Technical Report Documentation Page

1. Report No. FHWA/TX-03-1517-01-IMP-1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle CONSTRUCTION AND PRELIMINARY MONITORING OF THE GEORGETOWN, TEXAS PRECAST PRESTRESSED CONCRETE PAVEMENT		5. Report Date December 2002
7. Author(s) David K. Merritt, B. Frank McCullough, Ned H. Burns		6. Performing Organization Code
		8. Performing Organization Report No. 1517-01-IMP-1
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. 5-1517
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Covered Implementation Report (6/00 – 12/02)
		14. Sponsoring Agency Code

15. Supplementary Notes

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

16. Abstract

After completion of a feasibility study which examined the viability of using precast concrete panels to expedite pavement construction, FHWA and TxDOT funded an implementation project to construct a precast concrete pavement pilot project to test the concepts developed through the feasibility study.

This report will discuss the development and construction of this pilot project, which was constructed on a section of frontage road along IH 35 near Georgetown, Texas. The project incorporated the use of prestressed concrete panels to not only reduce the thickness of the pavement, but to enhance durability of the finished pavement.

This report will describe in detail all aspects of the Georgetown precast pavement project including design, panel fabrication, construction, and post-construction monitoring. Recommendations for construction of future precast concrete pavements are also included. This first precast pavement pilot project helped to develop viable design and construction techniques for use on future precast pavement projects in urban areas where expedited construction is needed the most.

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

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	168	

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

Construction and Preliminary Monitoring of the Georgetown, Texas Precast Prestressed Concrete Pavement

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Implementation Report 1517-01-IMP-1

Implementation Project 5-1517
Precast Prestressed Panels for Pavements on High-Volume Roads

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

December 2002

Disclaimers

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Acknowledgments

The researchers would like to acknowledge the TxDOT project director, Gary Graham, and all others involved with the successful completion of this implementation project including: Mark Swanlund, Suneel Vanikar, and Steve Forser (FHWA); Bill Garbade (TxDOT, Austin); John Waggner, Mark Herber, Jeff Tolson, David Boone (TxDOT, Georgetown); Ken Fults, German Claros (TxDOT, RTI); Texas Concrete Company (Victoria, Texas); Granite Construction Company (Austin); Tom D’Arcy (Consulting Engineers Group, PCMA); General Technologies, Inc. (Stafford, Texas); Dywidag-Systems International.

Research performed in cooperation with the Texas Department of Transportation and the U.S. Department of Transportation, Federal Highway Administration.

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1. Introduction

1.1 Background

With the rapid deterioration of the nation's infrastructure and the constant growth of traffic the need for rapid construction techniques is quickly gaining momentum. Pavements make up the majority of the infrastructure and therefore can have the biggest impact on the motoring public. Most pavements currently in service were not designed for the traffic they are now carrying and are in need of replacement.

Portland cement concrete pavements, when constructed properly, have proven to be durable long lasting-pavements. However, because of the nature of portland cement concrete, which requires a significant amount of time to reach adequate strength, it has not been the selected method for expedited pavement construction. Precast concrete, on the other hand, eliminates the problem of satisfactory set time required for portland cement concrete. Precast concrete pavement panels can be installed quickly and opened to traffic almost immediately, providing a durable high-performance concrete pavement with minimal traffic disturbance during construction.

1.1.1 Current Need for Expedited Pavement Construction

Traffic delays caused by construction can result in tremendous costs to the users of the roadway. Costs associated with construction include traveler time delay costs, such as lost work time, and vehicle operating costs, such as increased fuel consumption while traveling at slower speeds through work zones. There are also other costs associated with construction which are less tangible, but still significant. These include increased vehicle emissions, accidents, decreased local business access, and driver tension (Ref 1).

Based upon these considerations, it has become clear to transportation agencies that there is a great need to expedite pavement construction to minimize impacts on the motoring public. But expediting construction is not the only consideration. Building durable pavements that require minimal maintenance and will perform adequately for 40+ years is also important. Transportation agencies need viable new construction techniques that will allow them to *"get in, get out, and stay out."*

1.1.2 FHWA/TxDOT Sponsored Feasibility Study

Precast concrete pavement is not a new idea. Several different techniques have been developed in the U.S. and abroad for many different applications (Ref 2). The most prevalent use of precast concrete for pavement has been for small repairs, where a deteriorated section of pavement is cut out and replaced with precast panels. It has only been in recent years, however, that such an emphasis has been put on developing viable precast paving techniques for large-scale pavement construction.

In 1998, the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA) sponsored a feasibility study to investigate the use of precast panels for large-scale pavement construction (Ref 2). This research, conducted by the Center for Transportation Research (CTR), evaluated the feasibility of precast panels from the standpoint of design, construction, economics, and durability. The research revealed not only the feasibility of precast panels for pavement construction, but also the tremendous benefits in terms of economics and durability. The concept for precast pavement developed from this feasibility study, will be discussed more in-depth in Chapter 2.

The primary economic benefit of precast pavement will be realized through savings in user costs. Using precast panels, construction can take place during overnight or weekend construction windows, minimizing or even eliminating the impact on the motoring public during peak travel times. A sample analysis revealed that user delay costs could be reduced from $383,000 per day for conventional pavement construction (with 24-hour/day lane closures) to approximately $1,800 per day for precast concrete pavement (with night lane closures only) (Ref 2). Although the anticipated construction cost will be significantly higher, the user delay costs savings will far outweigh higher construction costs.

Durability is improved both by the use of precast concrete and the inclusion of prestressing. Casting pavement panels in a controlled environment will significantly reduce problems commonly encountered with cast-in-place concrete pavement, and the addition of prestressing will greatly reduce cracking, resulting in a durable, long-lasting pavement, which will require minimal maintenance. Furthermore, prestressing permits an effective pavement thickness that is 50–70% greater than the actual pavement thickness being placed. Hence the acceptable performance life of the pavement will be 5–8 times longer.

1.2 Implementation Study

The TxDOT/FHWA sponsored feasibility study presented a concept for precast prestressed concrete pavement. Although the concept was deemed viable by professionals in the precast and concrete paving industries, the only way to truly evaluate the concept was to implement it on actual projects. The recommendation from the feasibility study was a staged implementation strategy. Staged implementation would begin with lab testing and small-scale pilot projects. The pilot projects would be constructed on low-profile roadways, such as frontage roads or rest area driveways, which would have little or no impact on traffic. Pilot projects would be used to further evaluate and refine the precast pavement concept on actual paving projects.

Following pilot projects, larger scale rural projects would be constructed. These projects would allow the researchers to further streamline the construction process on projects constructed under more stringent time constraints on roadways with significant traffic volumes. Finally, staged implementation would conclude with construction under the most stringent time constraints, on urban freeways or intersections, where construction delays during peak travel times must not occur.

Based on this staged implementation strategy, the FHWA funded an Implementation Study in Summer 2000 to allow the researchers to conduct laboratory testing and construct two separate pilot projects. The implementation study provided funds for the researchers to complete laboratory testing and provide design and construction support for two pilot projects. The researchers were required to work with state DOTs to locate possible pilot project locations and provide support for the development and construction of test sections. Based on the outcome of the laboratory testing and pilot projects, the researchers would develop recommendations for construction of future precast concrete pavements.

1.3 Georgetown Precast Pavement Pilot Project

Upon initiation of the FHWA sponsored Implementation Study, CTR researchers met with the Austin District Engineer of TxDOT to present the precast pavement concept and discuss possible applications in the Austin District. The Austin District Engineer located a possible site for a test section in the Austin area. A 13-mile section of IH-35 just north of Georgetown, was being widened from four to six lanes. Part of the widening included reconstruction of a section of frontage road, which provided an ideal location for a precast pavement pilot project. A change order was initiated by the Austin District to replace the asphalt concrete pavement along the frontage road with precast prestressed concrete pavement. Although the project was constructed on the frontage road, the design was completed as if the pavement was to be placed on the mainlanes, allowing for the Austin District to evaluate precast pavement for possible future use on an urban freeway.

1.4 Report Objectives

The primary objective of this report will be to describe the precast prestressed pavement project constructed by TxDOT near Georgetown, Texas. This includes a description of the precast pavement concept, design for the pilot project, panel fabrication, laboratory testing, pavement construction, and monitoring. This report will also discuss recommendations for future precast pavement construction. The 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 and assembly, base preparation, post-tensioning, and grouting.

Chapter 3 presents the details of the Georgetown pilot project, including the scope of application and the project layout.

Chapter 4 presents the design for the Georgetown pilot project. This includes the design considerations for precast prestressed concrete pavement, the design procedure used for the Georgetown pilot project, and the final design recommendations.

Chapter 5 discusses the laboratory testing and trial assemblies that were completed prior to construction of the Georgetown pilot project.

Chapter 6 discusses the fabrication of the precast panels for the Georgetown pilot project. This includes the mix design, panel details, casting and curing procedures, and handling and storage. This chapter will also discuss the challenges encountered during the casting process.

Chapter 7 discusses the construction of the pavement on site. This includes base preparation, transportation, panel placement, post-tensioning, and grouting. This chapter also discusses the challenges and problems encountered during panel placement.

Chapter 8 presents the instrumentation and monitoring of the Georgetown pilot project. This includes the temperature instrumentation, monitoring of horizontal and vertical slab movements, and a long-term monitoring plan. This chapter will also present data from preliminary monitoring of the pavement.

Chapter 9 presents the condition survey, completed prior to opening to traffic, and post-construction testing.

Chapter 10 presents recommendations for future construction. This includes changes recommended by the researchers to aspects of the Georgetown pilot project to further simplify and streamline fabrication and construction procedures.

Chapter 11 presents conclusions drawn from the construction of the Georgetown pilot project and recommendations for the next step in precast pavement implementation.

1.5 Scope of Report

This report will discuss the implementation of precast pavement technology, developed through previous feasibility study, on a pilot project near Georgetown, Texas. The application was for large-scale full-depth pavement placement, as opposed to smaller repairs. Although the pilot project discussed in this report entailed placement of new pavement over a prepared base material, the application is valid for removal and replacement of existing pavement also.

2. Precast Concrete Pavement Concept

2.1 Introduction

As discussed in Chapter 1, the concept for the precast pavement constructed on the frontage road of IH-35 near Georgetown, Texas, was developed through a TxDOT/FHWA sponsored feasibility study. The concept was developed and evaluated by professionals in the precast concrete and concrete pavement industries prior to implementation in this first pilot project.

2.1.1 Full-Depth Panels

The concept developed from the feasibility study utilizes full-depth precast panels. Full-depth panels are the most efficient solution because an additional paving operation to place an asphalt or bonded concrete riding surface is both cost-and time-prohibitive. It was also believed that a smooth enough riding surface could be obtained with full-depth panels and occasional diamond grinding.

Two key factors that must be considered when using full-depth panels are: (1) base preparation and (2) vertical alignment of adjacent panels to achieve satisfactory ride quality over the joints. Base preparation will be discussed in more detail below. With regard to vertical alignment between adjacent panels, the proposed concept features continuous shear keys cast into the edges of the panels which will interlock the panels as they are set in place. Although shear keys along the panel edges require strict casting tolerances (possibly even match-casting), the panels can be set in place rapidly without the need for additional measures to level-up adjacent panels.

2.1.2 Prestressed Pavement

The concept incorporates prestressing into the pavement through both pretensioning and post-tensioning. The panels are pretensioned in the transverse direction (perpendicular to the flow of traffic) during fabrication, and post-tensioned together after placement in the longitudinal direction (parallel to the flow of traffic). The primary reasons for prestressing are increased durability and decreased pavement thickness. Perhaps the best example of the durability benefit of prestressed pavement is a 6 in. cast-in-place prestressed pavement developed by CTR and constructed on a 1-mile section of IH-35 near West, Texas (Ref 3). After more than 17 years, this 6 in. pavement is still in excellent condition despite heavy volumes (27%) of truck traffic. The expansion joints show no distresses and the ride quality is very good.

The other major benefit of incorporating prestress is a reduction in slab thickness. Cracking in concrete pavement, which eventually leads to pavement failure, is caused by tensile stresses from wheel loading in conjunction with environmental conditions. In general, the thicker a pavement is, the lower the stress in the pavement from wheel loading. The same effect is achieved by inducing a compressive stress in the pavement from prestressing. The additional compressive stress must first be overcome before tensile stresses develop and cracking occurs. Therefore, a thin prestressed pavement can be designed equivalent to a much thicker pavement by reducing the stresses in the thinner pavement through prestressing. This is very beneficial for situations where an older thinner pavement must be replaced by a thicker pavement, but overhead clearance will not permit the thicker pavement.

During the development of the West, Texas, prestressed pavement, it was discovered that pavements constructed in other states with only longitudinal prestress tended to develop longitudinal cracking. This is caused by a Poisson effect, i.e., stresses due to wheel loading are not unidirectional, they develop in both the longitudinal and transverse directions. Prestress in both the longitudinal and transverse directions is therefore essential, and is incorporated into the precast pavement concept through a combination of transverse pretensioning and longitudinal post-tensioning.

2.2 Panel Assembly

Figure 2.1 shows a typical precast pavement assembly. There are essentially three types of panels that make up a precast prestressed pavement: base panels, joint panels, and central stressing panels, shown individually in Figures 2.2–2.4. As described above, 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) to post-tension the panels together after they are set in place. The panels all incorporate keyed joints to ensure proper vertical alignment between panels as they are assembled, as described previously.

The panels are placed transverse to the flow of traffic, incorporating both traffic lanes and shoulders, if possible, in the panel. Similarly to providing tied shoulders for conventional concrete pavement, inclusion of the shoulders in the precast panels greatly reduce edge stresses caused by traffic traveling on the edge of the pavement. After placement of the panels, the pavement is post-tensioned in sections, with one section consisting of the panels between the expansion joints in the joint panels. The length of section will depend on the number of base panels placed between the central stressing panels and joint panels.

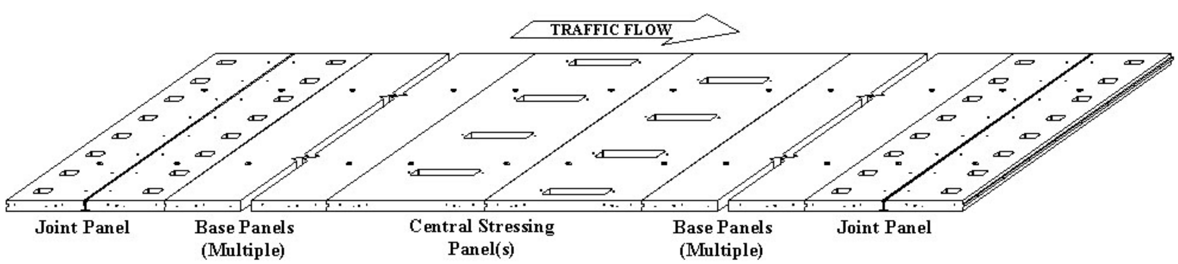


Figure 2.1 *Typical precast prestressed pavement assembly*

2.2.1 Base Panels

The base panels, shown in Figure 2.2, are the "filler" panels between the joint panels and central stressing panels. The number of base panels between the joint panels and central stressing panels will depend on the post-tensioned slab length (between expansion joints). Figure 2.2 shows the details of the base panels including the continuous shear keys along the panel edges, the post-tensioning ducts spaced equally across the panel, and the pretensioning strands equally spaced across the width of the panels. The "edge sleeves" shown in Figure 2.2 are used to pull the panels together as they are set in place, and will be described in greater detail in Chapter 5.

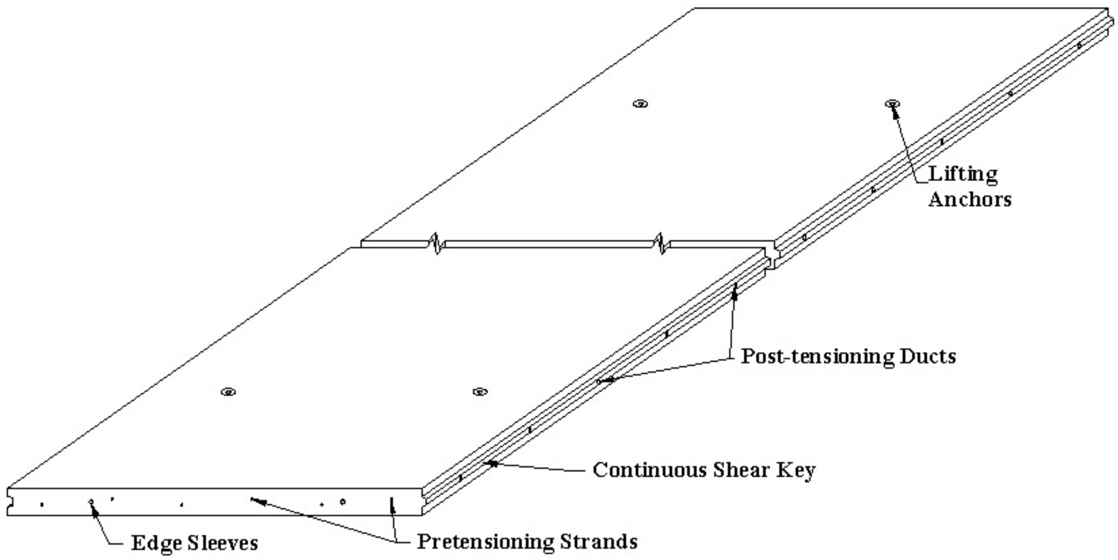


Figure 2.2 *Typical Base Panel*

2.2.2 Joint Panels

The joint panels, shown in Figure 2.3 contain an armored expansion joint, similar to that used for bridge decks. Similarly to long post-tensioned bridges, a precast, prestressed concrete pavement will expand and contract significantly with daily and seasonal temperature cycles. The armored expansion joints will accommodate this movement while providing load transfer across the joint. The expansion joint detail, described in more detail in Chapter 6, is based on the expansion joints used for the West, Texas, prestressed pavement, mentioned previously (Ref 3). This joint detail has shown virtually no distress after 17 years of heavy traffic on IH-35.

The joint panels also contain the post-tensioning anchorage. The post-tensioning anchors are spring-loaded anchors with self-seating wedges that automatically grip the post-tensioning strands as they are fed into the anchor. This permits the anchors to be cast as self-contained units into the joint panels and allows the strands to be fed blindly into the anchors. The anchors are bolted to the armored expansion joints to ensure transfer of the prestress force to the armored joint. Small access pockets (6 in. x 12 in.) cast into the joint panels permit inspection of the post-tensioning strand prior to insertion into the self-locking anchors. Grout inlets/vents are also cast into the joint panels just in front of the post-tensioning anchors for venting or pumping grout after the strands are tensioned.

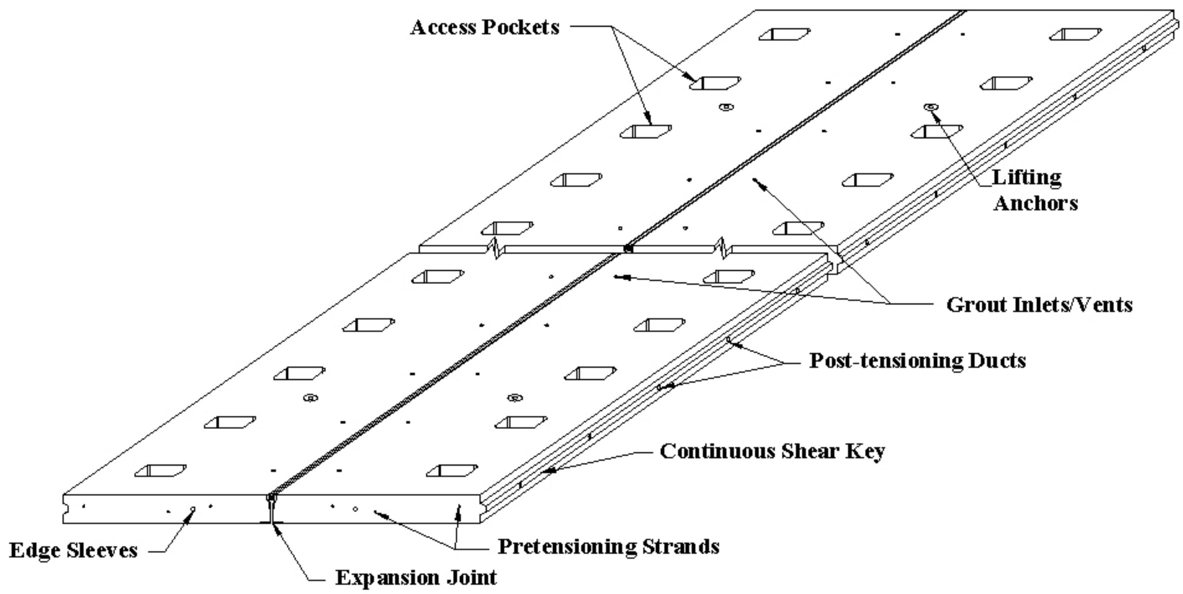


Figure 2.3 *Typical Joint Panel*

2.2.3 Central Stressing Panels

The third type of precast panel is the central stressing panel, shown in Figure 2.4. The central stressing panels are similar to the base panels, with the addition of large full-depth pockets (48 in. x 8 in.) cast into the panels at every post-tensioning duct. To prevent weakening of the panel, the pockets are alternated between two adjacent central stressing panels, depending upon the strand spacing, as shown in Figure 2.1. The purpose of the pockets is to allow for post-tensioning to be completed from the center of the slab rather than at the post-tensioning anchors, as is typically done with monostrand post-tensioning. This permits a more continuous pavement placement operation, as access to the end anchorage (in the joint panels) is not needed in order to tension the strands. The post-tensioning strands coming into the stressing pockets from either side of the slab are joined in the pocket using a coupler similar to that shown in Figure 2.5. A monostrand stressing ram is then used to tension the strands through the coupler. After stressing is complete, the stressing pockets are filled with a fast-setting concrete or temporarily covered to allow traffic onto the pavement immediately. Grout inlets/vents cast into the panels on either side of the stressing pockets allow for grouting the strands after filling the stressing pockets.

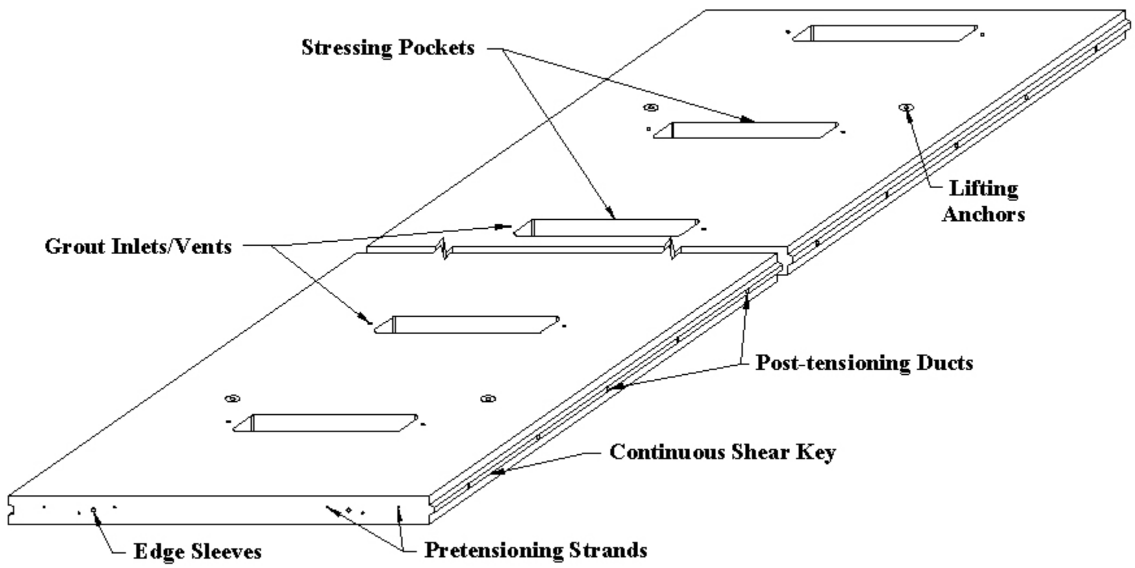


Figure 2.4 *Typical Central Stressing Panel*

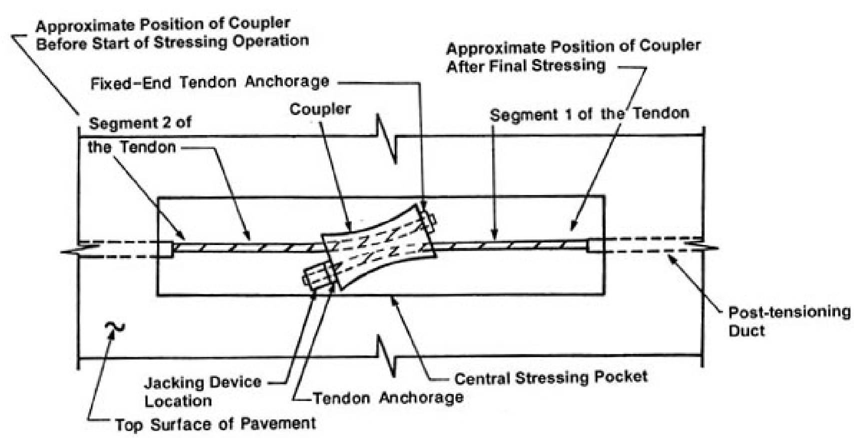


Figure 2.5 *Coupler used to join post-tensioning strands together in the stressing pockets*

2.2.4 Partial-Width Panels

In most pavement rehabilitation projects, it is not possible to replace the full roadway width at one time using a single precast panel. This requires the use of "partial-width" panels, placed adjacent to each other and tied together transversely, to achieve the full pavement width. Partial-width panels are similar to full-width panels, with the additional post-tensioning ducts cast into the panels in the transverse direction, as shown in Figure 2.6. A single duct is cast into each base panel, and two ducts are cast into each of the central stressing panels and joint panels on either side of the stressing pockets and expansion joint.

One important consideration with respect to the transverse post-tensioning duct is differential movement of adjacent slabs or slight misalignment of the transverse ducts. To accommodate this, the transverse post-tensioning ducts are flat, multi-strand ducts. A four-strand duct is used for only two post-tensioning strands. This allows for misalignment of adjacent slabs up to 1–2 in. Once the transverse post-tensioning has been completed, adjacent slabs will expand and contract together, eliminating the problem of differential movement.

The transverse post-tensioning strands are anchored at the outside edges of the panels using standard dead-end post-tensioning anchors. The strands are also grouted in place to provide corrosion protection where they cross the longitudinal joint between adjacent slabs.

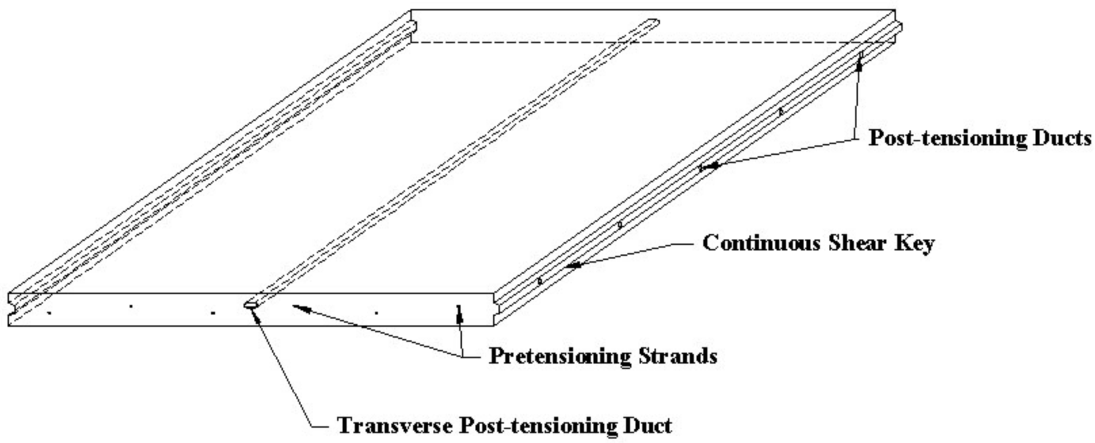


Figure 2.6 *Typical partial-width base panel showing the additional transverse post-tensioning duct*

2.3 Base Preparation

Base preparation is one of the most important considerations for full-depth precast panels. The panels should be placed on a flat, smooth surface which will properly support the panels. Imperfections in the base material can cause voids beneath the panels or prevent the keyed panel edges from matching up. Additionally, provision must be made to reduce the friction between the bottom of the pavement and the base material. This will allow the pavement to expand and contract with less resistance, reducing frictional restraint stresses in the pavement.

2.3.1 Leveling Course

Several different techniques for providing a smooth and flat surface for the precast panels were investigated during the original feasibility study (Ref 2). The method that appeared to be the most viable was placement of a thin, 1–2 in., asphalt pavement leveling course. An asphalt leveling course can be placed well in advance of the precast panels and opened to traffic prior to placement of the panels. An asphalt leveling course can also be placed fairly quickly and economically to relatively strict tolerances. Profile measurements of a newly placed asphalt pavement analyzed during the feasibility study revealed that asphalt pavement can be placed (under normal conditions) smooth enough that voids beneath the precast panels will be minimal.

Another advantage of a thin asphalt leveling course is that it is flexible, and the weight of the precast panels will cause the asphalt to conform to the bottom of the panels. This will minimize voids and flatten high spots in the leveling course.

An asphalt leveling course may not be suitable for all precast pavement applications, however. For the removal and replacement of a short section of concrete pavement, it may not be practical to mobilize asphalt paving equipment to place a short section of leveling course. This will be discussed more in-depth in Chapter 10. However, the scope of the Georgetown precast pavement permitted the use of an asphalt leveling course, and provided an ideal application for testing its use.

2.3.2 Friction Reduction Layer

The second consideration in base preparation for a precast pavement is provision of a friction reducing layer between the leveling course and the precast pavement. As mentioned previously, long prestressed pavement slabs will expand and contract significantly with daily and seasonal temperature cycles. If these movements are prevented by frictional resistance between the bottom of the pavement and the leveling course, very high stresses will develop in the pavement, depending upon how much movement is restrained. This is particularly critical when the slab contracts as the temperature of the concrete decreases at night or during a winter temperature cycle, as tensile stresses will develop from the frictional restraint.

Several different friction reducing materials have been tested for suitability in reducing frictional restraint (Refs 4, 5). One of the most effective and economical solutions has been a single layer of polyethylene sheeting. Polyethylene sheeting is not only effective in reducing friction by acting as a type of bond breaker between the pavement and leveling course, but is practical from the standpoint of economics and constructibility. Polyethylene sheeting was used successfully in the construction of the West, Texas, cast-in-place prestressed pavement (Ref 3), and should prove to be just as effective for precast pavement.

2.4 Post-Tensioning

As mentioned previously, the primary purpose of post-tensioning is to reduce slab thickness and improve durability. The post-tensioning also serves to tie all of the panels together so they act as a continuous slab. After placement of a section of panels over the leveling course and polyethylene sheeting, post-tensioning is completed. Each of the longitudinal post-tensioning strands are inserted into the ducts at the central stressing pockets and fed by hand into the post-tensioning anchors in the joint panels. The post-tensioning strands coming in to the stressing pockets from either end of the slab are then coupled together using an anchor similar to that shown in Figure 2.5. A monostrand stressing ram is then used to tension the strands by pulling on one strand while reacting against the other strand, thereby tensioning both strands at once. The post-tensioning sequence should start 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. If time constraints do not permit, post-tensioning can be completed during a subsequent construction window. 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.

2.5 Grouting

Following post-tensioning (and filling the stressing pockets), the final step in the precast pavement construction process is to grout the post-tensioning tendons. The primary purpose for grouting is to provide an extra layer of corrosion protection for the post-tensioning strands. This 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.

Grout is pumped into the ducts at inlets/vents located along each duct. At minimum, one inlet/vent is located at the post-tensioning anchors in the joint panel, and one next to the pockets in the central stressing panel. Additional intermediate vents in the base panels permit monitoring of the movement of grout in the duct, and provide alternative inlets if the inlets at either end are blocked.

Similarly to post-tensioning, grouting does not have to be completed before the pavement is opened to traffic. Grouting can be done during any subsequent construction window. It is essential, however, that all of the post-tensioning strands are stressed and the pockets are all filled prior to grouting. It is also essential that an adequate seal is provided around each post-tensioning duct between each panel to prevent grout from leaking and crossing into other ducts.

3. Georgetown Pilot Project

3.1 Project Scope

As mentioned in Chapter 1, the purpose of the Georgetown pilot project was to evaluate the concept for precast prestressed pavement developed through the TxDOT/FHWA sponsored feasibility study. The concept was presented to the TxDOT Austin District Engineer to generate interest for a possible pilot project in the Austin area. The District Engineer subsequently located possible locations for such a pilot project.

3.1.1 Location

The location selected for the Georgetown pilot project was along the east (northbound) frontage road of IH-35 between Airport Road and State Highway 195, just north of Georgetown, Texas. This section of frontage road was to be reconstructed as part of a much larger project involving the widening of the main lanes of IH-35. A new bridge was to be constructed, to raise the frontage road out of the Berry Creek floodplain, and the approaches on either side of the bridge were to be raised to meet the elevation of the bridge.

The frontage road was ideal for a pilot project location for three reasons. First, the section of frontage road could be closed to traffic during construction. This allowed for more flexibility with construction so that any problems encountered, which would delay the project, would not have a detrimental effect on traffic. Although the ultimate application for precast pavement will be on urban intersections and freeways, where traffic delays from construction must be eliminated, the purpose of this first pilot project was to test and refine the construction procedures without severe time constraints in order to streamline the process for future project.

Second, the frontage road geometry contained no horizontal curves or superelevations, and only a slight vertical curve at the south end of the reconstruction. Although these are issues which must eventually be addressed, a simple roadway geometry allowed for the focus of the project to remain on streamlining the construction process. Third, the frontage road location was ideal because it will experience a significant amount of traffic. A new exit ramp, (off IH-35) constructed just south of the precast pavement section along with a truck stop and State Highway 195 at the north end of the project should result in a significant amount of traffic on the pavement. Additionally, this section of frontage road will also become part of the interchange for State Highway 130 and IH-35, which will carry significant volumes of truck traffic.

3.1.2 Field Change

Although the plans had already been developed for the frontage road reconstruction, work had not yet begun, and incorporating precast pavement was a matter of submitting a field change order. Originally, the plans called for 2 in. of hot-mix asphalt pavement (Type C) over 12 in. of compacted base material over the embankment fill material. This was changed to 8 in. of precast-prestressed concrete pavement over 2 in. of hot-mix asphalt pavement (leveling-course) over the embankment fill material. The details of the pavement structure will be discussed more in-depth in Chapter 4.

In addition to the pavement structure, the pavement drainage was changed also. Originally, the plans called for a crown slope on the finished pavement, with a 2% slope on either side of the centerline. This was changed for a uniform cross-slope of 2%, sloping to the outside edge of the pavement. The pavement slope was changed to permit placement of panels which were the full width of pavement. With a crown slope, either two separate panels on either side of the centerline, or precast panels with a slight camber would have been required. Although this may be required on future projects, a uniform cross-slope was specified to simplify construction.

3.1.3 Pilot for Primary Lanes

As stated previously, one purpose of this first pilot project was to simulate what might be done on future projects constructed on the main lanes of urban freeways. Although only 2 in. of hot-mix asphalt pavement were needed to withstand the traffic experienced by the frontage road, the design was completed for main lane traffic loading. Although this significantly increased the cost of the frontage road reconstruction, it allowed TxDOT to visualize what would be required for future projects.

3.2 Project Layout

The project layout includes the roadway geometry, slab width, and slab length. Although the geometry was predetermined from the original plans, the slab length and width were decided by TxDOT based on the goals for this pilot project.

3.2.1 Geometry

The geometry of the frontage road was established during the initial plan development for the frontage road reconstruction. As stated in Section 3.1.2, the only geometric changes were to the slope of the finished pavement, changing from a crown slope to a uniform cross-slope.

One reason this section of frontage road was selected for a pilot project was because of a simple geometric layout. There were no horizontal curves or superelevations on the section, and only a minimal vertical curve on the south end of the project. Approximately 2,310 ft of precast pavement was placed on the frontage road; 1,230 ft north of the bridge, and 1,080 ft south of the bridge. The precast panels on the north side of the project were placed on a flat (0% slope) grade. The panels on the south side of the project, however, were placed on a slight vertical curve, transitioning from a 0% slope to a 5.4% slope, starting 180 ft south of the bridge.

The concern with placing precast panels on a vertical curve is the possibility of creating voids beneath the panels as well as a gap at the bottom of the joints between panels. These issues are illustrated in Figure 3.1.

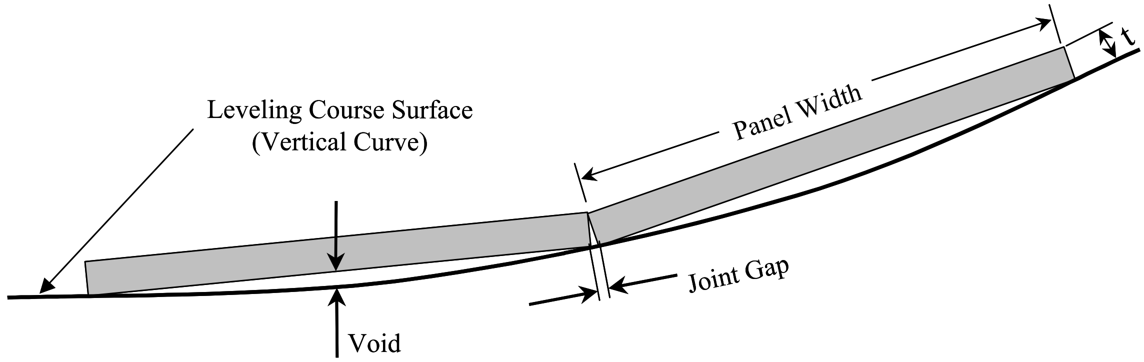


Figure 3.1 *Possible problems created when placing precast panels on a vertical curve*

To address these concerns, the depth of void beneath the panels and the gap at the bottom of the joint were determined using the vertical curve properties for the south end of the project. Figure 3.2 shows the calculated void depth and joint gap width for precast panels of varying width and thickness.

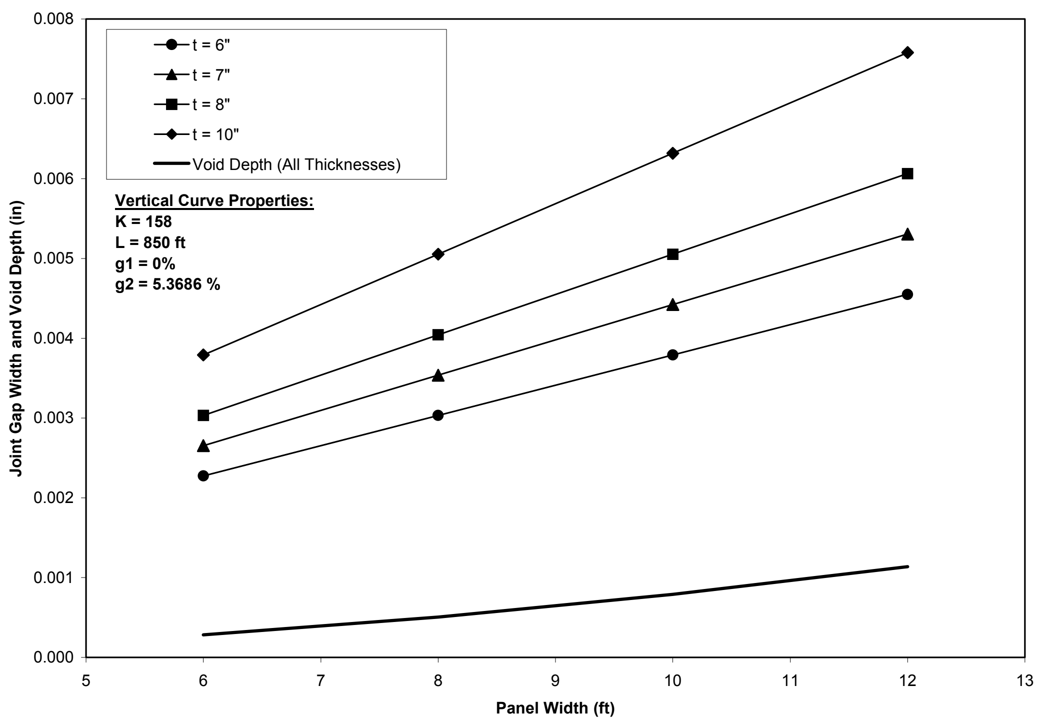


Figure 3.2 *Void depth and joint gap widths for vertical curve on Georgetown pilot project*

As Figure 3.2 shows, the depth of void created beneath the precast panels when placed on a vertical curve is less than 0.0012 in. for all panel widths and thicknesses. This void depth is essentially negligible, and will not affect the performance of the finished pavement. The joint gap widths, likewise, are very small (less than 0.008 in.) for all panel widths and thicknesses. This gap is also negligible and should easily be accommodated by the keyways in the panel joints.

3.2.2 Slab Width

The width of the frontage road pavement, from the original plans, is 36 ft. This corresponds to two 12 ft traffic lanes, an 8 ft outside shoulder, and a 4 ft inside shoulder. Based upon the concept for precast pavement, presented in Chapter 2, the precast panels were to be oriented transverse to the flow of traffic. To avoid the need for separate tied shoulders, it was decided to include the shoulders with the traffic lanes as part of the precast pavement. This required panels which would span the full 36 ft roadway width. To accomplish this, it was decided to use both "full-width" and "partial-width" (described in Chapter 2) panels. The full-width (36 ft) panels were placed north of the bridge, and the partial-width panels south of the bridge, as shown in Figure 3.3. The partial-width panels consisted of 16 ft and 20 ft panels placed next to each other to achieve the full 36 ft roadway width. The longitudinal joint between the 16 ft and 20 ft panels corresponded to the centerline of the roadway to minimize traffic loading on the longitudinal joint.

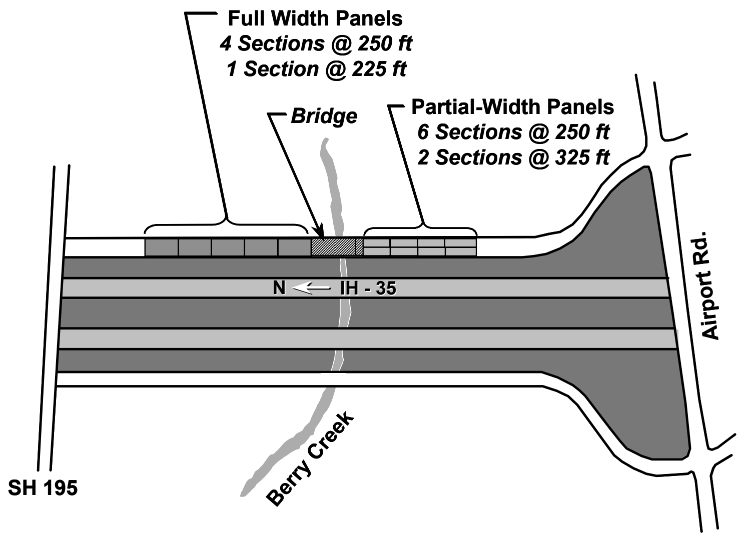


Figure 3.3 *Layout of the full width and partial width panels on the frontage road*

3.2.3 Slab Length

The slab length (between expansion joints) for the Georgetown pilot project was a critical design aspect. The advantage of longer slab lengths is that fewer expansion joints are required, which is beneficial for both the initial construction costs and life-cycle maintenance costs. The downside of longer slab lengths is that they expand and contract more than shorter slabs, requiring expansion joints that can accommodate large movements. An additional consideration is ensuring that all of the precast panels for a single slab can be placed in the allotted construction window. When using a slow-setting segmental bridge epoxy in the panel joints, all panels must be placed and post-tensioned within 24 hours.

Based upon these considerations, as well as previous experience with the cast-in-place post-tensioned pavement near West, Texas (Ref 3), it was decided that a standard post-tensioned slab length (between expansion joints) of 250 ft would be used. To evaluate the feasibility of placing a slightly longer slab length, however, a 325 ft long slab was also incorporated in order to meet the limits of the project.

After the slab length was selected, the next decision was how wide each precast panel would be. This determined how many precast panels would need to be placed for each slab. When deciding the width of the precast panels, fabrication, transportation, and on-site handling are taken into consideration. From a production standpoint, wider panels (i.e., 10–12 ft) are desirable because more pavement (linear feet) can be cast each time. However, from a transportation standpoint, fewer wide panels can be transported on each truck due to weight restrictions and may require special wide-load permits. With smaller (6 ft) panels, however, more panels can be transported on each truck. Finally, from a handling standpoint, smaller panels are easier to handle on-site, but more panels are required, substantially increasing installation time.

Based on these considerations, production (fabrication and installation on site) was the most important factor. Consequently a panel width of 10 ft was selected for all of the precast panels. Although only one full-width panel could be transported on each truck, more linear feet of pavement could be cast and installed in a shorter period. With a 10 ft panel width, twenty-five panels had to be placed and post-tensioned within 24 hours for each of the 250 ft slabs, which appeared reasonable to the contractor.

3.3 Partnering and Project Coordination

Partnering and project coordination were critical aspects of the Georgetown precast pavement pilot project. As this was an experimental project, and the first of its kind in the world, neither TxDOT nor the contractor had any experience with this type of construction. Therefore, communication between the researchers, TxDOT, and the contractor(s), was essential for every step of construction.

Approximately ten meetings with TxDOT, CTR, and the contractor(s) took place prior to construction. Initial meetings were used to familiarize everyone involved with the scope and the goals of the project. Subsequent meetings were then held to work out the details for the precast panels, fabrication, and installation on site.

Present at each meeting were the general contractor, TxDOT personnel from the Georgetown Area Office, TxDOT pavement design and materials personnel, and CTR personnel. Once the precast supplier and post-tensioning supplier had been selected, they also attended the meetings. In addition to those directly involved, other precast concrete and post-tensioning consultants were also present at several of the meetings.

Coordination of each step of the project with the contractor and suppliers was carried out through the TxDOT Georgetown Area Office. Design and construction assistance were also provided by CTR throughout the project. Any problems that arose over the course of the project were worked out through the Georgetown Area Office and CTR.

4. Design

4.1 Design Considerations

There are several factors which must be considered for the design of a precast prestressed concrete pavement. These design considerations primarily deal with prestress requirements, which are directly related to the thickness design, and slab movement, which is related to both the thickness design and expansion joint design. These design considerations are discussed in-depth in the original feasibility study (Ref 2), and are summarized below.

