Constructibility of Prestressed Concrete Panels for Use at Skewed Expansion Joints

by

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Abstract

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The Texas Department of Transportation (TxDOT) has typically used a thickened, cast-in-place slab in lieu of any special supports or diaphragms at concrete bridge deck expansion joints for many years. A new detail to replace the thickened cast-in-place slab was developed was developed under TxDOT research project 0-44118. The standard prestressed panels typically used at interior portions of the bridge are continued to the expansion joints in zero degree skew bridge decks. The new detail improved construction speed, safety, and economy.

The primary goals of the research project reported here were to evaluate the feasibility of producing trapezoidal-shaped prestressed concrete panels as well as address construction related issues so that use of the new panel detail can be extended to include skewed expansion joints. A total of eight trapezoidal panels were fabricated using two

different prestressing layouts and various geometries. In two panels, a flared prestressing strand pattern was used while in the other six panels, an arrangement with the prestressing strands parallel to the skewed end of the panel was used. A 45 degree skew angle was used in four of the panels and a 30 degree skew angle was used in the other four panels. A short edge length of 45 in. or 60 in. was used for the trapezoidal panels and all panels were 4 in. thick and 9 ft. 6 in. wide.

The results of the research project demonstrate that producing trapezoidal prestressed panels can be economical while accommodating a wide range of geometries. The research project showed that trapezoidal prestressed panels provide a feasible alternative to stay-in-place formwork at skewed expansion joints.

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CHAPTER 1

Introduction

1.1 INTRODUCTION

The bridge construction industry in the state of Texas has become increasingly efficient due to the speed and economy of utilizing precast concrete. One of the key elements is a precast concrete panel used as stay-in-place formwork for concrete bridge decks. The panels are easier and faster to install than conventional formwork thus increasing construction speed. However, the current Texas Department of Transportation (TxDOT) standard does not permit the use of panels at expansion joints because of the unsupported free end. Moreover, the standard panels are rectangular and therefore cannot be used with skewed expansion joints. TxDOT has typically used a thickened, cast-in-place slab in lieu of any special supports or diaphragms at such ends for many years.

1.2 BACKGROUND

Developed by TxDOT in 1963, precast panels were seldom used in Texas until changes to the bridge specification in 1983 made them a more viable alternative. This forming method is used to build approximately 85% of all bridges in Texas largely in part due to the increased speed of construction, cost savings, and increased safety. With half of the deck precast, only one mat of deck reinforcing must be tied and the volume of the cast-in-place concrete deck is roughly halved. A typical construction sequence is illustrated in Figures 1.1 and 1.2. The concrete girders are placed on the supports in the longitudinal direction with typical spacing between 6 ft. and 10 ft. (Figure 1.1). Next, the rectangular, 4 in. thick panels are placed on the girder flanges spanning between adjacent girders (Figure 1.2).

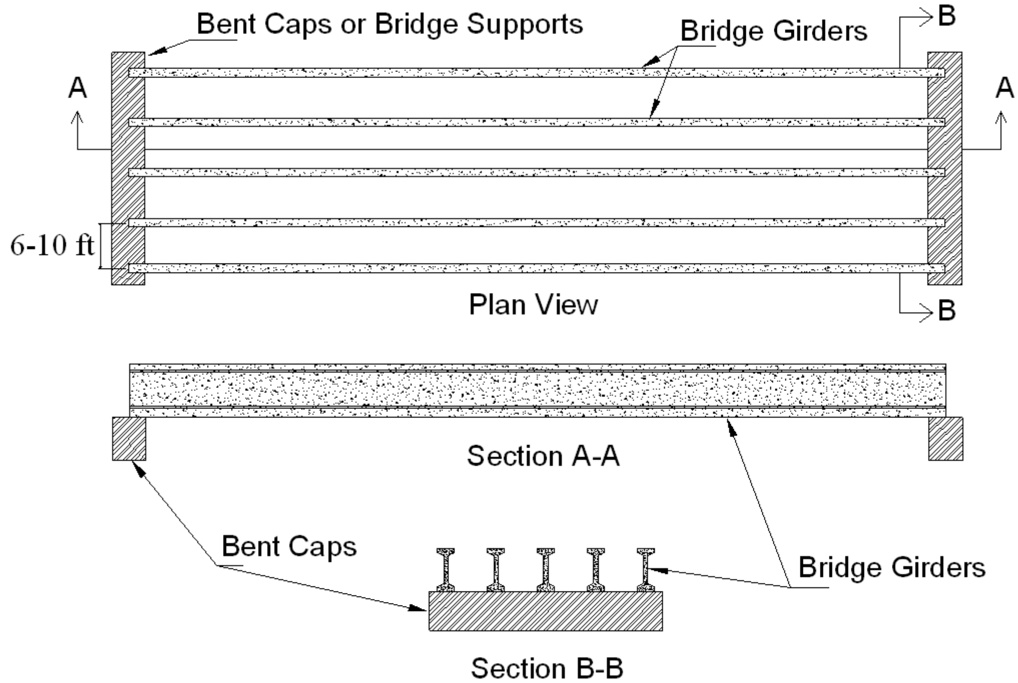


Figure 1.1: Typical bridge construction prior to placing bridge deck (Agnew 2007)

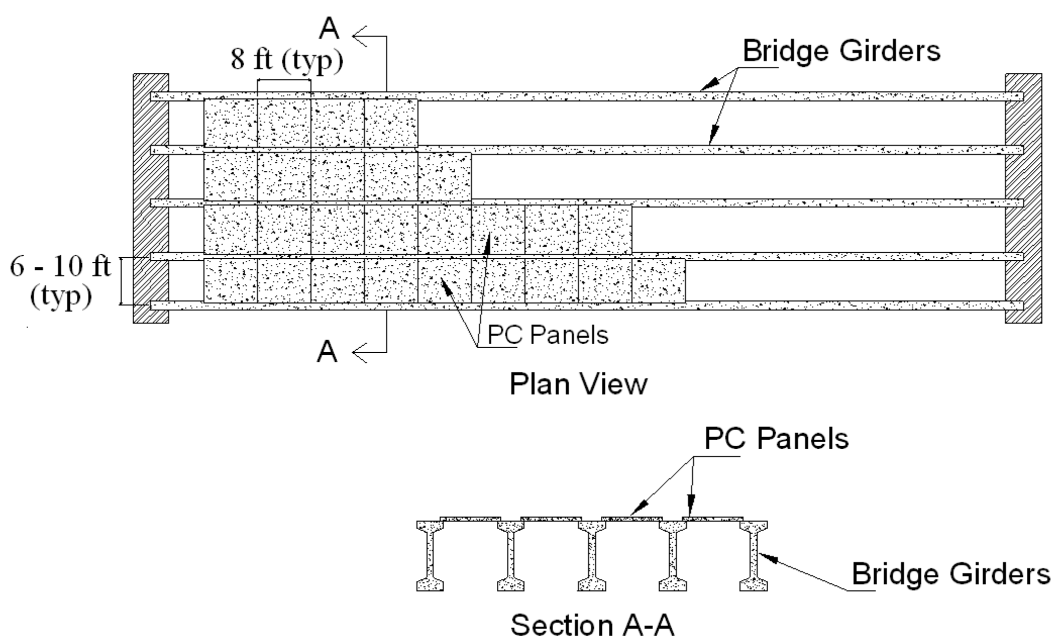


Figure 1.2: Typical bridge construction during placement of prestressed panels (Agnew 2007)

To allow the bridge structure to expand and contract with thermal fluctuation, small transverse breaks in the superstructure continuity, called expansion joints, are placed along the bridge. In the vicinity of these expansion joints, typically over a support, a full depth cast-in-place slab is used instead of precast panels. To achieve this, traditional formwork is erected to accommodate the expansion joint hardware and the skew angle of the joint. The panels themselves serve as formwork for the remainder of the interior of the bridge where ironworkers can then place a mat of reinforcing steel for the 4 in. thick concrete cast-in-place topping slab. The concrete topping slab combines with the panels to form an 8 in. thick composite bridge deck. Figure 1.3 shows the mat of reinforcing steel placed above the precast panels prior to casting the topping slab on a TxDOT bridge construction site.

Figure 1.3: Panels and reinforcing steel prior to casting topping slab (Agnew 2007)

At the expansion joint locations where conventional formwork is used, TxDOT currently uses the I-Beam Thickened Slab (IBTS) detail. The 10 in. thick, 4 ft. wide slab provides additional stiffness to the unsupported end of the deck, eliminating the need for diaphragms or additional supports. A cross-section drawing of the IBTS detail is shown in Figure 1.4.

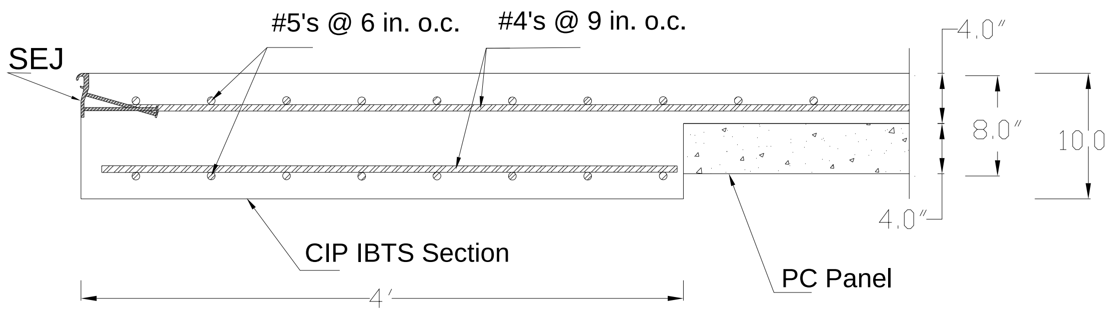


Figure 1.4: Cross-section of IBTS detail (Agnew 2007)

The conventional formwork for the IBTS detail must be installed prior to placing the topping slab and removed after curing (Figure 1.5). This formwork can become particularly difficult to construct in locations where the concrete girders are skewed with respect to the supports. Additionally, safety wires and temporary bridges must be erected to prevent workers and from falling through the portions of the deck where panels are not present. Figure 1.6 shows a TxDOT bridge construction site with a skewed expansion joint where formwork is complicated and an unsafe work environment exists until the formwork is completed. These processes can become unsafe, costly and time consuming, especially if the bridges are at high elevations or above water or traffic.

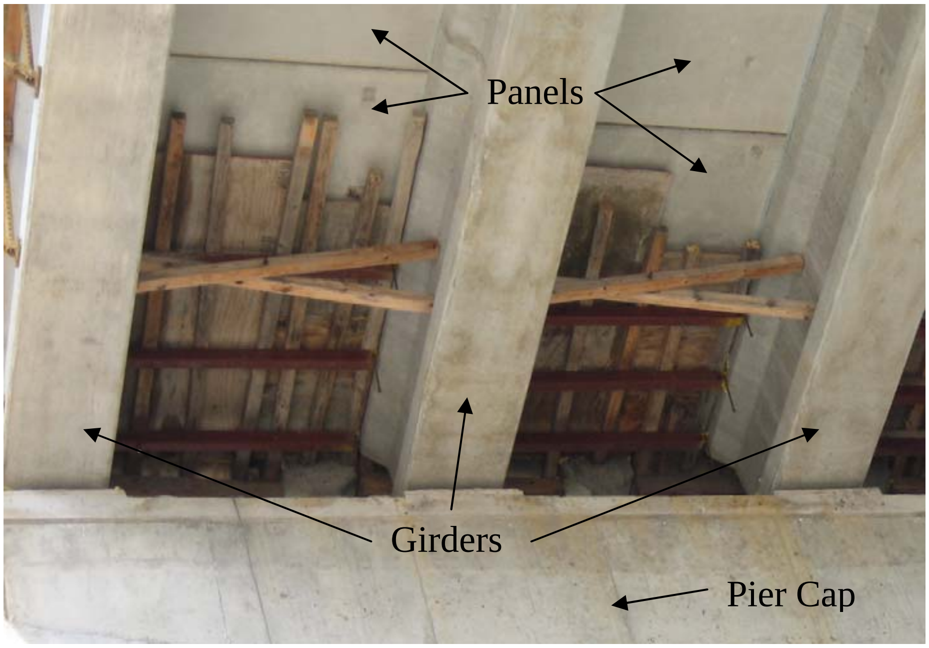


Figure 1.5: Temporary formwork erected for IBTS detail

Figure 1.6: Complex geometry and hazardous work environment at an unfinished skewed expansion joint (Agnew 2007)

The concerns regarding the IBTS detail influenced TxDOT to sponsor two research projects at the Ferguson Structural Engineering Laboratory at the University of Texas at Austin. In the first research project (0-4418) the performance of the IBTS detail was compared with a proposed Uniform Thickness Slab End (UTSE) detail for both 0 and 45 degree expansion joints. Additionally, the behavior and constructability of precast panels used at a non-skew expansion joints was investigated. The precast panel alternative presents many advantages for the contractor, including time and safety of the workforce. Results from these tests are discussed in Chapter 2. Based on recommendations from project 0-4418, a second research project (0-5367), a portion of which is the subject of this thesis, was awarded to further study the performance of precast panels at expansion joints.

1.3 OBJECTIVES AND SCOPE

The goals are to develop and evaluate a precast concrete panel detail for use at expansion joints for both skew and non-skew bridges. In the first phase of this project, 4 full-scale test specimens were constructed to asses the fatigue behavior of the panel detail under service and design loads at zero degree skew expansion joints. Results from these tests were reported by Agnew (2007). In the second phase of the project, trapezoidal-shaped panels with 30 and 45 degree skews were designed, constructed, and tested both statically and in fatigue. The practicality and constructibility of trapezoidal-shaped prestressed concrete panels for construction of bridge decks at skewed expansion joints is the focus of this thesis. The ability to produce trapezoidal panels economically was assessed and issues related to panel installation were examined. The panels produced during the study were used for full-scale composite bridge deck test specimens. The structural behavior of the trapezoidal panels is reported in Boswell (2008).

CHAPTER 2

Literature Review

2.1 INTRODUCTION

Since the focus of the study is the practicality and constructibility of trapezoidal-shaped panels for construction of bridge decks at expansion joints, research studies related to slab end details and skewed slabs have been reviewed and are presented in this chapter. Other subjects related to precast concrete panels, including prestressing strand development length, strand extensions, shear transfer, composite behavior, arching action, fatigue, and failure mechanisms, can also be found in these research studies, but will not be discussed.

2.2 RECENT TESTS OF EXPANSION JOINT BEHAVIOR

In recent years, TxDOT has sponsored research projects to investigate the performance of their current detail at expansion joints. Additionally, research has been undertaken to create new details with the goal of making bridge construction more economical. In the following discussions, research studies from various end detail tests are presented.

2.2.1 TxDOT Project 0-4418

In this research project, the behavior of TxDOT's IBTS detail was investigated. Project 0-4118 was initiated because of concerns regarding increased truck traffic on highways in Texas as a result of the North American Free Trade Agreement as well as TxDOT's recent 25% increase in bridge deck design loads with the implementation of

AASHTO LRFD Specification. The behavior of two alternative end details developed to potentially reduce bridge construction costs was studied.

2.2.1.1 Ryan (2003)

To investigate the ultimate capacity of the current TxDOT IBTS detail for slab ends of bridge decks, Ryan constructed and tested a full-scale bridge deck with a zero degree skew. The IBTS detail was used at one end and a Uniform Thickness Slab End (UTSE) detail was used at the other end as shown in Figure 2.4. The UTSE detail offered an alternative that would improve construction economy. The test specimen was a 32 ft. by 18 ft. composite bridge deck consisting of three spans with variable spacing and the two different end details. The entire bridge deck was cast-in-place.

The standard drawings for the IBTS detail are shown in Figures 2.1 and 2.2. The IBTS detail has an increased thickness of 10 in. to account for lack of end diaphragms to maintain stiffness. The thickened slab extends 4 ft. from the expansion joint and then transitions back to 8 in. where panels are typically used. Since the focus was on the slab behavior at expansion joints, however, panels were not used in the interior portions of this test specimen.

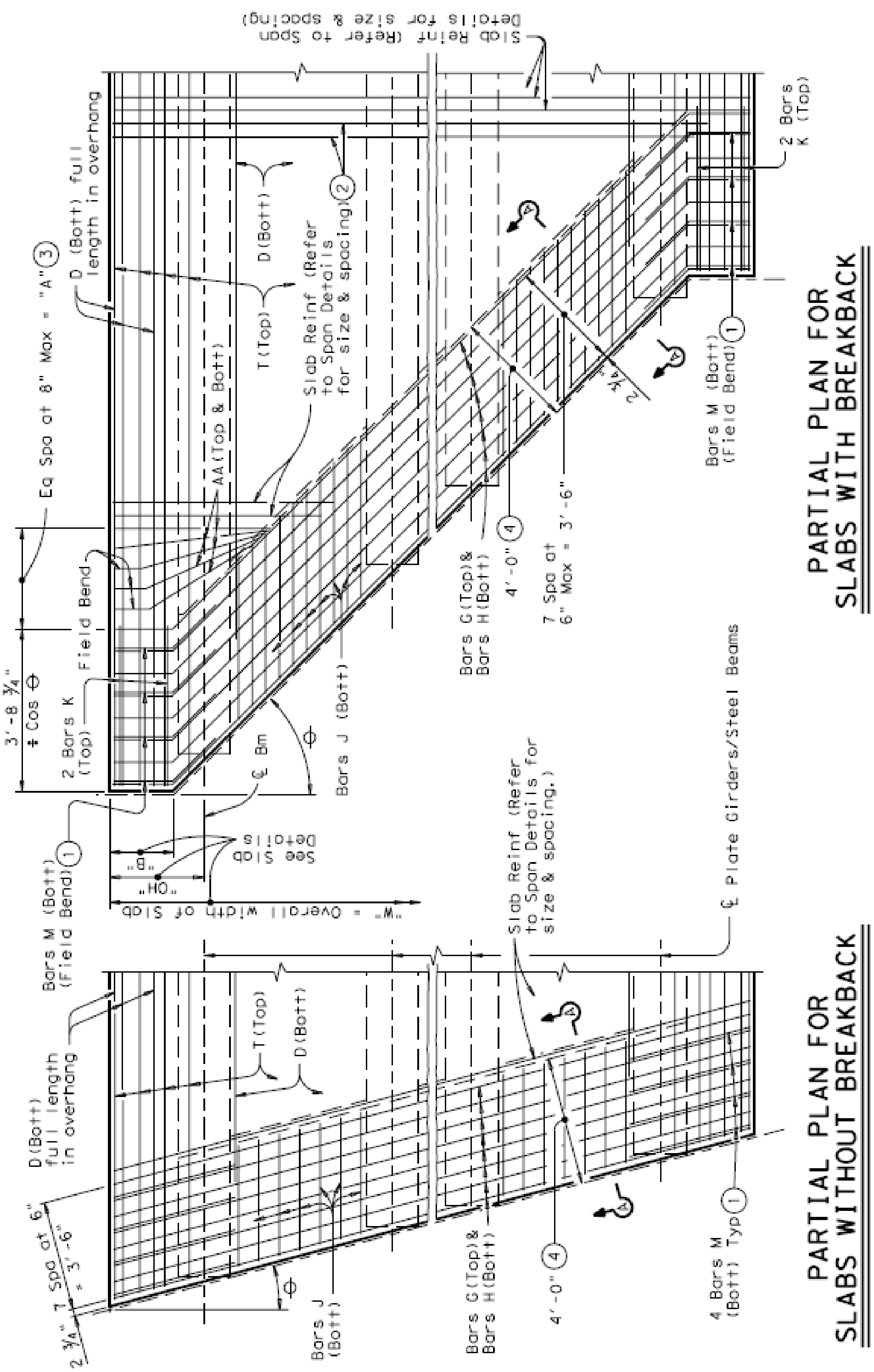


Figure 2.1: TxDOT IBTS detail, plan view

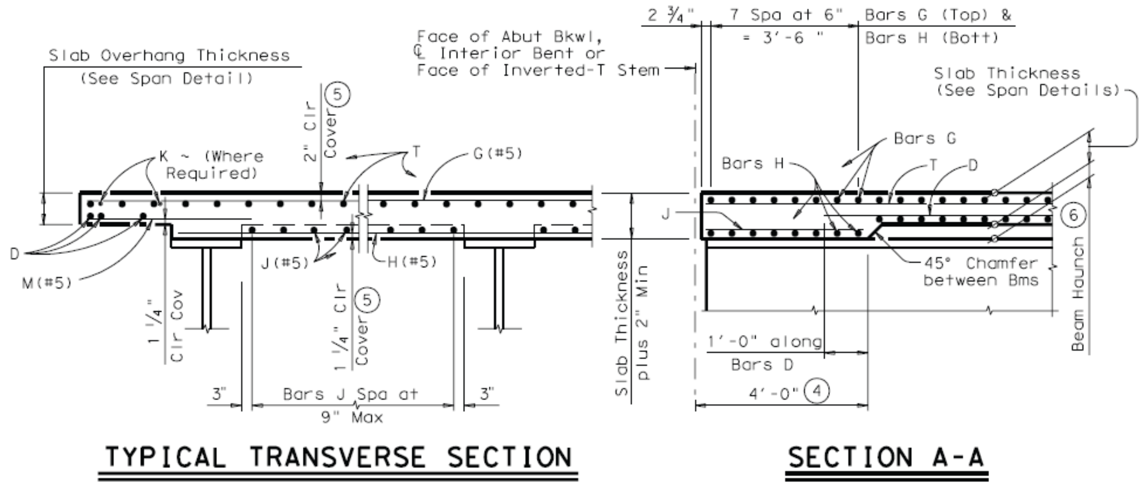


Figure 2.2: TxDOT IBTS detail, cross-sections from Figure 2.1

The UTSE detail, shown in Figure 2.3, was designed to simplify the end detail and reduce costs by decreasing the thickness of the slab to 8 in. To account for the loss of stiffness and to provide sufficient flexural capacity, a larger reinforcing ratio was used. Instead of placing #5's at 6 in. on center, as the IBTS detail specifies, #5's were set at 3-7/8 in. on center.

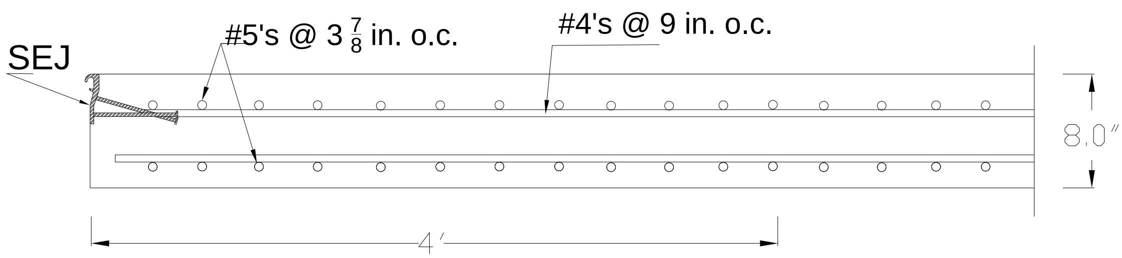


Figure 2.3: Simple cross-section view of UTSE detail (Ryan 2003)

The full-scale bridge deck specimen contained four different test areas. Both positive and negative moment behavior of the IBTS and UTSE details was studied. The test specimen along with load locations denoted by black rectangles can be seen in Figure

2.4. For the positive moment test of each detail, the max girder spacing used by TxDOT of 10 ft. was used. For the negative moment tests, load points were centered over Beam 3 with 8 ft. girder spacing to either side.

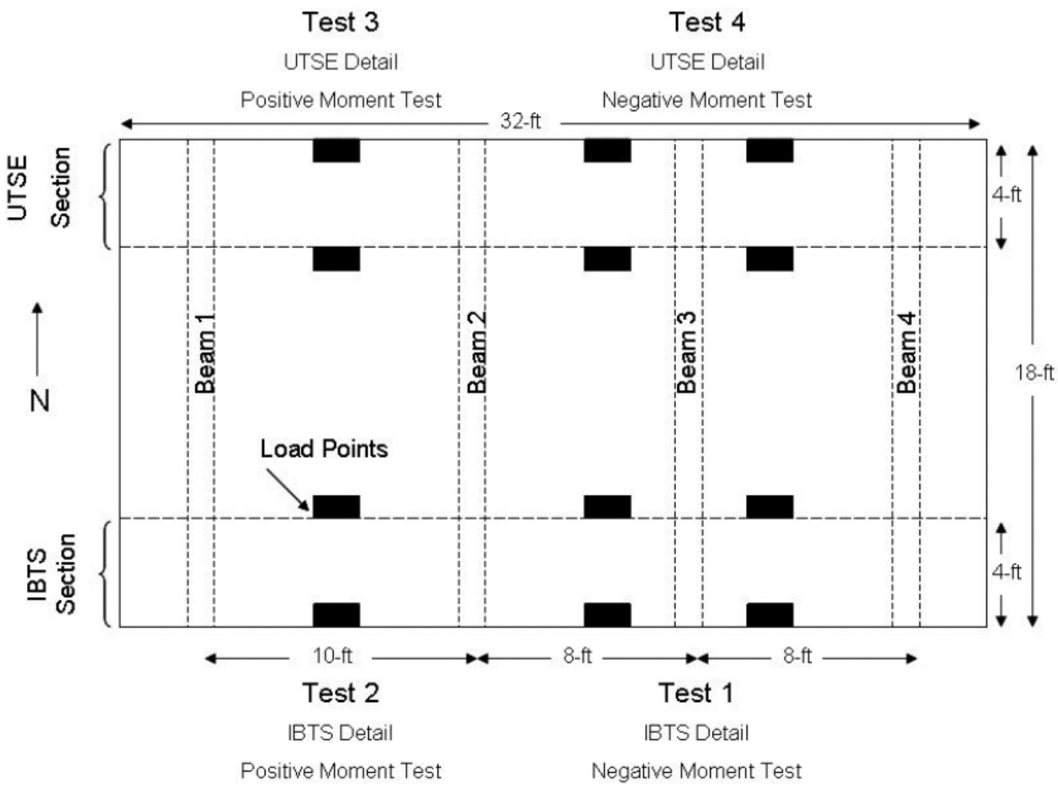


Figure 2.4: Plan view of zero degree skew bridge deck specimen (Ryan 2003)

The test areas were loaded with the AASHTO LRFD Bridge Design Specification HS-20 and HS-25 Design Tandem trucks. Afterwards, the test regions were loaded with typical design overloads of 20%, 75%, and 200%, and finally, loaded to failure. During both negative moment tests, the end details remained un-cracked up to a 200% overload. On the other hand, the positive moment tests began cracking at the design load level. However, a significant reduction in stiffness was not observed until approximately 2 times HS-25 truck load. The specimen failed in punching shear at the end load location in all four tests, but withstood loads significantly higher than design. The reserve strength recorded in each of the four test regions ranged from 4.9 to 6.1 times the HS-25 truck load.

2.2.1.2 Griffith (2003)

A second full-scale bridge deck was built to test the behavior of the TxDOT IBTS end detail with a 45 degree skew. Similarly, the test specimen contained the proposed UTSE detail at one end of the test specimen. Cross-sections of the IBTS and UTSE details are shown in Figures 2.2 and 2.3, respectively. The test specimen was a 21 ft. 6 in. by 33 ft. 7 in. skewed bridge deck built compositely with steel girders, containing three bays and two different end details (Figure 2.5). Once more, panels were not used in the interior portions of the bridge deck and the entire slab was cast-in-place.

