Construction of the California Precast Concrete Pavement Demonstration Project

PUBLICATION NO. FHWA IF 06 010

AUGUST 2004

U.S. Department of Transportation
Federal Highway Administration

Office of Infrastructure
Office of Pavement Technology
Washington, DC 20590

CONCRETE PAVEMENT
CPTP
TECHNOLOGY PROGRAM

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Technical Report Documentation Page

1. Report No. FHWA-IF-06-010	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Construction of the California Precast Concrete Pavement Demonstration Project		5. Report Date August 2004 6. Performing Organization Code
7. Author(s) David K. Merritt, B. Frank McCullough, Ned H. Burns, Robert Otto Rasmussen		8. Performing Organization Report No.
9. Performing Organization Name and Address The Transtec Group, Inc. 1012 East 38 1/2 St. Austin, Texas 78751		10. Work Unit No. (TRAIS) 11. Contract or Grant No. DTFH61-03-R-00105
12. Sponsoring Agency Name and Address U.S. Department of Transportation Federal Highway Administration 1200 New Jersey Avenue, SE, HIPT-20 Washington, DC 20590		13. Type of Report and Period Covered Final Report June 2003–August 2004 14. Sponsoring Agency Code HIPT-20

15. Supplementary Notes
Contracting Officer's Technical Representative: Sam Tyson, FHWA Office of Pavement Technology, 1200 New Jersey Avenue SE, HIPT-20 / E73-440, Washington, DC 20590,
E-mail: sam.tyson@dot.gov, Phone: 202-366-1326

16. Abstract

This report documents the construction of a precast prestressed concrete pavement demonstration project on Interstate 10 in El Monte, California. This work was conducted under a new effort by the Federal Highway Administration to further evaluate and refine precast prestressed pavement technology through the construction of additional demonstration projects. All aspects of the California demonstration project are presented, including the design, panel fabrication, pavement construction, and instrumentation and monitoring. An evaluation of each of these aspects is also discussed. This project represents the first of three demonstration projects to be constructed under this effort.

17. Key Word	18. Distribution Statement
precast pavement, prestressed pavement, concrete pavement, expedited construction	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
		82	

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

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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CHAPTER 1. INTRODUCTION

BACKGROUND

In the past decade, State transportation agencies have become desperate for construction techniques that will help them to "Get in, get out, and stay out!" To "get in, get out" implies rapid construction with minimal disruption to the motoring public. As if this is not challenging enough, agencies must also "stay out," requiring pavement structures that are long lasting and durable, with minimal maintenance over a 30- to 50-year lifespan.

Precast prestressed concrete pavement is a solution for this need, providing both improved durability and rapid construction. In 2000, a feasibility study was completed by the Center for Transportation Research (CTR) at The University of Texas at Austin that examined the use of precast prestressed panels for expedited pavement construction.[1] This study was followed by an implementation study funded by the Federal Highway Administration (FHWA) and conducted by CTR, which resulted in the construction of a 0.7-km (2,300 ft) precast prestressed concrete pavement pilot project near Georgetown, Texas.[2]

FHWA DEMONSTRATION PROJECTS

The Texas precast pavement pilot project demonstrated that precast prestressed concrete is a viable technique for rapid pavement construction. However, due to unfamiliarity with this new technology, it is difficult for agencies to evaluate this technology further. In response to this need, FHWA initiated an effort to construct additional precast pavement demonstration projects based on the concept developed from the CTR feasibility study. The purpose of these new demonstration projects was to further evaluate the viability of precast prestressed pavement for "real world" applications while also familiarizing contractors and agencies with this new technology. As part of this effort, the design, construction, and cost of each of these demonstration projects was evaluated.

This report discusses the first of these new demonstration projects, constructed on a section of Interstate 10 (I-10) in El Monte, California, in April 2004.

BENEFITS OF PRECAST CONCRETE PAVEMENT

One of the biggest benefits of precast concrete is the improvement in durability that can be realized by casting the panels in a controlled environment. Precast manufacturers are able to produce very consistent concrete mixes with a high degree of quality control.[3] Because the mixture is transported only a short distance from the batch plant to the forms, it permits the use of durable, low-permeability mixes with a low water-to-cementitious-materials ratio and reduces the probability of problems such as segregation and flash set. Other problems that are commonly encountered with cast-in-place concrete pavement, such as so-called "built-in curl" (from temperature and moisture gradients in the slab), surface strength loss (from insufficient curing), and inadequate air entrainment, can also be minimized or eliminated with precast concrete.[1]

Prestressing further benefits precast pavement durability by inducing a compressive stress in the pavement to greatly reduce or even prevent the occurrence of cracking.[1] This benefit was demonstrated by a cast-in-place prestressed pavement, 150 mm (6 in.) thick, constructed on I-35 near West, Texas, in 1985, that has required virtually no maintenance over its 19-year life.[4,5]

In addition to enhancing durability, prestressing also permits a significant reduction in slab thickness. With the introduction of a precompressive stress into thinner, precast pavement panels, tensile stresses caused by wheel loads can be limited to those of a thicker, non-prestressed pavement, resulting in a design life equivalent to a much thicker pavement. For example, the precast pavement in Georgetown, Texas, was designed, at 200 mm (8 in.) thick, to have a life equivalent to a continuously reinforced concrete pavement that is 355 mm (14 in.) thick.[2]

Perhaps the most obvious benefit of precast concrete for pavements is reduced construction time. One of the problems with conventional portland cement concrete pavement is the curing requirements. Portland cement concrete needs time to reach a specified strength before it can be opened to traffic. Although fast-setting concretes are available,[6] there is still some question regarding the early-age and long-term durability of these mixes.[7,8] Precast concrete panels, on the other hand, are cast and cured prior to placement, ensuring adequate time to reach the required design strength under controlled curing conditions. This means that the panels can be set in place and opened to traffic immediately, greatly reducing construction time by eliminating the cure time requirement. This benefit allows construction to take place during overnight or weekend operations.

Reduced construction time and improved durability will greatly reduce user costs associated with pavement construction and rehabilitation. User delay costs are accumulated when construction activities cause traffic congestion. These costs can be substantial, as demonstrated by the CTR feasibility study, which estimated daily user delay costs for a four-lane divided facility carrying 50,000 vehicles per day to be as high as $383,000 per day for 24-hour-per-day lane closure versus only $1,800 per day for nighttime lane closure only.[1] (It should be noted that 50,000 vehicles per day is very conservative. Many urban facilities carry in excess of 200,000 vehicles per day). Likewise, user costs resulting from poor pavement condition can be substantial. As a 2001 TRIP (The Road Information Program) report indicated, Americans spend over $41 billion annually on vehicle repair due to roads in poor condition.[9] Smoother and more durable pavements will substantially reduce these types of user costs.

Finally, another benefit of precast concrete that may not be immediately apparent is the potential for extending the construction season for paving. Because precast panels are cast and cured in a controlled environment, they are not as susceptible to on-site environmental conditions as conventional concrete. This will permit pavement construction to continue under adverse weather conditions, such as subfreezing or extremely hot temperatures, that would normally prohibit cast-in-place concrete paving.

REPORT OBJECTIVES

While this report introduces the background and fundamentals of precast prestressed concrete paving, the primary objective of the report is to describe a demonstration project constructed by the California Department of Transportation (Caltrans) in El Monte, California. The design, fabrication, and construction procedures are discussed and evaluated for applicability for future projects. This report also presents recommendations for future precast pavement construction.

Following is a summary of the remaining chapters of this report:

- Chapter 2 presents the precast pavement concept developed through the feasibility study, described previously. This includes the panel types, base preparation, panel assembly, post-tensioning, and grouting.

- Chapter 3 presents the details of the California demonstration project, including the scope of application and the project layout.

- Chapter 4 presents the design for the California demonstration project. This includes the design considerations for precast prestressed concrete pavement, the design procedure, and the final design recommendations.

- Chapter 5 discusses the fabrication of the precast panels for the California demonstration project. This includes the panel details and fabrication and handling procedures.

- Chapter 6 discusses the construction of the precast pavement on site. This includes base preparation, transportation, panel placement, post-tensioning, and grouting.

- Chapter 7 presents the instrumentation and monitoring of the California demonstration project, including instrumentation, condition survey, and monitoring of slab movements.

- Chapter 8 presents the overall project evaluation, including and assessment of design, fabrication, construction, and overall cost. Recommendations are also given for future precast pavement projects.

- Chapter 9 presents a summary of the project and recommendations for future projects based on observations from the California demonstration project.

CHAPTER 2. PRECAST CONCRETE PAVEMENT CONCEPT

INTRODUCTION

As mentioned in chapter 1, the California precast pavement demonstration project was based on a concept developed through a feasibility study completed by the Center for Transportation Research. While several different concepts for precast pavement have been developed in the past several years, the focus of these new FHWA demonstration projects is the prestressed precast pavement concept developed by CTR.[1] This concept was successfully implemented in a precast pavement pilot project constructed near Georgetown, Texas in 2002.[2] While several improvements have been made to the concept based on the Texas pilot project, the overall concept is essentially the same for the California demonstration project, as described below.

Full-Depth Panels

The precast pavement concept utilizes full-depth precast panels. Full-depth panels are a very efficient solution in that the pavement can be opened to traffic almost immediately after installation of the precast panels. Additional construction time is not required for a hot-mix asphalt or thin bonded concrete overlay wearing course. Full-depth panels were demonstrated to provide acceptable ride quality without the need to overlay or diamond grind for the Texas pilot project.[2] While it is anticipated that surface diamond grinding will be required to achieve more stringent ride-quality standards, full-depth panels can still be opened to traffic in the interim, prior to diamond grinding.

Full-depth panels require careful consideration of both base preparation and vertical alignment between panels. Based upon the conclusions from the CTR feasibility study and the Texas pilot project, it is apparent that either a hot-mix asphalt or portland cement base can be placed smooth and flat enough to serve as a leveling course beneath the precast panels. As demonstrated by the Texas pilot project, a hot-mix asphalt leveling course can actually deform under the weight of the precast panels, conforming to the bottom surface and minimizing voids.[2]

Vertical alignment between adjacent precast panels is ensured by casting continuous shear keys into the edges of the panels. The keyways have been demonstrated to “lock” the precast panels together vertically, providing load transfer prior to post-tensioning, and ensuring satisfactory ride quality of the finished surface. The keyways have also been shown to expedite panel installation by eliminating the requirement to “level” adjacent panels after installation.

Prestressed Pavement

The concept for precast pavement incorporates prestressing. As discussed in chapter 1, prestressing not only improves the durability of the pavement, but also permits a significant reduction in slab thickness by inducing a precompressive stress in the pavement that must be overcome before tensile stresses that lead to cracking can occur.

Prestressing in both the longitudinal (in the direction of traffic flow) and transverse (normal to traffic flow) directions is essential for prestressed pavements. Previous (cast-in-place) prestressed pavements constructed in the United States that had only longitudinal prestressing developed longitudinal cracking over time.[10] A cast-in-place prestressed pavement constructed on I-35 near West, Texas, which had both longitudinal and transverse prestress, has only recently developed minor cracking after 19 years of heavy truck traffic.[5]

Bi-directional prestressing is incorporated into the precast pavement through both pretensioning and post-tensioning. The precast panels are pretensioned in the transverse direction (long axis of the panel) during fabrication, and are post-tensioned together in the longitudinal direction after installation on site. Transverse pretensioning not only provides the necessary prestress for long-term durability, but also prevents cracking during lifting and handling of the panels. Likewise, longitudinal post-tensioning not only provides the necessary prestress for long-term durability, but also provides load transfer between panels.

PANEL ASSEMBLY

Figure 1 shows a typical precast panel assembly. The panels are installed transverse to the flow of traffic, incorporating both traffic lanes and shoulders if possible. The three types of panels that make up a precast prestressed pavement—base, joint, and central stressing panels—are shown in figures 2, 3, and 4. As described previously, all of the panels are pretensioned lengthwise (transverse to the flow of traffic), and monostrand post-tensioning ducts are cast into each panel widthwise (parallel to the flow of traffic) for longitudinal post-tensioning after the panels are all assembled. Note also the continuous shear keys cast into the edges of the panels to ensure vertical alignment and temporary load transfer, as described previously.

After each section of panels is installed (from joint panel to joint panel), the post-tensioning strands are fed into the post-tensioning ducts from the pockets in the central stressing panels (described below). The strands are pushed or pulled through all of the panels to the post-tensioning anchors cast into the joint panels. Post-tensioning is then completed from the pockets in the central stressing panels. Each post-tensioned slab acts independently of the adjacent slab in terms of expansion and contraction movements. Expansion joints are cast into the joint panels (described below) to permit adjacent slabs to move independently. The length of each post-tensioned slab can be adjusted by increasing or decreasing the number of base panels between the joint panels and central stressing panels.

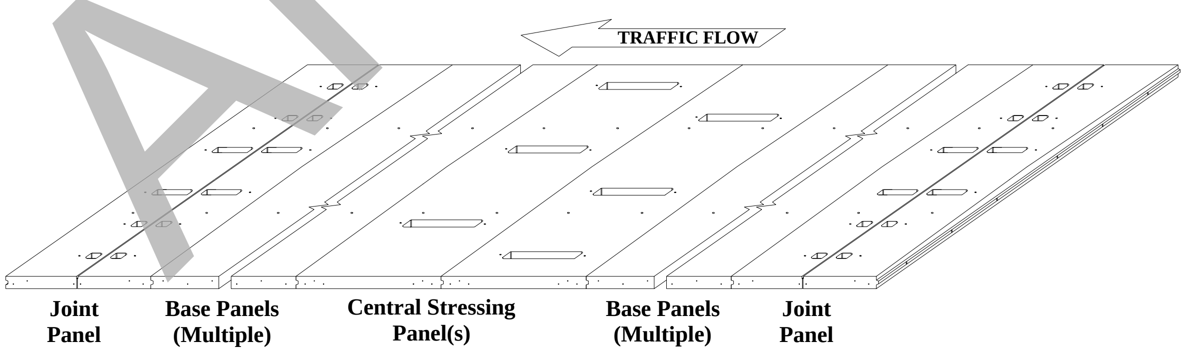

Figure 1. Illustration. Typical precast prestressed panel layout.

Base Panels

The base panels, shown in figure 2, are the most basic of the three panels and are the majority of the panels in each post-tensioned slab. Typical details of the base panels include continuous shear keys along the panel edges, post-tensioning ducts, pretensioning spaced evenly to provide uniform transverse prestress, and lifting anchors, located approximately 0.2L from each edge of the panel.

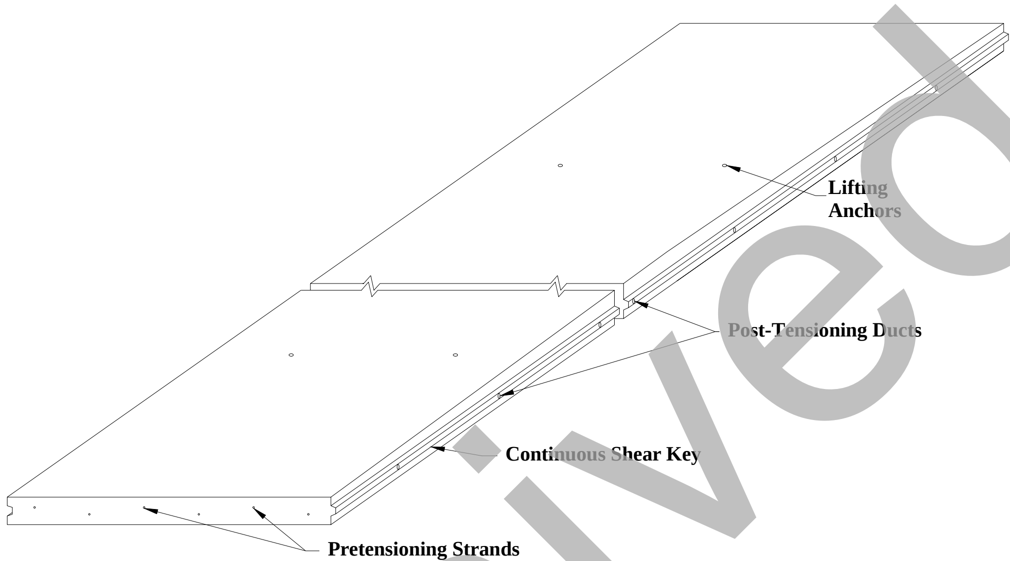

Figure 2. Illustration. Typical base panel.

Joint Panels

The joint panels, shown in figure 3, contain both the expansion joint and the post-tensioning anchorage. The expansion joint is designed to accommodate the significant amount of horizontal slab movement during daily and seasonal temperature cycles while providing load transfer across the joint.

The pockets cast into the joint panels provide access to the post-tensioning anchors. The pockets are used to manually install the post-tensioning chuck and wedges around the post-tensioning strand after the strands have been fed through the ducts. The two elongated pockets in the joint panel are used for feeding temporary post-tensioning strands into the ducts. The temporary post-tensioning strands are used to pull the panels together incrementally as each one is installed. Grout inlets/vents are also cast into the joint panels just in front of the post-tensioning anchors to facilitate grouting the ducts after the strands are tensioned.

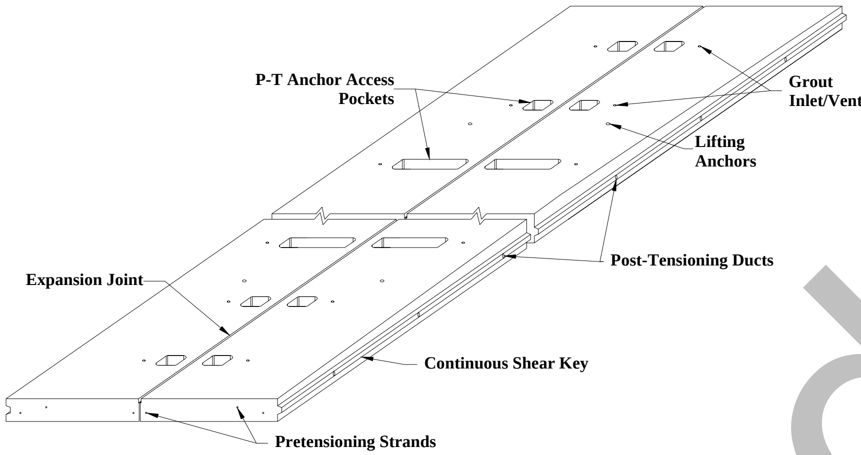

Figure 3. Illustration. Typical joint panel.