4.1.1 Wheel Load Repetition

Wheel loads on concrete pavements cause tensile stresses to develop at the bottom of the slab. Because concrete is inherently weak in tension, these stresses become the governing factor for thickness design. To determine the effect of wheel load repetitions, the magnitude and occurrence of various axle loadings are converted to the total number of passes of the equivalent 18 kip single axle load (ESAL). The fatigue caused by ESAL repetitions can then be predicted using various empirical equations. One of the most difficult aspects of evaluating wheel load repetitions, however, is predicting the number of ESAL applications the pavement will experience over its design life. The growth of ESAL applications is generally exponential as traffic volumes increase, percentage of truck traffic increases, and trucks become heavier. Fortunately, several traffic volume prediction models have been developed to more accurately predict the number of ESAL applications.

4.1.2 Temperature

Temperature has a significant effect on the design of prestressed concrete pavements. Because prestressed pavements generally consist of long slab lengths (between expansion joints), a significant amount of horizontal movement (expansion and contraction) and vertical movement (curling) is experienced during normal daily and seasonal temperature cycles. Expansion and contraction movements are resisted by friction between the bottom of the pavement and base material. This frictional resistance causes tensile and compressive stresses in the slab which must be accounted for. Horizontal slab movement also affects the width of the expansion joints which affects the ride quality of the pavement.

Vertical slab movement (curling) is cause by temperature gradients over the depth of the slab. During the warmest part of a daily temperature cycle (usually later afternoon), the top of the slab is warmer than the bottom of the slab, causing the ends of the slab to curl downward. However, because of the weight of the slab, which resists curling movement, tensile stresses are generated in the bottom of the slab. Conversely, during the coolest part of a daily temperature cycle (usually early morning), the bottom of the slab is warmer than the top, causing the ends of the slab to curl upward, generating tensile stresses in the top of the slab. A combination of all of these temperature effects must be considered in the design of a prestressed concrete pavement.

4.1.3 Moisture

Moisture has a similar effect as temperature in that moisture gradients cause curling and warping of the pavement 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 curling 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. This type of moisture gradient is inherent in conventional cast-in-place pavements as moisture becomes trapped in the bottom of the slab after casting. When this moisture gradient is large enough while the concrete is still gaining strength, the slab tends to retain the curl caused by the moisture gradient; this is referred to as "built-in" curl. One advantage of precast concrete is the fact that precast panels are able to "dry out" as they are stored after casting. This ensures a minimal moisture gradient in the precast panels prior to being installed, thereby reducing built-in curl.

Another consideration with respect to moisture effects is shrinkage. For cast-in-place prestressed pavements shrinkage can cause prestress losses, requiring additional prestress to compensate for these effects. Precast pavement panels, which are post-tensioned well after the majority of shrinkage has occurred, do not require additional prestress to compensate for shrinkage.

4.1.4 Slab-Base Interaction

As mentioned previously, frictional resistance to horizontal movement during daily and seasonal temperature cycles is developed between the bottom of a prestressed pavement and the base. The frictional resistance at the interface is the result of three components: bearing, adhesion, and shear (Ref 6). Bearing force is the weight of the slab on the subbase. Its direction is dependent on the base surface roughness, moisture condition, and temperature. Adhesion is the attraction the slab experiences relative to its base. Its magnitude is also dependent on the moisture condition and temperature of the base. The shear component is dependent on the rubbing characteristics of the two materials in contact when movement begins. It is also dependent on the magnitude and direction of the bearing component. It is possible for the combined forces of these three components to be such that the frictional restraint at the interface exceeds the internal strength of the base layer, resulting in failure of the base (Ref 5).

With precast pavement, the bottom of precast panels will be smooth, unlike that of a cast-in-place prestressed pavement, where the concrete conforms to the roughness of the base surface. The precast panels will also be rigid, spanning small voids in the base material, thereby reducing the contact between the bottom of the slab and supporting layer. These effects will result in a reduction of the shear effect at the slab-base interface. Because long-term movements from seasonal temperature changes occur at minute daily rates, as compared to daily temperature movements, they take place without significant frictional resistance. Frictional resistance to movements from daily temperature changes, however, produces stresses in the slab. 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 (Ref 7).

Horizontal movement of concrete pavement slabs caused by temperature variation decreases from a maximum at the ends of the slab to zero movement at the center, as shown in Figure 4.1(a). Likewise, frictional resistance also decreases from a maximum at the ends to zero at the center. The result is tensile stresses (for slab contraction) increasing from zero at the ends to a maximum at the center. In a prestressed (post-tensioned) pavement, frictional resistance has another effect. Frictional resistance causes a decrease in the amount of compressive stress transferred to the concrete from post-tensioning, as shown in Figure 4.1(b). The reduction of post-tensioning force along the slab requires that a higher post-tensioning force be applied at the ends of the slab.

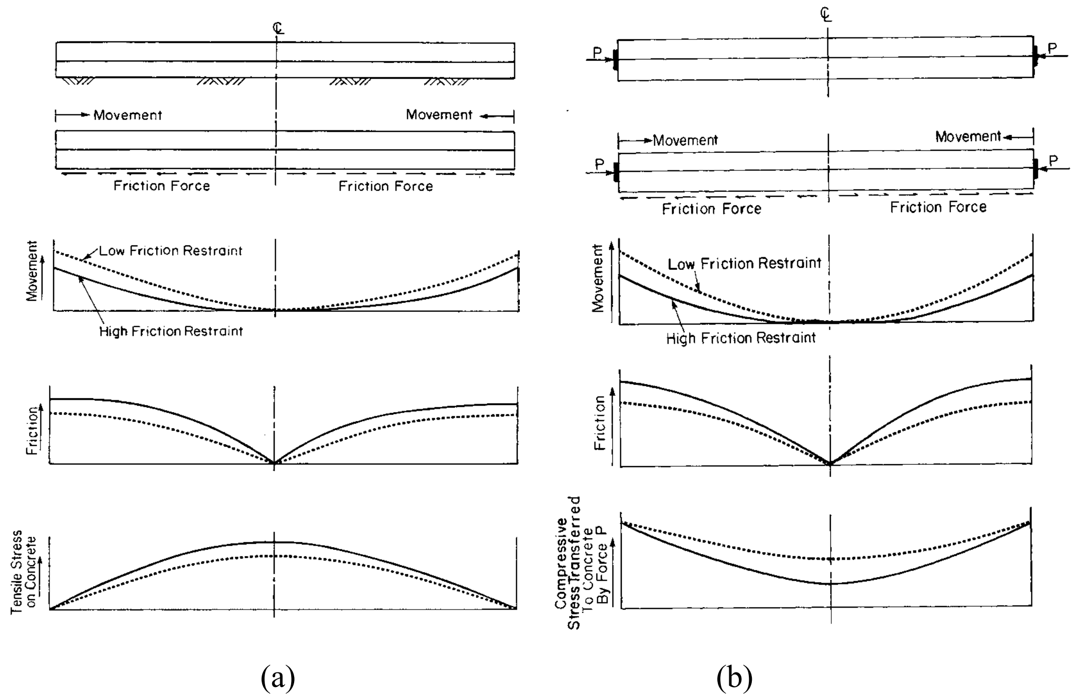


Figure 4.1 *Effects of frictional restraint on (a) normal PCCP slab, (b) prestressed PCCP slab*

To reduce the effects of frictional resistance, a friction-reducing membrane is placed beneath prestressed pavements to lower the coefficient of friction between the pavement slab and supporting base. The three main considerations in selecting a friction-reducing medium are (1) efficiency in reducing restraint, (2) practicality for road construction, and (3) economics (Ref 6). Previous research and experience have found a single layer of polyethylene sheeting to be a practical friction-reducing medium for meeting these requirements (Refs 4, 8).

4.1.5 Prestress Losses

Prestress losses are an important design consideration for post-tensioned (precast) pavements as they can greatly affect the amount of prestress force required to achieve an equivalent thickness design. Losses of 15–20% of the applied prestress force can be expected for a carefully constructed post-tensioned concrete pavement (Ref 9). The factors that contribute to prestress losses include:

- 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
- frictional resistance between the slab and base material

Extensive testing has produced methods to reliably predict the effects of these factors. A detailed discussion of each of these factors can be found elsewhere (Ref 9).

4.2 Design Procedure

The first step in the design of the Georgetown precast pavement was to determine the minimum prestress required for the selected slab thickness based on fatigue considerations. The slab thickness was selected based on the sample pavement design completed during the feasibility research, which revealed that an 8 in. slab thickness was attainable from the standpoint of required prestress (Ref 2). The fatigue effects were then computed using an accepted fatigue equation developed by Taute (Ref 10), which relates the number of 18 kip ESALs to the ratio of concrete flexural strength to tensile stress at the bottom of the pavement.

The second step in the design procedure was to determine the prestress requirements for the selected slab thickness based on environmental effects and varying slab lengths. In this step, the actual prestress applied to the pavement (during post-tensioning) was determined such that the minimum prestress requirements for fatigue were met. Design for environmental effects was used to determine the maximum stress in the pavement from simultaneous environmental (temperature) and wheel loading.

The final step in the design procedure was to check the anticipated expansion joint widths for typical summer and winter conditions for the varying slab lengths. This helped to determine the maximum permissible slab length. The joint widths were checked to ensure they would never be fully closed and never open more than 4 in. under extreme temperature conditions typical to Georgetown. The anticipated joint movement was also used to determine the required expansion joint width at panel placement based on anticipated ambient temperatures.

4.3 Design for Fatigue

Fatigue is the governing factor for the design of concrete pavements. As mentioned previously, fatigue is caused by wheel load repetitions under traffic. The effect these wheel loads will have depends on the state of stress of the pavement from environmental effects (such as curling and frictional resistance) and the support structure beneath the pavement. This section will discuss thickness design for fatigue based on wheel loading for a given pavement support structure.

4.3.1 Equivalent Thickness

Equivalent thickness design is used to compute the required prestress to achieve a precast pavement with an equivalent design life (in terms of fatigue) to a conventional concrete pavement. Continuously reinforced concrete pavement (CRCP) was selected for the equivalent thickness design comparison primarily because CRC pavements are commonly being constructed on major highways in Texas. For the Georgetown precast pavement, a 14 in. CRCP currently under construction on IH-35 in Hill County (Ref 11) was selected as the control section for the equivalent design. For the same design criteria and support structure a pavement thickness of 15 in. would be required for a jointed reinforced concrete pavement (JRCP). Although the support structure for the frontage road in Georgetown is different from that in Hill County and may result in a slightly different pavement design, a 14 in. pavement thickness is typical of that being constructed on interstate main lanes in Texas and should provide a realistic equivalent pavement design.

4.3.2 Support Structure

The support structure for the Georgetown precast pavement consists of embankment fill material for the subbase beneath a 2 in. asphalt leveling course. This support structure was used to determine the tensile stresses in the bottom of the equivalent CRC pavement using the loading shown in Figure 4.2. This loading condition represents the wheel load from dual wheels on a single axle truck. The properties of the different layers of the support structure were determined from known values from the project site. The modulus of resilience for the embankment fill was determined from triaxial tests on a sample of the embankment material. The modulus of the asphalt leveling course represents a worst case value for asphalt concrete base material.

With the support structure and loading condition shown above, the tensile stress (σ_T) at the bottom of the equivalent CRC (or JRCP) control pavement was determined using elastic layered theory analysis. The computer program BISAR (Bitumen Structures Analysis in Roads) was used for the elastic layered theory analysis. Stresses were computed beneath the loads and between the loads to determine the worst condition. The amount of slip between the asphalt leveling course and pavement was varied to determine the worst case condition, although there is generally no slip between a cast-in-place pavement and the base.

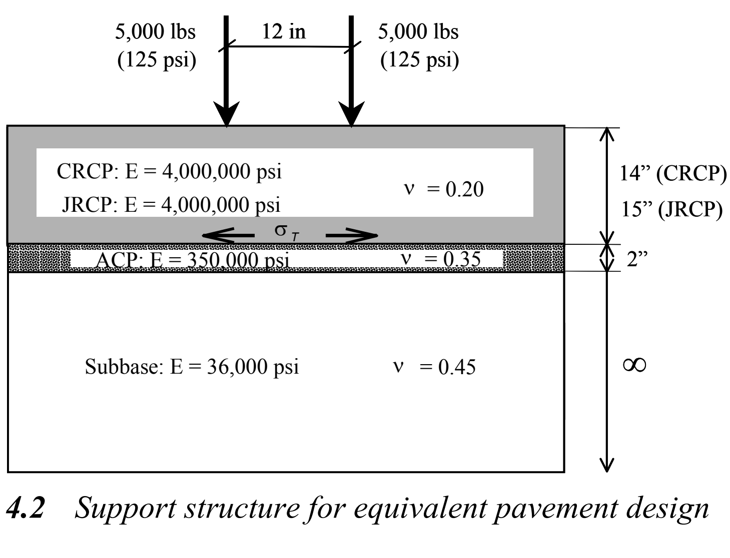


Figure 4.2 *Support structure for equivalent pavement design*

Table 4.1 summarizes the stresses for the equivalent CRC pavement for the varying slip conditions at locations directly beneath one of the loads and at the midpoint between loads. As the analysis shows, the highest tensile stresses occurred at the midpoint between loads for the frictionless slip condition. Although frictionless slip is probably not a realistic condition beneath a cast-in-place pavement, the variation between the no-slip condition and frictionless slip condition is very minimal (< 4 psi). Using the values from Table 4.1, the prestress requirements for fatigue design of the precast pavement can be computed.

Table 4.1 *Bottom fiber tensile stress at the bottom of the 14 in. equivalent CRC pavement*

	Bottom Tensile Stress, σ_T (psi)	
Slip Condition	***Beneath Load***	***Midpoint Between Loads***
No Slip	52.9	55.8
¼ Slip	55.0	58.0
Half Slip	55.7	58.8
Frictionless Slip	56.5	59.5

4.3.3 Prestress Requirements

The philosophy for designing a precast pavement with an equivalent fatigue life to a thicker conventional concrete pavement was to keep the ratio, R_e, of the bottom fiber tensile stress (from layered theory analysis) to the flexural strength equal to that of the control CRC pavement. Keeping this ratio constant should produce pavements that can withstand the same number of wheel loads over the life of the pavement. This methodology is based on the fatigue relationship given in Equation 4-1 (Ref 10)

$$N_{18} = 46{,}000\left(\frac{f}{\sigma_T}\right)^{3.00} \tag{4-1}$$

where: N_{18} = Number of 18 kip ESALs to serviceability failure
f = Concrete flexural strength (psi)
σ_T = Bottom fiber tensile stress from wheel loading

To determine the stress ratio, R_e, for a (precast) prestressed pavement, the applied prestress, σ_{PR}, was subtracted from the bottom fiber tensile stress (from elastic layered theory analysis) for an 8 in. pavement, as shown in Equation 4-2.

$$R_e = \left(\frac{\sigma_T - \sigma_{PR}}{f}\right) \tag{4-2}$$

where: R_e = Fatigue stress ratio
f = Concrete flexural strength (psi)

σ_T = Bottom fiber tensile stress in the precast pavement

σ_{PR} = Required prestress to achieve and equivalent stress ratio

Table 4.2 shows the results from the elastic layered theory analysis for an 8 in. precast pavement. As with the control pavement, the support structure shown in Figure 4.2 was used for varying slip conditions. Using these values, the stress ratios for the precast pavement were equated to that of the control pavement. Assuming a concrete flexural strength of 700 psi, the required prestress for an equivalent precast pavement was back-calculated, as shown in Table 4.3. Note that only the higher tensile stresses (between the loads) from Tables 4.1 and 4.2 were used for the back calculation.

Table 4.2 *Bottom fiber tensile stress at the bottom of the 8 in. precast pavement*

	Bottom Tensile Stress, σ_T (psi)	
Slip Condition	***Beneath Load***	***Midpoint Between Loads***
No Slip	124	125
¼ Slip	134	134
Half Slip	137	137
Frictionless Slip	139	140

Table 4.3 *Required prestress for the equivalent precast pavement*

	CRC Control Pavement		Equivalent Precast Pavement	
Slip Condition	***Bottom Fiber Stress (psi)***	R_e	***Bottom Fiber Stress (psi)***	**Required Prestress, σ_{PR} (psi)**
No Slip	55.8	0.080	125	69.2
¼ Slip	58.0	0.083	134	76.0
Half Slip	58.8	0.084	137	78.2
Frictionless Slip	59.5	0.085	140	80.5

As Table 4.3 shows, for the worst case conditions a prestress force of 80.5 psi is required at all points along the precast pavement in order to produce a pavement with an equivalent fatigue life to that of a 14 in. CRCP or 15 in. JRCP. Using Grade 270, 0.6 in. diameter post-tensioning strands tensioned to 80% of their ultimate strength, the required strand spacing would be approximately 72 in. to achieve this level of prestress along the length of the pavement. The next step in the design process is to consider environmental stresses and their effect on the prestress requirements.

4.4 Design for Environmental Effects

In the previous section, design for fatigue revealed that a prestress force of 80.5 psi was required to produce an 8 in. precast pavement with an equivalent design life to a 14 in. CRC pavement (15 in. JRCP). This means that at every point along the length of the slab, a minimum compressive stress of 80.5 psi must be maintained, under varying environmental conditions, in order to meet the fatigue requirements. Temperature is the primary environmental effect of concern. Temperature cycles cause horizontal slab movement, which results in frictional restraint stresses, as well as vertical slab movements, which result in curling stresses. Other factors such, as material properties and prestress losses, must also be considered with environmental effects.

4.4.1 PSCP2 Design Program

A powerful tool for analyzing the effects of environmental stresses on precast pavement design 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. PSCP2 takes into account the geometric properties of the pavement, material properties, slab-base interaction, and temperature effects. The program uses these parameters to determine curling stresses, prestress losses, frictional stresses, and vertical and horizontal slab movements. Although the program was originally developed for cast-in-place pavements, several adjustments were made to the inputs to simulate the differences of precast pavement.

4.1.1.1 Inputs

Geometric Properties

The geometric inputs for the PSCP2 program include the slab length (between expansion joints), slab width, and slab thickness.

Concrete Properties

The concrete properties include coefficient of thermal expansion, ultimate shrinkage strain, unit weight, Poisson's ratio, creep coefficient, and age-compressive strength relationship. As mentioned previously, it is necessary to adjust the inputs of the program to account for the differences of a precast pavement. This can be accomplished by specifying a low ultimate shrinkage strain and a high early-age strength to account for cured, full-strength concrete at the time the precast panels are installed.

Steel Properties

The steel properties include the strand cross-sectional area, yield strength, elastic modulus, thermal coefficient, and strand spacing. The spacing of the strands dictates the amount of prestress applied to the slab. During the design, the strand spacing is varied to adjust the amount of prestress in the pavement.

Prestress

The PSCP2 program allows for multiple-stage post-tensioning. This process is essential for cast-in-place prestressed pavements where it is essential to apply an initial prestress within the first several hours after concrete placement to prevent shrinkage cracking. For a precast concrete pavement, however, only one stage of post-tensioning is required. The program allows for specification of the amount of prestress (per strand) and the number of hours after panel placement at which post-tensioning takes place.

Slab-Base Interaction

The slab-base interaction inputs include the friction-displacement relationship and the stiffness of the slab support. The friction-displacement relationship can be specified as either a linear, multi-linear, or exponential relationship between the coefficient of friction and corresponding displacement.

Analysis Period

The analysis period input specifies the number of days after placement at which the pavement is to be analyzed. The program automatically analyzes the pavement for the first 24 hours. It allows for specification of multiple analysis periods beyond the first 24 hours to examine the stresses and end movements. For design purposes, an analysis period near the end of the expected design life should be specified, at minimum.

Temperature

Temperatures are usually specified for the first 24 hours after placement and for any future analysis periods. Mid-depth slab temperature, top-bottom temperature differential, and the time of day are all specified. The concrete setting temperature is also specified. For a precast concrete pavement, the setting temperature will be the temperature of the concrete at the time the precast panels are installed on site.

4.1.1.2 Analysis

The PSCP2 program computes the stress and movement (vertical and horizontal) for the number of points along the length of the slab specified in the input. In general, the slab is broken into fifty equally spaced segments, twenty-five on either side of the center of the slab. For each of these points, the output gives the prestress plus friction stress, bottom fiber curling stress, curling deflection, and horizontal movement. The output is given for every hour of the day specified in the input file.

For the Georgetown precast pavement design, the prestress plus friction stress was assumed to be constant over the depth of the slab, and the top fiber curling stress was assumed to be equal and opposite to the bottom fiber curling stress. Stresses were evaluated at the top and bottom of the slab, at mid-slab and at the end of the slab. However, only bottom fiber stresses were considered for meeting the fatigue requirements. Horizontal movement was evaluated only at the ends of the slab (at the expansion joints) where it is most critical.

Geometric Properties

A slab thickness of 8 in. and slab width of 36 ft were specified for the PSCP2 analysis as per the project specifications (Chapter 3). Slab lengths of 250 ft, 375 ft, and 500 ft were analyzed to determine the optimal length. The governing factor in slab length is the anticipated width of the expansion joints.

Concrete Properties

To account for the differences of precast pavement, the age-compressive strength relationship was specified such that the concrete had reached its full compressive strength of 4,000 psi at 0.01 days. The 3-, 7-, 14-, and 28-day compressive strengths were also specified as 4,000 psi. The ultimate shrinkage strain was specified at 0.0001 in./in., which is what might be expected for precast concrete. The coefficient of thermal expansion was specified as 8.5×10^{-6} in./in./ºF, as per the PCI Design Handbook (Ref 12), assuming a siliceous river gravel aggregate would be used for the precast panels. For the remaining inputs, a value of 150 lb/ft^3 was specified for the unit weight of the concrete, a value of 0.2 was specified for the Poisson's ratio, and a value of 2.3 was specified for the creep coefficient.

Steel Properties

A strand with a 0.6 in. diameter was specified for the prestressing (post-tensioning) steel, with a corresponding cross-sectional area of 0.217 in.^{2}, yield strength of 243 ksi, elastic modulus of 28.5×10^6 psi, and thermal coefficient of 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 placement. The strands were assumed to be stressed to 80% (216 ksi) of their ultimate strength, with only 72% (194 ksi) transferred to the pavement after accounting for seating losses.

Slab-Base Interaction

A value of 500 psi/in. was specified for the slab support. During the feasibility study, the sample design revealed that the slab support value had virtually no effect on the environmental stresses and horizontal slab movements (Ref 2). Stresses from slab support are taken into account with the wheel load stresses. The slab support value specified for the PSCP2 analysis does not correlate with the slab support values used for the elastic layered theory analysis described previously.

The friction-displacement relationship was assumed to be a linear relationship with a maximum coefficient of friction of 0.2 and corresponding displacement of 0.02 in. at sliding. Although extensive testing has found that the maximum coefficient for slabs placed on a single layer of polyethylene sheeting is around 0.92, the value used for this analysis was obtained from actual measurements of the cast-in-place prestressed pavement in McLennan County. As mentioned previously, the frictional resistance for precast panels will be different than that for cast-in-place pavements, but due to a lack of data on these differences, the values obtained from the McLennan County prestressed pavement were assumed, providing conservative results.

Analysis Period

The pavement is expected to have a design life of at least 30 years. At 30 years, the prestress will be at a minimum, owing to relaxation of the post-tensioning strands. Therefore, the number of days after placement for the final analysis was specified at 10,950. In addition, another analysis period at 1 year was specified to ensure that the critical stress combination would not occur earlier than 30 years. For analyzing maximum horizontal movements, a third analysis period of 90 days was also specified.

Temperature

Temperature data was specified for the first 24-hour period after placement and for a 24-hour period at each final analysis period (90 days, 1 year, 30 years). The temperature data used for the PSCP2 design was based upon the ambient temperature history for the previous four years in Georgetown, Texas. Daily temperature distributions for a typical summer day and a typical winter day were generated from the temperature history. Extreme conditions (extreme high and extreme low temperatures) from the past four years were then used to determine the worst-case ambient temperature distribution for extreme winter and summer conditions. Concrete temperatures were estimated from the empirical formula shown in Equation 4-3, which correlates concrete temperature to ambient temperature (Ref 13). Although concrete temperature is influenced by many other factors, such as cloud cover and precipitation, this equation provides a reasonable estimate for design purposes. Table 4.4 summarizes the temperatures used for the PSCP2 design.

$$T_C = 20.2 + 0.758 T_A \tag{4-3}$$

where: T_C = concrete temperature (^oF)
T_A = ambient temperature (^oF)

Table 4.4 *Temperature data used for the design of the Georgetown PPCP using the PSCP2 program*

	Summer Temperatures (°F)			Winter Temperatures (°F)		
Time of Day	***Ambient Temperature***	***Concrete Temperature***	***Top/Bottom Differential***	***Ambient Temperature***	***Concrete Temperature***	***Top/Bottom Differential***
12:00 AM	90.0	88.5	-5.9	17.2	33.2	-3.6
2:00 AM	86.7	85.9	-5.5	16.4	32.6	-3.4
4:00 AM	83.8	83.7	-5.2	16.1	32.4	-3.4
6:00 AM	82.7	82.9	-5.1	15.8	32.2	-3.5
8:00 AM	90.9	89.1	-1.7	16.1	32.4	-1.7
10:00 AM	99.4	95.5	8.0	19.9	35.3	7.2
12:00 PM	105.4	100.1	15.3	22.5	37.2	12.7
2:00 PM	108.8	102.7	15.4	23.7	38.2	9.1
4:00 PM	109.4	103.2	10.5	24.0	38.4	5.8
6:00 PM	107.0	101.3	3.1	21.3	36.3	-1.6
8:00 PM	97.9	94.4	-5.0	18.6	34.3	-2.9
10:00 PM	92.8	90.6	-6.4	17.7	33.6	-3.7

The setting temperature specified in the PSCP2 input file corresponded to the time of day the panels would be placed. Worst-case conditions for summer placement would correspond to temperatures at 4:00 p.m. and worst-case conditions for winter placement would correspond to temperatures at 6:00 a.m.

During the initial design four temperature conditions were considered. For each condition, one set of temperature data was specified for the initial 24-hour period after placement, and another set of temperature data was specified for the final analysis periods. The first case considered placement of the pavement in the winter and final analysis periods (90 days, 1 year, and 30 years), also in the winter. The second case considered placement in the winter and final analysis in the summer. The third case considered placement of the pavement in the summer and final analysis in the summer, while the final case considered placement in the summer and final analysis in the winter.

Wheel Load Stress

Wheel load stresses were accounted for in the fatigue design (Section 4.3). The wheel load stresses were calculated using the computer program BISAR, mentioned earlier. This wheel load stress is for interior wheel loading away from the edge of the slab. Because the shoulder is included in the precast panels, there should not be any edge loading of consequence, and the interior wheel load stress should be an accurate estimate of what the finished pavement will experience.

4.4.2 Longitudinal Prestress Requirements

As stated previously, the longitudinal prestress requirements (due to environmental effects) were determined by varying the spacing of the longitudinal strands and analyzing the stresses in the pavement using the PSCP2 computer program for the selected strand spacing. During the initial design, the slab length that would be constructed had not yet been determined. Consequently, a 500 ft slab length was analyzed as an upper limit.

Stresses in the pavement were analyzed at the ends and at mid-slab at both the top and bottom of the slab. Stresses were analyzed for the four different climatic conditions: summer placement/summer analysis, summer placement/winter analysis, winter placement/winter analysis, and winter placement/summer analysis. The limiting factor for the analysis was the maximum allowable stress in the bottom of the slab as determined from the fatigue design. As given in Table 4.3, this limiting value was a compressive stress of 80.5 psi. Figure 4.3 shows the results of the PSCP2 analysis for a 500 ft slab length for the four climatic conditions.

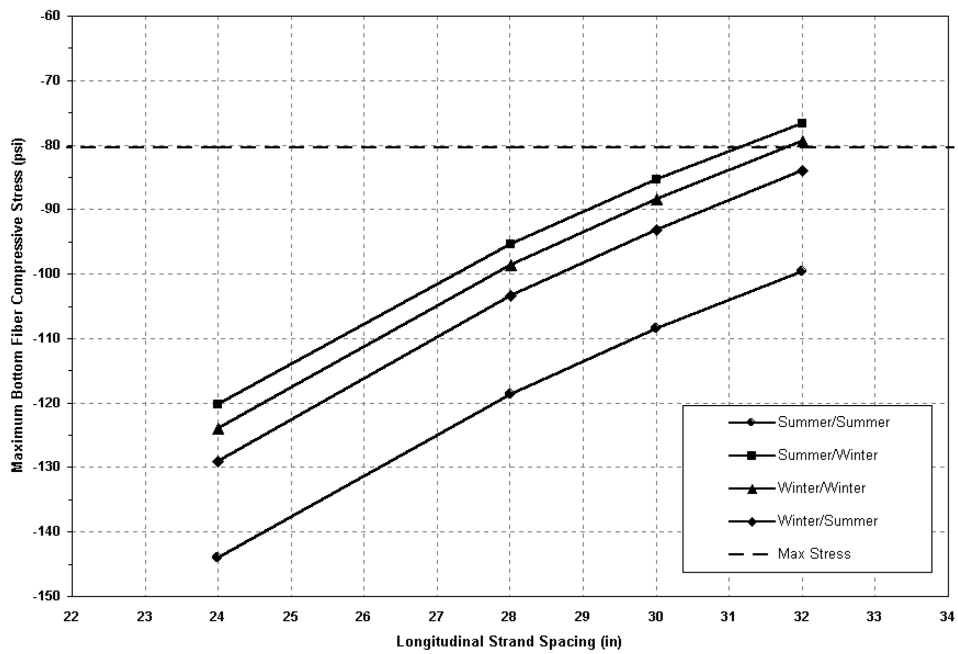


Figure 4.3 *Results of the Environmental Loading design for a 500 ft slab length*

As Figure 4.3 shows, the maximum permissible strand spacing to satisfy the prestress requirements was 30 in. Stresses in the pavement due to environmental effects during each of the four climatic conditions will never be greater than the required 80.5 psi (compressive) when the longitudinal strands are spaced at 30 in. or less. In all cases, the maximum stress occurred at mid-slab at the 30-year analysis period.

It should be noted that this analysis represents absolute worst-case conditions. The climatic conditions represent extreme high summer temperatures and extreme low winter temperatures, which may only be occasionally encountered over the life of the pavement. Realistically, the pavement will probably not be placed under these extreme conditions. Consequently, this design is conservative and will result in a more durable pavement than originally intended.

Further increasing the durability of the pavement, a strand spacing of 24 in. was selected by the designers when developing the panel drawings. This was done to simplify casting and standardize strand spacing for future projects. As Figure 4.3 shows, this will result in significantly higher compressive stresses in the pavement, greatly increasing the life of the pavement.

4.4.3 Transverse Prestress Requirements

As discussed earlier in this report, previous experiences with prestressed concrete pavements have shown that prestress in both the longitudinal and transverse directions is essential. Without transverse prestress, longitudinal cracks tend to form above the longitudinal tendons (Ref 9). Prestressed pavements will respond to environmental conditions in the transverse direction similar to that in the longitudinal direction. Accordingly, fatigue loading as well as environmental loading must be considered when determining transverse prestress requirements.

Precast panels present another consideration for transverse prestress design. Significant handling stresses are generated in large precast panels when removed from the forms and handled at the precast plant and on site. By pretensioning the panels during fabrication these stresses can be counteracted to prevent cracking from occurring during lifting. Pretensioning also minimizes the amount of mild steel reinforcement normally required in large precast panels to prevent cracks that form during lifting from opening up, and will allow much larger precast panels to be used.

Transverse prestress requirements were computed for both fatigue/environmental design and for handling. Fatigue and environmental prestress requirements were computed in the same manner as longitudinal prestress requirements. Handling stresses were computed in accordance with Section 5.2 of the Precast Prestressed Concrete Institute (PCI) Design Handbook (Ref 12). Handling stresses were limited such that cracking would not occur during the worst-case condition as the panels were removed from the forms. The modulus of rupture was computed to be 296 psi using the following equation from the American Concrete Institute (ACI):

$$f'_r = K\lambda\sqrt{f'_{ci}} \qquad (4\text{-}4)$$

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 recommendations
- λ = 1.0 for normal-weight concrete
- f'_{ci} = Concrete compressive strength at release of prestress, 3,500 psi for the Georgetown precast pavement project

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, where "L" is the length of the side 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 on these design parameters, the maximum lifting stress for 36 ft long panels was computed to be 338 psi. This stress is slightly higher than the modulus of rupture, requiring 42 psi of prestress force to prevent cracking.

Table 4.5 summarizes the transverse prestress requirements based on fatigue/environmental design and lifting considerations. As this table shows, prestress of approximately 129 psi is required to meet the fatigue/environmental design requirements, while only 42 psi is required to counteract lifting stresses.

Table 4.5 *Transverse prestress required for the precast panels*

Precast Panel Thickness	**Required Transverse Prestress (psi)**	
	Fatigue/Environmental Stresses	***Lifting Stresses***
8	129	42

Using 0.5 in. Grade 270 prestressing strand (stressed to 72% of ultimate strength after losses), five strands are required to achieve this level of prestress in each 10 ft wide precast panel. For symmetry (strands cannot be placed along the centerline of the joint panels and central stressing panels), six 0.5 in. pretensioning strands were specified for the transverse prestress.

4.4.4 Expansion Joint Movement

The final step in the design process was to determine the anticipated expansion joint movement in order to determine the maximum permissible slab length, based on the expansion joint width requirements. As discussed above, the initial design was completed assuming an upper limit on the slab length of 500 ft. The maximum slab length constructed, however, was significantly less (325 ft) due to expansion joint width limitations. The expansion joint width limits were set such that the joints would never be completely closed, and never be open more than 4 in.

The PSCP2 computer program was used to calculate expansion joint widths for slabs of varying length for the four climatic conditions mentioned previously: summer placement/summer analysis, summer placement/winter analysis, winter placement/summer analysis and winter placement/winter analysis. To ensure the analysis would predict the maximum slab movement, winter slab placement was assumed to take place at 6 a.m., while summer placement was assumed to take place at 4 p.m.

Figures 4.4 and 4.5 show the results of the PSCP2 analysis for expansion joint movement for slabs varying in length from 250 ft to 500 ft. Figure 4.4 shows the maximum amount of contraction movement of the end of the slab, which will cause the expansion joint to open, while Figure 4.5 shows the maximum amount of expansion movement of the end of the slab, which will cause the expansion joint to close. Because this movement is for the end of each slab, the total amount of joint opening or closure will be the sum of the movement of the slabs on either side of the expansion joint.

The maximum contraction movement (Figure 4.4) occurred for slabs placed in the summer and analyzed in the winter, while the maximum expansion movement (Figure 4.5) occurred for slabs placed in the winter and analyzed in the summer. The amount of contraction movement tended to increase over time to a maximum at the 30-year analysis period. Conversely, the amount of expansion movement tended to decrease over time from a maximum in the first 30 days after placement to a minimum at 30 years. This is due to creep effects in the concrete which cause the slabs to slowly shorten over time under the compression stress from post-tensioning. It should again be noted that the climatic conditions that cause these movements are extreme summer and winter temperatures during placement and analysis. Although it is unlikely that the slabs will ever experience this much movement, this analysis should provide conservative results for lack of better analysis/prediction methods.

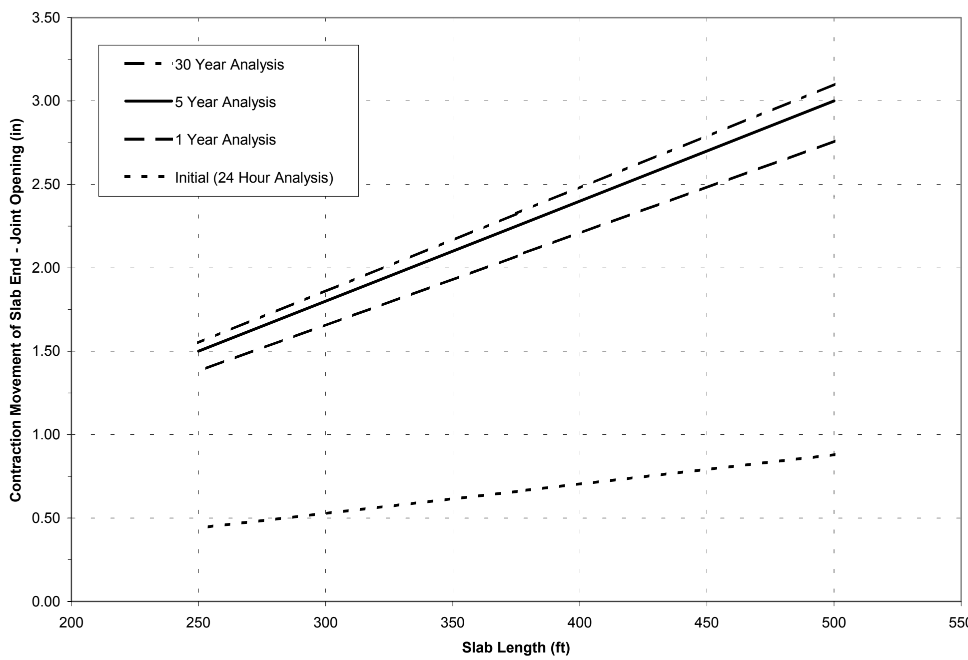


Figure 4.4 *Contraction movement of the slab end, causing opening of the expansion joint*

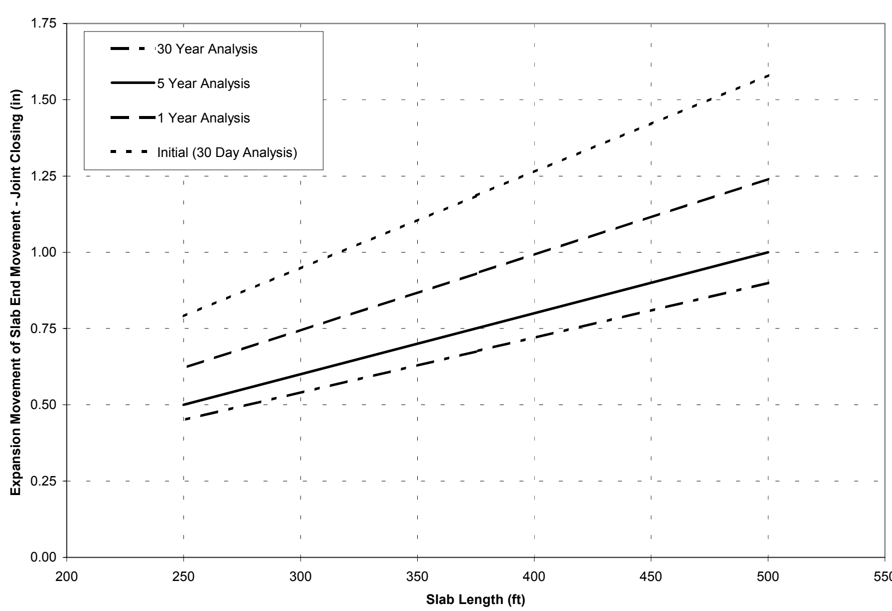


Figure 4.5 *Expansion movement of the slab end, causing closure of the expansion joint*

Based upon the PSCP2 analysis, the slab length to be constructed for the Georgetown pilot project was selected. With a maximum permissible expansion joint width of 4 in., the slab length was limited to 325 ft (see Figure 4.4). Based on these results and limitations on the number of precast panels that the contractor anticipated placing each day, a standard slab length of 250 ft was selected. However, to meet the project limits, a single slab 325 ft in length was constructed. The selection of a shorter (250 ft) slab length should benefit the ride quality of the pavement, as the expansion joints should never be open more than 3 in. (see Figure 4.4).

With the selection of the slab lengths to be constructed, the next step was to determine what width the expansion joints needed to be set at during panel placement. The initial expansion joint width is important because a joint that is initially set too narrow may close completely, while a joint that is initially set too wide may open more than the maximum width of 4 in. To address this, the data from the PSCP2 analysis was used to determine the maximum closure and opening movement of expansion joints between two 250 ft slabs, and between a 250 ft slab and 325 ft slab for a given set (placement) temperature. Figures 4.6 and 4.7 show the maximum amount of joint opening and closure movement, respectively, for the two slab lengths based upon concrete temperature at panel placement. The maximum joint opening was determined from the 30-year analysis period while the maximum joint closure was determined from the initial 30-day analysis period.

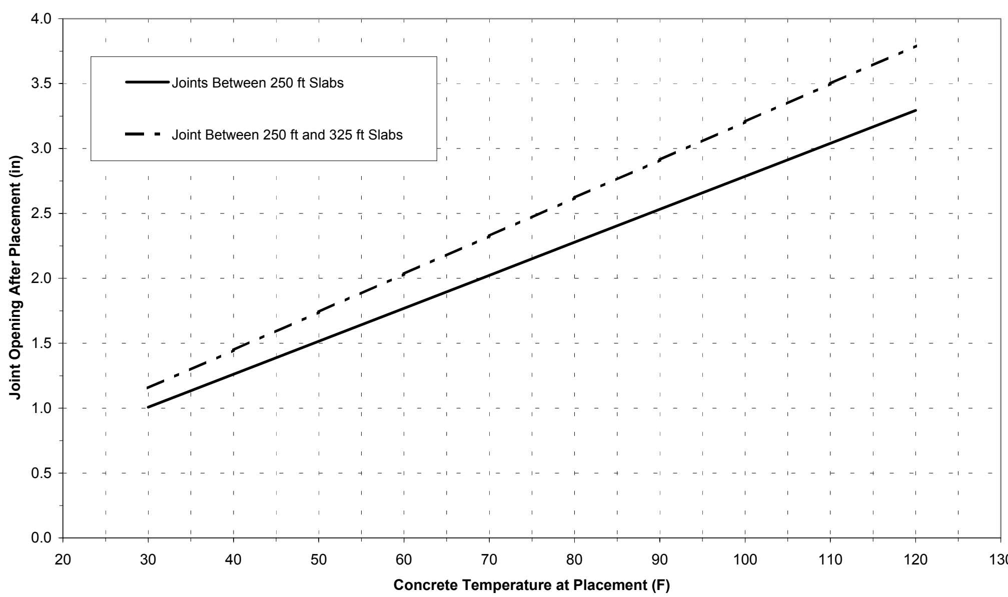


Figure 4.6 *Maximum joint opening for various concrete temperatures at panel placement*

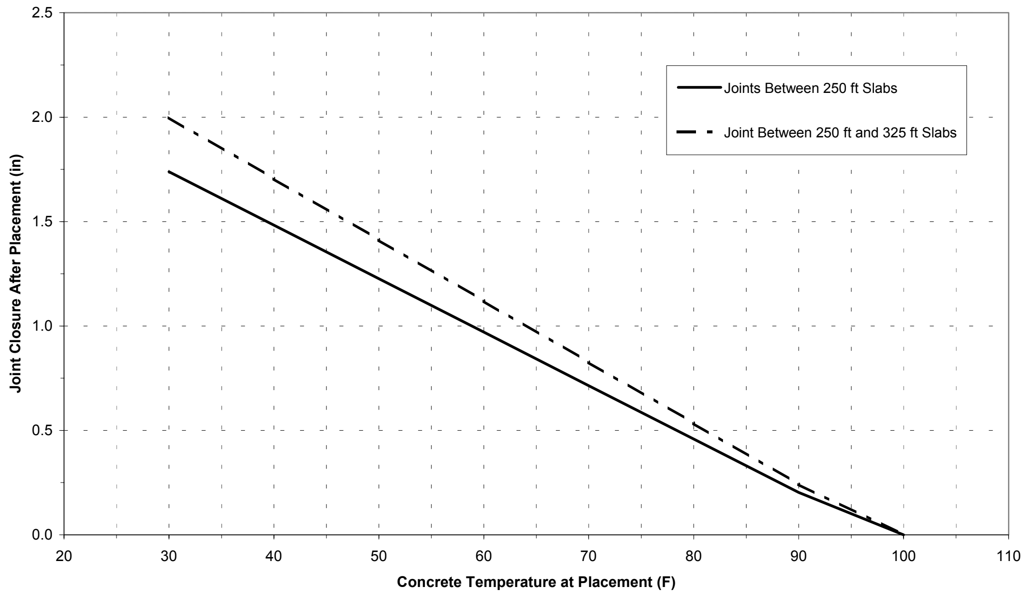


Figure 4.7 *Maximum joint closure for various concrete temperatures at panel placement*

Using the maximum joint movement shown in Figures 4.6 and 4.7, the recommended joint width at panel placement (prior to post-tensioning) was determined. To simplify the recommendations for inclusion in the construction drawings, the concrete temperature was correlated to ambient temperature. This allowed the contractor to use the graph shown in Figure 4.8 to determine the width for each expansion joint based on the ambient conditions when each panel was placed. Based on the anticipated season of construction, the initial expansion joint width was set at the precast plant prior to casting the panels. The initial joint width was set at 0.5 in. for the full-width panels for placement in late summer/early fall, and 0.75 in. for the partial-width panels for placement in mid–late fall.

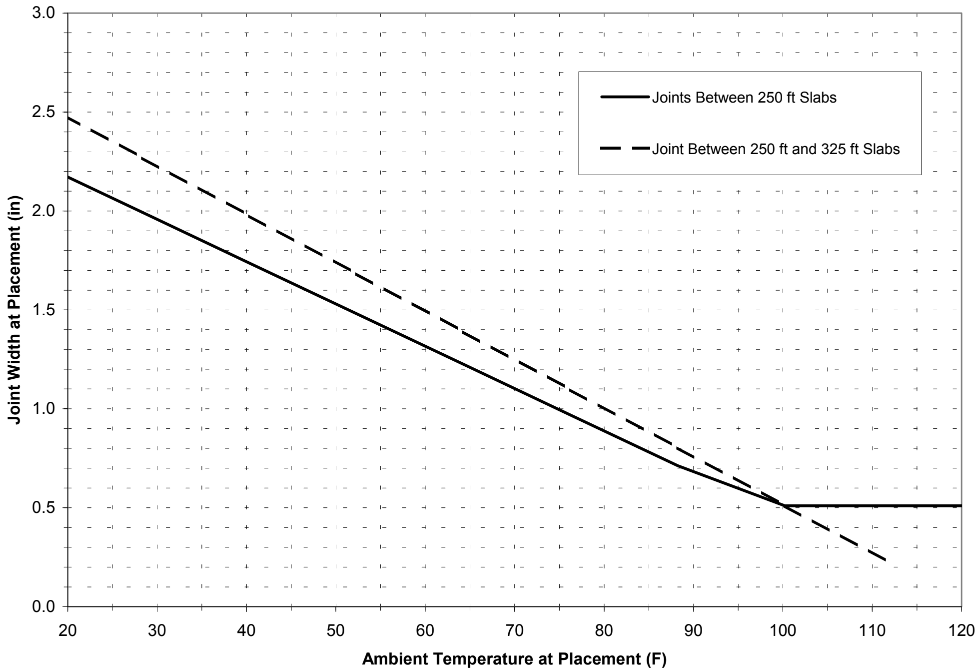


Figure 4.8 *Recommended expansion joint width at panel placement based on ambient condition*

5. Lab Testing and Trial Assemblies

5.1 Introduction

Prior to construction of the actual precast pavement test section on the frontage road near Georgetown, Texas, two trial assemblies were completed to evaluate some of the aspects of the proposed concept. The first of the trial assemblies was completed using small-scale precast panels in the laboratory. The successful completion of laboratory testing and assembly led to a full-scale assembly at the precast plant. Following the successful assembly of the panels at the precast plant, TxDOT gave approval for panel production. Both trial assemblies proved to be very valuable and provided useful information for actual construction at a minimal cost.