The test specimen was loaded in a similar fashion, as well, using the 10 ft. bay for a positive moment test with the loads centered between Beams 1 and 2, and the two 8 ft. bays for a negative moment test with the loads centered above Beam 3. The test regions were loaded with the AASHTO LRFD HS-20 and HS-25 Design Tandem trucks as well as overloads and ultimately to failure.

At service load levels, the negative moment region performed well without developing any cracks. The UTSE detail and the IBTS detail in the positive moment region, on the other hand, began cracking at the HS-20 and HS-25 load levels, respectively. Substantial change in stiffness was not observed until at least 1.6 times HS-25 load levels for any test region. All four test regions showed large reserve strength failing around 4 to 6 times HS-25 truck loads. Three of the four test areas failed in punching shear, with the exception being the IBTS positive moment test, which failed in one-way shear.

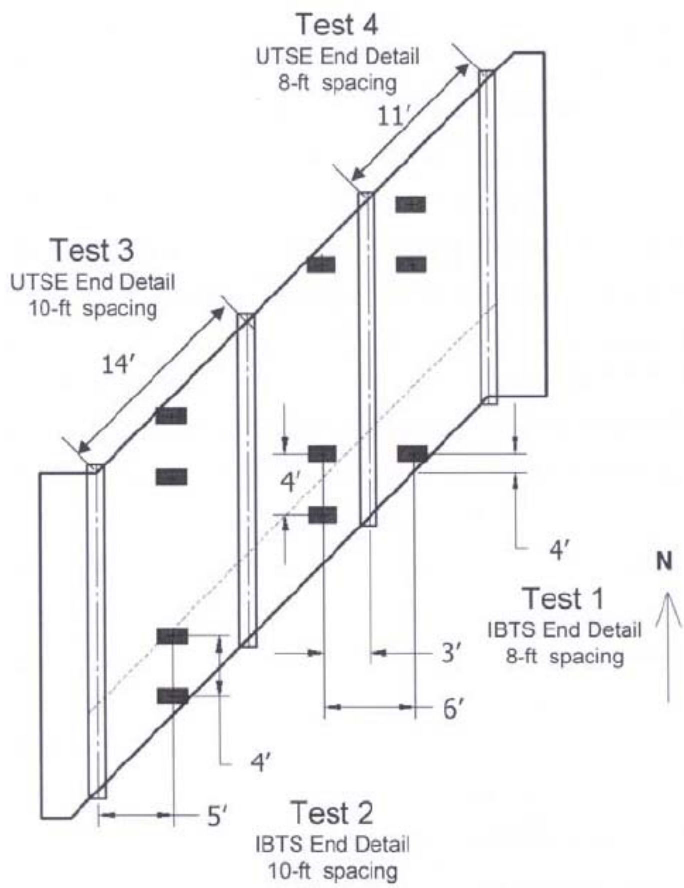


Figure 2.5: Plan view of 45 degree skew bridge deck (Griffith 2003)

2.2.1.3 Coselli (2004)

A third, full-scale bridge deck was constructed in order to study the behavior and constructability of prestressed concrete panels used at zero degree skew expansion joints. A cross-section of the panel end detail is shown in Figure 2.6. The test specimen was a 32 ft. by 18 ft. composite bridge deck consisting of one 10 ft. bay and two 8 ft. bays. The specimen, shown in Figure 2.7, contained a total of six test areas.

Aside from using the precast panel for the end detail, additional variables were introduced in these tests. The influence of the expansion joint armor was investigated by using two different types on the north side of the specimen, armor joint and sealed expansion joint, and no joint armor on the south side. Some changes in the anchorage details of the expansion joint hardware were needed so that the joint armor would be anchored only in the 4 in. cast-in-place topping slab. On the side without an armor joint, the top layer of transverse reinforcing was spaced at the TxDOT standard 6 in. on center for one negative moment test and 3-7/8 in. on center for the other negative moment test.

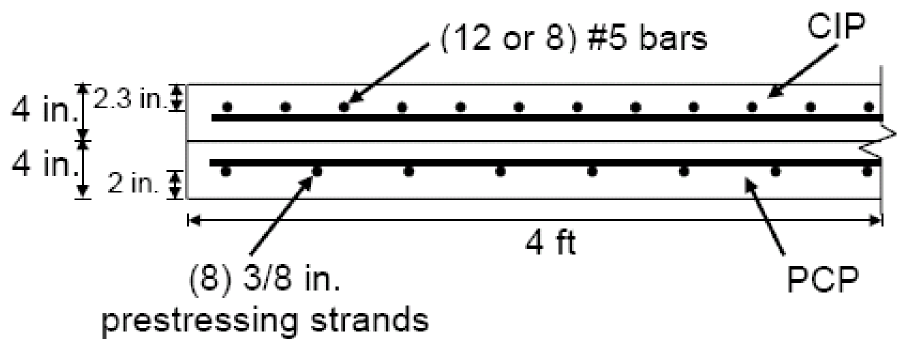


Figure 2.6: Cross-section of prestressed concrete panel end detail (Coselli 2004)

To prevent the influence of one test region on another, each test area was first loaded to service level loads with the AASHTO Bridge Specification HS-20 Design Tandem to observe cracking patterns. The loads were placed at mid-span of the 10 ft. bay to maximize positive moment and centered over the interior girders to maximize negative moment. Afterwards, four of the six test areas (1, 3, 4, and 6) were loaded until failure.

The results from all tests showed excellent performance under service level loads. No cracks were observed in any test until at least twice the design load. The tests on regions that included an expansion joint rail failed at loads 20-25% higher while experiencing less deflection. The only noticeable difference between the tests without an expansion joint rail was that cracks were more evenly distributed when the top transverse reinforcing was spaced closer together. All test areas that were taken to ultimate load failed in punching shear with load levels ranging from 5.4 to 7 times HS-20.

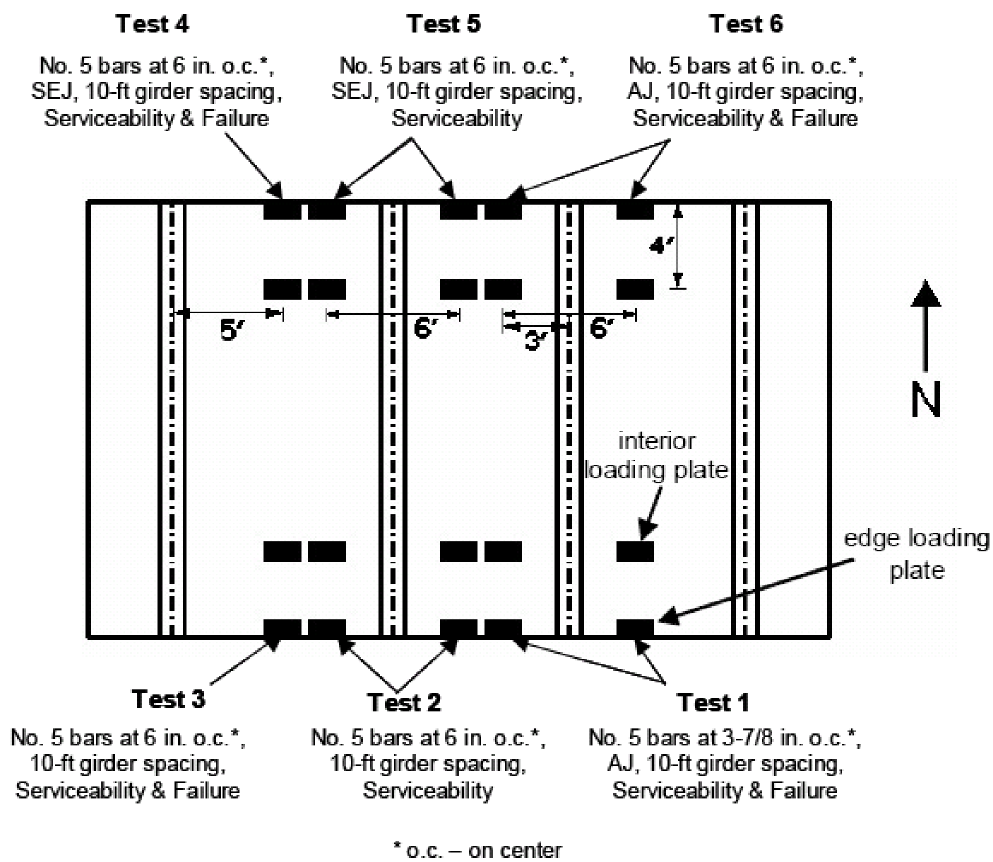


Figure 2.7: Plan view of zero degree skew bridge deck (Coselli 2004)

2.2.2 TxDOT Project 0-5367

Based on the results of the tests by Coselli (2003), project 0-5367 was initiated to further investigate the behavior of prestressed concrete panels used at expansion joints.

2.2.2.1 Agnew (2007)

In the first phase of the research project, Agnew (2007) tested four full-scale composite bridge deck specimens to determine the fatigue behavior of panels used at expansion joints of zero degree skew bridges. Rather than constructing one large bridge deck comprising multiple test regions, individual test specimens were built for each test. The two positive moment tests were 8 ft. by 11 ft. single bay bridges built with a single precast panel. Using finite element modeling, it was determined that the HL-93 Design Truck produced a larger stress at the slab end than the HL-93 Design Tandem. Therefore, a single load point representing a wheel load from the HL-93 Design Truck was placed at the end of the composite slab. An elevation and plan view of the positive moment test setup is shown in Figures 2.8 and 2.9.

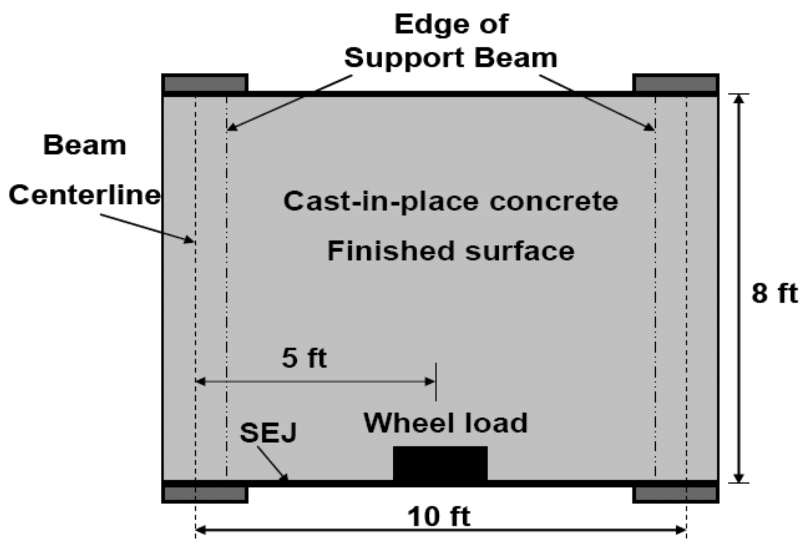


Figure 2.8: Plan view of positive moment fatigue test (Agnew 2007)

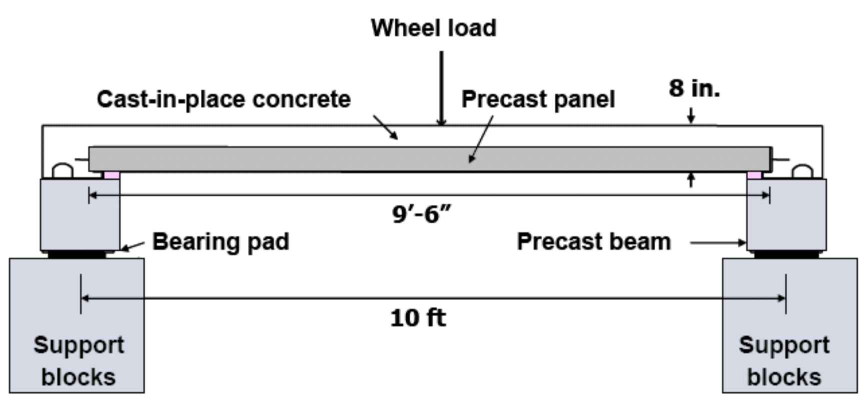


Figure 2.9: Elevation view of positive moment fatigue test (Agnew 2007)

The negative moment test specimens were 8 ft. by 21 ft. consisting of two bays and two panels. Two load points spaced at 6 ft. centered over the interior beam were used for the fatigue loading of these specimens. Figures 2.10 and 2.11 show elevation and plan views for the negative moment test specimens, respectively.

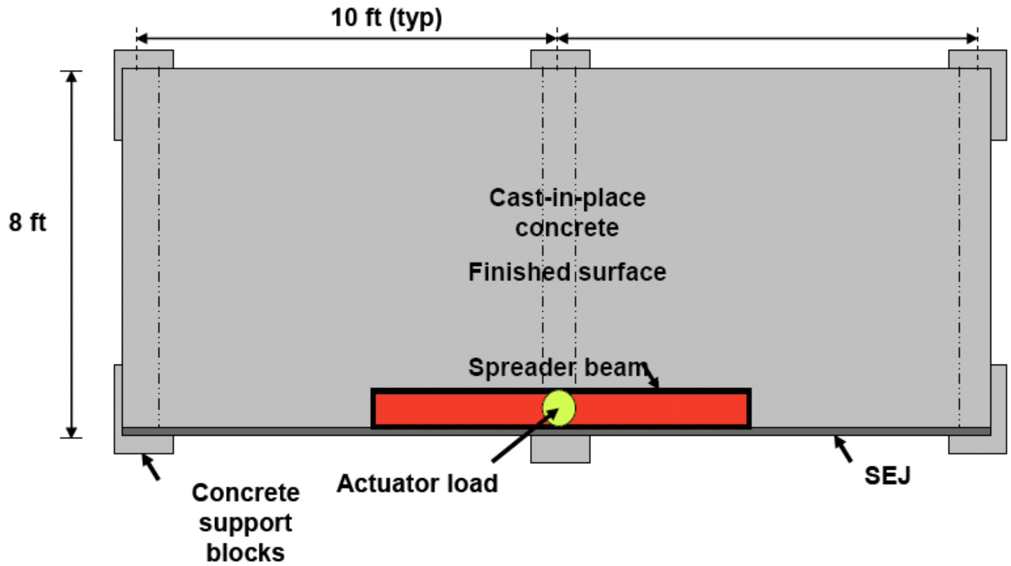


Figure 2.10: Plan view of negative moment fatigue test (Agnew 2007)

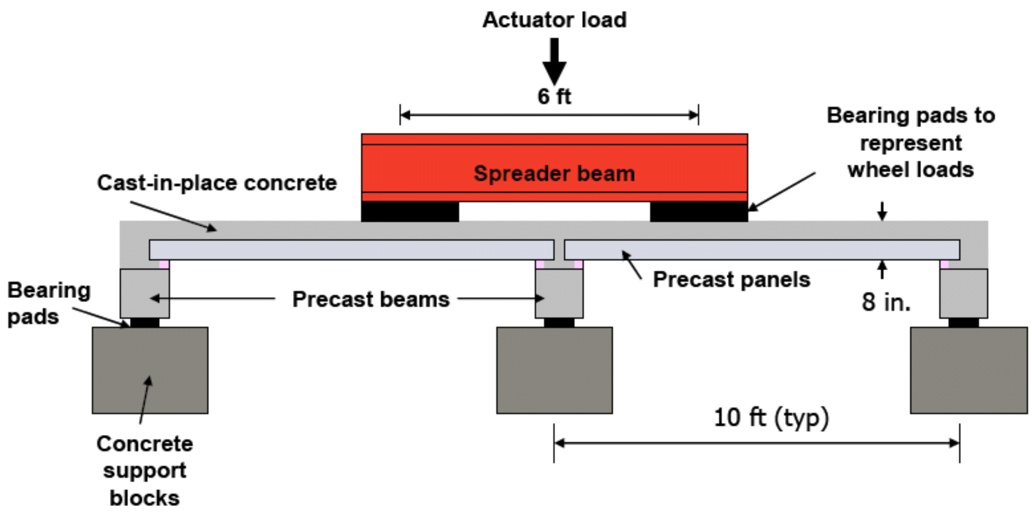


Figure 2.11: Elevation view of negative moment fatigue test (Agnew 2007)

Each specimen was first subjected to service or design level fatigue loads followed by a static overload test after 2 million cycles. The fatigue load testing was then continued up to 5 million cycles before finally conducting a static test to failure.

All four test specimens behaved excellently under the fatigue loading. During each test, the stiffness of the composite slab did not change during the first 2 million load cycles. After the static overload test, the stiffness decreased, but did not change appreciably throughout the remainder of the fatigue loading. No delamination was observed at the interface of the panel and the cast-in-place topping slab in any of the test specimens. When testing to failure, all four test specimens failed in punching shear at load levels exceeding 3.5 times design wheel loads.

2.3 Skewed Prestressed Panels and Slabs

In the following sections, topics involving skewed concrete bridge construction and behavior are discussed.

2.3.1 Abendroth, Pratanata, and Singh (1991)

Under research project HR-310 for the Iowa Department of Transportation, Abendroth, Pratanat, and Singh (1991) conducted a comprehensive research project to investigate the performance of precast concrete panels used as part of composite bridge deck slabs. The research project included surveys of design and fabrication agencies, field inspections, analytical models, and full scale testing. The focus of the discussion herein will be on the experimental testing of skewed panels and the results from the survey.

2.3.1.1 Experimental Testing of Trapezoidal Panels

The experimental portion of research project HR-310 involved five full-scale composite slab specimens. Three of the five specimens included skew angles of 15, 30, and 40 degrees each (Figures 2.12-2.15). Each composite slab was comprised of a 2-1/2 in. prestressed concrete panel and 5-1/2 in. of cast-in-place concrete topping. The support beams were spaced 8 ft. on center creating a 6 ft. 6 in. clear span. These dimensions were the largest allowable spacing permitted by Iowa DOT at the time of research. In addition to the two longitudinal support beams, the panels were supported by a third beam running along the skewed end of the panel. The panels were supported on 3/4 in. thick by 1 in. wide fiber-board on all bearing surfaces.

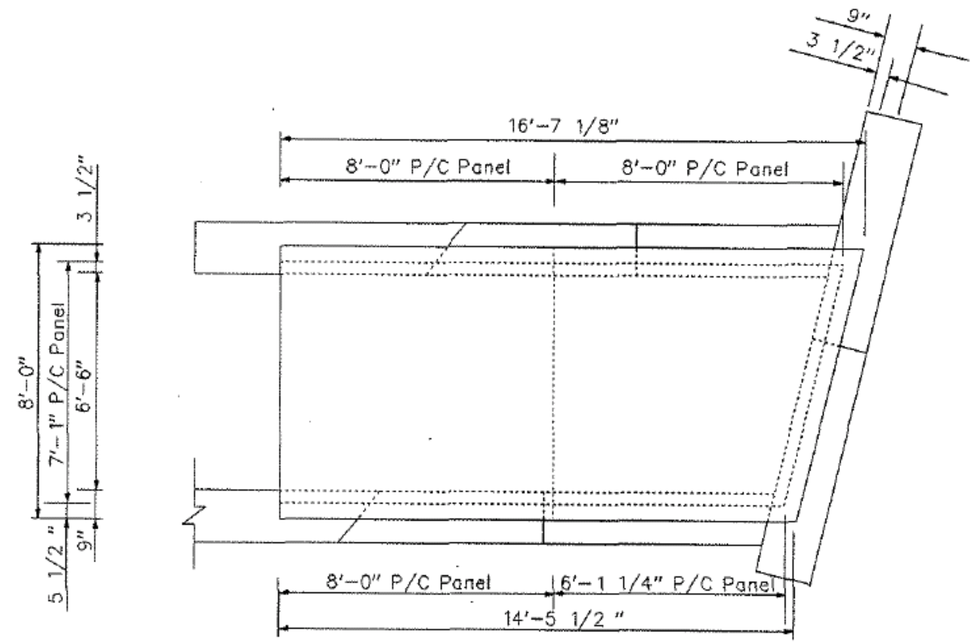


Figure 2.12: Plan view of 15 degree skew test (Abendroth 1991)

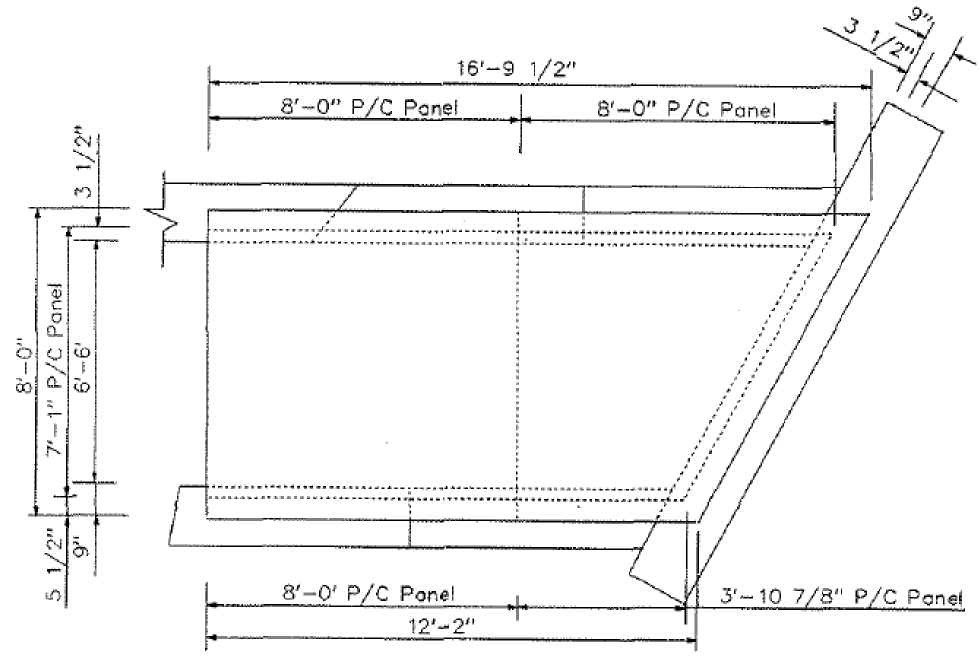


Figure 2.13: Plan view of 30 degree skew test (Abendroth 1991)

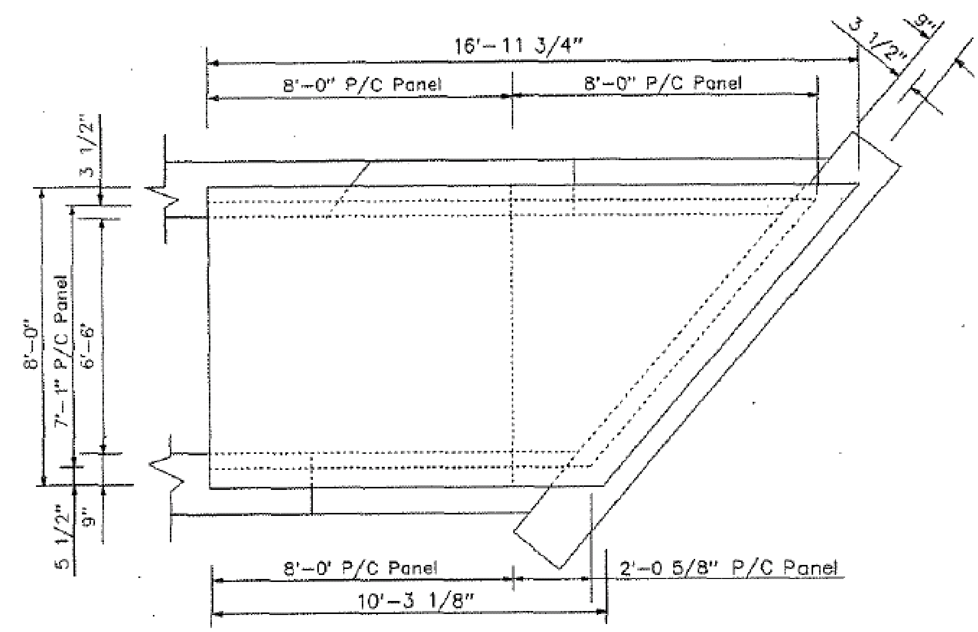


Figure 2.14: Plan view of 45 degree skew test (Abendroth 1991)

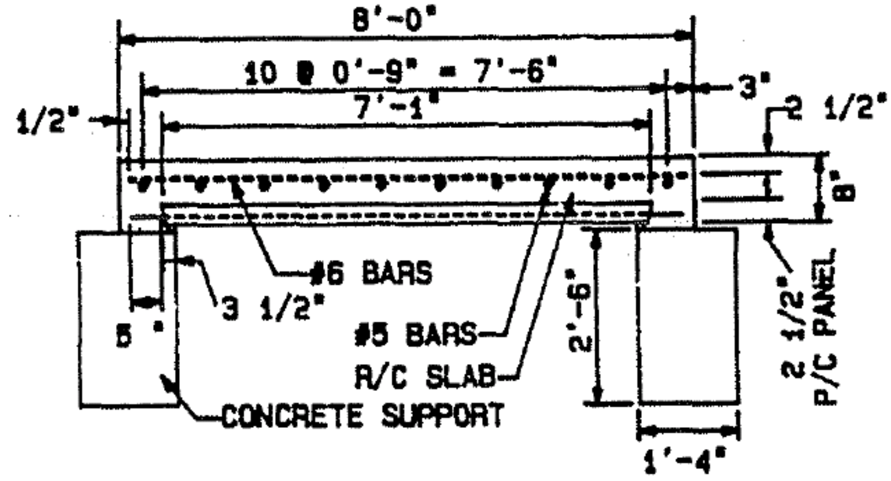


Figure 2.15: Cross-section of composite specimens (Abendroth 1991)

The precast concrete panels were 2-1/2 in. thick and 7 ft. 1 in. wide. The trapezoidal panels were all 8 ft. long on the long edge whereas the short edges were 6 ft. 1-1/4 in., 3 ft. 10-7/8 in., and 2 ft. 0-5/8 in. for the 15, 30, and 40 degree skews, respectively. The panels were prestressed using 3/8 in., 270 ksi low-relaxation prestressing strands positioned at the mid-depth of the panel 6 in. on center. In addition to the prestressing strands, the skewed panels had two #3 bars placed along the skewed end and a welded wire mesh placed directly above the strands. Plan and cross-section views of the trapezoidal panel designs are shown in Figure 2.16. The strands were oriented transverse to the main support beams tensioned to 17.2 kips each prior to concrete placement. To prevent the panel from cracking during the strand de-tensioning, two, three, and four of the shortest strands passing through the acute angle of the trapezoidal panel were sleeved on the 15, 30, and 40 degree skew panels, respectively. The fabrication of the panels was done at a precast plant in Iowa Falls, IA.

