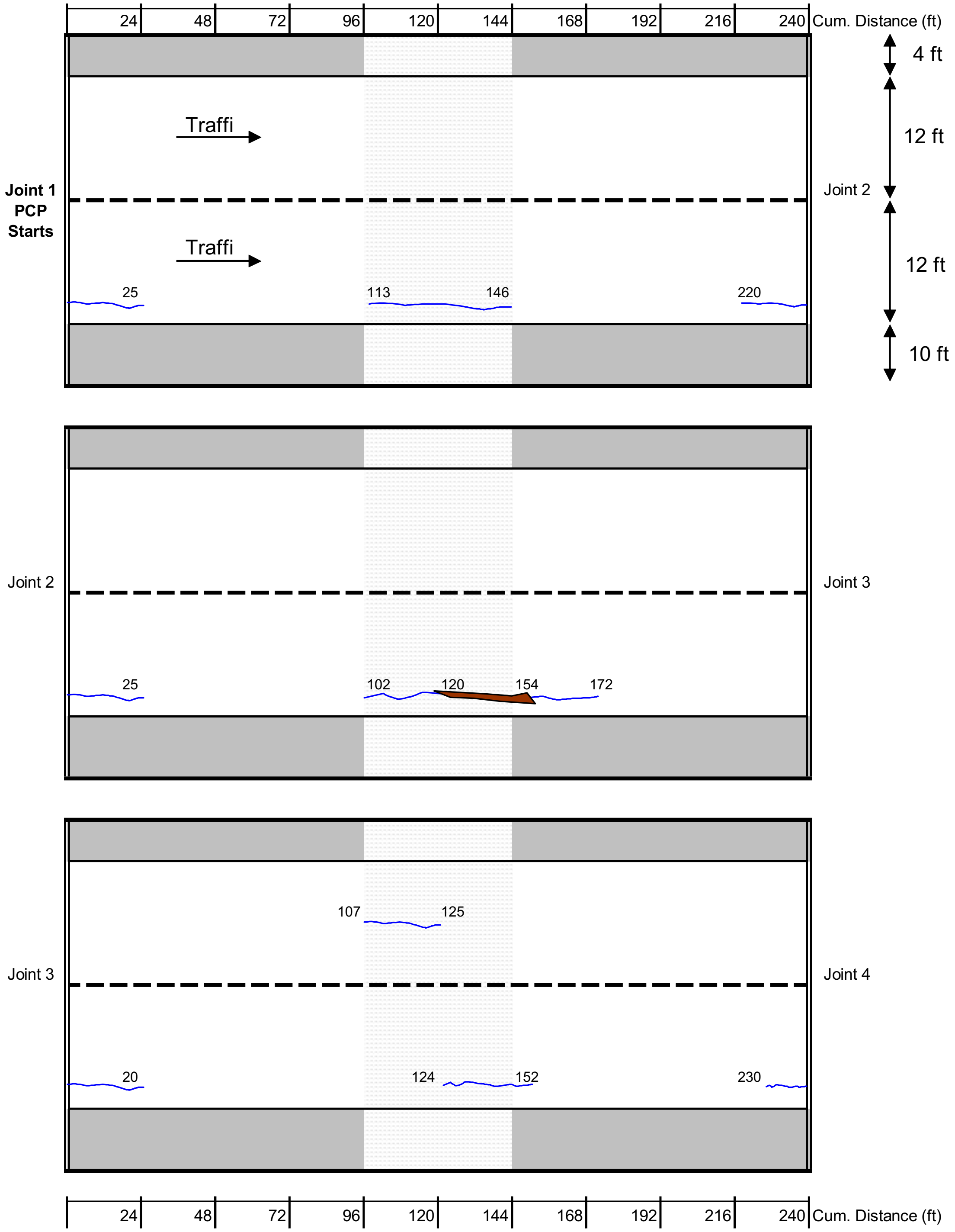
67. Economy History Services. “What Was the Inflation Rate Then? Internet Website http://www.eh.net/hmit/inflation/inflationr.php.

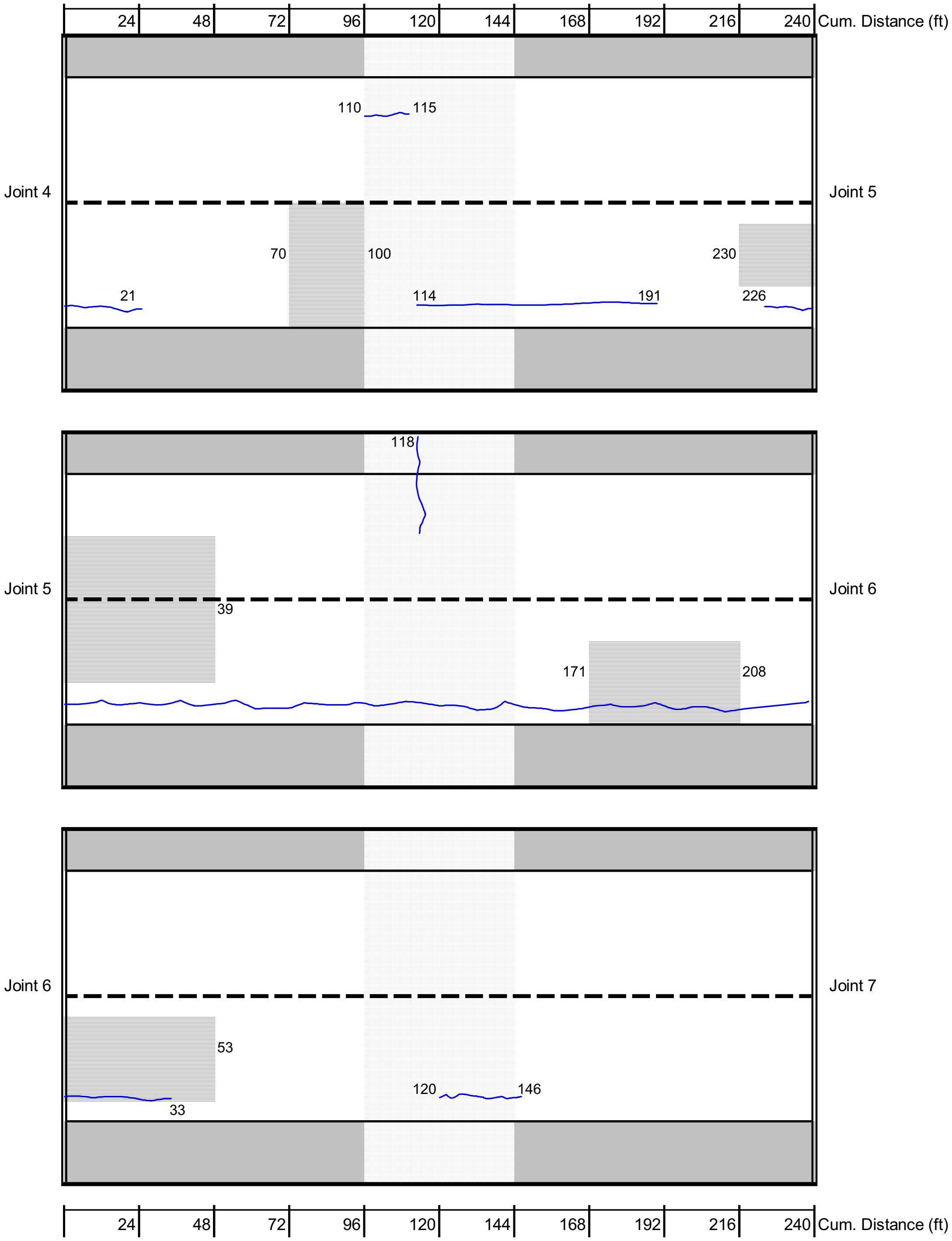
68. Booz Allen & Hamilton Inc., Hagler Bailly, and Parsons Brinckerhoff. "California Life-Cycle Benefit/Cost Analysis Model (Cal-B/C." *User's Guide Developed for the California Department of Transportation (CalTrans).* September 1999.

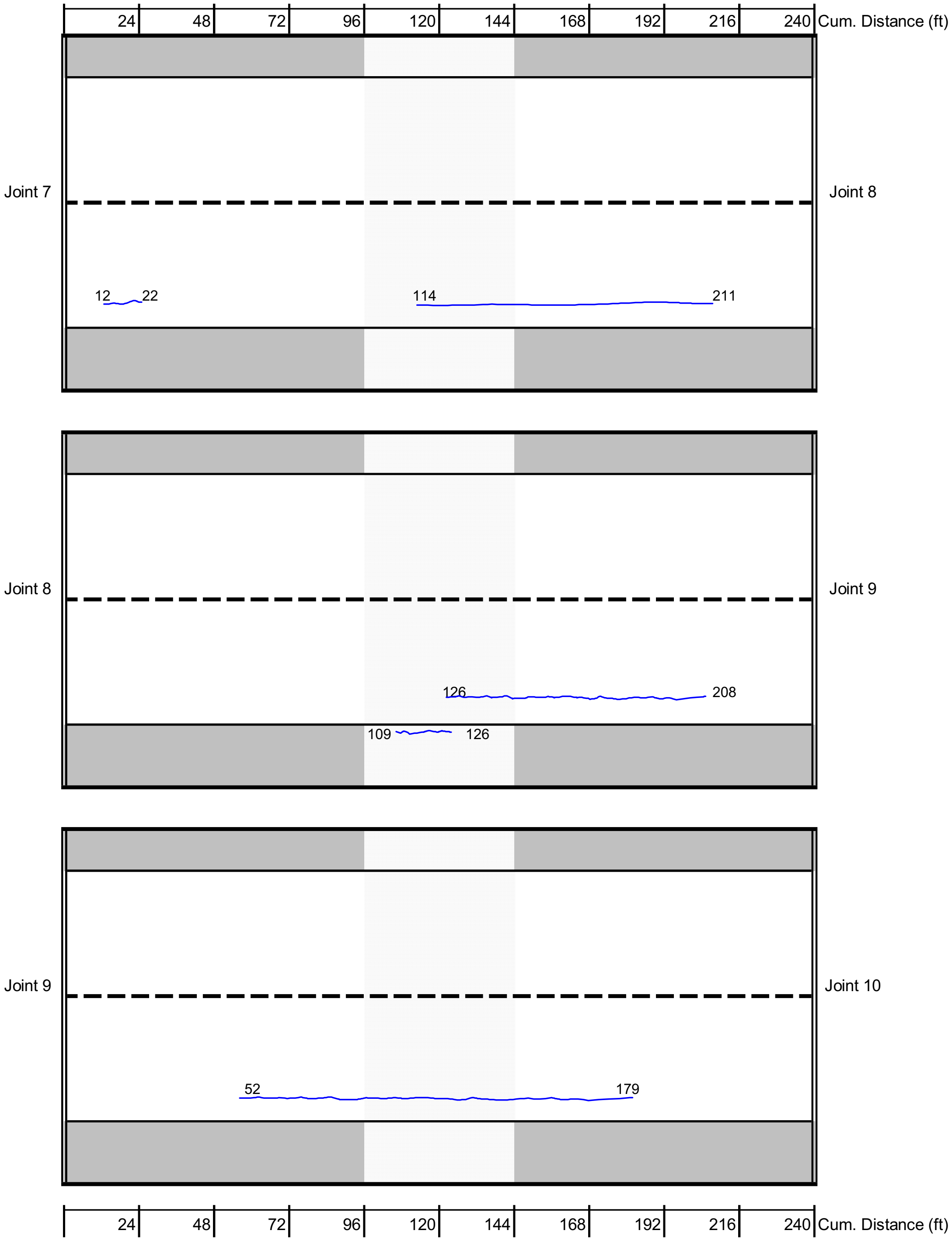
69. Mandel, Elliot, Jose Tena-Colunga, Kenneth Hankins, Ned Burns, and B. Frank McCullough. "The Performance of a Prestressed Concrete Pavement." Research Report 556-4F. Center for Transportation Research. The University of Texas at Austin, November 1989.