5.2 Lab Testing and Assembly

The first trial assembly was completed in the laboratory. This trial assembly was used to examine the viability of several aspects of the proposed precast pavement concept including:

1. Panel assembly
   - Lowering panels into position at a "nose-down" angle
   - Viability of continuous shear keys
   - Use of edge sleeves for pulling the panels together
   - Ease of assembly over plastic sheeting
2. Post-tensioning strand anchorage
3. Joint sealing
4. Strength of edge sleeves and post-tensioning anchors

Laboratory testing was conducted at Ferguson Structural Engineering Laboratory at the J. J. Pickle Research Campus of The University of Texas at Austin. Three panels were cast for the laboratory testing; two representing base panels, and one representing one half of a joint panel. The joint panel contained the spring-loaded post-tensioning anchors, as well as the access pockets for pushing the strands into the anchors.

The actual testing took place on two separate days. The first day consisted of a demonstration of the panel assembly, strand placement, and post-tensioning. Representatives from all parties involved with the Georgetown pilot project (CTR, TxDOT, general contractor, precast supplier, and post-tensioning supplier) were present for the demonstration. The second day consisted of testing joint sealant material, pull-out tests for the post-tensioning anchors, and testing the edge sleeves to failure.

5.2.1 Precast Panels

The three panels cast for the laboratory testing were all 8 in. thick and 62 in. wide. Two of the panels, representing base panels, were 10 ft long, and the third panel, representing one half of a joint panel was 5 ft long. The 62 in. panel width allowed for three post-tensioning ducts and anchors, which the researchers felt was sufficient for the initial testing.

The reinforcement used in the panels was the same as that used in the actual panels, with the exception of the transverse pretensioning strands. Mild reinforcement was substituted for pretensioning strands in the test panels. Other aspects, such as the post-tensioning ducts, keyed panel edges, trumpeted duct openings, and post-tensioning anchors were all the same as that used for the panels in the pilot project.

A 4,000 psi concrete mix with 3/4 in. limestone aggregate and 4 in. slump was specified for the panels. The panels were cast on two separate days due to a limited number of edge forms. Cylinders were cast with each pour to verify concrete strengths. A broom finish was applied to the top surface approximately one hour after casting. The panels were covered with plastic and allowed to cure for at least three days before the forms were removed. The panels were allowed to cure for at least 7 days before testing began to ensure that the concrete had reached its design strength.

5.2.2 Panel Assembly

The panel assembly demonstration was completed on March 7, 2001. Representatives from CTR, TxDOT, Granite Construction (general contractor), and Texas Concrete Company (precast supplier) were present for the demonstration.

The joint panel was set in place first, over a single layer of polyethylene sheeting, and anchored to the floor so that it could not move as the other panels were set in place. The base panels were then set in place one at a time, lowered into place at a “nose-down” angle, with slight tension in the lifting lines, to prevent the plastic sheeting from bunching up in the joint. By the time each base panel was fully resting on the plastic, the joint had already been pulled closed using come-alongs linked between the edge sleeves.

The keyed panel edges proved to be beneficial for achieving vertical alignment of the panels during assembly. The edge sleeves, likewise, also proved valuable for panel assembly. Come-alongs linked between the bars extending from the edge sleeves of adjacent panels pulled the panels together tightly as they were lowered into place.

Only minor spalling of the concrete was noticed around the edge sleeves, particularly the sleeves cast into the top of the panels. This spalling was attributed to having too much tension in the come-alongs as the panels were pulled together.

After assembly of the panels, initial joint-width measurements were taken for comparison with the joint widths after post-tensioning. Joint-width readings were taken using both dial calipers and an 8 in. (200 mm) gauge length Demec gauge on either side of both joints.

5.2.3 Strand Placement and Post-tensioning

Following assembly of the three panels, the post-tensioning strands were fed into the post-tensioning ducts. The strands were standard 0.6 in., grade 270, low relaxation strands. The strands were manually pushed into the ducts from the end of the slab until they reached the front of the post-tensioning anchors. The strands were then pushed as far as they could be pushed into the spring-loaded anchors from the access pockets in the joint panels.

After anchoring the post-tensioning strands, they were each stressed from the end of the assembled slab using a center-hole stressing ram and hydraulic pump. The strands were each stressed to 80% of ultimate strength (46.9 kips), as called for in the design. Standard post-tensioning chucks were used to anchor the strands at the end of the finished slab. Post-tensioning started with the middle strand followed by the outside strands.

After post-tensioning the joint widths were again measured using the dial calipers and Demec gauge. The results of the joint width measurements before and after post-tensioning, shown in Table 5.1, indicate very little closure (< 0.015 in.) of the joints from post-tensioning. This indicates that the come-alongs were able to pull the joints almost completely closed prior to post-tensioning.

Table 5.1 *Precast panel joint closure before (Initial Reading) and after (Final Reading) post-tensioning*

	Location	Initial Reading	Final Reading	Difference (in.)
Demec Gauge	*1*	0.03560	0.03067	-0.00493
	2	0.01116	0.00006	-0.01110
	3	0.01416	0.00009	-0.01407
	4	0.02833	0.01710	-0.01122
Dial Calipers	*1*	2.8855	2.8765	-0.0085
	2	2.8985	2.8910	-0.0075
	3	2.8990	2.8815	-0.0175
	4	2.8715	2.8630	-0.0085

5.2.4 Joint Sealing

Upon completion of panel assembly and post-tensioning, a method for sealing the joints between the individual panels was tested. The sealant material was a low viscosity, high molecular weight methacrylate commonly used to seal cracks in concrete.

Prior to applying the sealant material, the ends of the joints at the edge of the panels were sealed with silicon to ensure that sealant would not leak from the edges of the panels. The sealant material was poured into each of the joints and allowed to soak into the joint. Where the joints were very tight, the sealant material tended to pond at the surface of the panels. Where the joints were more open, however, the sealant flowed freely down into the joint. The sealant material was applied twice to the open joints, with approximately one hour between applications, to determine if the material was slowly building up in the joint.

When applying the sealant to the wider joints, sealant material was observed leaking from the post-tensioning ducts in the pockets in the joint panel. This indicated that not only was the sealant not staying in the joint, but also that the silicon sealant applied around the ducts prior to panel assembly was not properly sealing the ducts. This was later confirmed when the panels were separated and sealant material was found in both the post-tensioning ducts and underneath the panels.

Based on the joint sealing tests, it was determined that a low viscosity sealant would not work well for the joints between precast panels. Although the material did a good job of sealing those joints that were closed tightly, it did not adequately seal joints open more than 1/16 in., and could cause problems with post-tensioning.

5.2.5 Anchor and Edge Sleeve Strength Tests

After completion of panel assembly, post-tensioning, and joint sealing, a separate day of testing was used to test the strength of both the spring-loaded post-tensioning anchors and edge sleeves.

Anchor tests, often referred to as pull-out tests, are used to determine whether a post-tensioning anchor can withstand the prestress force exerted by a post-tensioning strand stressed to its ultimate strength. Ideally, the strand should fail before the anchor, as strand failure is a more gradual failure, while anchor failure can be more catastrophic.

Each of the three post-tensioning strands were stressed to just over 100% of their ultimate strength (58.6 kips), using a center hole stressing ram, held momentarily, and released. The pull-out tests resulted in no strand or anchor failures, and no distresses were noticed in any of the precast panels.

Following the strand pull-out tests, the edge sleeves were tested to failure. A 20 kip tension/compression stressing ram was used to pull #7 reinforcing bars inserted in the edge sleeves of the adjacent panels to failure. As expected, failure was governed by bending of the bars inserted in the edge sleeves. The edge sleeves on one side of the slab withstood a load of 5.1 kips, while the edge sleeves on the opposite side of the slab withstood a load of 6.2 kips. The sleeves on the top of the panels withstood a slightly higher load of 7.7 kips. Damage to the precast panels at failure was very minimal, in the form of minor spalling around the edge sleeves. Based upon these tests it was determined that the edge sleeves would provide a viable method for pulling the precast panels together during assembly.

5.2.6 Conclusions from Laboratory Trial Assembly and Testing

Based on the laboratory testing, lowering the panels into place at a slight "nose-down" angle (approximately 5°) is beneficial for preventing the plastic from bunching up in the joint and ensuring that the keyed edges mate properly. The edge sleeves proved to be a simple and effective method for pulling the joint between the panels closed before they are fully resting on the ground.

The lab testing also proved the viability of the spring-loaded post-tensioning anchors. The post-tensioning strands were easily threaded through the ducts and pushed into the anchors from the pockets in the joint panel. The anchors were shown to be capable of withstanding the full post-tensioning force from stressing the strands to 100% of their ultimate strength.

The lab testing also showed impracticality of a low viscosity methacrylate joint sealant. The sealant proved to be too thin to seal joints that are even slightly open, resulting in the material leaking into the post-tensioning ducts and pooling beneath the panels. Although the sealant did appear to work well for joints that are closed tightly, the precautions that must be followed when using the material may also make it impractical for field use.

Laboratory testing of small scale precast pavement panels proved very valuable for evaluating many aspects of the proposed precast pavement concept and may have prevented many problems from occurring during construction of the actual test section. Although the laboratory testing was done using 5 ft wide panels, as opposed to 36 ft wide panels, the techniques and concepts demonstrated in the testing were found to be applicable for 36 ft panels.

5.3 Precast Plant Trial Assembly

The second trial assembly was conducted at the precast plant in Victoria, Texas using full-size (36 ft x 10 ft) panels. Laboratory testing and assembly provided useful information on the viability of the keyed panel edges, use of edge sleeves for assembly, and the use of self-locking post-tensioning anchors. However, because the panels used for the laboratory assembly were only 5 ft wide, it was concluded that another trial assembly was needed using full-size panels.

5.3.1 Casting

The trial assembly at the precast plant not only allowed TxDOT and the researchers to evaluate the assembly of full-width panels, but also gave the precast supplier an opportunity to cast "trial" panels prior to full-scale production. The three panels cast for the trial assembly were standard panels with all of the details described in Chapter 6. One central stressing panel and two base panels were cast on a short (100 ft) pretensioning bed.

Casting the three test panels allowed the precast supplier to determine an optimum mix design and an optimum casting sequence. The precast supplier experimented with different concrete placement techniques, the use of intermediate curing compound, and timing for applying the surface texture. Casting a central stressing panel as one of the test panels allowed the precast supplier to determine the best method for casting around the stressing pocket block-outs.

5.3.2 Panel Assembly

Representatives from TxDOT, CTR, and Granite Construction (general contractor) were present for the panel assembly. The three panels were assembled on a flat concrete slab at the precast yard. The panels were each lowered into place at a slight "nose-down" angle using a 45 ton capacity forklift with a load spreading beam. No bowing or warping of the panels was observed as they were lifted. Similar to the laboratory assembly, come-alongs linked between steel bars inserted into the edge sleeves were used to pull the panels together as they were lowered into place. As observed during the laboratory assembly, the come-alongs were effective only when the panels were slightly suspended above the ground by the forklift. Due to the weight of the panels, it was not possible to pull them together with the come-alongs when they were fully resting on the ground. Alignment of the ducts was checked by pushing a 0.6 in. post-tensioning strand through several of the ducts.

The joint between the assembled base panels was approximately 1/8 in. to 3/16 in. wide at the top of the panels and uniform over the length of the joint. Examination of the edge of the panels revealed that the nose of the male keyway was flush with the adjoining keyway, preventing the joint from closing completely at the top. The joint between the base panel and central stressing panel was slightly tighter than the first joint, but not as uniform due to a slight bow in one of the side forms used to cast the central stressing panel. The side form was subsequently straightened to remove the bow prior to full-scale production.

The vertical alignment was satisfactory for both joints. The joint between the two base panels was essentially flush across the 36 ft length. The joint between the base panel and central stressing panel was slightly off at the center of the panels, but not more than 1/8 in., and did not extend the length of the entire joint.

5.3.3 Conclusions from Precast Plant Assembly

The trial assembly at the precast plant proved to be very informative for evaluating how well full-size precast panels fit together. Additionally, the casting process allowed the precast supplier to experiment with casting techniques on a small scale, and resulted in panels suitable for use in the actual frontage road pavement. The panel assembly demonstrated how well the panels fit together as well as how quickly they can be placed. The edge sleeves proved to be just as efficient and effective for pulling the full-width panels together as they were for the smaller laboratory panels. Overall, the joints appeared to be sufficiently tight and aligned, and should not degrade ride quality of the final pavement section.

6. Panel Fabrication

6.1 Introduction

A total of 339 panels were cast for the Georgetown precast pavement. This included 123 full-width panels (6 joint panels, 10 central stressing panels, and 107 base panels), and 216 partial-width panels (10 joint panels, 16 central stressing panels, and 190 base panels). Such a large casting operation required a concerted effort on the part of the precast supplier to ensure that a consistent product was produced with each set of panels cast. Some of the critical aspects of casting included tolerances, finishing, curing, and repairs to damaged panels.

6.2 Procedure

During the feasibility study (Ref 2) it was found that match-casting would not be economically feasible for large precast paving projects. Representatives from the precast industry indicated that, with strict tolerances, tight joints between panels could be achieved without the need for match-casting. The panels could be cast in a more productive manner on a long line casting bed. Based on this recommendation, all of the panels for the Georgetown precast pavement were cast on a long line casting bed, approximately 400 ft in length. This allowed for at least 10 full-width (36 ft) panels, and up to 20 partial-width panels to be cast at one time. The side forms for the male keyways were welded to the bottom form to ensure that they remained straight and in the same position throughout the casting process. The side forms for the female keyways were bolted to the bottom form prior to casting each set of panels for easy removal prior to lifting the panels out of the forms. Rigid steel bulkheads were used to form the ends of the panels and separate the panels from each other.

After the forms were set up, the pretensioning strands were extended the length of the casting bed, passing through all of the bulkheads, and anchored at the ends of the casting bed. After an initial tensioning of the strands, the rest of the components of each of the panels, including the post-tensioning ducts, lifting devices, and mild steel were tied in place in the forms. The pretensioning strands were then stressed to 80% of their ultimate strength in preparation for casting.

In general, one set (10–20) of panels were cast approximately every 3 days. The panels would be cast one day, removed from the forms the following day, and the bed set-up for casting the following day. This casting rate varied depending on the type of panels cast. The central stressing panels and joint panels generally required an additional day for setup. Although casting could have taken place the same day as the forms were set up, the precast supplier was required to cast in the morning only. Most of the panels were cast in the months of June–October where, in southeast Texas, ambient temperatures can exceed 100 °F in the afternoon, causing the metal forms to reach temperatures in excess of 140 ˚F. Not knowing what effect these high temperatures would have on the concrete, it was decided to only allow casting in the morning hours while the forms were relatively cool.

Concrete was placed in the forms using small (3 cubic yard) hoppers filled at a central batch plant at the precast plant. Handheld vibrators were used to consolidate the concrete around the reinforcement and keyways. A vibratory screed was then used to strike off the top surface of the panels level with the top of the side forms. After any necessary hand finishing the concrete was allowed to set until a surface texture could be applied. Following the surface texture, two coats of curing compound were applied to the panels to minimize moisture loss from the surface.

Cylinders (4 in. x 8 in.) cast at the same time as the panels were used to check the compressive strength of the concrete. Once the concrete had reached the specified release strength, the pretensioning strands were de-tensioned and the strands between each of the panels were cut. The side forms were then removed and the panels were lifted out of the forms. The panels were then stacked in the casting yard in preparation for shipment.

6.3 Tolerances

As discussed previously, it was determined that match-casting would significantly slow casting production and that tight joints between panels could be achieved without match-casting. This required special attention to tolerances, particularly along the edges of the panels. Any slight bulge or bow along a mating edge of a panel could have prevented it from matching up with an adjacent panel. Ensuring that the side forms were not damaged and remained true throughout casting was essential for meeting the specified tolerances.

After meeting with both the precast supplier and TxDOT inspectors at the precast plant, it was agreed that stringent tolerances for the panels would be specified initially and any regular deviation from these tolerances would be addressed on a case-by-case basis. Table 6.1 shows the tolerances for the precast panels as specified in the plans.

Table 6.1 *Table of panel tolerances from plans*

Measurement	Tolerance
Length (Longitudinal to C/L)	+/- 1/4″
Width (Transverse to C/L)	+/- 1/4″
Nominal Thickness	+/- 1/16″
Horizontal Alignment – Deviation from straightness of mating edge of panels	+/- 1/4″
Deviation of ends from shop plan dimension (Horizontal skew)	+/- 1/4″
Position of strands (horizontal and vertical)	+/- 1/8″
Position of handling devices	+/- 3″

The “Horizontal Alignment” tolerance relates to the straightness of the panel edges which mate to adjacent panels. This was a critical tolerance which dictated how well the panels matched up. The “Horizontal skew” tolerance was also very critical, particularly for the partial-width panels where each panel abutted to three other panels, making squareness very important. The “Nominal Thickness” tolerance was critical primarily at the edges of the panels, along the keyways. Deviation from this tolerance could result in vertical misalignment of adjacent panels, creating ridges at the panel joints.

6.4 Panel Details

The layout for each of the three types of panels is essentially the same, with the obvious additions of the stressing pockets in the central stressing panels, and armored expansion joints and access pockets in the joint panels. The common details for all panels and specific details for each of the panel types are discussed below.

6.4.1 Keyways

As stated previously, the purpose of the keyed panel joints was to aid with vertical alignment as the panels are assembled. The keyways proved to be very beneficial in decreasing the amount of time required to place each panel, as will be discussed in Chapter 7.

The keyway dimensions for all of the panels are shown below in Figure 6.1. The nose of the "male" keyway is slightly shorter and has a slightly steeper taper than that of the "female" keyway. This is to allow the top and bottom surfaces of the keyways to come together without the nose of the male keyway "bottoming out," ensuring a tight joint and maximum surface contact between panels.

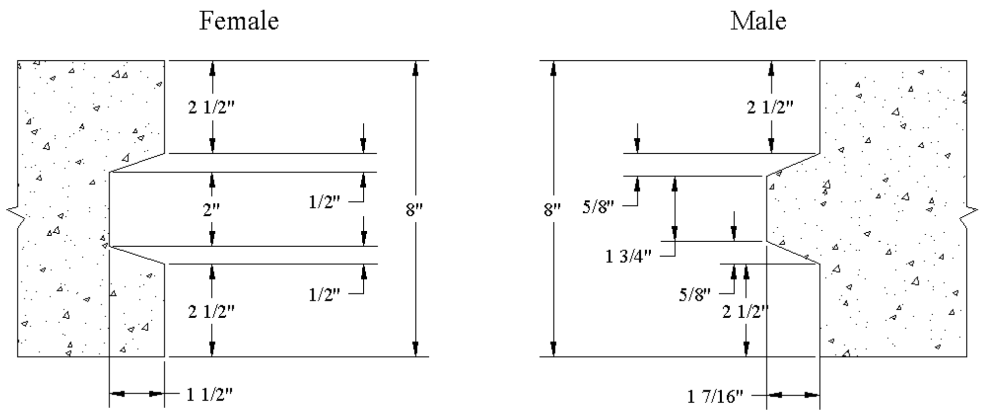


Figure 6.1 *Keyway dimensions for the precast panels*

6.4.2 Base Panels

As described in Chapter 2, "base panels" constituted the majority of the precast panels used for the Georgetown precast pavement. As with all of the other panels, the base panels were pretensioned in the transverse direction (long axis) during fabrication with six 0.5 in. Grade 270 prestressing strands. The vertical location of the six pretensioning strands was alternated above and below the post-tensioning duct, which was located at mid-depth, to prevent an eccentricity of prestress force which may have caused a slight camber in the panels. Mild reinforcement in the base panels was minimal, with #4 (Grade 60) deformed bar reinforcement around the perimeter of the panels at the top and bottom. A minimum of 2 in. of concrete cover was provided for all of the reinforcement. The dimensions and exact locations of all reinforcement is shown in the panel detail drawings in the Appendix.

6.4.3 Central Stressing Panels

The "central stressing panels" contain the pockets for post-tensioning, as described in Chapter 2. The pockets needed to be large enough to accommodate the post-tensioning ram, while accounting for elongation of the post-tensioning strands as they were stressed. Based upon these considerations and the size of pockets used for the West, Texas, post-tensioned pavement (Ref 3) the pocket dimensions specified were 8 in. x 48 in. The 8 in. pocket width provided enough room for workers to couple the post-tensioning tendons, and allowed for the post-tensioning ram to fit inside the pocket. Based on elongation calculations, a total elongation (both strands coming into each pocket) of approximately 2 in. (26 in. for the 325 ft slab) was anticipated. The 48 in. stressing pockets were sufficient to accommodate this movement.

As shown in the panel assembly diagram in Chapter 2, the stressing pockets were divided into two panels. With such large pockets, there was concern about handling a panel with pockets spaced every 2 ft over the length of the panel. By alternating the pockets between two panels, the pockets were spaced at 4 ft apart.

Another consideration with regard to the central stressing pockets was preventing cracks from propagating from the corners of the stressing pockets. Cracks appeared in the West, Texas, prestressed pavement shortly after construction (Ref 8), generally extending from the corner of one pocket to an adjacent pocket. It is believed that this cracking occurred because of stress concentrations at the corners of the pockets. To prevent this from happening in the Georgetown precast pavement, the corners of the stressing pockets were rounded with a 2 in. radius. Additionally, L-shaped reinforcement was placed at the end of each pocket to prevent cracks that may form from opening. The details of the mild reinforcement are shown in the panel detail drawings in the Appendix.

Mild reinforcement in the central stressing panels was similar to the base panels. In addition to the stressing pocket corner reinforcement, #4 reinforcing bars were placed around the perimeter of the panels at the top and bottom. Additional reinforcement was also included to tie the concrete used to fill the pocket to the rest of the panel. Two #4 reinforcing bars were placed such that they would cross each of the stressing pockets.

Pretensioning strands were located outside of the stressing pockets, spaced uniformly on either side of the pockets. Grout vents were also located on both sides of each stressing pocket for pumping or venting grout for the tendons.

6.4.4 Joint Panels

As discussed in Chapter 2, the "joint panels" contain the armored expansion joint which absorb the expansion and contraction movements of the post-tensioned slab due to temperature changes. The armored joint detail used for the Georgetown pilot project, shown in Figure 6.2, is similar to the joints used for the cast-in-place prestressed pavement near West, Texas, (Ref 3) which are in excellent condition after 18 years of service.

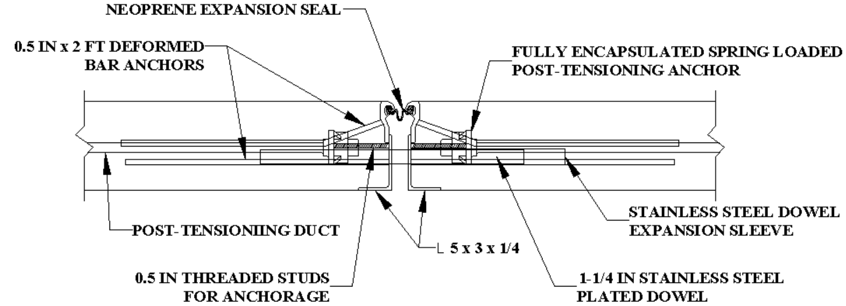


Figure 6.2 *Armored expansion joint detail used for the Georgetown pilot project*

The armored expansion joint is given structure by the steel angles welded to the bottom of the seal receiver. Deformed bar anchors are welded to the joint structure, alternating between the top and bottom every 8 in., to tie the joint structure to the precast panel. Stainless steel plated dowel bars are spaced every 12 in. and located just below mid-depth. Stainless steel dowel bar expansion sleeves are welded to one side of the joint structure to receive the dowel bars. To tie the post-tensioning tendons to the joint structure, 0.5 in. threaded studs are welded to the joint to receive the post-tensioning anchors. Tying the anchors to the joint structure ensures that the entire joint panel is prestressed.

Joint panels were perhaps the most complex of the panels to fabricate. The armored expansion joint, post-tensioning anchors, dowel bars, anchor access pockets, pretensioning strands, and mild steel were all components of the joint panels. The expansion joint was set into the forms as a single unit. Steel plates were tack welded to the joint structure to hold the joint at the specified width during casting.

Mild steel for the joint panels included top and bottom perimeter steel for each half of the joint panel, corner steel for the access pockets, and steel through the access pockets to tie the pocket concrete to the panel. Additional "bursting steel" was placed just in front of the post-tensioning anchors. The pretensioning strands were located on either side of the access pockets, with the strands alternating above and below the post-tensioning ducts. Grout inlets/vents were located just in front of the post-tensioning anchors. The layout and dimensions of the joint panels are shown on the panel detail sheets in the Appendix.

6.4.5 Partial-Width Panels

Partial-width panels were cast in much the same way as the full-width panels, and contained the same reinforcement, expansion joint detail, pocket dimensions, and mild reinforcement. As discussed in Chapter 2, however, the partial-width panels contained an additional flat post-tensioning duct for transverse post-tensioning. Transverse post-tensioning consisted of two 0.5 in. Grade 270 7-wire strands. Dead-end anchors were cast into the outside ends of each panel for the transverse tendons. To accommodate differential movement or slight misalignment between the 16 ft and 20 ft partial-width slabs, a 4-strand post-tensioning duct was used for the two transverse strands. A diverter at the end of the flat duct, channeled each of the two strands into their individual anchors. A single transverse duct was cast into each of the base panels and two ducts into each of the central stressing panels and joint panels. Grout vents for the transverse tendons were located below the anchors. Additional bursting steel was placed directly behind the post-tensioning anchors. The layout and dimensions of the transverse post-tensioning tendons is shown in the panels detail sheets in the Appendix.

6.5 Mix Design

Developing a mix design for precast pavement panels was one of the more challenging aspects for the precast supplier. As precast pavement is a new technology, TxDOT did not have specifications for the mix design. In general, concrete pavements in Texas are made with Type I/II cement with up to 35% fly ash replacement. In order for the precast supplier to be productive, it was not possible for them to use a typical pavement mix design. To make the best use of the casting beds, the precast supplier needed to be able to remove the panels from the forms the day after casting to prepare for the next casting. This required the precast supplier to use a typical precast bridge beam mix design which would give the necessary release strength of 3,500 psi within 24 hours and a 28-day strength of 5,000 psi. Bridge beams mix designs, however, are generally stiff, and do not permit finishing of a large surface area, as required for pavement panels. This required the use of a superplasticizer to increase workability.

The final mix design consisted of seven sacks of Type III cement, with a water/cement ratio of 0.42, superplasticizer, fine aggregate, and 1 1/2 in. maximum size coarse aggregate. Trial batches demonstrated this mix design to meet the strength requirements as well as the workability requirements for finishing such a large surface area. This mix did, however, generate significant heat in the panels as they cured. Temperature instrumentation in the panels recorded temperatures as high as 160 °F less than 8 hours after casting. This may have contributed to minor mid-slab cracking in the full-width panels, discussed in Section 6.9.

6.6 Finishing and Curing

As stated in the previous section, the precast supplier was required to develop a mix design which would not only meet the necessary strength requirements, but would allow them to finish the large surface area to acceptable pavement surface standards. The addition of a superplasticizer greatly improved the workability of a normally stiff concrete mix so that the surface of the panels could be finished. A vibratory screed, the full width of the casting bed was used to consolidate the concrete to provide a uniformly flat surface. To minimize water loss from the concrete during placement and finishing, an intermediate curing compound, similar to monomolecular film, was sprayed onto the surface of the panels. After the concrete had set to the point that a texture could be applied to the surface, a carpet drag finish was applied to the panels using a course Astroturf mat. Immediately after the carpet drag finish, two coats of curing compound were applied to the top surface of the panels, similar to conventional pavement construction.

After the panels were removed from the forms, usually 24 hours after casting, they were stacked on level two-point supports with battens between each of the panels. Each stack of panels was then covered with wet cotton mats and canvas tarps to allow the panels to cure for an additional 24 hours. Although the surface of the panels was coated with curing compound, the wet mats helped to minimize water loss from the edges and bottom surface of the panels as they were stored.

6.7 Handling and Storage

The lifting anchors used for the panels, shown in Figure 6.3, are referred to as “swift-lifts.” These lifting anchors allow for rapid attachment and detachment of the lifting lines to the panels using special lifting hooks. This allowed for much faster placement of the panels over conventional screw-type lifting devices. The one disadvantage of these lifting anchors, however, was the recess (approximately 4 in. in diameter) left in the surface of the panels, which had to be patched after placement of the panels.

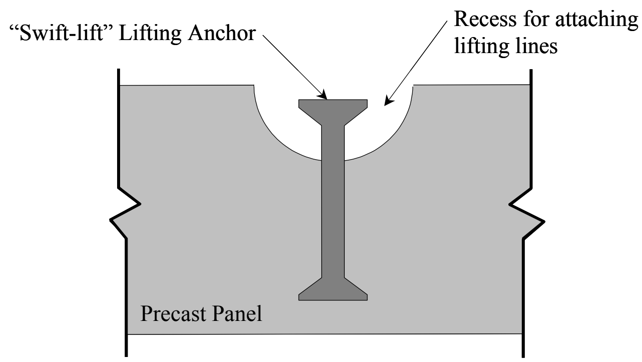


Figure 6.3 *Lifting Devices*

Four lifting anchors were cast into each panel, located between 0.2L and 0.25L from the edges of the panels, where "L" is the length of the side of the panel. A distance of 0.2071L minimizes the stresses in the panels due to bending moments during lifting, which is particularly important along the long axis of the panels. The prestress force from pretensioning helped to counteract the stresses generated in the panel during lifting. The prestress force was determined such that cracking would not occur when lifting the 36 ft panels.

After removal from the forms, the panels were always stored with a two-point support. This included during storage at the precast plant and during shipment on a flat-bed truck. The supporting battens were placed approximately at 0.2L from the ends of the panels and extended the full 10 ft width of the panels. The panels were stacked no higher than five panels at the precast plant.

6.8 Panel Repairs

From the start of casting, it was decided that repairs to panels damaged at the precast plant would be handled on a case-by-case basis by TxDOT inspectors. Damage to the panels generally occurred during removal from the forms or removal of the pocket formers, and was usually minor. Damage to a non-critical part of the panel, such as a corner break at the end of the panel, was not required to be repaired unless the corner break extended more than 12 in. from the outside edge of the panel. Damage to a critical part of the panel, however, was required to be repaired. This included damage to the top surface of the panel, such as spalling around the pockets, or damage to the keyways, particularly the top surfaces of the keyways.

Repairs to the keyways entailed sawcutting the repair area to "square-up" the repair area so that a minimum depth of 1 in. of patching material could be applied. Repairs to the surface of the panels required sawcutting to a minimum depth of 2 in. into the surface, ensuring that all corners on repairs were greater than 90°. Patching material for small repairs, such as repairs to the keyways, was a high-strength cementicious patching material. Patching for larger repairs to the surface of the panels was generally done after panel placement on site, using a normal strength concrete mix.

6.9 Challenges/Problems Encountered

Only minor problems were encountered during fabrication; some delayed production significantly, but most were minor issues which were easily corrected.

6.9.1 Straightness of Armored Joints

The biggest challenge during fabrication, which significantly delayed production, was the straightness of the armored joints. When the armored joint assemblies arrived from the steel fabricator most were significantly warped or bowed, requiring the precast supplier to straighten them before casting them into the joint panels. It is believed that the heat from welding the steel angle to the bottom of the joint receiver severely distorted the joint structure. Through a process of reheating the joint and tack welding it in a straight position, the precast supplier was able to straighten each of the joints to within specified tolerances.

6.9.2 Mid-panel Cracking

Another issue which was discovered during fabrication, but did not delay production, was the formation of a single hairline crack at approximately the middle of the 36 ft panels. The cracks would generally form less than 12 hours after concrete placement. The cracks would extend across the full 10 ft width of the panels, and penetrate approximately one-third of the depth of the panels. However, after release of prestress the cracks closed and were no longer noticeable. The prestress in the panels should prevent the cracks from ever opening in the future.

The cause of the mid-panel cracking is believed to be the temperature change experienced by the panels during the first 24 hours after casting. Temperature instrumentation in the panels recorded internal temperatures as high as 160 °F less than 8 hours after concrete placement. This is due to both the high heat of hydration of the concrete mix (7 sack, Type III) and high ambient temperatures during June–October when most of the panels were cast. Because the panels were cast early in the morning, the heat from cement hydration reached a maximum just as the ambient temperature was at a maximum. As the panels cooled overnight, however, the concrete would begin to contract. Because the panels were restrained from movement by the forms and pretensioning strands, stresses developed in the panels that caused the cracking. This behavior is similar to that of cast-in-place continuously reinforced concrete pavement during the first 24–48 hours after placement. Although mid-panel cracking was observed in most of the 36 ft panels, it rarely occurred in the 20 ft panels and never occurred in the 16 ft panels.

6.9.3 Movement of Post-tensioning Ducts

One issue observed early in the fabrication process was movement of the post-tensioning ducts as the concrete was placed in the forms. Even with the bar stiffeners in the post-tensioning ducts to hold them straight, the movement of a large mass of fresh concrete caused some of the ducts to bow horizontally. This issue was quickly solved by tying the ducts to all six of the pretensioning strands and by moving the concrete hopper along the length of the panel as concrete was being placed, rather than pouring it at one end and vibrating it down the length of the panel. The few post-tensioning ducts that were bowed did not cause a problem with post-tensioning in the field.

7. Pavement Construction

7.1 Introduction

The lab testing and trial assembly at the precast plant (Chapter 5) demonstrated the viability of full-width panel assembly. Upon completion of the trial assembly, approval was given for panel production and pavement construction. Pavement construction included placement of the asphalt leveling course, panel installation, post-tensioning, and grouting. Each of these aspects and the challenges encountered with each will be discussed below.

7.2 Base Preparation

After finishing construction of the bridge, the embankment fill material was placed and compacted to specification on either side of the bridge. The 2 in. thick asphalt leveling course was then placed over the embankment material. Type C (TxDOT specification) hot-mix asphalt concrete (HMAC) was specified for the leveling course. The leveling course was placed in three sections, each 12 ft wide. To ensure that the leveling course was as uniform as possible, thickness guides were staked to the embankment material every 25 ft along the length of the test section.

After final compaction of the asphalt leveling course, any obvious defects were marked and repaired. This included any high points, which may have caused the precast panels to rest unevenly, and any large depressions which may have created voids beneath the panels.

7.3 Transportation to Site

The panels were transported to the job site on flatbed semi-trucks. Due to weight restrictions, only one full-width or 2–3 partial-width panels could be transported on each truck. The panels were shipped from the precast plant in Victoria, Texas, approximately 150 miles to the project site just north of Georgetown, Texas. The panels were supported on the truck similarly to how they were supported at the precast plant, with two-point supports. The panels were strapped down using nylon straps to prevent damage during shipment.

7.4 Full-Width Panel Placement

As described in Chapter 3, the full-width panels were placed on the north side of the bridge, beginning at the bridge and moving towards the adjoining AC pavement. In total, 123 full-width panels were placed accounting for 2,460 lane-ft of pavement.

7.4.1 Procedure/Staging

The panels were placed with the male keyway facing the bridge. The trial assemblies (described in Chapter 5) showed that the most efficient method for panel assembly was to lower the panels into place at a “nose-down” angle with the nose of the male keyway mating into the adjacent panel.

A slow-setting (24-hour set) epoxy adhesive, similar to that used for segmental bridge construction, was applied to the keyways of each panel to seal the joint between panels and provide some degree of lubrication for assembly.

The polyethylene sheeting was rolled out prior to the placement of each panel. As each panel was lowered into place, come-alongs linked between the edge sleeves at the ends of the panels were used to pull the panels together while they were slightly suspended. Three panels were linked together with come-alongs at any given time to prevent the joints between them from opening up as subsequent panels were placed.

A 60 ton crane was used to set all of the panels in place. The crane was set up on half of the leveling course to allow the transport trucks to back in next to the crane. This minimized the distance the panels had to be moved into place. Lifting lines approximately 20 ft in length were used to minimize the angle of the lifting lines with respect to vertical to reduce bending stresses in the panel during lifting.

After each panel was set in place, a short (20 ft) length of post-tensioning strand was fed into the ducts of the panel to ensure the ducts were aligned with the previous panel. Occasionally, it was necessary to offset panels slightly to align the post-tensioning ducts.

7.4.2 Placement Rate

Panel placement rate was primarily dictated by how many panels could be set in place before the crane had to be moved, assuming the supply of panels was sufficient. Moving the crane took approximately 10–15 minutes. For the full-width panels, only two panels were set in place before the crane was moved. This was done to minimize the length of the boom on the crane, which affected the amount of sway as each panel was lifted. Panel placement rate increased as the project progressed. Initially, it took approximately 8 hours to place one section of 25 panels (see Figure 3.3). Placement of the final set of full-width panels, however, was completed in approximately 6 hours.

Panel placement rate was also dictated by the number of workers present and any complications encountered. A minimum of two workers was required for preparing and applying the epoxy, two workers to maneuver the panels into place, and one worker to direct the crane operator. Additional workers to help maneuver the panels, check alignment of the ducts, and help move the crane greatly improved the placement rate.

7.5 Partial-Width Panel Placement

Partial-width panel placement proceeded in much the same way as full-width panel placement, with the exception of placing two sections of panels next to each other. In total, 216 partial-width panels were placed, accounting for 2,160 lane-ft of pavement.

7.5.1 Procedure/Staging

Similarly to the full-width panels, the partial-width panels were oriented with the nose of the male keyway facing the bridge. However, three different placement strategies were tested for the partial-width panels. For the first strategy, one section (250 ft) of 20 ft panels were placed and post-tensioned longitudinally. The adjacent section of 16 ft panels were then placed and post-tensioned longitudinally. The two sections were then post-tensioned together transversely. This strategy was used to simulate a one-lane operation where only one lane of pavement can be placed at a time. This proved to be a viable strategy, although it did require the crane to swing each panel 180° from the transport truck to its final position, which may create problems for future one-lane operations.

The second placement strategy entailed placing a 20 ft panel and its adjacent 16 ft panel simultaneously. The panels were then post-tensioned longitudinally separately before being post-tensioned together transversely. Although this strategy is not applicable for a one-lane operation, it proved to be a viable method for future applications where partial-width panels are necessary for replacement of multiple lanes of pavement.

The third placement strategy involved unloading all precast panels and stacking them along the frontage road. The panels were then set in place the following day from where they were stacked. Although this was the least efficient placement strategy, it may be useful for future applications where the panels must be stockpiled on site prior to being set in place.

Using the first placement strategy, the panels were pulled together as they were lowered into place using come-alongs, similarly to the full-width panels. The 20 ft panels were pulled together using edge sleeves cast into the ends of the panels, while the 16 ft panels utilized sleeves on the outside edge of the panels and vertical sleeves cast into the top of the panels at the longitudinal joint (see panel detail drawings in the Appendix).

Using the second and third placement strategies, it was not possible to use the edge sleeves to assemble the 20 ft panels. Therefore, temporary post-tensioning strands were used to pull the panels together. Two 0.5 in. post-tensioning strands were fed through the ducts of each panel as it was set in place. The strands were located approximately one third of the panel length from each end. In general, 2–3 panels were placed then temporarily post-tensioned as the crane was being moved. Although this was a more time-consuming and labor-intensive process, it resulted in significantly tighter joints than just using the come-alongs and edge sleeves.

7.5.2 Placement Rate

The placement rate for the partial-width panels was slightly faster in terms of how many panels could be placed in a daily operation, but slower in terms of the amount (lane-ft) of pavement placed. Because of the lighter weight of the partial-width panels, more panels could be placed before it was necessary to move the crane. However, because each panel only accounted for one traffic lane, fewer lane-ft of pavement were placed with each panel. Placement of a single section (250 ft) of panels (either 16 ft or 20 ft panels) could be completed in approximately 6 hours. Placement of a section of both 16 ft and 20 ft panels, however, generally took more than 8 hours to complete.

As with full-width panel placement, placement rate was dictated by the number of workers present, and any complications that were encountered. Placement of the partial-width panels required special attention to the longitudinal joint between the 16 ft and 20 ft panels. Ensuring vertical alignment across the longitudinal joint occasionally caused delays in placement.

7.6 Post-Tensioning

As discussed in Chapter 2, post-tensioning greatly benefits the durability of precast concrete pavement. Post-tensioning not only ties all of the precast panels together, providing load transfer between panels, but greatly reduces cracking that will occur over the life of the pavement. Although post-tensioning does add additional complexity to fabrication and an extra step to panel assembly, the benefits it produces far outweigh these additional processes.

7.6.1 Longitudinal Post-tensioning

After placement of a section of panels (between expansion joints), the longitudinal post-tensioning strands were fed into the ducts, starting at the stressing pockets, and threaded through the panels in both directions to the anchorage in the joint panels. To prevent the strands from binding in the ducts, the end of each strand was beveled and a steel "bullet nose" cap was placed over the end of the strand. Once each strand reached the access pocket in the joint panel, the bullet nose was removed and the strand was inspected to ensure all seven wires were still tightly wound. The strands were then slowly pushed into the self-locking post-tensioning anchors. A vibratory hammer attached to the non-anchored end of the strand was used to vibrate the strand, as it was pushed into the anchor, to facilitate seating of the wedges around the strand. Because the longitudinal strands were anchored blindly, i.e., the anchors could not be visually inspected to ensure the strand was fully into the anchor and the wedges were properly seated around the strand, the distance each strand was pushed was carefully measured and recorded. The "tails" or non-anchored ends of the strands in the stressing pockets were also inspected to ensure that all seven wires of the strand were the same length. If there was any question as to whether the strand was properly seated, the strand was not stressed.

After all of the strands were anchored at the joint panels, the strands coming into each stressing pocket from either end of the slab were coupled together using a "ring anchor" (see Figure 2.5) and stressed. The strands were tensioned to 80% of their ultimate strength or 46.8 kips as specified in the plans using a monostrand stressing ram. The elongation of each tendon was measured by marking both strands in each pocket at known locations (with 20% of the ultimate load on the tendon), then measuring the movement of each mark after completion of stressing. The total elongation for each tendon was taken as the sum of the elongation of both strands.

The post-tensioning strands were spaced at 24 in. across the width of the pavement, resulting in 18 tendons to post-tension. Initially, stressing began with the tendons at the center of the slab, alternating outward to the tendons at the outside edges. However, due to safety concerns, it was decided instead to stress tendons sequentially, starting at one side of the slab and moving across to the other side.

Post-tensioning was required to be completed within 24 hours after placement of the panels to prevent the epoxy in the panel joints from setting up prior to stressing. Although generally less than 0.05 in., as much as 0.1 in. of closure was measured across the transverse joints from post-tensioning. Draw-in of the ends of the slab (at the joint panels) from post-tensioning was generally between 1/8 in. and 3/8 in. Temporary post-tensioning during assembly (partial-width panels) resulted in somewhat tighter transverse joints and reduced the amount of draw-in during final post-tensioning.

7.6.2 Transverse Post-tensioning

Transverse post-tensioning proved to be a simpler operation than longitudinal post-tensioning. After each set of 16 ft and 20 ft panels were in place, the transverse post-tensioning strands were fed into the ducts at the edge of the slab and pushed through both panels. Two 0.5 in. 7-wire strands were fed into each duct and anchored at standard dead-end anchors at the edges of the pavement. Although alignment of the transverse ducts was critical, the flat, oversized ducts (described in Chapter 6) permitted slight misalignment of the partial-width panels. Similarly to the longitudinal tendons, the transverse post-tensioning strands were stressed to 80% of their ultimate strength with a monostrand stressing ram. Elongations were measured and recorded for each strand.

7.7 Mid-Slab Anchor

To ensure the finished slabs would expand and contract outward from the center of the slab, it was necessary to anchor the center of each slab to the subbase. The method used to anchor the slab involved drilling and grouting anchor pins through the asphalt leveling course into the subbase at the stressing pockets. Two #8 deformed bars 1 ft in length were drilled and grouted into the subbase at each stressing pocket, as shown in Figure 7.1. The plans required that 4–6 in. of each bar protrude into the pockets. Filling the stressing pockets tied the anchor pins to the pavement slab, providing an economical and efficient solution for the mid-slab anchor.

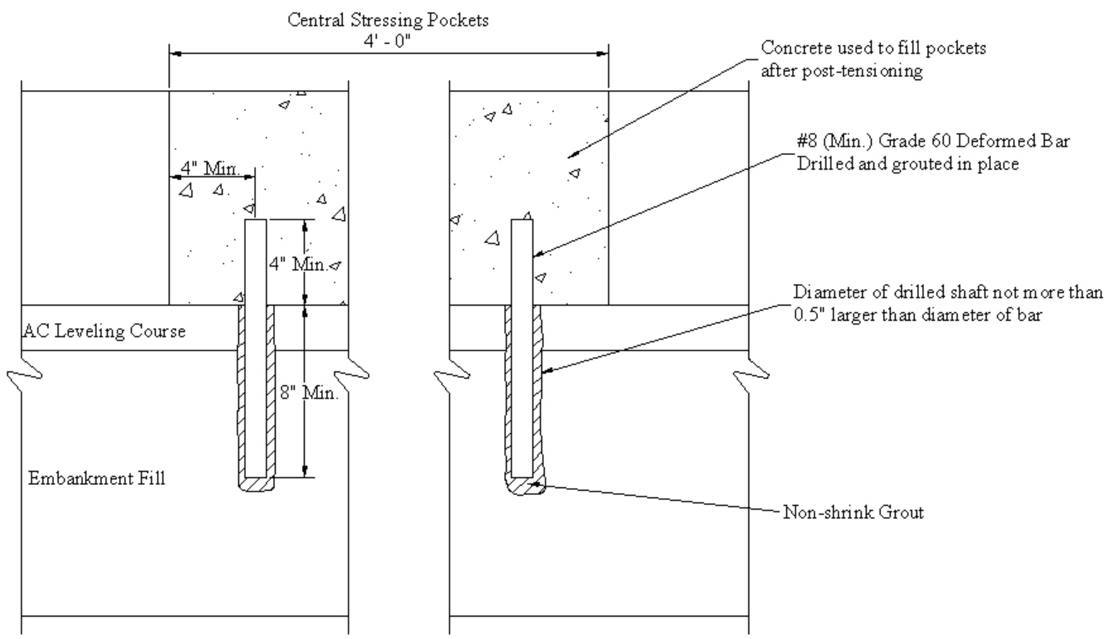


Figure 7.1 *Mid-slab anchor at the central stressing pockets*

7.8 Filling Pockets

After completion of post-tensioning and drilling and grouting the mid-slab anchors, the stressing pockets (central stressing panels) and anchor access pockets (joint panels) were filled. Although a fast-setting concrete was specified for the pockets, to allow traffic onto the pavement as soon as possible, the contractor opted to use normal set concrete as the frontage road was closed to traffic and the cost of fast-setting concrete was significantly higher. A carped drag texture was applied to the pockets to match the surrounding surface.