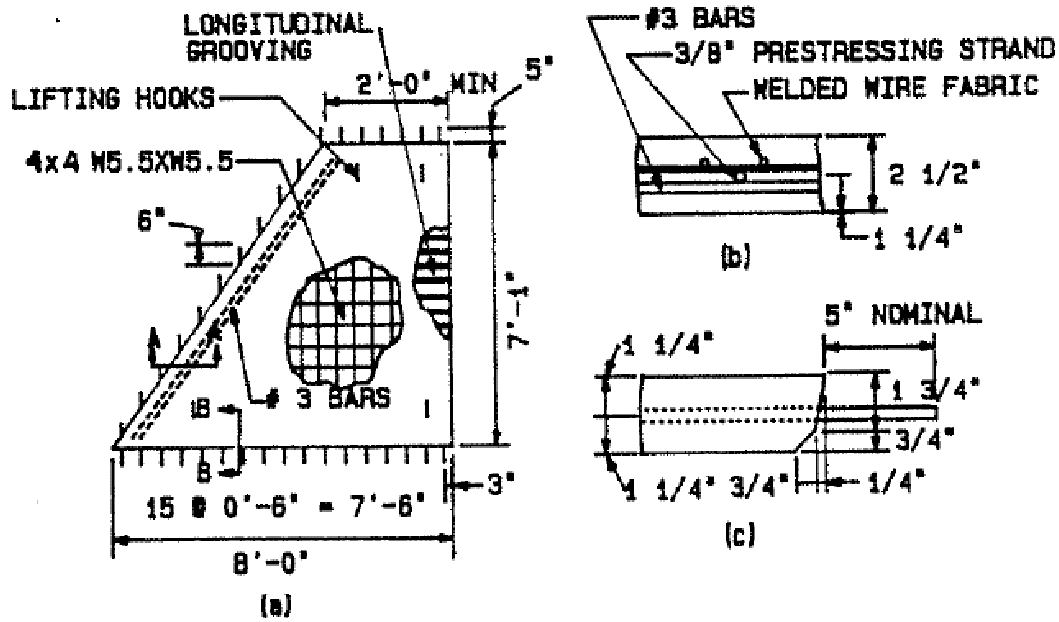


Figure 2.16: Trapezoidal prestressed concrete panels: (a) Plan view (b) Section A-A (c) Section B-B (Abendroth, et al. 1991)

Once the panels were set upon the fiber-board on the support beams, a mat of reinforcing was placed over the panels made up of longitudinal #5 bars spaced at 9 in. and transverse #6 bars spaced at 10 in. A concrete topping was placed on the panels for form a composite bridge slab.

The test specimens were first loaded with both single and double wheel service level loads based on the AASHTO HS-20 design truck plus impact at various locations around the slab. After service load tests were conducted for a give test specimen, the slabs were tested to failure. The locations for the failure load were generally in the center of the trapezoidal panel. All three test specimens failed in punching shear at load levels between 10 and 11 times HS-20 design truck loads.

2.3.1.2 Results from Panel Questionnaires

As part of the research project, surveys containing 82 multiple choice questions were distributed to design agencies and precast concrete plants across the United States and Canada. The purpose of the survey was to get an understanding of how other governing bodies permit usage of precast concrete panels during bridge construction as well as capabilities and opinions of manufactures that have made such panels.

The questionnaires sent to 121 departments of transportation and tollway authorities contained questions pertaining to general bridge and panel geometry, bearing details, design criteria and specifications, economy, and experience with panel usage. Only 29 of the 69 agencies that returned the surveys said that they have used panels in bridge construction. The design agencies expressed concerns regarding performance, serviceability, economy, and lack of composite bridge deck specifications by AASHTO. Only 16 of the 29 agencies that have used panels for bridge construction were still permitting their use at the time of the survey (1991).

Selected survey results from the questionnaires returned by design agencies relating to skewed bridge decks and panels are as shown in Table 2.1. The numbers in parenthesis correspond to the number of agencies reporting that answer. Considerable variability is evident from the survey results regarding the usage of skewed prestressed panels. Maximum skew angles and minimum short side length were reported as large as 50 degrees and as short as 1 ft., respectively. A majority of the agencies that permit the use of skewed panels reported that no additional reinforcing is required and no prestressing strands are de-bonded. To achieve the desired skew angle, most agencies allow the panels to be either sawn or cast. Overall, both positive and negative feedback was given with respect to panel usage. Many design agencies had yet to conduct an economic analysis or evaluation of performance.

Table 2.1: Selected survey results from design agencies (Abendroth 1991)

1. Maximum bridge skew for panels adjacent to abutments or pier diaphragms:

   (14) Not Specified
   (2) 15 Degree
   (3) 30 Degree
   (0) 45 Degree
   (4) Other [0,18,20,50 Degree]

2. For non-rectangular shaped panels that occur at abutment and pier diaphragms in skewed bridges, what is the minimum length of a panel side?

   (8) Panels not permitted at these locations
   (5) Not specified
   (0) 0 ft. (triangular shaped panel)
   (5) 1 ft. (trapezoidal shaped panel)
   (2) 2 ft. (trapezoidal shaped panel)
   (9) Other [Unspecified, one-half the length of the opposite side, 1.5 ft, 2.25 ft, 3 @ 3ft, 2 @ 3.25 ft]

3. Panel construction at skewed abutment or pier locations:

   (8) Panels are not used at these locations
   (4) Panels sawn to match the skew only
   (2) Panels cast to match the skew only
   (12) Panels sawn or cast to match the skew
   (4) Other [Panels not used when skew > 15 deg., C.I.P. full depth, C.I.P. slab if skew > 30 deg., May also cast closure in place without panel]

4. For non-rectangular panels, what type of additional reinforcement, other than the conventional rectangular panel reinforcement, is provided in the panel?

   (8) Non-rectangular panels are not permitted
   (15) None
   (2) R/C bars only
   (0) Wire Strands
   (1) WWF only
   (0) Any of the above
   (1) Other [Unspecified]

5. For a non-rectangular shaped panel, are some strands unbonded near the panel ends?

   (6) Only rectangular shaped panels are permitted
   (0) Always
   (0) Sometimes
   (21) Never

The surveys sent to 192 different manufacturers addressed topics relating to experiences, general bridge and panel geometry, design criteria and specifications, economy, inspections, and opinions. Of the 72 producers who returned the questionnaires, only 27 claimed they had made precast concrete panels for bridges. Many of the fabricators who said they did not make panels gave reasons such as local preference for cast-in-place bridge decks, difficult quality control, not economical, and competition within the market.

Selected survey results from the questionnaires returned by manufacturing agencies relating to skewed bridge decks and panels are as shown in Table 2.2. The numbers in parenthesis correspond to the number of agencies reporting that answer. Again, great variability is seen regarding the manufacturing of precast panels. The minimum panel edge length received a wide variety or answers ranging from as little as 0 ft to a maximum of 4 ft. Two, three, and four manufacturers reported maximum skew angles of 15, 30, and 45 degrees, respectively, whereas 10 claimed there was no maximum as long at the short side is 1 ft. wide. Eleven of the 20 producers who construct skewed panels reported using additional mild reinforcing bars along the skewed end. Casting the panels to match the desired skew angle was the most common method of construction, but several other manufactures reported sawing the panels to achieve the skew angle. When asked their opinion about skewed precast panels, some agencies claimed that the panels were difficult to de-tension and set properly. Additionally, many producers felt that precast panels would require better standardization and details to become an economic alternative. Nonetheless, when rating the overall panel usage, 19 of the 24 respondents reported a rating between *Good* and *Excellent*.

Table 2.2: Selected survey results from manufacturing agencies (Abendroth 1991)

1. For non-rectangular shaped panels that occur at abutment and pier diaphragms in skewed bridges, what is the minimum length of a panel side?

   (6) Only rectangular panels are cast [Mostly saw cut in the field]
   (4) 0 ft. (triangular shaped panel)
   (3) 1 ft. (trapezoidal shaped panel)
   (7) 2 ft. (trapezoidal shaped panel)
   (9) Other (trapezoidal shaped panel cast) [1 ft, 2.83 ft, 3 @ 3 ft, 4 ft]

2. Panel construction at skewed abutment or pier locations:
   (3) Panels are not used at these locations
   (3) Panels sawn to match the skew only
   (12) Panels cast to match the skew only
   (7) Panels can either be sawn or cast to match the skew
   (4) Other [N.A.]

3. For non-rectangular panels, what type of additional reinforcement, other than the conventional rectangular panel reinforcement, is provided in the panel?

   (6) Only rectangular panels without additional reinforcement are cast
   (5) None
   (2) Prestressing strands only
   (1) WWF only
   (11) Reinforcing bars only [Extra No. 4 bars, 8 No. 5 bars along future cutted skew location]
   (1) Other [Varies with job]

4. Maximum skew angle for casting non-rectangular panels to match the bridge skew for those panels adjacent to abutment or pier diaphragms:

   (6) Only rectangular panels are cast
   (2) 15 Degree
   (3) 30 Degree
   (4) 45 Degree
   (10) No maximum [Minimum edge length of 1 ft, No pointed corners]
   (1) Other [As long as the ratio of the long to short panel end is 2 or less]

5. For a non-rectangular shaped panel, are some strands unbonded near the panel ends?

   (6) Only rectangular shaped panels are cast
   (0) Always
   (5) Sometimes
   (14) Never

2.3.2 Rajagopalan (2006)

In his book *Bridge Superstructure*, Rajagopalan presents a chapter on the design of skew slab bridge decks. Although the post-tensioned slab bridge decks discussed are much larger than precast panels and typically parallelogram-shaped, the concepts presented are relevant.

For rectangular slab bridge decks, the principle moments act in the longitudinal and transverse directions with respect to the supports. Loads placed on the slab are transferred to the support directly through flexure. A small amount of torsion exists due to the unsupported edges, but the effect is negligible. For skewed slabs, however, the slab primarily bends along a line between the two obtuse angled corners. The width of this bending strip is a function of the skew angle and ratio of slab width to length. Loads placed to either side of this strip are not transferred directly to the supports; rather they are transferred to the strip in a cantilevered manner. This cantilevered transfer of forces can cause very large twisting moments on the strip with high skew angles. Diagrams showing the stress trajectories for various skew angles are shown in Figure 2.17.

The deflections and reactions resulting from torsion within the slab are not uniform or symmetrical. As the skew increases, the maximum deflection on the free end of the slab gets closer to the obtuse angled corner. A majority of the load is transferred to the obtuse angled corner and in the case of large skews, the acute angled corners can actually experience uplift. Diagrams showing deflection profiles in different regions of a skewed slab bridge deck are given in Figure 2.18.

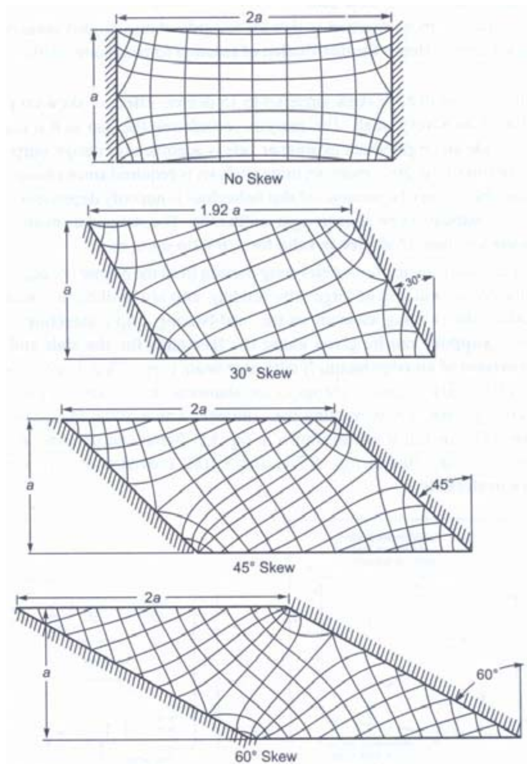


Figure 2.17: Principle stress trajectories for various skews (Rajagopalan 2006)

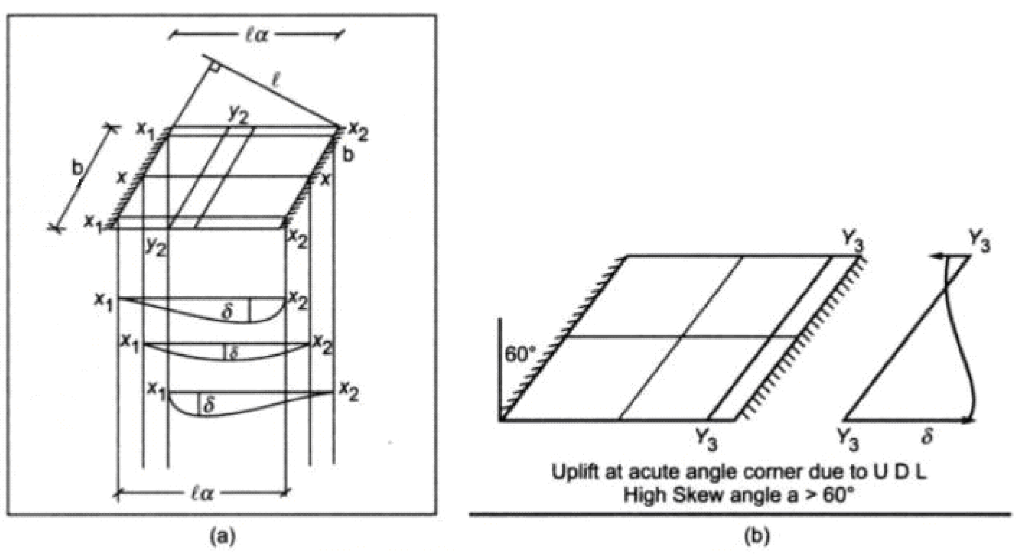


Figure 2.18: (a) Deflection profile in a skewed deck (b) 'S' shaped deflection profile near the support line for large skew angles (Rajagopalan 2006)

Because the principle moments are parallel to the strip between obtuse-angled corners, conventional post-tensioning oriented parallel to the free edges is not the most effective. Instead, differential post-tensioning would better counteract the higher moments around the obtuse-angled corner and lower moments around the acute-angled corner. This could be achieved by using a fan-shaped strand layout where strands are spaced closely together near the obtuse-angled corner and further apart around the acute-angled corner. A plan view of a fan-shaped strand pattern is shown in Figure 2.19.

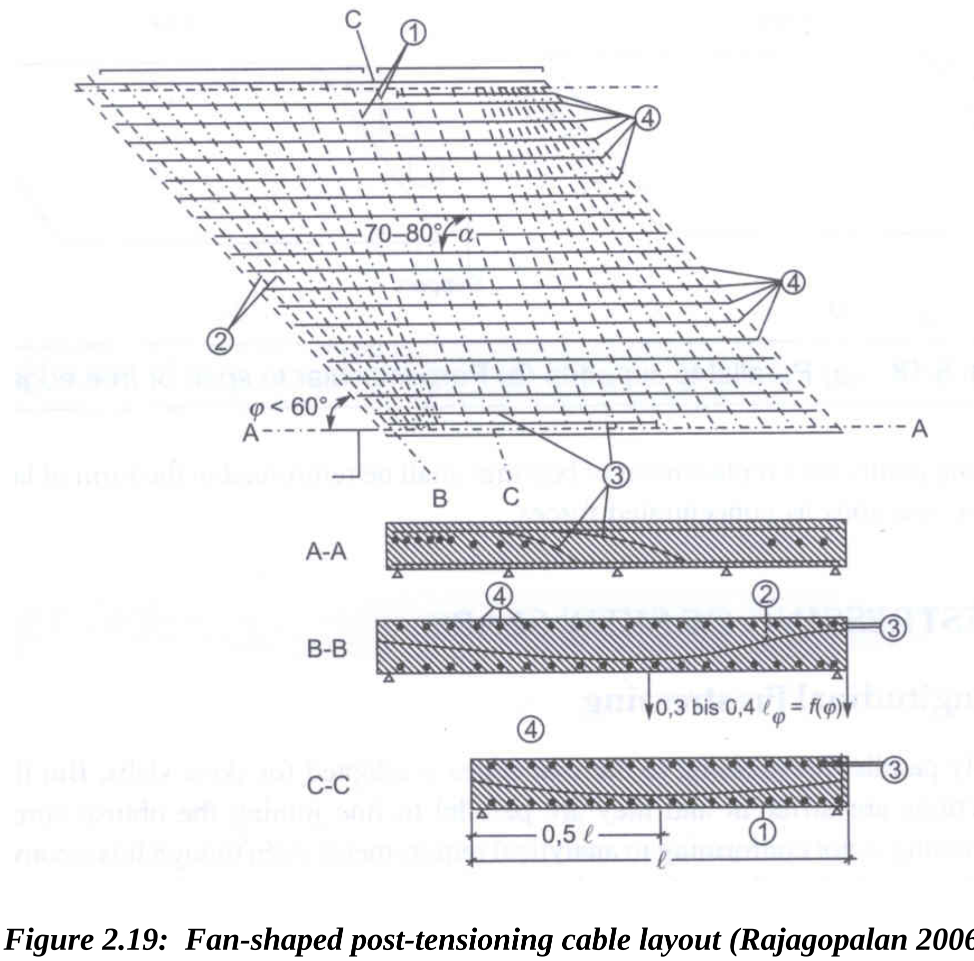


Figure 2.19: Fan-shaped post-tensioning cable layout (Rajagopalan 2006)

Some conclusions regarding the use of fanned post-tensioning are as follows:

- Slab behavior will become more similar to non-skew slabs.
- The larger moments near the obtuse-angled corners verify the need for fanned tendons.
- The location of maximum deflection shifts away from the obtuse-angled corner and closer to mid-span.
- Deformations transverse to the supports are more uniform along the length of the slab.
- The support reactions are more evenly distributed.

2.4 Research Significance

While it is evident that skewed precast prestressed concrete panels have been used in bridge construction, little research on the construction and behavior is reported. The tests conducted by Abendroth (1991) used panels at expansion joints, but the skewed end support represented an end diaphragm or abutment. Rajagopalan (2006) discussed theoretical fan-shaped post-tensioning of skewed slab bridge decks and its potential benefit. It was not reported if this method of construction was indeed practical, economical, or effective. Unsupported slab end tests conducted by Ryan (2003) and Griffith (2003) on non-skew and skewed bridge decks, respectively, showed significant reserve strength when compared with design parameters. Similarly, Coselli (2004) and Agnew (2007) demonstrated that non-skew panels were a viable alternative for an end detail that well exceeded design and fatigue capacities.

TxDOT's current construction method using precast concrete panels has become very economical. Construction could become even more efficient if panels were used along the entire length of the bridge. To permit use of trapezoidal panels at expansion joints, a standard detail must first be created and tested to demonstrate its reliability.

CHAPTER 3

Design of Test Specimens

3.1 INTRODUCTION

The objective of the experimental program was to evaluate the constructibility and practicality of producing skewed prestressed panels for use at expansion joints. In the investigation of a new product, many variables may need to be tested or held constant. The areas of primary concern included the skew angle, panel dimensions, and prestressing strand arrangement. Additional variables included prestressing force, concrete release strength, and supplementary deformed reinforcing bars for bursting or flexural reinforcement.

3.2 PRELIMINARY DESIGN

In September 2006, the research team held a joint meeting with representatives from TxDOT's bridge division, local precast concrete panel fabricators, and bridge construction contractors. The objective was to determine if contractors had an interest in skewed panels, if the fabricators could produce such panels, and if the requirements of TxDOT's bridge division could be satisfied. At the joint meeting, the discussion was open for any ideas from TxDOT, fabricators, or contractors. The main topics discussed were the panel geometries and methods of reinforcement. Several basic concepts came from this meeting and set the initial outline from which planning could proceed. A follow-up meeting held in November 2006 with the TxDOT representatives to establish the test parameters.

3.2.1 Panel Geometry

The most feasible panel geometries presented were:

- **Option 1 -** One large trapezoidal panel (Figure 3.1). Using a single panel to make the skew angle transition would require the fewest custom panels and minimize construction awkwardness. However, panels with large angles and widths cannot be produced on current prestressing beds and would require construction of new, wider ones. Also, depending on the orientation of the prestressing, some of the strands may have embedment lengths shorter than required to transfer forces into the panel. Areas with such strands might not provide the necessary strength.

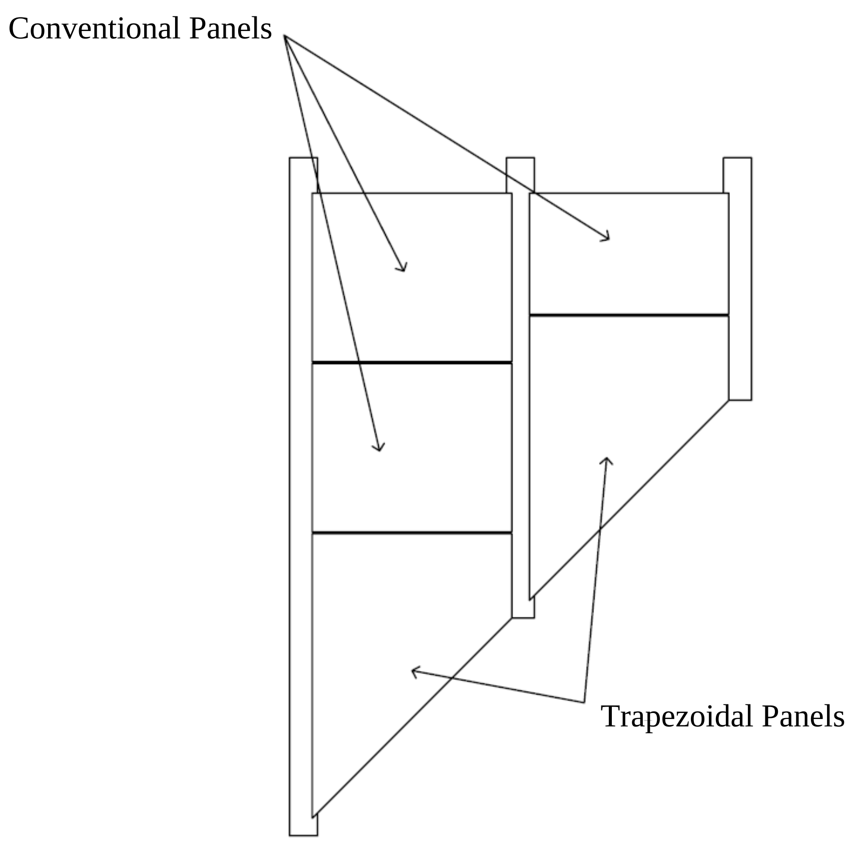


Figure 3.1: Option 1 - Single trapezoidal panels

- **Option 2 -** System of two, smaller trapezoidal panels (Figure 3.2). By breaking up the skew angle transition into two panels, each panel would be small enough to fabricate on current prestressing beds. As well, smaller angles on each panel may result in fewer strands lacking the proper embedment length. The downside to this method is that twice as many custom panels are required and construction crews have to manage the placement of more awkwardly shaped panels.

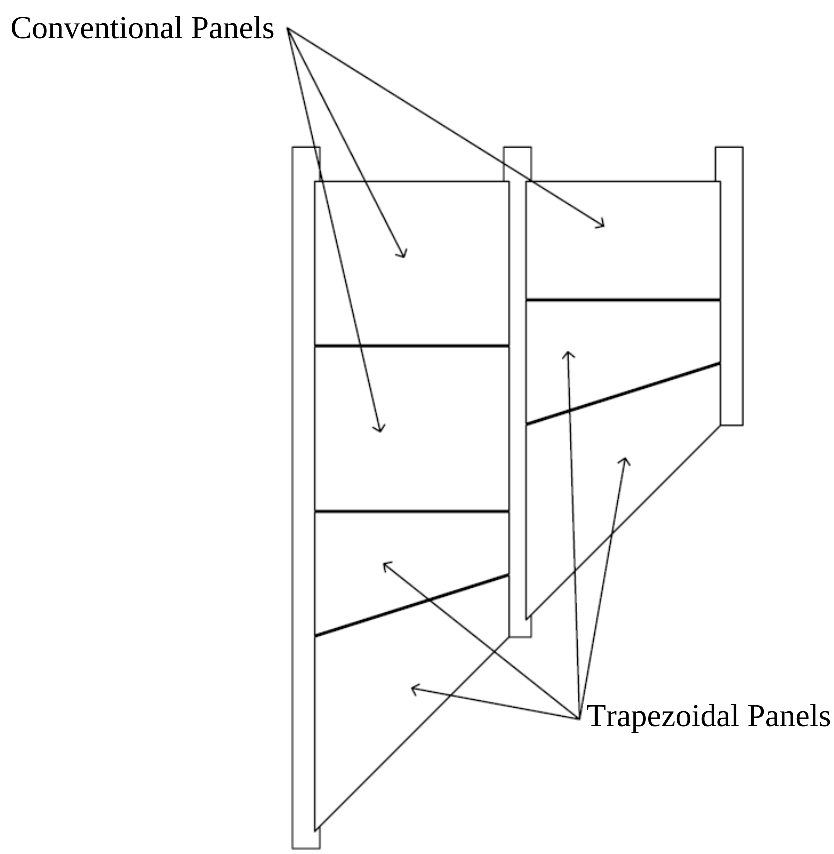


Figure 3.2: Option 2 - Combination of two trapezoidal panels

- **Option 3 –** Quadrilateral panel with parallel sides at expansion joint followed by trapezoidal panel (Figure 3.3). By making the edge panel a parallelogram, current prestressing beds could be used with skewed formwork. However, the second, trapezoidal shaped panel would still require a new casting bed for large skew angles and beam spacing, just like that in Option 1. The main benefit of this method would be to ensure a fully prestressed panel at the expansion joint. Furthermore, regions in the trapezoidal panels containing strands without sufficient embedment would be away from the expansion joint.

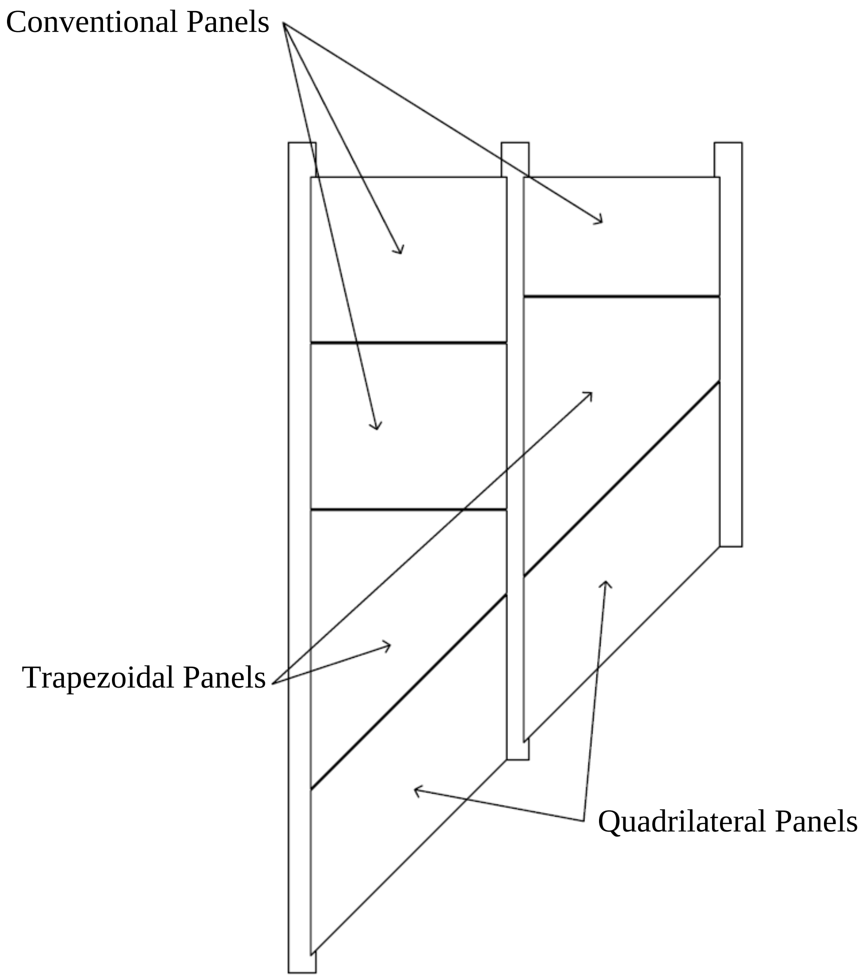


Figure 3.3: Option 3 – Combination of quadrilateral and trapezoidal panels

3.2.2 Reinforcement Alternatives

As mentioned in the panel options, the trapezoidal shaped panels have several different reinforcement alternatives. Considering typical casting bed layouts, prestressing strands could be oriented either perpendicular to the girders, parallel to the girders, or parallel with the skewed expansion joint. In each case, supplemental reinforcement would need to be placed at locations without effective prestressing. Figures 3.4 and 3.5 show different prestressing arrangements with the supplementary deformed reinforcement that would be needed. Another alternative that would not require any additional deformed bars is shown in Figure 3.6. By flaring the strands throughout the panel, strands are parallel to both the skewed and non-skewed ends. In each figure, strands that do not meet the embedment length requirement were omitted to show partially prestressed locations. Furthermore, typical temperature and shrinkage reinforcement required for panels is not shown.