Central Stressing Panels

The central stressing panels, shown in figure 4, contain large pockets where post-tensioning is completed. The pockets, approximately 1.2 m (48 in.) long by 200 mm (8 in.) wide (full depth), are cast into the panels at every post-tensioning duct. The pockets are split into two separate panels, with the pocket for each duct alternating between panels, to prevent weakening of the panel, as shown in figure 1. After stressing has been completed, the pockets are filled with a fast-setting concrete or temporarily covered to allow traffic onto the pavement immediately. As in the joint panels, grout inlets/vents are cast into the panels on either side of the stressing pockets for grouting the strands after post-tensioning.

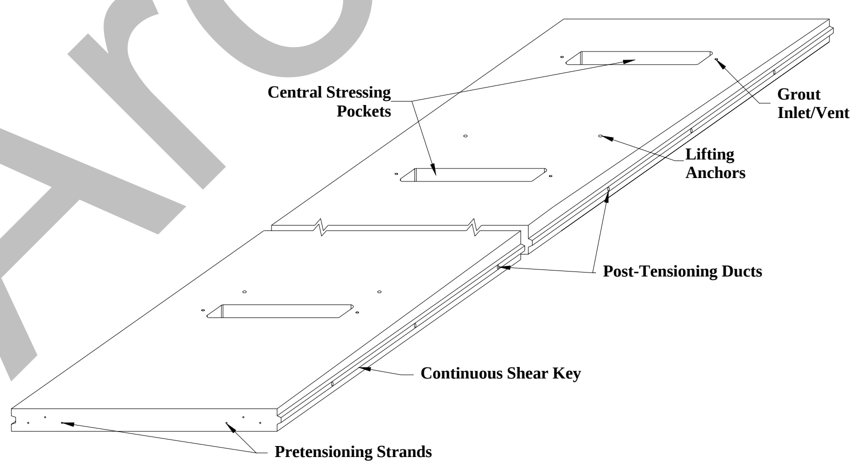

Figure 4. Illustration. Typical central stressing panel.

BASE PREPARATION

Base preparation consists of providing a smooth, flat surface to support the precast panels as well as providing a bond-breaking, friction-reducing material. As mentioned previously, the base preparation technique consists of using a hot-mix asphalt or cementitious material as a leveling course to support the precast panels. Care must be taken to ensure that high spots in the leveling course are minimized to prevent the panels from resting on the high spots, creating voids beneath the panels.

Over the leveling course, a friction-reducing material is required. The friction-reducing material prevents the precast pavement from bonding to the underlying base (leveling course), while also reducing the sliding friction between them. Because prestressed pavements are generally constructed as long, post-tensioned sections of pavement with no intermediate joints, a significant amount of horizontal expansion and contraction movement will take place. If this movement is restrained by friction between the pavement and leveling course, detrimental tensile stresses can develop in the slab, causing premature cracking and pavement failure.

A single layer of polyethylene sheeting (0.15 mm [0.006 in.] thickness, minimum) has proven to be an economical and effective friction-reducing material for prestressed pavements. This material was used successfully for the West, Texas, cast-in-place prestressed pavement(5) as well as for the Texas precast pavement pilot project.(2) Care must be taken during construction to prevent rips and tears and bunching of the sheeting between and beneath the panels.

POST-TENSIONING

As discussed previously, the purpose of longitudinal post-tensioning is to improve durability, reduce slab thickness, and provide load transfer between the precast panels. Longitudinal post-tensioning is applied through central stressing. Central stressing permits a more continuous pavement placement operation by eliminating the need for gap slabs to tension the strands and also reduces frictional losses during post-tensioning by reducing the effective length of the tendons. The post-tensioning strands coming into the central stressing pockets from either side of the slab are spliced in the pocket and tensioned using a “dogbone” or “ring anchor” coupler similar to that shown in figure 5. A monostrand stressing ram is used to tension the strands through the coupler. The post-tensioning sequence begins with the tendons at the middle of the slab moving outward, alternating to either side of the middle tendons.

Post-tensioning does not have to be completed before the pavement is opened to traffic. Post-tensioning can be completed during a subsequent construction operation if time constraints do not permit post-tensioning immediately after panel installation. Although post-tensioning is the primary mechanism for providing load transfer between panels, the keyways will provide some degree of load transfer prior to post-tensioning.

Post-tensioning ducts are located as close to mid-depth of the panel as possible. It is essential that the ducts line up between panels. It is also essential that the ducts are kept as straight as possible during panel fabrication to prevent post-tensioning losses due to “wobble.”

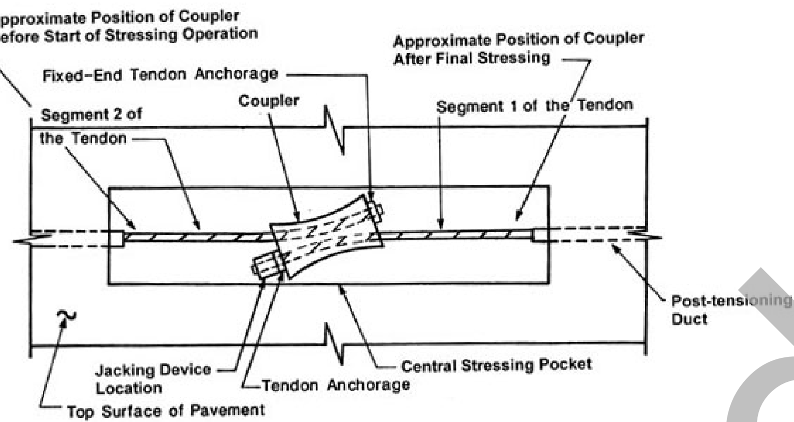

Figure 5. Illustration. Coupler used to join post-tensioning strands together in the stressing pockets.

GROUTING

Grouting consists of both post-tensioning tendon grouting and underslab void grouting. After the post-tensioning strands have been stressed and the pockets in the central stressing panels and joint panels are filled, the post-tensioning tendons are grouted. The primary purpose for grouting is to provide an extra layer of corrosion protection for the post-tensioning strands. This protection is particularly critical at the joints between precast panels where the post-tensioning duct is not continuous across the joint. However, post-tensioning also permanently bonds the strand to the pavement. This will prevent a loss of prestress if a strand is inadvertently cut or if a section of the pavement is cut out and replaced. Tendon grouting is accomplished by pumping grout into the ducts at inlets/vents located at the ends of each post-tensioning tendon in the joint panels and central stressing panels. Intermediate vents cast into the base panels also provide additional vent points if necessary. It is essential to ensure a tight seal around the post-tensioning ducts between panels to prevent grout leakage. A foam or neoprene gasket or thick epoxy can be used for this purpose.

Underslab grouting helps to ensure full support beneath the precast panels, filling any voids that may be present after panel installation. Underslab grouting is accomplished by pumping grout into grout channels cast into the bottom of the panels through inlets/vents at the surface of the panel. Grout is pumped into one inlet until it flows out of the vent at the other end of the grout channel. Care should be taken to seal/backfill the edges of the slab prior to underslab grouting to prevent grout leakage from beneath the slab.

Similarly to post-tensioning, grouting can be completed during a subsequent construction operation if time constraints do not permit grouting immediately following panel placement. Grouting should not be completed, however, until all post-tensioning tendons are stressed and the pockets in the joint panel and central stressing panels are patched.

CONSTRUCTION PROCESS

Figure 6 shows the overall construction process. Beginning with removal of the existing pavement, the base is then prepared by either smoothing the existing base or by placing a leveling course. The precast panels are then installed, followed by post-tensioning, patching of the stressing/access pockets, and grouting of the post-tensioning ducts. If needed, underslab grouting and diamond grinding can then be completed.

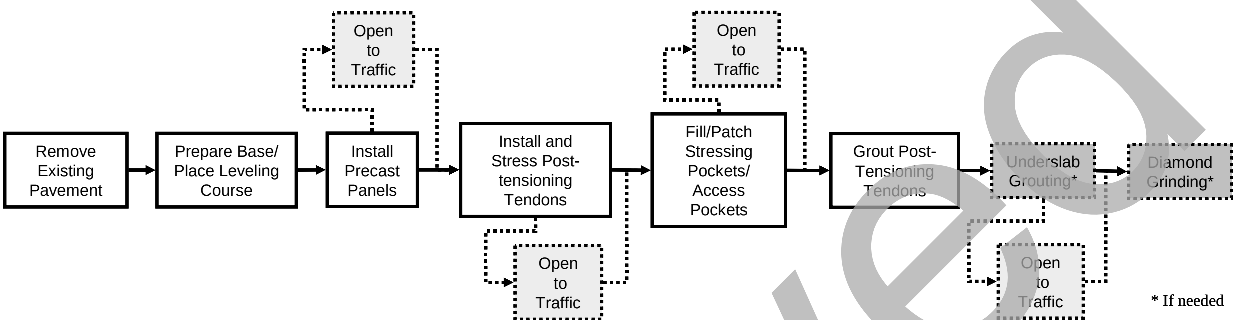

Figure 6. Illustration. Flowchart for the overall construction process.

As mentioned above, not all of these steps need to be completed before the pavement can be opened to traffic. If necessary, the pavement can be opened to traffic after placement of the precast panels; the keyways will provide load transfer between panels in the interim. The pavement can also be opened after post-tensioning; the stressing pockets and access pockets can be temporarily covered for traffic. The pavement can be opened prior to grouting also, provided that grouting is completed within a reasonable amount of time after post-tensioning and any water that may have entered the ducts is forced out prior to grouting. Likewise, the pavement can be opened to traffic prior to underslab grouting (if required) if the support beneath the pavement is deemed adequate to withstand traffic loading.

CHAPTER 3. CALIFORNIA DEMONSTRATION PROJECT

PROJECT SCOPE

The intent of the California precast pavement demonstration project was to further evaluate and refine the precast pavement concept developed through the FHWA feasibility study[1] and to familiarize Caltrans and its contractors with this new technology. The goal of this demonstration project was to construct the test section under realistic time constraints on the main lanes of a major freeway.

Location

The location selected by Caltrans for the demonstration project was a portion of a 5.1-km (3.2 mi) section of I-10, which was being widened from 8 lanes to 10 lanes to accommodate new high occupancy vehicle lanes. The location of the actual precast pavement test section is on eastbound I-10 just west of the Meeker Avenue overpass, approximately 3.2 km (2 mi) west of the San Gabriel River Freeway (I-605) in El Monte. The map in figure 7 shows the test section location.

Figure 7. Illustration. Location of the California precast pavement demonstration project.

Field Change

Due to the small-scale nature of this project, Caltrans issued a change order to incorporate precast pavement into the existing I-10 widening project. The existing plans at the test section location called for widening the existing pavement by 11.2 m (37 ft), including 8.2 m (27 ft) of main lanes and 3.0 m (10 ft) of shoulder, as shown in figure 8. The existing pavement design called for 250 mm (10 in.) of jointed plain concrete pavement (JPCP) over 150 mm (6 in.) of lean concrete base (LCB) over 220 mm (8.5 in.) of Class 3 aggregate base over the retaining wall embankment fill. For the change order, the 250 mm (10 in.) of JPCP was replaced with precast prestressed concrete pavement. It should be noted from figure 8 that the precast pavement was installed between fixed structures: the existing pavement and a new retaining/sound wall. This position required strict tolerances on the precast panels to ensure they would fit properly.

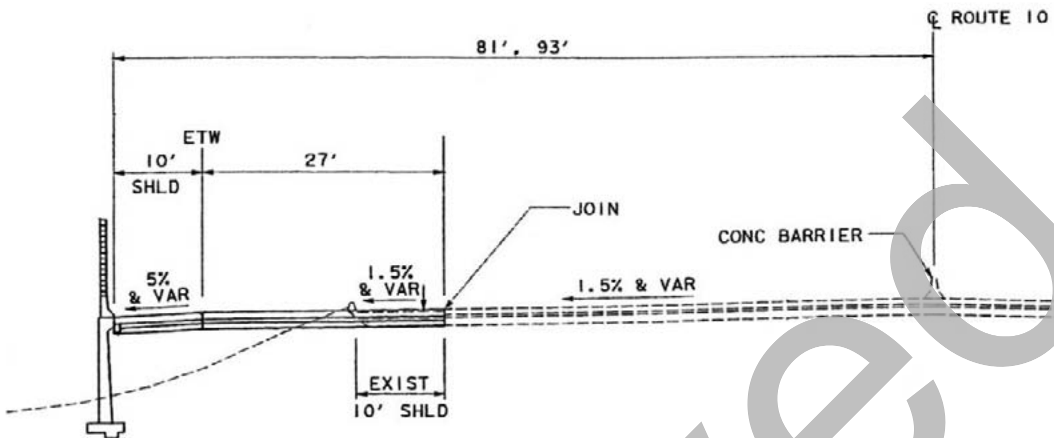

Figure 8. Illustration. Cross section of the original (proposed) pavement structure for widening I-10 (1 ft = 0.305 m).

PROJECT LAYOUT

The layout for the precast pavement demonstration project was dictated by the existing plans for the widening project. The goal of this demonstration project was simply to replace the original pavement design with precast concrete pavement—not to develop a new pavement structure and geometry for the precast pavement.

Geometry

A section with a "simple" geometry was selected for the precast pavement demonstration project. While pavements with complex geometry (i.e., superelevations) will eventually be encountered, the goal of this initial project in California was to work out the details of precast pavement fabrication and construction on a less complex project first. The selected test site had no change in vertical curvature and very minimal horizontal curvature. To ensure that the precast panels would fit flush against the existing pavement, this slight horizontal curvature was removed by sawcutting a straight edge onto the existing pavement over the length of the precast pavement test section.

While the longitudinal geometry of the test section was simple, the transverse geometry was more complex, with a change in cross slope from 1.5 percent in the traffic lanes to 5 percent in the shoulder, as shown in figure 8. It was therefore necessary to incorporate this change in cross slope into the precast panels, as will be discussed below.

Slab Width

As shown in figure 8, the original plans called for widening the existing pavement by 11.2 m (37 ft), with 8.2 m (27 ft) of main lanes (traffic lanes) and 3 m (10 ft) of shoulder. Two alternatives were presented to Caltrans:

1. Full-width panels: 11.2-m (37 ft) in length, with the change in cross slope cast into the surface of the panels.

2. Partial-width panels: 3-m (10 ft) shoulder panels installed at 5 percent cross slope and 8.2-m (27 ft) main lane panels installed at 1.5 percent cross slope; panels tied together with transverse post-tensioning.

Due to the additional expense of the second option, requiring more panels to be fabricated and the addition of transverse post-tensioning, Caltrans selected the first option. To cast panels with a "flat bottom," however, the cross slope of the LCB beneath the precast panels was changed to a uniform cross slope of 1.75 percent, rather than a range of 1.5 percent to 5 percent. While this resulted in an odd cross section for the precast panels, with 250-mm (10 in.) thickness at either end and 330-mm (13.1 in.) thickness at the edge of the traffic lanes, it greatly simplified the fabrication of the panels and installation over the LCB. Figure 9 shows the final cross section for the precast panels, which incorporate the cross slope change in the surface. Figure 10 shows one of the precast panels after fabrication.

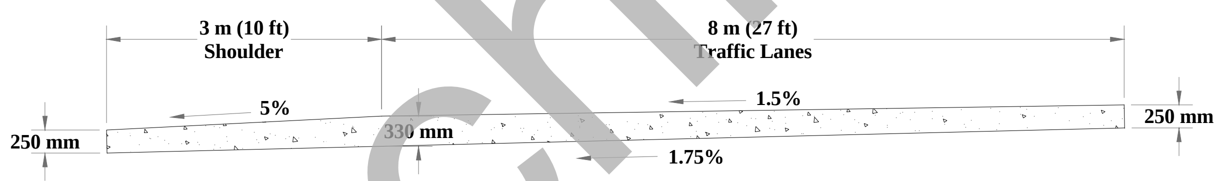

Figure 9. Illustration. Cross section of the precast panels, incorporating the change in cross slope into the surface of the panel.

Figure 10. Photo. Precast panel at the fabrication plant after removal from the forms.

Slab Length

The length of the demonstration project was contingent upon the funding available for construction. Based upon estimates from the fabricator and installation contractor, the total project length was limited to approximately 76 m (250 ft). The width of each precast panel was limited by transportation considerations. Although panel widths of 3 m (10 ft) or 3.7 m (12 ft) could have been fabricated, loads wider than 2.4 m (8 ft) required special permits for transportation. Therefore, panel width was limited to 2.4 m (8 ft). The total project length was 75.6 m (248 ft), or 31 panels. Although a single, post-tensioned slab could have been used for the full project length, it was decided to construct two slabs, each 37.8 m (124 ft) long. Each slab consisted of 12 base panels, 2 central stressing panels, and one-half of a joint panel at each end. The remaining half of each joint panel was tied into the existing pavement at either end of the test section.

PROJECT COORDINATION

Project coordination was a crucial aspect of this demonstration project. Because it was experimental in nature, Caltrans, the fabricator, and the installation contractor had no experience with this type of project. Prior experience by the design team with the Texas pilot project was very beneficial for the successful completion of the project. Flexibility on the part of Caltrans in meshing precast concrete and concrete pavement specifications was also very beneficial.

Several meetings were held during the development stage with the designers, Caltrans, precast fabricator, post-tensioning supplier, and installation contractor. Caltrans and the designers were involved throughout the fabrication and installation process. A pre-construction meeting involving all parties was held just before on-site installation began.