7.9 Grouting

Grouting post-tensioning tendons provides an extra layer of corrosion protection for the post-tensioning strands. This is particularly important at the transverse joints between precast panels where the duct is not continuous across the joint. Grouting also bonds the post-tensioning strands to the pavement. This will allow individual precast panels to be cut out and replaced in the future if needed without the need to de-tension the strands first. Although grouting does add an additional step to the construction process, the benefits are significant and should far outweigh the additional construction requirement.

7.9.1 Longitudinal Tendon Grouting

As described in Chapter 6, grout vents were cast into the joint panels—just in front of each post-tensioning anchor, into the central stressing panels—on either side of the stressing pockets, and in every fourth base panel. This resulted in tendons approximately 125 ft in length for the standard 250 ft slabs and 162 ft in length for the 325 ft slab, with intermediate grout vents spaced approximately every 40 ft for the 250 ft slabs and every 55 ft for the 325 ft slab. Although grout vents are generally not needed so close together there was a great deal of uncertainty with an experimental project such as this.

The grouting operation was started only after the stressing pockets and access pockets were filled. Grout was pumped from one end of each tendon, either at the joint panel or central stressing panels, to the other end, monitoring the movement of the grout at the intermediate grout vents. Grout was pumped until the efflux time of the grout at the outlet was the same as that at the inlet or until the pressure reached 150 psi (higher pressures caused the grout tube to separate from the duct). A pre-packaged cable grout was used for the longitudinal tendons. The grout was mixed in accordance with the manufacturer's recommendations. The fluidity of the grout was checked regularly with a flow cone using ASTM standard test method C 939. For the longitudinal tendons, the efflux time was generally between 15 and 20 seconds, depending on the weather conditions.

As will be discussed in more detail Section 7.10, fully grouting each tendon proved to be a tedious task. Grout tended to leak out of the ducts at the transverse joints, entering adjacent ducts and leaking out of the top and ends of the joints. Grout was pumped into each tendon until it either came out the correct vent on the same tendon, or until leakage was noticed. In order to completely grout each tendon, grout was pumped into each grout vent, including the intermediate vents. The movement of the grout was recorded for each tendon of each slab, as shown in the Appendix.

7.9.2 Transverse Tendon Grouting

Transverse tendon grouting proved to be a much simpler task than longitudinal grouting. As stated in Chapter 6, the transverse tendon grout vents were located below the post-tensioning anchors at the outside edges of the pavement. Due to the cross-slope of the pavement, one end of the transverse tendons was lower than the other. Grout was pumped from the low end of the tendon and vented at the high end. Grout was pumped until grout flowed out of the vent as well as from around the wedges of the anchors. For approximately six of the tendons, grout would not flow past the longitudinal joint and had to be pumped from the high end of the tendon as well.

Similar to the longitudinal tendons, the fluidity of the grout was measured with a flow cone. A thicker grout, with an efflux time 20 seconds, was used for the transverse tendons, because of the shorter tendon length and larger duct size. In general, no problems were experienced with grouting the transverse tendons. The movement of grout for the transverse tendons was monitored and recorded, as shown in the Appendix.

7.10 Challenges/Problems Encountered

As with any experimental project, construction of the Georgetown precast pavement presented several challenges. However, problems were expected from the beginning and were quickly mediated. The purpose of this first pilot project was to work out the details of precast pavement construction, addressing these problems as they arose in order to streamline the process for future projects. This section will discuss some of the problems encountered during pavement construction and the solutions developed.

7.10.1 Full-Width Panel Placement

Centerline Deviation – The first challenge discovered during full-width panel placement was keeping the centerline of the panels on the centerline of the frontage road. Even if some the panels were only slightly out of square, or if the transverse joint between two panels was not the same width on one side as the other, the centerline of the panels would deviate from the surveyed centerline of the road. To compensate for this, panels were slightly offset horizontally to bring the centerline of the panels back to the centerline of the roadway. However, offsetting the panels caused slight misalignment of the post-tensioning ducts which made pushing the post-tensioning strands more difficult, as will be discussed below. Although offsetting (not more than 1/8 in.) was a viable solution, a better solution was gapping one end of the joint between panels using shims.

Keyway Cracking – Another problem that was quickly discovered during full-width panel placement was the formation of a crack along the top lip of the female keyway as the panels were assembled. The crack was caused by a wedging action from the nose of the male keyway as the panels were assembled which was accentuated by the pressure exerted from the come-alongs. These were only hairline cracks and occurred on less than 25% of the panels. Because this cracking occurred only on the outside edges of the panels, on the shoulder of the finished roadway, no action was taken to repair these cracks.

Leveling Course Crown/Longitudinal Cracking – During the course of full-width panel placement, longitudinal cracking, or cracks across the 10 ft width of the panels, were noticed on several panels soon after placement. Most of the cracks occurred in roughly the same location, approximately 12 ft from one edge of the pavement. Closer examination of the asphalt leveling course revealed a slight crown in the leveling course at the joint between two of the sections of asphalt. It is believed that this slight crown was the cause of the panel cracking. A condition survey of the finished pavement revealed cracks in approximately 65% of the panels. However, these cracks appear to have formed during panel placement and should remain hairline because of the pretensioning in the panels. Accordingly, no measures were taken to repair the cracks, but they will be continuously monitored over the life of the pavement.

7.10.2 Partial-Width Panel Placement

Centerline Deviation – Centerline deviation became an even bigger issue with partial-width panel placement. Offsetting the partial-width panels resulted in an uneven longitudinal joint between the 16 ft and 20 ft panels. Because of this, offsetting was minimized, and joint gapping was used instead.

Panel Squareness – Similarly to the problem with centerline deviation, the squareness of the partial-width panels created some difficulties with panel installation. Although all panels met the tolerance for squareness, even slight out-of-square caused both centerline deviation and a non-uniform longitudinal joint between the 16 ft and 20 ft panels. In a few instances, the longitudinal joint between a 16 ft and 20 ft panel was as wide as 1/4 in. at one end and closed at the other end. Transverse post-tensioning did help to some degree to close these joints, however, and gapping the transverse joints helped to alleviate this problem. Overall, panel squareness was not a major problem, but should be carefully considered for future projects.

Corner Cracking – One issue created by having panels slightly out of square was the development of stress concentrations during transverse post-tensioning at the corners of the panels. This resulted in minor corner breaks at the surface of the pavement where the corners of the 16 ft and 20 ft panels came together. Although this was not an extensive problem (10–15 instances), it occurred on the riding surface at the centerline of the road. These corner breaks were repaired by saw cutting into the surface 2 in. deep to remove the break, then patching the area that was removed.

Asphalt leveling course – The asphalt leveling course beneath the partial-width panels experienced a significant amount of local traffic before panel placement. This exposure to traffic resulted in some depressions along the leveling course which created voids beneath the partial-width panels. In addition, the final 30 ft of the leveling course was placed much later than the original leveling course, with a rough finish, resulting in voids beneath the panels. To ensure voids were minimized as much as possible, holes were drilled into the panels for underslab grouting. A non-shrink portland cement grout was pumped under slight pressure (< 5 psi) beneath the partial-width panels to fill these voids.

7.10.3 Post-Tensioning

Blind Anchorage – Laboratory testing showed spring-loaded post-tensioning anchors cast into the joint panels to be a viable alternative to standard post-tensioning anchors. Large-scale implementation in the field, however, revealed some of the difficulties of using blind anchors. It was first discovered that while pushing the strand through the duct, friction caused some of the individual wires to slide back along the strand, leaving only five of the seven wires to go into the anchor. In general, the wires would only slide back 1–3 in. This problem was solved by putting a steel bullet-nose on the end of the strand prior to pushing it through the duct. The bullet nose was removed at the access pocket and the end of the strand was inspected. If any wires had slid back, the end of the strand was cut flush so all of the wires were the same length before being pushed into the anchor.

A similar problem was also encountered as the strands were pushed into the anchor (after being inspected at the access pockets). It was discovered that one to two of the individual wires would catch on a lip around the entrance to the post-tensioning anchor, leaving only five to six of the wires to go into wedges. This problem was alleviated by first grinding a chamfer onto the end of the strand, then rotating and vibrating the strand as it was pushed into the anchor. The combination of vibration and rotation prevented the individual wires from catching on the lip of the anchor.

Pushing Strands through Ducts – Another problem encountered with post-tensioning was resistance to pushing the strands through the panels. For the majority of the post-tensioning tendons, the strands could easily be pushed the length of the post-tensioning duct by hand with little resistance. However, in a few instances, the strand could not be pushed through the entire duct and it was necessary to cut into the top of the slab to clear obstructions. It was discovered that the primary cause of this resistance was misalignment of ducts due to offsetting the panels during placement. Gapping the joints between panels ensured alignment of the ducts and greatly reduced the occurrence of this problem.

7.10.4 Grouting

Grout Leakage – Grout leakage presented the biggest challenge to the grouting operation. Leakage occurred at the transverse joints with grout crossing over between ducts, leaking out of the top and bottom of the slab, and leaking out of the ends of the joints at the slab edge. Epoxy applied to the top and ends of the joints prevented external leakage, but nothing could be done to prevent crossover between ducts. Crossover was evident when grout was pumped into one duct and flowed out the vent of an adjacent duct. It became necessary to pump grout into every grout vent (including intermediate vents) to ensure the tendons were grouted as much as possible. It is believed that a significant amount of grout also leaked out the bottom of the joints beneath the slabs. Although this required a significant amount of extra grout, it most likely filled many small voids beneath the panels and should benefit the performance of the finished pavement. It was found that a heavier application of epoxy and the use of temporary post-tensioning during panel assembly greatly reduced the amount of grout leakage.

Grout Vent Blockage – Another challenge to grouting was blockage of many of the grout vents. Blockage was caused primarily by the post-tensioning strand pressing against the top of the post-tensioning ducts, covering the hole at the grout vents. This prevented grout from being pumped into the duct and required grouting from a different grout vent. Additionally, when the panels were cast, the duct was left as a continuous piece through the "T" at the grout vents. After casting, a hole was drilled into the duct at the T to allow grout to flow into the duct. It was found that this hole was usually not large enough and restricted the flow of grout into the duct. In several instances it was necessary to enlarge the hole by burning the plastic duct at the grout vent.

8. Instrumentation and Monitoring

8.1 Introduction

Instrumentation and monitoring the behavior of the Georgetown precast pavement is important for comparison to the behavior predicted during the design. This information will be used to calibrate computer design programs to better predict the behavior of precast prestressed concrete pavements, resulting in less conservative designs and savings in construction costs. As stated in Chapter 4, the design of the Georgetown precast pavement is extremely conservative due to the lack of better information on pavement behavior for design purposes.

8.2 Variables Monitored

In Chapter 4, several design considerations for prestressed pavements were presented. The main design considerations involve pavement response to environmental conditions, namely temperature. Daily and seasonal temperature cycles cause the pavement to expand and contract significantly. This movement is partially restrained by friction between the bottom of the pavement and the leveling course, resulting in prestress losses and stresses in the pavement. Temperature cycles also cause temperature gradients to develop over the depth of the slab, causing the slab to curl upward or downward. This vertical curling movement also causes significant stresses in the slab.

With these considerations in mind, the primary variables that must be monitored for the Georgetown precast pavement are:

- Concrete Temperature
  - Mid-depth
  - Top-Bottom differential
- Horizontal movement (width of expansion joints)
- Vertical movement (curling of slab ends)
- Ambient Temperature (for correlation to concrete temperature)

Each of these variables should be monitored in varying climatic conditions over the life of the pavement to ensure the behavior is well understood. At minimum, each variable should be monitored during winter and summer climatic conditions soon after construction and near the end of the service life of the pavement.

8.3 Temperature Instrumentation

Knowing the concrete temperature of a prestressed pavement is essential for correlation with the movement of the slab. Temperature instrumentation for prestressed pavements should measure top, mid-depth, and bottom temperatures of the slab. Mid-depth temperature provides a correlation for horizontal movement of the slab while the top-bottom temperature differential provides a correlation for the vertical (curling) movement at the ends of the slab.

Temperature instrumentation can be very difficult for concrete pavement. To obtain temperature data after the pavement is already in place, holes must be drilled into the slab for inserting a temperature device such as a thermocouple. This can be time consuming, and will only provide temperature data at discrete times unless an external logging device, such as a maturity meter, is left near the pavement. These logging devices, however, are susceptible to being stolen or damaged.

The temperature instrumentation used for the Georgetown precast pavement is a new device, which not only measures temperature, but also logs the temperature data internally. The devices are called i-Buttons® and are roughly 5/8 in. in diameter and 1/4 in. thick. Multiple i-Buttons® can be cast into the pavement at the various depths required for measurement. A single twisted-pair wire extending out of the edge of the pavement is used to download the data from the i-Button® to a personal computer. The computer is also used to "mission" the i-Buttons® by programming the sampling rate, sampling start delay, and temperature alarms into the device. The i-Buttons® can log approximately 2,000 data points at intervals as short as one minute, and the battery in the i-Buttons® can last up to 10 years depending on the sampling rate. Each individual i-Button® has a unique serial number which is recognized by the computer software to ensure data from each i-Button® is kept separate from the others.

A total of twenty-five sets of i-Buttons® were cast into the pavement panels for the Georgetown precast pavement. Twelve chains of three i-Buttons®, similar to that shown in Figure 8.1, were cast into the panels to obtain top, bottom, and multi-depth temperatures. The chains of three i-Buttons® were attached to a 6 in. length of rebar to hold them at the specified depths during casting. The rebar was tied to reinforcement in the precast panel to hold it in place, with the bottom i-Button® held one inch from the bottom of the panel. Thirteen additional sets of single i-Buttons® were cast into the panels to obtain mid-depth temperature.

The twisted-pair wire from each set of i-Buttons® was routed to a junction box cast into the end of the precast panel (outside edge of the pavement slab) where the RJ-11 (phone type) connector could be accessed after pavement construction was completed, as shown in Figure 8.2. The RJ-11 connector fits into a serial port adapter on a personal computer for downloading data or to mission the i-Button®. Each set of i-Buttons® was located a minimum of 24 in. from the end of the panel and 18 in. from the side of the panel to ensure the temperature measurement is representative of the whole pavement.

i-Buttons® were cast into almost every joint panel, as the end of the slab is where curling and horizontal movements are measured. Four sets were also cast into central stressing panels for comparison of slab end temperature to mid-slab temperature. Figure 8.3 shows the location of the temperature instrumentation. In general, the chains of three i-Buttons® were cast into the slabs where curling is measured.

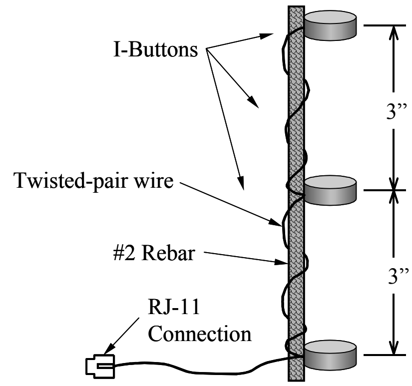


Figure 8.1 *Chain of three i-Buttons® tied to a length of rebar to obtain top, mid-depth, and bottom slab temperatures*

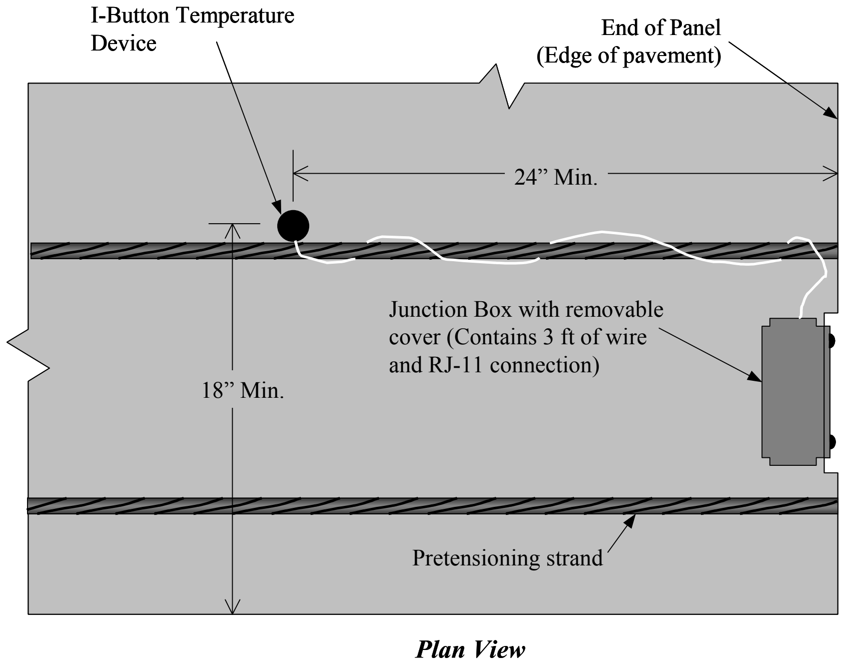


Figure 8.2 *Typical layout for a set of i-Buttons® with the twisted-pair wire from the i-Buttons routed to a junction box at the end of the panel (edge of the pavement)*

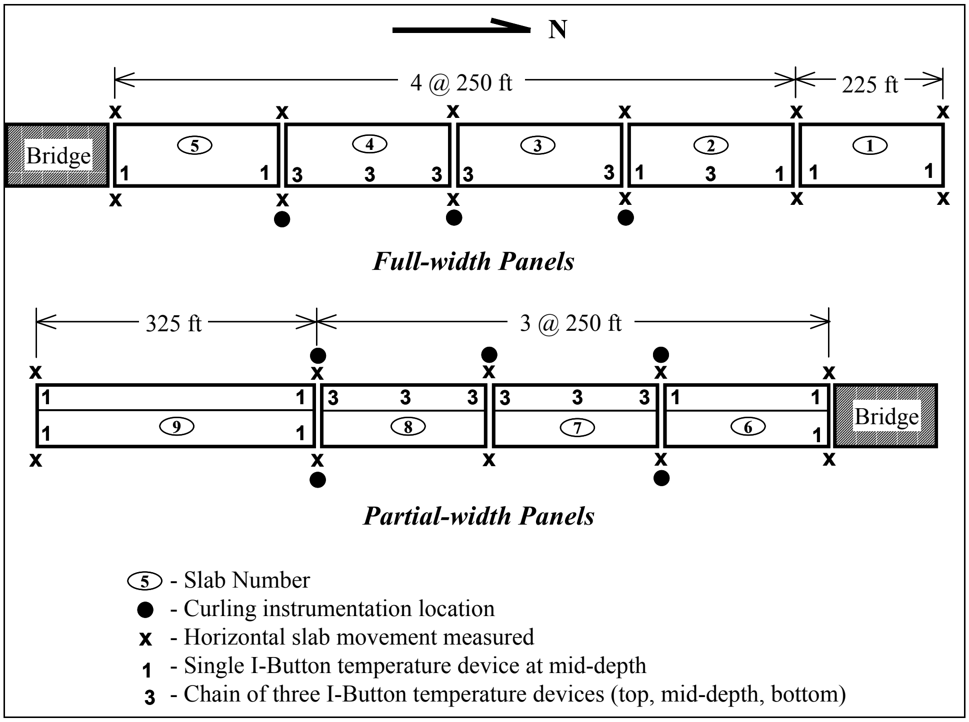


Figure 8.3 *Locations for the i-Buttons®, curling instrumentation, and horizontal movement measurements for the Georgetown precast pavement*

8.4 Curling Instrumentation

Curling instrumentation is used to measure the vertical movement of the ends of the post-tensioned sections of pavement. As discussed previously, vertical movement is caused by a temperature gradient over the depth of the slab. Curling is measured at the expansion joints from a stationary vertical reference point next to the pavement slab.

Curling instrumentation for the Georgetown precast pavement consists of a 6 in. x 12 in. "dead-man" concrete cylinder cast into the base material next to the slab, as shown in Figure 8.4. Horizontal steel arms, attached at a 90-degree angle to a vertical rod threaded into the dead-man cylinder, extend over the edge of the slab. Dial gages fixed to the ends of the horizontal arms measure the vertical and horizontal movement at the expansion joint. To ensure the steel arms and rods do not warp in direct sunlight, they are wrapped with white foam rubber pipe insulation.

Curling instrumentation was installed at three of the expansion joints of both the full-width and partial-width panels, as shown in Figure 8.3. To check for differential movement between the 16 ft and 20 ft partial-width panels, curling instrumentation was installed on both sides of the partial width panels.

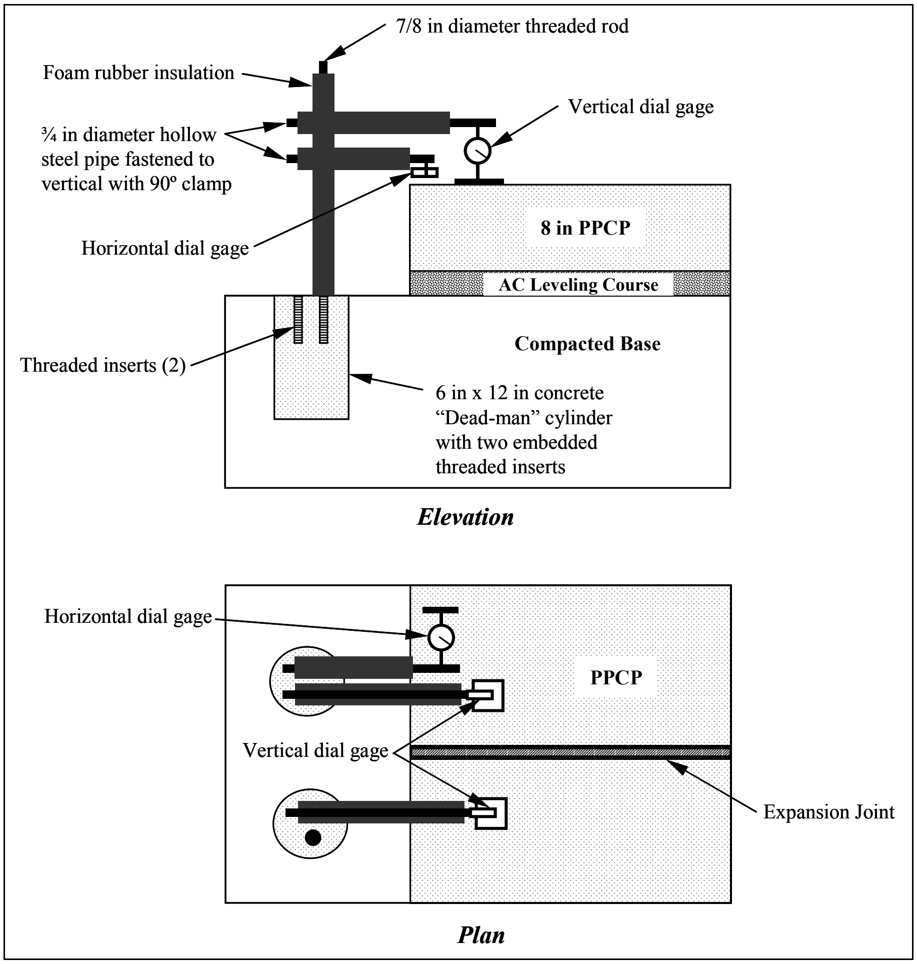


Figure 8.4 *Diagram of the curling instrumentation for the Georgetown precast pavement*

8.5 Horizontal Movement

Horizontal movement is measured to determine how much the post-tensioned slabs are expanding and contracting with temperature. It is an indicator of both the thermal expansion characteristics of the concrete as well as the amount of restraint from frictional resistance at the slab-base interface. This data is used for comparison to the horizontal movement predicted by the PSCP2 program during the design of the pavement.

Horizontal movement is determined by measuring the width of the expansion joints using dial calipers. Each measurement is taken at the same place on each expansion joint using reference marks stamped into the steel flange of the armored joint. The measurements are correlated to the mid-depth temperature (from the temperature logging devices) at the time the measurement was taken.

Measuring the width of the expansion joints only gives an indication of the total movement of the slabs on either side of the joint, and does not indicate the movement of each individual slab. Therefore, a horizontal dial indicator is mounted to the dead-man cylinder, as shown in Figure 8.4, to monitor the movement of an individual slab on one side of the joint.

8.6 Preliminary Monitoring

Preliminary monitoring was conducted soon after construction to evaluate initial slab behavior. This included temperature monitoring and monitoring of horizontal slab movement.

8.6.1 Temperature History

The i-Button® temperature devices were set to begin recording temperature just prior to casting the joint panels and central stressing panels. Figure 8.5 shows typical temperature data for the panels during casting and storage at the precast plant. The temperature differentials over the depth of the panel are evident from the i-Button® data. In general, the temperature at the top of the panel is significantly higher that that at the bottom during the afternoon hours when the sun is heating top of the panel. During the evening and early morning hours, however, the top, middle, and bottom temperatures are roughly the same as radiation from the sun is not a factor. As expected, the concrete temperature lags slightly behind the ambient temperature, i.e., the peak concrete temperature occurs just after the peak ambient temperature.

The disposition of the panels over the two-week period shown in Figure 8.5 is indicated at the top of the figure. The concrete temperature during casting and initial curing reached temperatures greater than 160 °F. This was caused by a concrete mix with a high heat of hydration which reached its peak temperature at nearly the same time as the peak ambient temperature. The panels then cooled down to less than 90 °F the following night before prestress was released. This significant temperature drop is the most likely cause of the mid-panel cracking discussed in Chapter 6.

Following casting and initial curing the panels were stacked and covered with a tarp for additional curing, resulting in very small top-bottom temperature differential in the panels. The large top-bottom temperature differential between September 1, 2001–September 5, 2001 indicates the top of the panel was most likely exposed to sunlight either at the precast plant or on the frontage road during that period.

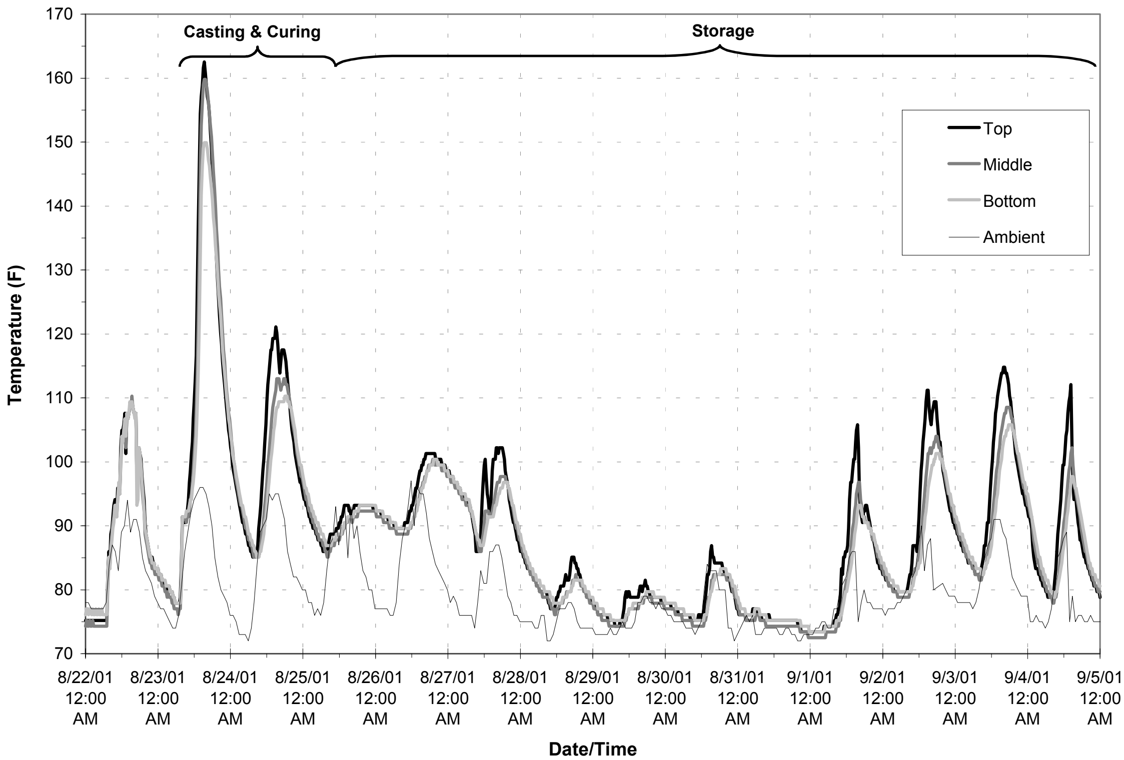


Figure 8.5 *Temperature history for one set of i-Buttons®*
during casting and storage at the precast plant

8.6.2 Horizontal Movement

Horizontal slab movement is determined by measuring the width of the expansion joints with dial calipers. This movement is then correlated with the mid-depth slab temperature to determine the relative movement with temperature. The initial set of horizontal movement data was collected over a 24-hour period from August 22, 2002–August 23, 2002, approximately 5 months after the pavement was opened to traffic. Figure 8.6 shows the temperature history for this first 24-hour monitoring period, representing summer climatic conditions.

Figure 8.7 shows the relative total joint movement with temperature for both the full-width and partial-width panels. This is a direct indication of how much each expansion joint opens or closes with a change in temperature. Measurements were taken on both the east and west sides of each expansion joint approximately 2 ft from the edge of the pavement. The average movement is shown in Figure 8.7. It should be noted that Joints 1, 6, 7, and 11 are only half joints, i.e., there is a post-tensioned slab on only one side of the joint. The movement, therefore, is roughly half of that for joints with full slabs on either side of the expansion joint. Additionally, Joints 1 and 2 are at either end of a 225 ft long slab, and Joints 10 and 11 are at either end of the 325 ft slab, and will exhibit slightly different movement than the joints between 250 ft slabs.

The relative movement of each of the joints is summarized in Table 8.1. As the data shows, the coefficient of variance for the movement of Joints 3, 4, 5, 8, and 9 is very low. This indicates that the joints are all behaving in a similar manner with regard to horizontal movement. The relative movement of Joints 6 and 7 are, likewise, very similar. Although it is not possible to compare the movements of Joints 1, 2, 10, and 11, the relative movement is roughly what would be expected, as shown in Figure 8.7.

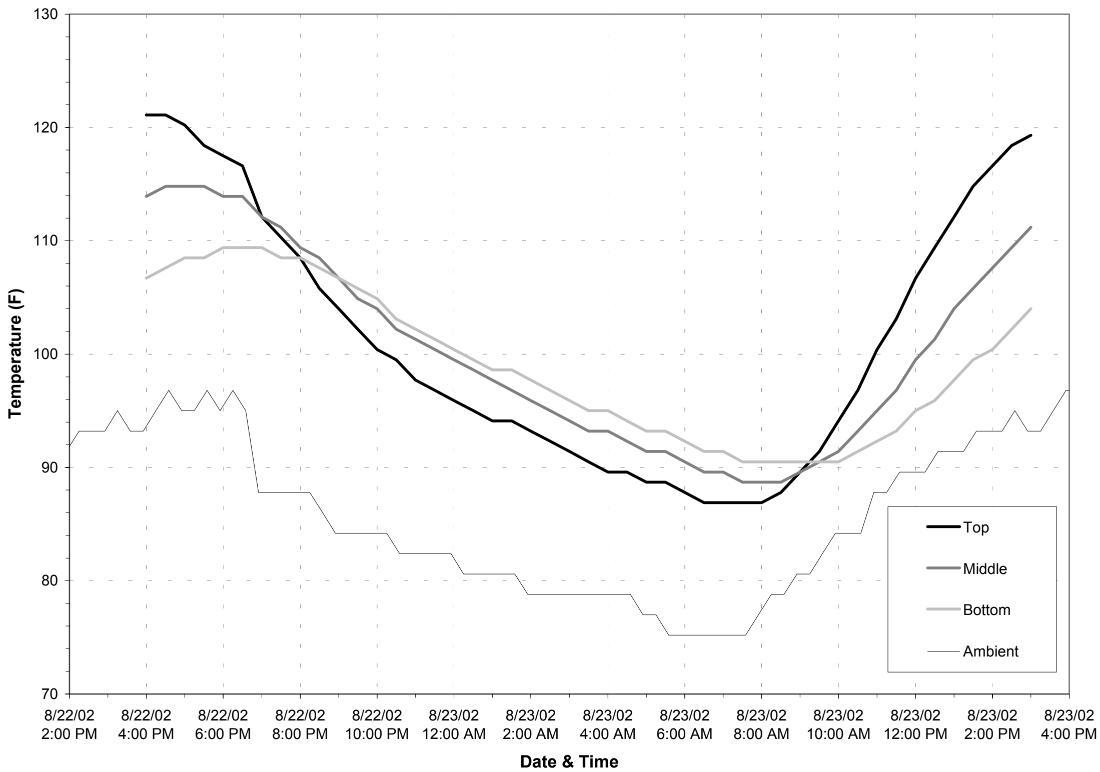


Figure 8.6 *Temperature history for initial monitoring of horizontal slab movement on 8/22/02-8/23/02*

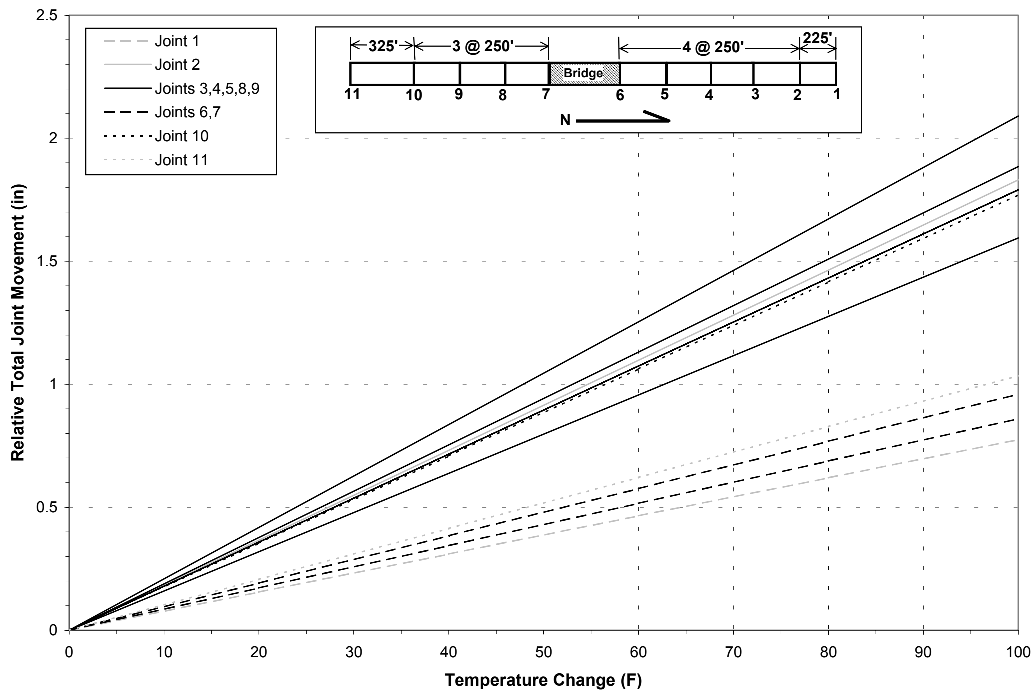


Figure 8.7 *Relative total movement with temperature for the Georgetown PPCP expansion joints as determined by measurements obtained from 8/22/02 – 8/23/02*

Table 8.1 *Summary of Relative movement with temperature for the Georgetown PPCP Expansion Joints*

Joint No.	Slope of Relative Movement	Mean	Coefficient of Variation
1	0.0078	N/A	N/A
2	0.0183	N/A	N/A
3	0.0179	0.0182	8.7%
4	0.0179		
5	0.0189		
8	0.0209		
9	0.0160		
6	0.0086	0.0091	5.5%
7	0.0096		
10	0.0177	N/A	N/A
11	0.0104	N/A	N/A

8.7 Proposed Monitoring Plan

The purpose of long-term monitoring of the Georgetown precast pavement is to examine the performance of the pavement over time for comparison to that predicted during the design.

Monitoring should include measurement of both horizontal and vertical slab movements, as well as temperature history for a minimum of a 24-hour period. Data should be collected during both summer and winter climatic conditions as well as during a spring or fall period. Monitoring during spring or fall climatic conditions will generally result in larger temperature differentials during a daily temperature cycle, resulting in significant temperature gradients over the depth of the pavement. Monitoring should be performed on a clear day with a significant temperature variation so that solar radiation will increase temperature gradients in the slab. As a rule of thumb, monitoring should be conducted when there will be at least a 20 °F ambient temperature differential over the 24-hour monitoring period.

Long-term monitoring should be done at periods approximately 1, 5, 10, and 30 years after construction to provide sufficient information on the long-term behavior of the pavement. For the Georgetown precast pavement, the following monitoring plan shown in Table 8.2, is proposed.

Table 8.2 *Proposed long-term monitoring plan for the Georgetown PPCP*

Time after Construction	**Climate/Season**	**Temperature History**	**Horizontal Movement**	**Vertical Movement**
< 6 months	*Summer*	X	X	
	Winter	X	X	
	Fall/Spring	X	X	X
1 Year	*Summer*	X	X	
	Winter	X	X	
	Fall/Spring	X	X	X
5 Years	*Summer*	X	X	
	Winter	X	X	
	Fall/Spring	X	X	X
10 Years	*Summer*	X	X	
	Winter	X	X	
	Fall/Spring	X	X	X
30 Years	*Summer*	X	X	
	Winter	X	X	
	Fall/Spring	X	X	X

Monitoring vertical slab movement requires frequent attention to the instrumentation, described previously, to ensure it is functioning properly. Horizontal movement, however, can be monitored at almost any time using dial calipers. Accordingly, it is proposed that horizontal movement be monitored more frequently while vertical movement should be monitored during fall/spring conditions when slab temperature gradients are greater.

9. Condition Survey and Post-Construction Testing

9.1 Introduction

Following construction, the researchers evaluated the finished pavement through a thorough condition survey and non-destructive testing. Researchers will use this data for comparison with data collected over the life of the pavement to evaluate the pavement performance.

9.2 Initial Project-Level Condition Survey

A project-level condition survey is a thorough examination of the condition of the pavement. This is used to identify and record any distresses in the pavement immediately after construction, prior to opening to traffic. The record of these distresses will be compared with records from future project-level condition survey to give an indication of the performance of the pavement under traffic and environmental loading. Typical condition surveys for portland cement concrete pavement record transverse crack spacing, longitudinal cracking, random cracking, D-cracking and Y-cracking, heavy spalling, punchouts, and patches. In general, however, these distresses are recorded over selected 1,000 ft sections, and assumed to be representative of the whole pavement. For a project-level condition survey, however, the exact locations of each distress are measured and recorded. For the Georgetown precast pavement, the distresses in each panel were measured and mapped. The distresses that were recorded included:

- Longitudinal pavement cracking (across the width of each panel)
- Transverse pavement cracking
- Random shrinkage cracking
- Condition of pockets and patched lifting anchor recesses
- Repairs (patches) during construction
- Distresses around expansion joints

9.2.1 Cracking

Cracking was the primary distress observed during the initial condition survey. The cracks were fairly minor, however, and should not open up due to the prestress in the pavement.

Longitudinal pavement cracking – The primary cracking distress observed was cracking across many of the full-width panels parallel to the flow of traffic. As discussed in Chapter 7, these cracks were primarily caused by a slight crown in the asphalt leveling course. Approximately 88 of the full-width panels (72%) exhibited some form of cracking. Of those 88 panels, 56 had a single crack, 21 had two cracks, 9 had three cracks, and 2 had minor random shrinkage cracking. However, of the 56 panels with a single crack, only 29 had cracks which extended across the full 10 ft width of the panel. Of the 30 panels with either 2 or 3 cracks, generally only one of the cracks extended across the full width of the panel. No longitudinal cracks were observed in any of the partial-width panels.

Transverse pavement cracking –No transverse cracks (perpendicular to the flow of traffic) were observed in either the full-width or partial-width panels.

Random shrinkage cracking – Only two (full-width) panels showed any random shrinkage cracking. This cracking was very minor, however, with only 5–6 random cracks observed in each panel.

9.2.2 Pockets and Lifting Anchor Recesses

Cracking was observed around several of the stressing and access pockets and lifting anchor recesses. This cracking was generally minor and should not affect the performance of the pavement. While these are probably only surface cracks, the additional reinforcement and post-tensioning strands should prevent a punchout from occurring at the pockets; and the head of the lifting anchors should prevent the recess patches from breaking out.

Stressing Pockets – Approximately 26 (16%) of the central stressing pockets showed minor cracking around the interface between the panel and the concrete used to fill the pockets. The majority of the cracks were 0.010 in. or less in width and generally occurred on only one side of the pocket. This cracking was most likely caused by shrinkage of the concrete used to patch the pockets and poor bond between the panel and the patch concrete.

Access Pockets – Cracking around the access pockets was observed on approximately 27 (7%) of the pockets in the joint panels. These were mostly hairline cracks (0.010 in.) and primarily resulted from propagation of longitudinal cracks (mentioned previously) in the joint panels.

Lifting Anchor Recesses – Minor hairline cracking was observed around approximately 105 (8%) of the patched lifting anchor recesses. Similarly to the stressing and access pockets, this cracking was minor and most likely caused by shrinkage of the patching material and poor bond at the interface between the panel and the patch.

9.2.3 Repairs and Other Minor Distresses

The size and location of any repairs made to the panels during construction, as well as any other minor distresses such as spalling, were recorded for future reference.

Repairs – Several repairs were made to the top surface of the panels during construction. These repairs were primarily caused by corner cracking on the partial-width panels and by cutting into the surface of the panels to clear obstructions in the post-tensioning ducts (discussed in Chapter 7). For the full-width panels there were only 2 corner repairs, located on the inside shoulder, and one repair near the centerline of the pavement. For the partial-width panels, there were 14 corner repairs primarily at the centerline or longitudinal joint, and approximately 21 repairs from cutting into the surface to clear the post-tensioning ducts. With a few exceptions, all repairs were minor, generally less than 1 sq. ft. in size.

Minor Distresses – Approximately 12 minor unrepaired distresses were observed during the condition survey. These distresses generally consisted of minor spalling at a joint between two panels, either at the end of the panels in the shoulder or near the centerline of the pavement. These distresses should not affect the ride quality or performance of the pavement.

9.3 Push-Off Tests

During construction the researchers conducted push-off tests to evaluate the frictional resistance between the precast panels and asphalt leveling course for comparison to the values assumed during the design phase. The assumed values were based on previous experience with post-tensioned (non-precast) pavements. The push-off tests were conducted on the frontage road over the actual asphalt leveling course. The tests were set up at the end of Slab 2, on the north side of the project, where there is no vertical curvature in the road profile. The end of Slab 2 was used as the reactionary force for the tests. The tests were completed on a sunny day with ambient temperatures in the mid-70s (°F).

Figure 9.1 shows the setup for the push-off tests. The precast panel used for the test was one of the panels used for the laboratory testing, described in Chapter 5. The panel dimensions were 120 in. x 62 in. x 8 in. thick, and the total weight of the slab was approximately 5,400 lb. A single hydraulic ram was used to push the precast panel with a spreader beam to distribute the load across the panel. A pressure gauge attached to the hydraulic pump, which was previously calibrated with the ram, was used to monitor the load applied to the panel. Two dial gauges were used to monitor the movement of each side of the slab. For comparison purposes, tests were conducted with and without the polyethylene sheeting beneath the panel. For each condition (with and without polyethylene sheeting), the panel was loaded until sliding occurred, then unloaded, the displacement gauges reset, and loaded again. Loading was applied in 100 psi (pressure gauge) increments or approximately 425 lbs of force until sliding occurred and no additional load was required to move the panel.

Figure 9.2 and Table 9.1 summarize the results from the push-off tests. The results are shown for the two tests for each condition. Test “1” is the initial push-off test while test “2” is the secondary push-off test after panel was unloaded and the gauges were reset. Table 9.1 lists the calculated maximum coefficient of friction and the movement as sliding. The coefficient of friction was calculated by dividing the applied force by the weight of the panel (5,400 lb). The movement at sliding was determined as the panel displacement at which no additional load was required to move the panel.

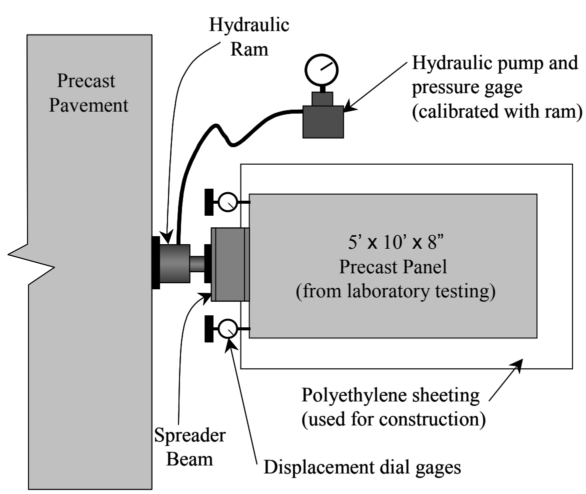


Figure 9.1 *Plan view of the push-off test setup over the actual leveling course*

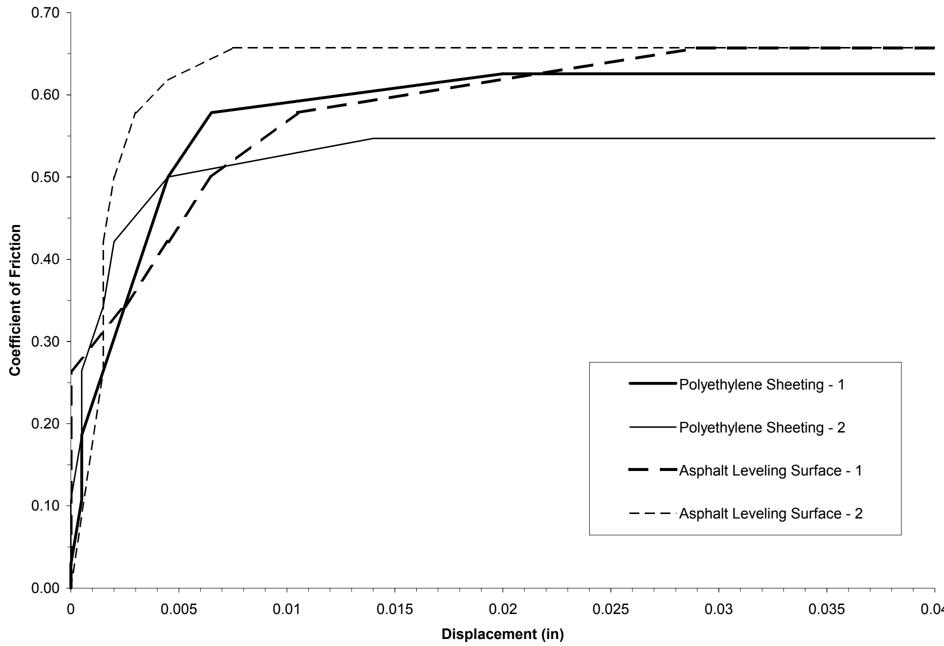


Figure 9.2 *Results of push-off tests showing coefficient of friction vs. panel displacement*

Table 9.1 *Summary of results for the push-off tests with and without polyethylene sheeting*

Condition	Test No. *1 = Initial push-off* *2 = Secondary push-off*	Maximum Coefficient of Friction	Movement at Sliding (in)
Polyethylene Sheeting	1	0.63	0.02
	2	0.55	0.014
Asphalt Leveling Course Surface	1	0.66	0.029
	2	0.66	0.0075

Several observations can be made about the push-off tests. First, there is very little difference in the coefficient of friction between the tests with the polyethylene sheeting and without the polyethylene sheeting. This is the result of a smooth concrete surface (precast panel) resting on a smooth asphalt surface. While there is still friction between the precast panel and asphalt leveling course, its effect is greatly reduced by the smoothness of the bottom of the precast panel and the stiffness (moderate ambient temperatures) of the leveling course. Had the test been conducted under very warm ambient temperatures, the asphalt would probably have conformed to the bottom surface of the panel and greatly increased the frictional resistance. The frictional interaction between the precast panels and asphalt leveling course is also time dependent. Over time, the asphalt leveling course will begin to conform to the bottom surface of the panels, greatly increasing surface contact and frictional resistance.