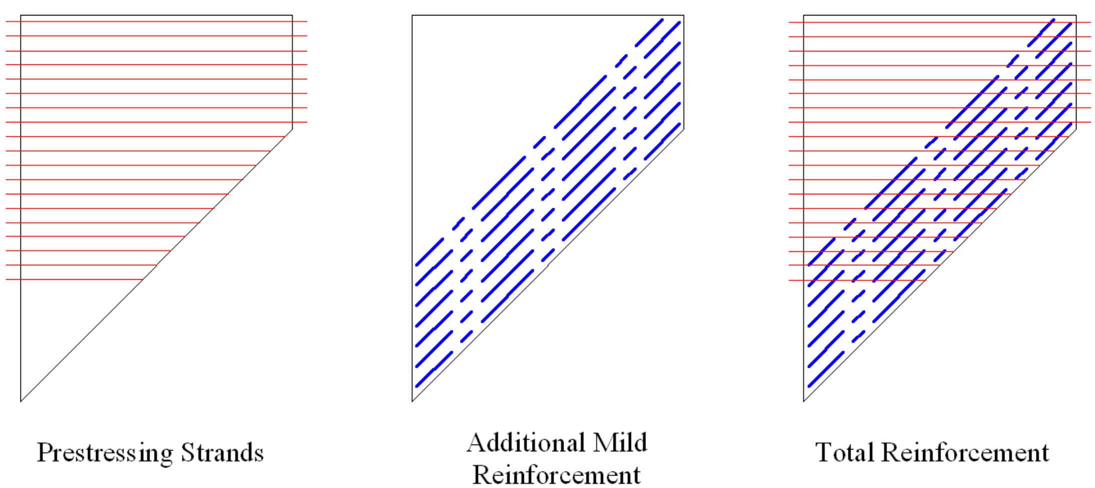


Figure 3.4: Strands oriented perpendicular to the girders

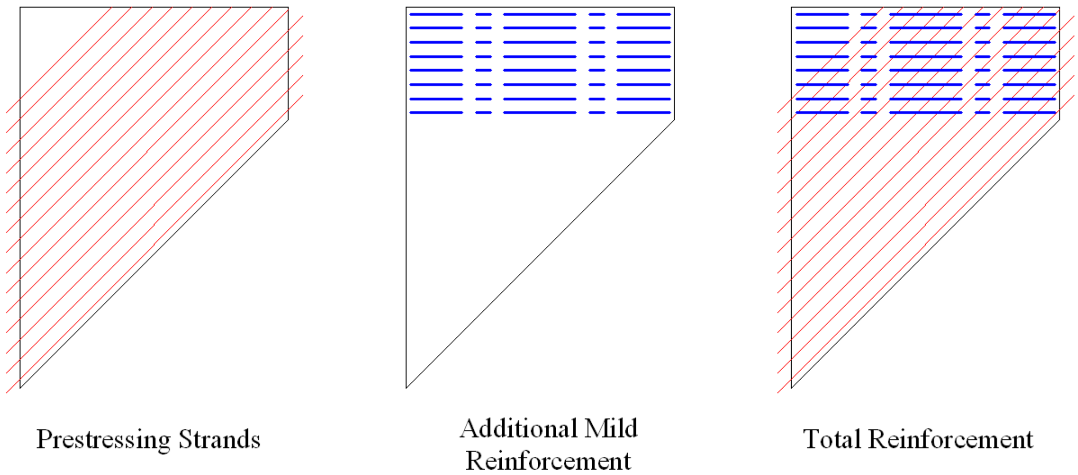


Figure 3.5: Strands oriented parallel to the skewed end

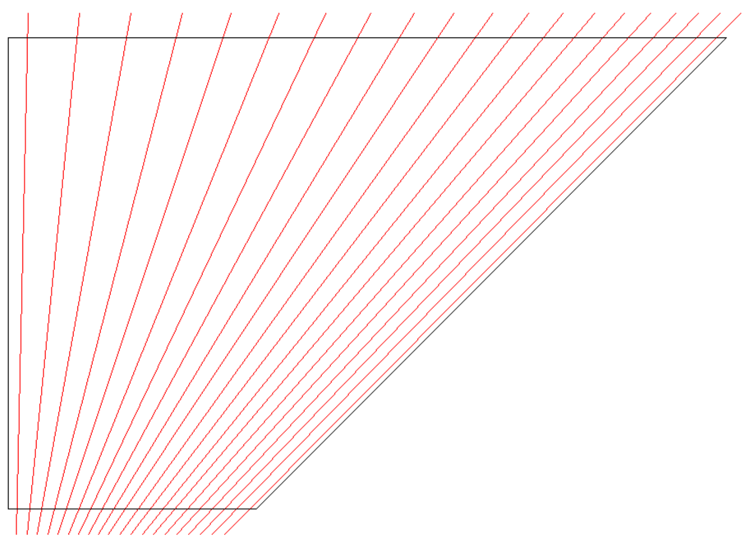


Figure 3.6: Strands flared throughout panel

In addition to prestressed panels, a conventionally reinforced panel and a post-tensioned panel using dywidag bars were discussed. The conventionally reinforced panel option was eliminated due to crack control requirements under the weight of the cast-in-place topping slab. Cracking would most likely occur and TxDOT engineers did not find that acceptable. The post-tensioned panel was rejected, as well, because of complications with anchorage and bar sizes.

3.2.3 Construction Issues

In general, contractor and fabricator representatives expressed support for the possibility of using skewed panels. However, they indicated that there were limitations on their capabilities that would restrict using some of the proposed alternatives. Additionally, TxDOT, as the owner of the structures, has established standards for the panels currently being used.

3.2.3.1 Contractor Requests

The contractors were in unanimous agreement that using skewed panels at expansion joints would benefit the construction process. The panels would eliminate the additional time used to form and shore the current IBTS detail, as well as the time required to remove such forms. In certain circumstances, such as over water, removal of formwork from the underside of the bridge can become costly and time consuming. Additionally, eliminating the temporary hole in the unfinished bridge deck before the formwork is in place at expansion joints could reduce insurance costs and create a safer work environment. The contractors also claimed they would willingly pay a premium, if necessary, for the specialty panels. The primary requests were to use a single panel and limit the panel weight to 6,000 pounds. Using a single panel between each girder would reduce the handling and setting of awkward panels. By limiting the weight of the panels, the contractors would not have to upgrade the cranes or other equipment currently used to place panels. The contractors also rejected the idea to saw cut standard panels to custom angles because saw blades wear down quickly while cutting through prestressing strands.

Another area of concern associated with using panels at expansion joints was the permitted spacing between panels. Using the current IBTS detail, formwork can easily be constructed to match the location of the end panel (Figure 3.7a). With a precast panel, however, geometric control in setting panels becomes more important because the panel dimensions on site are fixed. A strip of compressible foam, known as backer rod, is typically used to fill any gaps between panels up to 3/4 in., but a gap in this situation could become too large if the geometry control is not accurate (Figure 3.7b). One proposed alternative is to saw-cut two conventional rectangular panels on site for a custom fit (Figure 3.7c). Because the cut would be oriented parallel to the strands, the saw blade would last longer.

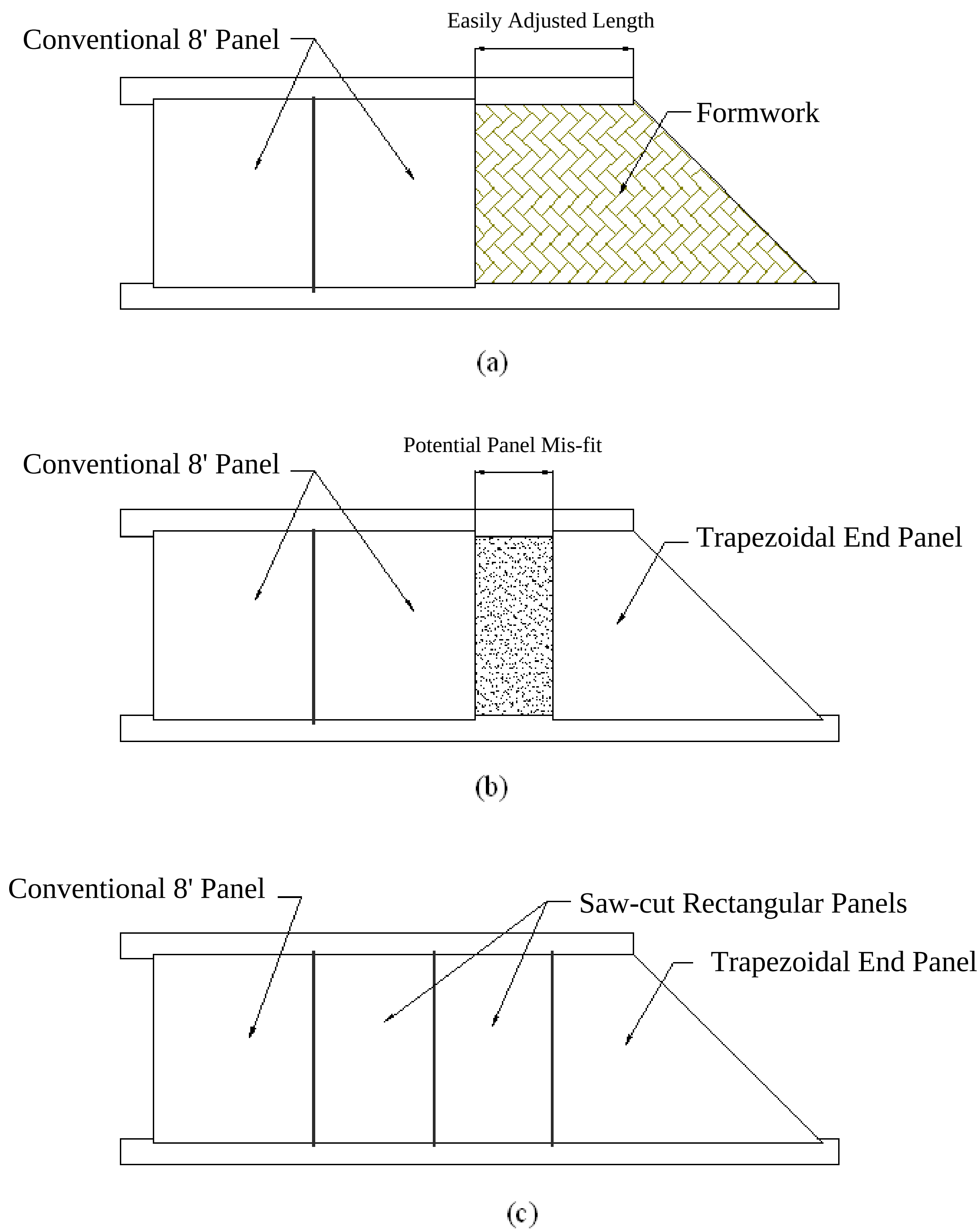


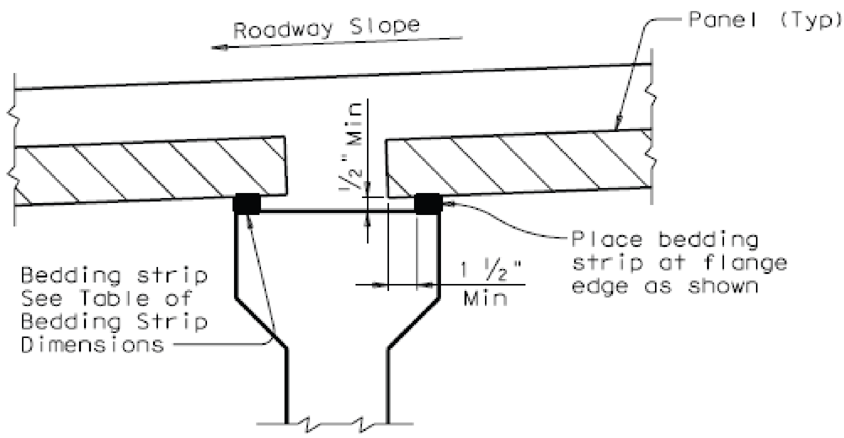
Figure 3.7: (a) Current construction techniques (b) Construction issue with end panels (c) Possible solution using two field-sawn panels

3.2.3.2 Fabricator Capabilities

The fabricator representatives indicated that producing skewed panels would not be difficult. They could easily produce custom wooden formwork for a given panel geometry as well as de-bond any strands necessary. De-bonding would be required if strands do not meet the minimum embedment length needed to transfer the prestressing force. The main problem they face is that the long lines used at the prestressing plants to produce the panels are permanently set to an 8 ft. width due to shipping restrictions. Anything wider than 8 ft. would necessitate special truck permits and cost extra money. However, many fabricators agreed that if the contractors were willing to pay the premium for the skewed panels, they would construct a new, wider casting bed to accommodate a wider panel dimension. Moreover, the fabricators claimed they could handle the shipping restrictions. These capabilities are dependent on the skewed panels having strands parallel to one another. For the flared prestressing strand alternative, no fabricator input was given. It was assumed that specialty casting beds would need to be constructed that would preclude mass production similar to current long-line methods.

3.2.3.3 TxDOT Requirements

The TxDOT bridge division representatives suggested that the panels utilize the current precast concrete panel standards as much as possible. This included concrete strength, the prestressing strand size, additional mild steel reinforcement, concrete release strength, panel thickness, and all bedding strip requirements (Figures 3.8 and 3.9). The other main variables were panel width, skew angle, and the short edge bearing length. Regardless of fabrication technique, the requirements needed to be flexible enough to accommodate variable skew angles and beam spacing. Current construction practice utilizes bedding strips with 40 or 60 psi strength, therefore, the TxDOT representatives wanted to keep bearing pressures within this range so that special materials need not be specified. Additionally, TxDOT expressed their concern for crack control at service load and preferred to see the prestressing strands parallel to the skewed end of the panels.



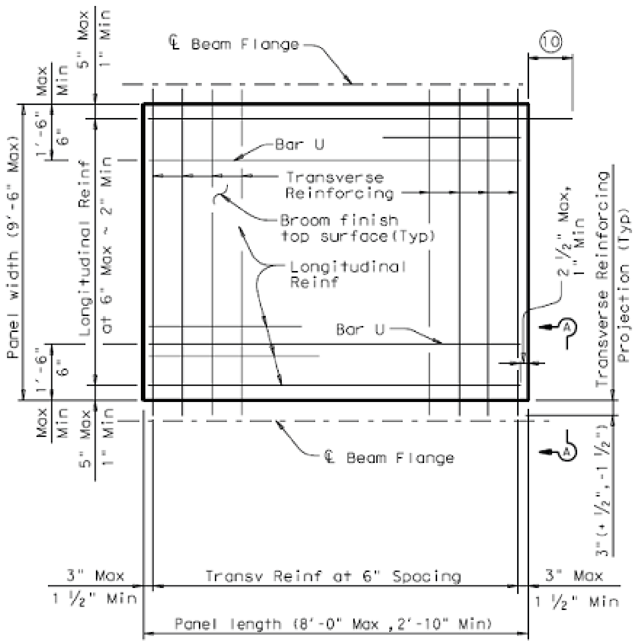
NORMAL GRADING DETAIL 1

Showing Prestressed Concrete I-Beams.
(Other Beam Types Similar)

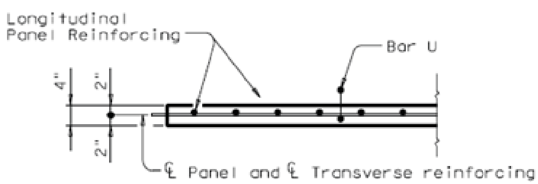
TABLE OF BEDDING STRIP DIMENSIONS

WIDTH	HEIGHT 2	
	Min	Max
1" (Min)	½"	2"
1 ¼"	½"	2 ½"
1 ½"	½"	3"
1 ¾"	½"	3 ½"
2" (Max)	½"	4"

Figure 3.8: Prestressed concrete panel bearing details



TYPICAL PANEL PLAN



SECTION A-A

FABRICATION NOTES:

All concrete for panels is to be Class H. Use Class H (HPC) concrete for panels if required elsewhere in plans. Release strength f'ci=4000 psi. Minimum 28 day strength f'c=5000 psi.

Remove laitance from top panel surface.

A minimum of 90 percent of the top surface area shall have the required broom finish.

Shop drawings for the fabrication of panels will not require the Engineer's approval if fabrication is in accordance with the details shown on this standard.

A panel layout which identifies location of each panel shall be developed by the fabricator. Permanently mark each panel in accordance with the panel layout. A copy of the layout is to be provided to the Engineer.

TRANSVERSE PANEL REINFORCEMENT:

For panel widths over 5', use $\frac{3}{8}$" or $\frac{1}{2}$" Dia (270k) prestressing strands with an initial tension of 16.1 kips per strand.

For panel widths over 3'-6" up to and including 5', use $\frac{3}{8}$" or $\frac{1}{2}$" Dia (270k) prestressing strands with an initial tension of 16.1 kip per strand. Optionally, #4 Grade 60 reinforcing bars may be used in lieu of prestressed strands.

For panel widths up to 3'-6", use #4 Grade 60 reinforcing bars (prestressed strands are not allowed).

Place transverse panel reinforcement at panel centroid and space at 6" Max.

LONGITUDINAL PANEL REINFORCEMENT:

Any of the following options may be used for longitudinal panel reinforcement:

1. (#3) Grade 60 reinforcing steel at 6" Max Spacing. No splices allowed.
2. $\frac{3}{8}$" Dia prestressing strands at 4 $\frac{1}{2}$" Max Spacing (unstressed). No splices allowed.
3. $\frac{1}{2}$" Dia prestressing strands at 6" Max Spacing (unstressed). No splices allowed.
4. Deformed Welded Wire Reinforcement (WWR) (ASTM A497) providing 0.22 sq in per foot of panel width. Wires larger than D11 not permitted. Provide transverse wires to ensure proper handling of reinforcing. One splice per panel is allowed. See WWR Splice Detail.

No combination of longitudinal reinforcement options in a panel is allowed.

Place longitudinal panel reinforcement above transverse panel reinforcement.

Figure 3.9: Prestressed concrete panel standard details

3.2.4 Selected Designs

Considering all of the options and opinions presented at the meetings, two types of skewed panels were selected to investigate. Because of the contractor requests, both types would be single trapezoidal panels that encompass the entire skew angle. The first alternative selected was the flared prestressing pattern. The second alternative has the prestressing parallel to the skewed edge with additional reinforcement perpendicular to the girders.

The current TxDOT maximum girder spacing is 10 ft. on center with a 9 ft. clear span between top flanges. With the minimum overhang of a precast panel over a flange equal to 3 in., the maximum panel width becomes 9 ft. 6 in. A new line of girders, TX-sections, that will soon be utilized in bridge designs permits girder spacing to extend up to 11 ft. on center. However, the new girders have wider top flanges creating an 8 ft. clear span between flanges. Therefore, a worst case condition of a 9 ft. 6 in. panel width was selected. Sketches of the two panel design options are shown in Figure 3.10.

Flared Prestressing **Prestressing Parallel to Skewed End**

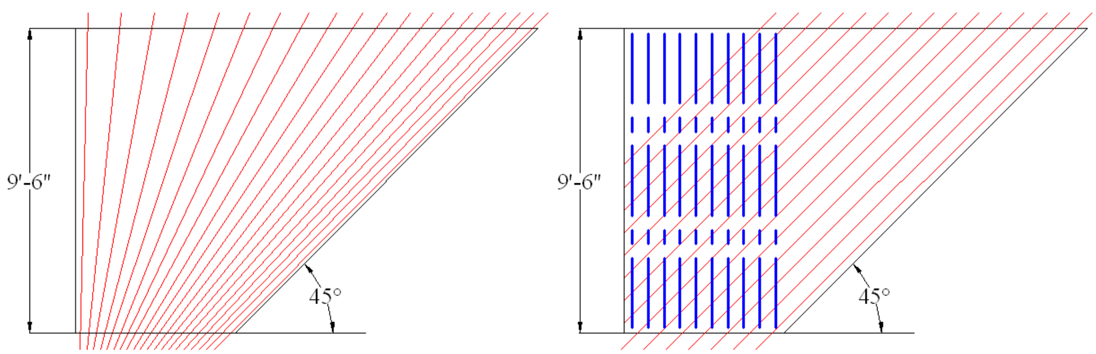


Figure 3.10: Selected design alternatives

In order to select a skew angle for design, bridge survey data from all bridges in Texas provided by TxDOT was reviewed. As seen in Table 3.1, targeting a 45 degree skew angle would encompass 97.9% of all bridges and 96.2% of prestressed I-girder bridges. This clearly covers a majority of bridge designs, therefore a 45 degree skew angle was chosen for the test program. Histograms showing the number of bridges with given skew angles are shown in Figures 3.11 and 3.12.

Table 3.1: Bridge skew angles in Texas

	Pretensioned I-Girders		All Bridge Types	
Total Bridges	8004		33201	
Skew	#	%	#	%
0°	3877	48.4%	21376	64.4%
≤ 15°	5095	66.7%	24058	73.6%
≤ 30°	6310	85.0%	28003	87.1%
≤ 45°	7055	96.2%	31164	97.9%

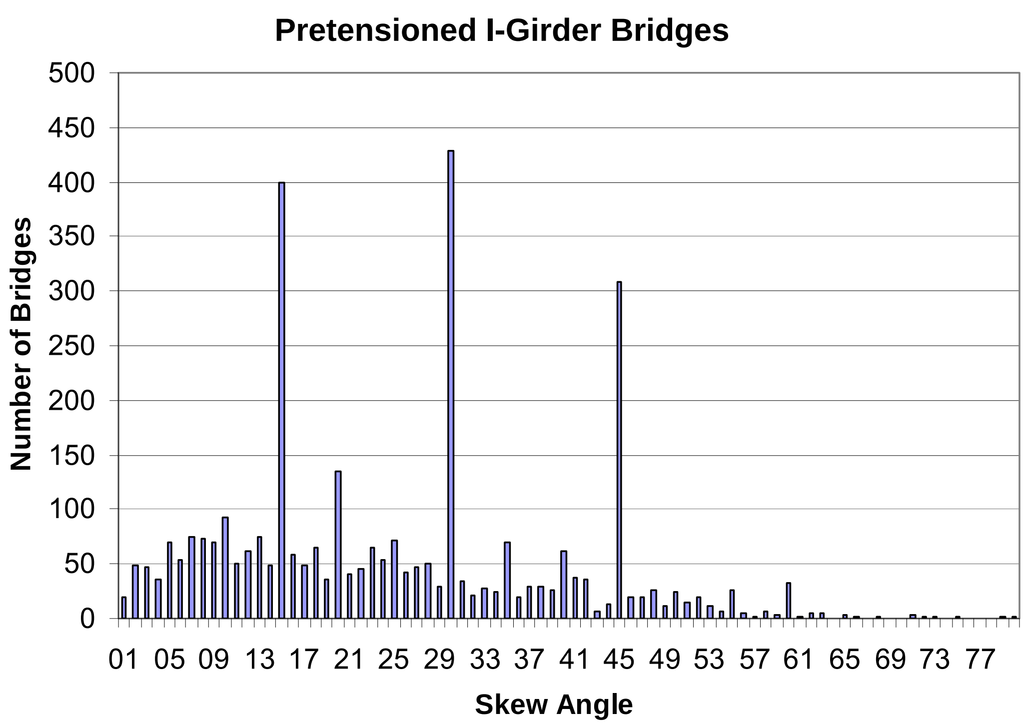


Figure 3.11: Number of pretensioned I-girder bridges with given skew angles (Van Landuyt 2006)

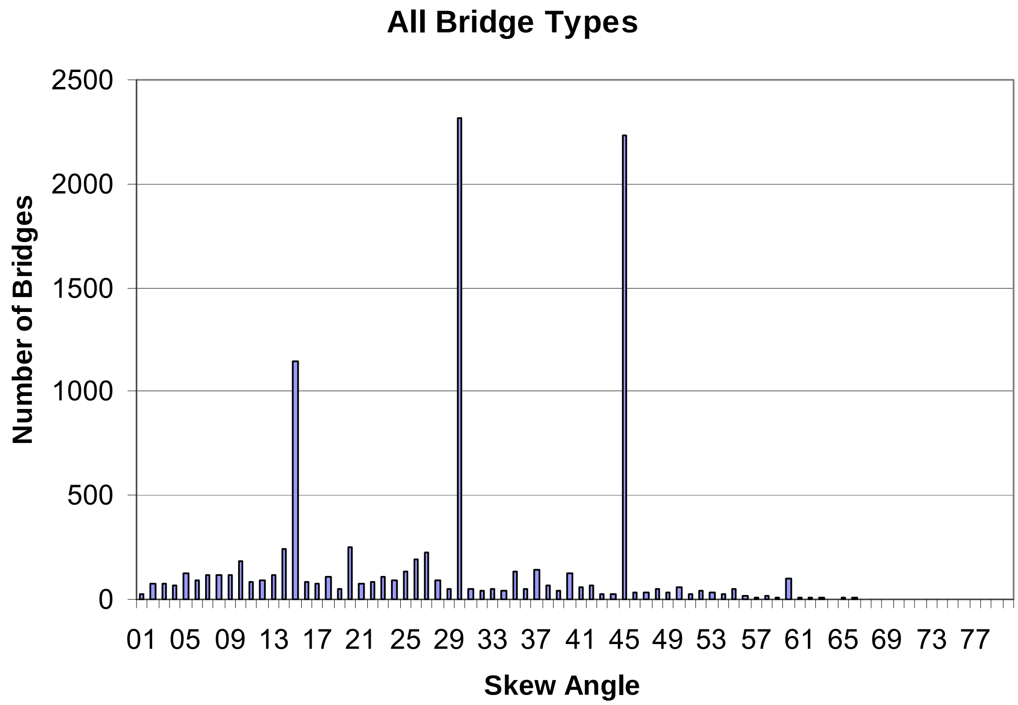


Figure 3.12: Number of all bridge types with given skew angle (Van Landuyt 2006)

Lastly, the short edge bearing length was determined based on bearing pressure calculations. For these calculations, the weight of the panel was conservatively assumed to have equal distribution to each girder. Furthermore, the bearing pressures were taken as equal along the length of each girder. Both 40 psi and 60 psi foam bedding strips are used in bridge construction, but 40 psi foam was selected as the more critical case. When using the TxDOT specified minimum width for rectangular panels of 34 in., bearing pressures for panels with large skew angles exceed 40 psi as shown in Table 3.2. Using the data shown in Table 3.3 the minimum bearing length of 55.8 in. for a 45 degree skew panel was rounded up to 60 in. for simplicity.