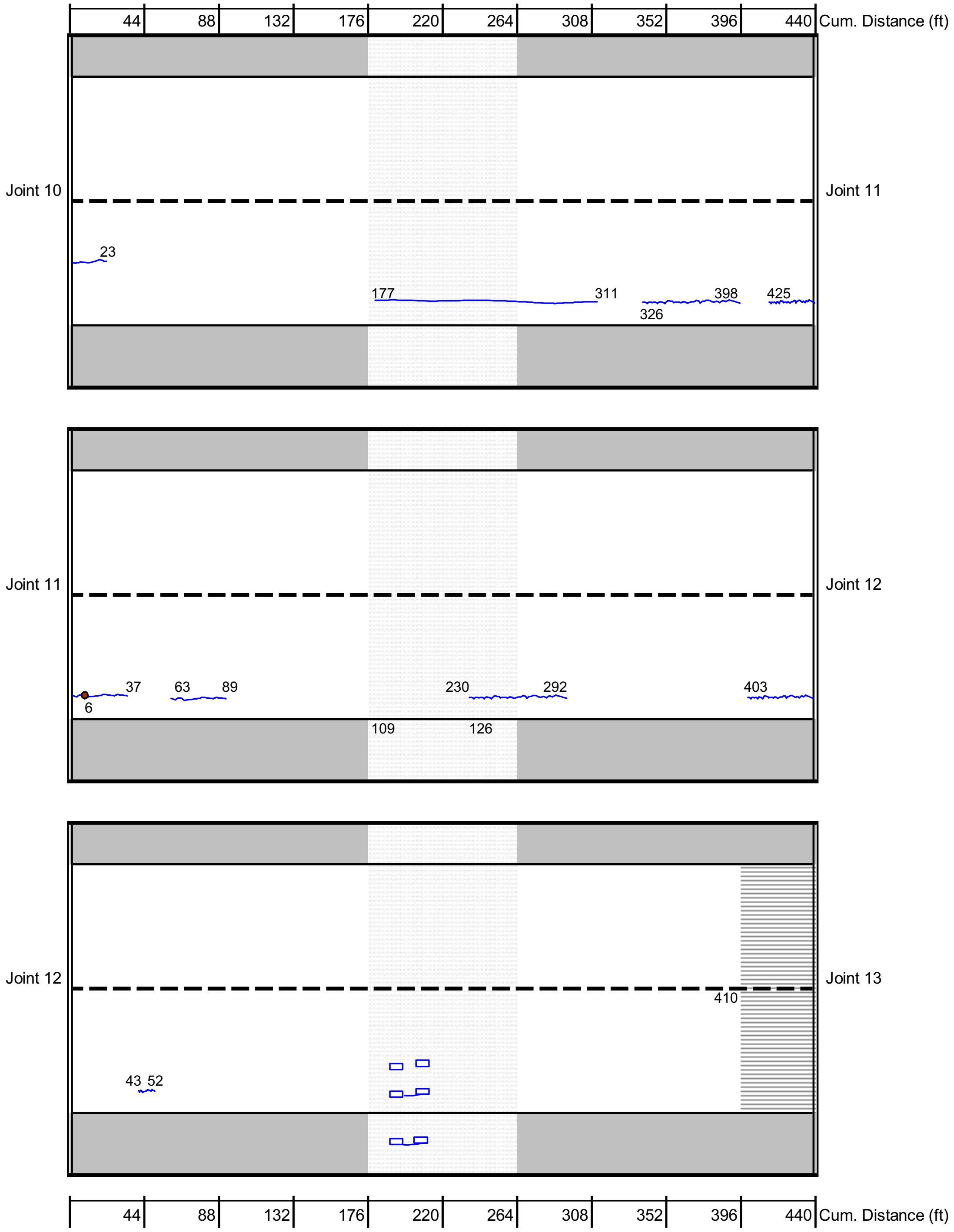
APPENDIX A

SCHEMATICS OF CONDITION SURVEY OF EXISTING PCP SECTION CONDUCTED IN APRIL 2001

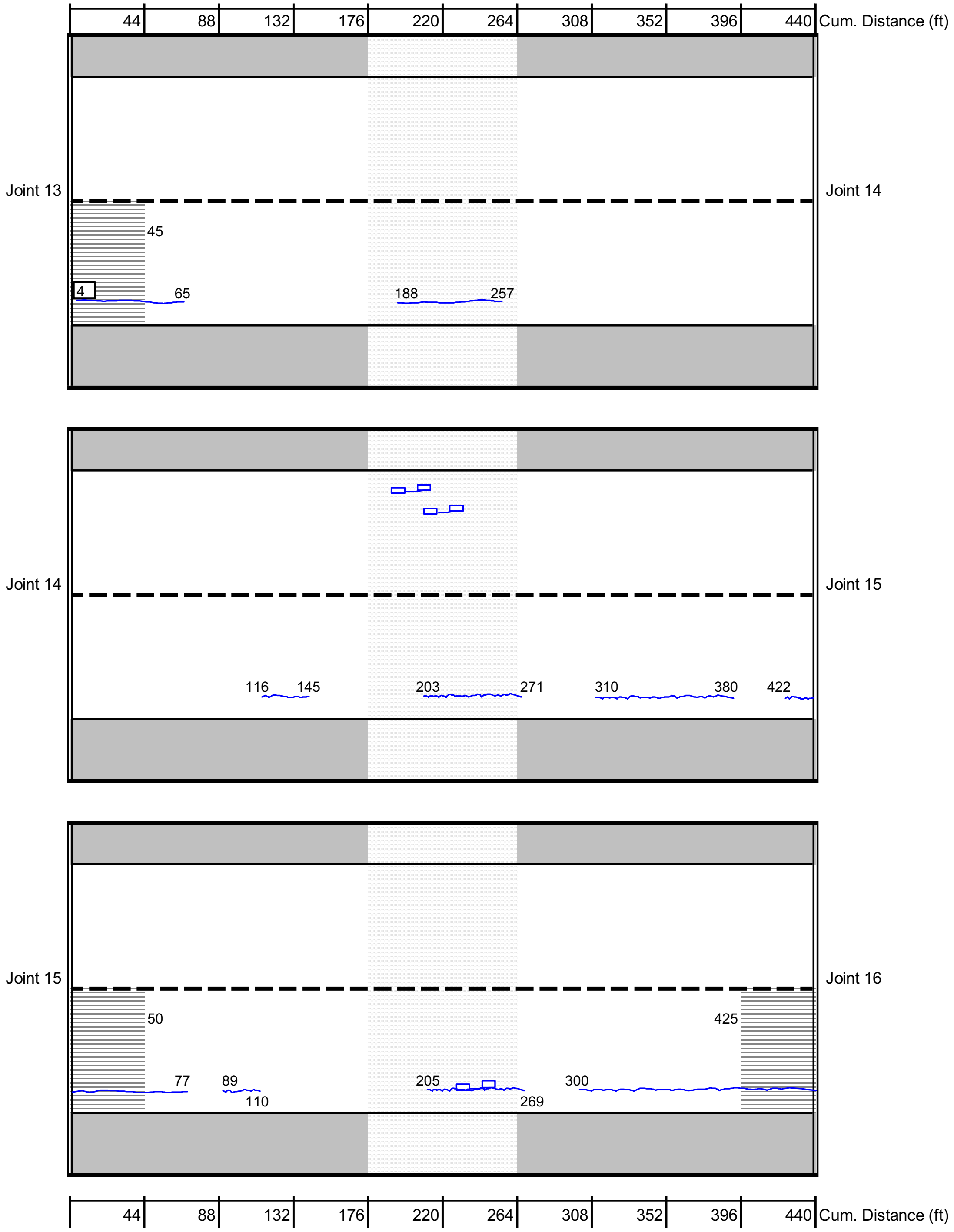

24
48
72
96
120
144
168
192
216
240
Cum. Distance (ft)
4 ft
12 ft
12 ft
10 ft
Traffi
Traffi
Joint 1
PCP
Starts
Joint 2
25
113
146
220
Joint 2
Joint 3
25
102
120
154
172
Joint 3
Joint 4
107
125
20
124
152
230
24
48
72
96
120
144
168
192
216
240
Cum. Distance (ft)

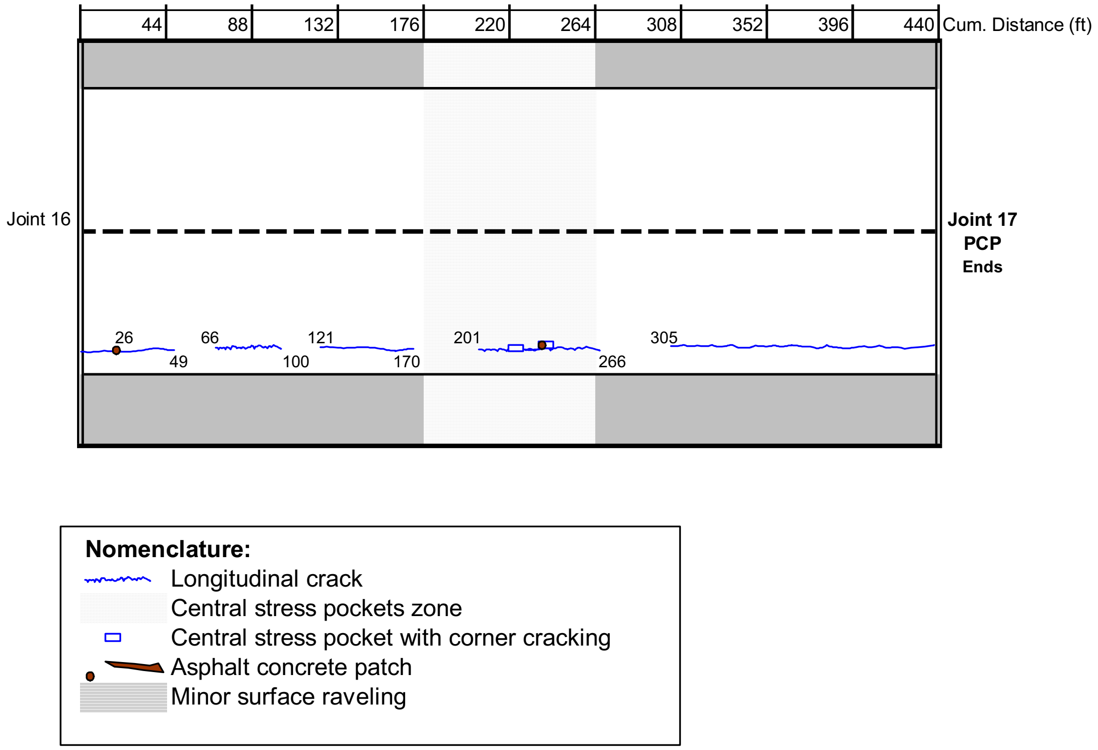

24
48
72
96
120
144
168
192
216
240
Cum. Distance (ft)
Joint 4
Joint 5
110
115
70
100
230
21
114
191
226
118
Joint 5
Joint 6
39
171
208
Joint 6
Joint 7
53
33
120
146
24
48
72
96
120
144
168
192
216
240
Cum. Distance (ft)


24
48
72
96
120
144
168
192
216
240
Cum. Distance (ft)
Joint 7
Joint 8
12
22
114
211
Joint 8
Joint 9
126
208
109
126
Joint 9
Joint 10
52
179
24
48
72
96
120
144
168
192
216
240
Cum. Distance (ft)



44
88
132
176
220
264
308
352
396
440
Cum. Distance (ft)
Joint 10
Joint 11
23
177
311
398
425
326
Joint 11
Joint 12
37
63
89
230
292
403
6
109
126
Joint 12
Joint 13
410
43
52
44
88
132
176
220
264
308
352
396
440
Cum. Distance (ft)


44
88
132
176
220
264
308
352
396
440
Cum. Distance (ft)
Joint 13
Joint 14
45
4
65
188
257
Joint 14
Joint 15
116
145
203
271
310
380
422
Joint 15
Joint 16
50
425
77
89
110
205
269
300


44
88
132
176
220
264
308
352
396
440
Cum. Distance (ft)
Joint 16
Joint 17
PCP
Ends
26
49
66
100
121
170
201
266
305
Nomenclature:
Longitudinal crack
Central stress pockets zone
Central stress pocket with corner cracking
Asphalt concrete patch
Minor surface raveling