A second observation is that the movement at sliding is reduced for the secondary push-off test compared to the initial push-off for both conditions. This is most likely caused by the adhesion between the panel or polyethylene sheeting and the asphalt leveling course being broken during the initial test, allowing the panel to slide easier during the secondary test. The reduction for the polyethylene sheeting (30%) is significantly less that the reduction for the non-polyethylene sheeting condition (74%), indicating less of an adhesion effect with the polyethylene sheeting. This is most likely also the reason for the significant reduction in coefficient of friction between the initial and secondary tests for the polyethylene sheeting condition.

A final observation is that the maximum coefficient of friction for the test with the polyethylene sheeting is significantly higher than that assumed during the design phase, while the movement at sliding is the same as that assumed. Values of 0.2 for the coefficient of friction and 0.02 in. for the movement at sliding were assumed during the design of the pavement based upon previous experience with cast-in-place prestressed pavements. A higher coefficient of friction will affect the stresses in the pavement, increasing tensile stresses during slab contraction, but should not significantly affect the design life of the Georgetown precast pavement due to the conservative nature of the original design. The calculated value from the push-off tests (0.63) should be used as a baseline for the design of future precast prestressed concrete pavements.

9.4 Final Profile

Prior to opening the pavement to public traffic, a high-speed inertial profilometer was used to evaluate the ride quality of the pavement. One pass was made in each of the two lanes over the length of the pavement. The FHWA computer program "ProVAL" developed by The Transtec Group, Inc., of Austin, Texas, was used to analyze the profile data. The data was initially filtered to eliminate the bridge and bridge approach slabs from the profile. The data was then further filtered to remove spikes in the profile data at the expansion joints.

Table 9.2 summarizes the International Roughness Index (IRI) values from the profile analysis. The data is subdivided into the individual wheel paths for each lane for both the partial-width and full-width panels. The average IRI was 165.5 in./mile for the partial-width panels and 147.1 in./mile for the full-width panels. The difference in IRI between the full-width and partial-width panels is most likely the result of the geometry of the frontage road. The majority of the partial-width panels were placed on a vertical curve, which may have caused irregularities in the final profile.

The TxDOT ride quality specification for new or reconstructed rigid pavements normally imposes a penalty pay adjustment for IRI values greater than 75 in./mile and requires corrective measures for IRI values greater than 95 in./mile. Although the values from the Georgetown precast pavement are significantly higher than these, a qualitative evaluation of the ride quality by TxDOT concluded that the pavement did not warrant diamond grinding or any other corrective measure.

Table 9.2 *IRI Values for the final profile of the finished pavement*

	Partial-Width Panels				**Full-Width Panels**			
	Inside Lane		***Outside Lane***		***Inside Lane***		***Outside Lane***	
	LWP	**RWP**	**LWP**	**RWP**	**LWP**	**RWP**	**LWP**	**RWP**
IRI (in/mi)	170.0	168.9	163.6	159.6	158.6	152.5	135.5	141.7

LWP = *Left Wheel Path,* **RWP** = *Right Wheel Path*

No irregularities, which could lead to dynamic loading and premature pavement failure, were observed in any of the panels. Although a ride quality specification was considered prior to construction, a hard aggregate was used in the panels and may have made diamond grinding cost-prohibitive. For future applications, a ride quality specification should be established prior to construction that will determine whether the finished pavement is smooth enough for immediate traffic use. Smoothness incentives and penalties will help to ensure a high-quality finished product from the contractor.

9.5 Deflection Measurements

Deflection measurements are commonly collected on concrete pavements to determine layer properties of the pavement structure, load transfer across joints or cracks, and to determine if voids are present beneath the pavement. The most common tool for collecting deflection data in Texas is the Falling Weight Deflectometer (FWD). The FWD was used to collect deflection measurement on the Georgetown precast pavement, prior to opening to traffic, in order to evaluate:

- Properties of pavement support structure (AC leveling course, embankment, subgrade)
- Load transfer across expansion joints, intermediate panel joints, and longitudinal joints (partial-width panels)
- Presence of voids beneath the pavement

For all deflection measurements, seven sensors were used on the FWD, with Sensor 1 at the load and the other six spaced at 1 ft apart moving away from the load (see Figure 9.3). Measurements were taken on both the asphalt leveling course prior to panel placement, and after all of the panels were in place. Deflections were collected for four different load values at each measurement location. Although the applied load is not exactly the same each time, the deflection measurements were normalized for loads of: 5,250 lb, 8,500 lb, 11,750 lb, and 15,500 lb. The 15,500 lb load was primarily used for the analysis.

9.5.1 Expansion Joints

Load transfer across the expansion joints was evaluated by taking two sets of deflection measurements, one each side of the joint, as shown in Figure 9.3. Measurements were taken on all eleven expansion joints along the outside wheel load path. Additional measurements were taken along the inside wheel load path of the inside lane of the partial-width panels for comparison of behavior between the 16 ft and 20 ft panels.

Load transfer was evaluated by examining the difference in the ratio of Sensor 2 deflection to Sensor 1 deflection. For Drop 1, Sensor 1 was located at the load on one side of the expansion joint, while Sensor 2 was located on the opposite side of the expansion joint. For Drop 2, both Sensor 1 and Sensor 2 are located on the same side of the expansion joint, as shown in Figure 9.3. The deflection bowl created by Drop 2 is indicative of perfect load transfer because all of the sensors are on the same slab, while the deflection bowl for Drop 1 indicates the degree of load transfer across the joint. Comparison of the deflection bowl from Drop 1 to Drop 2, specifically looking at Sensors 1 and 2, allows one to quantify the load transfer effectiveness of the expansion joint. Equation 9-1 is used to compute the load transfer efficiency as follows:

$$LTE = \frac{\delta_{1-2} / \delta_{1-1}}{\delta_{2-2} / \delta_{2-1}} \times 100 \tag{9-1}$$

where: LTE = Load Transfer Efficiency
$\delta_{i\text{-}j}$ = Deflection for Drop i at Sensor j

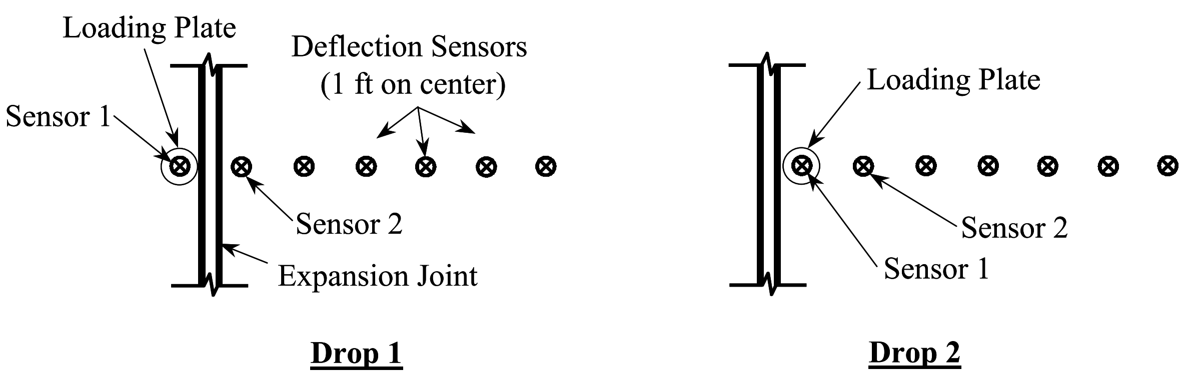


Figure 9.3 *Diagram of FWD deflection measurement at the expansion joints*

Figure 9.4 shows the deflection bowls from Expansion Joint 6 for Drops 1 and 2. The substantial decrease in deflection between Sensors 1 and 2 for Drop 1 indicates poor load transfer (44.2%) across the joint. This change in deflection was compared to the change in deflection for Drop 2 using Equation 9-1 to determine load transfer efficiency. Figure 9.5 shows the deflection bowls for Expansion Joint 2. Although the actual deflections are somewhat different between Drops 1 and 2, the change in deflection between Sensors 1 and 2 is relatively the same for both drops, indicating excellent load transfer (97.2%).

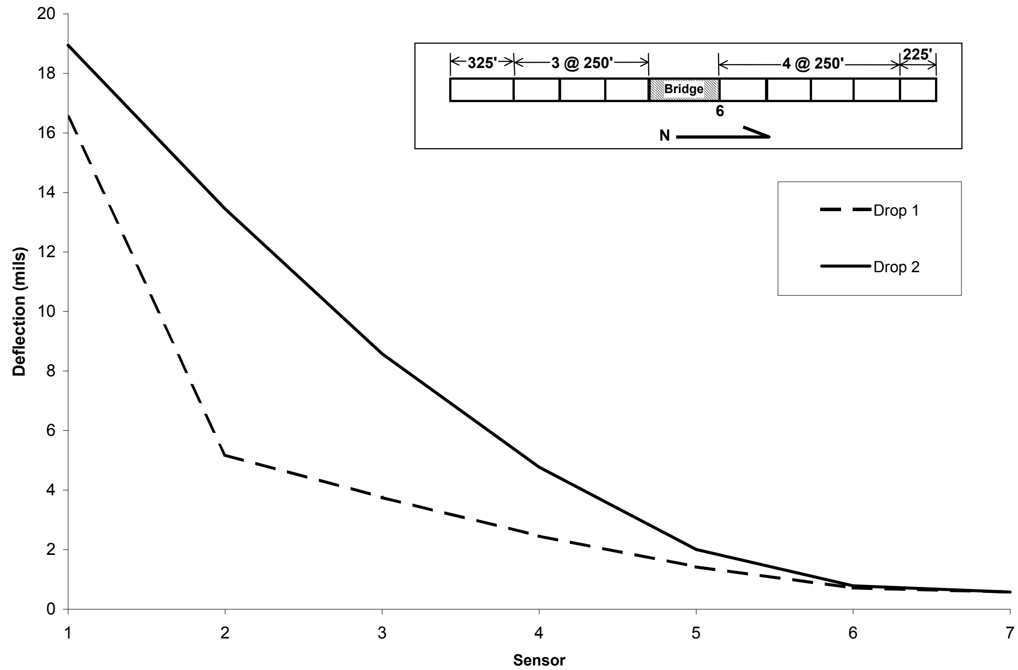


Figure 9.4 *Deflection measurements for Expansion Joint 6 showing poor load transfer*

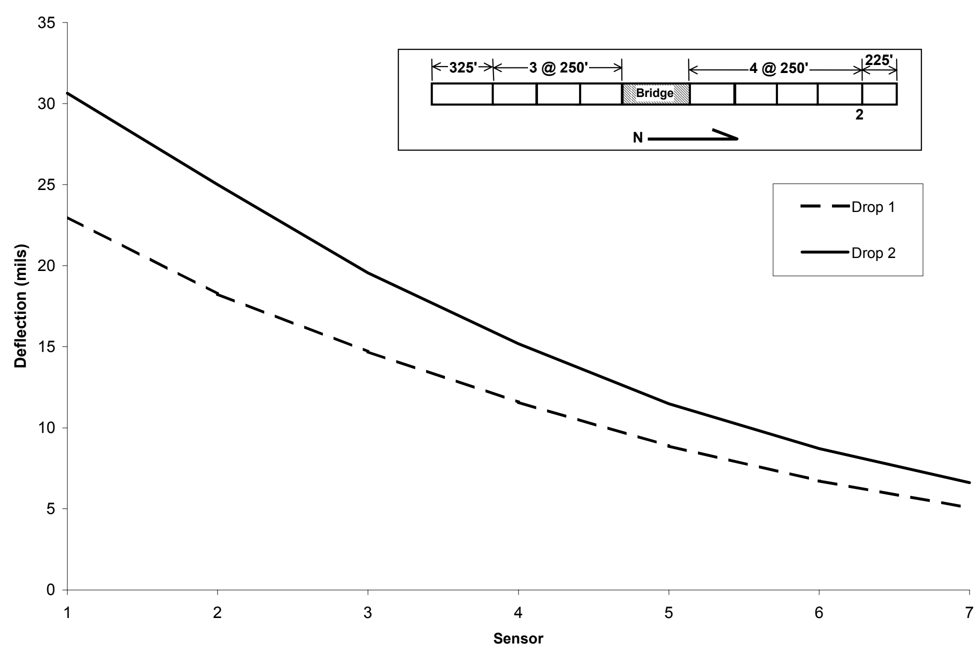


Figure 9.5 *Deflection measurements for Expansion Joint 2 showing excellent load transfer*

Table 9.3 summarizes the load transfer efficiency calculated from Equation 9-1 for each of the expansion joints. The table also indicates whether there is the possibility of a void beneath the expansion joint. The presence of voids was determined from the deflection measurements, with deflections greater than 15 mils indicating possible voids.

As Table 9.3 shows, only six joints showed load transfer efficiency of 50% or greater, with only two joints indicating nearly perfect load transfer. Four of the joints (three full-width, one partial-width) indicate that the FWD measurement was taken over a void, as determined by deflections greater than 15 mils, although it was not possible to determine the actual presence and extent of any voids.

The main cause of poor load transfer across many of the expansion joints is believed to be the dowel bar expansion sleeves cast into one side of the joint (see Figure 6.2). The different diameters of the dowel and expansion sleeve create a space around the dowel which allows for some amount of vertical movement of the dowel when a load is applied to the joint. The dowel must be in contact with the expansion sleeve before load is transferred across the joint through the dowel. Although all measured deflections were less than 31 mils, the space between the dowel and expansion sleeve could result in deflections up to 62.5 mils as there is a 1/8 in. difference between the outside diameter of the dowel and inside diameter of the sleeve.

Although the expansion joint detail used for the Georgetown precast pavement has shown excellent performance after 17 years in the West, Texas, cast-in-place prestressed pavement, mentioned previously, the detail should be modified for future projects to eliminate the space around the dowel bar. It should be noted, however, that the Georgetown precast pavement has yet to show any signs of distress from this detail.

Table 9.3 *Summary of calculated load transfer efficiency (LTE) and indication of possible voids for each expansion joint*

Expansion Joint	LTE		Possible Void
1		92.6 %	N
2		97.2 %	Y
3		55.7 %	N
4		47.1 %	Y
5		54.8 %	N
6		44.2 %	Y
7	**OL**	34 %	N
	IL	15.6 %	N
8	**OL**	45.8 %	N
	IL	24.8 %	N
9	***OL***	33.9 %	N
	IL	76 %	N
10	***OL***	31.7 %	Y
	IL	57.7 %	N
11	***OL***	27.1 %	N
	IL	36.6 %	N

OL = ***Outside Lane,*** *IL* = ***Inside Lane***

9.5.2 Longitudinal Joints

A similar procedure to that described above was used to evaluate load transfer efficiency for the longitudinal joint between the partial-width panels. Similar to that shown in Figure 9.3, deflection measurements were collected for drops on either side of the longitudinal joint. Measurements were collected at random locations along the longitudinal joint with two measurements from Slab 9, three measurements from Slab 8, and one measurement from Slab 7.

The load transfer efficiency for the longitudinal joints proved to be very good. Load transfer efficiency ranged from 86% to 95% with an average of 91.7% (coefficient of variation = 3.9%). This indicates that the combination of post-tensioning and epoxy along the longitudinal joint provides excellent load transfer.

Additionally, all deflection measurements, with the exception of one, were in the range of 3–7 mils, indicating voids are most likely not present beneath the majority of the longitudinal joint. The outlying measurement, taken on Slab 8, indicated deflections of 13–14 mils. Based upon the other measurements, this may indicate that a measurement was taken over a void, but it is difficult to determine the actual presence and extent of any void.

9.5.3 Intermediate Panel Joints

For the intermediate panel joints, only one deflection measurement was taken at each joint. Sensor 1, located at the loading plate was on one side of the joint, and sensors 2–7 were located on the other side of the joint, similar to that shown in Figure 9.3 for Drop 1. At least two intermediate panel joints were tested on each slab, with all of the joints tested for Slab 3. The deflection measurements were taken along the outside wheel path of the outside lane, approximately 2 ft from the shoulder stripe. Additional deflection data was also collected along the inside wheel path of the inside lane on each of the partial-width slabs.

Figure 9.6 shows the deflection bowls generated from the FWD measurements on the intermediate panel joints. A visual inspection of the data indicates excellent load transfer across all of the intermediate panel joints. The decrease in deflection from Sensor 1 to Sensor 2 is similar to that measured on a monolithic slab.

The FWD measurements also indicate, however, the possible presence of voids. The mean deflection at Sensor 1 for all of the measurements was 5.7 mils, with a standard deviation of 2.92. Considering a range of two standard deviations, deflections outside of this range (above 11.6 mils) most likely indicate the presence of voids. From the FWD data (Figure 9.6), voids may have been present beneath three of the locations measured. Two of these locations occurred on Slab 9 and one on Slab 1. The extent of these voids is difficult to determine, but should be evaluated and monitored over the life of the pavement.

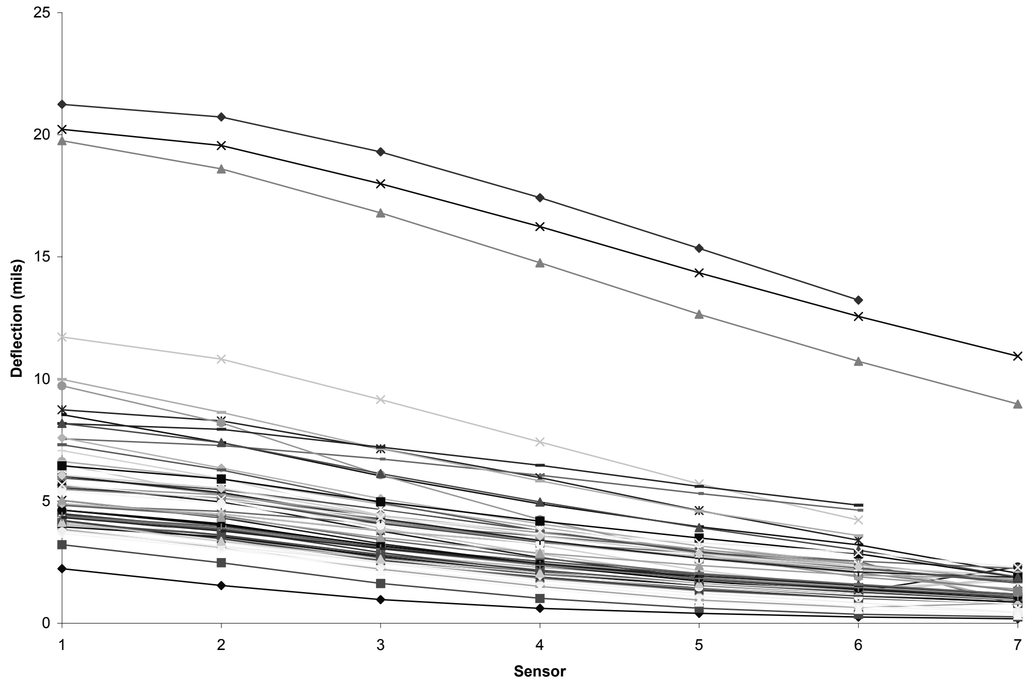


Figure 9.6 *Deflection bowls for intermediate panels joints*

9.5.4 Properties of Support Structure/Presence of Voids

In addition to the deflection measurements collected at the various joints, deflection data was also collected at selected locations in the middle of a panel, away from any joints. This was used to determine what typical mid-panel deflections should be expected and evaluate the possibility of voids beneath the pavement.

The first step in this analysis was to evaluate the properties of the pavement support structure using the FWD measurements collected over the asphalt leveling course. The properties of the different support layers were determined from back-calculation using the FWD data and known characteristics, such as layer thickness. With the properties of the support structure, the next step was to calculate expected deflections for the complete pavement structure using layered theory analysis. The final step was to compare the expected deflections to the measured deflections (on the precast pavement) to evaluate the possible presence of voids.

FWD deflection data was collected approximately every 50 ft over the length of the asphalt leveling course at the centerline of the frontage road. Using this data and the computer program MODULUS version 5.1 (Ref 14), the properties of the pavement support structure were back-calculated. Figure 9.7 shows the pavement support structure determined from the analysis. The thickness of the embankment material was varied for each slab based on the construction drawings. This support structure was used for predicting deflections with the precast pavement in place. The computer program BISAR (Ref 15) was used to predict the FWD deflections using layered theory analysis for the varying loads mentioned previously.

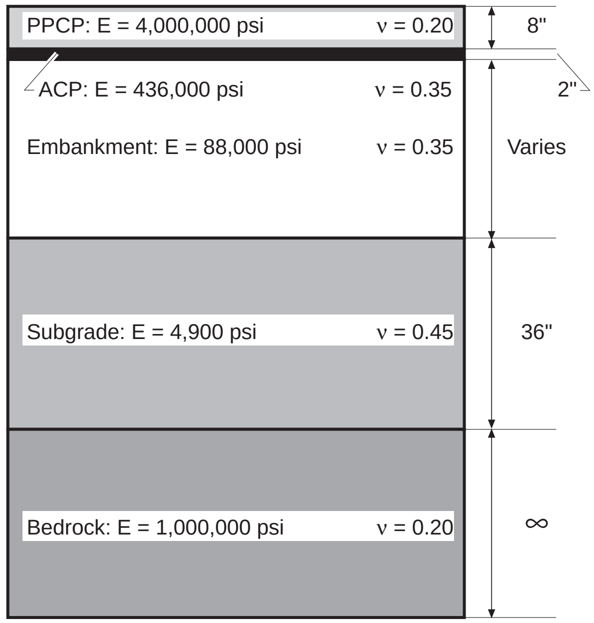


Figure 9.7 *Pavement support structure as determined from FWD measurements and back-calculation*

The deflection bowls predicted by layered theory analysis for each of the nine slabs under the heaviest load (15,500 lb) are shown in Figure 9.8. Deflections varied based upon the varying thickness of the embankment layer. As Figure 9.8 shows, the range of deflections at the load (Sensor 1) range from approximately 3.5–8 mils. Although these predictions are not exact, due to minor variations in the actual modulus and layer thickness, they serve as a baseline for comparison with the measured values.

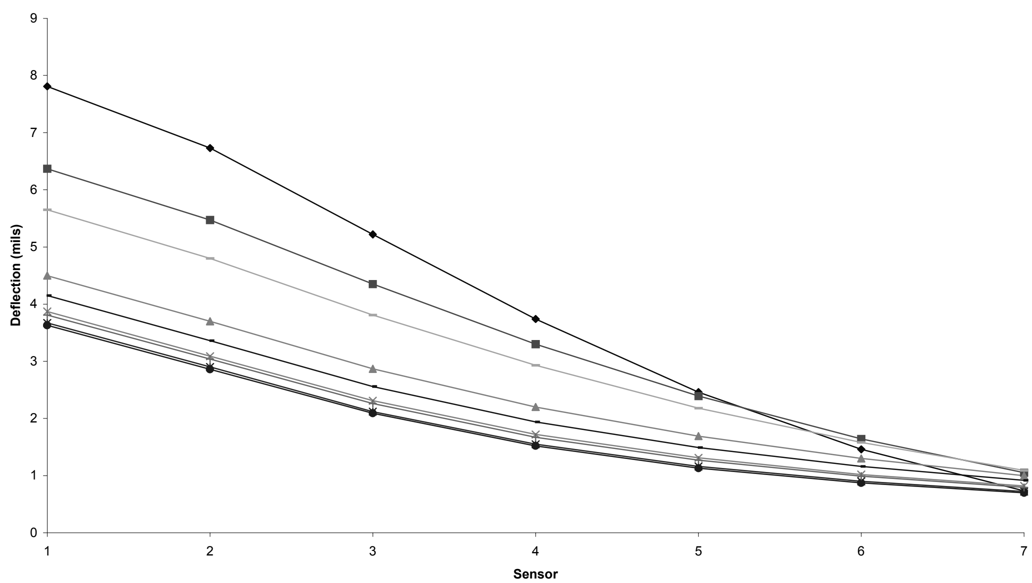


Figure 9.8 *Deflection bowls predicted from the layered theory analysis for each of the nine slabs.*

Figure 9.9 shows the deflection bowls from the FWD measurements on the actual precast pavement under the heaviest load (15,500 lb). The measurements were taken in the middle of various precast panels, away from any joints. Similarly to that predicted by elastic layered theory analysis, the majority of Sensor 1 deflections are in the range of 3–8 mils. There were several measurements, however, outside of this range, indicating the possible presence of voids.

To confirm the typical deflection range for an 8 in. concrete pavement, data collected from an 8 in. continuously reinforced concrete pavement (CRCP) was examined. Figure 9.10 shows the deflection bowls from FWD measurements on an 8 in. concrete pavement placed over asphalt on I-30 near Ft. Worth, Texas. This deflection data, likewise, shows a typical range of deflections of 3–8 mils. Because this was a relatively new cast-in-place pavement, and because the deflection data is fairly homogeneous, voids are most likely not present beneath this pavement.

Based on this analysis, it can be concluded that deflection data outside of the 3–8 mil range probably indicates the presence of a void at the location the FWD data was collected. Examination of the data from the Georgetown precast pavement (Figure 9.9) reveals six (out of twenty-one) locations where there is most likely a void present. These six locations occurred on slabs 1, 4, 6, 7 (two locations), and 9. Unfortunately, it is difficult to determine the actual presence and the extent of these voids.

Any voids that are present beneath the Georgetown precast pavement are believed to be shallow and small in size, caused primarily by minor variances in the asphalt leveling course. The integrity of the leveling course should prevent these voids from becoming any larger. Additionally, observations made during construction revealed that most shallow (1/32–1/16 in.) voids observed immediately after placement of a panel disappeared within 24 hours due to the weight of the panel resting on the leveling course. Even if permanent voids do exist, the prestress in the pavement will help it to span these voids, preventing any loss of life of the pavement.

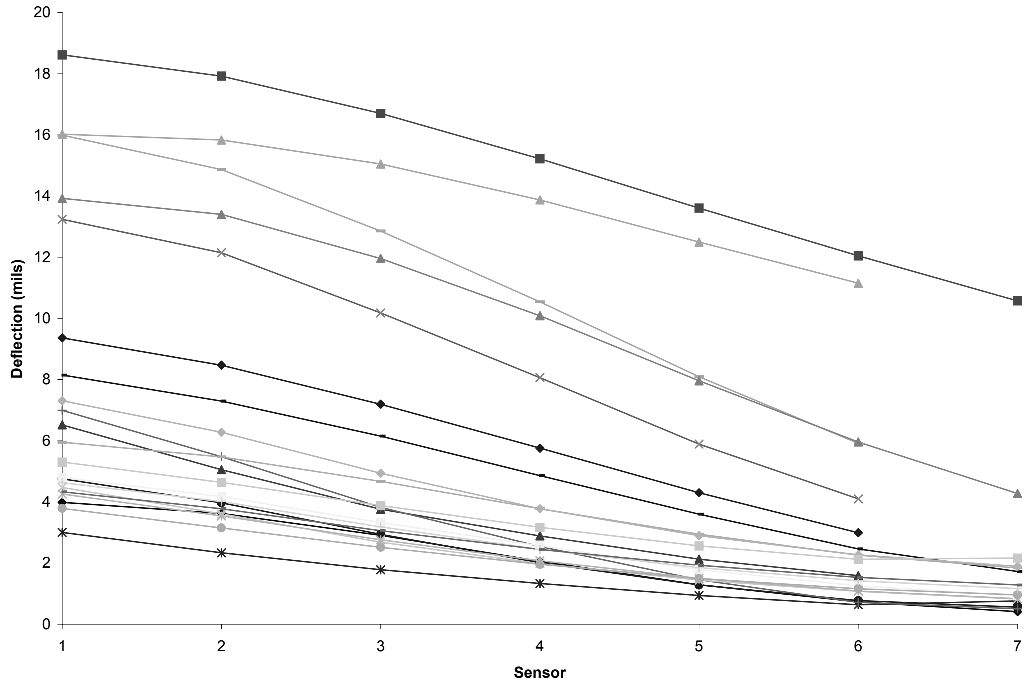


Figure 9.9 *Deflection bowls from mid-panel FWD measurements*

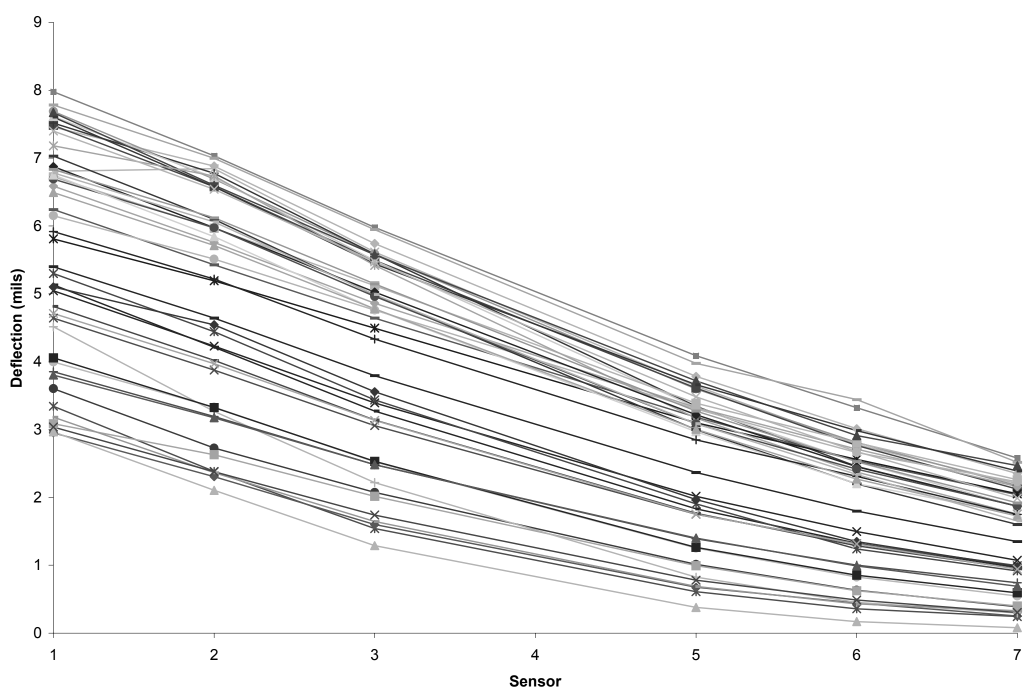


Figure 9.10 *Deflection bowls from FWD measurements on an 8" CRCP in Ft. Worth, Texas*

10. Recommendations for Future Construction

10.1 Introduction

The Georgetown precast pavement pilot project provided a great deal of information for construction of future precast prestressed pavements. Although an excellent product resulted, many challenges were encountered throughout the design process, panel fabrication, and construction. This chapter will present recommendations for future construction based on what was learned from the design and construction of the Georgetown pilot project.

10.2 Design Issues

Perhaps the greatest challenge encountered in the design process was the lack of standardized design procedures for precast concrete pavement. Although CTR has experience with designing cast-in-place prestressed concrete pavements, precast presented some unique challenges.

The use of a computer model, such as PSCP2, is highly recommended for designing precast prestressed pavements. PSCP2 has been found to accurately predict the behavior of prestressed concrete pavements, and has been calibrated with data collected from actual pavements. However, a modified version of PSCP2 is needed for precast pavement. This modified version should take into account the different material properties and behavior of a precast pavement.

In addition to the use of a computer model for design variables of precast pavement, a better understanding and characterization of all of the design variables for precast pavement is needed. This includes better estimation of the slab-base frictional interaction, realistic prediction of climatic conditions the pavement will experience, and better prediction of the stresses the pavement will experience over its design life. The latter two variables are inherently related as the climatic conditions greatly influence the stresses in the pavement. Using worst-case variables, as was done with the Georgetown precast pavement, results in a conservative design, increasing the cost of the pavement.

10.3 Panel Fabrication

Several challenges were encountered during panel fabrication, including both panel details and casting procedures. Several other recommendations for panel fabrication, based on experiences from the Georgetown precast pavement, are discussed below.

10.3.1 Panel Details

The majority of the panel details for the Georgetown precast pavement, including panel dimensions, reinforcement, and stressing pocket configuration, proved to be viable for future precast pavements. However, there are some details which should be improved for future projects.

Armored Joints – The primary cause of delays in casting was the armored expansion joints. As described in Chapter 6, the armored joints from the steel fabricator were severely warped and bowed. For future projects this warping must be considered and corrected, if needed, by the steel fabricator who supplies the armored joint. Alternatively, a non-armored expansion joint detail could be used as long as the joint movement is limited accordingly. Maximum joint movement for a non-armored expansion joint should be limited to 0.5 in. This will require limiting the length of post-tensioned slabs or using a concrete mix with a very low coefficient of thermal expansion.

Another recommendation with regard to armored joints is elimination of the expansion sleeves around the dowels. As discussed in Chapter 9, the expansion sleeves caused a loss in load transfer across the expansion joints due to the difference in diameter of the dowel and expansion sleeve. For future applications, an expansion cap on each end of a greased dowel is recommended to replace the expansion sleeves.

Keyways – The keyways along the edges of the panels proved to be invaluable during panel assembly on site. The keyways ensured vertical alignment between panels, greatly increasing the placement rate. During assembly, however, some joints between panels did not close completely because of wedging action along the nose of the male keyway. Although this was a very minor occurrence, the keyway dimensions should be evaluated for future projects to allow for a looser fit of the keyways. The keyway dimensions should allow for the top and bottom surfaces to make full contact, as shown in Figure 10.1. Additionally, a slight chamfer should be included on the bottom of the keyways (Figure 10.1). This chamfer will allow for easier removal of the panels from the forms and prevent corner breaks such as those encountered with the panels for the Georgetown pavement.

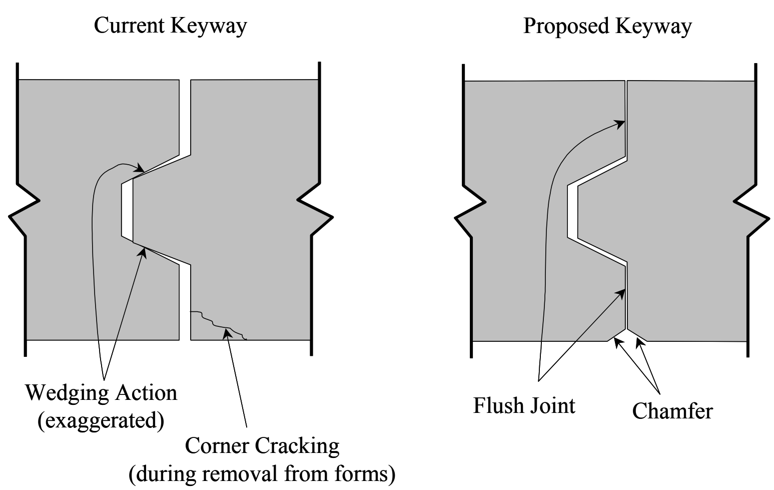


Figure 10.1 *Keyway detail for the Georgetown PPCP and proposed keyway for future projects*

Lifting Devices – The lifting devices used for the Georgetown precast pavement panels proved effective for rapid lifting and handling of the panels. However, the recesses left by the lifting devices required a significant amount of time to prepare and patch. For future projects a lifting device that only requires minimal patching, and will allow the pavement to be opened to traffic prior to patching, is recommended. Several types of lifting devices, such as screw-type devices, are currently available and should be considered for future applications. It is important, however, that the lifting lines can be attached and removed from the panels as quickly as possible so that panel placement is not slowed.

Tolerances – The precast contractor was able to maintain the required tolerances for all of the panels. However, as discussed in Chapter 7, it was discovered that a tighter tolerance was needed for the squareness of the partial-width panels. During placement of the partial-width panels, several corner cracks occurred from stress concentrations at the longitudinal joint between the 16 ft and 20 ft panels when post-tensioned transversely. On future projects a squareness tolerance of no more than 1/8 in. should be specified for panels abutting adjacent panels.

10.3.2 Casting Procedures

No major problems were experienced with casting the panels for the Georgetown precast pavement. However, there were some areas which could be improved on future projects.

Mix Design – The mix design for the Georgetown precast pavement proved to be a viable mix for meeting the needs of the precast contractor. The mix was workable enough that a carpet drag finish could be applied while also achieving the necessary release strength within 24 hours of casting so the panels could be removed. One undesirable effect of this mix, however, was the high heat of hydration, which caused the concrete temperature to exceed 160 °F. This resulted in mid-panel cracking in the 36 ft long panels, as discussed in Chapter 6. The cost of the mix was also significantly higher than typical paving mixes due to the superplasticizer used to achieve the necessary workability. Future projects should consider the use of a less expensive workable mix in conjunction with alternative curing techniques, such as steam curing, to achieve the necessary release strength.

Casting Time – The majority of the panels for the Georgetown precast pavement were cast between the months of June and October. During these months temperatures are regularly in the mid to upper 90 °F range and can exceed 100 °F in Victoria, Texas, where the panels were cast. For this reason, the precast contractor was required to cast the panels early in the morning while the forms were still relatively cool. Although this was beneficial for preventing excessive water loss/evaporation during casting, the concrete mix generally reached its peak heat of hydration during the hottest part of the day. This resulted in very high temperatures in the panels and led to mid-slab cracking. Ideally, panels cast in hot temperatures using a concrete mix with such a high heat of hydration should be cast late in the afternoon as the ambient temperature is beginning to decrease. However, this requires the forms to be cooled before casting, which may not be feasible, depending on the size of the casting bed. For future projects the climatic conditions the panels will be cast under should be examined carefully. Timing of casting should be such that the ambient temperatures will not exasperate the concrete temperature by either adding to the heat of hydration or greatly slowing cure time due to colder temperatures.

Curing – The curing procedure for the Georgetown precast pavement panels was deemed to be sufficient and should be the minimum curing procedure for future projects. Curing consisted of both an intermediate curing compound applied to the panels during casting to minimize water loss, followed by two coats of standard concrete pavement curing compound immediately after the carpet drag finish was applied. After the panels were removed from the forms and stacked, each stack was covered with wet cotton mats and canvas tarps for an additional 24 hours. Although this was sufficient, the preferred method of curing would be to cover the panels with wet cotton mats or plastic sheeting for at least the first 24 hours, while in the forms, followed by wet cotton mats or plastic sheeting for an additional 24–48 hours after they are stacked. Alternatively, steam curing or fog curing would also be advantageous, but may require a special casting bed.

10.4 Pavement Construction

Assembly of the precast panels was very successful with only a few minor problems over the entire project. Minor adjustments on future projects, however, will prevent these problems from occurring again.

10.4.1 Base Preparation (Asphalt Leveling Course)

The asphalt leveling course proved to be a viable method for providing a smooth, flat surface for supporting the precast panels. The leveling course was placed quickly in one day, and did not require additional preparation prior to panel placement.

Tolerances – The leveling course for the Georgetown precast pavement was placed in three passes on either side of the bridge to achieve the full 36 ft roadway width. The critical issue with placing the leveling course in separate passes is the joint between the different placements. A low point, or sag, at the joint between placements will result in a void beneath the precast panels. This void will normally be small, and prestressed pavement panels should be able to span the void without affecting the performance of the pavement. A high point or ridge between placements, however, will cause a stress concentration in the precast panels. This was the case for many of the full-width panels of the Georgetown precast pavement. A single crack, located approximately at the joint between two sections of asphalt, was noticed in many of the full-width panels, as discussed in Chapter 9. Stricter tolerances on the smoothness of the leveling course will prevent this from occurring on future projects. A general recommendation for the asphalt leveling course tolerance, based on the Georgetown precast pavement is as follows:

Width of AC Leveling Course	Tolerance
≤ 10 ft	± 1/8”
> 10 ft (not to exceed 40 ft)	± 1/4”

Removal and Replacement Applications – The Georgetown precast pavement represented new pavement construction over a prepared roadbed. Many applications for precast pavement, however, will be removal and replacement operations, where the old pavement is removed and the new precast pavement is placed during overnight or weekend operations. Theses applications will most likely also consist of much shorter sections of pavement. This presents some issues when using an asphalt leveling course, such as the economic viability of placing a short section of asphalt leveling course, mobilization of the paving equipment into the cavity left by the old pavement, and viability of placing precast panels over a hot asphalt pavement. For these reasons, it may not be practical in all cases to use an asphalt leveling course. Some alternative solutions for base preparation include:

- Precision grading equipment
- Screeded gout bed
- Cold-mix asphalt concrete

Although none of these methods of base preparation were tested on the Georgetown precast pavement project, precision grading and screeded grout beds have been used successfully on other precast pavement projects.

10.4.2 Panel Placement

No major problems were experienced during placement of the precast panels. Many different techniques were tested, however, and provided useful information for future construction.

Joint Treatment – Sealing the joints between panels is particularly important for protecting the post-tensioning strands crossing the joints. Many different ideas were examined for the Georgetown precast pavement. However, the use of a segmental bridge epoxy appeared to me the most viable option. Epoxy not only seals the joints between panels, but also acts as a lubricant for the keyed joints during assembly. A slow setting (24-hour set) epoxy is recommended for precast pavement construction to ensure enough time to apply some amount of post-tensioning before the epoxy sets. Other alternatives, such as foam rubber or neoprene seals around the individual ducts will better help to protect the post-tensioning strands, but will not prevent water from penetrating the joints between the ducts. Ideally, a combination of epoxy and individual duct seals will provide the best protection for the post-tensioning strands while also sealing the joint and providing lubrication during assembly.

Temporary Post-Tensioning – The panels for the Georgetown precast pavement were pulled together as each was lowered into place using come-alongs linked between the edge sleeves in the ends of the panels. This proved to be an effective and efficient method for assembly of the full-width panels. For the partial-width panels, however, the ends of the panels at the longitudinal joint were not accessible. The solution was to use temporary post-tensioning to pull the panels together. Two 0.5 in. post-tensioning strands were fed into the ducts at the third points (one third of the panel length from each end) and used to pull the panels together. Between two and three panels were placed before the temporary post-tensioning was applied. Although temporary post-tensioning was not specified in the original design, it proved to be an efficient and effective method for closing the joints between panels as much as possible during panel placement. For future projects, temporary post-tensioning should be included in the construction plans with a provision for accommodating the stressing ram used for temporary post-tensioning. At least two temporary post-tensioning strands should be used, located between the third and quarter points.

Panel/Duct Alignment – As discussed in Chapter 7, many of the panels were offset slightly to compensate for deviation from the road centerline during panel placement. The deviation was caused primarily by variation in the joint width at either end of the panels. Offsetting the panels, however, caused misalignment of the longitudinal post-tensioning ducts, making strand insertion more difficult. During placement of the partial-width panels, however, gapping the joints at either end proved to be effective for correcting the centerline deviation while not compromising duct alignment. For future projects, gapping the joints, rather than offsetting the panels, is recommended to prevent any misalignment of the longitudinal ducts.

Mid-slab Anchor – The mid-slab anchor for the Georgetown precast pavement was achieved by drilling 1 1/2 in. diameter holes into the base material at the stressing pockets and then grouting 1 in. diameter deformed bars in the holes. Filling the stressing pockets tied the bars to the pavement, preventing the center of each slab from moving. This proved to be an efficient and effective solution for the mid-slab anchor, and is recommended for future projects. At least two bars should be anchored at each stressing pocket, and the bars should extend at least 12 in. into the leveling course and base.

Stressing/Access Pockets – Because the Georgetown precast pavement was closed to traffic during construction, the stressing pockets and access pockets were filled with normal strength/normal set concrete. For future applications, however, the use of fast-setting concrete or temporary covers may be required. If a fast-setting concrete is used, enough time should be permitted for adequate curing before the pavement is opened to traffic. If temporary covers are used, they should be tested to ensure they can withstand heavy truck traffic and will not come loose under traffic loading.

10.5 Post-Tensioning

Post-tensioning initially presented the most challenges to the precast pavement construction. Minor adjustments to the post-tensioning materials and procedures, however, should prevent these issues from occurring on future projects.

10.5.1 Strand Placement/Anchorage

Strand Placement – Inserting the strands into the ducts at the central stressing pockets and feeding them by hand to the anchors proved to be a simple process. With properly aligned ducts, minimal effort was required to feed the strands through the ducts. When ducts were slightly misaligned due to panel offsetting, however, feeding the strands took considerably more effort. In general, the more resistance to strand insertion that was encountered, the more likely one or more of the wires had slid back. This required a procedure for first inspecting the strands at the access pockets and cutting the end flush if needed. The strand was then pushed up to the grout vent in front of the anchor, marked in the stressing pockets (for measuring total movement), and then slowly vibrated into the anchor using a hammer-drill. The movement of the strand from the grout vent to the back of the anchor was then measured to determine whether the strand was fully inserted into the anchor. This procedure proved to be effective for determining whether each strand was fully anchored, and should be used on future projects where blind, self-locking post-tensioning anchors are used.

Anchorage – As discussed above and in Chapter 7, the use of blind, self-locking post-tensioning anchors required special procedures for ensuring the strands were fully anchored. For future projects, the researchers recommend that self-locking anchors not be used unless the back of the anchor is accessible to ensure the strand is intact and the wedges are properly seated around the strand. This can be accomplished by moving the access pockets in the joint panels behind the anchors rather than in front.