Table 3.2: Bearing pressures using TxDOT minimum 34 in. bearing length

Bearing Pressures Using 34" Short Edge Length				
Angle (degrees)	Short Edge Length (in)	Panel Weight (lbs)	Total Load (lbs)	Pressure (psi)
45	34	3602	7204	53
40	34	3239	6478	48
35	34	2926	5851	43
30	34	2648	5297	39
25	34	2398	4796	35
20	34	2167	4334	32
15	34	1950	3901	29
10	34	1744	3487	26
5	34	1543	3086	23

Table 3.3: Minimum short edge bearing lengths for 40 psi and 60 psi bedding strips

Required Short Edge Lengths				
Angle (degrees)	Foam Strength (psi)	Panel Weight (lbs)	Total Load (lbs)	Short Edge Length (in)
45	40	4466	8932	55.8
40	40	3747	7495	46.8
35	40	3127	6254	39.1
30	40	2578	5157	32.2
45	60	3367	6734	28.1
40	60	2825	5650	23.5
35	60	2357	4715	19.6
30	60	1944	3888	16.2

3.3 FLARED PRESTRESSING PATTERN

The first two specimens were designed using the flared prestressing pattern so that the entire panel was fully prestressed. The basic arrangement of the strands for this alternative is shown in Figure 3.6. The following sections describe the final design.

3.3.1 Strand Layout

As noted previously, the width of the panel was set to 9 ft. 6 in. with a 45 degree skewed end and 60 in. short edge length. This set the basic geometry from which to design the prestressing strand locations. The goal for this design was to produce a uniform peak stress along the length of the panel when loaded with fresh concrete during deck placement. Casting the topping slab for the deck was seen as a critical loading condition for the panel since it carries the entire load. Keeping the panel uncracked during this phase is essential to satisfactory long term performance. Once the topping slab is cured, the panel and slab act compositely.

In the flared pattern, the embedment length of each individual strand varies. This leads to slight differences in seating loss, elastic shortening, creep and shrinkage, and strand relaxation. Analysis using the strip method with trapezoidal-shaped sections was done to determine the exact spacing between strands. Because the strips analyzed were trapezoidal, the effective prestressed area varied along the length of the strip. This led to a non-uniform prestressed force from one edge of the panel to the other. The strand spacing on the short edge of the panel was set to 3 in. on center to reduce local stresses and provide more space for the chuck and barrel anchoring assemblies. The selected strand spacing is shown in Figure 3.13.

Using ACI 318-05, the maximum allowable tensile stress in prestressing steel due to the jacking force is 0.94 times the yield stress. The yield stress for prestressing strands is approximately 0.85 times the ultimate strength. Using 270 ksi prestressing steel, the maximum allowable stress becomes 215.7 ksi, which equals a jacking force of 18.3 kips on a 3/8 in. strand with cross sectional area of 0.085 in^2. Because seating losses are much more critical for shorter strands, a target jacking force of 18 kips was selected instead of the 16.1 kips specified on the precast panel standard drawings. A seating loss of 1/4 in.

was assumed, which could reduce the strand stress by as much as 20% for short strands. Creep and shrinkage coefficients of 2.9 and 0.0008 were selected and modified for a 60 day time frame to 1.56 and 0.000417, respectively.

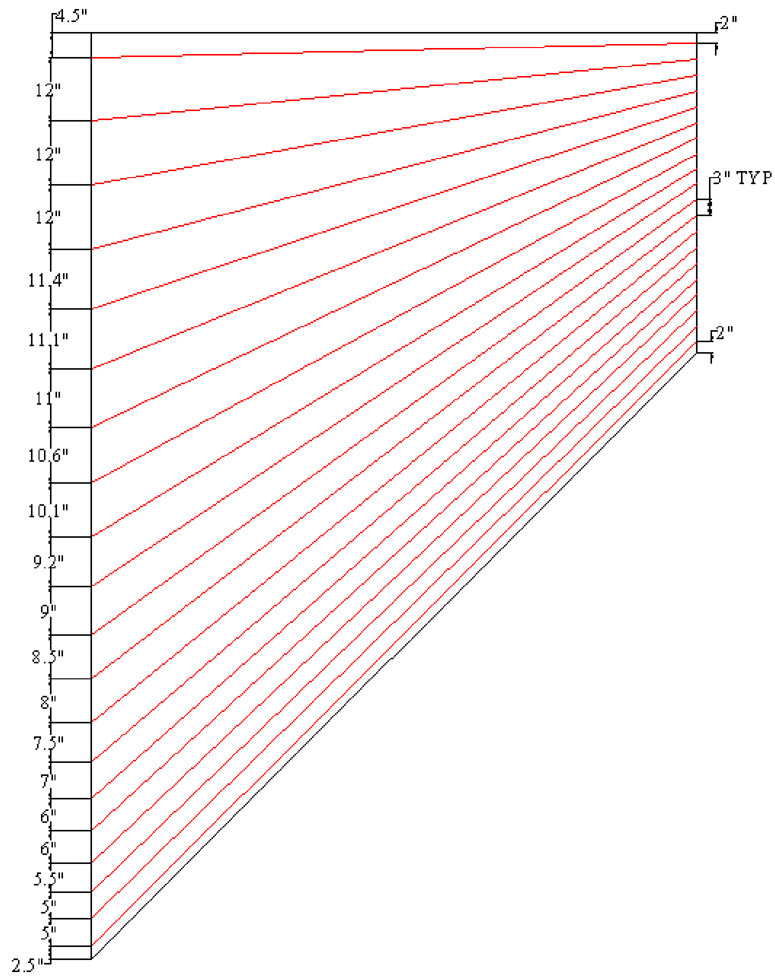


Figure 3.13: Strand spacing for flared strand pattern

3.3.2 Additional Mild Reinforcement

In the flared prestressing pattern, no two strands within the panel were parallel to each other. It was believed that this varying vector of compressive load would result in a splitting action between strands. No additional bursting steel is required for the current panels, but it is typical in other prestressing applications to prevent rupture and control cracking. To account for these conditions, #3 bars bent 180 degrees (hairpins) were placed between strands. A single hairpin bar could span the gap between several strands, so an equal number of bars to strands were not necessary. The hairpins arrangement, shown in Figure 3.14, was chosen so that they crossed the strands as close to orthogonal as possible.

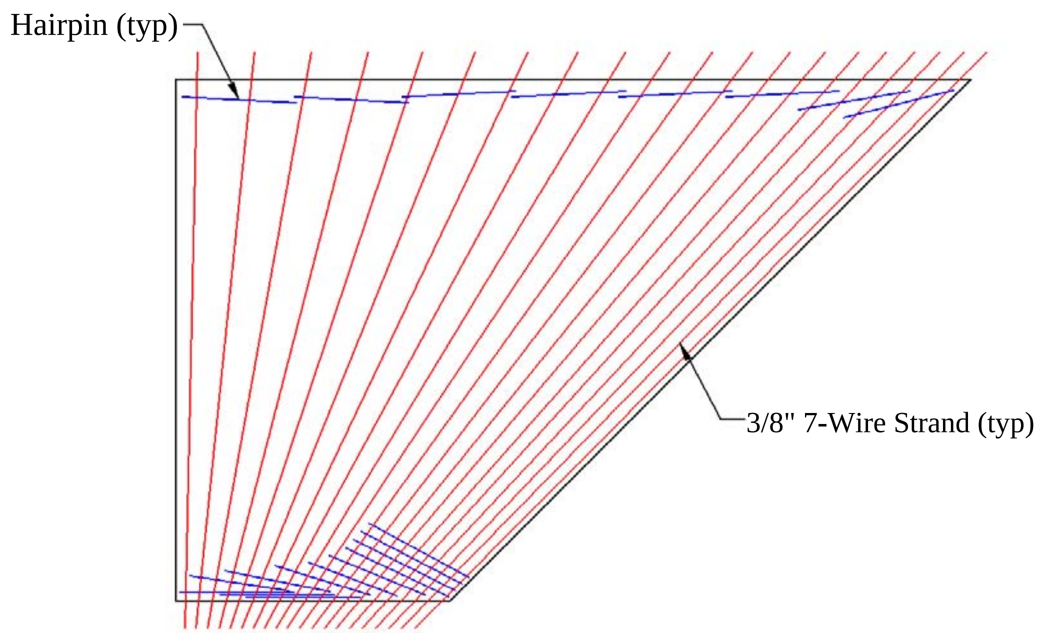


Figure 3.14: Hairpin layout for flared strand pattern

The first test specimen had 12 hairpins on the short edge and 8 on the long edge. Because the strands were spaced much further apart on the long edge, fewer hairpins were necessary. The second test specimen did not contain any hairpins in order determine whether they were necessary. To fulfill the longitudinal panel reinforcement requirements, #3 deformed bars spaced at 6 in. on center were used in each test specimen.

3.3.3 Release Strength

The TxDOT standard for concrete release strength in precast panels is 4,000 psi. Because of the high local compressive force at the obtuse and acute angles in the skewed panel, the first test specimen had a target release compressive strength of 5000 psi. The target compressive strength at release for the second specimen was 4,000 psi in order to match the current TxDOT standard for precast panels.

3.4 Parallel Prestressing Pattern

The second set of test specimens was designed with a prestressing arrangement in which the strands are parallel to one another as well as the skewed end. To account for partially prestressed or non-prestressed portions of the panel due to lack of strand embedment, supplemental deformed reinforcement was placed in this region.

3.4.1 Panel and Strand Geometry

Similar to the flared prestressing panels, the width of these panels was set to 9 ft. 6 in. to capture the largest beam spacing possible. Likewise, a skew angle of 45 degrees was chosen to include a majority of bridges in Texas. The first test specimen with the parallel strand arrangement had a short edge length of 60 in. to maintain continuity with the flared strand panels. Because this strand arrangement facilitates fabrication of smaller panels, the short edge length was reduced to 45 in. in a second specimen for comparison.

The motivation behind the strand layout was to match the current precast panel standard and casting lines at fabrication plants. Therefore, the strand spacing for the parallel pattern was set at 6 in. on center. A triangular region of the panel contains strands that do not meet the required embedment length to transfer the prestressing force. Using ACI 318-08 equation 12-4, 3/8 in. strands tensioned to 16.1 kips require approximately 24 in. of embedment to transfer the force. Fabricators typically use a de-bonding agent or simply wrap the strand when no force transfer is desired. For these test specimens, any strand that would have an embedment length less than 48 in. was omitted (24 in. from each face of the panel). As a result, the first test specimen with a 60 in. short edge length required 14 strands spaced at 6 in. The second test specimen only required 12 strands since the short edge length was 15 in. shorter. The strand layouts for both designs are shown in Figure 3.15.

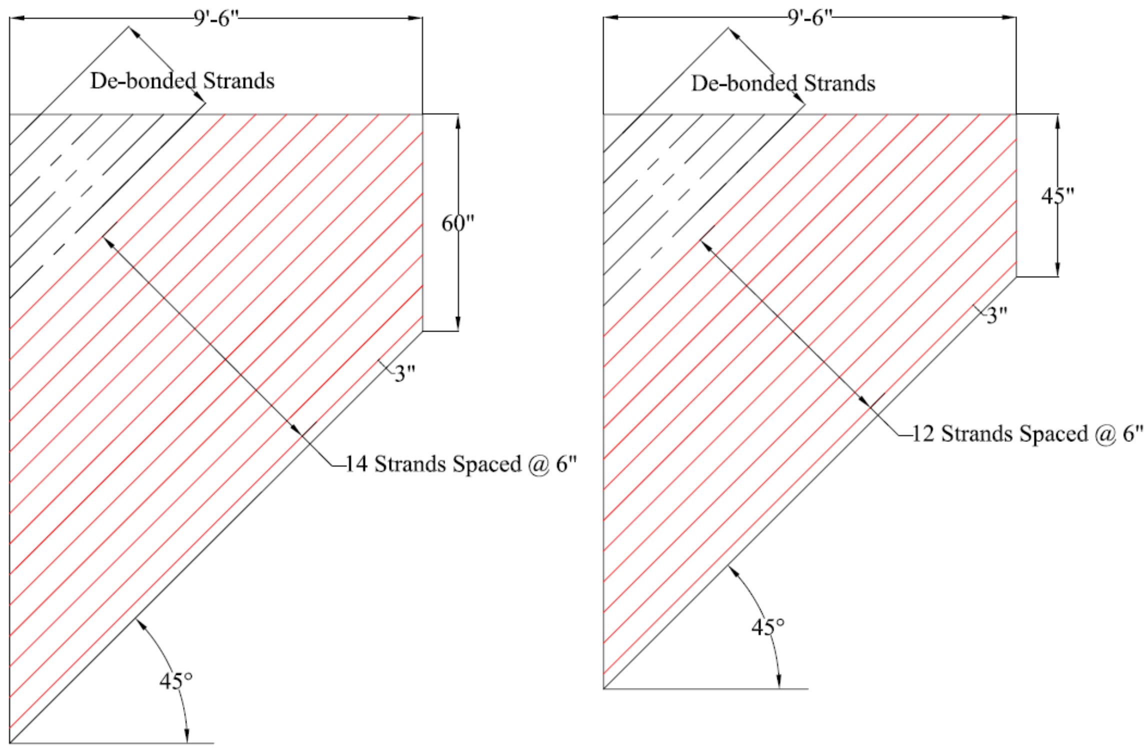


Figure 3.15: Parallel strand panel designs

3.4.3 Additional Mild Reinforcement

The strand spacing in these test specimens matched that of the TxDOT standard panels, so no additional bursting steel was used. However, these test specimens contained an entire corner region where no prestressing was present. To account for this, additional transverse mild steel reinforcing was used. To achieve a higher bending strength, the bars were placed beneath the prestressing strands. However, due to cover restrictions, the size of the bar was limited. The final design consisted of placing #4 bars with a 4 in. center to center spacing parallel to the non-skewed end. The number of bars used was selected to cover the entire non-prestressed region. The first specimen with the 60 in. short edge contained 14 transverse bars and the second specimen with a 45 in. short edge used 11 transverse bars. To fulfill the longitudinal reinforcement requirements, #3 deformed bars spaced at 6 in. were used in each test specimen. The ordinary reinforcing layouts for both designs are shown in Figure 3.16.

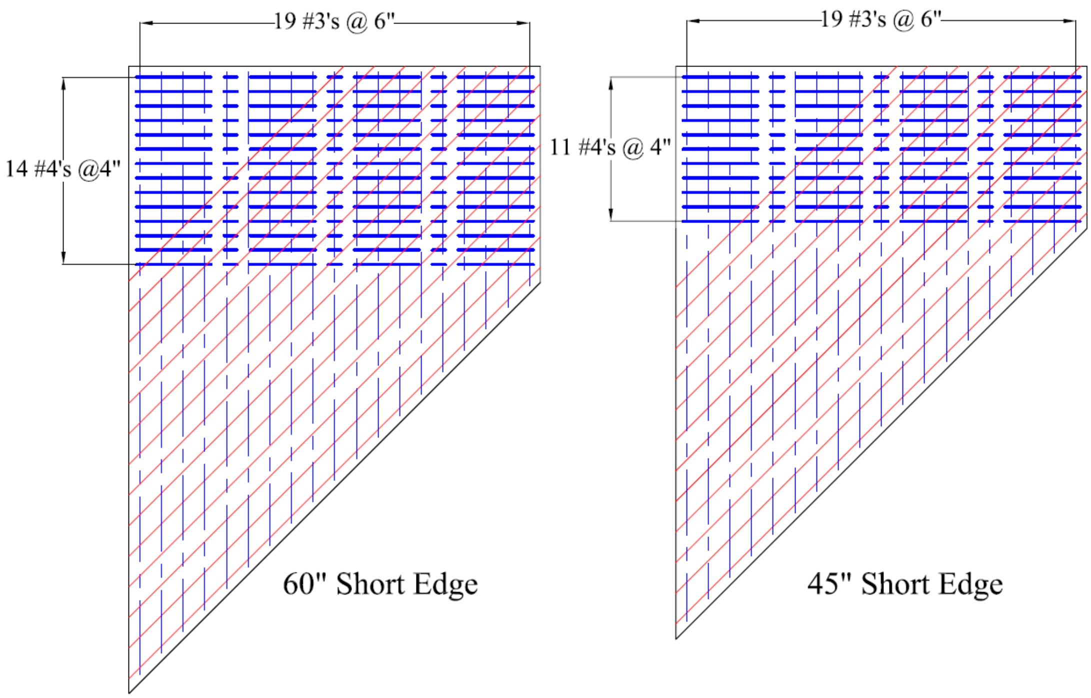


Figure 3.16: Additional deformed bars in parallel strand panels

3.4.4 Commercial Fabrication

In addition to the four test specimens fabricated by the researchers, another set of test specimens was fabricated by a precast plant. The design matched the 45 degree skew parallel strand pattern specimens, but because the precasting beds used for panels were only 8 ft. wide, the geometry of the panels was restricted. A skew of 30 degrees was selected which limited the short edge length to 45 in. However, for this skew angle, the bearing pressure on the bedding strip was deemed to be satisfactory. Figures 3.17 through 3.21 show the set of drawings sent to the precast concrete plant for fabrication.

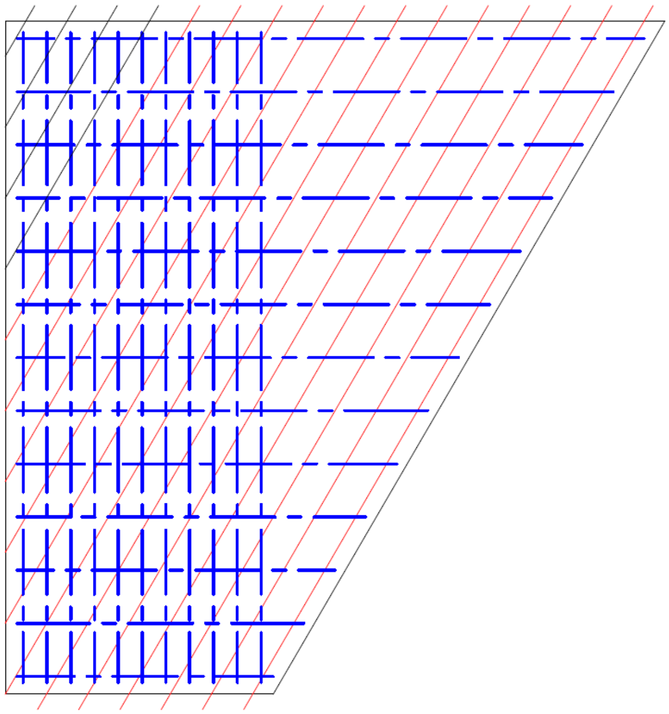


Figure 3.17: 30 Degree skew panel general view

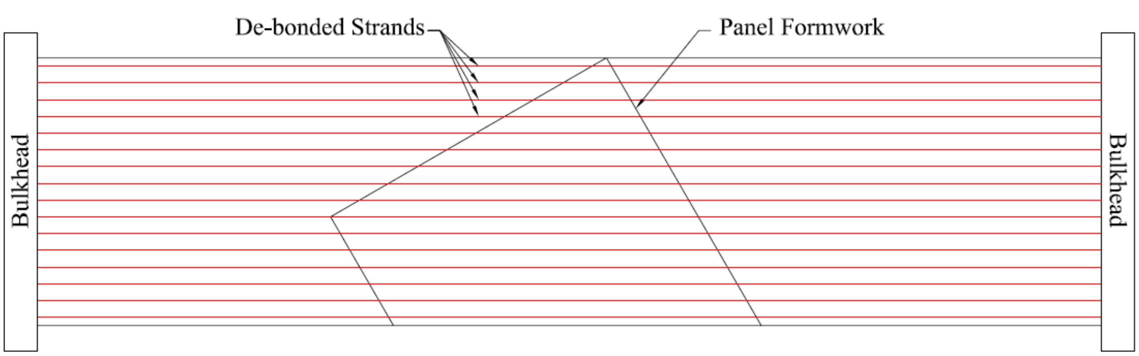


Figure 3.18: 30 Degree skew panel arrangement in prestressing bed

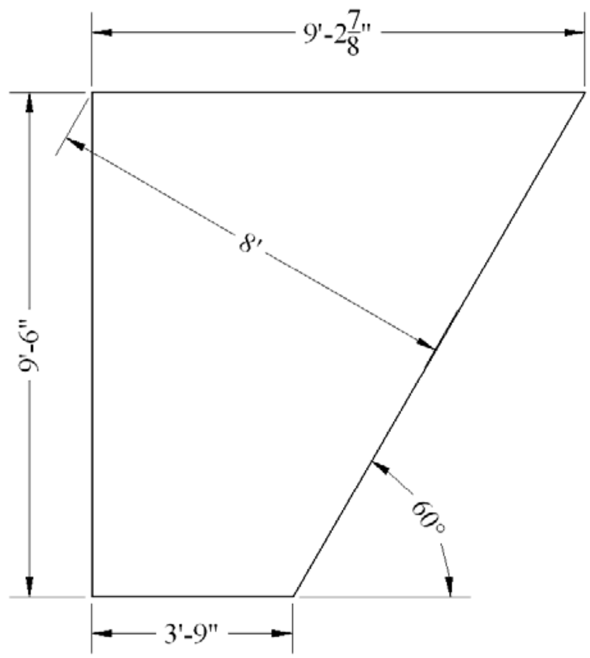


Figure 3.19: 30 Degree skew panel dimensions

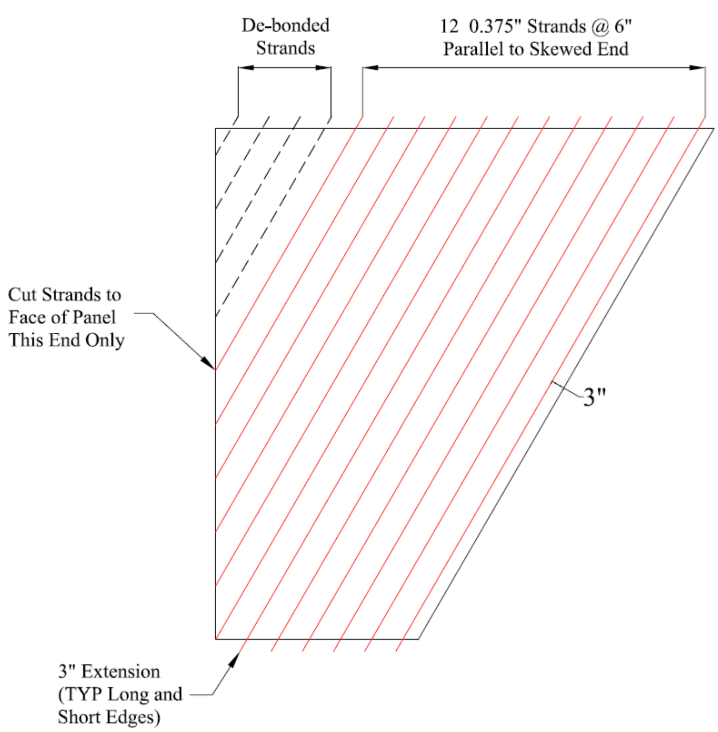


Figure 3.20: 30 Degree skew panel prestressing

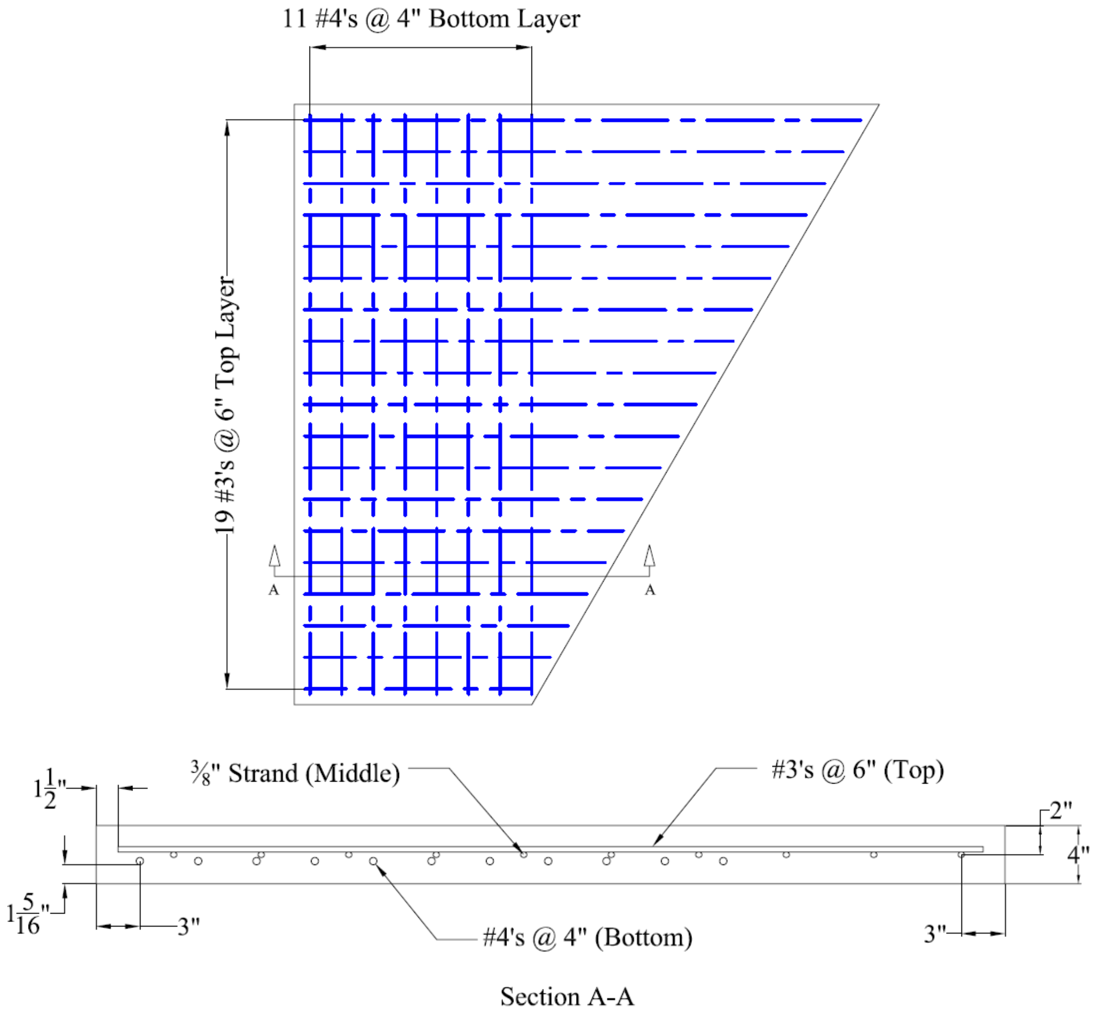


Figure 3.21: 30 Degree skew panel ordinary reinforcing layout and detail

3.5 Design Summary

A total of eight prestressed panels were produced using five different designs. All 45 degree skew panels were fabricated within Ferguson Laboratory. The 30 degree skew panels were commercially fabricated. Table 3.4 provides a summary of all panels produced. The nomenclature used to identify the panels is as follows: 1) Prestressed panel (P), 2) 30 degree (30) or 45 degree (45) skew, 3) Flared (F) or Parallel (P) strands, 4) 45 in. (45) or 60 in. (60) short edge bearing length. The number following the dash indicates the specimen number using that panel design. For example, the second panel with a 45 degree skew using the flared prestressing pattern and a 60 in. short edge length is designated P45F60-2.