CHAPTER 4. DESIGN

DESIGN CONSIDERATIONS

There are several factors which must be considered for the design of a precast prestressed concrete pavement. A detailed discussion of these design considerations is beyond the scope of this report, but the primary factors are summarized below. For more information on design considerations, the reader is referred to CTR Reports 1517-1[(1)] and 1517-1-IMP.[(2)]

Traffic Loading

Traffic loading is a factor in the design of all pavements. Wheel loads induce a bending action in concrete pavements that generates tensile stresses in the bottom of the slab and compressive stresses in the top. Pavement designs are generally governed by the expected number of 80-kN (18 kip) equivalent single-axle loads (ESALs) that the pavement will experience over its intended design life. The repetition of these wheel loads over time will fatigue the concrete, eventually leading to pavement failure. Accurate prediction of the number of ESALs a pavement will experience will help ensure that the pavement is adequately designed.

As discussed previously, one of the advantages of prestressed concrete pavement is that the tensile stresses generated by wheel loads can be counteracted through prestressing. This allows for a thinner pavement slab to be designed with a fatigue life that is equivalent to a much thicker slab.

Temperature and Moisture Effects

Temperature has a significant effect on all concrete pavements, but the effect is particularly pronounced in prestressed concrete pavements. Daily and seasonal temperature cycles cause concrete pavements to expand, contract, and curl. Because prestressed pavement slabs are generally very long (between expansion joints), a significant amount of horizontal movement (expansion and contraction) can be expected at the ends of the slab. If this movement is restrained fully or partially by friction between the bottom of the pavement and the base material, stresses will develop in the slab. Because it is impossible to construct a pavement that is completely unrestrained, temperature effects will always generate stresses in prestressed concrete pavements that must be accounted for in design.

Slab curling also generates stresses in concrete pavements. Concrete pavement slabs rarely have a uniform temperature throughout the depth of the slab. The resulting temperature gradients cause the ends of the slab to curl upward or downward. As the sun heats the surface of the pavement in the morning, for example, it causes the ends slab to curl downward. However, because of the weight of the slab resisting this movement, tensile stresses also develop in the bottom of the slab. Due to the length of most prestressed slabs, this effect is even more pronounced and must be accounted for in design.

Moisture has a similar effect as temperature in that moisture gradients cause warping of the concrete slab. In general, moisture gradients are such that the bottom of the slab has a higher moisture content than the top of the slab due to the ease with which moisture can evaporate from the top surface. This moisture gradient will cause upward warping of the ends of the slab, resulting in tensile stresses in the top of the slab and compressive stresses in the bottom of the slab. In conventional cast-in-place pavements, this moisture gradient can become “built in” as the fresh concrete sets. This is one advantage of precast concrete pavements: so-called built-in temperature and moisture gradients are essentially eliminated.

Slab–Support Interaction

The interaction at the slab–support interface essentially consists of four components: friction, interlock, adhesion, and cohesion.(11) Friction is the result of the rubbing together of the two materials, interlock is the restraint caused by the macrotexture of the two surfaces in contact, adhesion is the attraction or “sticking” together of the two materials, and cohesion is related to the internal deformability capacity of the base layer. It is possible for the combined forces of these four components to be such that the restraint at the interface exceeds the internal strength of the base layer, resulting in failure of the base.(12)

It is essential to minimize restraint between the slab and base to reduce the magnitude of stresses that develop in the slab (and base) during horizontal slab movements. In general, compressive stresses will develop when the slab expands, while tensile stresses will develop when the slab contracts. The latter situation is more critical, as these tensile stresses may be additive to those tensile stresses caused by wheel loads and curling to such an extent that the slab may crack.(13) For the sake of simplicity in design, the stresses generated during slab expansion and contraction are generally assumed to be constant over the depth of the slab. Fortunately, precast panels tend to be fairly smooth on the bottom, which helps to reduce friction and interlock between the slab and base but still requires a friction-reducing material, such as polyethylene sheeting, to further reduce frictional restraint while also preventing adhesion.

Prestress Losses

Prestress losses are another important design consideration for prestressed concrete pavements. In general, losses of 15 to 20 percent of the applied prestressing force can be expected for a carefully constructed post-tensioned concrete pavement.(10) Some of the factors that contribute to prestress losses include the following:

- Elastic shortening of the concrete.
- Creep of the concrete (shrinkage is not a factor for precast pavements).
- Relaxation of the stressing tendons.
- Slippage of the stressing tendons in the anchorage.
- Friction between the stressing tendons and ducts.
- Horizontal restraint between the slab and support.

Fortunately, most of the factors are fairly well understood and can be estimated with reasonable accuracy. A more detailed discussion of each of these factors can be found elsewhere.(10)

DESIGN PROCEDURE

The design procedure for the California precast pavement is similar to that used for the design of the Georgetown, Texas, precast pavement(2) and will only be briefly summarized here. The design procedure is based on the principle of equivalent thickness design, meaning that the precast pavement is designed for an equivalent design life (fatigue life) to that of a thicker pavement. The first step in the design procedure is to determine the minimum prestressing required to achieve the equivalent thickness. This is accomplished through a layered elastic analysis of pavement stresses.

The second step is to adjust the prestressing requirements for the precast pavement slab to account for environmental effects. At this stage, the slab–support interaction, prestress losses, and temperature and moisture effects are accounted for. The result is the actual prestressing that must be applied to the pavement (during post-tensioning) to meet the equivalent thickness design requirements.

The final step in the design procedure is to check maximum horizontal slab movement (expansion and contraction) to ensure that the maximum permissible expansion joint width is not exceeded. At this stage, the slab length (between expansion joints) can be adjusted if necessary.

DESIGN FOR FATIGUE

The first step in the design procedure is to determine the required prestressing for a precast pavement with an equivalent fatigue life (design life) to that of a thicker conventional cast-in-place pavement. The basic premise of equivalent fatigue design is that the same bottom fiber tensile stress in slabs of different thickness will produce pavements with the same fatigue life.

Equivalent Thickness

The original pavement design for widening I-10 in El Monte called for a 250-mm (10 in.) JPCP. While it would have been possible to design a 150-mm (6 in.) or 200-mm (8 in.) precast pavement with an equivalent thickness to the 250-mm (10 in.) JPCP, for constructability reasons it was decided that a 250-mm (10 in.) precast concrete pavement would be constructed. However, for the sake of comparison, the precast concrete pavement was actually designed to be equivalent to a JPCP that is 355 mm (14 in.) thick.

Support Structure and Prestress Requirements

The support structure for the El Monte precast pavement consists of 150 mm (6 in.) of LCB over 220 mm (8.5 in.) of aggregate base (Class 3) over the existing subgrade, as shown in figure 11. This support structure was used to determine the tensile stresses in the bottom of the equivalent 355-mm (14 in.) JPCP under the loading shown in figure 11. This loading condition represents the typical theoretical wheel load from dual wheels on a single-axle truck. The properties of the different layers of the support structure were estimated based upon Caltrans *Standard Specifications*.[14] The resilient modulus values for the aggregate base and subgrade were determined from the R-value requirements given in the Special Provisions for this project. The minimum R-value given was 50, corresponding to a resilient modulus of 75.8 MPa (11,000 lbf/in²).

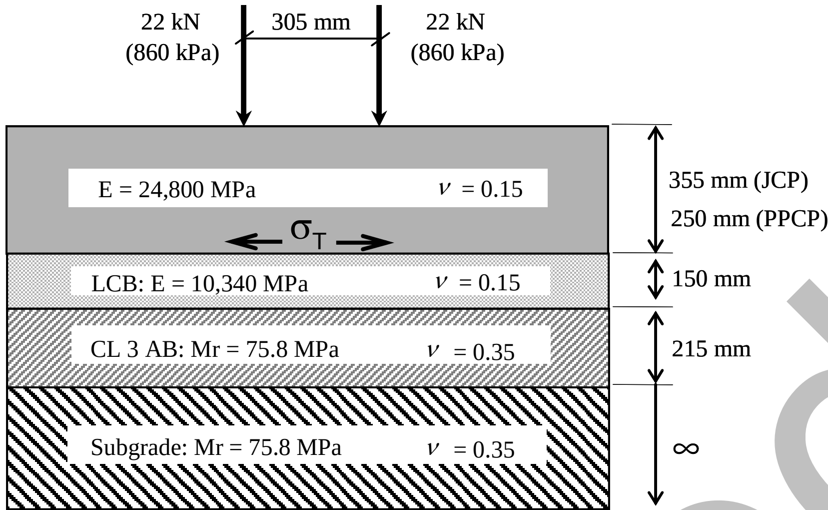

Figure 11. Illustration. Support structure for equivalent pavement design (subgrade, class 3 aggregate base, and lean concrete base), topped by either jointed concrete pavement or precast prestressed concrete pavement.

Using the support structure and loading condition shown above, the tensile stress (σ_T) at the bottom of the equivalent JPCP control pavement was determined using layered elastic analysis. The computer program BISAR (Bitumen Structures Analysis in Roads)(15) was used for the layered elastic analysis. Stresses were computed beneath the loads and between the loads to determine the worst condition.

Table 1 summarizes the stresses for the two pavement thicknesses (250 mm and 355 mm). The highest tensile stresses occurred at the midpoint between loads, but the largest difference in bottom fiber stress occurred beneath the load. The difference in bottom fiber tensile stress of 92 kPa (13.3 lbf/in²) is the magnitude of prestressing required in the 250-mm (10 in.) precast concrete pavement to achieve an equivalent JPCP thickness of 355 mm (14 in.).

Table 1. Bottom Fiber Tensile Stress at the Bottom of the 250-mm (10 in.) and 355-mm (14 in.) Equivalent Pavements

	Bottom Tensile Stress, σ_T, kPa	
Pavement Thickness	**Beneath Load**	**Midpoint Between Loads**
250 mm	329	337.8
355 mm	237	246.8
Difference	**92**	91

1 kPa = 0.145 lbf/in²

Fatigue Life Implications

The benefit of a thicker equivalent pavement will be realized through the increase in fatigue life of the pavement. Fatigue life can be predicted using pavement fatigue equations such as that given in figure 12, developed by Taute et al.[16] This fatigue equation predicts the number of 80-kN (18 kip) ESAL repetitions to serviceability failure.

$$N_{18} = 46{,}000\left(\frac{f}{\sigma_T}\right)^{3.00}$$

Figure 12. Equation. Fatigue life prediction based on number of 80-kN (18 kip) ESALs experienced by the pavement.

where: N_{18} = Number of 80-kN (18 kip) ESALs to serviceability failure

f = Concrete flexural strength (MPa)

σ_T = Bottom fiber tensile stress from wheel loading (MPa)

Using the equation given in figure 12 and the stresses computed in table 1 (worst case: midpoint between loads), the predicted fatigue life, in terms of 80-kN (18 kip) ESALs, of the original 250-mm (10 in.) pavement and the "equivalent" 355 mm (14 in.) pavement were calculated as shown in table 2. Concrete flexural strength was calculated from the American Concrete Institute's modulus of rupture equation for 41.4 MPa (6,000 lbf/in²) compressive strength concrete to be 4 MPa (580 lbf/in²). It is evident from this analysis that the reduction in bottom fiber tensile stress of only 27 percent will significantly increase the fatigue life of the pavement.

Table 2. Total Predicted ESALs to Serviceability Failure for 250-mm (10 in.) Versus 355-mm (14 in.) JPCP

Pavement Thickness	**Bottom Fiber Tensile Stress (kPa)**	**Predicted ESALs to Serviceability Failure**
250-mm JPCP	338	76,300,000
250-mm prestressed pavement (equivalent to 355-mm JPCP)	247	195,600,000
1 kPa = 0.145 lbf/in²		

To calculate the effect of this increase in fatigue life in terms of years, traffic data for I-10 in El Monte were obtained from Caltrans and used to predict the number of ESALs in the design lane, assuming a constant growth rate of 2 percent. As table 3 shows, the 250-mm (10 in.) pavement is adequate for a 30-year design life. However, a 250-mm (10 in.) precast, prestressed pavement (equivalent to a 355-mm JPCP), will have a design life just over 57 years. It should be noted that if a 3 percent growth rate were used, the design life of the. 250-mm (10 in) pavement would be only 27 years, while that of the 350-mm (14 in.) pavement would be 48 years.

Table 3. Total Predicted ESALs for the Design Lane on I-10

Prediction	Design Period (years) 30	57
Growth factor (%)	2	2
ESAL volume, 1st year	4,579,451	4,579,451
% ESALs in design lane	40	40
Total predicted ESALs	185,779,515	478,962,126
Total design lane ESALs	74,311,806	191,584,850

Although long-term traffic volumes are difficult to predict with a great deal of certainty, and actual pavement performance is also difficult to predict due to the many variables that affect pavement performance, this analysis demonstrates the relative benefit of prestressed pavement in terms of design life.

DESIGN FOR ENVIRONMENTAL EFFECTS

In the previous section, the fatigue design revealed that a prestress force of 92 kPa (13.3 lbf/in²) was required to produce a 250-mm (10 in.) precast, prestressed pavement with an design life equivalent to a 350-mm (14 in.) JPCP pavement. Therefore, at every point along the length of the slab, a minimum compressive stress of 92 kPa (13.3 lbf/in²) must be maintained under varying environmental conditions to meet the fatigue requirements. This is the residual pavement stress caused by environmental conditions (i.e., slab expansion and contraction, curling, etc.) and does not include wheel loads, which were accounted for in the fatigue design.

PSCP2 Design Program

A very powerful tool for analyzing the effects of environmental conditions on precast, prestressed pavement is the computer program PSCP2. This program was originally developed at CTR as a design and analysis tool for cast-in-place post-tensioned pavements, but can also be used to analyze precast, prestressed pavements by adjusting the inputs to the program. A full description of the PSCP2 program is beyond the scope of this report, but the reader is referred to Mandel et al.[5] and Merritt et al.[1] for a more detailed discussion of the PSCP2 program and its application to precast pavement design.

To summarize the functionality of the PSCP2, the program calculates horizontal slab movement, frictional restraint stresses, prestress losses, curling stresses, and curling movement at a specified number of points along the length of the slab for any number of days (or years) after construction. The user inputs slab geometry, material properties (concrete and prestressing steel), slab–support frictional characteristics, prestress application, and expected slab temperatures into the program. After running the program, the user can evaluate total slab stress and movement at any point in the pavement's design life.

PSCP2 Analysis

The inputs used for the PSCP2 analysis for the California precast pavement are summarized below.

Geometric Properties

- Slab thickness: 250 mm (10 in.).
- Slab width: 11.2 m (37 ft).
- Slab length: 38.1 m (125 ft) (76.2 m was also evaluated).

Concrete Properties

- Compressive strength: 41.4 MPa (6,000 lbf/in²).
- Ultimate shrinkage strain: 0.00019 mm/mm (for precast concrete).
- Coefficient of thermal expansion: 9×10^{-6} mm/mm/ºC (5×10^{-6} in/in/ºF)–from *PCI Design Handbook*(3) for granitic coarse aggregate.
- Unit weight: 2,322 kg/m³ (145 lb/ft^3).
- Poisson's ratio: 0.15.
- Creep coefficient: 2.1.

Post-Tensioning Steel Properties

- Strand diameter: 15 mm (0.6 in.).
- Cross-sectional area: 140 mm² (0.217 in^2).
- Yield strength: 1,675 MPa (243,000 lbf/in²).
- Elastic modulus: 19.7×10^4 MPa (28.5×10^6 lbf/in²).
- Thermal coefficient: 12.6×10^{-6} mm/mm/ºC (7×10^{-6} in/in/ºF).

Prestress

The pavement was assumed to be post-tensioned in the longitudinal direction, in one stage, 6 hours after panel installation, although the timing of post-tensioning is irrelevant for this analysis. The strands were assumed to be stressed to 80 percent (1,490 MPa) of their ultimate strength. Strand spacing was initially set at 0.9 m (36 in.) as this is deemed to be the maximum permissible strand spacing for a prestressed pavement with monostrand tendons. This spacing was adjusted during the PSCP2 analysis as needed to meet the fatigue requirements.

Slab-Support Interaction

A k-value of 135.7 kPa/mm (500 lbf/in²/in) was specified for the slab support, although this input has essentially no effect on the analysis. The friction-displacement relationship was assumed to be a linear relationship with a maximum coefficient of friction of 0.2 and corresponding displacement of 0.5 mm (0.02 in.) at sliding.

Analysis Period

The pavement was analyzed at 90 days, 1 year, 5 years, and 30 years to determine the worst-case condition for slab movement and stress. The 30-year analysis proved to be the controlling design for both slab movement and stresses.