An alternative solution is the use of looped post-tensioning tendons, such as that shown in Figure 10.2. With a looped system, the strands are fed around a curved section of duct in the joint panel. The ends of the strands are then coupled in the central stressing pocket as before. This system will require tying the curved section of duct to the armored expansion joint, which can be accomplished using deformed bar stirrups welded to the armored joint which loop around the curved duct. The material for the curved duct should be smooth material, such as galvanized steel conduit, which will not inhibit the movement of the strand, and should be a larger diameter than the normal longitudinal duct. A grout vent at the tip of the curved section of duct will ensure the duct is fully grouted. Depending upon the spacing of the longitudinal tendons, it may be necessary to overlap the curved sections to accommodate the minimum bending radius of the strand. This solution would not only eliminate the dead-end anchorage in the joint panels, but would also eliminate the access pockets. It is critical, however, to ensure the tendons are tied to the armored joints.

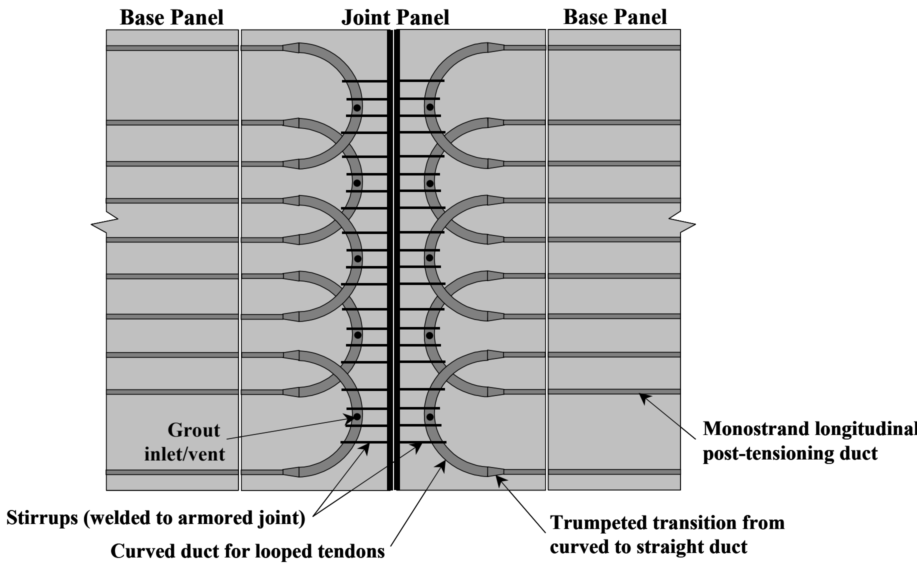


Figure 10.2 *Proposed looped tendon anchorage at the joint panels (Note: dowels, keyways, pretensioning strands, and mild reinforcement are not shown for simplification)*

10.5.2 Transverse Post-tensioning

Anchorage – Transverse post-tensioning did not present any difficulties for the Georgetown precast pavement. The use of a flat multi-strand duct allowed for slight misalignment of the partial-width panels and any differential movement that occurred prior to post-tensioning. The anchorage, however, at the outside edges of the pavement will need to be modified for a future removal and replacement applications, as access to the edges of the slabs may not be available. One alternative for the transverse post-tensioning anchorage would be to provide access pockets behind each anchor at the outside edges of the slab. Another alternative would be to use a looped tendon system, similar to that described above with a single stressing pocket.

Alternatives to Transverse Post-tensioning – As discussed previously, the purpose of transverse post-tensioning is to tie adjacent partial-width slabs together. This ensures load transfer across the longitudinal joint and ensures adjacent slabs will expand and contract together. Because the panels are pretensioned, transverse post-tensioning does not provide a significant structural benefit. Therefore, any alternative method for tying adjacent slabs together can also be used to achieve the same effect. The most important consideration is ensuring adequate load transfer across the longitudinal joint. Additionally, if different slab lengths are used for adjacent slabs or the expansion joints do not match, the transverse tie-in detail must allow for differential movement of the slabs.

One technique commonly used for conventional concrete pavements that could be used for this application is crack stitching. Crack stitching involves drilling holes at an angle across the longitudinal joint between adjacent slabs, as shown in Figure 10.3. A dowel or reinforcing bar is then inserted into the shaft and grouted in place. The stitches should be located approximately every 1 to 2 ft along the length of the longitudinal joint.

This technique has proven to be effective for restoring load transfer across cracks in conventional concrete pavement, and should be effective for the longitudinal joints between adjacent precast pavement slabs.

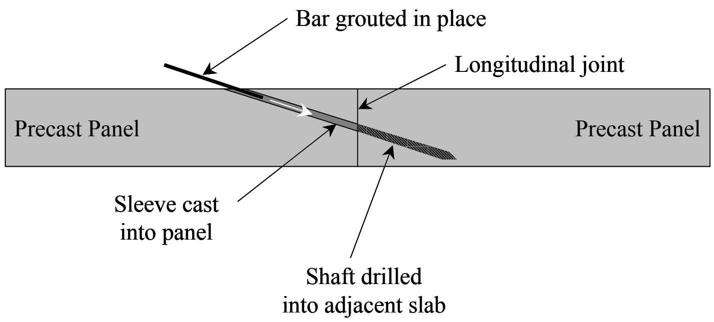


Figure 10.3 *Alternative transverse tie-in detail between adjacent precast slabs*

10.6 Grouting

Although grouting is an additional operation in the construction process, it provides several benefits to the post-tensioning system. One benefit is that it provides an additional layer of corrosion protection for the post-tensioning strands, particularly where they cross the joints between panels. Another major benefit is that it bonds the post-tensioning strands to the pavement. This will allow for the removal of damaged precast panels over the life of the pavement without having to de-tension the post-tensioning strands. Consequently, grouting is highly recommended for future precast, post-tensioned concrete pavements.

Grouting the longitudinal post-tensioning strands proved to be a challenging aspect of the Georgetown precast pavement. As discussed in Chapter 7, problems were experienced with grout leakage from the joints onto the top of the pavement and into other ducts. The following recommendations should help to prevent these problems from occurring in the future.

10.6.1 Panel Details

Ducts – The longitudinal post-tensioning ducts used for the Georgetown precast pavement are normally used for the grout vents of large diameter post-tensioning ducts. The main problem with this duct material was getting the grout into the ducts. In several instances, the post-tensioning strands were pressing against the top of the duct after they were tensioned, blocking the opening in the duct at the grout inlets/vents. Since completion of the Georgetown pilot project, alternative monostrand ducts have been developed to address this problem. Currently polyethylene monostrand ducts are available which contain grout channels along the length of the duct. These channels provide a route for the grout to flow when the duct is partially blocked by the strand or other material.

Joint Seal – The main problem with grouting was the lack of a tight seal around each duct between the panels. This resulted in grout leakage onto the top of the pavement, out the ends of the joints, and between ducts. For future applications, individual seals should be provided around each post-tensioning duct. This seal can be a foam rubber, neoprene, or other deformable material that will compress when the panels are post-tensioned together. This joint seal, combined with epoxy along the keyways will greatly minimize grout leakage at the panel joints.

10.6.2 Procedures/Materials

Grout – The grout material used should be suitable for post-tensioning tendon applications. A thixotropic grout, which is less susceptible to segregation when pumped, should be used. The grout mix should have low bleed and low shrinkage characteristics and should meet the necessary strength and fluidity requirements. The fluidity of the grout should be checked regularly during the grouting operation to ensure it has the ability to flow the necessary length between grout inlets/vents without segregating.

Procedures – Grout vents should be located just in front of the post-tensioning anchorage and next to the stressing pockets at minimum. Additional intermediate vents should also be located in the base panels at the quarter and half points between the end vents. Intermediate vents will provide additional grout inlets if the end inlets become blocked or if grout is leaking excessively near the end inlets. The movement of the grout should be carefully traced and recorded to determine whether each duct is fully grouted and where leaks are occurring.

11. Conclusions and Recommendations

11.1 Summary

The Georgetown precast, prestressed concrete pavement pilot project demonstrated the successful implementation of the precast pavement concept proposed by the feasibility study by Merritt, McCullough, Burns, and Schindler (Ref 2). The concept incorporated the use of pretensioning and post-tensioning to improve the durability of the pavement while greatly reducing the required pavement thickness.

The main objective of this implementation study was to test and refine the precast pavement concept developed from the feasibility study. Through small-scale laboratory testing and construction of the full-scale pilot project near Georgetown, Texas, the researchers were able to accomplish this objective. The successful completion of this implementation study will hopefully lead to the use of precast pavement on urban applications where expedited construction is needed most.

11.2 Conclusions

Precast concrete has proven to be a viable alternative for expediting construction of portland cement concrete pavements. Approximately 2,310 linear ft (0.9 lane-miles) of precast, prestressed concrete pavement was installed on this first pilot project near Georgetown, Texas. The construction of this pavement demonstrated not only the advantages of precast pavement over conventional pavement, but also the viability of several specific aspects of this particular precast pavement concept including:

- **Full-depth precast panels** – Full-depth precast panels were successfully installed and did not require additional measures, such as an asphalt concrete overlay or diamond grinding, to improve the final ride quality.
- **Keyways** – Keyways were incorporated into the edges of the panels to ensure vertical alignment between adjacent panels. The keyways greatly increased the panel placement rate by eliminating the need to level-up adjacent panels through some other means.
- **Asphalt leveling course** – The asphalt leveling course provided an adequately flat and smooth surface for supporting the precast panels. The leveling course was placed quickly and was opened to local traffic prior to panel installation.
- **Pretensioning** – Pretensioning was used successfully to provide transverse prestress, which previous experience has found to be essential for the long-term performance of prestressed pavements. Pretensioning also compensated for handling stresses in the panels, allowing longer panels to be used.
- **Post-tensioning** – Post-tensioning was successfully incorporated in the pavement to provide longitudinal prestress. Post-tensioning also served to tie all of the panels (between expansion joints) together so they would act as a continuous slab. Additional transverse post-tensioning was used to tie the partial-width slabs together and provide load transfer across the longitudinal joint. The combination of pretensioning and post-tensioning greatly reduced

the required slab thickness, and should significantly benefit the long-term durability of the pavement.

- **Polyethylene sheeting** – A single layer of polyethylene sheeting was successfully incorporated as a friction reducing membrane between the bottom of the pavement and the asphalt leveling course. The polyethylene sheeting was not only effective in reducing the frictional resistance, but efficient and economical for construction.
- **Grouting** – Grouting the post-tensioning tendons, although a challenging process, proved to be viable for precast pavement. Grouting not only provides an additional layer of corrosion protection for the post-tensioning strands, but also bonds the strands to the panels. This will allow for the removal and replacement of individual precast panels over the life of the pavement without the need to de-tension the post-tensioning strands.
- **Placement Rate** – The Georgetown pilot project revealed that a reasonable panel placement rate, not including placement of the leveling course, is approximately 25 panels in 6 hours. The length of the section will depend on the size (length and width) of the precast panels. For the Georgetown project, 25 panels represented 250 linear ft or 500 lane-ft for the full-width panels. Post-tensioning is an additional process, but does not need to be completed at the same time as panel installation.
- **Costs** – The construction costs for precast concrete pavement are significantly higher than conventional pavement at present. The final cost of the Georgetown precast pavement, including placement of the leveling course, was approximately \$160/SY. Conventional concrete pavement can be placed for as low as \$40/SY, depending on the size of the job. This higher construction cost is due primarily to the experimental nature of this project and unfamiliarity with this new technology on the part of the contractors. Construction cost will decrease over time as precast pavement becomes more widely used and standardized construction practices are developed. The biggest economic benefit of precast pavement will be realized through savings in user costs. Overnight or weekend construction using precast panels will always result in substantially lower user costs over conventional pavement construction.

11.3 Recommendations for Future Implementation

Although several challenges were encountered along the way, the Georgetown precast pavement pilot project provided a great deal of useful information for TxDOT, FHWA, and the researchers. This first pilot project allowed the researchers to evaluate the original precast pavement concept and further refine many of the details for future implementation.

Following the staged implementation strategy proposed in the initial feasibility study (Ref 2), this project serves as part of the first stage of implementation: laboratory testing and pilot projects. An additional pilot project, developed for the California Department of Transportation (Caltrans), will provide further information on additional refinements to the precast pavement concept. Although pilot projects are costly, they provide a great deal of information for future projects.

The next stage in precast pavement implementation is a pavement constructed under strict time constraints on a low profile project. This stage will require construction to take place during overnight or weekend operations to minimize effects on traffic. However, the pavement should be constructed on a low profile project, which will not have a severe impact on the motoring public if problems or delays are encountered and construction cannot be completed overnight or over a weekend. A project of this nature may be constructed on a rural highway pavement or intersection with a significant, but not excessive, amount of traffic.

The final stage of precast pavement implementation will be an urban application with high-traffic volumes and the strictest constraints on lane closures. This project may be constructed with incentives or penalties for any construction outside of a prescribed time frame. This application will serve as the ultimate test of precast pavement technology. Not only will the construction process need to be fully refined, but the durability of the pavement must be adequate for the traffic the pavement will experience. A typical project for this application would be an urban freeway or intersection.

The Georgetown precast pavement pilot project is only the beginning for precast pavement construction. Through the design, fabrication, and construction techniques developed through this initial pilot project, the next stage of implementation will proceed with a greater understanding of what is involved in a precast paving operation. The completion of the first pilot projects will allow transportation agencies to better develop precast pavement specifications, and will help contractors to better understand precast paving procedures. In the end, precast paving techniques should be something that is acceptable to both transportation agencies and contractors, and easily incorporated into rehabilitation projects.

References

1. Wilde, James W., Steve Waalkes, and Rob Harrison. *Life Cycle Cost Analysis of Portland Cement Concrete Pavements*, Research Report 1739-1, Center for Transportation Research, The University of Texas at Austin, September 1999.

2. Merritt, David K., B. Frank McCullough, Ned H. Burns, and Anton K. Schindler. *The Feasibility of Using Precast Concrete Panels to Expedite Highway Pavement Construction*, Research Report 1517-1, Center for Transportation Research, The University of Texas at Austin, February 2000.

3. Mendoza Diaz, Alberto, B. Frank McCullough, and Ned H. Burns. *Design of the Texas Prestressed Concrete Pavement Overlays in Cooke and McLennan Counties and Construction of the McLennan County Project*, Research Report 555/556-1, Center for Transportation Research, The University of Texas at Austin, February 1986.

4. Chia, Way Seng, B. F. McCullough, and Ned H. Burns. *Field Evaluation of Subbase Friction Characteristics*, Research Report 401-5, Center for Transportation Research, The University of Texas at Austin, September 1986.

5. Rozycki, Dan K., and Robert Otto Rasmussen. *Assessment of Slab-Base Interaction in PCC Airfield Pavements*, research paper, The Transtec Group, Inc., Austin, Texas, 1998.

6. Wesevich, James W., B. F. McCullough, and Ned H. Burns. *Stabilized Subbase Friction Study for Concrete Pavements*, Research Report 459-1, Center for Transportation Research, The University of Texas at Austin, April 1987.

7. Mendoza Diaz, A., B. F. McCullough, and Ned H. Burns. *Behavior of Long Prestressed Pavement Slabs and Design Methodology*, Research Report 401-3, Center for Transportation Research, The University of Texas at Austin, September 1986.

8. Mandel, Elliott, Jose Tena-Colunga, Kenneth Hankins, Ned H. Burns, and B. Frank McCullough. *The Performance of a Prestressed Concrete Pavement*, Research Report 556-4F, Center for Transportation Research, The University of Texas at Austin, November 1989.

9. Cable, Neil D., B. F. McCullough, and N. H. Burns. *New Concepts in Prestressed Concrete Pavement*, Research Report 401-2, Center for Transportation Research, The University of Texas at Austin, December 1985.

10. Taute, A., B. F. McCullough, and W. R. Hudson. *Improvements to the Materials Characterization and Fatigue Life Prediction Methods of Texas Rigid Pavement Overlays Design Procedure*, Research Report 249-1, Center for Transportation Research, The University of Texas at Austin, March 1982.

11. Schindler, Anton K., Tony S. Krauss, and B. F. McCullough. *The Design Documentation of a High Performance Concrete Pavement Located on IH-35 in the Waco District, Texas*, Research Report 98-0215-1F, Center for Transportation Research, The University of Texas at Austin, May 1999.

12. Precast and Prestressed Concrete Institute. *PCI Design Handbook*, Precast and Prestressed Concrete Institute, Chicago, Illinois, 5th ed., 1999.

13. Otero-Jimenez, M., B. F. McCullough, and K. Hankins. *Monitoring of Siliceous River Gravel and Limestone Continuously Reinforced Concrete Pavement Test Sections in Houston 2 Years after Placement, and Development of a Crack Width Model for the CRCP-7 Program*, Research Report 1244-4, Center for Transportation Research, The University of Texas at Austin, March 1992.

14. Michalak, Chester H., and Tom Scullion. *MODULUS 5.0 User's Manual*, Research Report 1987-1, Texas Transportation Institute, The Texas A&M University, November 1995.

15. Whiteoak, C.D. "Analytical Pavement Design Using Programs for Personal Computers," *Highways and Transportation*, v. 37, n. 8, August 1990, pp. 31–35.

Appendix

Photos of Panel Fabrication and Construction


Long-line casting bed at the precast plant in Victoria, TX


Form for a typical full-width Base panel


Looking down one of the side forms of the casting bed


Bed set up for casting Central Stressing Panels

Forms set up for casting a Joint Panel.

IButton temperature instrumentation (chain of three IButtons) cast into a Central Stressing Panel

Placement of concrete in the forms.

Vibratory screed used to create a flat top surface on the panels

Application of the carpet drag finish to a finished set of panels

Application of two coats of curing compound to a finished set of panels

Texture created by the carpet drag finish

Finished set of Central Stressing Panels


Removal of the panels from the forms


Panels stacked at the precast plant

Placement of the asphalt leveling course on the frontage road

Finished asphalt leveling course for the partial-width panels

Trucks waiting to unload full-width panels

Placement of a joint panel over the polyethylene sheeting

Placement of a full-width Central Stressing Panel

Application of segmental bridge epoxy to the panel keyways

Come-alongs linked between the edge sleeves were used to pull the panels together as they were lowered into place

The come-alongs were used to hold the joints between panels closed as subsequent panels were placed.

A full section of panels (between expansion joints) were placed in 6 –8 hours

A typical joint between panels prior to post-tensioning

Removable covers provide access to the temperature instrumentation at the edge of the slab

As panel placement continues, the post-tensioning strands are cut to length

The longitudinal post-tensioning strands are fed into the ducts at the Central Stressing Panels and fed by hand to the anchors in the Joint Panels.

The strands coming into the stressing pockets from either end of the slab are coupled together with a ring anchor

A monostrand stressing ram is used to tension the entire tendon by pulling on one strand while reacting against the other strand

Holes are drilled into the base material at each stressing pocket for the mid-slab anchor

A #7 deformed bar is grouted in the hole to provide the mid-slab anchor

The stressing pockets and access pockets are filled prior to grouting the tendons

Grout tubes are attached to each inlet/vent for tendon grouting

The tendons are grouted by pumping grout into the inlets/vents along the slab

Placement of a section of
20-ft partial-width panels

Placement of the adjacent set of 16-ft
partial-width panels

Staging for partial-width panels placement

Placement of a partial-width panel over
the polyethylene sheeting

After placement of a pair of partial-width
panels, the transverse post-tensioning strands
were fed into the ducts.

The transverse post-tensioning tendons
were stressed with a monostrand stressing
ram at the edge of the pavement.

After stressing, the transverse post-tensioning tendons were grouted.

Looking down the centerline of the full-width panels after opening to traffic

Looking down the centerline of the partial-width panels after opening to traffic

Finished precast pavement pilot project near Georgetown, Texas


Typical mid-panel crack which occurred overnight on many of the 36 ft panels


Corner breaks occurred when removing some of the panels from the forms.


The armored joints were significantly bowed when they arrived from the steel fabricator.


The last section of the leveling course was roughly finished and some voids were present

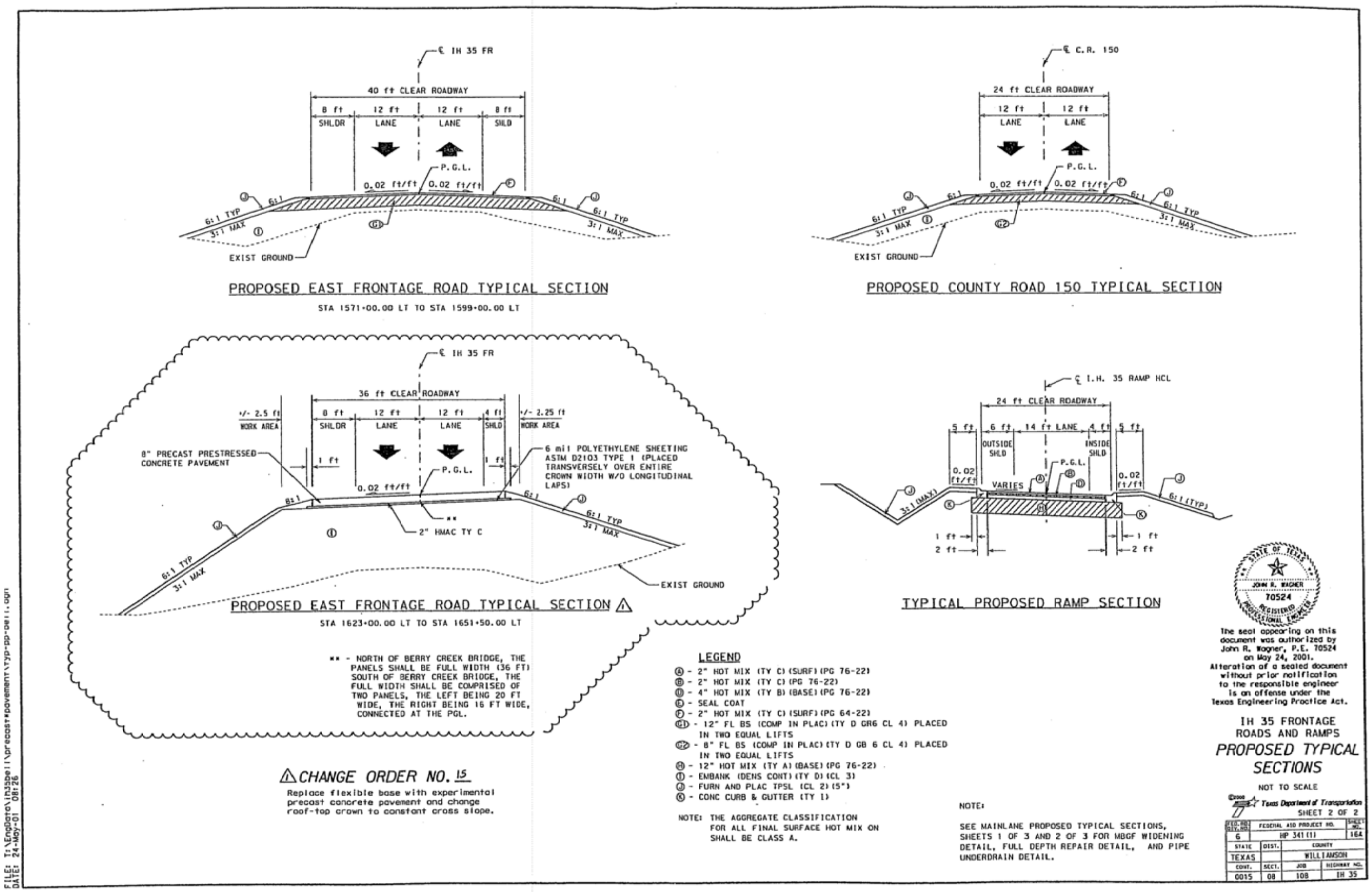

Grout leakage onto the top of the slab at the panel joints

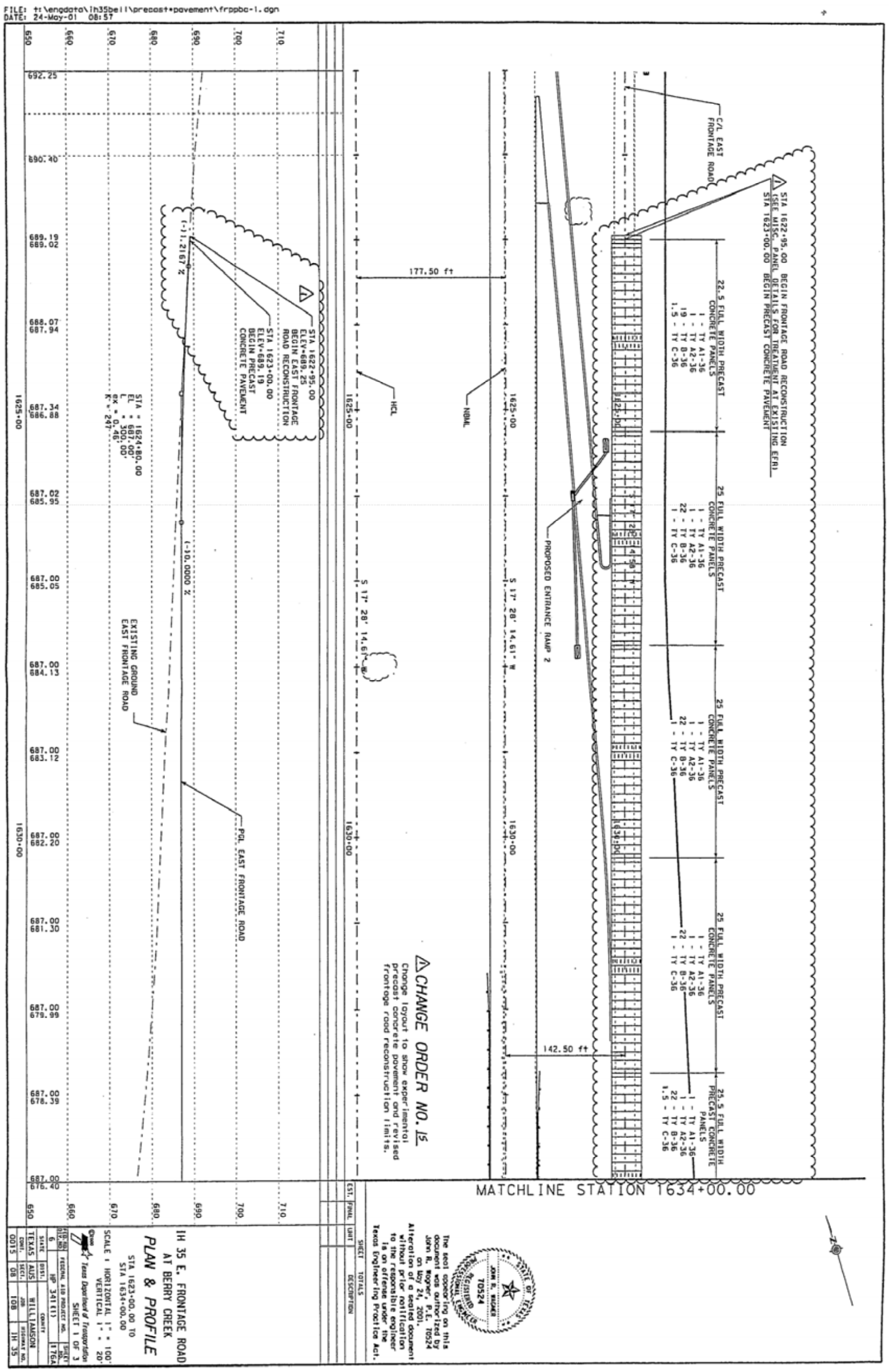

When pushing the strands through the ducts, some of the individual wires would push back, leaving only 4 – 5 wires to go into the anchor.

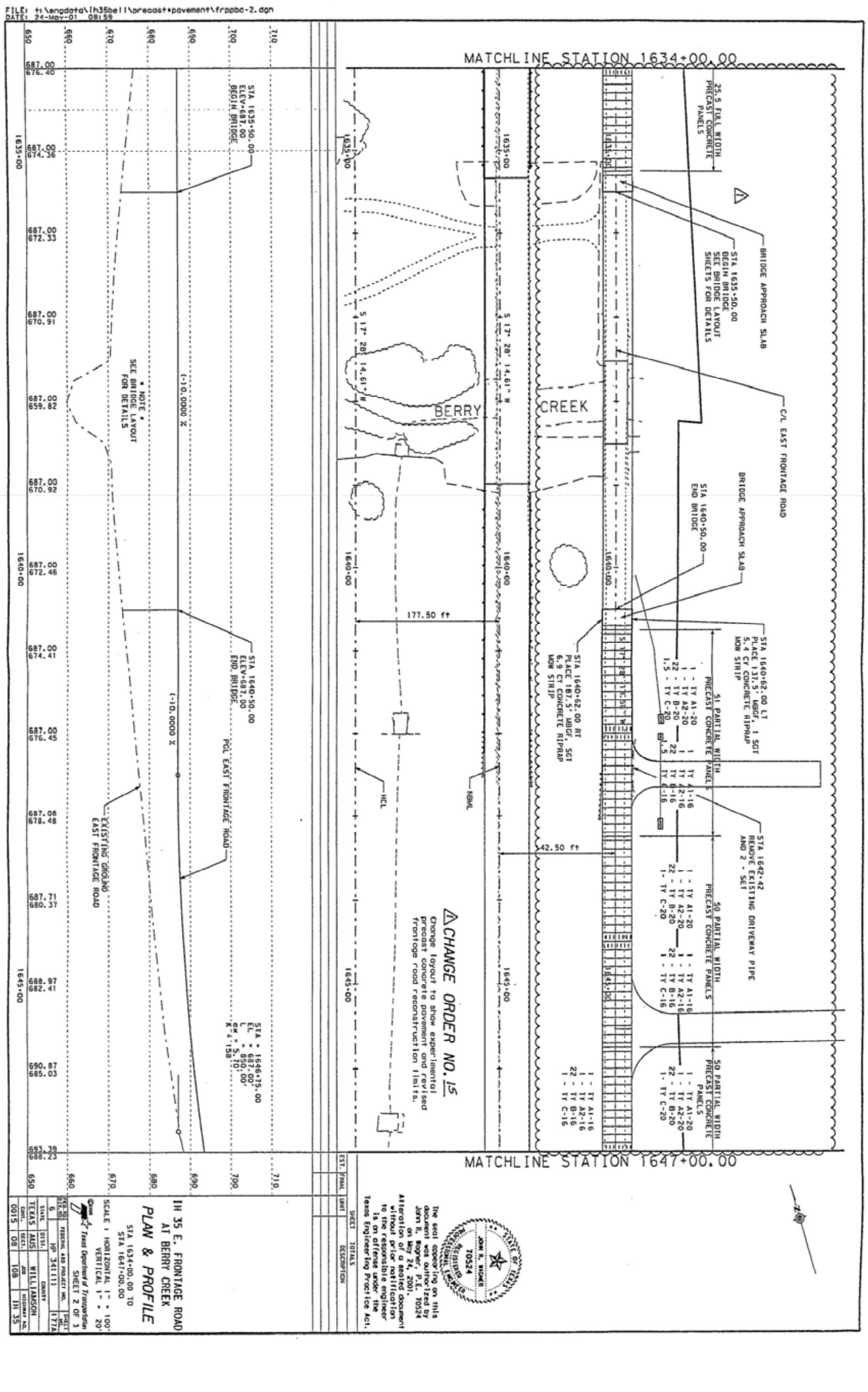

Typical on-site repair to a corner crack in a partial-width panel at the joint between 16-ft and 20-ft panels

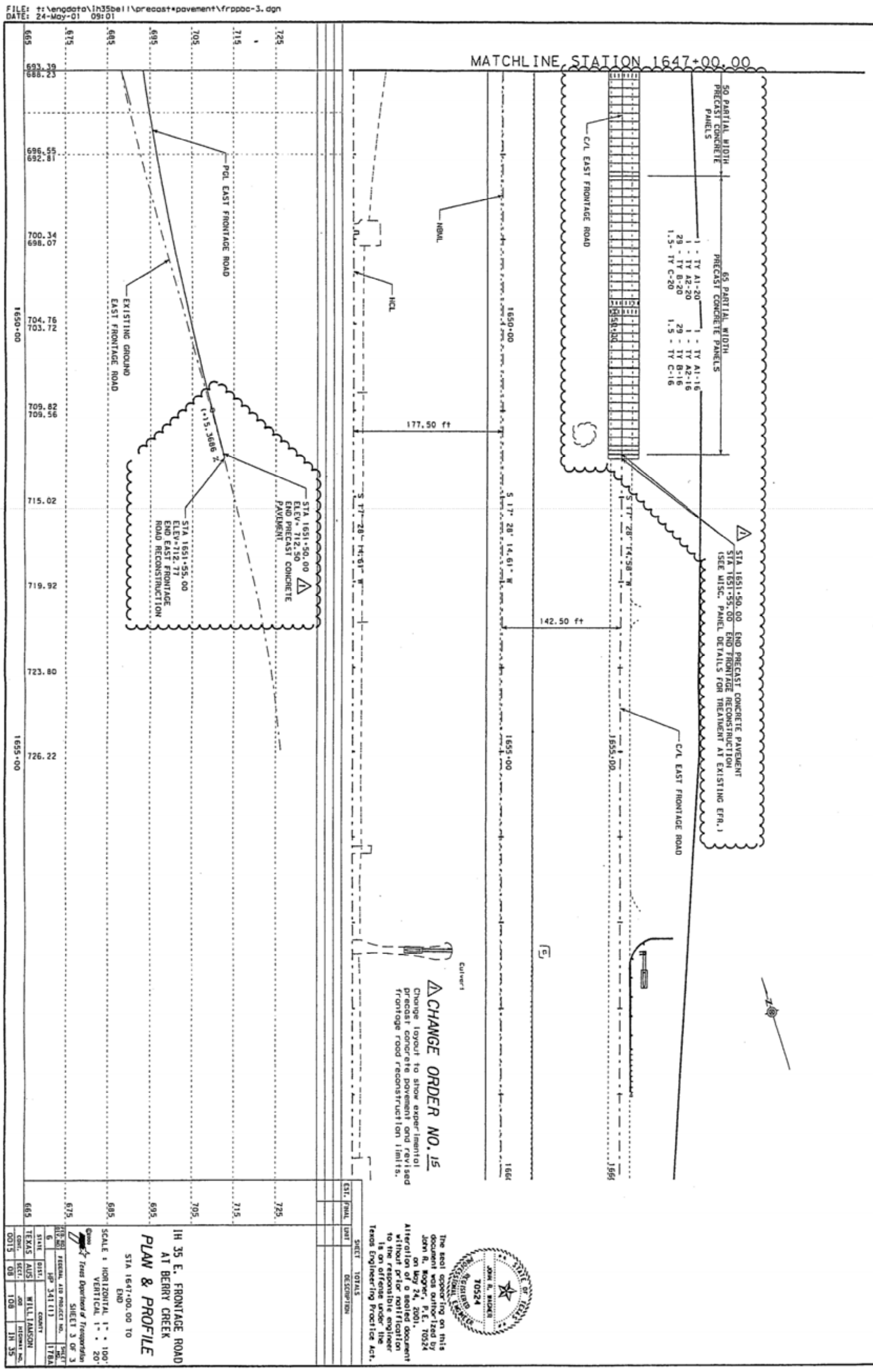

Grout leakage from the ends of the panel joints

Project Plans and Panel Drawings


FILE: T:\EngData\ih35bell\precast*pavement\typ-pp-bell.dgn
DATE: 24-May-01 08:26
℄ IH 35 FR
40 ft CLEAR ROADWAY
8 ft
12 ft
12 ft
8 ft
SHLDR
LANE
LANE
SHLD
P.G.L.
0.02 ft/ft
0.02 ft/ft
6:1
6:1 TYP
3:1 MAX
EXIST GROUND
PROPOSED EAST FRONTAGE ROAD TYPICAL SECTION
STA 1571+00.00 LT TO STA 1599+00.00 LT
℄ C.R. 150
24 ft CLEAR ROADWAY
PROPOSED COUNTY ROAD 150 TYPICAL SECTION
36 ft CLEAR ROADWAY
+/- 2.5 ft
WORK AREA
4 ft
+/- 2.25 ft
WORK AREA
8" PRECAST PRESTRESSED CONCRETE PAVEMENT
1 ft
6 mil POLYETHYLENE SHEETING ASTM D2103 TYPE 1 (PLACED TRANSVERSELY OVER ENTIRE CROWN WIDTH W/O LONGITUDINAL LAPS)
2" HMAC TY C
PROPOSED EAST FRONTAGE ROAD TYPICAL SECTION
STA 1623+00.00 LT TO STA 1651+50.00 LT
** - NORTH OF BERRY CREEK BRIDGE, THE PANELS SHALL BE FULL WIDTH (36 FT) SOUTH OF BERRY CREEK BRIDGE, THE FULL WIDTH SHALL BE COMPRISED OF TWO PANELS, THE LEFT BEING 20 FT WIDE, THE RIGHT BEING 16 FT WIDE, CONNECTED AT THE PGL.
CHANGE ORDER NO. 15
Replace flexible base with experimental precast concrete pavement and change roof-top crown to constant cross slope.
℄ I.H. 35 RAMP HCL
24 ft CLEAR ROADWAY
5 ft
6 ft
14 ft LANE
4 ft
5 ft
OUTSIDE SHLD
INSIDE SHLD
VARIES
3:1 (MAX)
6:1 (TYP)
2 ft
TYPICAL PROPOSED RAMP SECTION
LEGEND
Ⓐ - 2" HOT MIX (TY C) (SURF) (PG 76-22)
Ⓑ - 2" HOT MIX (TY C) (PG 76-22)
Ⓓ - 4" HOT MIX (TY B) (BASE) (PG 76-22)
Ⓔ - SEAL COAT
Ⓕ - 2" HOT MIX (TY C) (SURF) (PG 64-22)
G1 - 12" FL BS (COMP IN PLAC) (TY D GR6 CL 4) PLACED IN TWO EQUAL LIFTS
G2 - 8" FL BS (COMP IN PLAC) (TY D GB 6 CL 4) PLACED IN TWO EQUAL LIFTS
Ⓗ - 12" HOT MIX (TY A) (BASE) (PG 76-22)
Ⓘ - EMBANK (DENS CONT) (TY D) (CL 3)
Ⓙ - FURN AND PLAC TPSL (CL 2) (5")
Ⓚ - CONC CURB & GUTTER (TY 1)
NOTE: THE AGGREGATE CLASSIFICATION FOR ALL FINAL SURFACE HOT MIX ON SHALL BE CLASS A.
NOTE:
SEE MAINLANE PROPOSED TYPICAL SECTIONS, SHEETS 1 OF 3 AND 2 OF 3 FOR MBGF WIDENING DETAIL, FULL DEPTH REPAIR DETAIL, AND PIPE UNDERDRAIN DETAIL.
STATE OF TEXAS
JOHN R. WAGNER
70524
REGISTERED PROFESSIONAL ENGINEER
The seal appearing on this document was authorized by John R. Wagner, P.E. 70524 on May 24, 2001.
Alteration of a sealed document without prior notification to the responsible engineer is an offense under the Texas Engineering Practice Act.
IH 35 FRONTAGE ROADS AND RAMPS
PROPOSED TYPICAL SECTIONS
NOT TO SCALE
Texas Department of Transportation
SHEET 2 OF 2
FEDERAL AID PROJECT NO.
6
HP 341 (1)
164
STATE
DIST.
COUNTY
TEXAS
WILLIAMSON
CONT.
SECT.
JOB
HIGHWAY NO.
0015
08
108
IH 35


FILE: t:\engdata\ih35bell\precast+pavement\frppbc-1.dgn
DATE: 24-May-01 08:57
MATCHLINE STATION 1634+00.00
CHANGE ORDER NO. 15
Change layout to show experimental precast concrete pavement and revised frontage road reconstruction limits.
PROPOSED ENTRANCE RAMP 2
PGL EAST FRONTAGE ROAD
EXISTING GROUND EAST FRONTAGE ROAD
IH 35 E. FRONTAGE ROAD AT BERRY CREEK
PLAN & PROFILE
STA 1623+00.00 TO STA 1634+00.00


MATCHLINE STATION 1634+00.00
MATCHLINE STATION 1647+00.00
BERRY
CREEK
BRIDGE APPROACH SLAB
C/L EAST FRONTAGE ROAD
177.50 ft
42.50 ft
PGL EAST FRONTAGE ROAD
EXISTING GROUND EAST FRONTAGE ROAD
CHANGE ORDER NO. 15
Change layout to show experimental precast concrete pavement and revised frontage road reconstruction limits.
IH 35 E. FRONTAGE ROAD AT BERRY CREEK
PLAN & PROFILE
STA 1634+00.00 TO STA 1647+00.00



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DATE: 24-May-01 09:01
MATCHLINE, STATION 1647+00.00
50 PARTIAL WIDTH PRECAST CONCRETE PANELS
65 PARTIAL WIDTH PRECAST CONCRETE PANELS
C/L EAST FRONTAGE ROAD
PGL EAST FRONTAGE ROAD
EXISTING GROUND EAST FRONTAGE ROAD
STA 1651+50.00 ELEV= 712.50 END PRECAST CONCRETE PAVEMENT
STA 1651+55.00 ELEV=712.77 END EAST FRONTAGE ROAD RECONSTRUCTION
STA 1651+50.00 END PRECAST CONCRETE PAVEMENT
STA 1651+55.00 END FRONTAGE RECONSTRUCTION
(SEE MISC. PANEL DETAILS FOR TREATMENT AT EXISTING EFR.)
177.50 ft
142.50 ft
1650+00
1655+00
CHANGE ORDER NO. 15
Change layout to show experimental precast concrete pavement and revised frontage road reconstruction limits.
IH 35 E. FRONTAGE ROAD AT BERRY CREEK
PLAN & PROFILE
STA 1647+00.00 TO END
SCALE : HORIZONTAL 1" : 100' VERTICAL 1" : 20'
Texas Department of Transportation
SHEET 3 OF 3
TEXAS AUS WILLIAMSON
0015 08 108 IH 35

GENERAL NOTES:

PROVISIONS SHALL BE MADE FOR LIFTING PANELS USING STARCON ANCHOR SYSTEM OR EQUIVALENT AS APPROVED BY THE ENGINEER. THE LOCATION OF THE ANCHORS SHALL BE BETWEEN 0.2071L AND 0.25L (APPROXIMATE QUARTER POINTS), WITH A MINIMUM OF FOUR (4) ANCHORS.

PANELS SHALL HAVE A CARPET DRAG FINISH AS PER ITEM 360, "CONCRETE PAVEMENT."

CENTRAL STRESSING POCKETS, JOINT PANEL POCKETS, AND LIFTING ANCHORS SHALL BE FILLED AS SPECIFIED UNDER ITEM 361. THE SURFACE OF THE PATCH SHALL RECEIVE A FINISH THAT MATCHES AS CLOSE AS POSSIBLE THE SURFACE OF THE SURROUNDING PANEL.

PANELS SHOULD BE CURED USING POLYETHELENE SHEETING OR COTTON MATS AS PER ITEM 420, "CONCRETE STRUCTURES."

PANELS SHALL REACH A MINIMUM STRENGTH OF 3500 PSI AT RELEASE OF PRESTRESS AND 5000 PSI AT 28 DAYS.

PANELS OF ALL DIMENSIONS, REGARDLESS OF TYPE, SHALL MEET THE REQUIREMENTS UNDER ITEM 425, "PRESTRESSED CONCRETE STRUCTURAL MEMBERS."

ALL 7-WIRE STRAND SHALL BE GRADE 270 (LOW RELAXATION) AS PER ITEM 426, "PRESTRESSING."

LONGITUDINAL AND TRANSVERSE POST-TENSIONING DUCTS SHALL BE STRAIGHT (LOOSE BAR STIFFENERS OR RMC MAY BE PLACED INSIDE P-T DUCTS DURING CASTING). ALL P-T DUCTS SHALL MEET THE SPECIFICATIONS FOR GROUTED TENDONS AS PER ITEM 426.

ALL PLAIN REINFORCEMENT SHALL BE GRADE 60 AS PER ITEM 440, "REINFORCING STEEL."

THE THICKNESS OF THE VISQUEEN SHEET BETWEEN THE PANELS AND THE HMACP SHALL BE 6 MILS.

PANEL TOLERANCES SHALL MEET THE REQUIREMENTS IN THE "TABLE OF ALLOWABLE TOLERANCES" BELOW.

MATERIALS IN THE PANELS SHALL MEET THE REQUIREMENTS AS SET FORTH IN THE FOLLOWING TXDOT SPECIFICATIONS:

ITEM 421 "PORTLAND CEMENT CONCRETE"
ITEM 426 "PRESTRESSING"
ITEM 433 "JOINT SEALANTS AND FILLERS"
ITEM 440 "REINFORCING STEEL"
ITEM 454 "SEALED EXPANSION JOINTS"

REFERENCE ITEMS SHALL BE MEET THE REQUIREMENTS AS SET FORTH IN THE FOLLOWING TXDOT SPECIFICATIONS:

ITEM 360 "CONCRETE PAVEMENT" [CARPET DRAG FINISH]
ITEM 361 "FULL-DEPTH REPAIR OF EXISTING CONCRETE PAVEMENT" [PATCHES]
ITEM 420 "CONCRETE STRUCTURES"
ITEM 421 "PORTLAND CEMENT CONCRETE"
ITEM 424 "PRECAST CONCRETE STRUCTURES (FABRICATION)"
ITEM 426 "PRESTRESSING"
ITEM 427 "SURFACE FINISHES FOR CONCRETE"
ITEM 435 "ELASTOMERIC MATERIALS"
ITEM 440 "REINFORCING STEEL"
ITEM 442 "METAL FOR STRUCTURES"
ITEM 524 "HYDRAULIC CEMENT"
ITEM 5295 "EPOXY AND ADHESIVES"
ITEM 5591 "RIDE QUALITY FOR PAVEMENT SURFACES"

THE RIDE QUALITY REFERENCE SPECIFICATION SHALL APPLY TO THE ASPHALT SURFACE BELOW THE CONCRETE PANELS.

THE CONTRACTOR MAY BE REQUIRED TO DIAMOND GRIND THE TOP SURFACE OF THE PANELS TO PROVIDE A RIDE ACCEPTABLE TO THE ENGINEER.

VOIDS BENEATH THE PANELS, AS DETERMINED FROM VISUAL INSPECTION DURING PANEL PLACEMENT AND/OR NON-DESTRUCTIVE TESTING OF THE FINISHED PAVEMENT, SHALL BE FILLED USING GROUT INJECTION AT THE DISCRETION OF THE ENGINEER.