Table 3.4: Summary of panel designs

Panel Name	Skew Angle (Degrees)	Short Edge Length (Inches)	Strand Pattern	Supplementary Reinforcement	Release Strength (psi)
P45F60-1	45	60	Array	Hairpins	5000
P45F60-2	45	60	Array	None	4000
P45P60-1	45	60	Parallel	Flexural	4000
P45P45-1	45	45	Parallel	Flexural	4000
P30P45-1	30	45	Parallel	Flexural	4000
P30P45-2	30	45	Parallel	Flexural	4000
P30P45-3	30	45	Parallel	Flexural	4000
P30P45-4	30	45	Parallel	Flexural	4000

3.6 Concrete Mix Design

The TxDOT required 28-day strength for precast panels is 5,000 psi. The mix design for the test specimens was obtained from a local panel fabricator in order to emulate typical practice. Because precast concrete plants rely on quick product turnover, Type III cement is used to achieve high early strengths. As a result, the 28-day strength of the mix design used was 7,500 psi. The concrete cubic-yard batch weights are shown in Table 3.5. The weights for the aggregates are based on saturated surface dry (SSD) conditions.

Table 3.5: Concrete mix design properties

Cement (lbs/yd^3)	SSD Fine Aggregate (lbs/yd^3)	SSD Coarse Aggregate (lbs/yd^3)	Water (lbs/yd^3)	Super Plasticizer (oz./100lb Cement)	Retarder (oz./100lb Cement)
658	1276	1776	250	6	4

CHAPTER 4

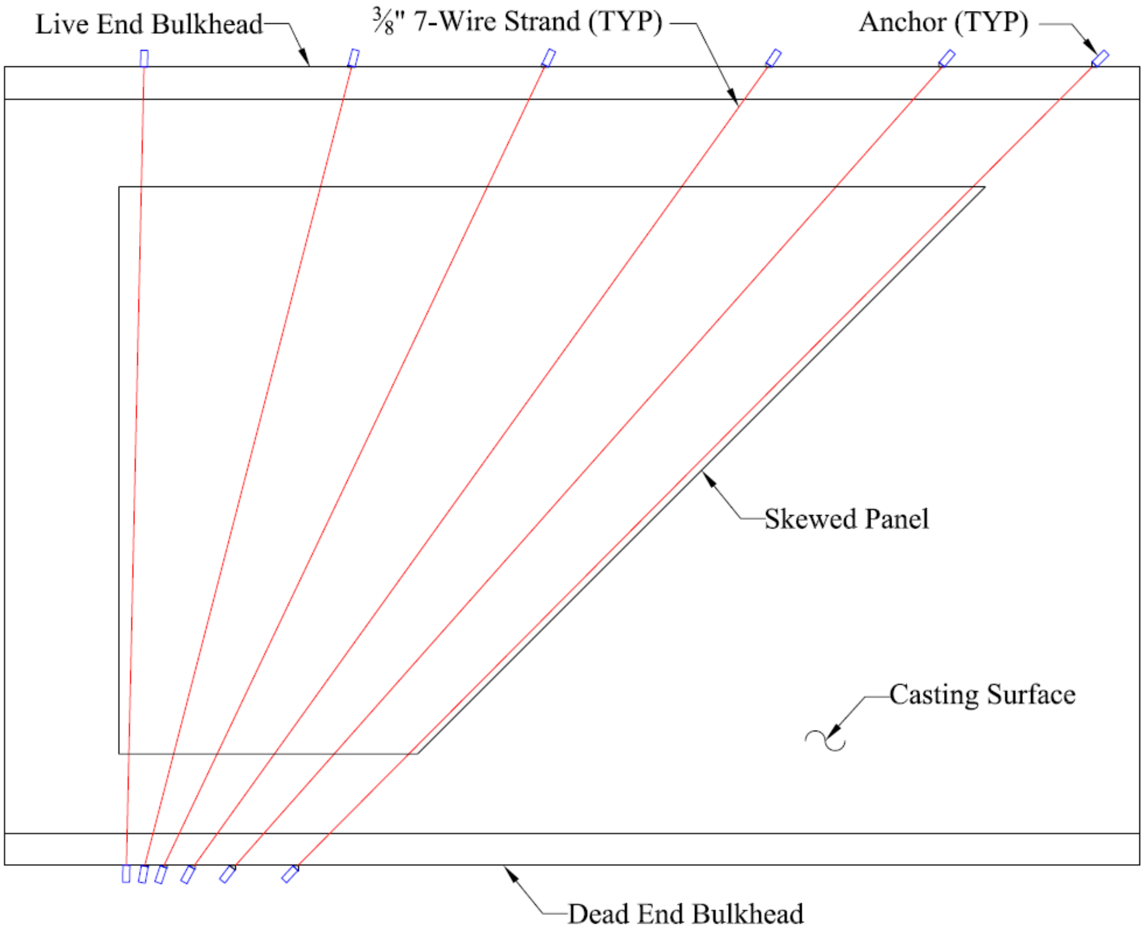
Prefabrication Activities

4.1 INTRODUCTION

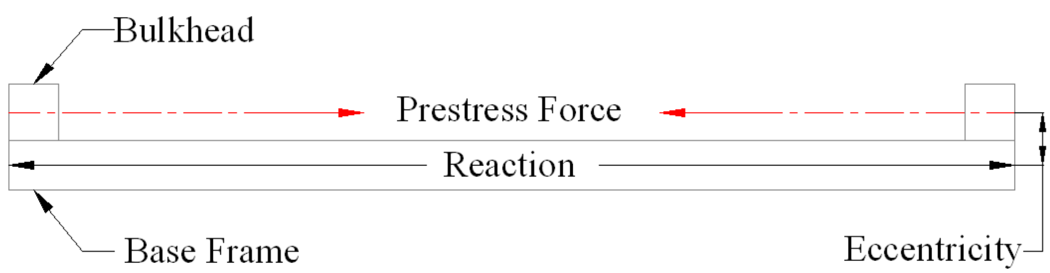
Before fabrication of trapezoidal prestressed panels could begin, production methods had to be developed. First, the construction of a self-reacting prestressing frame and the strand anchorage system is described. Next, the strand stressing operation and methods used to monitor stress levels, including instrumentation and calibration, are discussed. The concrete batching process and the techniques used to monitor early strength gain are also explained. Lastly, the procedure for releasing the strands once the concrete is cured sufficiently is discussed.

4.2 PRESTRESSING FRAME

With no fabricator in Texas currently equipped to produce a skewed prestressed panel, a self-reacting prestressing frame was designed and constructed for production of the panels at Ferguson Laboratory. The frame needed to contain a large open casting bed and to resist significant eccentric loads produced by the tensioned strands. Because the strands in the arrayed prestressing pattern are skewed relative to each other (Figure 4.1), the forces are multi-directional. With the help of RISA-3D modeling, the prestressing frame was designed using relatively small wide-flange and channel sections. These shapes were selected because they were relatively easy to handle, modify, and transport with the facilities available in Ferguson Laboratory.



a) Prestressing bed layout (not all strands shown for clarity)



b) Cross-section of frame showing eccentricity from prestressing strands

Figure 4.1: Prestressing bed schematic

4.2.1 Base Reaction Frame

The main structural frame for the prestressing bed consisted of W8x24 steel shapes. An 'X' pattern was chosen because it most efficiently resisted the large racking forces imposed by the skewed strands. A flat surface was essential to the placement of other frame elements so full penetration welds were used at all connections instead of bolted gusset plates. The arrangement of the elements in the base reaction frame is shown in Figure 4.2 and the completed frame is shown in Figure 4.3.

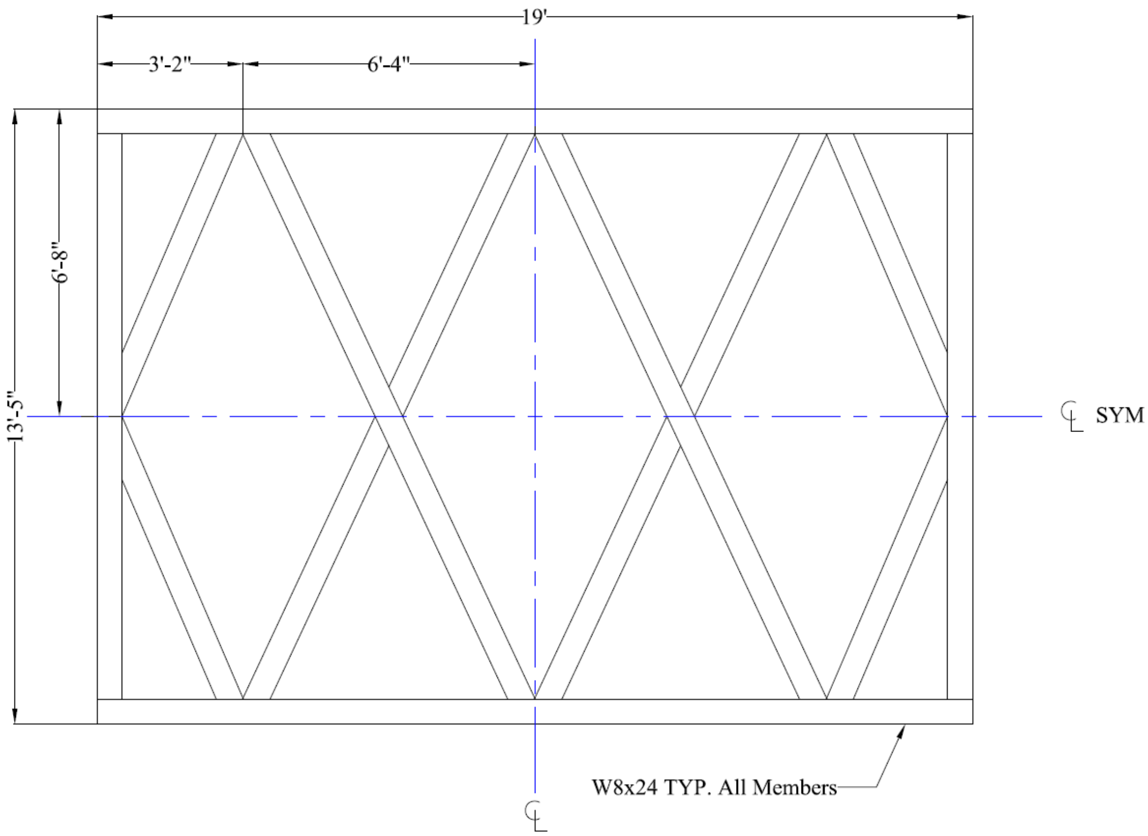


Figure 4.2: Base reaction frame

Figure 4.3: Completed base reaction frame

Covering the frame with thin steel plates to produce a casting surface was not a practical alternative for handling and economic reasons. Instead, a series of wide channels, C12x20.7 were selected because of their large width and small height (2.5 in.). These C12x20.7 shapes helped minimized the eccentricity of the load from the strands with respect to the frame. The channels were spaced at 19 in. to help provide a flat surface with minimal gaps. Figures 4.4 and 4.5 show elevation and plan views of the channel floor beams, respectively. A plywood decking was placed over the channels for a casting surface. The channels were attached at all intersections with the base reaction frame using fillet welds to help brace the base frame.

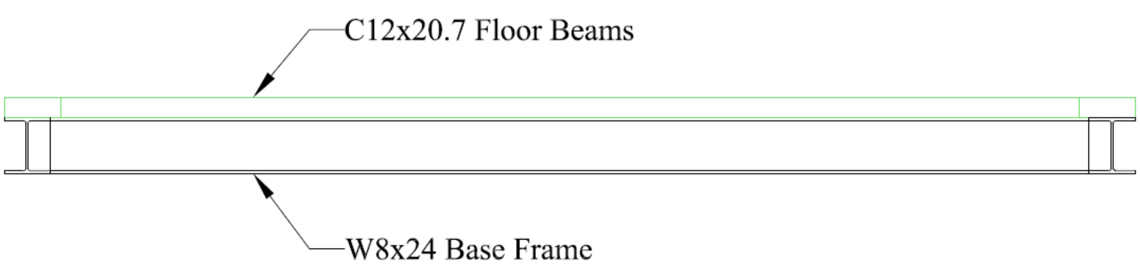


Figure 4.4: Elevation of base frame with channel floor beams

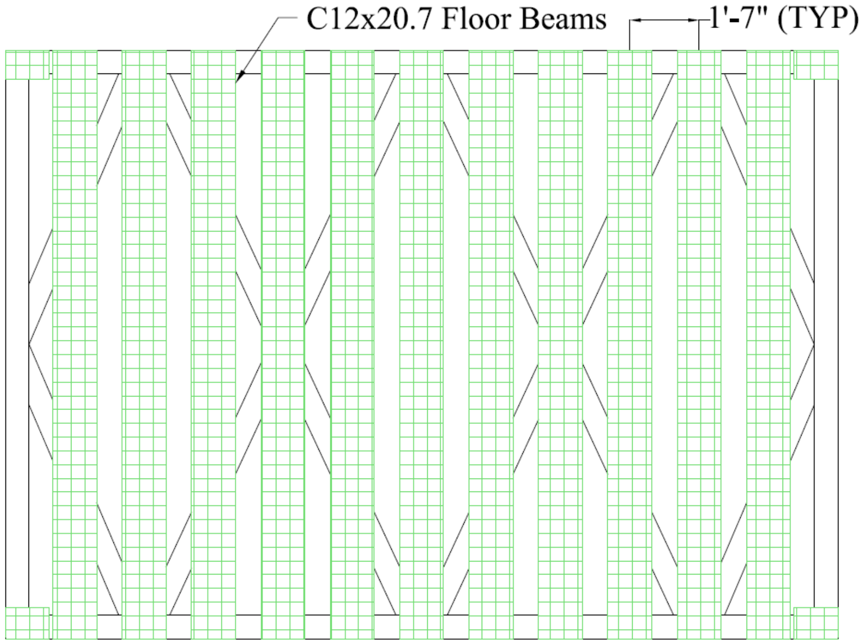


Figure 4.5: Plan view of base frame with channel floor beams

4.2.2 Bulkheads

Typical prestressing beds are built with bulkheads at either end denoted live end or dead end. The live end indicates at which end the tensioning will take place. The strand ends on the dead end are permanently anchored throughout the process. For ease of accessibility, the short edge of the trapezoidal panels was selected to always face the dead end bulkhead. Because the strands in the flared prestressing pattern pass through the bulkhead at variable angles, holes could not easily be drilled through a thick plate as is typically done in prestressing frames. Instead, back-to-back C8x18.75 sections were used with a 1 in. spacing between enabling strands to pass through freely. The shallow 8 in. section helped to reduce congestion of the anchors on the short edge of the panel by minimizing the distance between the face of the panel and the anchor location. Because the skewed strands converge onto one another as they extend from the short edge, this distance becomes critical. An illustration of this problem is shown in Figure 4.6.

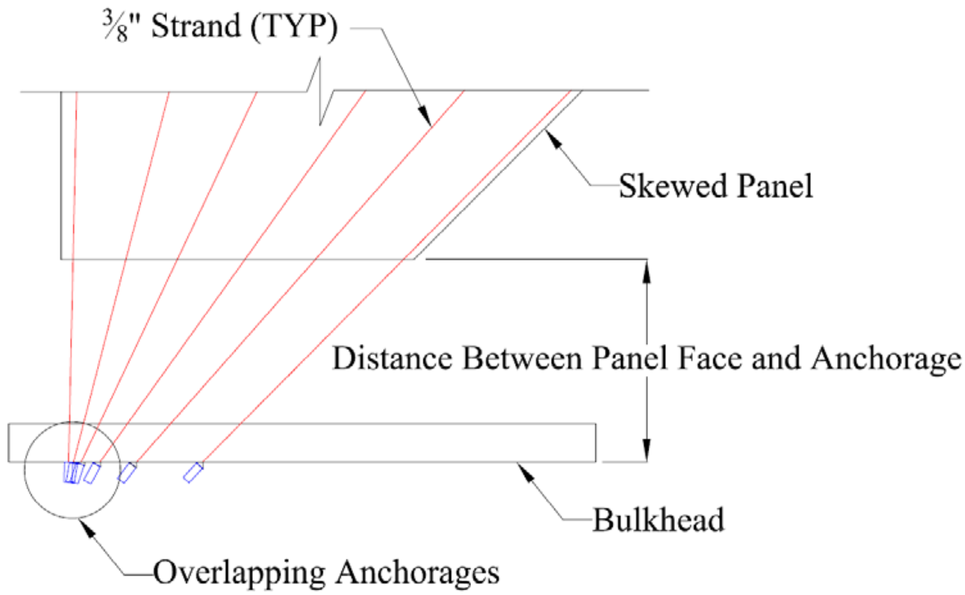


Figure 4.6: Overlapping dead end anchorages

In order to transfer the prestressing force to the structural steel base frame, the back-to-back C8x18.75 channels were bolted to the top flanges of the W8x24 wide-flange members of the base frame using high strength threaded rods as shown in Figure 4.7. Four 1-1/8 in. rods were used at every intersection with a decking channel on the dead side because of the grouping of strands over a short distance. Only two rods were used on the live end where the strands were spaced further apart. In the areas where strands also pass through the bulkhead, the bolts had to be precisely placed in order to avoid intersecting the strands.

To help transfer the shear force into the reaction frame, an additional 6 in. by 4 in. by 1/2 in. thick angle was placed on the inside of the bulkhead. This angle was bolted down to the decking channels using 6 bolts at every intersection. This was a stronger alternative than welding the bottom edge to the channels. The 4 in. leg of the angle was cut to 1-1/2 in. using a track torch so as not to obstruct the gap between the back-to-back channels. The completed casting bed with bulkheads is shown in Figure 4.8.

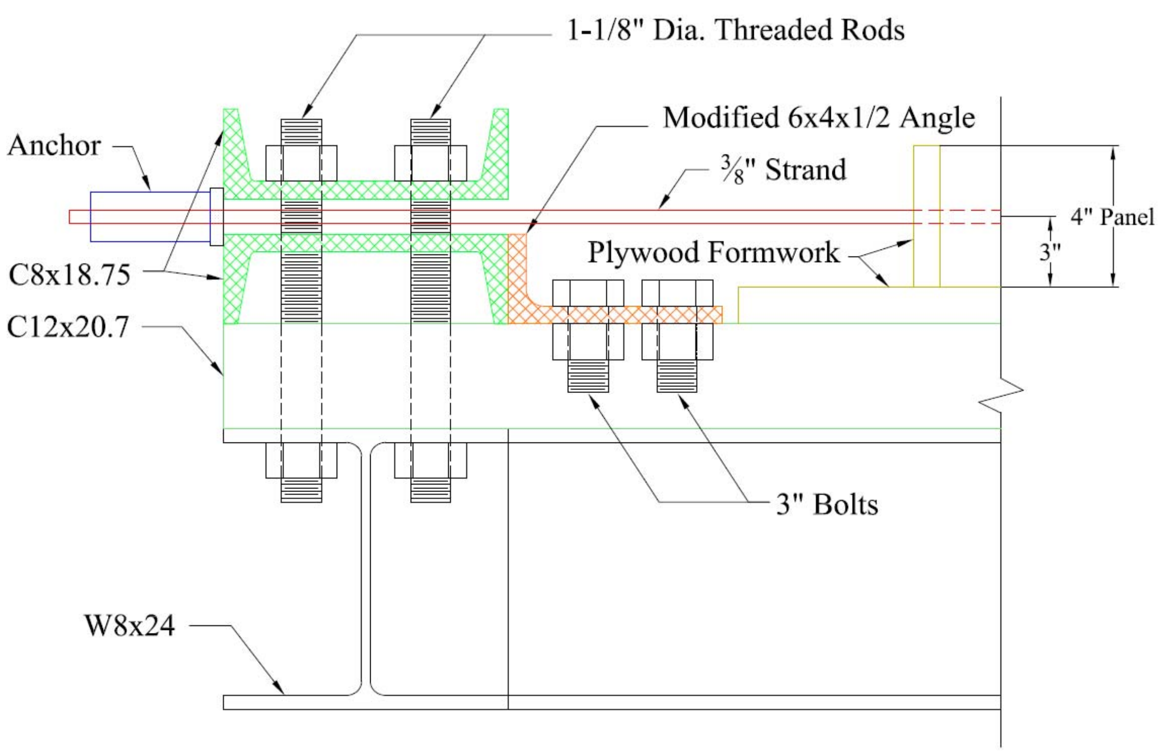


Figure 4.7: Cross-section of dead end bulkhead

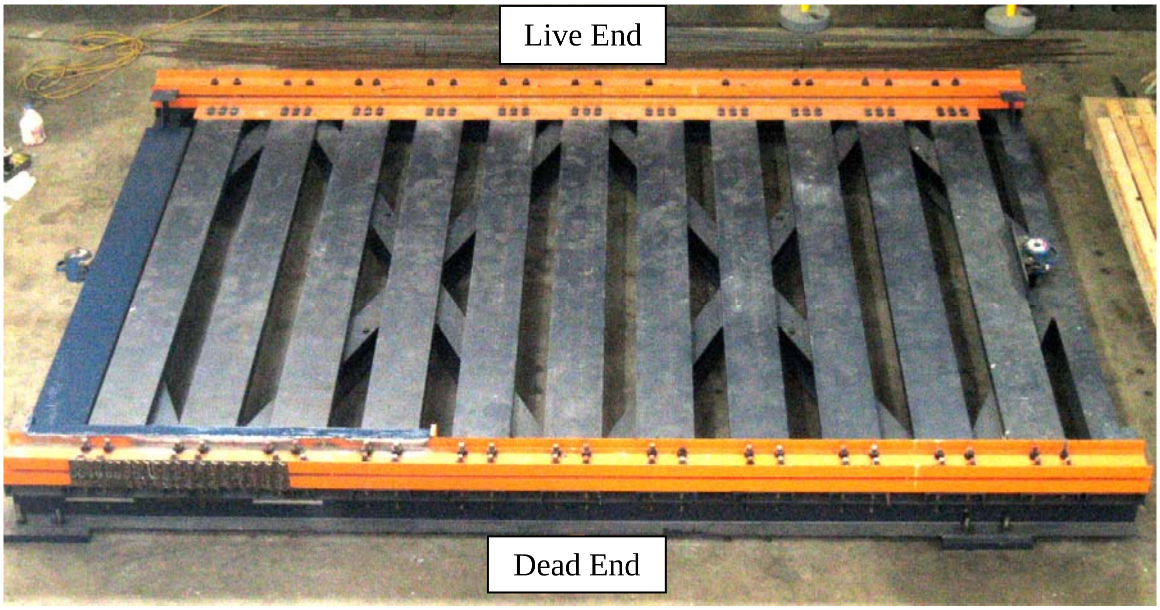


Figure 4.8: Prestressing bed prepared for flared panel fabrication

4.2.3 Anchoring

Standard multi-use chuck and barrel assemblies were used for anchoring the strands on both the dead and live ends. These assemblies consist of a set of wedges to grip the strand set inside a tapered steel cylinder with a spring loaded cap as shown in Figure 4.9. Due to the uniqueness of the flared prestressing pattern, a special bearing system for the chuck and barrels had to be designed. A similar, simplified design was used with the parallel prestressing pattern.

Figure 4.9: Typical chuck and barrel anchoring assemblies

4.2.3.1 Flared Prestressing Pattern

Devising a way to anchor twenty prestressing strands at different angles was challenging since it would have been nearly impossible to produce and set bearing plates with the precision required. Additionally, the anchoring region on the dead end was very congested because the distance between the non-parallel strands reduces as the distance from the panel increases. The solution was to use very thin spherically dished washers (Figure 4.10) in conjunction with angled shims and simple bearing plates. Because the washers had a rotational capacity of +/-2.5 degrees with the 3/8 in. strand, the angled shims were cut to the nearest 5 degree increment of the desired angle. With this system, the barrels aligned themselves during the stressing operation. The bearing assemblies were constructed by welding half of the spherical washer onto one side of a 3/8 in. thick bearing plate containing a 9/16 in. hole. The angled shim plates were then welded to the other side of the bearing plate. A sketch of a typical bearing assembly is shown in Figure 4.11. The entire bearing assembly was welded onto the bulkheads to resist the high shear forces associated with stressing. To further reduce congestion, a set of narrow chuck and barrel assemblies was used on the dead end while ordinary assemblies were used on the live end. Photos of the completed dead and live end anchorages are shown in Figures 4.12 and 4.13, respectively.

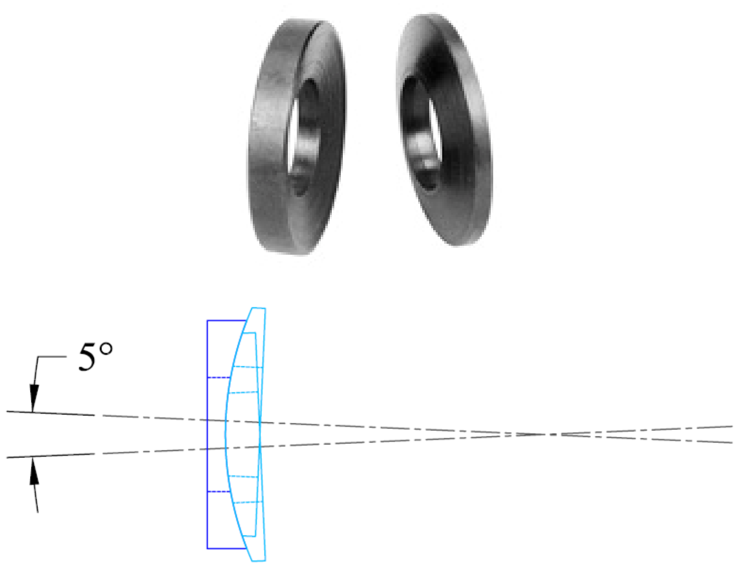


Figure 4.10: Spherically dished washers with diagram showing rotational capacity

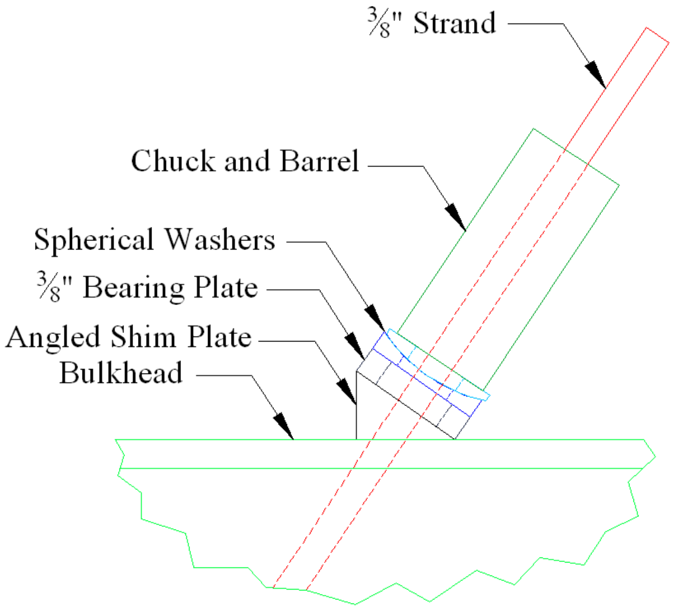


Figure 4.11: Sketch of skewed anchor assemblies

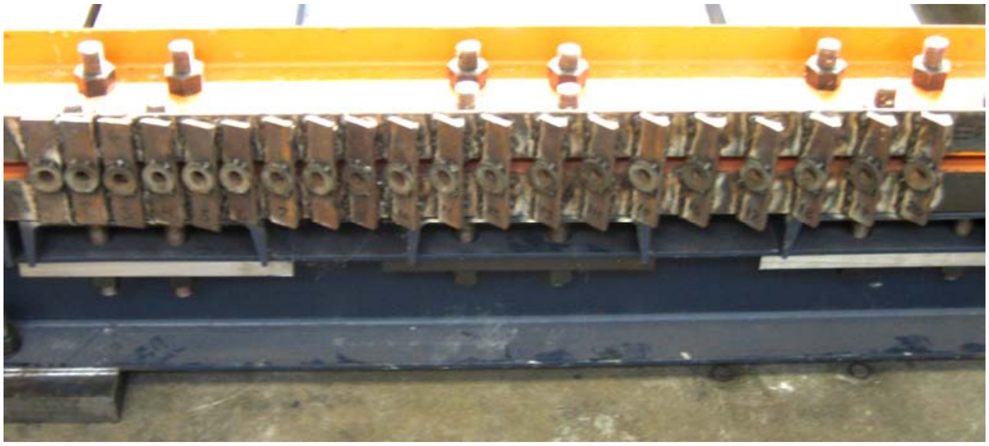


Figure 4.12: Completed dead end anchors for flared strands

Figure 4.13: Portion of completed live end anchors for flared strands

4.2.3.2 Parallel Prestressing Pattern

All of the strands in the parallel prestressing arrangement were the same angle with respect to the bulkhead, making bearings less complicated. Bearing seats were cut out of 2 in. thick plate to match the required angle. Holes were drilled through the blocks to permit the strand to pass through. The bearing blocks were then welded directly to the bulkheads to resist the shear forces from stressing. Photos of the typical bearings are shown in Figures 4.14 and 4.15. No dished washers or shim plates were necessary because the angle created by the bearing block was accurate enough. Instead, the chuck and barrel assemblies were seated directly against the bearing blocks.