APPENDIX B

FWD RAW DATA FOR OUTSIDE LANE OUTSIDE WHEELPATH

```
R80    327   010607161I35K636F20
711031008002-06970267.3502111  8
150 0   305 610 914 121915241829 5.9   0     12    24    36    48    60    72
C:\WACO\                    .FWD
09161 ih0035
S    5170k6         31 25 328       88  77  Heights ............................
S    5256k6         31 25 328       88  77  Heights ............................
800 208027493735.012    .979
18  15  3.5 5   2   15  2   8
Ld 401   .994  87.5
D1 1213  1.003 .9954
D2 948   1.003 1.005
D3 943   1.008 .9933
D4 944   1     1.071
D5 945   1.006 1.047
D6 949   1.003 1.014
D7 947   1.011 1.010
D0 941   1.005 1.022
D* 946   .984  1.080
D0 950   1     1.045
Miles E. Hicks
11030600........................
0   0   0   0    ...............
*0000 +00.0
DtCty PxNnnnS
  Cty P Nnnn
*0000 +00.0
................................
10422.04-10422.0    0   1Peak...32  0   ......
12345...........................................................................
C12342222222222233333333444444444444444444444444444444444444444444444444444444444
................................................................................
.*******************************************************************************
................................................................................
................................................................................
LOAD ZONE - '98
*0000 +00.0
S      64k6            24 I60828        74  Heights ............................
 442  41  27  22  20  15  12  11  7023  1.61  1.07  0.88  0.78  0.61  0.49  0.45
 598  64  48  40  37  33  29  27  9498  2.50  1.90  1.56  1.45  1.29  1.13  1.07
 835  84  62  52  48  42  37  33 13260  3.31  2.43  2.06  1.89  1.66  1.45  1.30
1006 103  79  66  61  54  48  43 15989  4.06  3.12  2.61  2.39  2.12  1.89  1.69
'first test in south bound outside lane outside wheel path on first slab
S     116k6         31 23 I60833    88  73  Heights ............................
 445  94  87  75  63  48  37  29  7063  3.69  3.43  2.96  2.48  1.90  1.47  1.15
 601 114 108  93  75  57  45  36  9546  4.48  4.27  3.65  2.95  2.25  1.77  1.41
 839 153 143 124 104  81  62  50 13324  6.03  5.65  4.86  4.09  3.17  2.44  1.97
1008 177 166 144 122  95  74  59 16021  6.95  6.55  5.66  4.79  3.74  2.89  2.32
'middle of slab #1
S     220k6         31 23 I60834    88  72  Heights ............................
 441  54  48  37  31  27  24  20  7008  2.13  1.88  1.44  1.23  1.07  0.94  0.77
 597  62  54  40  35  28  23  20  9486  2.46  2.14  1.57  1.37  1.09  0.90  0.79
 852  95  84  65  58  47  41  35 13534  3.73  3.30  2.54  2.27  1.87  1.62  1.37
1027 115 100  80  70  59  50  43 16323  4.52  3.95  3.15  2.76  2.30  1.97  1.69
'end of slab #1
S     246k6         31 24 I60835    88  74  Heights ............................
 450  45  38  31  27  22  19  18  7151  1.76  1.49  1.24  1.06  0.88  0.75  0.69
 595  57  49  42  36  32  27  25  9455  2.26  1.94  1.67  1.43  1.24  1.06  0.97
 850  81  72  59  52  42  37  33 13511  3.17  2.81  2.33  2.04  1.65  1.47  1.30
1024  98  86  73  65  56  48  43 16268  3.86  3.39  2.87  2.54  2.19  1.90  1.68
'beginning of slab #2
S     355k6         31 23 I60837    88  73  Heights ............................
 463  36  36  30  26  22  19  17  7357  1.42  1.41  1.19  1.02  0.86  0.74  0.66
 600  53  49  37  33  27  25  22  9530  2.10  1.93  1.47  1.31  1.07  0.96  0.87
 850  76  69  57  51  43  38  34 13503  2.97  2.72  2.22  2.00  1.69  1.49  1.35
1046  93  85  72  64  54  48  44 16613  3.67  3.33  2.82  2.53  2.12  1.91  1.72
'middle of slab #2
S     463k6         31 23 I60838    88  73  Heights ............................
 445  65  55  35  29  22  20  18  7075  2.54  2.15  1.39  1.15  0.85  0.79  0.72
 598  86  72  53  42  32  29  24  9502  3.37  2.84  2.07  1.65  1.27  1.14  0.94
 847 123 104  78  63  50  43  37 13455  4.86  4.08  3.07  2.47  1.96  1.71  1.46
1017 150 127  95  76  61  53  45 16160  5.89  5.01  3.72  3.00  2.38  2.09  1.76
'end of slab #2
S     476k6         31 23 I60839    88  73  Heights ............................
 439  50  42  33  28  23  20  18  6972  1.97  1.64  1.31  1.11  0.91  0.78  0.70
 602  66  56  44  37  31  26  22  9570  2.61  2.21  1.73  1.44  1.23  1.02  0.85
 850  97  83  65  55  46  40  33 13507  3.82  3.25  2.54  2.16  1.83  1.57  1.31
1017 116  99  79  67  56  48  41 16152  4.56  3.88  3.10  2.65  2.20  1.87  1.61
'beginning of slab #3
```

```
S     590k6         31 24 I60841    88  74  Heights ..........................
 449  35  32  26  23  19  17  15  7127  1.39  1.25  1.02  0.89  0.76  0.65  0.59
 607  47  41  34  29  26  22  20  9637  1.85  1.60  1.32  1.14  1.03  0.86  0.79
 856  68  59  48  42  36  32  28 13602  2.67  2.31  1.89  1.66  1.41  1.24  1.11
1017  81  71  60  53  45  40  37 16160  3.17  2.78  2.36  2.07  1.78  1.56  1.44
'middle of slab #3
'middle of slab #3 and close to mile maker 351
S     695k6         31 24 I60843    88  74  Heights ..........................
 452  52  53  31  22  17  19  14  7186  2.05  2.10  1.23  0.87  0.68  0.74  0.54
 601  68  62  43  33  27  25  20  9546  2.69  2.45  1.71  1.30  1.06  0.97  0.77
 846  92  83  57  45  34  32  28 13435  3.62  3.26  2.25  1.78  1.34  1.26  1.09
1026 111  99  71  57  47  42  37 16299  4.35  3.89  2.80  2.22  1.83  1.67  1.45
'end of slab #3
S     711k6         31 24 I60844    88  74  Heights ..........................
 443  55  38  29  25  21  19  16  7039  2.17  1.50  1.12  0.98  0.84  0.73  0.64
 599  68  50  39  33  28  25  22  9522  2.67  1.96  1.52  1.31  1.11  0.97  0.87
 838  95  71  54  48  40  35  30 13308  3.74  2.81  2.14  1.87  1.59  1.37  1.17
1012 114  88  67  58  50  43  36 16077  4.47  3.44  2.63  2.30  1.97  1.69  1.43
'beginning of slab #4
S     828k6         31 24 I60847    88  74  Heights ..........................
 449  70  58  42  30  23  19  17  7131  2.76  2.28  1.63  1.17  0.89  0.74  0.67
 600  90  75  52  38  28  23  20  9534  3.54  2.94  2.06  1.48  1.10  0.92  0.79
 846 125 105  77  56  42  34  30 13439  4.91  4.13  3.02  2.19  1.65  1.35  1.20
1009 146 123  89  65  49  40  34 16033  5.74  4.85  3.50  2.56  1.93  1.59  1.35
'middle of slab #4
S     933k6         31 23 I60849    88  73  Heights ..........................
 434  67  58  44  35  27  22  15  6888  2.62  2.28  1.73  1.39  1.07  0.87  0.60
 598  87  74  56  46  36  29  23  9498  3.42  2.93  2.20  1.80  1.40  1.16  0.91
 843 116 100  76  63  50  41  34 13399  4.55  3.92  3.01  2.48  1.96  1.60  1.34
1024 137 118  93  77  61  50  42 16264  5.40  4.64  3.67  3.03  2.39  1.97  1.65
'end of slab #4
S     953k6         31 23 I60851    88  73  Heights ..........................
 445  57  49  38  33  27  24  22  7075  2.25  1.91  1.49  1.30  1.06  0.94  0.88
 600  70  58  45  37  31  25  22  9530  2.76  2.27  1.78  1.45  1.21  1.00  0.87
 836  95  81  63  54  43  37  32 13276  3.76  3.20  2.48  2.11  1.67  1.44  1.26
1014 116  97  79  67  56  49  43 16105  4.58  3.83  3.10  2.65  2.21  1.91  1.68
'beginning of slab #5
S    1066k6         31 24 I60853    88  74  Heights ..........................
 460  40  35  30  27  22  19  17  7302  1.59  1.36  1.19  1.07  0.85  0.73  0.67
 608  49  42  34  30  25  21  18  9661  1.94  1.65  1.34  1.17  0.97  0.81  0.71
 847  70  60  48  40  34  28  24 13451  2.74  2.35  1.89  1.58  1.33  1.11  0.94
1020  88  76  65  58  49  42  38 16208  3.48  2.99  2.57  2.30  1.92  1.65  1.48
'middle of slab #5
S    1170k6         31 23 I60854    88  73  Heights ..........................
 452  51  47  38  32  26  21  18  7186  2.02  1.86  1.49  1.26  1.03  0.83  0.71
 602  70  64  51  43  37  30  25  9562  2.76  2.50  2.02  1.71  1.44  1.17  1.00
 848 100  90  74  63  50  42  35 13471  3.93  3.56  2.90  2.46  1.98  1.64  1.39
1024 123 111  91  78  63  52  44 16272  4.84  4.37  3.59  3.06  2.47  2.05  1.72
'end of slab #5
S    1195k6         31 24 I60856    88  74  Heights ..........................
 444  44  39  31  27  23  21  18  7059  1.74  1.52  1.20  1.05  0.89  0.83  0.72
 603  57  48  41  37  32  27  26  9582  2.23  1.88  1.62  1.44  1.26  1.06  1.04
 846  84  72  58  52  45  42  37 13439  3.30  2.83  2.30  2.06  1.79  1.64  1.44
1019 108  93  80  71  63  57  52 16196  4.24  3.65  3.14  2.81  2.48  2.23  2.06
'beginning of slab #6
S    1305k6         31 24 I60857    88  74  Heights ..........................
 462  48  43  35  30  25  23  21  7341  1.89  1.68  1.39  1.19  0.97  0.90  0.81
 619  61  53  43  39  33  27  24  9840  2.42  2.08  1.68  1.52  1.30  1.07  0.94
 853  90  77  63  57  50  43  39 13558  3.54  3.02  2.46  2.25  1.96  1.70  1.54
1013 107  89  70  66  56  46  40 16093  4.20  3.51  2.76  2.61  2.19  1.81  1.57
'middle of slab #6
S    1409k6         31 24 I60858    88  74  Heights ..........................
 439  61  55  45  41  35  31  28  6980  2.40  2.17  1.77  1.60  1.37  1.23  1.11
 591  66  60  48  41  33  27  23  9387  2.61  2.36  1.87  1.59  1.29  1.07  0.90
 831 106  96  78  67  56  50  42 13209  4.19  3.76  3.06  2.63  2.19  1.98  1.67
1009 128 117  94  81  67  62  51 16033  5.02  4.62  3.69  3.20  2.65  2.44  2.02
'end of slab #6
S    1425k6         31 24 I60859    88  75  Heights ..........................
 448  61  52  43  38  33  28  25  7119  2.40  2.05  1.69  1.50  1.29  1.09  0.99
 596  76  66  50  46  40  32  27  9463  2.98  2.59  1.96  1.82  1.56  1.27  1.07
 826 111  96  80  72  63  54  47 13125  4.36  3.76  3.13  2.83  2.46  2.12  1.87
 997 142 124 104  95  83  73  65 15846  5.57  4.87  4.09  3.75  3.27  2.89  2.55
'beginning of slab #7
S    1541k6         31 24 I60901    88  75  Heights ..........................
 447  48  41  34  28  26  22  17  7103  1.89  1.63  1.32  1.09  1.01  0.88  0.67
 600  61  50  44  43  39  35  34  9526  2.41  1.97  1.72  1.70  1.52  1.37  1.33
 839  82  69  59  52  47  42  37 13332  3.24  2.71  2.30  2.06  1.85  1.64  1.46
1016 102  84  73  68  61  55  51 16148  4.00  3.30  2.89  2.69  2.40  2.15  1.99
'middle of slab #7
```

```
S    1646k6         31 24 I60902    88  75  Heights ..........................
 427  50  43  31  29  23  20  18  6789  1.98  1.68  1.21  1.12  0.92  0.79  0.72
 584  73  62  49  44  36  33  29  9272  2.85  2.44  1.94  1.72  1.43  1.30  1.14
 835 101  85  68  61  51  45  40 13260  3.96  3.36  2.68  2.39  2.02  1.78  1.59
1017 124 105  84  74  62  55  47 16152  4.89  4.14  3.31  2.91  2.45  2.15  1.87
'end of slab #7
S    1664k6         31 25 I60903    88  76  Heights ..........................
 451  59  48  40  34  28  24  23  7159  2.31  1.87  1.58  1.34  1.09  0.94  0.89
 622  84  68  56  49  42  37  34  9880  3.29  2.69  2.20  1.91  1.65  1.46  1.35
 815 117  98  80  71  59  52  48 12954  4.60  3.87  3.16  2.79  2.31  2.05  1.88
1007 136 117  93  82  67  59  54 16005  5.35  4.60  3.68  3.23  2.65  2.33  2.12
'beginning of slab #8
S    1777k6         31 24 I60904    88  74  Heights ..........................