Temperature

Temperature data were specified for the first 24-hour period after placement and for a 24-hour period at each of the analysis periods (90 days, 1 year, 5 years, 30 years). Both mid-slab temperature and top–bottom slab temperature differential were specified. Ambient temperature data were obtained from the National Weather Service for El Monte, California. Daily temperature distributions for a typical summer day and a typical winter day were generated from the temperature history for El Monte. Concrete temperatures were estimated from ambient temperature using the empirical formula shown in figure 13.[17] Top–bottom temperature differentials were estimated from previous prestressed pavement measurements.[18] Table 4 summarizes the temperatures used for the PSCP2 design.

$$T_C = 20.2 + 0.758 T_A$$

Figure 13. Equation. Prediction of concrete temperature based on ambient temperature.

where: T_C = concrete temperature (^{o}F)

T_A = ambient temperature (^{o}F)

Table 4. Temperature Data for a 24-Hour Period Used for the PSCP2 Analysis of the California Demonstration Project

Time of Day	Summer Temperatures (°C)			Winter Temperatures (°C)		
	Ambient Temperature	Concrete Temperature	Top–Bottom Differential	Ambient Temperature	Concrete Temperature	Top–Bottom Differential
12:00 a.m.	24.3	25.3	-6.0	5.3	10.9	-6.0
2:00 a.m.	23.8	24.9	-6.3	4.7	10.5	-6.3
4:00 a.m.	23.5	24.7	-6.3	4.3	10.2	-6.3
6:00 a.m.	24.1	25.2	-6.1	3.9	9.9	-6.1
8:00 a.m.	27.0	27.4	-4.2	4.9	10.6	-4.2
10:00 a.m.	31.2	30.6	2.1	7.9	12.9	2.1
12:00 p.m.	33.2	32.1	5.2	9.9	14.4	5.2
2:00 p.m.	33.0	31.9	6.3	10.5	14.8	6.3
4:00 p.m.	31.6	30.9	4.0	9.5	14.1	4.0
6:00 p.m.	28.9	28.9	-0.4	7.8	12.8	-0.4
8:00 p.m.	25.6	26.3	-3.3	6.7	12.0	-3.3
10:00 p.m.	24.8	25.7	-4.4	5.8	11.3	-4.4

Longitudinal Prestress Requirements

Previous precast pavement analyses have revealed that the controlling case for design of longitudinal prestress occurs when the pavement is placed under summer temperature conditions and evaluated under winter conditions. In other words, the worst-case stress condition will occur when a pavement constructed under summer climatic conditions is evaluated under winter climatic conditions. Therefore, the summer/winter PSCP2 analysis was considered the controlling design case.

When analyzing longitudinal prestress, the spacing of the post-tensioning strands is adjusted in a "trial and error" analysis until the longitudinal prestress requirements are met. For this project, 92 kPa (13.3 lbf/in²) was the minimum longitudinal prestress requirement. As mentioned previously, a "rule of thumb" maximum strand spacing for post-tensioned pavements is 0.9 m (36 in.). Based on the PSCP2 analysis for the summer/winter condition and 0.9 m (36 in.) strand spacing, the maximum bottom fiber stress was calculated as 685 kPa (99.3 lbf/in²) (compressive), which is well above the required 92 kPa (13.3 lbf/in²) (compressive). Therefore, longitudinal strand spacing was specified as 0.9 m (36 in.).

Transverse Prestress Requirements

While transverse prestressing is essential for the long-term performance of prestressed pavements, transverse prestress requirements are generally governed by handling stresses for precast panels. Transverse prestressing is specified such that no cracking will occur during handling of the panels. As a "rule of thumb" from the Texas pilot project, a minimum of 1.4 MPa (200 lbf/in²) prestressing is recommended to counteract both handling stresses and long-term *in situ* pavement stresses. To check the adequacy of 1.4 MPa (200 lbf/in²) for handling, stresses were calculated in accordance with Section 5.2 of the *PCI Design Handbook*.[3] Removal of the panels from the forms is generally the critical design case, as the panels have not reached their full 28-day design strength. The modulus of rupture at removal was computed to be 2.18 MPa (316 lbf/in²) using the following equation from the *PCI Design Handbook* (Eq. 5.2.1):

$$f'_r = K \lambda \sqrt{f'_{ci}}$$

Figure 14. Equation. Equation for estimation of modulus of rupture from compressive strength.

where: f'_r = Modulus of rupture.

K = Constant prescribed by ACI as 7.5, reduced by a factor of safety of 1.5 to 5 as per *PCI Design Handbook* recommendation.

λ = 1.0 for normal-weight concrete.

f'_{ci} = Concrete compressive strength at release of prestress, 27.6 MPa, (4,000 lbf/in²) for the California demonstration project.

Lifting stresses were computed using moments calculated for a two-point pick-up (four lifting points located approximately 0.2L from each edge of the panel) as recommended by the *PCI*

Design Handbook. An equivalent static-load multiplier of 1.3 was added to the unit weight of the concrete to account for stripping and dynamic forces as the panels are removed from the forms. Based upon these design parameters, the maximum lifting stress for 11.2 m x 2.4 m x 290 mm (average thickness) panels was computed to be 2.2 MPa (320 lbf/in^2). This stress is slightly higher than the modulus of rupture (2.18 MPa [316 lbf/in^2]), but is counteracted by 1.4 MPa (200 lbf/in^2) of compression from pretensioning. Assuming that 13-mm (0.5 in.), 1,860 MPa (270,000 lbf/in^2) prestressing strand (stressed to 80 percent of ultimate strength) will be used, six strands were required to achieve the 1.4 MPa (200 lbf/in^2) transverse prestress requirement.

Expansion Joint Limitations

The last step in the design process is to check horizontal slab movement under varying temperature conditions to ensure that the expansion joints limitations are satisfied. Expansion joint widths should be checked to ensure they will never fully close and never open excessively. In the case of the California demonstration project, the maximum expansion joint width was limited to 25 mm (1 in.).

The PSCP2 computer program was used to predict long-term slab movement for estimation of expansion joint widths. Horizontal slab movement was examined for two worst-case scenarios: placement of the pavement under summer climatic conditions and evaluation in winter (summer/winter), and placement under winter climatic conditions and evaluation in the summer (winter/summer). The summer/winter condition generally governs for determining maximum expansion joint width (i.e., how wide the joint will open over time), while the winter/summer condition generally governs for determining the minimum required expansion joint width (i.e., the minimum width that must be provided to allow for slab expansion). Based on the PSCP2 analysis, the maximum amount of slab contraction predicted was 7.5 mm (0.3 in.) for the 38.1-m (125 ft) slab length. For the expansion joints, this translates to 15 mm (0.6 in.) of total movement (7.5 mm [0.3 in.] on either side of the expansion joint). The maximum amount of slab expansion predicted was 2.5 mm (0.1 in.), or 5 mm (0.2 in.) of total expansion joint movement (closure). It should be noted that these are long-term predictions, with the maximum (15-mm) joint contraction movement anticipated at 30 years after construction.

While these predicted slab movements were within the expansion joint limitations (maximum width of 25 mm), it was necessary to develop recommendations for the width of the expansion joints during construction. The expansion joint widths (at the level of the dowels) after completion of post-tensioning were given in the project plans to ensure the joints will never fully close and never open more than 25 mm (1 in.). The contractor adjusted the width of the expansion joints to meet these requirements after post-tensioning. Table 5 shows the expansion joint width requirements. The width was specified based upon the approximate ambient temperature after post-tensioning. It should be noted that these recommendations were based upon the climatic conditions and climatic history in El Monte, California. Different regions of the country will have different requirements.

Table 5. Required Expansion Joint Width After Post-Tensioning
(from the project plans)

Approximate Ambient Temperature (°C)	**Joint Width (at height of the dowels)**
T < 16 º	13 mm
16º ≥ T ≤ 32º	6 mm
T > 32º	0 mm

1 mm = 0.039 in.

CHAPTER 5. PANEL FABRICATION

PROCEDURE

The precast panels for the California demonstration project were fabricated by Pomeroy Corporation of Perris, California. The fabrication plant was located approximately 97 km (60 mi.) from the test site in El Monte. Because of the unique shape of the side forms (change in cross slope cast into the panel surface) and the small number of panels to cast, it was cost-prohibitive for the precast fabricator to purchase side forms for more than two panels, limiting production to two panels per casting. In total, 31 panels were cast, including 3 joint panels, 4 central stressing panels, and 24 base panels.

The panels were fabricated on a long-line casting bed, with two panels cast end to end. The pretensioning strands extended through both panels, anchored at permanent stressing abutments at either end of the casting bed. After each pair of panels was cast and had reached the specified release strength of 27.6 MPa (4,000 lbf/in^2), the pretensioning strands were de-tensioned and cut at the ends of the panels and the panels were removed from the casting bed to a storage area in preparation for shipment. Depending on set-up time for the casting bed, which varied by panel type, one set of panels was generally cast every other day.

TOLERANCES

Because the panels were not match-cast, tolerances were particularly critical to ensure that adjoining panels would fit together and provide a satisfactory riding surface. Table 6 summarizes the tolerances for the precast panels, as specified in the project drawings. Based upon previous experience with the Texas pilot project, a “squareness” tolerance was added to ensure that a “curve” was not built into the pavement by panels that were out-of-square. The position and straightness of the post-tensioning ducts were also critical to ensuring that the ducts would line up between adjacent panels and that wobble would be minimal over the length of the entire duct.

It should be noted that table 6 is not an exhaustive list of tolerances for the precast panels. Tolerances such as clear cover for reinforcement were dictated by Caltrans *Standard Specifications* for Portland Cement Concrete Pavement (section 40), Prestressing Concrete (section 50), Concrete Structures (section 51), Reinforcement (section 52), Portland Cement Concrete (section 90), and others.[14]

Tolerances were checked at the precast plant by Caltrans inspectors, and any dimensions out of tolerance were generally handled on a case-by-case basis. The precast fabricator indicated that there were no problems with meeting the required tolerances.

Table 6. Table of Panel Tolerances From the Project Plans

Measurement	Tolerance
Length (longitudinal to centerline)	+/- 6 mm (1/4 in.)
Width (transverse to centerline)	+/- 6 mm (1/4 in.)
Nominal thickness	+/- 1.5 mm (1/16 in.)
Horizontal alignment (upon release of prestress)—deviation from straightness of mating edge of panels	+/- 6 mm (1/4 in.)
Deviation of ends from shop plan dimension (horizontal skew)	+/- 6 mm (1/4 in.)
Position of strands (horizontal and vertical)	+/- 3 mm (1/8 in.) vertical +/- 3 mm (1/8 in.) horizontal
Position of post-tensioning ducts at transverse joints	+/- 3 mm (1/8 in.) vertical +/- 3 mm (1/8 in.) horizontal
Straightness of post-tensioning ducts	+/- 6 mm (1/4 in.)
Squareness (corner–corner measurement)	+/- 3 mm (1/8 in.)
Position of lifting anchors	+/- 76 mm (3 in.)

PANEL DETAILS

The majority of the details for the different panel types were essentially the same with the exception of the expansion joints in the joint panels and stressing pockets in the central stressing panels, as described below.

Keyways

As discussed previously, the primary purpose of the keyways along adjoining panel edges was to ensure vertical alignment between panels as they were assembled and to provide temporary load transfer until post-tensioning was completed. Because the panels were not match-cast, keyway dimensions were specified such that there would be a slight amount of "play" in the keyways. As figure 15 shows, the nose of the "male" keyway was tapered slightly more than the "female" keyway to prevent a wedging action in the keyway as the panels were assembled. The nose of the male keyway was also shortened to ensure that the vertical faces above and below the keyway would contact and the nose of the male keyway would not "bottom out." Figure 16 shows the end of a keyway for two panels assembled at the fabrication plant (without post-tensioning).

A tapered opening was cast into the panels at the ends of the post-tensioning ducts, as shown in figure 15. The purpose of this taper was to help guide the post-tensioning strands into the ducts as they passed between panels. Additionally, a recess was cast into the female keyways at each post-tensioning duct to receive a foam gasket. The foam gasket helped to seal the post-tensioning ducts at the joints between panels.

Although the thickness of the panels varied from 250 mm (10 in.) at the ends to 330 mm (13.1 in.) at 3 m (10 ft) from one end, the keyways remained parallel to the bottom of the panel, resulting in a variable-depth vertical face above the keyways (figures 10, 15). Likewise, the vertical center of the post-tensioning ducts was maintained parallel to the bottom of the panel, at the center of the keyways. This helped to greatly simplify the formwork and assembly of the panels.

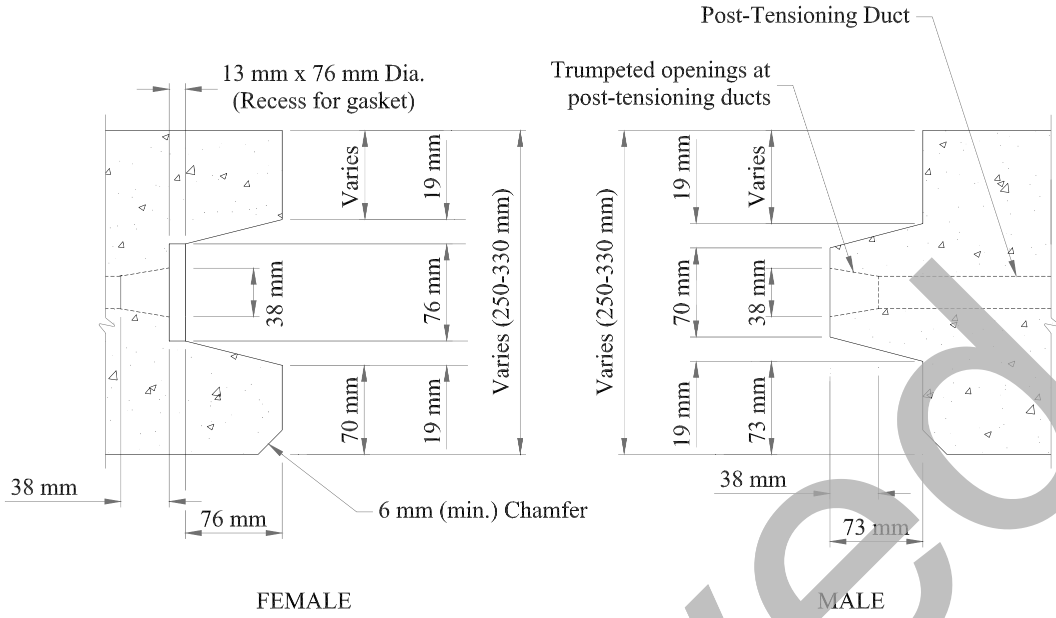

Figure 15. Illustration. Keyway dimensions for the precast panels.

Figure 16. Photo. Keyway of two assembled panels at the fabrication plant.

Grout Channels

Grout channels were cast into the bottom of each panel to allow for underslab grouting after completion of post-tensioning. Two “half-round” channels with a 13-mm (1/2 in.) radius were cast into the bottom of each panel parallel to the long axis of the panel. Each channel was located approximately 0.5 m (20 in.) from the edge of the panel and was stopped 0.3 m (1 ft) from the ends of the panel. Grout inlets/ports for the channels were spaced at 2.4 m (8 ft) along the length of the channel, extending vertically to the top surface of the panel.

Base Panels

The base panels were the most basic of the three panel types. Six pretensioning strands, 13 mm (0.5 in.) in diameter, were spaced evenly across the 2.4-m (8 ft) width of the panel, parallel to the bottom of the panel, alternating just above and just below the post-tensioning ducts. Twelve galvanized steel post-tensioning ducts, 25 mm (1 in.) in diameter (inside), were spaced at 1 m (3 ft) on center across the length of the panel, beginning 0.6 m (2 ft) from each end. Non-prestressed reinforcement was minimal, with 13-mm (0.5 in.) (ASTM A706) deformed bars around the top and bottom perimeter of each panel. A minimum of 50 mm (2 in.) of concrete cover was provided for all of the reinforcement. Lifting anchors were located approximately 0.2L from the edges of the panels. Intermediate grout vents/inlets for the post-tensioning ducts were cast into approximately every fourth base panel.

Central Stressing Panels

The central stressing panels were similar to the base panels with the addition of the stressing pockets for post-tensioning, as described in chapter 2. The pockets were 200 mm (8 in.) wide by 1.2 m (4 ft) long, through the panel, large enough to accommodate a monostrand stressing ram and the “movement” of the strand coupler from elongation of the post-tensioning strands.

As shown in the panel assembly diagram in chapter 2, the stressing pockets were divided into two panels to prevent weakening of the panels from having too many pockets. The pockets for adjacent tendons were alternated between the two panels. The corners of each of the pockets were rounded to reduce stress concentrations and associated cracking from the corners of the pockets. Tendon grout inlets/vents were located on either side of the stressing pockets.

The six pretensioning strands, 13 mm (0.5 in.) in diameter, were spaced evenly on either side of the stressing pockets. Non-prestressed reinforcement (A706, 13 mm) was provided around the top and bottom perimeter of the panels, similar to the base panels. Two additional bars were provided through the bottom of each panel to help hold the concrete for the pockets in place, and 90-degree angle bars were provided at the ends of each stressing pocket to arrest any cracks that may propagate from the corners of the pockets. Figure 17 shows a central stressing panel at the fabrication plant during storage.

Figure 17. Photo. Typical central stressing panel at the fabrication plant prior to shipment.

Joint Panels

The joint panels contained the expansion joints for "absorbing" the horizontal expansion and contraction movements of the post-tensioned slabs. The expansion joint detail is shown in figure 18. While an armored expansion joint detail was used for the Texas pilot project,(2) a plain dowelled joint was specified for the California project due primarily to the fact that diamond grinding was anticipated, and armored expansion joints cannot be ground very easily. Fortunately, the anticipated expansion joint movement (< 25 mm) is much less than that of the Texas project, eliminating the need for such a robust armored joint.

As figure 18 shows, stirrups (A706, 13 mm) spaced approximately every 100 to 150 mm (4 to 6 in.), were provided to transfer the prestress from the face of the post-tensioning anchors back to the expansion joint, ensuring the entire joint panel is prestressed. Non-prestressed reinforcement was also provided in front of the post-tensioning anchors and at the corners of the stirrups. Stainless-steel-clad dowel bars and expansion sleeves were spaced at 0.3 m (1 ft) on center, parallel to the bottom of the panel, 130 mm (5 in.) from the bottom. The pockets behind the post-tensioning anchors were provided for manual fitting of the wedges around the strands. The majority of the pockets were 150 mm (6 in.) by 200 mm (8 in.), through the panel, but as mentioned previously, two larger pockets (150 mm by 0.6 m), through the panel) were provided for feeding the temporary post-tensioning strands into the ducts during panel assembly (figures 3 and 19).

Two of the six pretensioning strands (13 mm [0.5 in.] in diameter) were located behind the anchor access pockets (between the pockets and expansion joint), and the other four were located in front of either of the post-tensioning anchors. Additional non-prestressed reinforcement was provided around the top and bottom perimeter of each "half" of the joint panel.