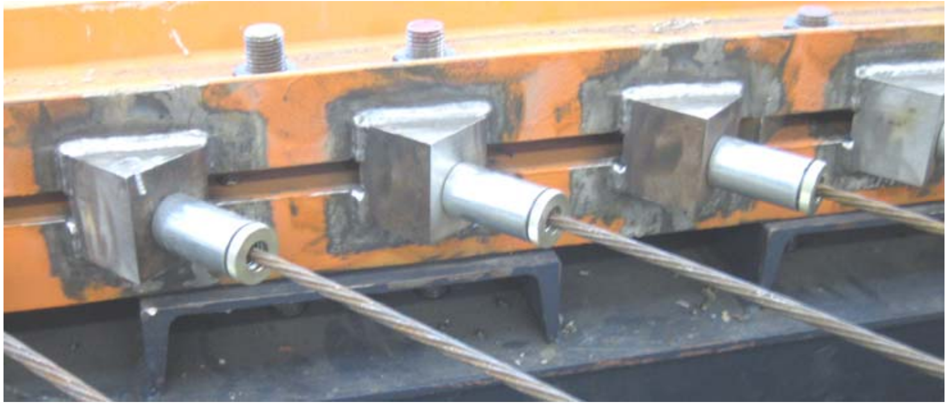


Figure 4.14: Typical bearing for parallel strands

Figure 4.15: Plan view of typical bearing for parallel strands

4.2.4 Modifications

During the first stressing operation the prestressing frame began to deform. Due to the large diagonal, eccentric forces from the strands, opposing corners in line with the longest strand lifted off the ground. To prevent this problem from recurring, large steel plates were welded to the bottom of the frame so that the entire bed could be bolted to the floor (Figure 4.16). No further uplift or undesirable deformations were encountered.

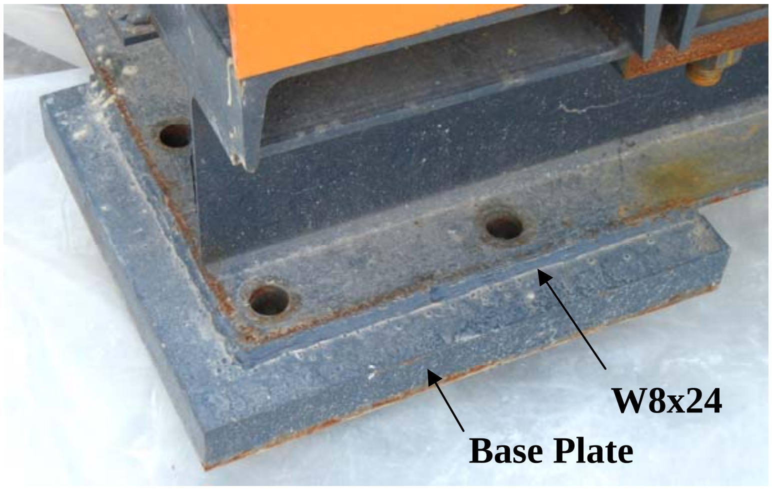


Figure 4.16: Typical base plate with holes to bolt to strong floor

Secondly, the top bulkhead channel on the dead end deflected inward towards the panel. The threaded rods as well as the bearing plates bent along with the channel. To avert this problem, another large steel plate was cut to fit directly against the top bulkhead channel and welded into place along with vertical stiffeners at either end (Figure 4.17). After completion, deflection in the top bulkhead channel reduced dramatically and no further problems were encountered.

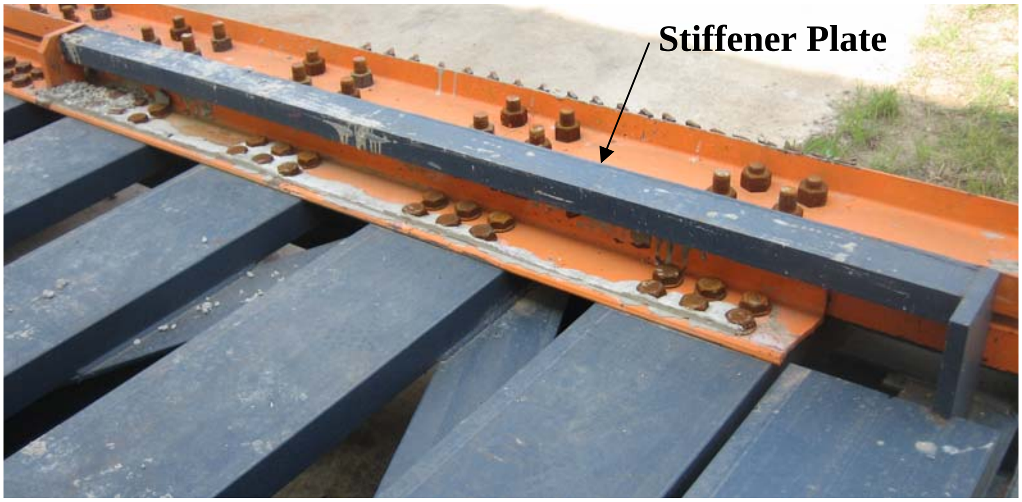


Figure 4.17: Stiffener plate

4.3 Instrumentation

In order to monitor the stress levels in the prestressing strands both during the stressing operation and testing, strain gages were attached near the mid-span of each strand as illustrated in Figure 4.18. Additionally, strain gages were attached to deformed bars (Figure 4.19) used in some of the panels.

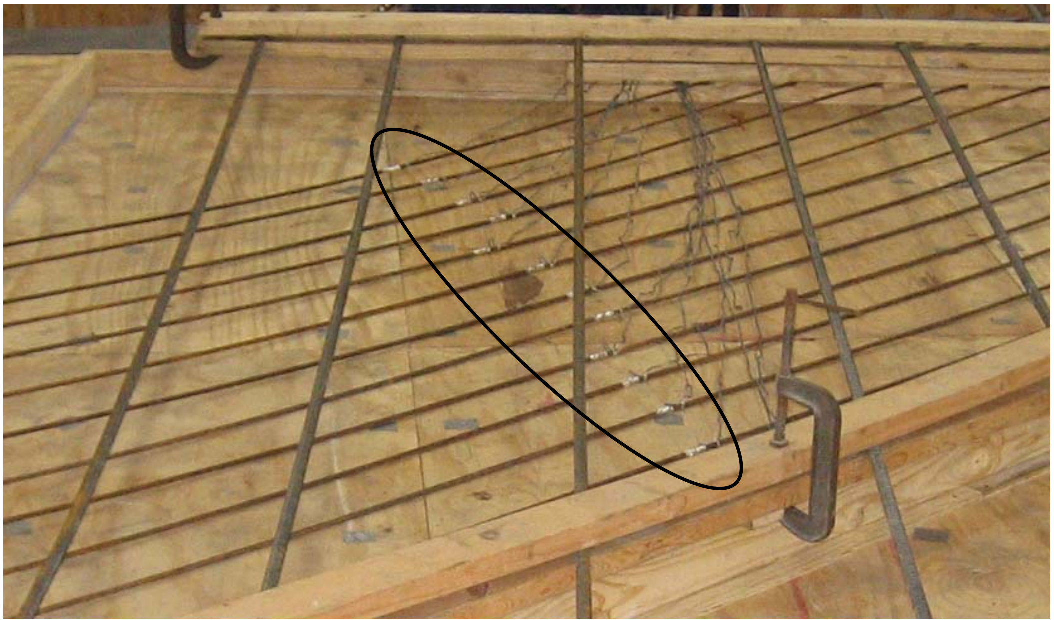


Figure 4.18: Strain gages attached to the strands

The strain gages applied to a single wire on the strand were 5 millimeters long with 5 meter, pre-attached lead wires. The prestressing strand was smoothed using high grade sand paper to minimize the amount of material removal. The area was then cleaned with a conditioner and neutralizer. The gage itself was bonded to the strand using cyanoacrylate adhesive. An acrylic coating was used to waterproof the strain gage. For protection against damage during stressing and concrete placement and to further waterproof the strand, a neoprene rubber pad was placed over the gage and held in place with foil tape.

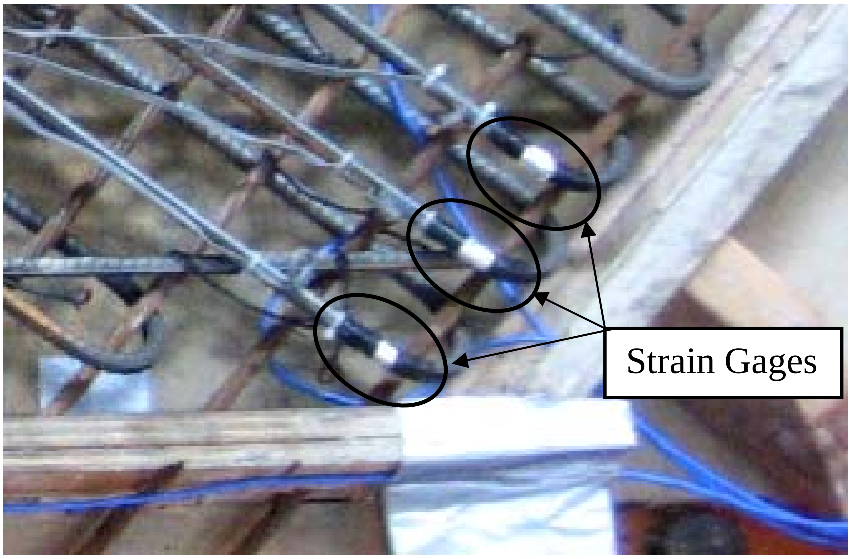


Figure 3.19: Strain gages attached to hairpins

4.4 PRESTRESSING STRAND TESTS

Strain gages applied to the individual wires on the prestressing strands were not perfectly aligned with the direction of force due to the helical arrangement of the wires. Therefore the strain gage data had to be calibrated to obtain an accurate force value. To facilitate this process, a set of tests was done beforehand using a tensile testing machine. The ends of three 6 ft. long strands were grouted into an 18 in. long, 1 in. diameter pipe, which was gripped in the testing machine. Using the strain gage output together with the load readings, an apparent modulus of elasticity of 29,500 ksi was calculated. The correlation between the calibrated load calculation and the actual load is shown in Figure 4.2.

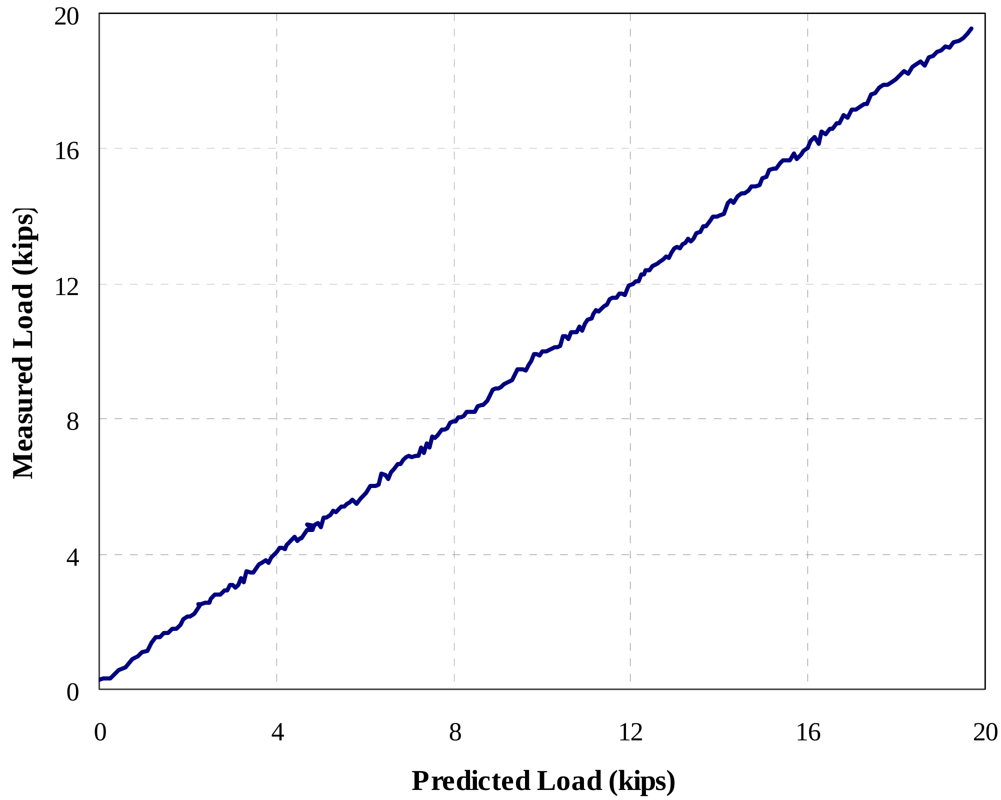


Figure 4.20: Calibrated strain data using apparent modulus of 29,500 ksi

4.5 STRAND STRESSING

To apply the necessary force for prestressing the strands, a 45 kip hydraulic jack with two hand pumps was used. A picture of the apparatus is shown in Figure 4.21. The jack bears against the live end anchorage while pulling the strand. To lock the strand in place, the wedges inside the barrel are forced down the tapered interior walls using a second pump on the jack. Once seated, the pressure in the pump is relieved and the jack removed. Because the strands were stressed individually and the frame has some inherent flexibility, the strands were stressed in two phases. All strands were stressed sequentially to 50% of full force to remove any slip in the frame and then all strands were stressed to 100%. The strands were tensioned in consecutive order starting and finishing with the strand closest to the non-skewed end. This allowed the operators to always be away from fully stressed strands. For additional safety, a set of six #8 reinforcing bars was clamped orthogonal to the strands to prevent a broken strand from whipping, should a failure occur.

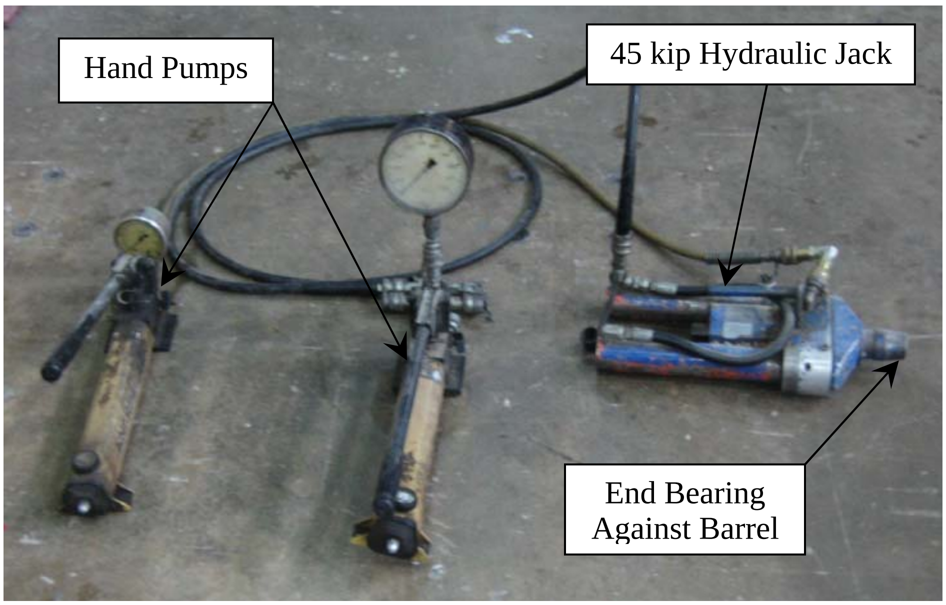


Figure 4.21: Hydraulic jack apparatus

Throughout the stressing operation, a data acquisition system was used to monitor the strains in the strands so that the required jacking force could be applied. The first set of strands to be stressed was governed mostly using strain gage data rather than pressure readings from the hydraulic system. However, due to the inherent gage variability and possible damage to gages, a target pressure was chosen during the first stressing operation that corresponded well with the data. The jacking force in the strands for the remainder of the panels was then monitored by pressure readings.

4.6 CONCRETE BATCHING

The concrete mix-design was selected to match that of a local producer of precast panels. Because the precast plants produce the panels as fast as possible, they utilize Type III cement to achieve a high early strength. Local ready-mix concrete producers do not currently offer concrete with Type III cement, so the concrete for the panels was mixed at Ferguson Laboratory.

To ensure a good mixture, three cubic yards of concrete were made, whereas the largest panels only required 1.15 cubic yards. The mixing truck arrived from the ready-mix plant with the coarse and fine aggregate already loaded. On arrival at the laboratory, approximately 80 gallons of water was added to the truck. Next, 21 sacks of Type III cement were loaded into the truck while the barrel rotated at about 75% of mixing speed to get the cement down to the bottom of the barrel. The remaining 10 gallons of water was added to flush any cement remaining in the hopper down into the truck. The speed of the barrel was then increased to full mixing speed for 200 rotations. The retarding and super-plasticizing admixtures were then added and the batch was mixed again at full speed for another 100 turns.

4.8 EARLY CONCRETE STRENGTH

The time taken for the concrete to reach the desired release strength was monitored. To get an accurate estimate of the early strength gain, two trial batches were made and 4x8 in. cylinders were tested in accordance with ASTM C39 standards at early hours to determine the compressive strength. The results showed that the concrete reached 4,000 psi in roughly 13 hours and 5,000 psi in approximately 18 hours (Figure 4.23). However, the amount of retarder, ambient temperature, water content of the aggregate, as well as other factors contribute to the early strength gain, so cylinders were made for each panel and tested before releasing strands to ensure proper strength.

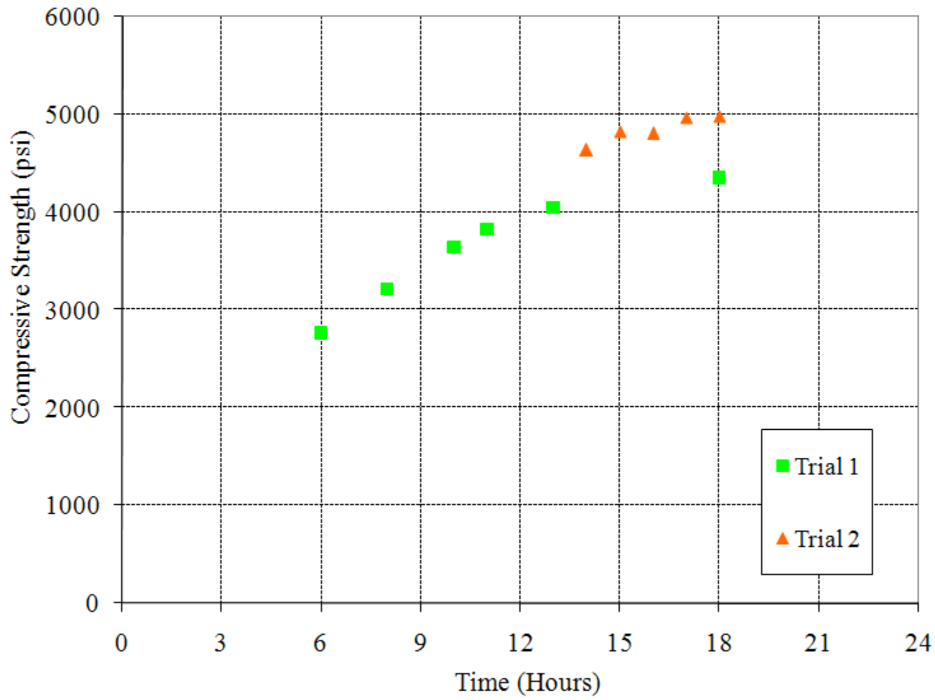


Figure 4.22: Cylinder test results from trial batches

4.9 Release

The strands were cut using an acetylene torch after the concrete had gained sufficient compressive strength. To help prevent cracking in the middle of the panel, the strands were released individually on alternate edges. Figure 4.24 displays a schematic of the strand cutting order. Due to accessibility, the strands were cut on the live end for the panels with flared strands, and the dead end for panels with parallel strands. To cut the strands the red flame (low heat) was gently passed over roughly 12 in. of the exposed length if possible, making the release process less violent. Using this method, the force from the strand was gradually transferred to the panel to reduce the possibility of cracking at release.

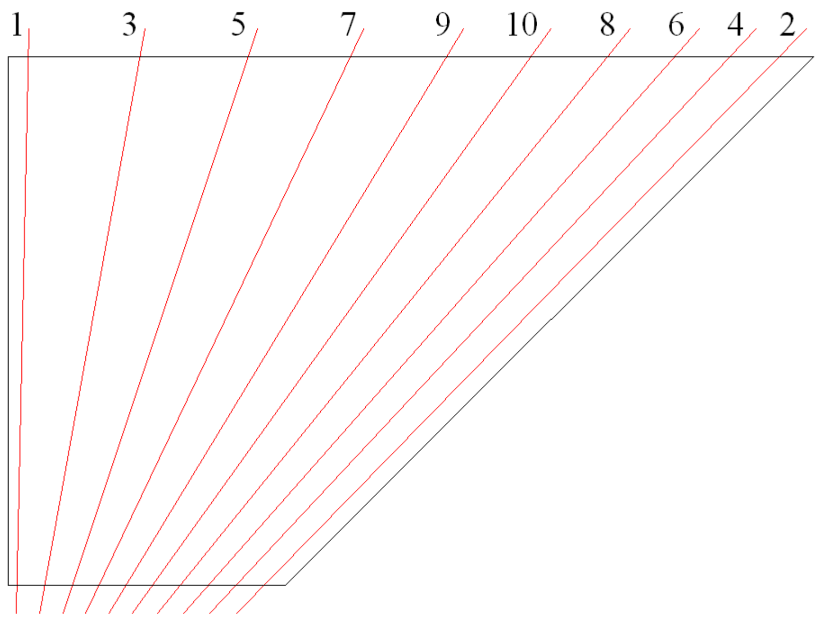


Figure 4.23: Typical strand release order (not all strands shown for clarity)

4.10 SUMMARY

The prestressing frame, bulkheads, and anchorages developed for this research project facilitated the production of trapezoidal panels with skewed prestressing strands. The strain gages applied to the prestressing strands allowed the stress in the strands to be monitored during the stressing operation, concrete curing, and future load testing. Finally, the concrete batching process and strand release methods ensured replication of typical concrete strengths and release stresses found at precast plants. With all of the necessary apparatus and procedures established, prestressed panels could be produced.

CHAPTER 5

Construction of Prestressed Panels

5.1 INTRODUCTION

Four 45 degree skew panels were fabricated using the custom prestressing bed described in Chapter 4. Four 30 degree skew panels were produced by a precast concrete plant. The fabrication processes undertaken to construct the 45 degree skew panels are described. The techniques used by the precast concrete plant are provided along with comments made by the workers in the plant. Refer to Table 3.4 for a summary of panel designs and designations.

5.2 45 DEGREE SKEW PANELS

The prestressing bed was configured to produce panels with a flared strand pattern. Once they were constructed, small modifications in the bearing plates needed to tension the tendons were made before panels with parallel strands could be produced. For each panel, the strands were stressed, formwork was built, concrete was placed, and the strands were cut when the concrete reached the desired strength.

5.2.1 Stressing and Relaxation

Once the stressing bed was attached to the rigid test floor in the laboratory, the stressing operations were very successful. Typical seating losses were between 1/8 in. and 3/16 in., lower than the assumed 1/4 in. This resulted in higher net forces than anticipated, but close to the value specified by TxDOT.

Figure 5.1: Stressing strands for panel P45F60-1

The strands were numbered in consecutive order for each panel beginning with the strand furthest from the skewed end. Of the 16 strain gages used in P45F60-1, forces between 16 and 18 kips (Figure 5.2) were typically recorded. Only the even numbered strands in P45F60-2 were gauged and slightly smaller forces ranging between 14 and 16 kips (Figure 5.3) were reported. The strain gages in P45P60-1 were quite variable, but forces in the range of 15 to 18 kips (Figure 5.4) were reported by the gages that can be considered accurate. During the stressing of P45P45-1, it became noticeably more difficult to stress the strands than the previous three panels. The same target pressure was used, but the resulting forces recorded were much higher than before. Almost all of the force readings given by the strain gages were between 19 and 21 kips (Figure 5.5). Because the strain data is so consistent, it is assumed that there was a problem with the pressure gage. Moreover, the normal strain loss due to sequential stressing was not nearly as evident with P45P45-1. The first several strands to be stressed up to 50% typically lost 25-50% of their force as the remaining strands were stressed. This loss happened because slight flexure and slip of the bulkhead took place as more force was introduced into the prestressing frame. In P45P45-1, the maximum loss was 25%.

Due to the length of time required to stress the strands, tie the rebar cage, and batch the concrete, the concrete was placed the day after tensioning the strands. The length of time between tensioning the strands and placing the concrete allowed more relaxation in the strands and loss of prestress force. The strands in P45F60-1 lost an average of 0.20 kips over a 20 hour period. The strands in P45F60-2 lost an average of 0.12 kips during a 16 hour period. The strands in P45P60-1 lost an average of 0.49 kips during 23 hours. The data acquisition system was inadvertently turned off between the stressing and casting of P45P45-1, so no relaxation data is available.