 439  54  46  38  35  32  30  28  6980  2.11  1.81  1.51  1.39  1.26  1.18  1.09
 594  61  50  41  38  33  31  27  9431  2.39  1.98  1.61  1.49  1.31  1.20  1.07
 841  85  72  61  54  48  43  39 13368  3.36  2.81  2.42  2.11  1.90  1.67  1.52
1020 104  88  75  68  60  53  49 16204  4.08  3.47  2.96  2.67  2.35  2.10  1.93
'middle of slab #8
S    1885k6         31 24 I60905    88  75  Heights ..........................
 444  38  30  24  21  16  14   9  7059  1.48  1.20  0.94  0.81  0.64  0.53  0.37
 598  51  42  35  29  25  22  17  9502  1.99  1.67  1.36  1.15  1.00  0.85  0.67
 840  78  65  52  45  37  31  28 13340  3.08  2.55  2.06  1.78  1.47  1.23  1.09
1011  93  77  63  54  46  39  35 16065  3.65  3.05  2.46  2.11  1.80  1.54  1.38
'end of slab #8
S    1902k6         31 25 I60907    88  76  Heights ..........................
 435  51  40  29  20  13   8   5  6912  1.99  1.57  1.14  0.79  0.52  0.30  0.20
 585  63  52  36  28  19  13  10  9292  2.48  2.03  1.43  1.08  0.74  0.52  0.39
 833 101  84  63  51  39  32  27 13236  3.97  3.32  2.50  2.01  1.54  1.25  1.08
1022 126 106  83  66  52  43  37 16236  4.95  4.17  3.28  2.61  2.04  1.70  1.44
'beginning of slab #9
S    2015k6         31 24 I60908    88  75  Heights ..........................
 438  57  59  31  24  21  17  13  6956  2.25  2.33  1.24  0.94  0.84  0.68  0.52
 584  73  70  43  35  29  25  21  9280  2.89  2.77  1.69  1.39  1.15  0.99  0.84
 825 105  98  64  53  45  38  30 13101  4.13  3.86  2.52  2.09  1.78  1.50  1.17
1001 120 108  76  62  52  44  37 15910  4.72  4.24  2.99  2.43  2.04  1.71  1.44
'middle of slab #9
S    2123k6         31 25 I60909    88  76  Heights ..........................
 432  36  31  24  21  18  16  14  6865  1.43  1.21  0.93  0.82  0.70  0.64  0.56
 594  48  42  31  28  23  21  19  9431  1.87  1.65  1.22  1.11  0.92  0.82  0.73
 823  70  57  47  40  34  30  27 13074  2.74  2.24  1.85  1.59  1.33  1.17  1.04
 986  86  71  59  51  43  37  33 15672  3.37  2.78  2.31  1.99  1.70  1.46  1.30
'end of slab #9
S    2139k6         31 25 I60910    88  77  Heights ..........................
 445  68  47  36  29  22  17  20  7063  2.67  1.85  1.42  1.16  0.88  0.69  0.78
 600  95  74  51  43  36  31  27  9526  3.72  2.91  2.01  1.70  1.40  1.21  1.07
 812 126  96  71  60  48  40  36 12895  4.98  3.76  2.80  2.35  1.89  1.58  1.43
 982 156 120  89  76  61  52  52 15604  6.13  4.73  3.51  2.98  2.39  2.05  2.05
'beginning of slab #10
S    2353k6         31 25 I60912    88  77  Heights ..........................
 434  52  43  31  26  22  19  17  6900  2.04  1.67  1.24  1.03  0.85  0.74  0.67
 591  68  56  42  35  29  26  23  9387  2.68  2.19  1.63  1.37  1.13  1.02  0.92
 834  97  81  64  55  47  42  38 13244  3.83  3.18  2.50  2.15  1.83  1.64  1.51
1023 115  95  76  64  54  47  42 16252  4.54  3.74  2.98  2.54  2.12  1.86  1.66
'middle of slab #10
S    2558k6         31 25 I60913    88  76  Heights ..........................
 439  45  39  33  27  24  19  17  6976  1.79  1.52  1.28  1.08  0.93  0.75  0.65
 592  63  54  44  39  34  29  26  9399  2.46  2.14  1.74  1.55  1.33  1.14  1.01
 840  86  75  62  55  47  41  36 13352  3.37  2.94  2.45  2.17  1.85  1.61  1.40
1017 109  95  81  72  62  55  48 16160  4.31  3.76  3.18  2.82  2.44  2.16  1.90
'end of slab #10
S    2573k6         31 24 I60914    88  75  Heights ..........................
 435  58  39  32  28  23  20  18  6912  2.26  1.55  1.24  1.08  0.91  0.79  0.72
 589  72  51  38  34  28  24  21  9355  2.82  2.00  1.49  1.34  1.10  0.95  0.83
 828  98  70  53  46  38  32  28 13149  3.86  2.74  2.10  1.80  1.50  1.27  1.10
 997 117  85  65  55  46  38  31 15842  4.61  3.33  2.56  2.18  1.80  1.48  1.21
'beginning of slab #11
S    2789k6         31 24 I60916    88  75  Heights ..........................
 432  54  47  38  31  24  22  19  6861  2.11  1.85  1.48  1.21  0.96  0.85  0.76
 585  69  60  49  41  32  28  25  9288  2.72  2.36  1.92  1.59  1.26  1.10  1.00
 832  97  85  68  56  45  40  34 13221  3.81  3.33  2.66  2.20  1.75  1.57  1.35
1023 120 106  86  74  61  54  48 16252  4.70  4.17  3.39  2.93  2.39  2.13  1.89
'middle of slab #11
S    2994k6         31 25 I60917    88  76  Heights ..........................
 435  51  40  28  21  14   8   5  6904  2.00  1.58  1.11  0.81  0.55  0.32  0.20
 587  66  58  43  36  29  25  22  9324  2.58  2.26  1.70  1.42  1.15  0.97  0.86
 838  90  77  58  49  39  33  29 13316  3.55  3.04  2.29  1.91  1.54  1.29  1.12
1024 111  96  75  64  53  46  41 16268  4.38  3.78  2.96  2.51  2.08  1.79  1.60
'end of slab #11
S    3010k6         31 25 I60918    88  77  Heights ..........................
```

```
 433  33  28  21  19  18  16  13  6872  1.30  1.10  0.83  0.76  0.69  0.63  0.53
 588  44  37  29  29  25  22  20  9347  1.72  1.45  1.16  1.12  0.96  0.87  0.79
 826  65  55  47  42  36  32  27 13129  2.56  2.16  1.83  1.66  1.42  1.25  1.06
 993  80  69  57  52  45  38  35 15783  3.13  2.70  2.25  2.04  1.77  1.51  1.37
'beginning of slab #12
S    3223k6         31 25 I60919    88   77  Heights ..........................
 438  45  35  30  26  24  18  18  6956  1.77  1.39  1.19  1.04  0.93  0.71  0.69
 590  61  48  40  34  30  25  22  9367  2.38  1.89  1.57  1.35  1.17  0.98  0.87
 828  84  68  56  49  42  36  33 13149  3.30  2.67  2.19  1.91  1.65  1.43  1.30
1006 100  82  67  59  51  45  40 15982  3.93  3.21  2.64  2.31  1.99  1.78  1.57
'middle of slab #12
S    3428k6         31 26 I60921    88   78  Heights ..........................
 430  40  32  23  17  11   7   5  6837  1.59  1.26  0.91  0.66  0.45  0.29  0.20
 585  70  60  49  43  34  29  25  9292  2.76  2.37  1.94  1.69  1.35  1.13  1.00
 828  99  86  69  58  47  39  33 13157  3.91  3.39  2.70  2.27  1.85  1.54  1.28
1011 119 104  85  72  60  51  44 16069  4.68  4.09  3.36  2.85  2.36  2.02  1.73
'end of slab #12
S    3443k6         31 25 I60922    88   77  Heights ..........................
 423 101  65  41  33  28  25  23  6726  3.97  2.54  1.63  1.29  1.11  0.99  0.89
 586 125  80  52  41  37  33  26  9304  4.93  3.17  2.04  1.62  1.46  1.28  1.04
 817 168 110  74  61  53  47  41 12978  6.63  4.31  2.89  2.40  2.08  1.83  1.62
 982 196 129  88  73  63  55  48 15596  7.73  5.06  3.47  2.89  2.49  2.18  1.91
'beginning of slab #13
S    3657k6         31 26 I60926    88   78  Heights ..........................
 443  34  30  25  23  20  18  16  7043  1.33  1.17  0.98  0.90  0.79  0.69  0.63
 592  47  38  33  30  26  24  22  9399  1.86  1.51  1.28  1.19  1.04  0.94  0.85
 834  68  57  49  45  40  36  32 13256  2.69  2.23  1.94  1.76  1.57  1.41  1.26
1008  81  67  58  53  47  42  37 16017  3.20  2.63  2.27  2.08  1.84  1.64  1.45
'middle of slab #13
S    3863k6         31 25 I60928    88   77  Heights ..........................
 429  36  30  24  22  20  17  15  6813  1.41  1.16  0.96  0.88  0.78  0.65  0.58
 589  47  38  33  30  25  21  19  9355  1.87  1.51  1.30  1.16  1.00  0.84  0.76
 832  70  57  48  44  37  32  28 13225  2.76  2.24  1.90  1.72  1.47  1.24  1.09
1015  88  73  62  56  48  41  36 16121  3.45  2.86  2.43  2.21  1.89  1.63  1.41
'end of slab #13
S    3879k6         31 25 I60929    88   76  Heights ..........................
 431  40  34  27  23  19  17  15  6853  1.59  1.34  1.05  0.89  0.75  0.65  0.61
 587  53  42  35  31  26  22  19  9324  2.08  1.65  1.38  1.20  1.04  0.88  0.75
 832  77  63  52  46  39  33  29 13217  3.04  2.48  2.04  1.79  1.52  1.31  1.12
1017  95  77  64  57  48  41  35 16164  3.72  3.04  2.52  2.23  1.89  1.62  1.38
'beginning of slab #14
S    4090k6         31 25 I60931    88   76  Heights ..........................
 426  37  30  25  22  19  16  14  6773  1.44  1.16  0.96  0.86  0.76  0.64  0.57
 579  46  38  32  29  26  23  20  9192  1.82  1.50  1.26  1.14  1.00  0.89  0.79
 835  68  57  48  45  40  36  32 13272  2.68  2.24  1.91  1.77  1.57  1.40  1.26
1036  82  68  58  53  47  42  38 16454  3.22  2.67  2.27  2.09  1.84  1.66  1.48
'middle of slab #14
S    4298k6         31 25 I60933    88   77  Heights ..........................
 422  42  33  27  25  21  19  17  6702  1.64  1.31  1.05  0.96  0.84  0.76  0.69
 574  55  44  37  33  28  25  22  9125  2.15  1.72  1.45  1.30  1.12  0.99  0.87
 831  81  68  53  49  41  36  31 13209  3.19  2.67  2.08  1.92  1.62  1.43  1.24
1030  97  81  64  59  50  45  39 16363  3.83  3.20  2.54  2.33  1.97  1.76  1.54
'end of slab #14
S    4315k6         31 26 I60934    88   78  Heights ..........................
 452  42  33  29  23  21  18  16  7186  1.65  1.28  1.15  0.91  0.81  0.71  0.63
 588  52  45  41  33  30  26  23  9343  2.06  1.77  1.61  1.30  1.18  1.02  0.90
 843  74  63  54  47  39  36  31 13387  2.90  2.48  2.13  1.85  1.54  1.40  1.22
1021  89  77  65  58  50  43  38 16220  3.50  3.02  2.56  2.27  1.95  1.70  1.51
'beginning of slab #15
S    4528k6         31 26 I60936    88   78  Heights ..........................
 428  47  37  28  24  21  19  17  6793  1.85  1.46  1.11  0.95  0.81  0.73  0.66
 584  61  48  38  32  27  24  22  9272  2.41  1.89  1.48  1.26  1.07  0.95  0.85
 835  84  67  53  46  39  34  30 13260  3.31  2.63  2.08  1.81  1.54  1.35  1.19
1027 101  82  66  58  50  45  39 16315  3.98  3.21  2.59  2.29  1.96  1.77  1.54
'middle of slab #15
S    4734k6         31 25 I60939    88   77  Heights ..........................
 437  37  32  22  19  17  14  12  6936  1.45  1.27  0.87  0.76  0.68  0.55  0.46
 587  48  42  33  30  27  24  21  9328  1.90  1.66  1.29  1.17  1.04  0.94  0.84
 838  70  62  50  45  39  34  32 13308  2.77  2.43  1.95  1.77  1.52  1.35  1.24
1019  85  74  63  56  48  42  38 16184  3.36  2.92  2.46  2.21  1.89  1.65  1.48
'end of slab #15
S    4752k6         31 25 I60940    88   77  Heights ..........................
 430  53  31  23  20  16  16  14  6825  2.08  1.23  0.89  0.79  0.65  0.63  0.55
 586  70  43  32  28  26  24  22  9316  2.77  1.68  1.25  1.11  1.02  0.94  0.88
 827  97  61  45  40  34  33  30 13145  3.81  2.39  1.78  1.57  1.35  1.30  1.20
1014 114  70  54  47  42  38  36 16105  4.48  2.76  2.11  1.87  1.65  1.51  1.42
'beginning of slab #16
S    4966k6         31 25 I60942    88   77  Heights ..........................
 427  45  37  30  26  22  19  16  6785  1.76  1.47  1.19  1.04  0.87  0.74  0.64
```

```
 583  57  48  40  37  30  27  24  9260  2.26  1.87  1.57  1.47  1.19  1.06  0.95
 836  82  69  55  48  40  34  30 13284  3.24  2.71  2.17  1.87  1.57  1.34  1.17
1023 105  88  72  65  54  47  41 16252  4.11  3.46  2.85  2.54  2.14  1.86  1.62
'middle of slab #16
S    5170k6         31 25 I60944    88  77  Heights ...........................
 450  39  39  21  22  19  18  18  7151  1.52  1.55  0.82  0.87  0.74  0.69  0.72
 600  61  58  54  52  49  50  49  9526  2.39  2.30  2.13  2.04  1.93  1.96  1.91
 848  76  69  61  56  50  45  41 13467  2.98  2.72  2.39  2.19  1.97  1.78  1.62
1029  89  81  68  61  53  47  41 16355  3.48  3.17  2.66  2.41  2.10  1.83  1.60
'end of slab #16 last test taken
EOF
•
```