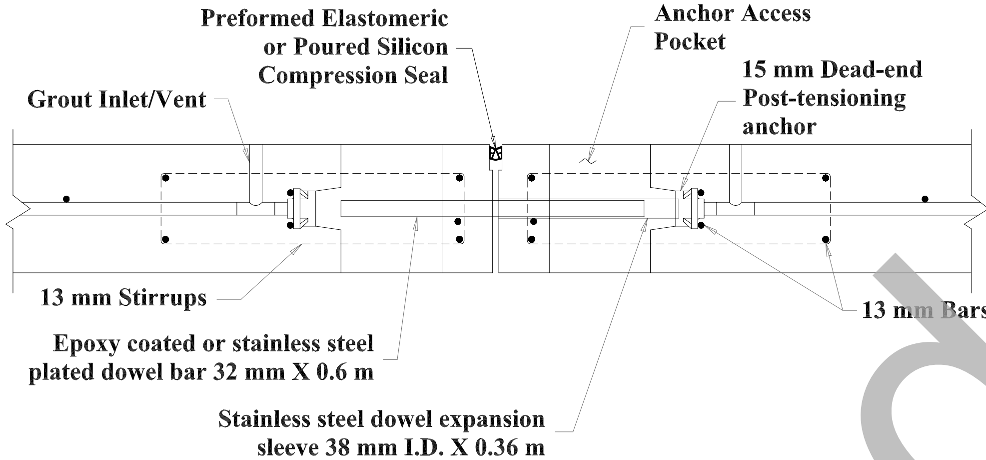

Figure 18. Illustration. Expansion joint detail for the California pilot project.

For the two joint panels abutting the new cast-in-place pavement at either end of the precast pavement test section, slots were cast into the joint panel to receive tie bars extending from the existing pavement. The tie-bar slots were cast into the bottom half of the joint panels at 0.3 m (1 ft) on center, with grout ports extending to the surface of the panel for grouting the tie-bars in place. For the non-post-tensioned half of the joint panels abutting the existing pavement, 13-mm (0.5 in.) A706 bars were provided at mid-depth at 0.3 m (1 ft) on center over the length of the panel.

Joint panels were fabricated by casting one half at a time. Dowels were left protruding from the face of the expansion joint, and a bond-breaker was applied to the hardened concrete prior to casting the other half of the joint panel. This helped to ensure a clean joint between the two halves of each joint panel. Figure 19 shows one of the joint panels at the precast plant prior to shipment to the installation site. The anchor access pockets, grout vents, and lifting devices are visible in this picture. The strongbacks used to hold the two halves of the panel together during handling are also visible.

Figure 19. Photo. Typical joint panel at the fabrication plant prior to shipment.

MIXTURE DESIGN

To remain productive, precast fabricators must continually keep their casting beds in use. This usually requires a concrete mixture design that allows them to remove a product from the forms the day after casting. Fortunately, experience has led to mixture designs that will meet this requirement without compromising quality or durability. For the precast pavement panels, a mixture was needed that would not only give the precast fabricator the required release strength the day after casting, but would also meet the finishablity requirements for a pavement surface.

The mixture design used for the panels consisted of 340 kg (8 sacks) of Type II cement (Prestress/Mojave) with 15 percent Type F fly ash replacement, a water-to-cementitious materials ratio of 0.37, superplasticizer, fine aggregate, and 13-mm (1/2 in.) maximum size granitic coarse aggregate. The mixture had a design release strength of 27.6 MPa (4,000 lbf/in^2) and 28-day strength of 41.4 MPa (6,000 lbf/in^2). The maximum pouring slump was 100 mm (4 in.), and unit weight was 2,355 kg/m^3 (145 lb/ft^3). The precast fabricator indicated that there were no problems with this mixture design for pavement panels. Figure 20 shows placement of concrete for one of the base panels at the fabrication plant.

Figure 20. Photo. Placement of concrete for a base panel.

FINISHING AND CURING

Surface finishing was a critical step as the finish greatly affects the ride quality of the finished pavement. Finishing the precast panels was somewhat challenging for two reasons. First, the change in cross slope cast into the panels formed a “peak” in the surface of the panel where the thickness was 330 mm (13.1 in.) versus 250 mm (10 in.) at the ends. Because fresh concrete tends to flow downhill, the mixture needed to be stiff enough to prevent sloughing at the high point on the surface. Secondly, grout vents and lifting anchor supports were protruding from the surface, forcing the fabricator to hand finish the surface of the panels around these embedments. After hand finishing the panel surface, an Astroturf drag texture was applied. Figure 21 shows workers screeding and hand finishing the surface of a joint panel around the grout ports protruding from the surface of the panel.

Following the finishing process, the panels were covered with tarps and steam cured in accordance with Caltrans *Standard Specifications*, section 90-7.04.(14) Steam curing was usually applied for 12 hours or until the panels reached the necessary release strength. Upon release of prestressing, the panels were removed from the forms and stored on site.

Figure 21. Photo. Screeding the surface of a joint panel.

HANDLING AND STORAGE

One of the recommendations from the Texas pilot project was to use lifting anchors that would only leave a small hole or void to be patched.[2] Based on this recommendation, screw-type lifting anchors were used for this project. Although screw-type lifting anchors are slower for attaching lifting lines, the recesses left by these anchors are much smaller and easier to patch, and the pavement can be opened to traffic prior to patching them.

An important consideration with respect to lifting and handling such large precast panels is to ensure that the lifting lines are as near to vertical as possible to minimize any bending stresses caused by the lifting angle. The *PCI Design Handbook* recommends a lifting angle (angle between the top surface of the panel and the lifting line) of at least 60°.[3] Based on this recommendation, and assuming the four lifting lines come to a common point at the crane hook, the minimum length for the lifting lines for the panels used for this project was 6.7 m (22 ft). While the precast fabricator used a travel-lift with a spreader beam and nearly vertical lifting lines for handling at the plant, the 6.7-m (22 ft) minimum lifting line length was required for the crane used for on-site installation. Figure 22 shows a panel being removed from the forms at the precast plant after curing using a travel-lift with a spreader beam.

Figure 22. Photo. Removal of a panel from the forms after curing.

All of the precast panels were stored at the fabrication plant until installation. Because of the uneven surface of the panels, it was not possible to stack the panels on top of one another. The panels were stored at the plant with two supports at approximately the same location as the lifting anchors. The panels were continuously checked to ensure cracking did not occur during storage.

TRIAL ASSEMBLY

Several weeks prior to installation of the panels on site, the precast fabricator conducted a trial assembly of three panels at the precast plant. The panels were assembled over a prepared base material to simulate on-grade installation. The trial installation demonstrated excellent fit between the panels.

PANEL REPAIRS

Panel repairs during fabrication were addressed on a case-by-case basis. The precast fabricator was required to repair any damage to the keyways, pockets, or surface of the panels that occurred during fabrication. Repairs to keyways were required to be such that the repaired area did not protrude (bulge) beyond the adjacent true edge of the keyway. Repairs to the surface of the panels required sawcutting a clean void around the repair area and patching with a Caltrans-approved patching material.

Only one repair was required due to damage that occurred during the trial assembly of the panels. During the trial assembly, the top lip of the female keyway of one of the panels ruptured at the outside edge (pavement shoulder) of the panel. To repair this, the precast fabricator was required to sawcut and remove the damaged area, drill and epoxy rebar into the existing concrete so that it would extend into the repair area, and patch the keyway such that it would not affect the fit of the panel. This repair was completed and did not affect assembly of the panel during the on-site installation.

CHALLENGES AND PROBLEMS ENCOUNTERED

Corner Break—The only major problem encountered during fabrication was the corner break discussed previously, which occurred during the trial assembly at the precast plant. The reason for the break is not known, but it could have been caused by debris in the keyway during assembly or by a slight irregularity in the keyway that resulted in a wedging action when the panels were assembled. A similar problem encountered during construction is discussed in chapter 6.

Mid-panel Cracking—An issue that was encountered during the Texas pilot project was the formation of a hairline crack in the top surface of the panel approximately mid-way from the ends.(2) This crack was believed to have been caused by the temperature change experienced by the panel in the first 24 hours after casting. As the panel cools after reaching the peak hydration temperature and tries to contract, the movement is restrained by the casting bed, causing the crack to form. Similar mid-panel cracking was observed in some of the panels for the California project, most likely due to the same condition. Fortunately, these are only hairline cracks, which are essentially unnoticeable after release of prestressing, and they will be held closed by the pretensioning in the panels. Figure 23 shows one of these mid-panel hairline cracks, which are nearly invisible to the unaided eye.

Figure 23. Photo. Mid-panel hairline crack after curing.

CHAPTER 6. PAVEMENT CONSTRUCTION

PRE-CONSTRUCTION

The week prior to installation, a pre-construction meeting took place at the office of the installation contractor. Representatives from Yeager Skansa, Inc. (installation contractor), Caltrans, Pomeroy Corporation (precast fabricator), Dywidag Systems International (post-tensioning contractor/supplier), and The Transtec Group, Inc., were present at the meeting. The purpose of the pre-construction meeting was to ensure coordination of the installation effort. The precast fabricator was responsible for delivery of the panels to the site, the installation contractor was responsible for panel installation, and the post-tensioning supplier was responsible for temporary post-tensioning during installation and permanent post-tensioning and grouting after installation. Caltrans was responsible for oversight and inspection of the construction process.

BASE PREPARATION

The precast pavement test section was constructed over embankment fill behind a new retaining wall as part of the widening of I-10. The LCB and aggregate base were placed well in advance of the precast panels. Due to unforeseen changes in the project schedule, the actual test site was not where it was originally proposed, and therefore strict tolerances on smoothness were not enforced during the placement of the LCB. While this was not desirable, it provided a good indication of the smoothness that could be expected for a typical LCB. Any obvious “high spots” in the LCB were ground off before the panels were installed.

The polyethylene sheeting (friction-reducing material) for each panel was rolled out just before the panel was placed to minimize any damage to the sheeting caused by foot traffic or construction vehicles and equipment. The surface of the LCB was cleared of loose rocks and debris before the polyethylene sheeting was unrolled.

TRANSPORTATION TO SITE

The panels were transported to the job site on flat-bed tractor trailers. Due to weight restrictions, only one panel could be transported on each truck. The panels were shipped from the fabrication plant in Perris approximately 97 km (60 mi) to the project site in El Monte. The panels were supported on the truck with two-point supports, as they had been supported at the precast plant. The panels were strapped down using nylon straps to prevent damage during shipment, as shown in figure 24.

Figure 24. Photo. Panels were shipped to the job site one per truck.

PANEL PLACEMENT

Precast panel installation took place on the nights of April 12 and 13, 2004. Installation was completed before 5 a.m., and post-tensioning was started immediately after panel installation was completed each night.

Procedure/Staging

Panel installation took place at night so that two of the four existing lanes of I-10 could be closed to allow the trucks delivering the precast panels to stage outside of the construction barrier. The lane closures commenced at 10 p.m. and were removed by 5 a.m. A 90.7-Mg (100-ton) crane stationed on the LCB was used to lift the panels off of the trucks, swing them over the construction barrier, and set them in place. The different steps of the installation process are shown in figures 25 through 29.

Each panel was lowered into place at a "nose down" angle to facilitate mating the panel with the adjacent panel already on the ground. The panels were lowered into place with the female keyway as the leading edge, mating to the adjacent panel. The panels were aligned using a mark on the surface of each panel, directly above one of the post-tensioning ducts. This prevented any misalignment that may have been caused by aligning the panels by their ends. A slow-setting epoxy adhesive was applied to the keyways prior to installing each panel. Although the purpose of the epoxy was both to provide lubrication for assembling the panels and to seal the joints between panels, it proved to be too thin to provide lubrication and did not seal joints that were open more than 3 mm (1/8 in.).

As each panel was installed, it was temporarily set approximately 0.3 m (1 ft) from its final position while temporary post-tensioning strands were fed from the panels already in place into the new panel. The panel was then lifted slightly, moved into place, and the temporary post-tensioning strands were stressed to pull the panels together as tight as possible. The lifting lines were then removed, the temporary post-tensioning anchor was removed, and the process was repeated with the next panel.

Figure 25. Photo. A precast panel is lifted off the truck and over the concrete barrier and set in place.

Figure 26. Photo. Workers detach the lifting lines from a precast panel. Note the polyethylene sheeting, rolled out just prior to placement of each panel.

Figure 27. Photo. Lowering a panel into place. Each panel was lowered at a nose-down angle to facilitate mating it with the panel already in place.

Figure 28. Photo. Workers apply epoxy to the joint prior to assembling the panels.

Figure 29. Photo. Workers use temporary post-tensioning to pull the panels together as they are assembled.

Placement Rate

During the 1st night of panel installation, 16 panels were set in place in approximately 5 hours. During the 2nd night, the process was more efficient, and the remaining 15 panels were installed in just over 3 hours. Efficiency was improved the 2nd night by adding a second set of lifting hooks and applying the epoxy to the panel while it was on the truck waiting to be installed. Overall, the average placement rate was approximately 15 minutes per panel, including the time necessary to move the crane (after every 3 panels) and apply temporary post-tensioning.

POST-TENSIONING

Final longitudinal post-tensioning began following placement of the last panel each night. The post-tensioning strands were precut to length and delivered to the site prior to panel installation. The strands were uncoiled, fed into the post-tensioning ducts from the central stressing pockets, and pushed by hand to the anchors in the joint panels, as shown in figure 30. An anchor chuck was threaded onto the end of each strand from the access pockets in the joint panels, and the wedges were seated around the strand. A "dogbone" or "ring anchor" coupler (shown in figure 5) was threaded onto the ends of the two strands in the central stressing pockets, and wedges were seated around each of the strands, as shown in figure 31. A monostrand stressing ram was then used to tension the strands from the stressing pockets. The stressing ram tensioned both strands simultaneously by gripping one strand while reacting against the other strand by pushing against the coupler, as shown in figure 32. Each tendon was initially stressed to 20 percent of the ultimate load, each strand was then marked, and then the tendon was stressed to the ultimate load. The elongation of the tendon was then determined by measuring the distance the mark on each strand had moved when stressing it from 20 percent to the full load.

Post-tensioning strand coated with a fine-grit-impregnated epoxy was used for the longitudinal tendons. Epoxy-coated strand provides an extra layer of protection against corrosion, which is particularly critical at the joints between panels. The grit in the epoxy helps to provide better bond between the strand and the grout. This strand requires special wedges for the post-tensioning anchors, specifically designed for epoxy-coated strand. Previous testing by the strand supplier demonstrated that grit-impregnated strand would not damage the post-tensioning ducts as it was fed through them.

Post-tensioning (stressing) required approximately 5 to 10 minutes per tendon, barring any problems with equipment. Stressing began with the strands at the center of the slab (centerline of the pavement) and alternated outward to the strands on either side of the centerline. In total, 12 tendons were stressed for each slab, 24 tendons in all.

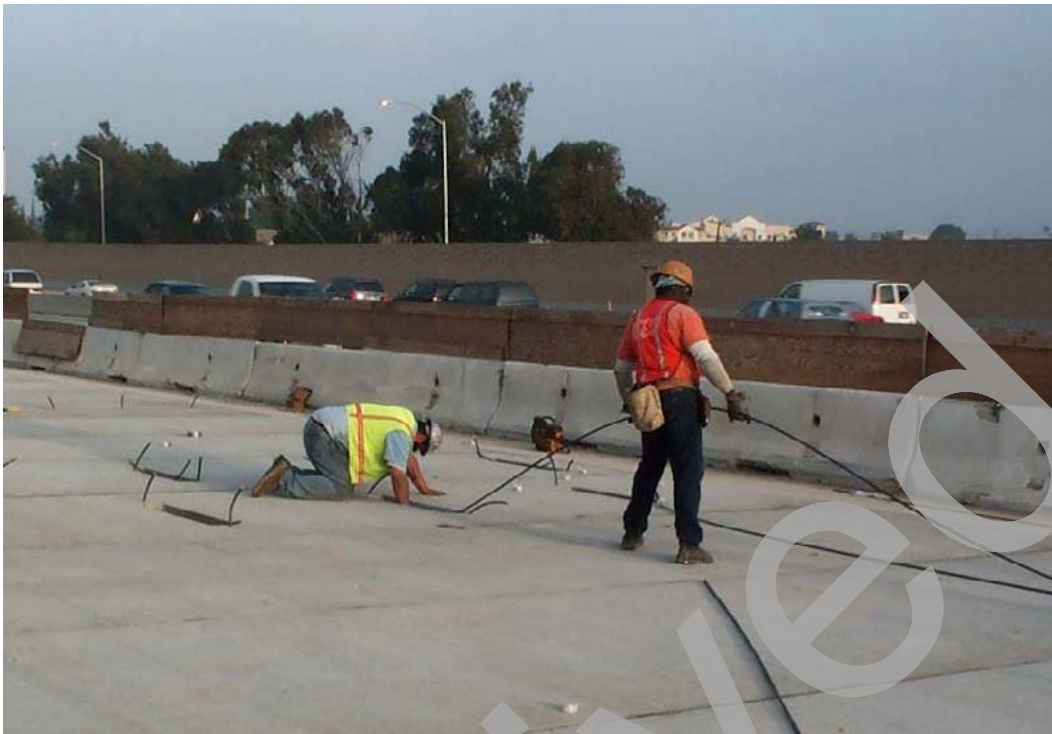

Figure 30. Photo. Workers feed the post-tensioning strands into the ducts at the central stressing pockets and push them through the ducts to the anchors.

Figure 31. Photo. A “dogbone” coupler is used to splice the post-tensioning tendons together in the stressing pockets. The stressing ram tensions both tendons at the same time through the coupler.

Figure 32. Photo. A monostrand stressing ram is used to tension each of the post-tensioning tendons.

EXPANSION JOINT SEAL

A recess for the expansion joint seal (figure 18) was cast into the joint panels. This recess was widened prior to installation of the seal by sawcutting. A preformed elastomeric seal was then installed in the joint. The seal was selected such that it could accommodate the anticipated total expansion joint movement. The seal was installed after completion of post-tensioning, and was installed slightly lower than the finished pavement surface to permit diamond grinding.