THE PROCESS FOR CURING THE PANELS SHALL BE AS FOLLOWS:

1. CURING COMPOUND SHALL BE APPLIED IMMEDIATELY AFTER THE CARPET DRAG HAS BEEN COMPLETED.
2. A "TARP TENT" MADE OF HEAVY MATERIAL SHALL BE PLACED OVER THE PANELS UNTIL TIME THE PANELS HAVE REACHED RELEASE STRENGTH.
3. THE FORMS SHALL THEN BE STRIPPED AND THE PANELS STACKED HORIZONTALLY IN SUCH A MANNER TO INSURE THAT ONLY MINIMAL CREEP DEFORMATION OF CONCRETE WILL BE INTRODUCED TO THE PANEL. (MOVING AND STACKING THE PANELS WILL NOT COUNT AGAINST CURING TIME)
4. THE PANELS SHALL THEN BE COVERED WITH WET MATS AND "TARP TENTS".

CURING SHALL BE MAINTAINED FOR A MINIMUM OF 48 HOURS AFTER COMPLETION OF THE FIRST APPLICATION OF CURING COMPOUND.

REPAIRS TO ANY PART OF A PANEL RIDING SURFACE THAT WILL BE IN CONTACT WITH ANOTHER PANEL WHEN PLACED IN THE FIELD SHALL BE REPAIRED IN THE FIELD IN ACCORDANCE WITH ITEM 361 AFTER THE PANELS HAVE BEEN POST-TENSIONED TOGETHER. ALL OTHER REPAIRS SHALL BE PERFORMED AT THE PLANT AT THE DISCRETION OF THE ENGINEER.

ANY REPAIRS TO KEYWAYS OR ADJOINING EDGES OF PANELS SHALL BE SUCH THAT THE REPAIRED AREAS WILL NOT PROTRUDE PAST THE ADJACENT TRUE EDGES OF THE KEYWAY OR PANEL EDGE AS TO MINIMIZE STRESSES IN THE JOINTS DUE TO THE REPAIRS.

RECOMMENDED JOINT WIDTH AT PLACEMENT OF PANELS

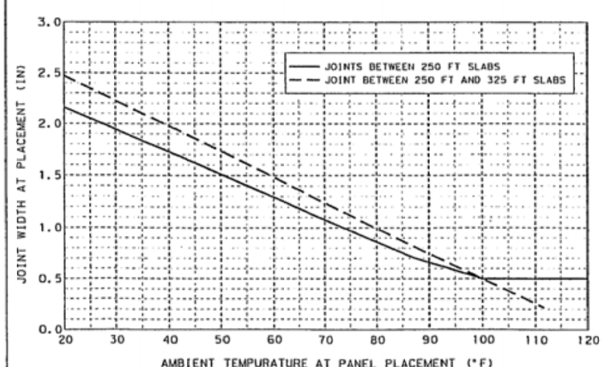

TABLE OF ALLOWABLE TOLERANCES

LENGTH (LONGITUDINAL TO C/L)		+/- 1/4"
WIDTH (TRANSVERSE TO C/L)		+/- 1/4"
NOMINAL THICKNESS		+/- 1/16"
HORIZONTAL ALIGNMENT (UPON RELEASE OF STRESS) - DEVIATION FROM STRAIGHTNESS OF MATING EDGE OF PANELS		+/- 1/4"
DEVIATION OF ENDS FROM SHOP PLAN DIMENSION (HORIZONTAL SKEW)		+/- 1/4"
POSITION OF STRANDS	HORIZ	+/- 1/8"
	VERT *	+/- 1/8"
POSITION OF HANDLING DEVICES		+/- 3"

NOTES: * MEASURED FROM BOTTOM OF PANEL

STATE OF TEXAS
BENJAMIN F. McCULLOUGH
19914
REGISTERED PROFESSIONAL ENGINEER

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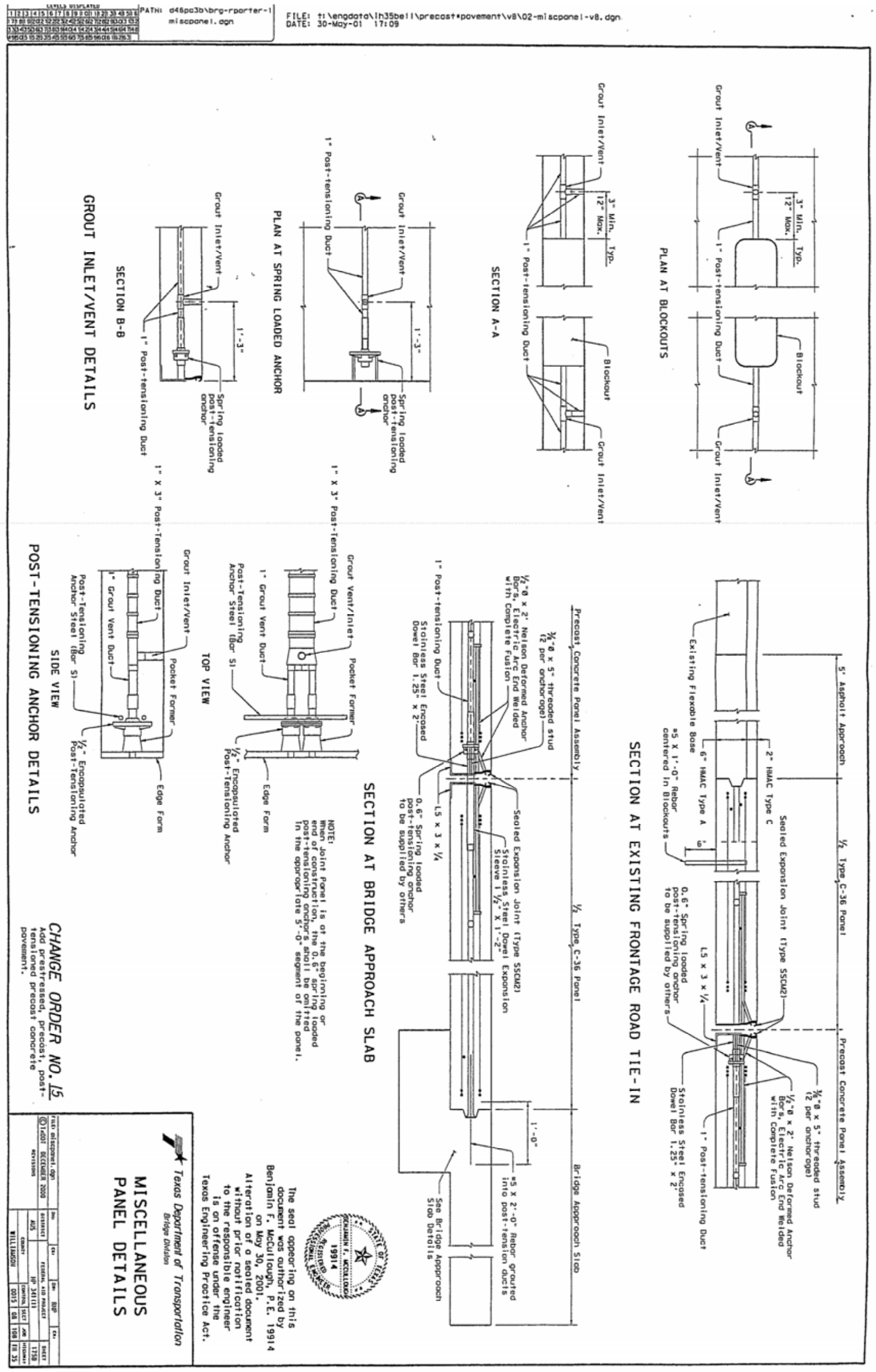
CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.

Texas Department of Transportation
Bridge Division

GENERAL NOTES
FOR ALL
PANEL WIDTHS

FILE: full joint.dgn	DN:	CK:	DW: RMP	CK:
© TxDOT	DISTRICT AUS	FEDERAL AID PROJECT IM 34(11)		SHEET 175A
REVISIONS	COUNTY WILLIAMSON	CONTROL 0015	SECT 08	JOB 108	HIGHWAY IH 35



PLAN AT BLOCKOUTS
SECTION A-A
PLAN AT SPRING LOADED ANCHOR
SECTION B-B
GROUT INLET/VENT DETAILS
SECTION AT EXISTING FRONTAGE ROAD TIE-IN
SECTION AT BRIDGE APPROACH SLAB
TOP VIEW
SIDE VIEW
POST-TENSIONING ANCHOR DETAILS
NOTE: When Joint Panel is at the beginning or end of construction, the 0.6" spring loaded post-tensioning anchors shall be omitted in the appropriate 5'-0" segment of the panel.
CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.
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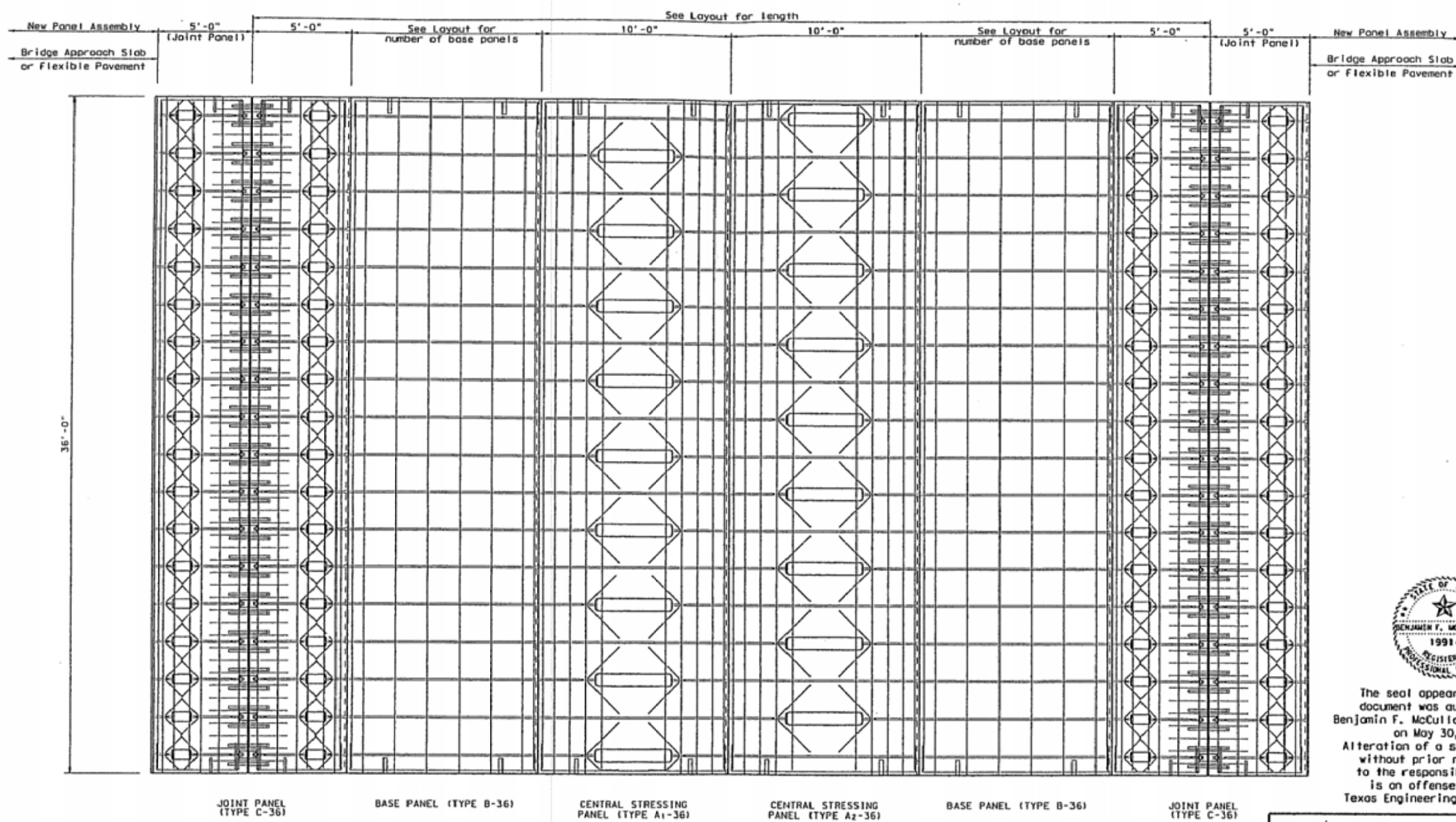
Texas Department of Transportation
Bridge Division
MISCELLANEOUS PANEL DETAILS

FILE: t:\engdata\ih35bell\precast*pavement\v8\04-assembly36-v8.dgn
DATE: 30-May-01 17:16

PATH: d48pc3b\brg-rporter-1
miscpanel.dgn



TYPICAL PANEL ASSEMBLY

NOTE:
When Joint Panel is at the beginning or end of construction, the 0.6" spring loaded post-tensioning anchors and grout inlets/vents may be omitted in the appropriate 5'-0" segment of the panel.

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Texas Department of Transportation
Bridge Division

36' FULL WIDTH PANEL ASSEMBLY

CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.

FILE: miscpanel.dgn | © TxDOT DECEMBER 2000 | DISTRICT 14 | FEDERAL AID PROJECT IM 341(1) | SHEET 1750 | COUNTY WILLIAMSON | CONTROL 0015 | SECT 08 | JOB 108 | HIGHWAY IH 35

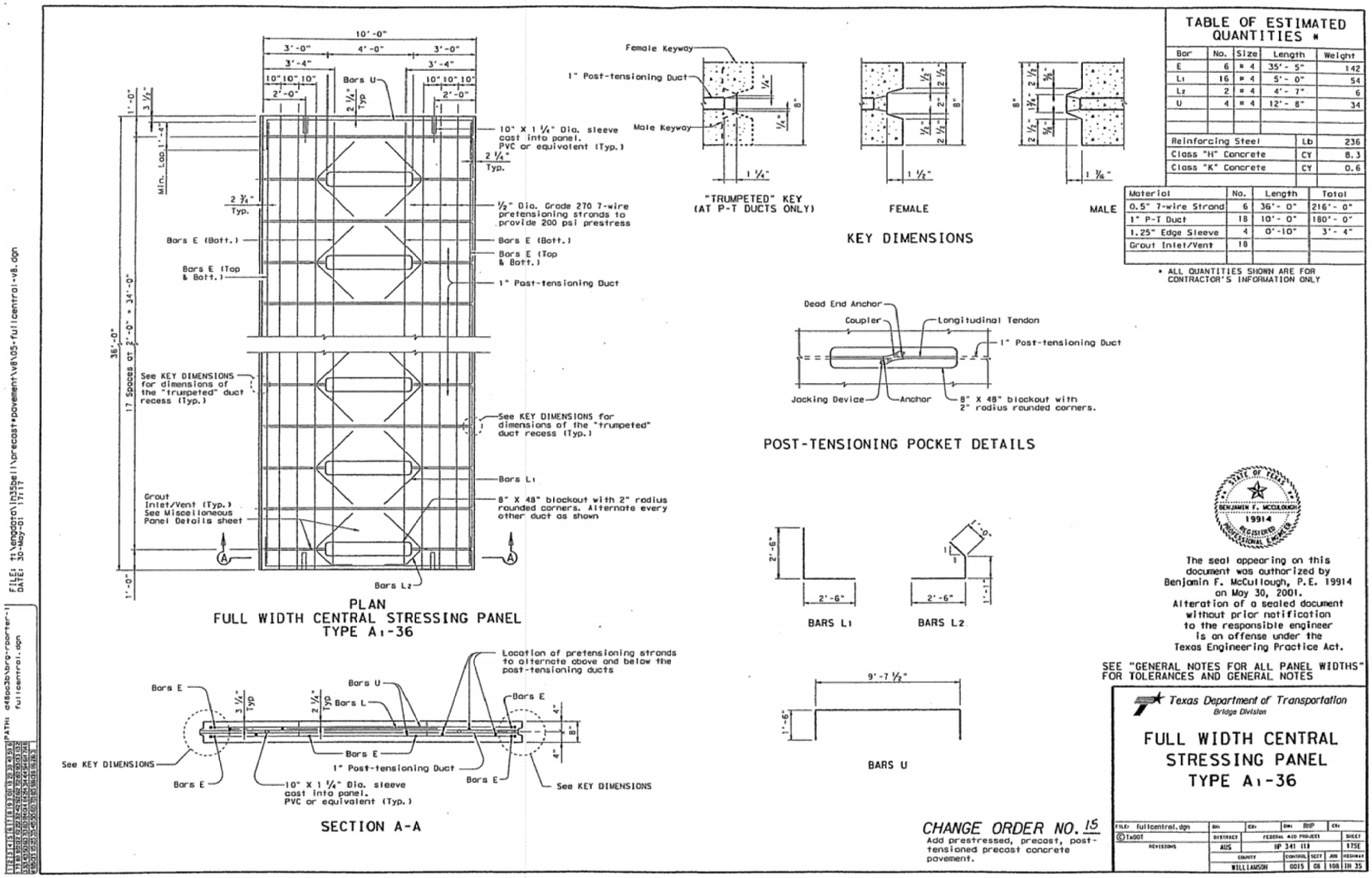


TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
E	6	# 4	35'- 5"	142
L1	16	# 4	5'- 0"	54
L2	2	# 4	4'- 7"	6
U	4	# 4	12'- 8"	34
Reinforcing Steel			Lb	236
Class "H" Concrete			CY	8.3
Class "K" Concrete			CY	0.6

Material	No.	Length	Total
0.5" 7-wire Strand	6	36'- 0"	216'- 0"
1" P-T Duct	18	10'- 0"	180'- 0"
1.25" Edge Sleeve	4	0'-10"	3'- 4"
Grout Inlet/Vent	18		

* ALL QUANTITIES SHOWN ARE FOR CONTRACTOR'S INFORMATION ONLY

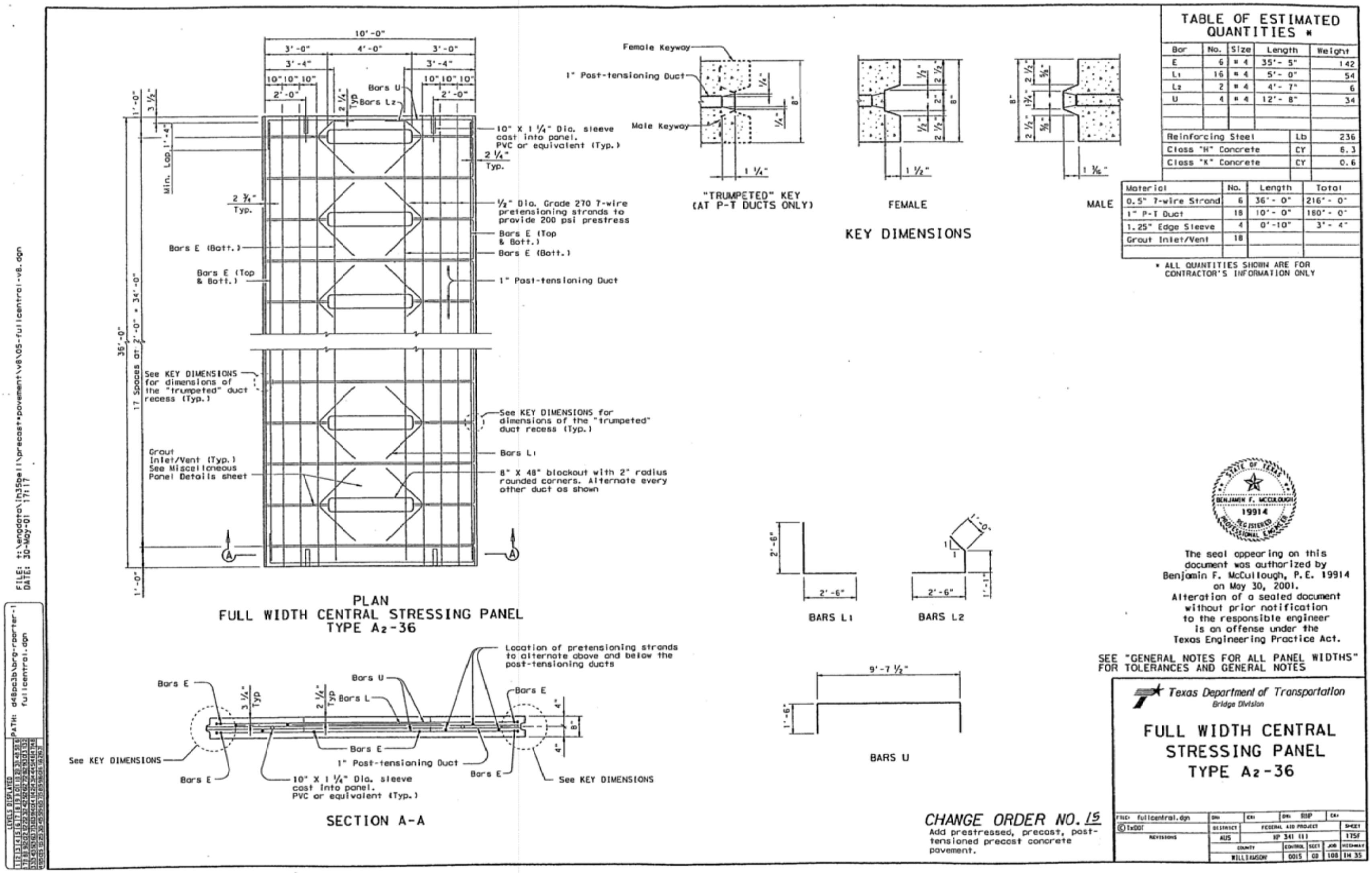


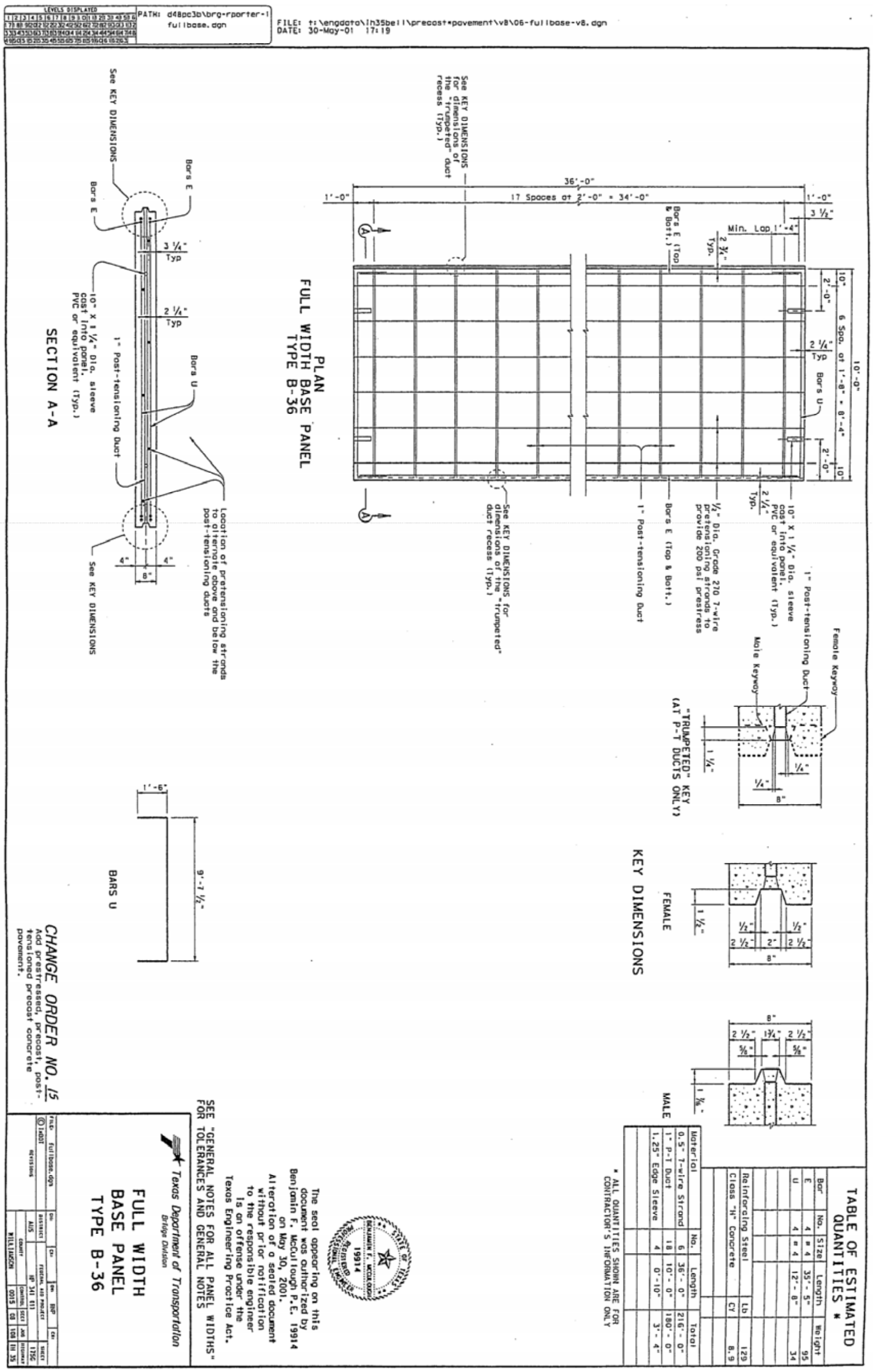
TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
E	6	# 4	35'- 5"	142
L1	16	# 4	5'- 0"	54
L2	2	# 4	4'- 7"	6
U	4	# 4	12'- 8"	34

Reinforcing Steel	Lb	236
Class "H" Concrete	CY	6.3
Class "K" Concrete	CY	0.6

Material	No.	Length	Total
0.5" 7-wire Strand	6	36'- 0"	216'- 0"
1" P-T Duct	18	10'- 0"	180'- 0"
1.25" Edge Sleeve	4	0'-10"	3'- 4"
Grout Inlet/Vent	18		

* ALL QUANTITIES SHOWN ARE FOR CONTRACTOR'S INFORMATION ONLY


PLAN
FULL WIDTH BASE PANEL
TYPE B-36
SECTION A-A
KEY DIMENSIONS
"TRUMPETED" KEY
(AT P-T DUCTS ONLY)
FEMALE
MALE
BARS U
TABLE OF ESTIMATED QUANTITIES *
* ALL QUANTITIES SHOWN ARE FOR CONTRACTOR'S INFORMATION ONLY
SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES
CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.
Texas Department of Transportation
FULL WIDTH
BASE PANEL
TYPE B-36

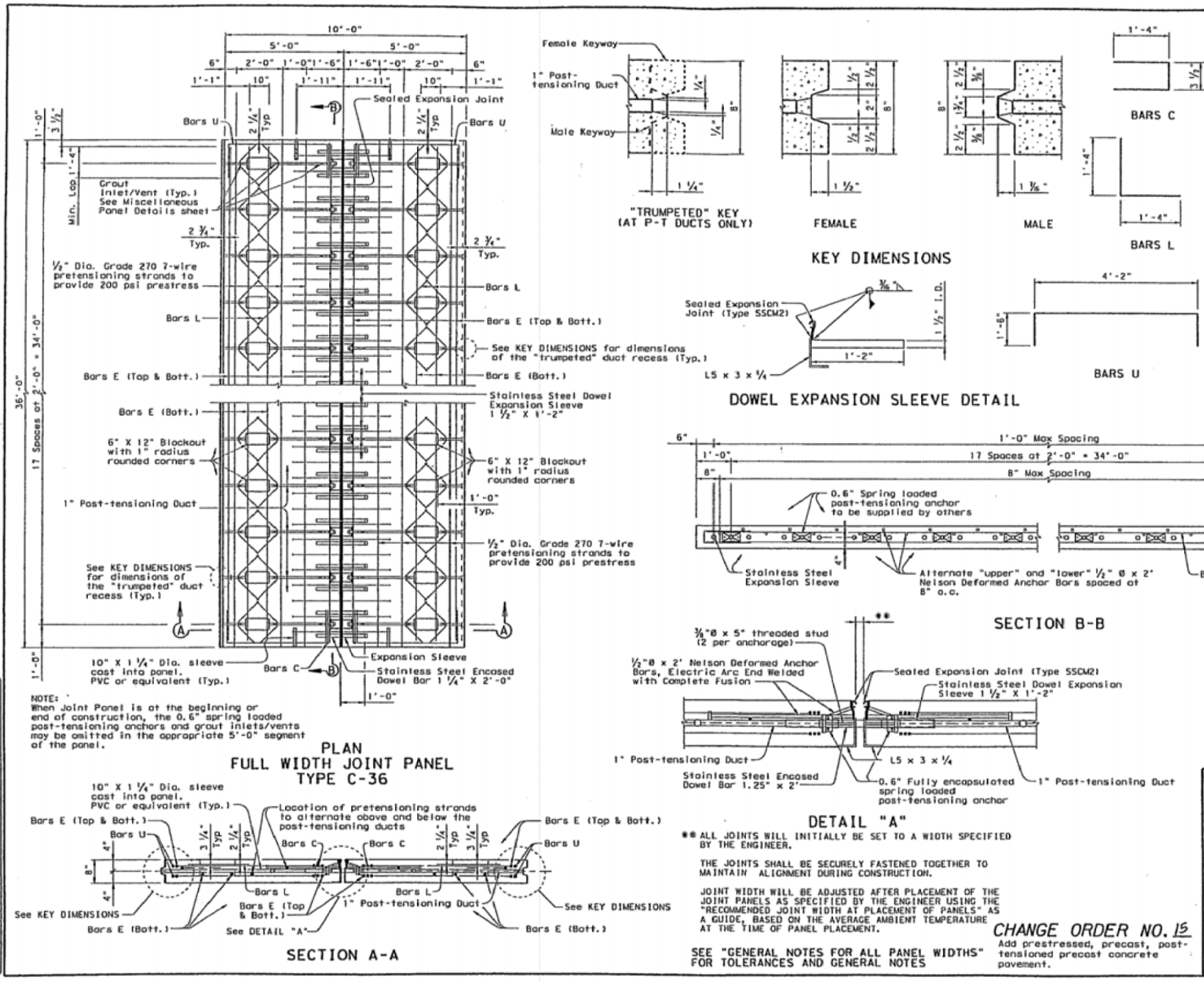


TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
C	4	# 4	3'- 0"	8
E	12	# 4	35'- 5"	284
L	72	# 4	2'- 8"	128
U	8	# 4	7'- 2"	38

Reinforcing Steel	Lb	458
Class "H" Concrete	CY	6.5
Class "K" Concrete	CY	0.4

Material	No.	Length	Total
0.5" 7-wire Strand	6	36'- 0"	216'- 0"
1" P-T Duct	18	10'- 0"	180'- 0"
1.25" Edge Sleeve	4	0'-10"	3'- 4"
Sprng Ld P-T Anchr	36		
L 5 x 3 x ¼"	2	36'- 0"	72'- 0"
Steel Exp. Joint	2	36'- 0"	72'- 0"
0.5" Nlsn Def Bar	106	2'- 0"	212'- 0"
Dowel Exp. Sleeve	36		
1.25" Dowel Bar	36	2'- 0"	70'- 0"
Grout Inlet/Vent	108		

* ALL QUANTITIES SHOWN ARE FOR CONTRACTOR'S INFORMATION ONLY

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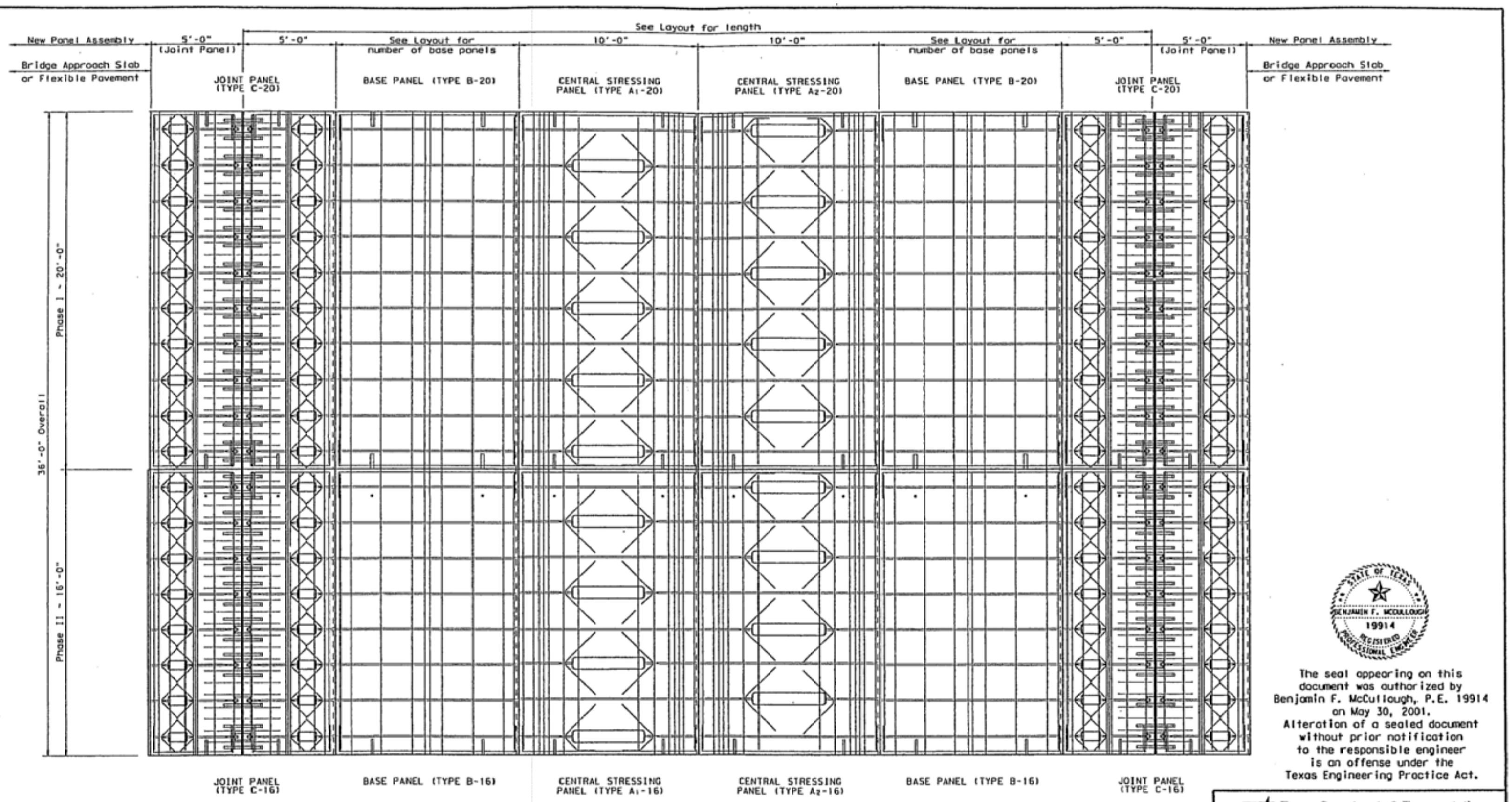
Texas Department of Transportation
Bridge Division

FULL WIDTH
JOINT PANEL
TYPE C-36



NOTE:
When Joint Panel is at the beginning or end of construction, the 0.6" spring loaded post-tensioning anchors and grout inlets/vents may be omitted in the appropriate 5'-0" segment of the panel.

TYPICAL PANEL ASSEMBLY

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CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.

Texas Department of Transportation
Bridge Division

16' PARTIAL WIDTH & 20' PARTIAL WIDTH PANEL ASSEMBLY

FILE: miscpanel 16.dgn
©TxDOT DECEMBER 2000
AUS | WILLIAMSON | 0015 | 08 | 108 | IH 35 | 1751

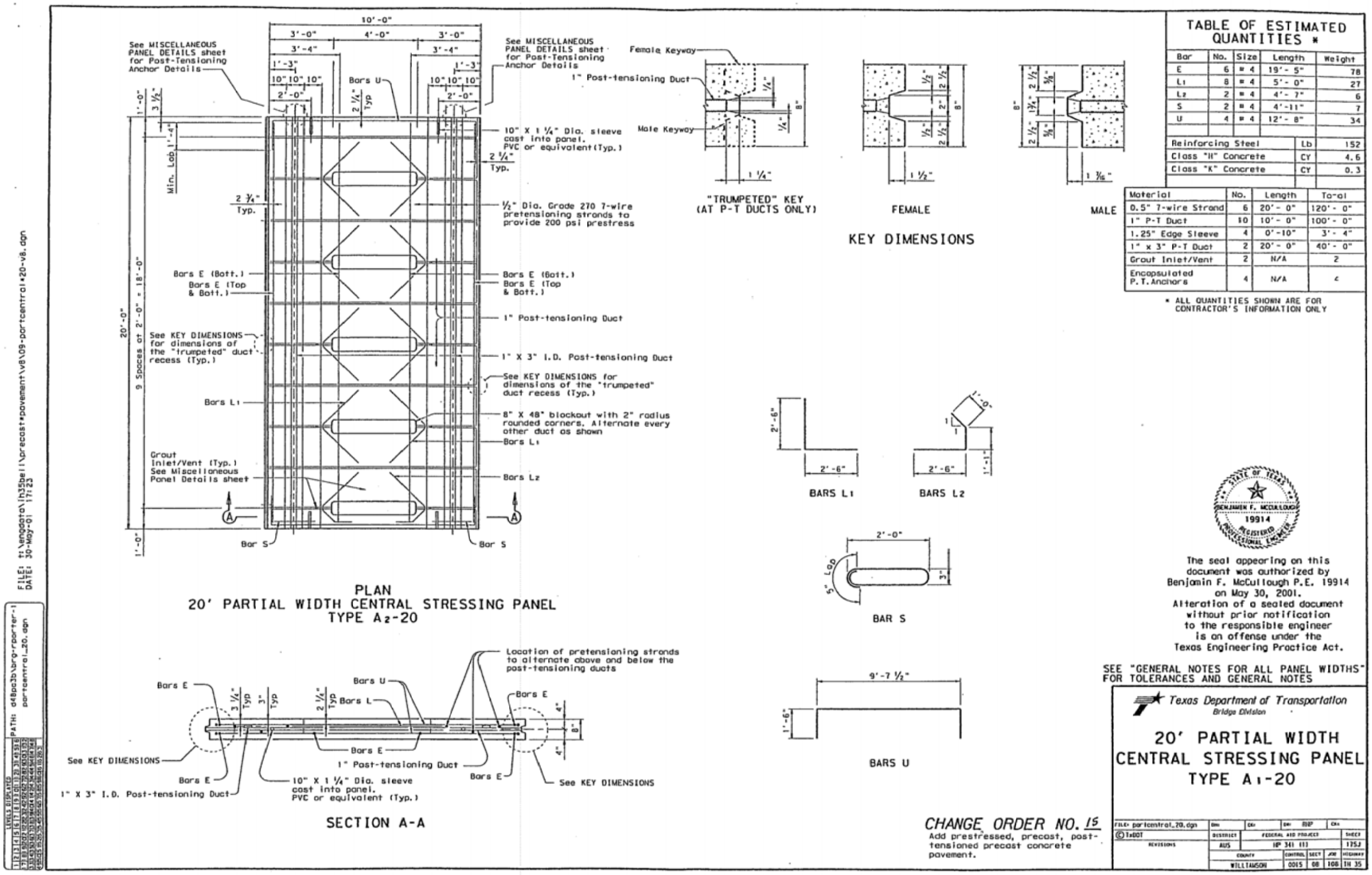

TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
E	6	# 4	19'- 5"	78
L1	8	# 4	5'- 0"	27
L2	2	# 4	4'- 7"	6
S	2	# 4	4'-11"	7
U	4	# 4	12'- 8"	34
Reinforcing Steel			Lb	152
Class "H" Concrete			CY	4.6
Class "K" Concrete			CY	0.3

Material	No.	Length	Total
0.5" 7-wire Strand	6	20'- 0"	120'- 0"
1" P-T Duct	10	10'- 0"	100'- 0"
1.25" Edge Sleeve	4	0'-10"	3'- 4"
1" x 3" P-T Duct	2	20'- 0"	40'- 0"
Grout Inlet/Vent	2	N/A	2
Encapsulated P.T. Anchors	4	N/A	4

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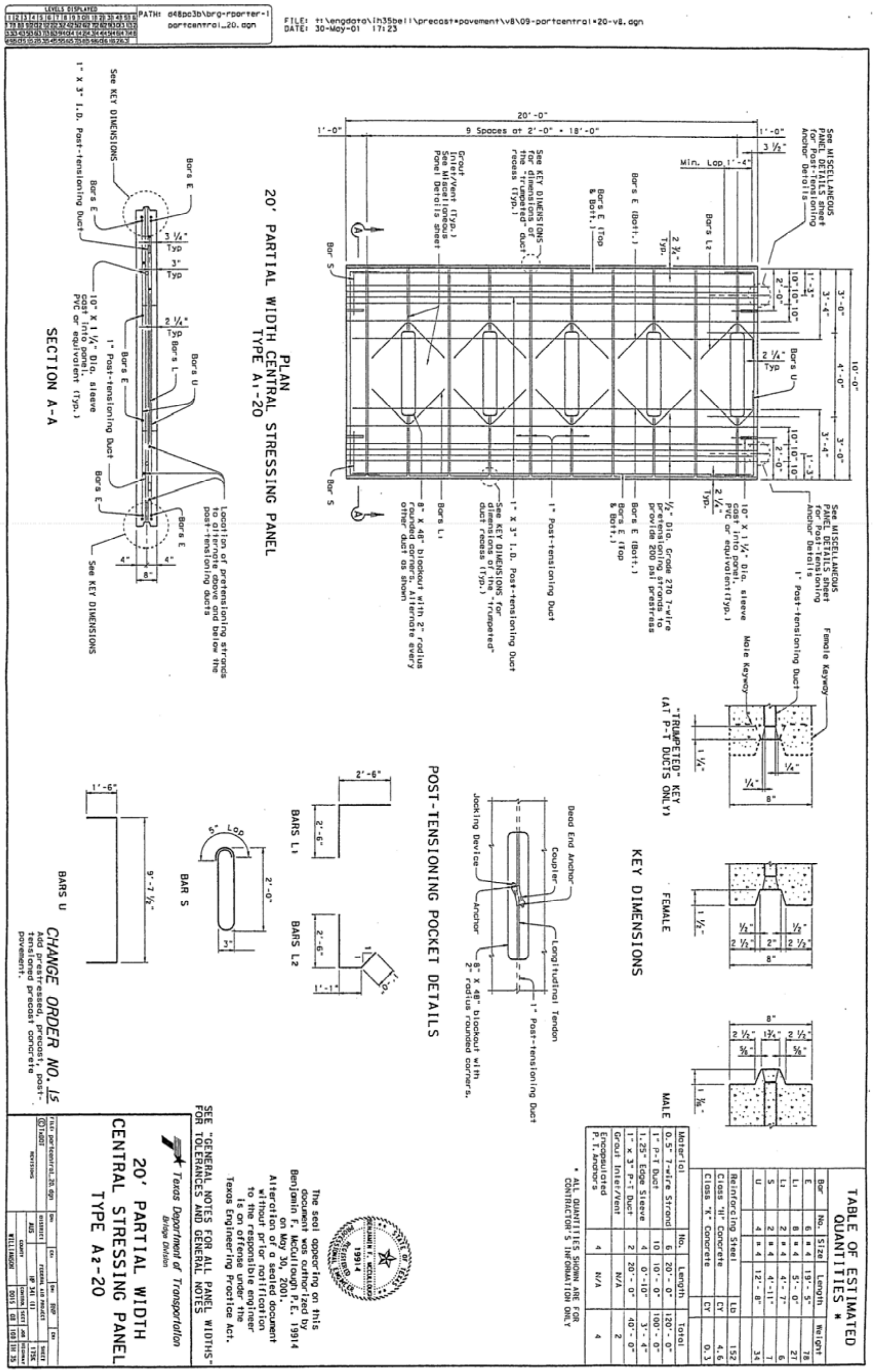
SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES

Texas Department of Transportation
Bridge Division

20' PARTIAL WIDTH CENTRAL STRESSING PANEL TYPE A1-20

CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.



PLAN
20' PARTIAL WIDTH CENTRAL STRESSING PANEL
TYPE A1-20
SECTION A-A
KEY DIMENSIONS
FEMALE
MALE
"TRUMPETED" KEY (AT P-T DUCTS ONLY)
POST-TENSIONING POCKET DETAILS
BARS L1
BARS L2
BAR S
BARS U
CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.
TABLE OF ESTIMATED QUANTITIES *
SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES
Texas Department of Transportation
Bridge Division
20' PARTIAL WIDTH CENTRAL STRESSING PANEL TYPE A2-20

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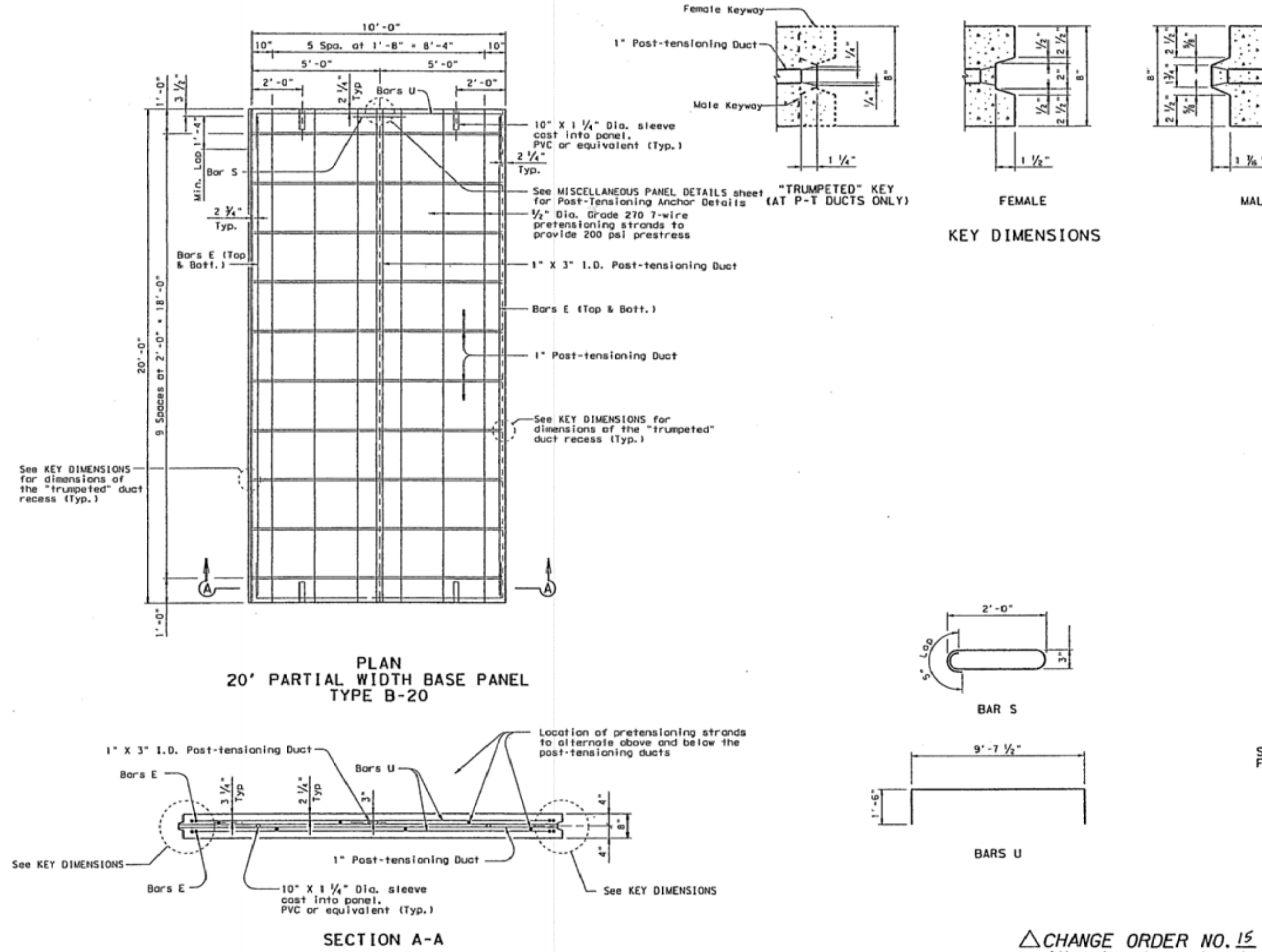

TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
E	4	# 4	19'- 5"	52
S	1	# 4	4'-11"	3
U	4	# 4	12'- 8"	34
Reinforcing Steel			Lb	89
Class "H" Concrete			CY	4.9

Material	No.	Length	Total
0.5" 7-wire Strand	6	20'- 0"	120'- 0"
1" P-T Duct	10	10'- 0"	100'- 0"
1.25" Edge Sleeve	4	0'-10"	3'- 4"
1" x 3" P-T Duct	1	20'- 0"	20'- 0"
Grout Inlet/Vent	1	N/A	1
Encapsulated P.T. Anchors	2	N/A	2

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SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES

△ CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.