APPENDIX C

DETAILED T-TEST ANALYSIS OF THE PCP JOINTS

Steps followed for the t-test analysis

For equal sample sizes:

1. Specify the null and the alternative hypotheses.

   Null Hypothesis is: $H_0 : \mu_1 - \mu_2 = 0$

   Alternative Hypothesis is: $H_1 : \mu_1 - \mu_2 \neq 0$

2. Choose a significance level (α).

   A value of $\alpha = 0.05$ was chosen in all cases.

3. Compute the difference between sample means (M_d).

   $$M_d = \mu_1 - \mu_2$$

4. Compute p, the probability of obtaining a difference between the two sample means and the value specified by the null hypothesis as larger or larger than the difference obtained in the experiment:

   $$t = \frac{M_d - (\mu_1 - \mu_2)}{SM_d}$$

   When the two sample sizes are equal the mean square error equals:

   $$MSE = \frac{\sigma_1^2 + \sigma_2^2}{2}$$

   $$SM_d = \sqrt{\frac{2MSE}{n}}$$

   The probability value for t can be determined using a t table.

5. The probability is compared to the significance level. If the probability value is less than the significance level, the effect is significant.
6. If the effect is significant, the null hypothesis is rejected.
7. Create a report of this experimental result.

   For unequal sample sizes:

Steps 1-3 and Steps 5- 7 are the same. The only difference is in Step 4 as follows:

When $n_1 \neq n_2$

$$\text{MSE} = \frac{\text{SSE}}{\text{df}}$$

where

$$\text{df} = (n_1 - 1 + n_2 - 1)$$

$$\text{SSE} = \text{SSE}_1 + \text{SSE}_2$$

$$\text{SSE}_1 = \sum_{i-1}^{n} (X_i - \mu_1)^2$$

$$\text{SSE}_2 = \sum_{i-1}^{n} (X_i - \mu_2)^2$$

$$\text{SM}_d = \sqrt{\frac{2\text{MSE}}{n_k}}$$

$$n_k = \frac{2}{\frac{1}{n_1} + \frac{1}{n_2}}$$

Using a spreadsheet, Cases 1 through 4 were analyzed using the Student's t-Test and the results are as follows:

Case 1

Slab length: 240ft
Sample 1: North end (start)
Sample 2: South end (end)

	Deflections at start of slab			Deflections at end of slab		
	Xi	Xi-Xmean	(Xi-Xmean)^2	Xi	Xi-Xmean	(Xi-Xmean)^2
	2.76	-0.55	0.31	3.21	-0.22	0.05
	3.24	-0.08	0.01	4.17	0.73	0.54
	3.26	-0.06	0.00	3.18	-0.26	0.07
	3.34	0.02	0.00	4.00	0.56	0.32
	2.88	-0.44	0.19	3.39	-0.04	0.00
	3.83	0.51	0.26	3.56	0.12	0.01
	3.89	0.57	0.32	3.42	-0.01	0.00
	3.35	0.03	0.00	2.55	-0.88	0.78
$\Sigma_{1,2}$	**26.54**	**0.00**	**1.09**	**27.46**	**0.00**	**1.77**
$N_{1,2}$	**8.00**			**8.00**		
$df_{1,2}$	**7.00**			**7.00**		
$\mu_{1,2}$	**3.32**			**3.43**		
$\sigma_{1,2}^2$	**0.16**			**0.25**		
Md	**0.11**					
MSE	**0.20**					
SM_d	**0.23**					
t	**-0.51**					

For α = 0.05, df=14, and a two-tailed distribution:
t table = 2.14 2.14>-0.51
Then, H_0 cannot be rejected and $\mu 1=\mu 2$
There is not significant difference between means

Case 2

Slab length: 240ft

Sample 1: North end (start)

Sample 2: Mid span

	Deflections at start of slab			Deflections at mid span of slab		
	Xi	Xi-Xmean	$(\text{Xi-Xmean})^2$	Xi	Xi-Xmean	$(\text{Xi-Xmean})^2$
	2.76	-0.55	0.31	2.54	-0.44	0.20
	3.24	-0.08	0.01	2.27	-0.71	0.51
	3.26	-0.06	0.00	4.24	1.25	1.57
	3.34	0.02	0.00	2.44	-0.55	0.30
	2.88	-0.44	0.19	3.01	0.03	0.00
	3.83	0.51	0.26	2.89	-0.10	0.01
	3.89	0.57	0.32	2.99	0.00	0.00
	3.35	0.03	0.00	3.50	0.51	0.26
$\Sigma_{1,2}$	**26.54**	**0.00**	**1.09**	**23.87**	**0.00**	**2.85**
$N_{1,2}$	**8.00**			**8.00**		
df $_{1,2}$	**7.00**			**7.00**		
$\mu_{1,2}$	**3.32**			**2.98**		
$\sigma_{1,2}^2$	**0.16**			**0.41**		
Md	**-0.33**					
MSE	**0.28**					
SM_d	**0.27**					
t	**1.26**					

For α = 0.05, df=14, and a two-tailed distribution:

t table=2.14 2.14>1.26

Then, H_0 cannot be rejected and $\mu1=\mu2$

There is not significant difference between means

Case 3

Slab length: 440ft

Sample 1: North end (start)

Sample 2: South end (end)

Deflections at start of slab		
Xi	Xi-Xmean	(Xi-Xmean)^2
3.39	0.59	0.35
2.18	-0.62	0.38
2.61	-0.19	0.04
2.53	-0.27	0.07
3.29	0.49	0.24

Deflections at end of slab		
Xi	Xi-Xmean	(Xi-Xmean)^2
2.98	0.18	0.03
3.13	0.33	0.11
3.24	0.44	0.19
2.37	-0.43	0.18
2.70	-0.10	0.01
2.37	-0.43	0.18

$\Sigma_{1,2}$	**13.99**	**16.79**
$N_{1,2}$	**5.00**	**6.00**
$df_{1,2}$	**4.00**	**5.00**
$\mu_{1,2}$	**2.797**	**2.798**
$SSE_{1,2}$	**1.08**	**0.71**
SSE	**1.79**	
df	**9.00**	
MSE	**0.20**	
n_h	**5.45**	
SM_d	**0.27**	
t	**-0.004**	

For $\alpha = 0.05$, df=9, and a two-tailed distribution:

t table=2.262 2.262>-0.004

Then, H_0 cannot be rejected and $\mu 1=\mu 2$

There is not significant difference between means

Case 4

Slab length: 440ft

Sample 1: North end (start)

Sample 2: Mid span

Deflections at start of slab		
Xi	Xi-Xmean	(Xi-Xmean)^2
3.39	0.59	0.35
2.18	-0.62	0.38
2.61	-0.19	0.04
2.53	-0.27	0.07
3.29	0.49	0.24

Deflections at mid span of slab		
Xi	Xi-Xmean	(Xi-Xmean)^2
3.27	0.45	0.20
3.34	0.51	0.26
2.85	0.02	0.00
2.27	-0.55	0.30
2.29	-0.53	0.28
2.89	0.07	0.00
2.84	0.02	0.00

$\Sigma_{1,2}$	**13.99**	**19.74**
$N_{1,2}$	**5.00**	**7.00**
df $_{1,2}$	**4.00**	**6.00**
$\mu_{1,2}$	**2.80**	**2.82**
$SSE_{1,2}$	**1.08**	**1.06**
SSE	**2.14**	
df	**10.00**	
MSE	**0.21**	
n_h	**5.83**	
SM_d	**0.27**	
t	**-0.09**	

For α = 0.05, df=10, and a two-tailed distribution:

t table=2.228 2.228>-0.09

Then, H_0 cannot be rejected and $\mu 1=\mu 2$

There is not significant difference between means

APPENDIX D

EVERCALC INPUT VALUES FOR BACK-CALCULATION AND OUTPUT RESULTS

EVERCALC input parameters for back-calculation and calculated modulus

Outside Lane

Run No.1

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	5461
2	ACP layer	2	0.35	750	300	1000	316
3	JCP layer	12	0.15	4500	500	10000	1266
4	Granular base	5	0.35	100	10	500	233
5	Natural soil	N/A	0.40	30	10	250	41

Run No. 2

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	6454
2	ACP layer	2	0.35	250	100	500	137
3	JCP layer	12	0.15	4500	500	10000	2821
4	Granular base	5	0.35	100	10	500	118
5	Natural soil	N/A	0.40	30	10	250	41

Run No. 3

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	5210
2	ACP layer	2	0.35	750	400	1200	425
3	JCP layer	12	0.15	4500	500	10000	1093
4	Granular base	5	0.35	100	10	500	270
5	Natural soil	N/A	0.40	30	10	250	42

Run No. 4

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	6276
2	ACP layer	2	0.35	750	100	1200	263
3	JCP layer	12	0.15	4500	500	10000	2668
4	Granular base	5	0.35	100	10	500	163
5	Natural soil	N/A	0.40	30	10	250	41

Run No. 5

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	5600
2	ACP layer	2	0.35	400	300	700	306
3	JCP layer	12	0.15	4500	500	10000	1177
4	Granular base	5	0.35	100	10	500	268
5	Natural soil	N/A	0.40	30	10	250	41

EVERCALC input parameters for back-calculation and calculated modulus

Inside Lane

Run No.1

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	4550
2	ACP layer	2	0.35	750	300	1000	420
3	JCP layer	12	0.15	4500	500	10000	906
4	Granular base	5	0.35	100	10	500	174
5	Natural soil	N/A	0.40	30	10	250	45

Run No. 2

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	5395
2	ACP layer	2	0.35	250	100	500	207
3	JCP layer	12	0.15	4500	500	10000	1282
4	Granular base	5	0.35	100	10	500	155
5	Natural soil	N/A	0.40	30	10	250	44

Run No. 3

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	4416
2	ACP layer	2	0.35	750	400	1200	528
3	JCP layer	12	0.15	4500	500	10000	791
4	Granular base	5	0.35	100	10	500	210
5	Natural soil	N/A	0.40	30	10	250	45

Run No. 4

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	5199
2	ACP layer	2	0.35	750	100	1200	302
3	JCP layer	12	0.15	4500	500	10000	1203
4	Granular base	5	0.35	100	10	500	180
5	Natural soil	N/A	0.40	30	10	250	44

Run No. 5

Layer No.	Description	Thickness (in)	Poisson Ratio	Initial Modulus (ksi)	Minimum Modulus (ksi)	Maximum Modulus (ksi)	Calculated Modulus (ksi)
1	PCP layer	6	0.15	4500	500	10000	4058
2	ACP layer	2	0.35	400	300	700	432
3	JCP layer	12	0.15	4500	500	10000	980
4	Granular base	5	0.35	100	10	500	150
5	Natural soil	N/A	0.40	30	10	250	45

APPENDIX E

CALCULATION OF REQUIRED PRESTRESS FOR PCP SLABS (LONGITUDINAL AND TRANSVERSE DIRECTIONS)

Calculation of Required Prestress for Longitudinal Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 3.00E-06				α = 3.00E-06				α = 3.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 250				L = 300				L = 350			
Temperature Differential (°F/in)	ΔT = 3				ΔT = 3				ΔT = 3			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	191	214	238	σ_c =	191	214	238	σ_c =	191	214	238
Friction Stress (psi)	σ_F =	115	115	115	σ_F =	138	138	138	σ_F =	161	161	161

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design | **From Fatigue and Elastic Analyses** | **Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)**

Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
250	8	83.1	191	115	350	39	27	142	215	**215**	5482	1361	2821	9664	**36992**	**22**
250	9	70.7	214	115	350	50	17	132	215	**215**	5482	1361	2821	9664	**36992**	**19**
250	10	66.7	238	115	350	70	14	129	215	**215**	5482	1361	2821	9664	**36992**	**17**
300	8	83.1	191	138	350	62	27	165	238	**238**	6499	1361	2821	10681	**35975**	**19**
300	9	70.7	214	138	350	73	17	155	238	**238**	6499	1361	2821	10681	**35975**	**17**
300	10	66.7	238	138	350	93	14	152	238	**238**	6499	1361	2821	10681	**35975**	**15**
350	8	83.1	191	161	350	85	27	188	261	**261**	7490	1361	2821	11672	**34984**	**17**
350	9	70.7	214	161	350	96	17	178	261	**261**	7490	1361	2821	11672	**34984**	**15**
350	10	66.7	238	161	350	116	14	175	261	**261**	7490	1361	2821	11672	**34984**	**13**

Calculation of Movement at Joint Ends

			Temperature						
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.369	0.178	0.252	0.450	0.225	0.143	**1.617**
300	0.055	138	0.443	0.204	0.294	0.540	0.270	0.190	**1.941**
350	0.075	161	0.517	0.227	0.332	0.630	0.315	0.244	**2.264**

Calculation of Required Prestress for Longitudinal Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Siliceous River Gravel

Concrete and Pavement Properties:												
28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 6.00E-06				α = 6.00E-06				α = 6.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 250				L = 300				L = 350			
Temperature Differential (°F/in)	ΔT = 3				ΔT = 3				ΔT = 3			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	381	429	476	σ_c =	381	429	476	σ_c =	381	429	476
Friction Stress (psi)	σ_F =	115	115	115	σ_F =	138	138	138	σ_F =	161	161	161