MID-SLAB ANCHOR

To ensure that the finished precast pavement would expand and contract outward from the middle of the slab (at the central stressing panels), it was necessary to anchor the center of each slab to the base/subbase. These mid-slab anchors were created by drilling two holes, 38 mm (1 1/2 in.) in diameter, a minimum of 200 mm (8 in.) into the base/subbase at each of the central stressing pockets; and inserting bars, 25 mm (1 in.) in diameter, into the holes, as shown in figure 33. The bars were grouted in-place in the holes with nonshrink grout, with a minimum of 100 mm (4 in.) of the bar protruding into the stressing pocket. After the central stressing pockets were patched, the mid-slab anchor bars restricted the middle of the slab from movement.

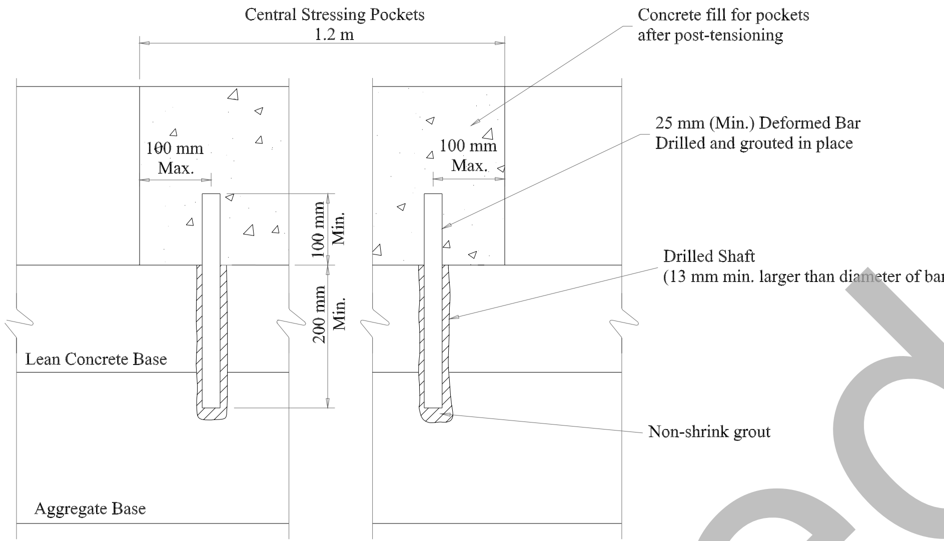

Figure 33. Illustration. Mid-slab anchor at the central stressing pockets.

FILLING/PATCHING POCKETS

After completion of post-tensioning and after the mid-slab anchor bars were drilled and grouted in place, the stressing pockets (central stressing panels) and anchor access pockets (joint panels) were filled and finished flush with the pavement surface. Although a fast-setting concrete could have been used to permit traffic onto the pavement as soon as possible, a "pea gravel" mixture was used to fill the pockets, as shown in figure 34. The patches were given a surface texture similar to the surrounding concrete, and two coats of curing compound were applied.

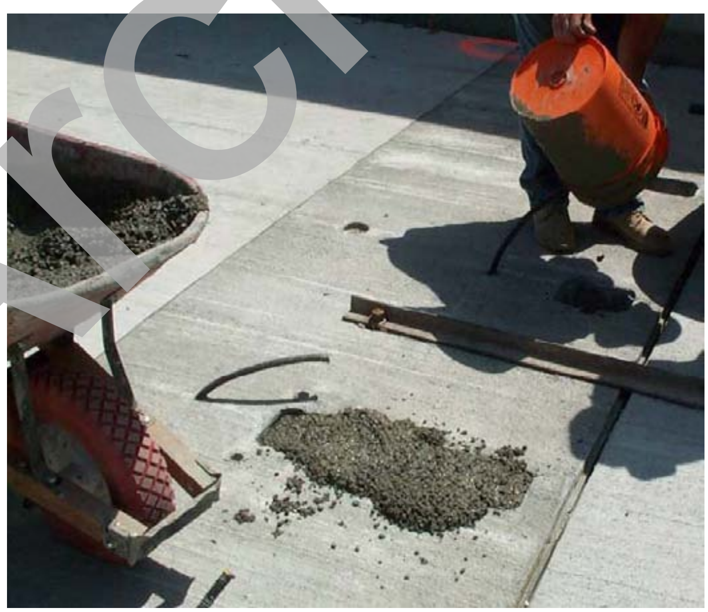

Figure 34. Photo. Workers fill the post-tensioning pockets with a "pea gravel" concrete mixture after post-tensioning.

SURFACE REPAIRS AND DIAMOND GRINDING

Any major damage to the surface of the pavement that could affect ride quality or pavement durability was required to be repaired prior to opening the pavement to traffic. Minor spalling at the joints between panels was repaired at the discretion of the engineer on site. The only major distresses that occurred resulted from the panels coming together too quickly during assembly, causing spalling of the joint or corner breaks. Only one major corner break that required repair was encountered during construction; it is shown in figure 35. Had any cracking occurred during panel installation, the cracks would have been sealed using an approved method such as epoxy injection.

To achieve the Caltrans ride quality requirements, the finished surface of the precast pavement was diamond ground in the traffic lanes. Ride quality after diamond grinding was measured by the contractor using a California profilograph. Four separate passes were made with the profilograph across the width of the traffic lanes, and the trace was recorded on an automated profilogram. The profile indexes, measured for each of the four passes using a 5-mm (0.2 in) blanking band, were 15.6 mm/0.1km (9.9 in/mi), 9.7 mm/0.1km (6.2 in/mi), 1.9 mm/0.1km (1.2 in/mi), and 11.7 mm/0.1km (7.4 in/mi).

Figure 35. Photo. Corner break that occurred during panel installation—the only major distress encountered.

GROUTING

Grouting was essentially the final major step in the construction process, and took place after the central stressing pockets and anchor access pockets were patched. In preparation for grouting, the exposed longitudinal edge of the slab was backfilled and the bottom of each of the expansion joints was sealed with expansive spray foam to prevent grout from leaking into the joint from beneath the slab.

Longitudinal Tendon Grouting

As discussed previously, the purpose of grouting the longitudinal post-tensioning tendons is two-fold. First, the grout provides an additional layer of protection for the tendons against corrosion. Secondly, the grout bonds the tensioned strands to the precast panels. This bonding will prevent a complete loss of prestress in the slab if one or more of the tendons are cut, either inadvertently or to remove a damaged precast panel.

Grouting was performed by the post-tensioning contractor/supplier. A high-capacity grout pump/mixer was used for both tendon grouting and underslab grouting. The grout mixture consisted of Type II/V cement and 19 L (5 gal) of water per sack. Grout was pumped into one end of each tendon, at either the joint panel or the central stressing panel, until it flowed out of each intermediate grout vent and finally out of the vent at the opposite end of the tendon, as shown in figure 36. Each intermediate vent was closed off when grout flowed out of it.

Figure 36. Photo. Grout is pumped into one end of the post-tensioning tendon at the stressing pocket until it flows out of the far end.

In some instances, grout began to leak from the joints between panels onto the surface of the pavement. In those instances, the grouting hose was moved to an intermediate grout vent and pumped until it flowed out of the end vent. If grout did not flow out of the end after a significant amount of pumping (indicating that grout was leaking from a joint to beneath the slab), the hose was moved to the end vent and pumped until it flowed from an intermediate vent. Each tendon was grouted until the inspectors were confident it was fully grouted.

Underslab Grouting

Underslab grouting ensured that any voids beneath the precast panels were filled so that the panels were fully supported. Underslab grouting was completed in much the same way as tendon grouting, using the same mixer/pump and grout mixture. Grout was pumped into the inlet at the low end of each underslab grout channel until it was observed flowing either out of an intermediate vent or out of an adjacent grout channel vent.

CHALLENGES AND PROBLEMS ENCOUNTERED

Due to the experimental nature of this demonstration project, which is only the second known precast prestressed pavement constructed in the United States, minor problems were anticipated during construction. This section will discuss some of these problems and the solutions that were developed.

Panel Placement

Leveling Course Smoothness—The primary issue encountered during panel placement was the smoothness of the LCB leveling course. Because LCB is a rigid material, it did not conform to the bottom of the precast panels as the asphalt leveling course did in the Texas pilot project.[2] This rigidity resulted in a significant number of voids beneath the precast panels that required a large volume of grout to fill. Fortunately, no cracking due to the uneven leveling course was observed. Smoothness of the leveling course is an issue that must be addressed on future projects, however.

Joint Spalling/Corner Breaks—As discussed previously in this chapter, the only major distress that required repair was a corner break, which occurred as a panel was installed. This break was most likely caused by the panel impacting the adjacent panel as it was lowered into place. It is possible that debris may also have been present in the keyway, causing a stress concentration. There were also several incidences of minor spalling at the top surface of the pavement at the joints. This spalling, likewise, was attributed to the panels impacting each other during assembly. These problems can be avoided on future projects by carefully monitoring the assembly process to ensure that the keyways are free from debris and the panels do not impact each other as they are assembled.

Post-Tensioning

Pinched Duct—As the post-tensioning strands were being pushed through the ducts, one of the strands could not be pushed past a certain point. Measurement of the location where the strand had stopped revealed that it was in the middle of one of the panels and not at a joint. It is believed this was caused by a pinched post-tensioning duct that prevented the strand from sliding freely. Fortunately, with enough force, the workers were able to force the strand past this point to the anchorage. This was only an isolated incident and can be prevented on future projects by carefully checking all post-tensioning ducts at the precast plant for blockages or pinched areas.

Seating of Wedges—Another isolated incident occurred during post-tensioning when a set of wedges would not seat around the post-tensioning strand at the coupler in the stressing pocket. As the operator began to release the tension in the stressing ram, one set of wedges would not seat around the strand, forcing the workers to de-tension the entire tendon and replace the wedges. Inspection of the wedges revealed that the epoxy coating had clogged the teeth, preventing them from gripping the strand. This was most likely caused by repeated seating of the wedges during the tensioning process, which required up to two resets of the stressing ram to get the full tension in the strand. This problem could be prevented on future projects by using a stressing ram with a long enough stroke to fully tension the strand with one pull (if epoxy-coated strands are used).

Stressing Pocket Dimensions—Although the workers were able to successfully tension all of the tendons, the size of the stressing pockets made post-tensioning more difficult. The width of the stressing pockets (200 mm [8 in.]) was not wide enough for the stressing ram to lie flat, and the length of the pockets required the ram to stick out of the pockets at an angle during stressing (see figure 32). While this did not cause any problems with stressing, larger pocket dimensions would have simplified the operation.

Grouting

Grout Leakage—The only problem encountered during grouting was leakage of the grout onto the surface of the pavement. This was caused by an inadequate seal around some of the post-tensioning ducts at the joint between panels. Although the leakage was not a major problem (about 6 percent of the ducts leaked at a joint), it did slow the grouting process and required the excess grout to be cleaned off of the surface. It may never be possible to eliminate this problem completely, but it can be minimized by carefully inspecting the gaskets around the ducts prior to panel assembly and by ensuring that the joints between panels are closed as tightly as possible during panel installation. Epoxy applied to the keyways may also reduce the occurrence of grout leakage.

CHAPTER 7. INSTRUMENTATION AND MONITORING

INTRODUCTION

As part of the demonstration project, it was necessary to monitor the behavior and performance of the test section. Monitoring of the behavior provides an indication of the accuracy of the assumptions made during the design process and the accuracy of the PSCP2 program. Monitoring of pavement performance also helps in identifying design details that need improvement and provides an indication of the true long-term durability of prestressed precast pavement.

Chapter 4 presented many of the variables that were part of the pavement design procedure. Temperature was one of the most important variables because it affects both horizontal (expansion and contraction) and vertical (curling) slab movements and their associated stresses. In addition to temperature, actual slab movements will be monitored over time.

PROJECT-LEVEL CONDITION SURVEY

Following construction, a project-level condition survey was performed to establish the initial as-constructed condition of the precast pavement prior to opening to traffic. This initial project-level condition survey will be used for comparison with future condition surveys to identify any new distresses that occur over time. For the condition survey, a distress map was developed, noting any distresses that occurred during construction, including longitudinal and transverse cracking, random shrinkage cracking, spalling, and corner breaks.

The distress map for the as-constructed condition is shown in figure 43 at the end of this chapter. To summarize the as-constructed condition:

- No cracking was observed in any of the precast panels.
- Three instances of minor spalling were noted, with each spall less than 6 mm (1/4 in.) in depth and less than 0.1 m^2 (1 ft^2) in area. All spalling occurred at the joints between panels.
- Five instances of moderate spalling were noted, with each spall less than 13 mm (1/2 in.) in depth and less than 0.1 m^2 (1 ft^2) in area. All spalling occurred at the joints between panels.
- One corner break was observed. The corner break was less than 100 mm (4 in.) in depth and extended approximately 1.2 m (4 ft) from the slab edge.

The corner break was required to be repaired, but repairs to spalling were not required. Most of the minor spalls were removed by diamond grinding, and most of the moderate spalls were reduced to minor spalls during grinding.

TEMPERATURE INSTRUMENTATION

Temperature instrumentation was embedded in the precast panels to monitor slab temperature during fabrication and over the life of the pavement. The temperature data are used to correlate measured slab movements to slab temperature. Temperature sensors were embedded at 25 mm (1 in.) from the top surface of the panel, at mid-depth, and at 25 mm (1 in.) from the bottom of the panel. Because panel thickness varied from 250 mm (10 in.) to 330 mm (13.1 in.), separate sets of sensors were positioned at those two panel thicknesses. The sensors were located at least 0.3 m (1 ft) from the panel edges to prevent "edge effects" in the temperature measurements.

The sensors used for measuring and recoding slab temperature are Thermochron iButtons®. These sensors, which are roughly 16 mm (5/8 in.) in diameter and 6 mm (1/4 in.) thick, measure and log the temperature internally. They are capable of logging over 2,000 temperature measurements at prescribed intervals between 1 minute and 4 hours. A single twisted-pair wire attached to the sensor is used to download the temperature data to a personal computer or handheld device.[19] Each temperature data point is recorded with its own date/time stamp. To ensure that the data from each sensor are kept separate from other sensor data, each sensor has a unique serial number that is recognized by the computer software.

In total, 15 temperature sensors were embedded in the precast panels in sets of three (top, middle, bottom). Two sets were embedded where the panel thickness is 250 mm (10 in.), and the remaining three sets were embedded where the panel thickness is 330 mm (13.1 in.). The sets of sensors were embedded in the joint panels, where slab movement is measured.

Each set of sensors was tied to a wood dowel to ensure proper spacing of the sensors, as shown in figure 37. The wood dowel with the sensors attached was tied to reinforcement in the panels to keep it in place during casting. The wires from each sensor were extended to junction boxes at the edge of the slab. The lids on the boxes are removable to permit access to the sensor leads over the life of the pavement. Figure 38 shows the general location of the sensors and junction boxes in the precast panels. (The location of the temperature sensors in the overall precast pavement test section is shown in the condition survey in figure 43 at the end of this chapter, with sensor identification and pavement thickness in accompanying table 8.)

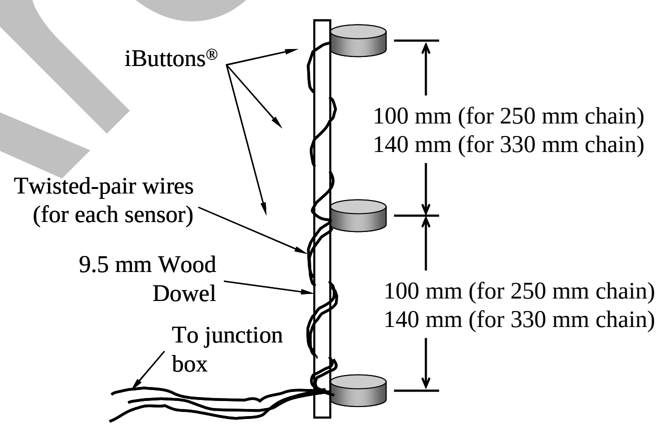

Figure 37. Illustration. Chain of three iButton® sensors for top, mid-depth, and bottom slab temperatures (1 mm = 0.039 in.).

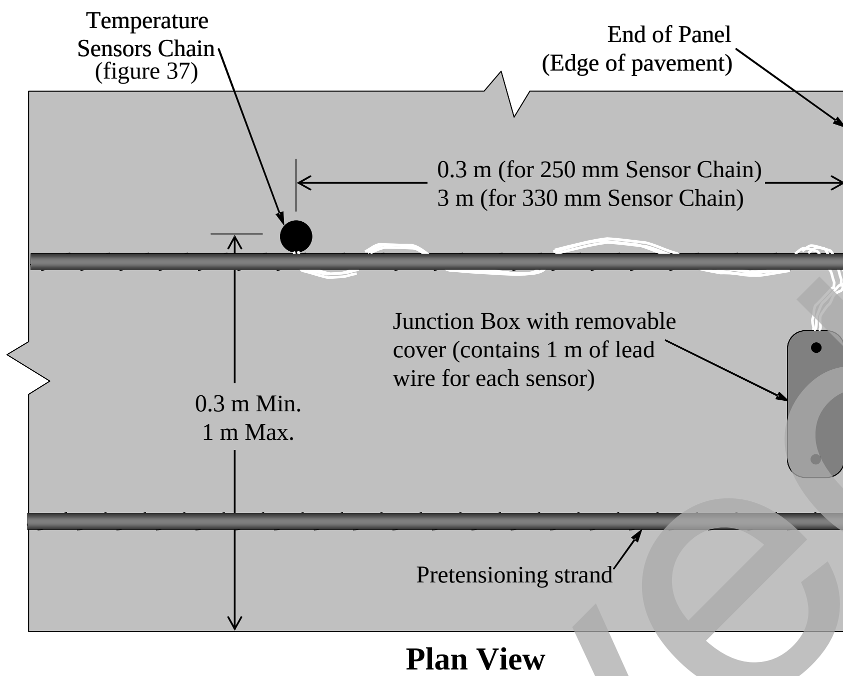

Figure 38. Illustration. Typical layout for a set of iButtons® with the twisted-pair wire from the sensors routed to a junction box at the end of the panel (edge of the pavement).