Texas Department of Transportation
Bridge Division

20' PARTIAL WIDTH
BASE PANEL
TYPE B-20

FILE: partbase_20.dgn | DN: | CK: | DW: RNP | CK:
© TxDOT | DISTRICT: AUS | FEDERAL AID PROJECT: HP 341 (1) | SHEET: 175L
REVISIONS | COUNTY: WILLIAMSON | CONTROL: 0015 | SECT: 08 | JOB: 108 | HIGHWAY: IH 35

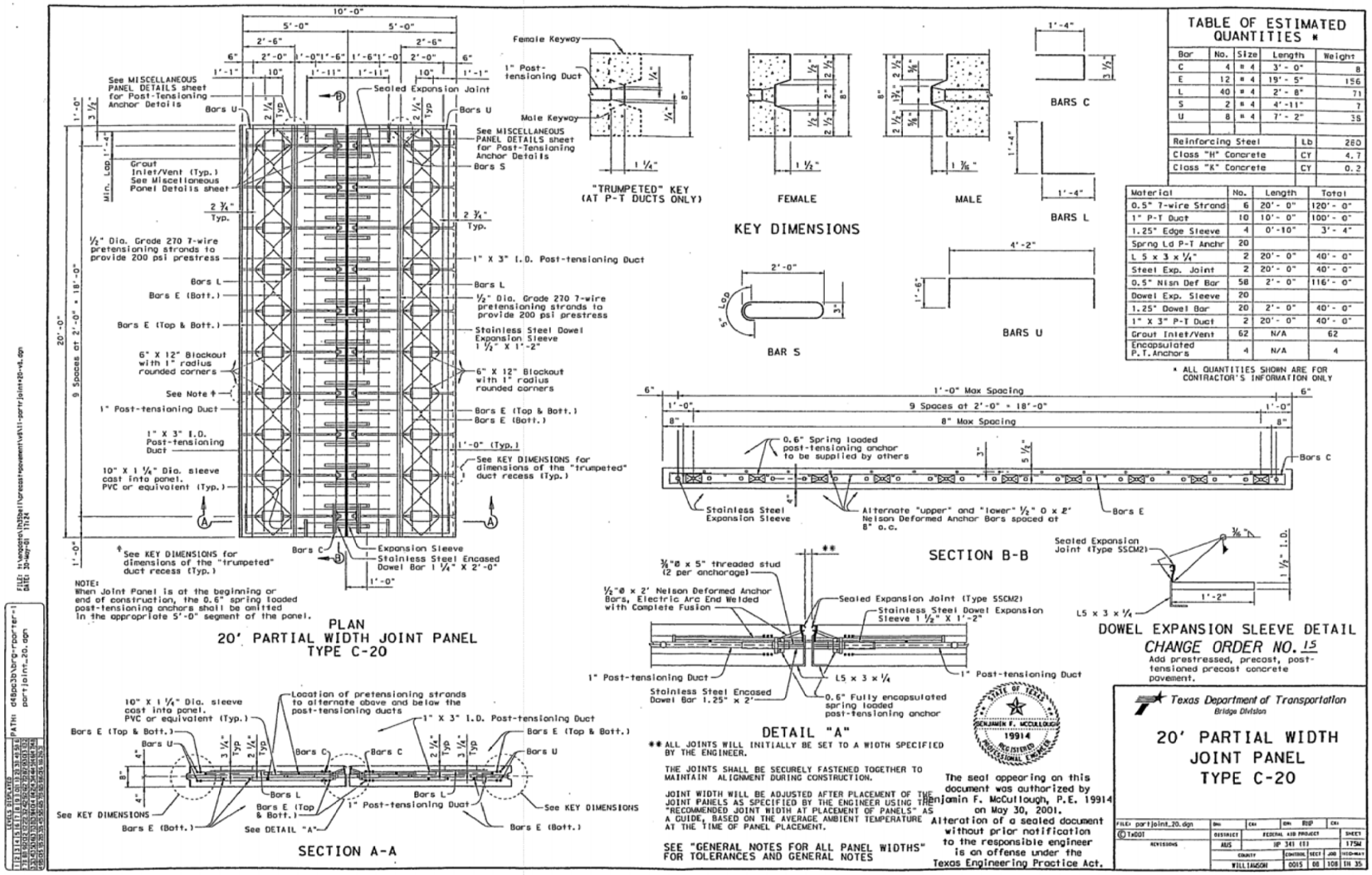
TABLE OF ESTIMATED QUANTITIES *
Bar | No. | Size | Length | Weight
C | 4 | # 4 | 3' - 0" | 8
E | 12 | # 4 | 19' - 5" | 156
L | 40 | # 4 | 2' - 8" | 71
S | 2 | # 4 | 4' -11" | 7
U | 8 | # 4 | 7' - 2" | 35
Reinforcing Steel | Lb | 280
Class "H" Concrete | CY | 4.7
Class "K" Concrete | CY | 0.2
Material | No. | Length | Total
0.5" 7-wire Strand | 6 | 20' - 0" | 120' - 0"
1" P-T Duct | 10 | 10' - 0" | 100' - 0"
1.25" Edge Sleeve | 4 | 0' -10" | 3' - 4"
Sprng Ld P-T Anchr | 20
L 5 x 3 x 1/4" | 2 | 20' - 0" | 40' - 0"
Steel Exp. Joint | 2 | 20' - 0" | 40' - 0"
0.5" Nlsn Def Bar | 58 | 2' - 0" | 116' - 0"
Dowel Exp. Sleeve | 20
1.25" Dowel Bar | 20 | 2' - 0" | 40' - 0"
1" X 3" P-T Duct | 2 | 20' - 0" | 40' - 0"
Grout Inlet/Vent | 62 | N/A | 62
Encapsulated P.T. Anchors | 4 | N/A | 4
* ALL QUANTITIES SHOWN ARE FOR CONTRACTOR'S INFORMATION ONLY
PLAN
20' PARTIAL WIDTH JOINT PANEL
TYPE C-20
NOTE:
When Joint Panel is at the beginning or end of construction, the 0.6" spring loaded post-tensioning anchors shall be omitted in the appropriate 5'-0" segment of the panel.
See KEY DIMENSIONS for dimensions of the "trumpeted" duct recess (Typ.)
"TRUMPETED" KEY (AT P-T DUCTS ONLY)
FEMALE
MALE
KEY DIMENSIONS
BARS C
BARS L
BARS U
BAR S
SECTION B-B
DETAIL "A"
SECTION A-A
DOWEL EXPANSION SLEEVE DETAIL
CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.
** ALL JOINTS WILL INITIALLY BE SET TO A WIDTH SPECIFIED BY THE ENGINEER.
THE JOINTS SHALL BE SECURELY FASTENED TOGETHER TO MAINTAIN ALIGNMENT DURING CONSTRUCTION.
JOINT WIDTH WILL BE ADJUSTED AFTER PLACEMENT OF THE JOINT PANELS AS SPECIFIED BY THE ENGINEER USING THE "RECOMMENDED JOINT WIDTH AT PLACEMENT OF PANELS" AS A GUIDE, BASED ON THE AVERAGE AMBIENT TEMPERATURE AT THE TIME OF PANEL PLACEMENT.
SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES
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Texas Department of Transportation
Bridge Division
20' PARTIAL WIDTH
JOINT PANEL
TYPE C-20
FILE: partjoint_20.dgn
PATH: d48pc3b\brg-rporter-l partjoint_20.dgn

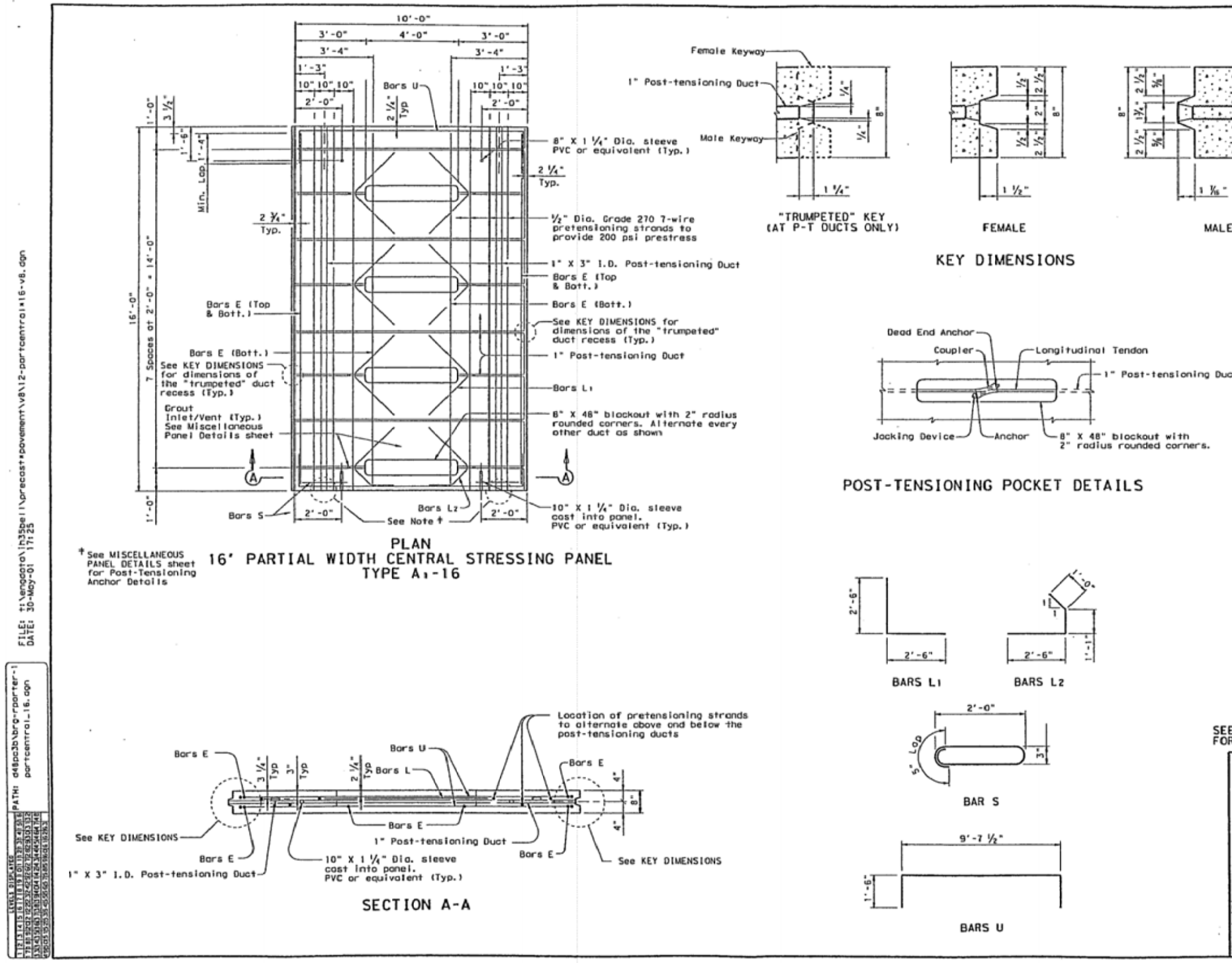

TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
E	6	# 4	15'- 5"	62
L1	6	# 4	5'- 0"	20
L2	2	# 4	4'- 7"	6
S	2	# 4	4'-11"	7
U	4	# 4	12'- 8"	34
Reinforcing Steel			Lb	129
Class "H" Concrete			CY	3.7
Class "K" Concrete			CY	0.3

Material	No.	Length	Total
0.5" 7-wire Strand	6	16'- 0"	96'- 0"
1" P-T Duct	8	10'- 0"	80'- 0"
1.25" Edge Sleeve	2	0'-10"	1'- 8"
1" x 3" P-T Duct	2	16'- 0"	32'- 0"
1.25" Vert. Sleeve	2	0'- 8"	1'- 4"
Grout Inlet/Vent	10	N/A	10
Encapsulated P.T. Anchors	4	N/A	4

* ALL QUANTITIES SHOWN ARE FOR CONTRACTOR'S INFORMATION ONLY

STATE OF TEXAS
BENJAMIN F. MCCULLOUGH
19914
REGISTERED PROFESSIONAL ENGINEER

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CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.

SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES

Texas Department of Transportation
Bridge Division

16' PARTIAL WIDTH CENTRAL STRESSING PANEL TYPE A1-16

FILE: partcentral_16.dgn	DN:	CK:	DW: RUP	CK:
© TxDOT	DISTRICT	FEDERAL AID PROJECT		SHEET
REVISIONS	AUS	IP 341 (1)		175N
	COUNTY	CONTROL / SECT / JOB		HIGHWAY
	WILLIAMSON	0015 / 08 / 108		IH 35

PATH: d48pc3b\brg-rporter-1 partcentral_16.dgn

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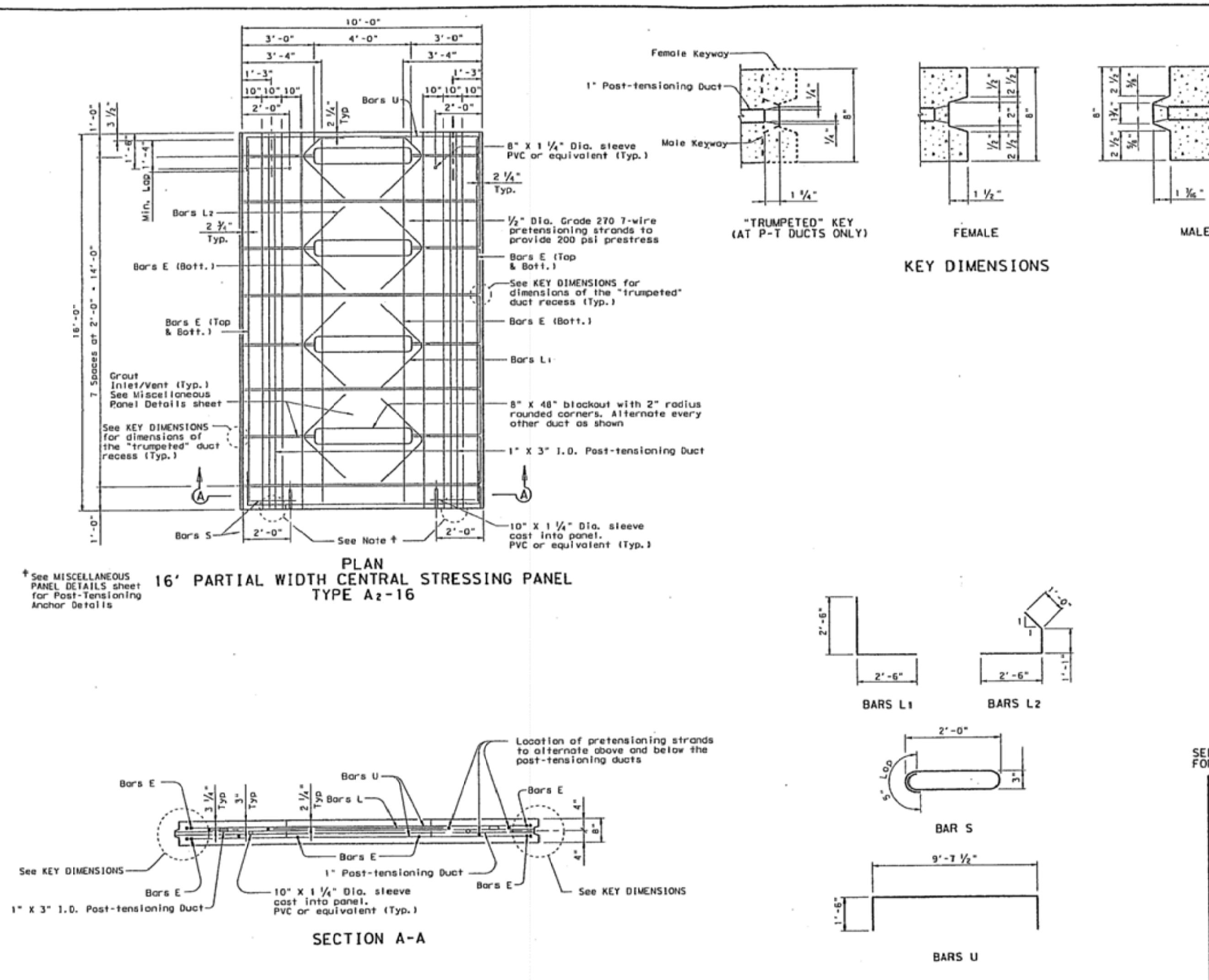

TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
E	6	# 4	15'- 5"	62
L1	6	# 4	5'- 0"	20
L2	2	# 4	4'- 7"	6
S	2	# 4	4'-11"	7
U	4	# 4	12'- 8"	34

Reinforcing Steel	Lb	129
Class "H" Concrete	CY	3.7
Class "K" Concrete	CY	0.3

Material	No.	Length	Total
0.5" 7-wire Strand	6	16'- 0"	96'- 0"
1" P-T Duct	8	10'- 0"	80'- 0"
1.25" Edge Sleeve	2	0'-10"	1'- 8"
1" x 3" P-T Duct	2	16'- 0"	32'- 0"
1.25" Vert. Sleeve	2	0'- 8"	1'- 4"
Grout Inlet/Vent	10	N/A	10
Encapsulated P.T. Anchors	4	N/A	4

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CHANGE ORDER NO. 15
Add prestressed, precast, post-tensioned precast concrete pavement.

SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES

Texas Department of Transportation
Bridge Division

16' PARTIAL WIDTH CENTRAL STRESSING PANEL TYPE A2-16

FILE: partcentral_16.dgn	DN:	CK:	DW: RNP	CK:
© TxDOT	DISTRICT AUS	FEDERAL AID PROJECT HP 341 (1)		SHEET 1750
REVISIONS	COUNTY WILLIAMSON	CONTROL 0015	SECT 08	JOB 108 HIGHWAY IH 35

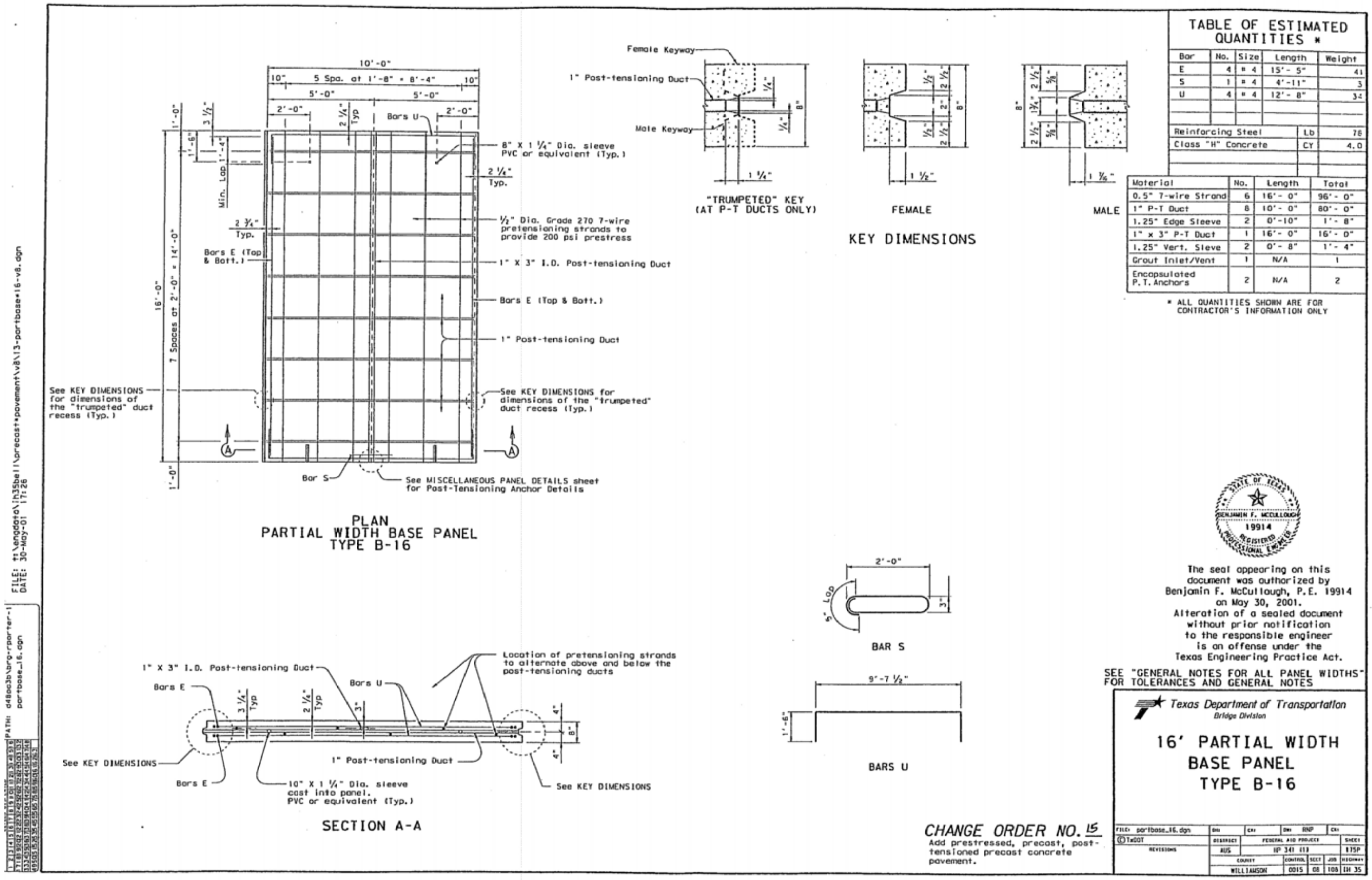

TABLE OF ESTIMATED QUANTITIES *

Bar	No.	Size	Length	Weight
E	4	# 4	15'- 5"	41
S	1	# 4	4'-11"	3
U	4	# 4	12'- 8"	34
Reinforcing Steel			Lb	78
Class "H" Concrete			CY	4.0

Material	No.	Length	Total
0.5" 7-wire Strand	6	16'- 0"	96'- 0"
1" P-T Duct	8	10'- 0"	80'- 0"
1.25" Edge Sleeve	2	0'-10"	1'- 8"
1" x 3" P-T Duct	1	16'- 0"	16'- 0"
1.25" Vert. Sleve	2	0'- 8"	1'- 4"
Grout Inlet/Vent	1	N/A	1
Encapsulated P.T. Anchors	2	N/A	2

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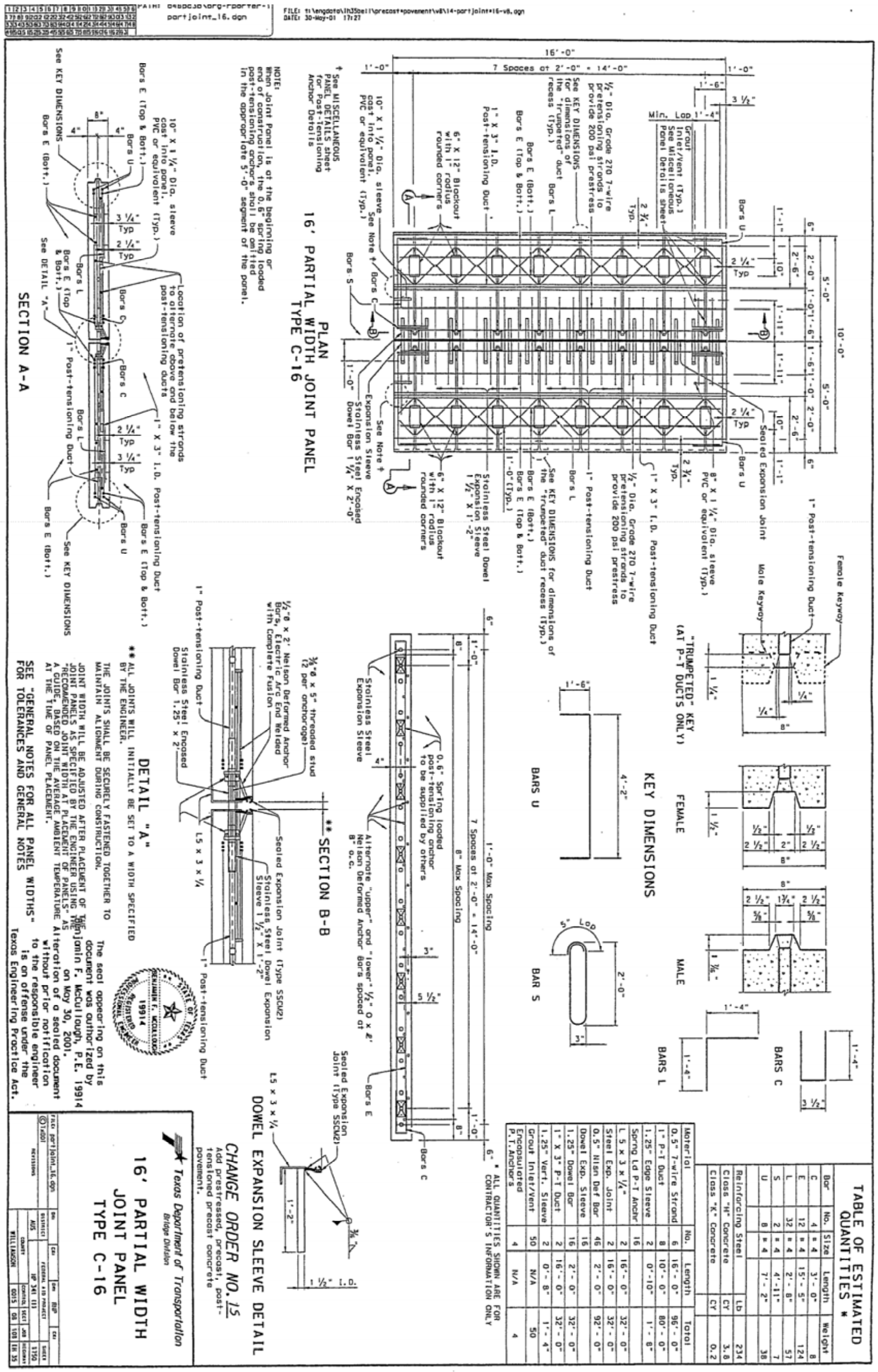
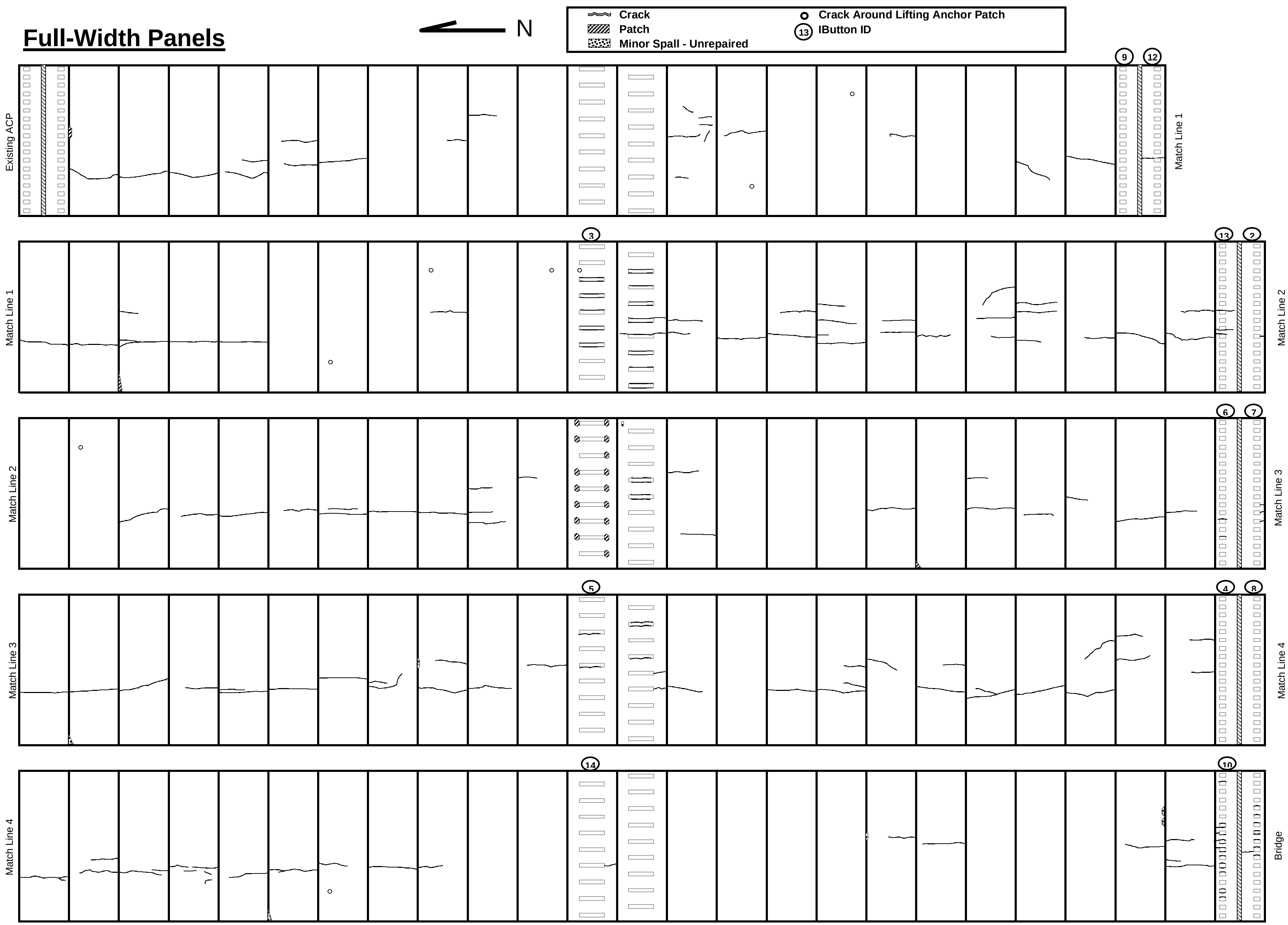
SEE "GENERAL NOTES FOR ALL PANEL WIDTHS" FOR TOLERANCES AND GENERAL NOTES

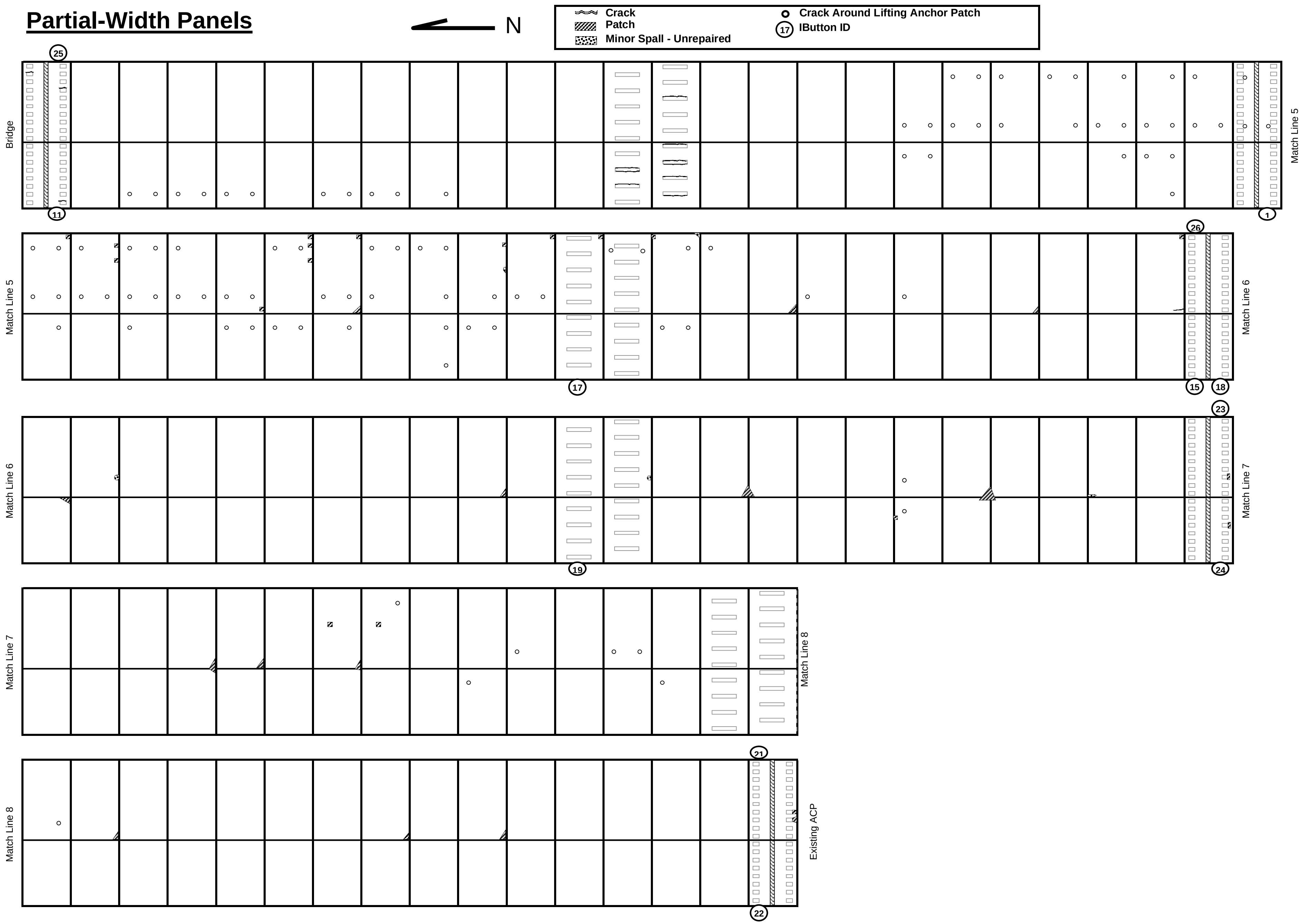

16' PARTIAL WIDTH JOINT PANEL
TYPE C-16
PLAN
SECTION A-A
DETAIL "A"
SECTION B-B
KEY DIMENSIONS
TABLE OF ESTIMATED QUANTITIES *
DOWEL EXPANSION SLEEVE DETAIL
CHANGE ORDER NO. 15
Texas Department of Transportation

Post-Construction Condition Survey and Temperature Instrumentation Locations

Full-Width Panels



Partial-Width Panels



IButton ID	Serial Number		
	Top	*Middle*	*Bottom*
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2	4B34C00000CFE921	FE34C00000D05121	8A34C00000D18D21
3	C534C00000D20821	4A34C00000D06221	5034C00000CFAB21
4	E534C00000D08421	0334C00000CFF721	D134C00000D08421
5	B134C00000D1E621	0234C00000D07C21	8F34C00000CF3921
6	7B34C00000D05621	6334C00000D08221	2E34C00000D20D21
7	2134C00000D0C321	F834C00000CF6A21	4034C00000CF3321
8		2234C00000D05521	
9		5034C00000D1BC21	
10		DA34C00000D14721	
11		9A34C00000D00C21	
12		AC34C00000D19121	
13		BB34C00000CFAE21	
14		5334C00000D03321	
Partial-Width Panels			
15	F234C00000D12221	CF34C00000D06521	9534C00000D1E921
16	E334C00000D21421	E234C00000CFAD21	1634C00000D1DB21
17	6B34C00000D1D721	8A34C00000CF9A21	5834C00000CFE721
18	5934C00000D17521	4B34C00000CEF021	9F34C00000D0AF21
19	C634C00000CFA221	F834C00000D17D21	4A34C00000D17B21
20	5134C00000D13921	F934C00000CEF621	E334C00000D13F21
21		CE34C00000CFEE21	
22		5B34C00000CF7121	
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27		DE34C00000D16121	

Minutes from Post-Construction Meeting

PRECAST PAVEMENT IMPLEMENTATION ON IH 35
FRONTAGE ROAD NEAR GEORGETOWN, TEXAS

Post-Construction Meeting

MARCH 18, 2002

Attendees:

Mark Leintz	*Granite Construction*
Kurt Knebel	*Granite Construction*
Lyle R. Clark	*Granite Construction*
Ricky Morris	*Granite Construction*
Frank O'Malley	*Granite Construction*
Burson Patton	*Texas Concrete Company*
Butch George	*Texas Concrete Company*
Kris Kriofske	*Dywidag Systems Int'l*
Joe Harrison	*General Technologies Inc.*
Bill Garbade	*TxDOT – Austin District Engineer*
Mark Herber	*TxDOT – Georgetown Area Office*
David Boone	*TxDOT – Georgetown Area Office*
Jeff Tolson	*TxDOT – Georgetown Area Office*
Gary Graham	*TxDOT – Pavement Design*
Joe Roche	*TxDOT – CST/M&P*
Andy Naranjo	*TxDOT – CST/M&P*
George Vogt	*TxDOT – CST/M&P – Victoria*
Frank McCullough	*Center for Transportation Research*
David Merritt	*Center for Transportation Research*
Ned Burns	*CTR/Ferguson Structural Lab*

Discussion:

I. Panel Fabrication
   A. Production
      1. Longer panels (36 ft) preferred from a production standpoint – size of crane required to place panels dictates how long panels can be.
      2. 10 ft panel width limit for transportation.
   B. Mid-slab cracking in forms
      1. May not be possible to control for long precast panels.
      2. Should not cause a problem with durability – pretensioning and mild steel should prevent cracks from opening.
      3. Possibility of casting panels in the afternoon so that heat generation in the panels is not so high – may reduce likelihood of cracking. However, this requires attention to cooling the forms, cooling the aggregate stockpiles, and will result in a later release of prestress, which will slow production.
   C. Armored joint tolerances

1. Tolerances need to be specified for the steel fabricator so the precast plant does not have to straighten the joints.
2. Minimize amount of welding on armored joint
3. Possibility of using a more rigid section for the seal receiver.
4. Possibility of using a preformed metal joint (PMJ) containing a solid expansion material.

D. Panel Tolerances
1. Tighter tolerances are needed.
2. Possibility of a squareness tolerance to ensure ends of panels are square with edges.

E. Removal from forms
1. Breakage reduced by leaving panels in forms longer (overnight as opposed to removal same day as casting).
2. Chamfer along bottom edge of panels may eliminate breakage along bottom edge of panels.

F. Curing
1. No problems experienced with curing.
2. Intermediate curing compound (Confilm) is essential when casting in hot and dry weather.

G. Panel Storage (creep)
1. Slight sag noticed in some of the older panels when they were delivered to the site.
2. Panels supported at approximately 0.21L from ends of the panels (two support points) during storage.
3. Difficult to get more than two support points perfectly level in the storage area – additional supports may cause more problems.
4. Possibility of "H" support configuration.

H. Panel Repairs
1. Specifications needed for repairs to panels at precast plant – i.e. qualifications for repair, depth of repair, materials for repair, grounds for rejection, etc.

II. Asphalt Leveling-course

A. Crowns/Sags
1. Too much traffic was allowed onto the leveling course (south end) after placement, and may have caused unevenness observed.
2. Difficult to get longitudinal joint between adjacent asphalt sections perfectly level.

B. Future Applications
1. Difficult to get a paving train into a short section to get AC placed to the required tolerances.
2. Problem with putting equipment/construction vehicles on a thin layer of hot asphalt – may result in indentations/unevenness.
3. Possibility of incorporating under-slab grouting.

III. Panel Placement

A. Keyway Dimensions

1. Keyway dimensions should be modified so that the fit is not as tight – will help with getting the joints tighter when assembled and provide flexibility with keeping the adjacent panels square and on-line.
2. Epoxy will compensate for some “slop” in keyways – still sealing joints while allowing for adjustment to keep the panels on-line.

B. Panel Offsetting
   1. Inserting shims in the joint to gap one side proved more desirable than offsetting the panels.
   2. Panel offsetting should only be considered as a last resort and if larger ducts, which allow for misalignment, are used.
   3. Possibility of incorporating an “alignment tab” cast into the panels at the centerline to ensure alignment of panels.

C. Full width panel cracking (asphalt crown)
   1. Difficult to control cracking with a slight crown in the AC leveling course.
   2. Cracks should not open up because of pretensioning.
   3. Cracks will not be sealed, but will be monitored over the life of the pavement.

D. Partial-width panel alignment (squareness)
   1. Consider incorporation of a squareness tolerance for panel fabrication.

E. Longitudinal joint load transfer (partial-width panels)
   1. Possibility of incorporating dowel bars in one panel with an oversized slot (to be grouted after placement) in the adjacent panel.
   2. Possibility of a “ship-lap” type joint between panels.
   3. Possibility of incorporating separate ducts in the panels for inserting rebar/dowel bars across the joint after placement. Rebar/dowel bar would be grouted in-place across the joint.

F. Temporary Stressing
   1. Temporary stressing resulted in tighter joints during placement of the partial width panels.
   2. Special pockets needed in the joint panels to accommodate the stressing ram for temporary stressing so that rams do not need to be moved.
   3. Provision for anchor plates against the keyways of each panel for temporary stressing is needed to prevent damage to the keyways.

G. Expansion Joint width
   1. Expansion joint should be shipped to the precast yard as a single unit, tack welded together at a specified joint width.
   2. Recommended joint width table (based upon approximate ambient temperatures during panel placement) should be used throughout construction, adjusting joint width when necessary.
   3. Joint width should be easy to adjust in the field.

H. Epoxy
   1. Epoxy will tend to take up small amounts of “slop” in the joint as the panels are placed.
   2. Post-tensioning strands should be fed through the ducts and post-tensioning completed while the epoxy is fresh.
   3. Better method for application of epoxy is needed to get a more uniform application.
   4. Possibility of using epoxy with an even longer pot life.

IV. Post-Tensioning
   A. Blind Anchors/Strand Insertion
      1. The use of “blind” anchors should be eliminated.
      2. Incorporate a way to ensure the post-tensioning strands are fully anchored if spring-loaded anchors or standard dead-end anchors are used – observation pocket, etc.
      3. Possibility of using a looped tendon system at the expansion joints, as opposed to dead-end anchors. 3 ft minimum radius using galvanized steel (schedule 40/80) duct, welded/tied to the expansion joint. 2 – 4 strand capacity. Dogbone couplers in the stressing pockets with larger pocket dimensions for the stressing ram.
   B. Duct size
      1. Larger duct size needed for 0.6” diameter strand – more flexibility with panel misalignment, easier to push strands by hand, easier to grout.
      2. Possibility of using a new type of duct with channels in the top, bottom, and sides of the duct to facilitate grouting.
   C. Panel offset/Duct alignment
      1. Panel offset should be minimized and larger ducts should be used to prevent problems with strand insertion.
   D. Transverse strand insertion
      1. A different layout is needed for the transverse tendons to make transverse strand insertion easier – i.e. longer diverter length.
      2. Possibility of separate transverse ducts for each strand.

V. Grouting
   A. Tightness of panel joints/Keyway dimensions
      1. Keyway dimensions should be adjusted to the panels will fit together tighter.
   B. Sealing panel joints (transverse and longitudinal)
      1. The use of a rubber or foam gasket material should be considered to get a better seal around both transverse and longitudinal joints.
   C. Duct size
      1. Larger duct size should be used to facilitate grouting.
   D. Grout vent opening
      1. New duct style, which incorporates channels along the length of the duct, will facilitate grouting, even if the strand is blocking the opening at the grout vent.
      2. Better Ts are needed for the gout vents so that the duct does not need to be drilled – should be available for the new duct style.
      3. Grout vents in the top of the panels should be minimized to simplify fabrication and improve ride quality of the finished pavement.
      4. Possibility of a “network” of grout vents for grouting multiple ducts from a single vent.

VI. Finished Pavement
   A. Ride Quality
      1. Finished ride quality appears to be just as good as the cast-in-place post-tensioned pavement near West, Texas.
   B. Expansion Joint width

1. Expansion joint width should be adjustable in the field and set according to the recommended joint width table in the plans.

C. Lifting devices
   1. Screw-type lifting inserts are available, but are more time consuming to use and will slow down production/construction.
   2. Screw-type lifting devices would greatly reduce patching required on the finished pavement.

VII. Supplier's/Contractor's Overall Perspectives and Recommendations

A. Precast Supplier
   1. Joint panels proved difficult to set up and cast.
   2. Too many gout vents/lifting devices are protruding from the panel surface – slows down production and required chipping into the surface of the panels.
   3. Difficult to get enough trucks for transportation.
   4. May be difficult to handle panels longer than 36 ft – borderline on stresses in the panels during handling.
   5. Possibility of stockpiling "standard" panels under a separate contract, then using the panels for different projects as they arise.

B. General Contractor
   1. Base preparation is the main consideration for rapid reconstruction projects/projects constructed under strict time constraints and staging area restrictions.
   2. End treatment (end of slab transition to existing pavement) needs to be considered for removal and replacement applications.
   3. A more streamlined construction process is needed for truly expedited construction.

C. Design
   1. Need to address issues associated with intersection construction, etc.