Prestressing Tendon Properties:			
Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR}+\sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
250	8	83.1	381	115	350	229	27	142	215	**229**	5482	1361	2821	9664	**36992**	**20**
250	9	70.7	429	115	350	265	17	132	215	**265**	5482	1361	2821	9664	**36992**	**16**
250	10	66.7	476	115	350	308	14	129	215	**308**	5482	1361	2821	9664	**36992**	**12**
300	8	83.1	381	138	350	252	27	165	238	**252**	6499	1361	2821	10681	**35975**	**18**
300	9	70.7	429	138	350	288	17	155	238	**288**	6499	1361	2821	10681	**35975**	**14**
300	10	66.7	476	138	350	331	14	152	238	**331**	6499	1361	2821	10681	**35975**	**11**
350	8	83.1	381	161	350	275	27	188	261	**275**	7490	1361	2821	11672	**34984**	**16**
350	9	70.7	429	161	350	311	17	178	261	**311**	7490	1361	2821	11672	**34984**	**13**
350	10	66.7	476	161	350	354	14	175	261	**354**	7490	1361	2821	11672	**34984**	**10**

Calculation of Movement at Joint Ends

			Temperature						
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.738	0.394	0.543	0.450	0.225	0.153	**2.503**
300	0.055	138	0.886	0.463	0.642	0.540	0.270	0.202	**3.003**
350	0.075	161	1.033	0.530	0.739	0.630	0.315	0.257	**3.504**

Calculation of Required Prestress for Longitudinal Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f =	700			f =	700			f =	700		
Safety Factor (ACI=2)	SF =	2			SF =	2			SF =	2		
Allowable Flexural Stress	f_{design} =	350			f_{design} =	350			f_{design} =	350		
Concrete Modulus of Elasticity (psi)	E =	4500000			E =	4500000			E =	4500000		
Concrete Poisson's Ration	μ =	0.15			μ =	0.15			μ =	0.15		
Coefficient of Thermal Expansion (/°F)	α =	3.00E-06			α =	3.00E-06			α =	3.00E-06		
Concrete Unit Weight (pcf)	γ=	144			γ=	144			γ=	144		
Slab Length (ft)	L =	250			L =	300			L =	350		
Temperature Differential (°F/in)	ΔT =	1			ΔT =	1			ΔT =	1		
Coefficient of Friction (slab-support)	μ_{max}=	0.92			μ_{max}=	0.92			μ_{max}=	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	64	71	79	σ_c =	64	71	79	σ_c =	64	71	79
Friction Stress (psi)	σ_F =	115	115	115	σ_F =	138	138	138	σ_F =	161	161	161

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	- Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR}+\sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
250	8	83.1	64	115	350	-88	27	142	215	**215**	5482	1361	2821	9664	**36992**	**22**
250	9	70.7	71	115	350	-93	17	132	215	**215**	5482	1361	2821	9664	**36992**	**19**
250	10	66.7	79	115	350	-89	14	129	215	**215**	5482	1361	2821	9664	**36992**	**17**
300	8	83.1	64	138	350	-65	27	165	238	**238**	6499	1361	2821	10681	**35975**	**19**
300	9	70.7	71	138	350	-70	17	155	238	**238**	6499	1361	2821	10681	**35975**	**17**
300	10	66.7	79	138	350	-66	14	152	238	**238**	6499	1361	2821	10681	**35975**	**15**
350	8	83.1	64	161	350	-42	27	188	261	**261**	7490	1361	2821	11672	**34984**	**17**
350	9	70.7	71	161	350	-47	17	178	261	**261**	7490	1361	2821	11672	**34984**	**15**
350	10	66.7	79	161	350	-43	14	175	261	**261**	7490	1361	2821	11672	**34984**	**13**

Calculation of Movement at Joint Ends

			Temperature						
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.369	0.178	0.252	0.450	0.225	0.143	**1.617**
300	0.055	138	0.443	0.204	0.294	0.540	0.270	0.190	**1.941**
350	0.075	161	0.517	0.227	0.332	0.630	0.315	0.244	**2.264**

Calculation of Required Prestress for Longitudinal Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Siliceous River Gravel

Concrete and Pavement Properties:

28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 6.00E-06				α = 6.00E-06				α = 6.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 250				L = 300				L = 350			
Temperature Differential (°F/in)	ΔT = 1				ΔT = 1				ΔT = 1			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	127	143	159	σ_c =	127	143	159	σ_c =	127	143	159
Friction Stress (psi)	σ_F =	115	115	115	σ_F =	138	138	138	σ_F =	161	161	161

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	-	Min
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
250	8	83.1	127	115	350	-25	27	142	215	**215**	5482	1361	2821	9664	**36992**	**22**
250	9	70.7	143	115	350	-21	17	132	215	**215**	5482	1361	2821	9664	**36992**	**19**
250	10	66.7	159	115	350	-9	14	129	215	**215**	5482	1361	2821	9664	**36992**	**17**
300	8	83.1	127	138	350	-2	27	165	238	**238**	6499	1361	2821	10681	**35975**	**19**
300	9	70.7	143	138	350	2	17	155	238	**238**	6499	1361	2821	10681	**35975**	**17**
300	10	66.7	159	138	350	14	14	152	238	**238**	6499	1361	2821	10681	**35975**	**15**
350	8	83.1	127	161	350	21	27	188	261	**261**	7490	1361	2821	11672	**34984**	**17**
350	9	70.7	143	161	350	25	17	178	261	**261**	7490	1361	2821	11672	**34984**	**15**
350	10	66.7	159	161	350	37	14	175	261	**261**	7490	1361	2821	11672	**34984**	**13**

Calculation of Movement at Joint Ends

			Temperature					Elastic	
			Seasonal	Summer	Winter	Shrinkage	Creep	Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.738	0.394	0.543	0.450	0.225	0.143	**2.493**
300	0.055	138	0.886	0.463	0.642	0.540	0.270	0.190	**2.992**
350	0.075	161	1.033	0.530	0.739	0.630	0.315	0.244	**3.490**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f =	700			f =	700			f =	700		
Safety Factor (ACI=2)	SF =	2			SF =	2			SF =	2		
Allowable Flexural Stress	f_{design} =	350			f_{design} =	350			f_{design} =	350		
Concrete Modulus of Elasticity (psi)	E =	4500000			E =	4500000			E =	4500000		
Concrete Poisson's Ration	μ =	0.15			μ =	0.15			μ =	0.15		
Coefficient of Thermal Expansion (/°F)	α =	3.00E-06			α =	3.00E-06			α =	3.00E-06		
Concrete Unit Weight (pcf)	γ=	144			γ=	144			γ=	144		
Slab Length (ft)	L =	25			L =	49			L =	71		
Temperature Differential (°F/in)	ΔT =	3			ΔT =	3			ΔT =	3		
Coefficient of Friction (slab-support)	μ_{max}=	0.92			μ_{max}=	0.92			μ_{max}=	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	191	214	238	σ_c =	191	214	238	σ_c =	191	214	238
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	$\Delta\ FS_1$	$\Delta\ FS_2$	$\Delta\ FS_3$	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
25	8	83.1	191	12	350	0	27	39	0	**39**	580	1361	2821	4761	**41895**	**136**
25	9	70.7	214	12	350	0	17	29	0	**29**	580	1361	2821	4761	**41895**	**163**
25	10	66.7	238	12	350	0	14	26	0	**26**	580	1361	2821	4761	**41895**	**164**
49	8	83.1	191	23	350	0	27	50	0	**50**	1129	1361	2821	5311	**41345**	**104**
49	9	70.7	214	23	350	0	17	40	0	**40**	1129	1361	2821	5311	**41345**	**116**
49	10	66.7	238	23	350	0	14	37	0	**37**	1129	1361	2821	5311	**41345**	**113**
71	8	83.1	191	33	350	0	27	60	0	**60**	1627	1361	2821	5809	**40847**	**86**
71	9	70.7	214	33	350	0	17	50	0	**50**	1627	1361	2821	5809	**40847**	**91**
71	10	66.7	238	33	350	0	14	47	0	**47**	1627	1361	2821	5809	**40847**	**88**

Calculation of Movement at Joint Ends

			Temperature					Elastic	
			Seasonal	Summer	Winter	Shrinkage	Creep	Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
25	0.000	12	0.037	0.021	0.029	0.045	0.0225	0.003	**0.157**
49	0.001	23	0.072	0.041	0.056	0.088	0.044	0.006	**0.307**
71	0.003	33	0.105	0.058	0.079	0.128	0.0639	0.011	**0.446**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Siliceous River Gravel

Concrete and Pavement Properties:

28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 6.00E-06				α = 6.00E-06				α = 6.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 25				L = 49				L = 71			
Temperature Differential (°F/in)	ΔT = 3				ΔT = 3				ΔT = 3			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	381	429	476	σ_c =	381	429	476	σ_c =	381	429	476
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	-	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86		45	41
Temperature Change During Summer Day (ΔT Summer) =	110		86	24
Temperature Change During Winter Day (ΔT Winter) =	48		10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design — **From Fatigue and Elastic Analyses** — **Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)**

Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	Governing Value (psi)	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	Final prestress force (lbs)	Strand Spacing (in)
25	8	83.1	381	12	350	0	27	39	0	**39**	580	1361	2821	4761	**41895**	**136**
25	9	70.7	429	12	350	0	17	29	0	**29**	580	1361	2821	4761	**41895**	**163**
25	10	66.7	476	12	350	0	14	26	0	**26**	580	1361	2821	4761	**41895**	**164**
49	8	83.1	381	23	350	0	27	50	0	**50**	1129	1361	2821	5311	**41345**	**104**
49	9	70.7	429	23	350	0	17	40	0	**40**	1129	1361	2821	5311	**41345**	**116**
49	10	66.7	476	23	350	0	14	37	0	**37**	1129	1361	2821	5311	**41345**	**113**
71	8	83.1	381	33	350	0	27	60	0	**60**	1627	1361	2821	5809	**40847**	**86**
71	9	70.7	429	33	350	0	17	50	0	**50**	1627	1361	2821	5809	**40847**	**91**
71	10	66.7	476	33	350	0	14	47	0	**47**	1627	1361	2821	5809	**40847**	**88**

Calculation of Movement at Joint Ends

			Temperature					Elastic	
			Seasonal	Summer	Winter	Shrinkage	Creep	Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
25	0.000	12	0.074	0.043	0.058	0.045	0.0225	0.003	**0.244**
49	0.001	23	0.145	0.083	0.112	0.088	0.044	0.006	**0.479**
71	0.003	33	0.210	0.120	0.162	0.128	0.0639	0.011	**0.694**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 3.00E-06				α = 3.00E-06				α = 3.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 25				L = 49				L = 71			
Temperature Differential (°F/in)	ΔT = 1				ΔT = 1				ΔT = 1			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	64	71	79	σ_c =	64	71	79	σ_c =	64	71	79
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	-	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86		45	41
Temperature Change During Summer Day (ΔT Summer) =	110		86	24
Temperature Change During Winter Day (ΔT Winter) =	48		10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	Δ FS_1	Δ FS_2	Δ FS_3	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
25	8	83.1	64	12	350	0	27	39	0	**39**	580	1361	2821	4761	**41895**	**136**
25	9	70.7	71	12	350	0	17	29	0	**29**	580	1361	2821	4761	**41895**	**163**
25	10	66.7	79	12	350	0	14	26	0	**26**	580	1361	2821	4761	**41895**	**164**
49	8	83.1	64	23	350	0	27	50	0	**50**	1129	1361	2821	5311	**41345**	**104**
49	9	70.7	71	23	350	0	17	40	0	**40**	1129	1361	2821	5311	**41345**	**116**
49	10	66.7	79	23	350	0	14	37	0	**37**	1129	1361	2821	5311	**41345**	**113**
71	8	83.1	64	33	350	0	27	60	0	**60**	1627	1361	2821	5809	**40847**	**86**
71	9	70.7	71	33	350	0	17	50	0	**50**	1627	1361	2821	5809	**40847**	**91**
71	10	66.7	79	33	350	0	14	47	0	**47**	1627	1361	2821	5809	**40847**	**88**

Calculation of Movement at Joint Ends

			Temperature						
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
25	0.000	12	0.037	0.021	0.029	0.045	0.0225	0.003	**0.157**
49	0.001	23	0.072	0.041	0.056	0.088	0.044	0.006	**0.307**
71	0.003	33	0.105	0.058	0.079	0.128	0.0639	0.011	**0.446**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP overlay for existing pavement in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Siliceous River Gravel

Concrete and Pavement Properties:

28-day Flexural Strength	f =	700			f =	700			f =	700		
Safety Factor (ACI=2)	SF =	2			SF =	2			SF =	2		
Allowable Flexural Stress	f_{design} =	350			f_{design} =	350			f_{design} =	350		
Concrete Modulus of Elasticity (psi)	E =	4500000			E =	4500000			E =	4500000		
Concrete Poisson's Ration	μ =	0.15			μ =	0.15			μ =	0.15		
Coefficient of Thermal Expansion (/°F)	α =	6.00E-06			α =	6.00E-06			α =	6.00E-06		
Concrete Unit Weight (pcf)	γ=	144			γ=	144			γ=	144		
Slab Length (ft)	L =	25			L =	49			L =	71		
Temperature Differential (°F/in)	ΔT =	1			ΔT =	1			ΔT =	1		
Coefficient of Friction (slab-support)	μ_{max}=	0.92			μ_{max}=	0.92			μ_{max}=	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3	σ_t =	63.9	54.4	51.3
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14	σ_{PR} =	27	17	14
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7	$\sigma_{t\ design}$ =	83.1	70.7	66.7
Curling Stress at Slab Center (psi)	σ_c =	127	143	159	σ_c =	127	143	159	σ_c =	127	143	159
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	Governing Value (psi)	ΔFS_1	ΔFS_2	ΔFS_3	Total losses (lbs)	Final prestress force (lbs)	Strand Spacing (in)
25	8	83.1	127	12	350	0	27	39	0	**39**	580	1361	2821	4761	**41895**	**136**
25	9	70.7	143	12	350	0	17	29	0	**29**	580	1361	2821	4761	**41895**	**163**
25	10	66.7	159	12	350	0	14	26	0	**26**	580	1361	2821	4761	**41895**	**164**
49	8	83.1	127	23	350	0	27	50	0	**50**	1129	1361	2821	5311	**41345**	**104**
49	9	70.7	143	23	350	0	17	40	0	**40**	1129	1361	2821	5311	**41345**	**116**
49	10	66.7	159	23	350	0	14	37	0	**37**	1129	1361	2821	5311	**41345**	**113**
71	8	83.1	127	33	350	0	27	60	0	**60**	1627	1361	2821	5809	**40847**	**86**
71	9	70.7	143	33	350	0	17	50	0	**50**	1627	1361	2821	5809	**40847**	**91**
71	10	66.7	159	33	350	0	14	47	0	**47**	1627	1361	2821	5809	**40847**	**88**

Calculation of Movement at Joint Ends

			Temperature					
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	Total Movement (in)
25	0.000	12	0.074	0.043	0.058	0.045	0.0225	0.003	**0.244**
49	0.001	23	0.145	0.083	0.112	0.088	0.044	0.006	**0.479**
71	0.003	33	0.210	0.120	0.162	0.128	0.0639	0.011	**0.694**

Median

Calculation of Required Prestress for Longitudinal Direction

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 3.00E-06				α = 3.00E-06				α = 3.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 250				L = 300				L = 350			
Temperature Differential (°F/in)	ΔT = 3				ΔT = 3				ΔT = 3			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	σ_c =	191	214	238	σ_c =	191	214	238	σ_c =	191	214	238
Friction Stress (psi)	σ_F =	115	115	115	σ_F =	138	138	138	σ_F =	161	161	161

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR}+\sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
250	8	149.5	191	115	350	105	63	178	215	**215**	5482	1361	2821	9664	**36992**	**22**
250	9	129.0	214	115	350	108	47	162	215	**215**	5482	1361	2821	9664	**36992**	**19**
250	10	112.3	238	115	350	116	34	149	215	**215**	5482	1361	2821	9664	**36992**	**17**
300	8	149.5	191	138	350	128	63	201	238	**238**	6499	1361	2821	10681	**35975**	**19**
300	9	129.0	214	138	350	131	47	185	238	**238**	6499	1361	2821	10681	**35975**	**17**
300	10	112.3	238	138	350	139	34	172	238	**238**	6499	1361	2821	10681	**35975**	**15**
350	8	149.5	191	161	350	151	63	224	261	**261**	7490	1361	2821	11672	**34984**	**17**
350	9	129.0	214	161	350	154	47	208	261	**261**	7490	1361	2821	11672	**34984**	**15**
350	10	112.3	238	161	350	162	34	195	261	**261**	7490	1361	2821	11672	**34984**	**13**

Calculation of Movement at Joint Ends

			Temperature						
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.369	0.178	0.252	0.450	0.225	0.143	**1.617**
300	0.055	138	0.443	0.204	0.294	0.540	0.270	0.190	**1.941**
350	0.075	161	0.517	0.227	0.332	0.630	0.315	0.244	**2.264**

Calculation of Required Prestress for Longitudinal Direction

Concrete and Pavement Properties:

28-day Flexural Strength	$f =$	700			$f =$	700			$f =$	700		
Safety Factor (ACI=2)	$SF =$	2			$SF =$	2			$SF =$	2		
Allowable Flexural Stress	$f_{design} =$	350			$f_{design} =$	350			$f_{design} =$	350		
Concrete Modulus of Elasticity (psi)	$E =$	4500000			$E =$	4500000			$E =$	4500000		
Concrete Poisson's Ration	$\mu =$	0.15			$\mu =$	0.15			$\mu =$	0.15		
Coefficient of Thermal Expansion (/°F)	$\alpha =$	6.00E-06			$\alpha =$	6.00E-06			$\alpha =$	6.00E-06		
Concrete Unit Weight (pcf)	$\gamma =$	144			$\gamma =$	144			$\gamma =$	144		
Slab Length (ft)	$L =$	250			$L =$	300			$L =$	350		
Temperature Differential (°F/in)	$\Delta T =$	3			$\Delta T =$	3			$\Delta T =$	3		
Coefficient of Friction (slab-support)	$\mu_{max} =$	0.92			$\mu_{max} =$	0.92			$\mu_{max} =$	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	$D =$	8	9	10	$D =$	8	9	10	$D =$	8	9	10
Tensile Stress at Bottom of Slab (psi)	$\sigma_t =$	115	99.2	86.4	$\sigma_t =$	115	99.2	86.4	$\sigma_t =$	115	99.2	86.4
Critical Stress Factors (edge condition)	$CSF =$	1.3										
Required Fatigue Prestress (psi)	$\sigma_{PR} =$	63	47	34	$\sigma_{PR} =$	63	47	34	$\sigma_{PR} =$	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design} =$	149.5	129.0	112.3	$\sigma_{t\ design} =$	149.5	129.0	112.3	$\sigma_{t\ design} =$	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	$\sigma_c =$	381	429	476	$\sigma_c =$	381	429	476	$\sigma_c =$	381	429	476
Friction Stress (psi)	$\sigma_F =$	115	115	115	$\sigma_F =$	138	138	138	$\sigma_F =$	161	161	161

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	$Ss = 270000$	Tendon Yield Strength (psi)	$Sy = 229500$
Tendon Elastic Modulus (psi)	$E_s = 28000000$	Tendon Yield Force (lbs)	$Fy = 49572$
Tendon Area, 0.6 in. Diameter (in^2)	$a = 0.216$	Tendon Prestressing Force (lbs)	$F = 34700$
Tendon Wooble Coefficient	$K = 0.001$	Time After Stressing (design hr)	$t_{hr} = 262800$
Design Tendon Strenght (decimals)	$DS_s = 0.8$		$Fjack = 46656$
Concrete Shrinkage Strain (in/in)	$\varepsilon_s = 0.00015$		
Concrete Creep Strain (in/in)	$\varepsilon_k = 0.000075$		
Concrete Strain (Shrinkage and Creep)	$\varepsilon_{cu} = 2.25E-04$		
Time After Stressing (design yrs)	$t = 30$		

Climatic Factors:	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	$\sigma_F + 100$ (psi)	**Governing Value (psi)**	ΔFS_1	ΔFS_2	ΔFS_3	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
250	8	149.5	381	115	350	296	63	178	215	**296**	5482	1361	2821	9664	**36992**	**16**
250	9	129.0	429	115	350	323	47	162	215	**323**	5482	1361	2821	9664	**36992**	**13**
250	10	112.3	476	115	350	354	34	149	215	**354**	5482	1361	2821	9664	**36992**	**10**
300	8	149.5	381	138	350	319	63	201	238	**319**	6499	1361	2821	10681	**35975**	**14**
300	9	129.0	429	138	350	346	47	185	238	**346**	6499	1361	2821	10681	**35975**	**12**
300	10	112.3	476	138	350	377	34	172	238	**377**	6499	1361	2821	10681	**35975**	**10**
350	8	149.5	381	161	350	342	63	224	261	**342**	7490	1361	2821	11672	**34984**	**13**
350	9	129.0	429	161	350	369	47	208	261	**369**	7490	1361	2821	11672	**34984**	**11**
350	10	112.3	476	161	350	400	34	195	261	**400**	7490	1361	2821	11672	**34984**	**9**

Calculation of Movement at Joint Ends

				Temperature					
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.738	0.394	0.543	0.450	0.225	0.197	**2.547**
300	0.055	138	0.886	0.463	0.642	0.540	0.270	0.255	**3.056**
350	0.075	161	1.033	0.530	0.739	0.630	0.315	0.319	**3.566**

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Siliceous River Gravel

Calculation of Required Prestress for Longitudinal Direction

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f =	700			f =	700			f =	700		
Safety Factor (ACI=2)	SF =	2			SF =	2			SF =	2		
Allowable Flexural Stress	f_{design} =	350			f_{design} =	350			f_{design} =	350		
Concrete Modulus of Elasticity (psi)	E =	4500000			E =	4500000			E =	4500000		
Concrete Poisson's Ration	μ =	0.15			μ =	0.15			μ =	0.15		
Coefficient of Thermal Expansion (/°F)	α =	3.00E-06			α =	3.00E-06			α =	3.00E-06		
Concrete Unit Weight (pcf)	γ=	144			γ=	144			γ=	144		
Slab Length (ft)	L =	250			L =	300			L =	350		
Temperature Differential (°F/in)	ΔT =	1			ΔT =	1			ΔT =	1		
Coefficient of Friction (slab-support)	μ_{max}=	0.92			μ_{max}=	0.92			μ_{max}=	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	σ_c =	64	71	79	σ_c =	64	71	79	σ_c =	64	71	79
Friction Stress (psi)	σ_F =	115	115	115	σ_F =	138	138	138	σ_F =	161	161	161

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	- Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	Governing Value (psi)	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	Final prestress force (lbs)	Strand Spacing (in)
250	8	149.5	64	115	350	-22	63	178	215	**215**	5482	1361	2821	9664	**36992**	**22**
250	9	129.0	71	115	350	-35	47	162	215	**215**	5482	1361	2821	9664	**36992**	**19**
250	10	112.3	79	115	350	-43	34	149	215	**215**	5482	1361	2821	9664	**36992**	**17**
300	8	149.5	64	138	350	1	63	201	238	**238**	6499	1361	2821	10681	**35975**	**19**
300	9	129.0	71	138	350	-12	47	185	238	**238**	6499	1361	2821	10681	**35975**	**17**
300	10	112.3	79	138	350	-20	34	172	238	**238**	6499	1361	2821	10681	**35975**	**15**
350	8	149.5	64	161	350	24	63	224	261	**261**	7490	1361	2821	11672	**34984**	**17**
350	9	129.0	71	161	350	11	47	208	261	**261**	7490	1361	2821	11672	**34984**	**15**
350	10	112.3	79	161	350	3	34	195	261	**261**	7490	1361	2821	11672	**34984**	**13**

Calculation of Movement at Joint Ends

			Temperature					
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.369	0.178	0.252	0.450	0.225	0.143	**1.617**
300	0.055	138	0.443	0.204	0.294	0.540	0.270	0.190	**1.941**
350	0.075	161	0.517	0.227	0.332	0.630	0.315	0.244	**2.264**

Calculation of Required Prestress for Longitudinal Direction

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Siliceous River Gravel

Concrete and Pavement Properties:

28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 6.00E-06				α = 6.00E-06				α = 6.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 250				L = 300				L = 350			
Temperature Differential (°F/in)	ΔT = 1				ΔT = 1				ΔT = 1			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	σ_c =	127	143	159	σ_c =	127	143	159	σ_c =	127	143	159
Friction Stress (psi)	σ_F =	115	115	115	σ_F =	138	138	138	σ_F =	161	161	161

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:

	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	**Governing Value (psi)**	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	**Final prestress force (lbs)**	**Strand Spacing (in)**
250	8	149.5	127	115	350	42	63	178	215	**215**	5482	1361	2821	9664	**36992**	**22**
250	9	129.0	143	115	350	37	47	162	215	**215**	5482	1361	2821	9664	**36992**	**19**
250	10	112.3	159	115	350	36	34	149	215	**215**	5482	1361	2821	9664	**36992**	**17**
300	8	149.5	127	138	350	65	63	201	238	**238**	6499	1361	2821	10681	**35975**	**19**
300	9	129.0	143	138	350	60	47	185	238	**238**	6499	1361	2821	10681	**35975**	**17**
300	10	112.3	159	138	350	59	34	172	238	**238**	6499	1361	2821	10681	**35975**	**15**
350	8	149.5	127	161	350	88	63	224	261	**261**	7490	1361	2821	11672	**34984**	**17**
350	9	129.0	143	161	350	83	47	208	261	**261**	7490	1361	2821	11672	**34984**	**15**
350	10	112.3	159	161	350	82	34	195	261	**261**	7490	1361	2821	11672	**34984**	**13**