HORIZONTAL MOVEMENT

Accurate prediction of horizontal slab movement is important for determining the maximum permissible slab length during the design process. Measurement of horizontal slab movement provides an indication of both the thermal expansion characteristics of the concrete as well as the amount of restraint from frictional resistance at the slab–support interface.

Horizontal slab movement is monitored by measuring the width of the expansion joints using dial calipers. Demec points epoxied to the surface of the joint panels on either side and at either end of the expansion joint provide a reference location for the measurements. Measurements are taken at various times of the day and night to obtain a thorough correlation of joint width and slab temperature.

PRELIMINARY MONITORING

Preliminary monitoring was conducted during panel fabrication and installation and soon after construction, prior to opening the pavement to traffic, as described below.

Temperature History

Figure 39 shows the temperature history from one set of temperature sensors during panel fabrication. The sensors revealed that the internal slab temperature during the peak heat of hydration (during steam curing) for this particular panel reached nearly 52 ºC (125 °F). The top–

bottom slab temperature differential became much more evident after casting, while the panels were stored at the precast plant. A top–bottom temperature differential of nearly 17 ºC (30 °F) was recorded for several days during storage.

Figure 40 shows the temperature history during a 24-hour period after panel installation, prior to opening the pavement to traffic. The slab temperature shown is the mid-depth temperature from two different sensors (sensors 1-M and 5-M). This plot clearly demonstrates the lag between ambient temperature and slab temperature that is typical of concrete pavements. The plot also shows the insulating effect of the base on the slab temperature. In contrast to the slab temperature during storage (figure 39), the on-grade slab temperature is almost always higher than ambient temperature.

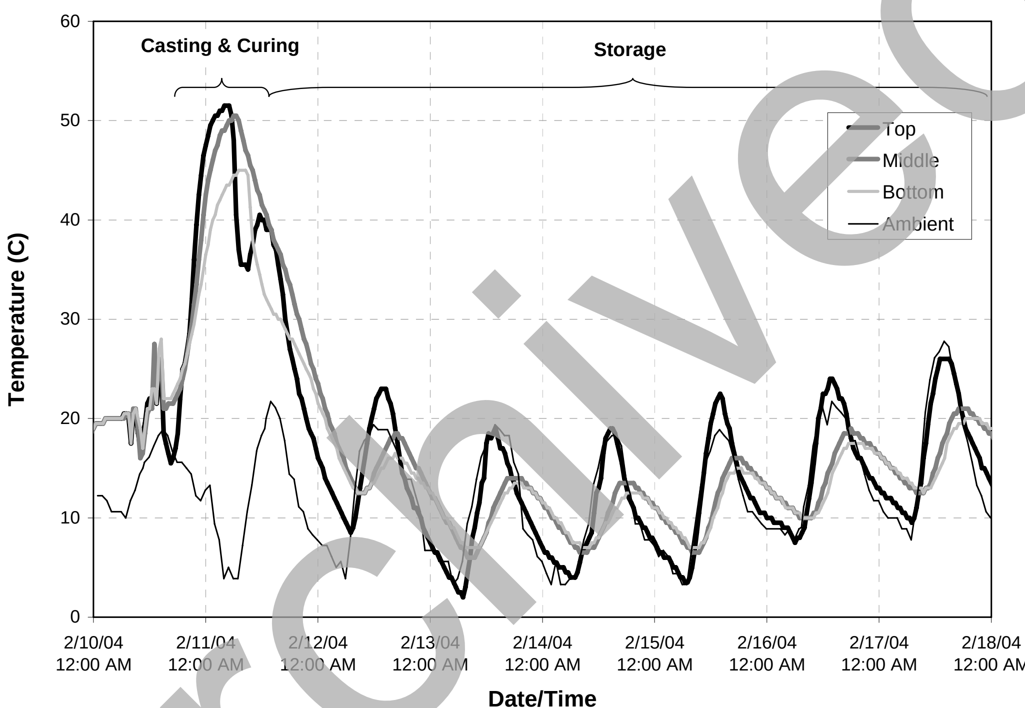

Figure 39. Graph. Temperature history for one set of temperature sensors during casting and storage at the precast plant.

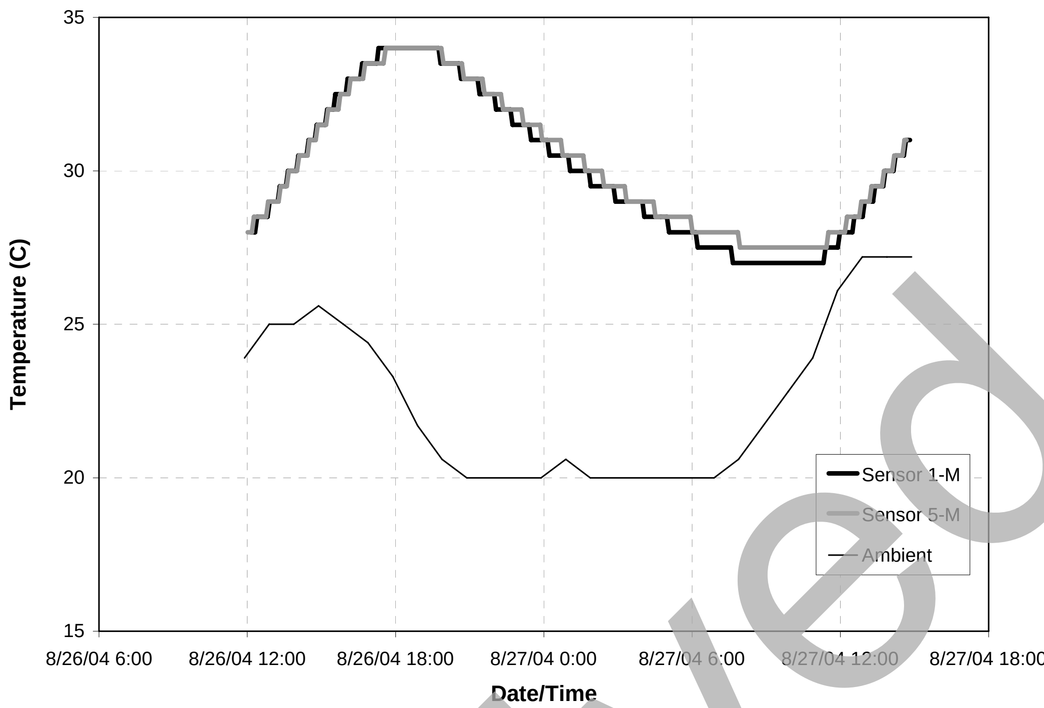

Figure 40. Graph. Temperature history for two different mid-depth sensors prior to opening to traffic.

Horizontal Movement

Horizontal slab movement (expansion joint width) was monitored for a 24-hour period approximately 3 months after panel installation. Dial calipers were used to measure the width of each of the three expansion joints (see figure 43 for joint numbers) using stainless steel Demec points as gage points for the measurements (figure 41). Each joint was measured at each end (north and south) and at the middle. These measurements were then correlated to mid-depth slab temperature. Figure 42 shows a normalized plot of the relative movement of the expansion joints with change in mid-depth slab temperature. The mid-depth slab temperature and ambient temperature for the measurement period were shown in figure 40 (August 26–27, 2004).

As figure 42 shows, roughly the same relative movement was measured for joints 1 and 3 at the north end and middle of each joint. Because joints 1 and 3 abut the cast-in-place pavement at either end of the test section, this relative movement represents the movement of one end of each of the 37.8 m (124 ft) slabs. The relative movement of joint 2, however, represents the movement of the two slabs on either side of joint 2, or 75.6 m (248 ft) of pavement. It would be expected, therefore, that the relative movement of joint 2 would be approximately twice that of joints 1 and 3. As figure 42 indicates, however, this was not the case. It is not known why joint 2 experienced so little movement, but it was most likely due to a temporary precast concrete median barrier resting across the joint during the period of measurement. The barrier, as well as a significant amount of debris in the joint, probably restrained joint movement during the measurement period.

Figure 41. Photo. Dial calipers were used to measure expansion joint movement using Demec points on either side of the joint as gage points.

Figure 42 also shows less relative movement at the south end of joints 1 and 3 than at the middle and north end. This was most likely caused by a shadow cast over the south side of the pavement by an abutting sound wall. The sound wall shades the south side of the pavement throughout the day, preventing slab temperatures from reaching as high as at the middle and north side of the slab, resulting in less movement at that end of the joints. Table 7 summarizes relative slab movement based on a linear regression of the field data, shown in figure 42, along with the degree of correlation (R^2 value) of the data to the regression. A high degree of correlation was observed for the north end and middle of joints 1 and 3, but a lower degree of correlation at the south end, most likely due to the shading effect mentioned above. Only the south end of joint 2 showed good correlation.

Measured Versus Predicted Movement

To check the predictions made by the PSCP2 program during the design (chapter 4), the actual slab temperatures during construction (April 14, 2004) and the slab temperatures during the measurement period (August 26–27, 2004), were input into the PSCP2 program. (The analysis period input for comparison was 134 days after construction.) Based on the temperature inputs, and the design inputs discussed in chapter 4, the relative movement predicted by the PSCP2 program was 0.150 mm/°C (0.0033 in/°F). This value for relative movement is very close to that measured on site (for joints 1 and 3), indicating that the assumptions used during the design were accurate for predicting expansion joint movement.

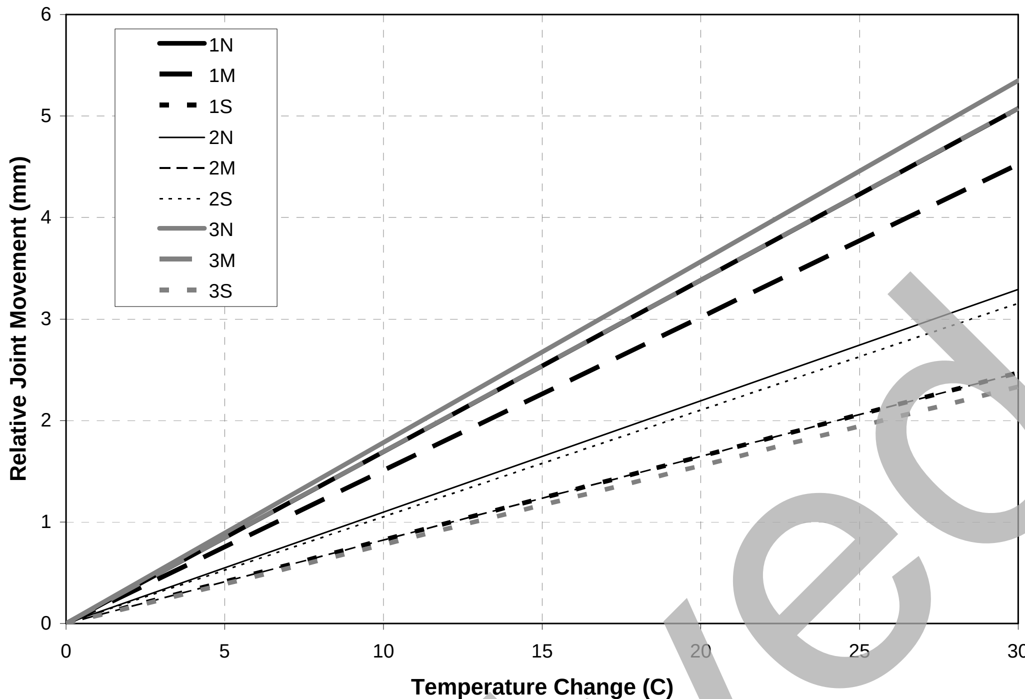

Figure 42. Graph. Relative total movement of three expansion joints with temperature at the north, middle, and south ends (measurements obtained August 26–27, 2004).

Table 7. Summary of Relative Slab Movement and the Corresponding Degree of Correlation for Each of the Joints

Joint	Location (north/middle/south)	Relative Slab Movement mm/°C	Degree of Correlation (R^2)
1	N	0.169	0.835
	M	0.151	0.916
	S	0.082	0.640
2	N	0.110	0.439
	M	0.082	0.649
	S	0.105	0.844
3	N	0.178	0.888
	M	0.169	0.835
	S	0.078	0.541

Monitoring Plan

Long-term monitoring is important for evaluating the behavior and performance of precast prestressed pavement. While pavement performance is the more critical aspect, monitoring slab movements will help to improve the design process through better prediction of pavement behavior.

Ideally, long-term monitoring should be performed at intervals approximately 1, 5, 10, and 30 years after construction. As a minimum, a condition survey should be completed to document changes to existing pavement distresses and any new distresses that may have appeared. If traffic conditions permit, horizontal and vertical slab movements and slab temperature should be measured also. Measurement of horizontal and vertical slab movements should be completed over a minimum of 24 hours each time. The measurement period should be during a season when large differences in daily high and low temperatures are expected (usually fall or spring) and preferably on days with a clear sky.

Table 8. Temperature Sensor Chain IDs Corresponding to Figure 43

Chain ID (*Figure 43*)	**Length**	**iButton® Serial No.**	**Sensor Location (Top/Middle/Bottom)**
3	330 mm (13 in.)	1725400006955020	M
		B825400006A0DE21	T
		7B254000059F5321	B
5	330 mm (13 in.)	B3254000059EF021	M
		4525400006778A21	T
		8A2540000316A521	B
1	330 mm (13 in.)	1F254000058BE321	B
		A825400005944821	T
		A425400005658321	M
2	250 mm (10 in.)	7E25400003022921	T
		BB254000057F9421	M
		2225400005A05521	B
4	250 mm (10 in.)	BC2540000557E721	T
		FA25400005A62121	M
		71254000067AA821	B

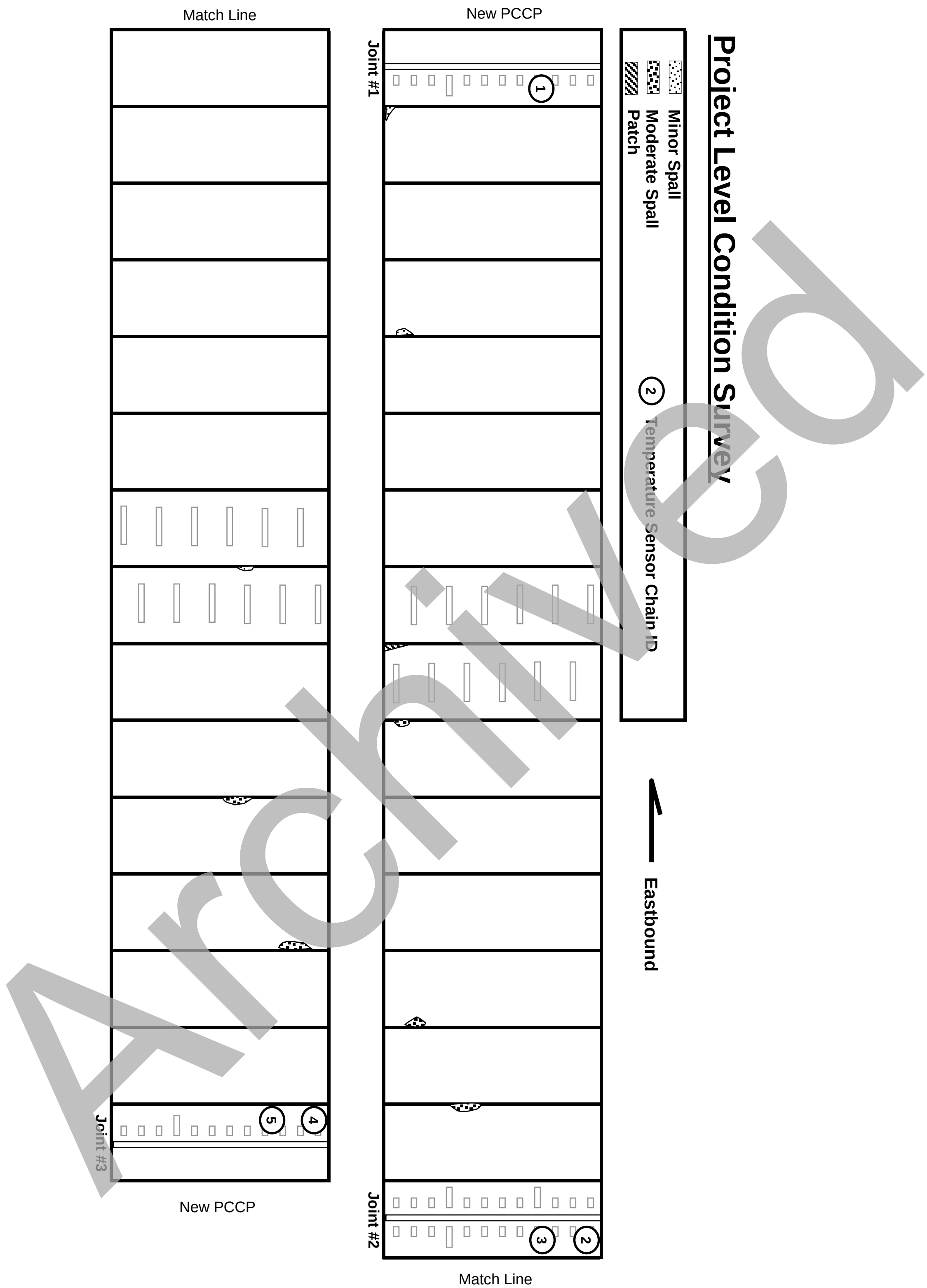

Figure 43. Illustration. Project Level condition survey after construction and location of temperature sensors.