Calculation of Movement at Joint Ends

			Temperature					
		Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
250	0.038	115	0.738	0.394	0.543	0.450	0.225	0.143	**2.493**
300	0.055	138	0.886	0.463	0.642	0.540	0.270	0.190	**2.992**
350	0.075	161	1.033	0.530	0.739	0.630	0.315	0.244	**3.490**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f =	700			f =	700			f =	700		
Safety Factor (ACI=2)	SF =	2			SF =	2			SF =	2		
Allowable Flexural Stress	f_{design} =	350			f_{design} =	350			f_{design} =	350		
Concrete Modulus of Elasticity (psi)	E =	4500000			E =	4500000			E =	4500000		
Concrete Poisson's Ration	μ =	0.15			μ =	0.15			μ =	0.15		
Coefficient of Thermal Expansion (/°F)	α =	3.00E-06			α =	3.00E-06			α =	3.00E-06		
Concrete Unit Weight (pcf)	γ=	144			γ=	144			γ=	144		
Slab Length (ft)	L =	25			L =	49			L =	71		
Temperature Differential (°F/in)	ΔT =	3			ΔT =	3			ΔT =	3		
Coefficient of Friction (slab-support)	μ_{max}=	0.92			μ_{max}=	0.92			μ_{max}=	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	σ_c =	191	214	238	σ_c =	191	214	238	σ_c =	191	214	238
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max -	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	Governing Value (psi)	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	Final prestress force (lbs)	Strand Spacing (in)
25	8	149.5	191	12	350	0	63	75	0	**75**	580	1361	2821	4761	**41895**	**70**
25	9	129.0	214	12	350	0	47	59	0	**59**	580	1361	2821	4761	**41895**	**80**
25	10	112.3	238	12	350	0	34	46	0	**46**	580	1361	2821	4761	**41895**	**92**
49	8	149.5	191	23	350	0	63	86	0	**86**	1129	1361	2821	5311	**41345**	**60**
49	9	129.0	214	23	350	0	47	70	0	**70**	1129	1361	2821	5311	**41345**	**66**
49	10	112.3	238	23	350	0	34	57	0	**57**	1129	1361	2821	5311	**41345**	**73**
71	8	149.5	191	33	350	0	63	96	0	**96**	1627	1361	2821	5809	**40847**	**53**
71	9	129.0	214	33	350	0	47	80	0	**80**	1627	1361	2821	5809	**40847**	**57**
71	10	112.3	238	33	350	0	34	67	0	**67**	1627	1361	2821	5809	**40847**	**61**

Calculation of Movement at Joint Ends

			Temperature						
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
25	0.000	12	0.037	0.021	0.029	0.045	0.0225	0.005	**0.159**
49	0.001	23	0.072	0.041	0.056	0.088	0.044	0.011	**0.312**
71	0.003	33	0.105	0.058	0.079	0.128	0.0639	0.018	**0.452**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Summer

Aggregate Type:
Siliceous River Gravel

Concrete and Pavement Properties:												
28-day Flexural Strength	f = 700				f = 700				f = 700			
Safety Factor (ACI=2)	SF = 2				SF = 2				SF = 2			
Allowable Flexural Stress	f_{design} = 350				f_{design} = 350				f_{design} = 350			
Concrete Modulus of Elasticity (psi)	E = 4500000				E = 4500000				E = 4500000			
Concrete Poisson's Ration	μ = 0.15				μ = 0.15				μ = 0.15			
Coefficient of Thermal Expansion (/°F)	α = 6.00E-06				α = 6.00E-06				α = 6.00E-06			
Concrete Unit Weight (pcf)	γ= 144				γ= 144				γ= 144			
Slab Length (ft)	L = 25				L = 49				L = 71			
Temperature Differential (°F/in)	ΔT = 3				ΔT = 3				ΔT = 3			
Coefficient of Friction (slab-support)	μ_{max}= 0.92				μ_{max}= 0.92				μ_{max}= 0.92			
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	σ_c =	381	429	476	σ_c =	381	429	476	σ_c =	381	429	476
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:			
Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	-	Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86		45	41
Temperature Change During Summer Day (ΔT Summer) =	110		86	24
Temperature Change During Winter Day (ΔT Winter) =	48		10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	Governing Value (psi)	Δ FS_1	Δ FS_2	Δ FS_3	Total losses (lbs)	Final prestress force (lbs)	Strand Spacing (in)
25	8	149.5	381	12	350	0	63	75	0	**75**	580	1361	2821	4761	**41895**	**70**
25	9	129.0	429	12	350	0	47	59	0	**59**	580	1361	2821	4761	**41895**	**80**
25	10	112.3	476	12	350	0	34	46	0	**46**	580	1361	2821	4761	**41895**	**92**
49	8	149.5	381	23	350	0	63	86	0	**86**	1129	1361	2821	5311	**41345**	**60**
49	9	129.0	429	23	350	0	47	70	0	**70**	1129	1361	2821	5311	**41345**	**66**
49	10	112.3	476	23	350	0	34	57	0	**57**	1129	1361	2821	5311	**41345**	**73**
71	8	149.5	381	33	350	0	63	96	0	**96**	1627	1361	2821	5809	**40847**	**53**
71	9	129.0	429	33	350	0	47	80	0	**80**	1627	1361	2821	5809	**40847**	**57**
71	10	112.3	476	33	350	0	34	67	0	**67**	1627	1361	2821	5809	**40847**	**61**

Calculation of Movement at Joint Ends

			Temperature						
			Seasonal	Summer	Winter	Shrinkage	Creep	Elastic Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
25	0.000	12	0.074	0.043	0.058	0.045	0.0225	0.005	**0.247**
49	0.001	23	0.145	0.083	0.112	0.088	0.044	0.011	**0.484**
71	0.003	33	0.210	0.120	0.162	0.128	0.0639	0.018	**0.701**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Limestone

Concrete and Pavement Properties:

28-day Flexural Strength	f =	700			f =	700			f =	700		
Safety Factor (ACI=2)	SF =	2			SF =	2			SF =	2		
Allowable Flexural Stress	f_{design} =	350			f_{design} =	350			f_{design} =	350		
Concrete Modulus of Elasticity (psi)	E =	4500000			E =	4500000			E =	4500000		
Concrete Poisson's Ration	μ =	0.15			μ =	0.15			μ =	0.15		
Coefficient of Thermal Expansion (/°F)	α =	3.00E-06			α =	3.00E-06			α =	3.00E-06		
Concrete Unit Weight (pcf)	γ=	144			γ=	144			γ=	144		
Slab Length (ft)	L =	25			L =	49			L =	71		
Temperature Differential (°F/in)	ΔT =	1			ΔT =	1			ΔT =	1		
Coefficient of Friction (slab-support)	μ_{max}=	0.92			μ_{max}=	0.92			μ_{max}=	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	σ_c =	64	71	79	σ_c =	64	71	79	σ_c =	64	71	79
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	- Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	Governing Value (psi)	ΔFS_1	ΔFS_2	ΔFS_3	Total losses (lbs)	Final prestress force (lbs)	Strand Spacing (in)
25	8	149.5	64	12	350	0	63	75	0	**75**	580	1361	2821	4761	**41895**	**70**
25	9	129.0	71	12	350	0	47	59	0	**59**	580	1361	2821	4761	**41895**	**80**
25	10	112.3	79	12	350	0	34	46	0	**46**	580	1361	2821	4761	**41895**	**92**
49	8	149.5	64	23	350	0	63	86	0	**86**	1129	1361	2821	5311	**41345**	**60**
49	9	129.0	71	23	350	0	47	70	0	**70**	1129	1361	2821	5311	**41345**	**66**
49	10	112.3	79	23	350	0	34	57	0	**57**	1129	1361	2821	5311	**41345**	**73**
71	8	149.5	64	33	350	0	63	96	0	**96**	1627	1361	2821	5809	**40847**	**53**
71	9	129.0	71	33	350	0	47	80	0	**80**	1627	1361	2821	5809	**40847**	**57**
71	10	112.3	79	33	350	0	34	67	0	**67**	1627	1361	2821	5809	**40847**	**61**

Calculation of Movement at Joint Ends

			Temperature					Elastic	
			Seasonal	Summer	Winter	Shrinkage	Creep	Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
25	0.000	12	0.037	0.021	0.029	0.045	0.0225	0.005	**0.159**
49	0.001	23	0.072	0.041	0.056	0.088	0.044	0.011	**0.312**
71	0.003	33	0.105	0.058	0.079	0.128	0.0639	0.018	**0.452**

Calculation of Required Prestress for Transverse Direction

Description:
Design of PCP for new pavement on median in Hillsboro, Texas

Season:
Winter

Aggregate Type:
Siliceous River Gravel

Concrete and Pavement Properties:

28-day Flexural Strength	f =	700			f =	700			f =	700		
Safety Factor (ACI=2)	SF =	2			SF =	2			SF =	2		
Allowable Flexural Stress	f_{design} =	350			f_{design} =	350			f_{design} =	350		
Concrete Modulus of Elasticity (psi)	E =	4500000			E =	4500000			E =	4500000		
Concrete Poisson's Ration	μ =	0.15			μ =	0.15			μ =	0.15		
Coefficient of Thermal Expansion (/°F)	α =	6.00E-06			α =	6.00E-06			α =	6.00E-06		
Concrete Unit Weight (pcf)	γ=	144			γ=	144			γ=	144		
Slab Length (ft)	L =	25			L =	49			L =	71		
Temperature Differential (°F/in)	ΔT =	1			ΔT =	1			ΔT =	1		
Coefficient of Friction (slab-support)	μ_{max}=	0.92			μ_{max}=	0.92			μ_{max}=	0.92		
Thickness (in)		D_1	D_2	D_3		D_1	D_2	D_3		D_1	D_2	D_3
	D =	8	9	10	D =	8	9	10	D =	8	9	10
Tensile Stress at Bottom of Slab (psi)	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4	σ_t =	115	99.2	86.4
Critical Stress Factors (edge condition)	CSF =	1.3										
Required Fatigue Prestress (psi)	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34	σ_{PR} =	63	47	34
Wheel Load Stress (corrected, psi)	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3	$\sigma_{t\ design}$ =	149.5	129.0	112.3
Curling Stress at Slab Center (psi)	σ_c =	127	143	159	σ_c =	127	143	159	σ_c =	127	143	159
Friction Stress (psi)	σ_F =	12	12	12	σ_F =	23	23	23	σ_F =	33	33	33

Prestressing Tendon Properties:

Tendon Ultimate Strength (psi)	Ss = 270000	Tendon Yield Strength (psi)	Sy = 229500
Tendon Elastic Modulus (psi)	E_s = 28000000	Tendon Yield Force (lbs)	Fy = 49572
Tendon Area, 0.6 in. Diameter (in^2)	a= 0.216	Tendon Prestressing Force (lbs)	F = 34700
Tendon Wooble Coefficient	K= 0.001	Time After Stressing (design hr)	t_{hr} = 262800
Design Tendon Strenght (decimals)	DS_s= 0.8		Fjack= 46656
Concrete Shrinkage Strain (in/in)	ε_s = 0.00015		
Concrete Creep Strain (in/in)	ε_k = 0.000075		
Concrete Strain (Shrinkage and Creep)	ε_{cu} = 2.25E-04		
Time After Stressing (design yrs)	t = 30		

Climatic Factors:	Max	- Min	
Seasonal Temperature Change from Summer to Winter (ΔT Season) =	86	45	41
Temperature Change During Summer Day (ΔT Summer) =	110	86	24
Temperature Change During Winter Day (ΔT Winter) =	48	10	38

Required Prestress at Ends of Slabs in Longitudinal Direction

From Elastic Design							From Fatigue and Elastic Analyses				Prestress Losses (Strand Friction, Shrinkage and Creep, and Steel Relaxation)					
Slab Length (ft)	D (in)	σ_t +	σ_c +	σ_F -	f_{design} =	Elastic Prestress σ_p (psi)	Fatigue Prestress σ_{PR} (psi)	$\sigma_{PR} + \sigma_F$ (psi)	σ_F + 100 (psi)	Governing Value (psi)	Δ FS$_1$	Δ FS$_2$	Δ FS$_3$	Total losses (lbs)	Final prestress force (lbs)	Strand Spacing (in)
25	8	149.5	127	12	350	0	63	75	0	**75**	580	1361	2821	4761	**41895**	**70**
25	9	129.0	143	12	350	0	47	59	0	**59**	580	1361	2821	4761	**41895**	**80**
25	10	112.3	159	12	350	0	34	46	0	**46**	580	1361	2821	4761	**41895**	**92**
49	8	149.5	127	23	350	0	63	86	0	**86**	1129	1361	2821	5311	**41345**	**60**
49	9	129.0	143	23	350	0	47	70	0	**70**	1129	1361	2821	5311	**41345**	**66**
49	10	112.3	159	23	350	0	34	57	0	**57**	1129	1361	2821	5311	**41345**	**73**
71	8	149.5	127	33	350	0	63	96	0	**96**	1627	1361	2821	5809	**40847**	**53**
71	9	129.0	143	33	350	0	47	80	0	**80**	1627	1361	2821	5809	**40847**	**57**
71	10	112.3	159	33	350	0	34	67	0	**67**	1627	1361	2821	5809	**40847**	**61**

Calculation of Movement at Joint Ends

			Temperature					Elastic	
			Seasonal	Summer	Winter	Shrinkage	Creep	Shortening	
Slab Length (ft)	d_f (in)	σ_F	d_1 (in)	d_2 (in)	d_3 (in)	d_4 (in)	d_5 (in)	d_6 (in)	**Total Movement (in)**
25	0.000	12	0.074	0.043	0.058	0.045	0.0225	0.005	**0.247**
49	0.001	23	0.145	0.083	0.112	0.088	0.044	0.011	**0.484**
71	0.003	33	0.210	0.120	0.162	0.128	0.0639	0.018	**0.701**