CHAPTER 8. PROJECT EVALUATION AND RECOMMENDATIONS FOR FUTURE PROJECTS

DESIGN

The design procedures and details for the California demonstration project were based primarily on the recommendations from the Texas pilot project.[2] An evaluation of the design procedures and details and recommendations for future projects are discussed below.

Design Procedures

The design procedure for the California demonstration project was originally developed during the FHWA precast pavement feasibility study.[1] Essentially, the precast pavement section is designed to have a fatigue life equivalent to a much thicker conventional pavement. While the original design only called for a 250-mm (10 in.) jointed concrete pavement with a 30-year design life, the prestressed precast pavement, which varies in thickness from 250 mm (10 in.) to 330 mm (13.1 in.) will have a significantly longer fatigue life, as shown in chapter 4.

The design procedure is mechanistic in nature, incorporating both layered elastic analysis for fatigue evaluation and separate analysis using the PSCP2 program for behavior specific to prestressed pavement. While the PSCP2 design program has been calibrated using data from actual prestressed pavements, it was not originally intended to be used for precast pavements and should be modified for this purpose. The program should also account for both bonded and unbonded post-tensioning tendons. A more user-friendly, Windows-based interface that incorporates the layered elastic analysis and the PSCP2 analysis would greatly simplify the design process.

Other aspects of the design procedure, such as slab-support restraint and long-term performance, have also not been well established. Slab-support restraint is dependent upon the material on which the precast panels are placed. While push-off tests were conducted on the hot-mix asphalt leveling course for the Texas pilot project,[2] frictional characteristics for other materials, such as the LCB used for this project, have not been established, and therefore values must be assumed. Likewise, long-term performance has not been established due to the lack of precast prestressed pavements constructed thus far. Long-term performance data will help with refinement of the design procedure for a more accurate prediction of pavement life.

Design Details

The design details for this demonstration project did not cause any problems with fabrication or assembly of the precast panels. Several changes were made to the design details from the original feasibility study based on the recommendations from the Texas pilot project, however. These changes greatly improved both the fabrication and construction of the precast pavement section.

Cross Slope—Perhaps the most significant change from the previous project in Texas was the change in cross slope (from 1.5 percent to 5 percent) cast into the top surface of the panels. While this change presented a challenge to the fabrication process, requiring special forms and casting procedures, the finished product demonstrated that this detail can easily be incorporated into a precast pavement.

Expansion Joints—The expansion joints were designed specifically for this project. Because Caltrans was anticipating diamond grinding the finished pavement, plain doweled joints were specified in lieu of the armored joints used for the Texas pilot project.[2] Although plain doweled joints may not be as durable under repeated wheel loads, they should provide similar performance to typical concrete pavement joints. This plain doweled joint design is not recommended, however, when excessive expansion joint opening is expected. Based on the climate, thermal expansion characteristics of the coarse aggregate, and the selected slab length, a maximum expansion joint width of 25 mm (1 in.) was anticipated for this project. It is recommended that plain doweled expansion joints only be used when the anticipated maximum expansion joint width is 25 mm (1 in.) or less.

Post-Tensioning Anchors—One of the main issues with post-tensioning during the Texas project was the self-locking spring-loaded post-tensioning anchors. Due to problems with these anchors, they were replaced with standard dead-end chuck anchors for this project. Steel plates with circular collars to receive the post-tensioning ducts were cast into the joint panels, with the face of the plate open to the anchor access pockets, where standard post-tensioning chucks (for epoxy-coated strands) were fitted over the post-tensioning strands and seated flush against the steel plates. This detail eliminated problems with anchoring the post-tensioning strands and improved the efficiency of the post-tensioning process.

Keyways—The dimensions of the keyways for the Texas pilot project occasionally resulted in a wedging action as the panels were assembled, preventing some joints from closing completely and causing minor corner cracking from stress concentrations.[2] As shown in chapter 5, the dimensions of the keyways for the California project were relaxed slightly, resulting in a better fitting keyway. For future projects, the keyway dimensions should always permit the top and bottom vertical faces of the keyways to come together, while still providing load transfer across the joint. Additionally, when casting panels with varying thickness (due to changes in cross slope) the keyways should always be straight, or parallel to the bottom of the panel.

Another keyway detail that was incorporated into the panels was a chamfer along the bottom edge of the panels, as shown in figures 15 and 16. This chamfer prevented corner breaks when the panels were removed from the forms, which was a common occurrence during the Texas pilot project.[2] A 6-mm (1/4 in.) minimum (13 mm [1/2 in.] preferred) chamfer should be cast along the bottom edge of all panels for future projects.

Lifting Anchors—While the lifting anchors used for the Texas pilot project allowed workers to attach and remove the lifting lines from the panels faster, they left a recess 100 mm (4 in.) in diameter to be cleaned and patched.[2] For this project, screw-type lifting anchors were used, greatly reducing the size of the hole to be patched. The holes from the lifting anchors were small enough that they would not have any effect on traffic prior to being patched if it were necessary to open the pavement to traffic immediately. For future projects, screw-type lifting anchors are recommended unless alternative anchors that leave an equally small or smaller hole can be utilized.

Grout Channels—Another detail added to the panels was underslab grout channels. These channels and corresponding inlets/vents were cast into the panels to permit grouting beneath the precast panels after installation. Although only minimal underslab grouting was required for the Texas project,[2] these channels allowed the contractor to perform underslab grouting as needed without having to drill holes into the finished pavement. Grout channels are highly recommended for future projects.

Stressing Pockets—The dimensions of the stressing pockets proved to be slightly too small for the stressing ram used for longitudinal post-tensioning. For future projects where stressing pockets are used, it is recommended that the pocket dimensions be adjusted according to the dimensions of the stressing ram to be used. Alternatively, if the tendons could be tensioned at their ends from smaller pockets, this would better facilitate opening the pavement to traffic sooner. This option should be explored further for future projects.

PANEL FABRICATION

Overall, no major problems were encountered during the fabrication process. However, there were several differences between the fabrication of the panels for this project and the previous project in Texas. Below are some of the key aspects of fabrication, including changes made after the Texas project, and recommendations for future projects.

Tolerances—While the actual tolerances did not change for this project, additional tolerances were added. Most notably, a "squareness" tolerance, or the measured distance from corner to diagonal corner over the top surface of the panels, was added. The Texas pilot project revealed this to be a key tolerance to maintain alignment of the panels. For future projects, additional tolerances should be specified, including tolerances for vertical alignment (camber), horizontal skew, vertical batter, dowel location and alignment, and expansion joint straightness and width.

Casting Bed—The "long line" casting bed, where the panels are cast end-to-end with the pretensioning strands extending through all of the panels, has proven to be an efficient and effective technique for panel fabrication. The process is much faster than match-casting, and no problems have been encountered to date with assembling non-match-cast panels. A key element of the fabrication process is casting panels with a flat bottom. Any variations in cross slope should be cast into the surface of the panel only. The flat bottom will better ensure full support when the panels are installed over a flat base rather than trying to match a grade break in the base with the panels.

Finishing—The finish of the top surface of the panel has a major impact on the smoothness of the finished pavement. The panels for this project were finished by hand due to the grout vents protruding from the surface of the panels. While the final result was still a very smooth and flat finish, the panels should ideally be screeded with a vibratory screed followed by minimal hand finishing. A vibratory screed will provide a more uniform and flat surface. Unfortunately, a vibratory screed cannot pass over grout vents or lifting anchor supports protruding from the surface. If possible, the grout vents should be capped just below the finished surface until screeding and carpet drag texturing have been applied. The vents can then be carefully uncovered by removing the thin layer of concrete above them.

An issue that was encountered during casting was slight sloughing of the fresh concrete at the edges of the panels. Because the sideforms varied in depth from 250 mm (10 in.) at each end to a "peak" of 330 mm (13.1 in.), the fresh concrete mixture wanted to flow "downhill" away from the peak, resulting in slight sloughing at the peak. Although this did not significantly affect the finished pavement surface, it could be minimized on future projects by using a "drier" concrete mix that will not flow as readily, and by vibrating the mixture as little as necessary.

Steam Curing—While the panels for the Texas pilot project received only two coats of curing compound after casting, the panels for this project were steam cured overnight. Steam curing is a preferred option for precast pavement panels because it is a "moist cure" technique that helps to ensure the concrete has sufficient moisture for hydration during the critical hours after casting. Although not all fabrication plants are set up for steam curing, any "moist cure" technique, such as fogging, wet mats, or steam curing is preferable to curing compound, particularly in hot or arid climates. Steam curing also promotes strength gain, permitting the use of non-high-early-strength cements such as Type II cement rather than Type III cement.

PAVEMENT CONSTRUCTION

As discussed in chapter 6, pavement construction was very successful, with only minor problems encountered during panel installation and post-tensioning. Evaluation of the construction process and recommendations for future construction are presented below.

Base Preparation (Lean Concrete Base)

LCB was not an ideal material for supporting precast pavement panels. While it is a high-strength material, providing excellent structural support, it is a rigid material that will not conform to the bottom of the precast panels. Because it was difficult to finish LCB to strict tolerances, voids were present beneath the panels after they were installed. Although underslab grouting filled these voids, minimizing these voids before grouting is always preferred. If LCB is to be used for the leveling course, the use of a precision laser screed is recommended to achieve the smoothness tolerances described below. Under stringent time constraints, such as 6-to-10-hour construction windows, LCB may not be the best material since it requires time to cure before the precast panels can be installed over it.

Tolerances—A tolerance for the LCB was not specified in the original plans for the precast pavement. For future projects, however, it is recommended that the tolerances given in table 9 be specified for the leveling course.[2]

Table 9. Recommended Tolerances for Leveling Courses

Width of Leveling Course	Tolerance
≤ 3 m (10 ft)	± 3 mm (1/8 in.)
> 3 m (not to exceed 12 m)	± 6 mm (1/4 in.)

Alternative Materials—While hot-mix asphalt and LCB have proven to work for the leveling course material, neither are ideal materials for construction under very short construction windows, such as for overnight construction. Some alternatives should be considered for future projects:[2]

- Precision-graded aggregate base.
- Precision-graded, fast-setting LCB.
- Screeded grout bed.
- Cold-mix asphalt concrete.

Precision-graded base and screeded grout bed have proven successful for other precast pavement projects.[20, 21]

Panel Placement

Several modifications to the panel placement process from the Texas pilot project greatly improved the process. These modification and further recommendations are discussed below.

Joint Treatment/Epoxy—Segmental bridge epoxy proved to be beneficial not only for sealing the joints between panels, but also as a lubricant during assembly of the Texas pilot project.[2] Due to material supplier issues, however, a different epoxy was used for this project. The epoxy was much thinner in consistency, and therefore did not provide the same lubrication or sealing qualities. The epoxy was only effective in sealing the joints where the joint was tighter than 3 mm (1/8 in.), and was only applied to the top and bottom vertical faces of each keyway. This did demonstrate, however, that the panels could still be assembled without the epoxy as a lubricant. Although a thick consistency epoxy is recommended for future projects to both seal the joint and act as a lubricant, it is not absolutely essential for panel installation.

Temporary Post-Tensioning—Temporary post-tensioning consisted of two post-tensioning strands, each 15-mm (0.6 in.) in diameter, inserted into elongated access pockets in the joint panels and incrementally fed into each new precast panel as it was installed. After each panel was set as close as possible to its adjoining panel, the two strands were tensioned to pull the panels together as tightly as possible. This technique proved to be very efficient, increasing installation time only marginally, and very effective in ensuring the joints between panels were as tight as possible. Provisions for temporary post-tensioning should be incorporated into all future projects.

Panel/Duct Alignment—One issue encountered during the Texas pilot project was misalignment of post-tensioning ducts caused by using the ends of the panels as a reference point.[2] For this project, a chalk line was projected onto the surface of each panel exactly above the center of a given post-tensioning duct. This helped to ensure the post-tensioning ducts were consistently aligned by using one of the ducts as a reference. This technique for panel alignment is recommended for all future projects.

Post-Tensioning

Ducts—Corrugated galvanized steel ducts, which are standard materials for Caltrans, were used for the longitudinal post-tensioning ducts. The primary benefit of using this material (vs. plastic duct) was that it remained straight in the forms during fabrication, eliminating the need for bar stiffeners. While these ducts normally are less susceptible to crushing, one incidence of a pinched duct was encountered when the strands were fed through the panels, as discussed in chapter 6. This material, or comparable rigid plastic duct, is recommended for use on future projects. A minimum inside diameter of 25 mm (1 in.) for 15-mm strand is recommended. Although the larger duct requires more grout, it provides a larger tolerance for panel misalignment.

Strand—Fine-grit-impregnated epoxy-coated strand was used for longitudinal post-tensioning. Although the grit increases friction and makes strand insertion slightly more difficult, the workers did not have any problems feeding the strand through 18 to 21 m (60 to 70 ft) of duct.

The corrosion protection offered by the epoxy coating is a significant benefit over bare strand. As mentioned in chapter 6, however, a stressing ram with a long enough stroke to apply the full prestress force should be used to prevent multiple “bites” through the epoxy coating during post-tensioning.

Stressing Pockets—As discussed above, elimination of the central stressing pockets would greatly benefit the construction process by eliminating the need to patch the pockets before opening to traffic, although a fast-setting mixture could always be used for patching. For future projects, alternative stressing schemes should be examined to eliminate or reduce the stressing pockets.

Grouting

Joint Seal/Gasket—The most significant improvement to the grouting process, based on recommendations from the Texas pilot project, was the use of a foam gasket around each of the post-tensioning ducts between each of the panels. The gaskets were fitted into a recess cast around each duct along the female keyway when the panels arrived on site. A foam gasket, 25 mm (1 in.) thick and compressible to at least 13 mm (1/2 in.), was used at each duct. While grout leakage was not completely eliminated, it was significantly reduced over the Texas pilot project. A foam or neoprene gasket is recommended for all future applications where the tendons are to be grouted.

Grout Vents—For applications where the precast pavement may need to be opened to traffic prior to tendon grouting, it will not be possible to leave grout vent tubing protruding from the surface of the pavement. Alternative grout vents or vent locations should be examined for future projects to eliminate this potential conflict. Eliminating grout vents protruding from the surface will also improve finishing and pavement smoothness. Grout vents were spaced approximately every 9 to 12 m (30 to 40 ft) along each tendon. Intermediate vents were important to the grouting operation, and therefore it is recommended that grout vents be spaced not more than 15 m (50 ft) apart for future projects.

PROJECT COST

The final cost for the California Demonstration Project was approximately \$228,000. This cost includes all fabrication and installation costs, including traffic control. The total translates to a unit cost of approximately \$268 per m^2 (\$224 per yd^2) of finished pavement. This unit cost is significantly higher than that of the Texas pilot project, which was approximately \$194 per m^2 (\$162 per yd^2). However, the amount of pavement constructed for this project was only 853 m^2 (1,020 yd^2), whereas the Texas pilot project was 7,690 m^2 (9,200 yd^2). As with any project, there are economies of scale that decrease unit costs as quantity increases.

As might be expected, the largest portion of the cost (about 83 percent) was for panel fabrication and post-tensioning. Panel fabrication cost included significant capital investment for the panel forms. On future projects, it should be possible to reuse sideforms, significantly reducing the cost of fabrication. It should also be noted that this was an experimental project with many unknowns. As contractors and fabricators become more familiar with precast paving technology, unit costs will decrease. Also, it is anticipated that unit costs for competitively bid projects (rather than for change orders such as this project was) will be significantly less.

CHAPTER 9. SUMMARY AND RECOMMENDATIONS

SUMMARY

The precast prestressed pavement project on I-10 in El Monte demonstrated the viability of construction of prestressed precast concrete pavement. Although the project length was small (76 m [249 ft]) compared to the Texas pilot project (700 m [2,297 ft]), this project demonstrated a similar fabrication and installation rate.(2) Additionally, several new challenges were presented with this project:

- Panel placement only at night.
- Construction over LCB.
- Incorporating a change in cross slope in the top surface of the panels.
- Placement of the panels between fixed structures—the existing pavement and a sound wall.

The project also incorporated many of the recommended improvements from the Texas pilot project:(2)

- Modified keyway dimensions to ensure tighter joints between panels.
- Gaskets around the post-tensioning ducts to prevent grout leakage.
- Modified post-tensioning anchors (non-spring-loaded).
- Epoxy-coated post-tensioning strand to prevent corrosion.
- Screw-type lifting anchors, which leave a much smaller hole to patch.
- Underslab grout channels to facilitate filling voids beneath the panels.
- Temporary post-tensioning during panel assembly to ensure tighter joints.

The main objective of this demonstration project was to further evaluate and refine the precast prestressed pavement concept developed during the original FHWA feasibility study(1) while also familiarizing Caltrans and local contractors with this innovative rapid paving technique. While the project was constructed behind a concrete barrier and was not required to be opened to traffic immediately, the goal of this project was to evaluate its potential for future use under stringent time constraints, such as during overnight or weekend construction windows. Based on observations from the construction of this project, this technology appears to be suitable for such applications.

RECOMMENDATIONS FOR FUTURE CONSTRUCTION

The new challenges presented by this project demonstrated the adaptability of this precast paving technique to whatever demands a given project may have. Although this particular precast paving technique is better suited for large-scale pavement rehabilitation/construction, it can be adapted to either single-lane or full-width (up to 12 m [39 ft]) pavement construction.

Although this project was not required to be opened to traffic immediately after construction, panel placement was limited to nighttime only. Future demonstration projects should focus on limiting the entire construction operation to a nighttime or weekend timeframe, with the pavement open to traffic outside of these construction windows. Future projects should also be constructed under competitively bid contracts rather than change orders to better evaluate realistic construction costs. It is believed that as contractors and owner agencies become more familiar and comfortable with this technology, costs will become competitive with conventional fast-track concrete paving.

Figure 44. Photo. Finished precast pavement just prior to opening to traffic.

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FHWA-IF-06-010