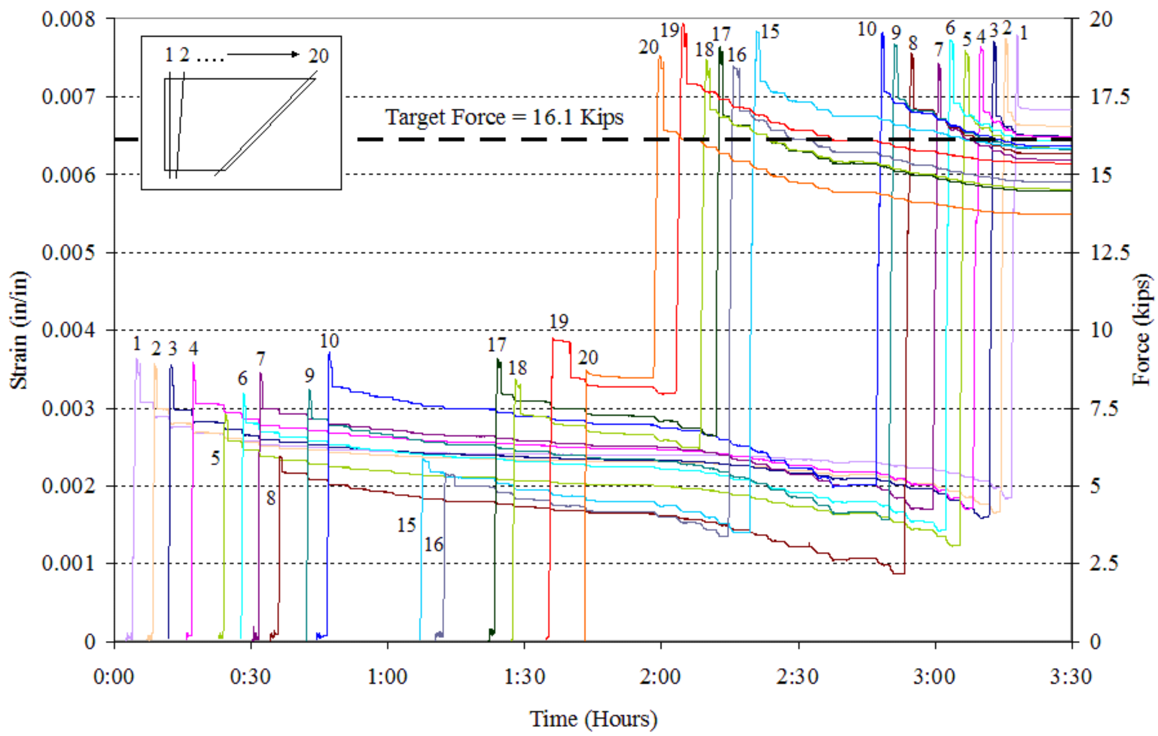


Figure 5.2: Strains recorded from strain gages for P45F60-1

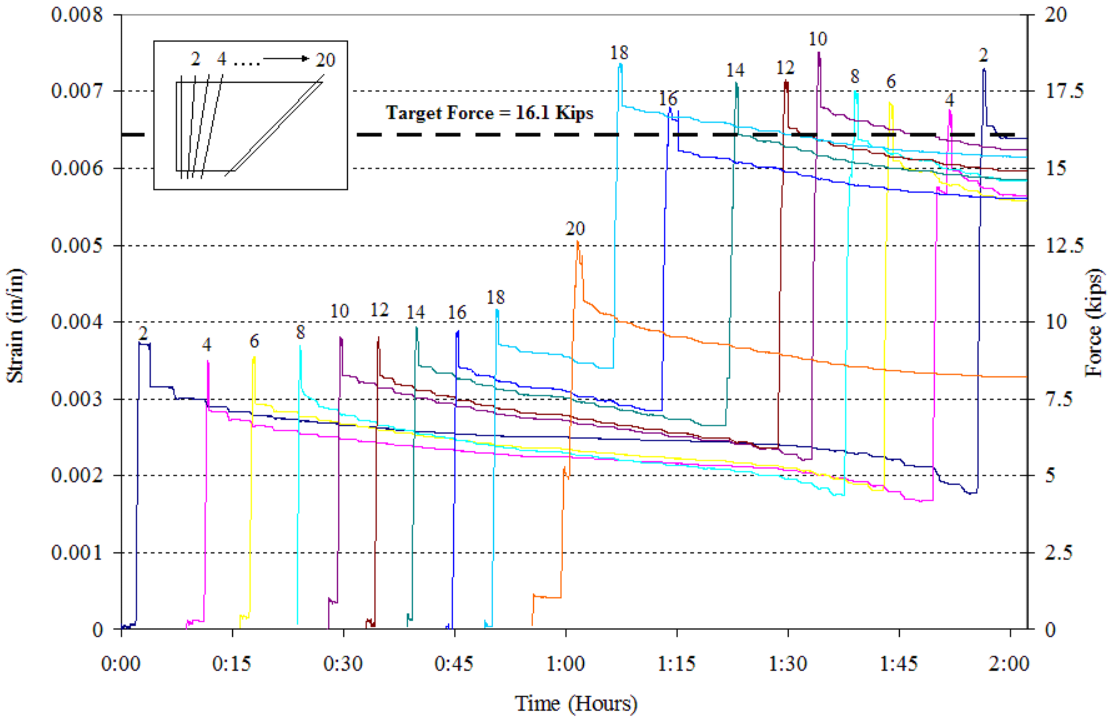


Figure 5.3: Strains recorded from strain gages for P45F60-2

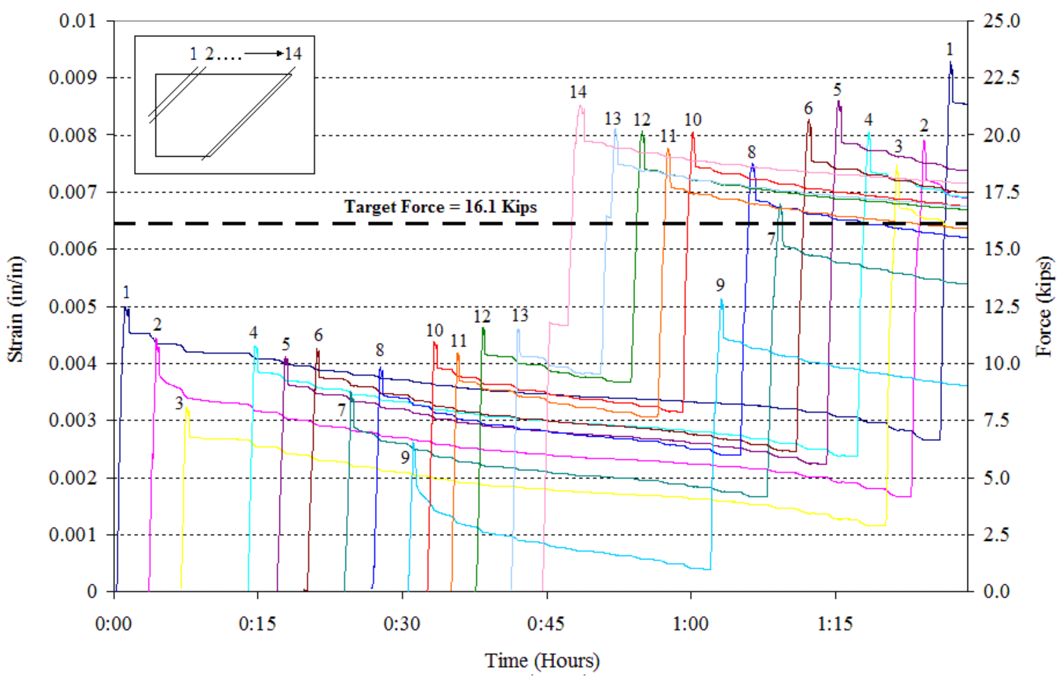


Figure 5.4: Strains recorded from strain gages for P45P60-1

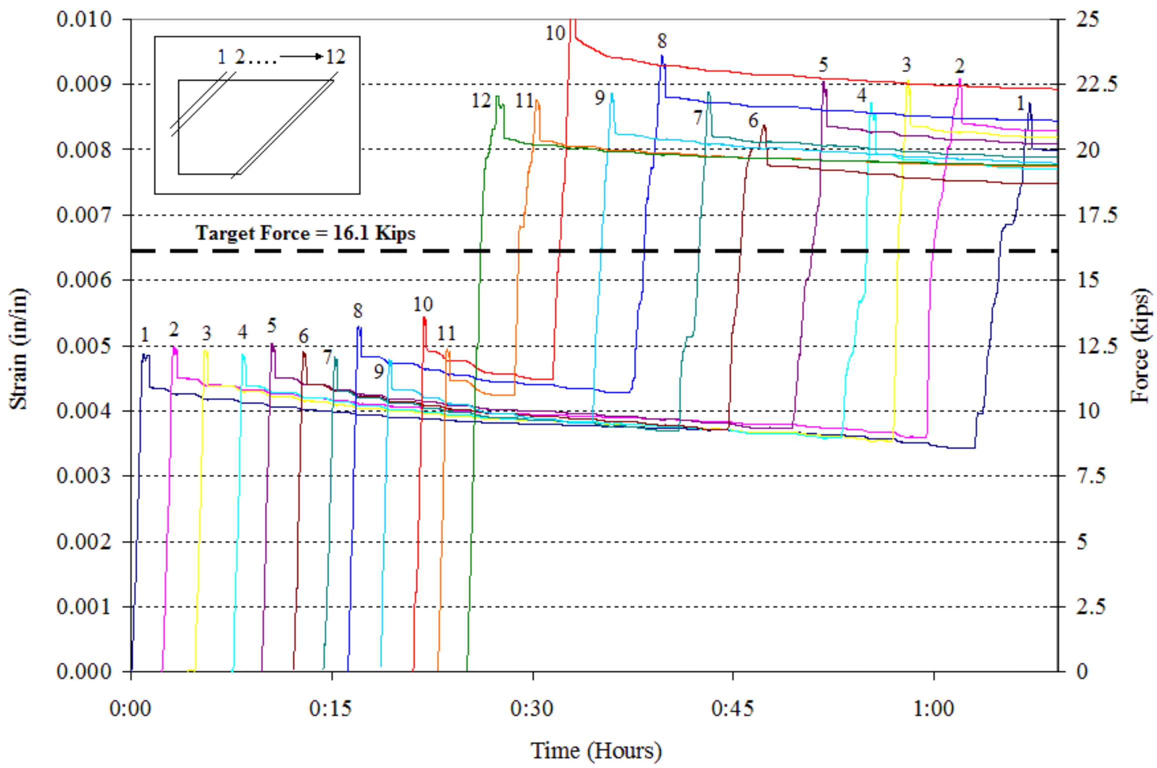


Figure 5.5: Strains recorded from strain gages for P45P45-1

The appearance of the anchorages after stressing is shown in Figures 5.6-5.8. The congestion at the dead end of P45F60-1 with flared strands is evident in Figure 5.6. Although the live end anchors shown in Figure 5.7 have sufficient space, the angles are different for each strand. Both the dead and live ends of the panels with the parallel strands look the same with constant spacing of similar bearing blocks. The live end of P45P60-1 is shown in Figure 5.8.

Figure 5.6: Congested dead end anchorage after stressing strands for P45F60-1

Figure 5.7: Live end anchorage after stressing strands for P45F60-1

Figure 5.8: Live end anchorage after stressing strands for P45P60-1 (similar to dead end)

5.2.2 Formwork and Rebar

After the strands were stressed, the formwork for the panels was completed and the deformed reinforcing bars were placed and tied. For all panels, nineteen #3 bars were placed with 6 in. center-to-center spacing above the strands parallel with the short and long edges of the panel. This arrangement is one of the longitudinal panel reinforcement options primarily for temperature and shrinkage crack control. Additionally, lifting hoops were tied into the cage of all panels to facilitate transportation.

The flared strand pattern panels did not require any additional flexural reinforcement because they were fully prestressed. However, to control cracking due to bursting stresses, an additional twenty hairpins were placed in Panel 1 – twelve on the short edge and eight on the long edge. Because there was no visible cracking after the release of P45F60-1, no hairpin bars were placed in P45F60-2. Figure 5.9 the reinforcement in place prior to casting P45F60-1.

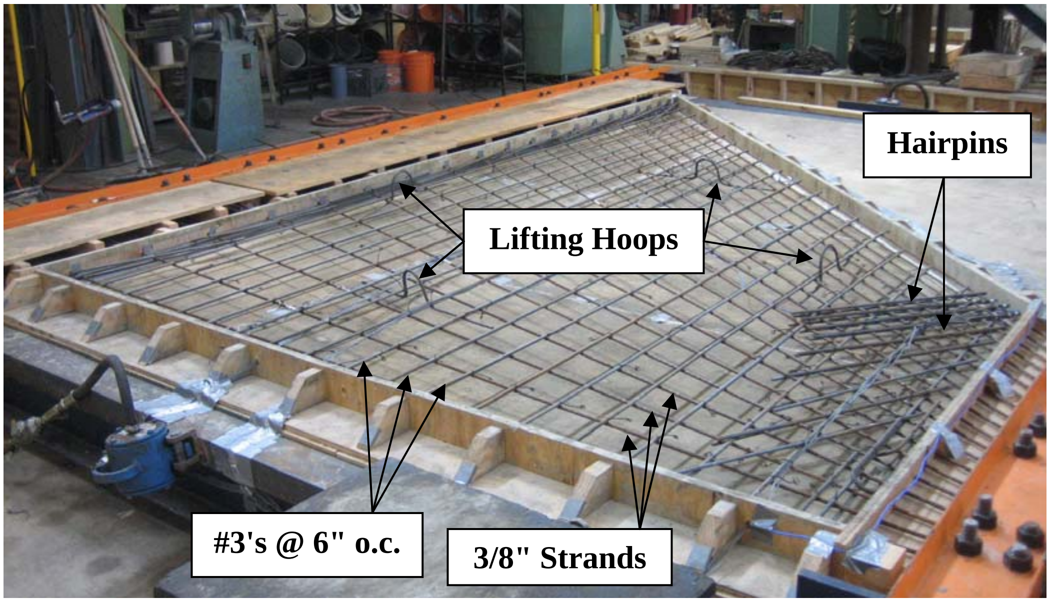


Figure 5.9: Completed formwork and rebar cage for P45F60-1

For P45P60-1 and P45P45-1, no hairpin bars were placed, but both contained additional flexural reinforcement in the regions without prestressing strands. In P45P60-1, fourteen #4's spaced at 4 in. on center were placed beneath the strands parallel to the non-skewed end. In Figure 5.10, the completed reinforcing cage for P45P60-1 is shown and a close up of the additional flexural reinforcement is shown in Figure 5.11. Eleven #4's were placed with 4 in. center-to-center spacing in P45P45-1 because the short edge was 15 in. shorter than P45P60-1. The completed reinforcement cage and close-up of the additional deformed bars for P45P45-1 are shown in Figures 5.12 and 5.13, respectively.

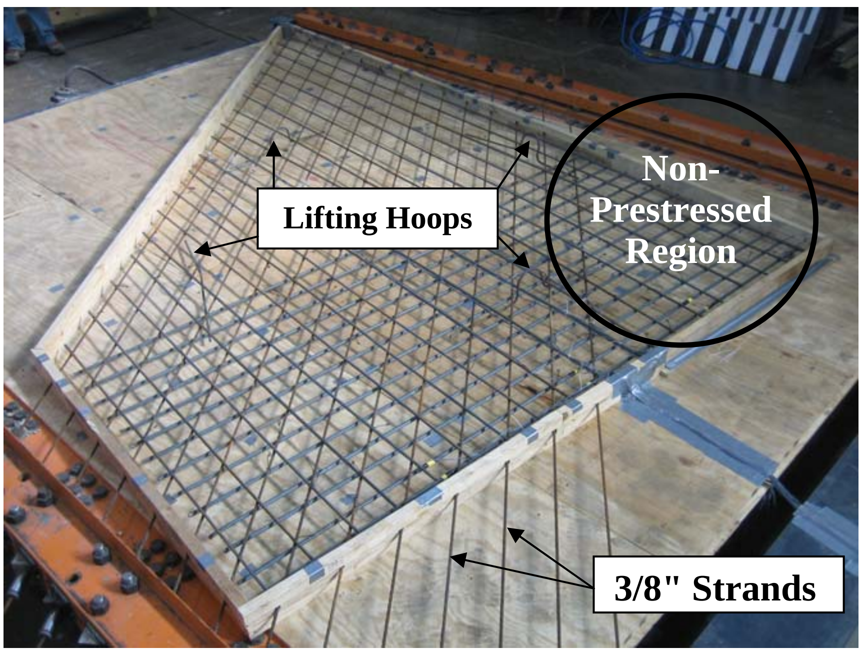


Figure 5.10: Completed formwork and rebar cage for P45P60-1

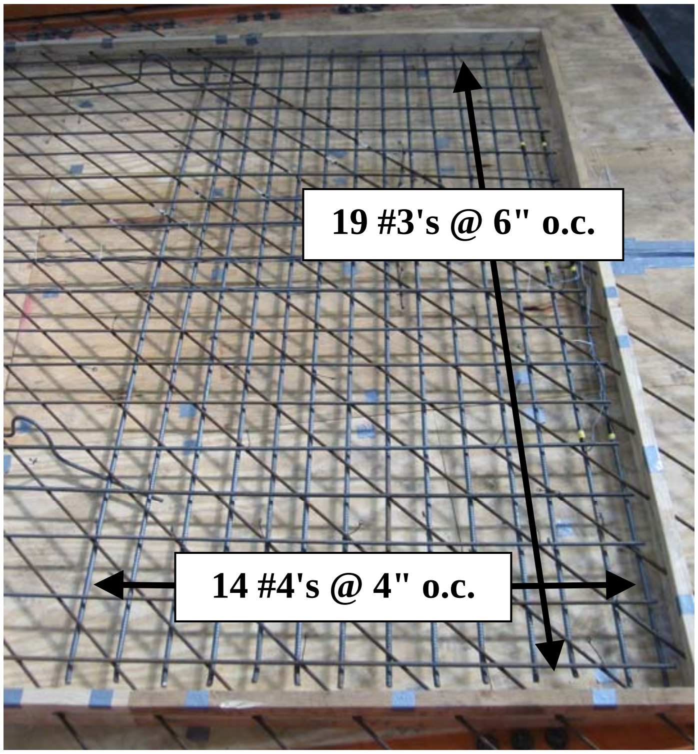


Figure 5.11: Additional flexural reinforcement for P45P60-1

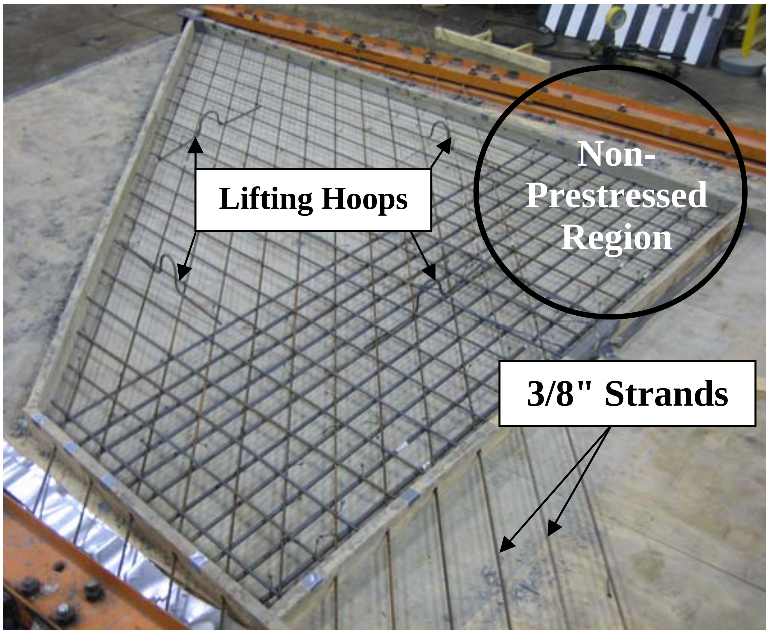


Figure 5.12: Completed formwork and rebar cage for P45P45-1

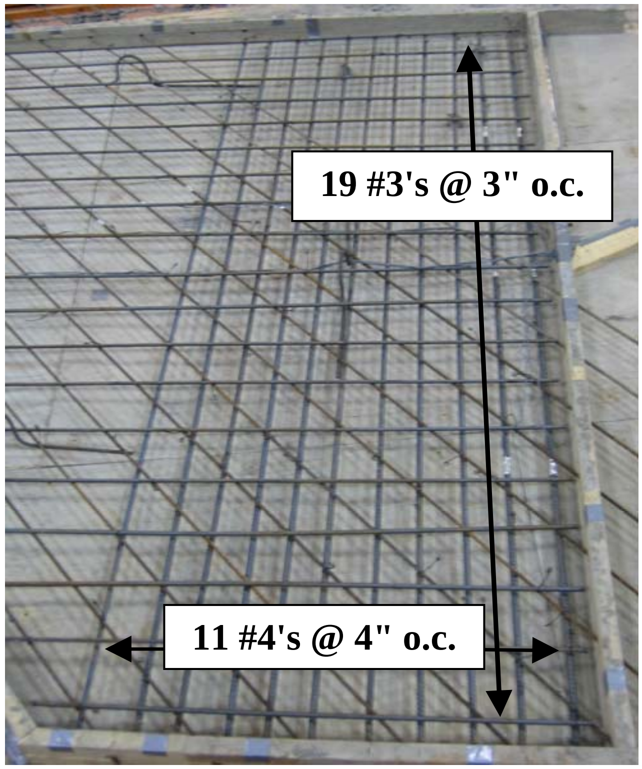


Figure 5.13: Additional flexural reinforcement for P45P45-1

5.2.3 Casting

From the time the ready-mix concrete truck arrived with the coarse and fine aggregates, the batching process took approximately 30 minutes. Once the concrete mixing was completed, a slump test was conducted. Twenty 4x8 in. cylinders were cast to monitor the concrete compressive strength. Concrete was transported to the casting bed using a one-yard bucket carried by the overhead crane. The concrete was placed and spread evenly with hoes and trowels. Screed boards were used to create a level surface even with the tops of the forms. Due to the shallow depth of the concrete, ordinary stinger vibrators were not used. Instead, compressed air form vibrators mounted on the reaction frame were used to vibrate the concrete. The panels were not trowel finished because a rough surface is specified on precast panels. When the concrete began to set, a broom was dragged across the surface to produce a TxDOT 'standard broom finish.' Figures 5.14 through 5.19 contain pictures taken during various stages of production.

Figure 5.14: Placing concrete using one-yard bucket

Figure 5.15: Leveling concrete with screed board

Figure 5.16: Form vibrator mounted on prestressing frame

Figure 5.17: Completed panel prior to roughening the surface

Figure 5.18: Finish created from broom

5.2.4 Release

The prestressing strands were cut as soon as the desired concrete strength was achieved as determined by cylinder testing. An image of the strands being torched is shown in Figure 5.20. The curing time required varied by panel due to slight differences in the concrete batch as well as ambient air temperature. Panel 1 was released after nineteen hours when the concrete strength reached 5,000 psi. Panel 2 was released after only nine hours when the concrete strength reached 4,000 psi. Panels 3 and 4 were released at nineteen and twenty hours, respectively, once the compressive strength reached 4,000 psi. The latter two panels took much longer to reach 4,000 psi because they were both cast in the winter when the ambient air temperature was below 40 degrees Fahrenheit. In all four panels, no cracking was evident after the strands were cut. The release strength and time as well as 28-day compressive strengths are summarized in Table 5.1.

Table 5.1 Summary of concrete release and 28-day strengths

Panel	Strength at Release (psi)	Time (hours)	28-Day Strength (psi)
P45F60-1	5120	19	8830
P45F60-2	4430	9	10200
P45P60-1	4100	19	8800
P45P45-1	4120	26	8530

Figure 5.19: Cutting tensioned strands with acetylene torch

5.2.5 Assessment of Panel Construction

Two panels with flared prestressing strands and two panels with parallel prestressing strands were fabricated at Ferguson Laboratory. The hands-on experience provided valuable insight as to which pattern of prestressing is a more viable alternative.

The angled shim plates and spherical washers performed very well for aligning the flared prestressing strands, but the bearing assemblies took a long time to produce. Although the bulkhead and anchorage method used may not be the most efficient way to capture variable angles and strand spacing, any alternate solutions would still be more complex than current prestressing methods. Angled blocks were produced for the parallel strand pattern bearings in order to conform to the geometry of the constructed prestressing frame, however, typical solid plate bulkheads with drilled holes often used by precast plants could have been used with a different prestressing bed. The standard strand spacing from the parallel strand pattern allows the same bulkhead to be used for all panel geometries and eliminates potential error and fabrication time associated with setting individual bearing assemblies.

The congestion of the dead end anchorage for the flared strand pattern, as seen in Figure 5.6, is an area of concern. The convergence of strands extending beyond the short edge could govern panel geometry. Conversely, the parallel strand pattern has no anchorage related problems and could accommodate any panel geometry.

The skewed prestressing pattern would prevent the trapezoidal panels from being mass produced on a long-line casting bed. Deviators could be used to create the flared arrangement down the length of the casting bed, but the geometry and prestressing strand forces would be difficult to control. The parallel prestressing pattern, however, can easily utilize the efficiency of long-line production since strand spacing remains constant. Since current casting beds are typically 8 ft. wide, larger casting beds would need to be constructed to produce panels with large skew angles and widths. Both prestressing strand arrangements require custom formwork regardless of casting location.

No cracking was observed in any panel upon strand release after concrete compressive strengths reached 4,000 psi. Therefore, so bursting reinforcement is required for either prestressing pattern. No mild reinforcing in addition to the longitudinal reinforcing is required for the flared strand pattern, whereas parallel strand pattern requires transverse bars tied beneath the prestressing strands. The additional transverse bars were easy to place and required minimal extra time.

5.3 30 DEGREE SKEW PANELS

All of the 30 degree skew panels were produced by a precast concrete fabricator. The first two were cast in early November 2007, and the second two were cast on January 3, 2008. Visiting the plant during the fabrication of the second set permitted firsthand observation of the entire process.

5.3.1 Formwork and Rebar

The formwork for the panels was constructed using lumber and plywood in two phases. The bottom half of the forms were placed prior to tensioning the strands (Figure 5.20). These were made from a split 2x4 with 1/2 in. plywood strip spacers between strands (Figure 5.21). After the strands were tensioned, the strands with short embedment lengths were de-bonded by wrapping them in a plastic tube and sealing it with duct tape (Figure 5.22). Next, the additional flexural reinforcement was placed beneath the strands and tied into place. Rather than cutting each longitudinal #3 bar prior to installation, the iron workers tied over-length bars into place and then cut them to length with large bolt cutters. All reinforcing for both panels was completed in approximately 25 minutes. Once all of the reinforcement was tied, the top half of the wooden formwork was nailed into place. To keep the formwork from moving, small clamps were placed on the strands to brace the wood (Figure 5.24). The 30 degree skew panels were then ready for casting. Pictures taken during the placement of the formwork and reinforcing bars are shown in Figures 5.20 through 5.25.

Figure 5.20: Bottom half of wooden forms for 30 degree skew panels

Figure 5.21: De-bonded strands for 30 degree skew panel

Figure 5.22: Finished rebar cage for 30 degree skew panel prior to formwork completion

Figure 5.23: Flexural reinforcement in 30 degree skew panels

Figure 5.24: Clamp attached to strands to brace formwork

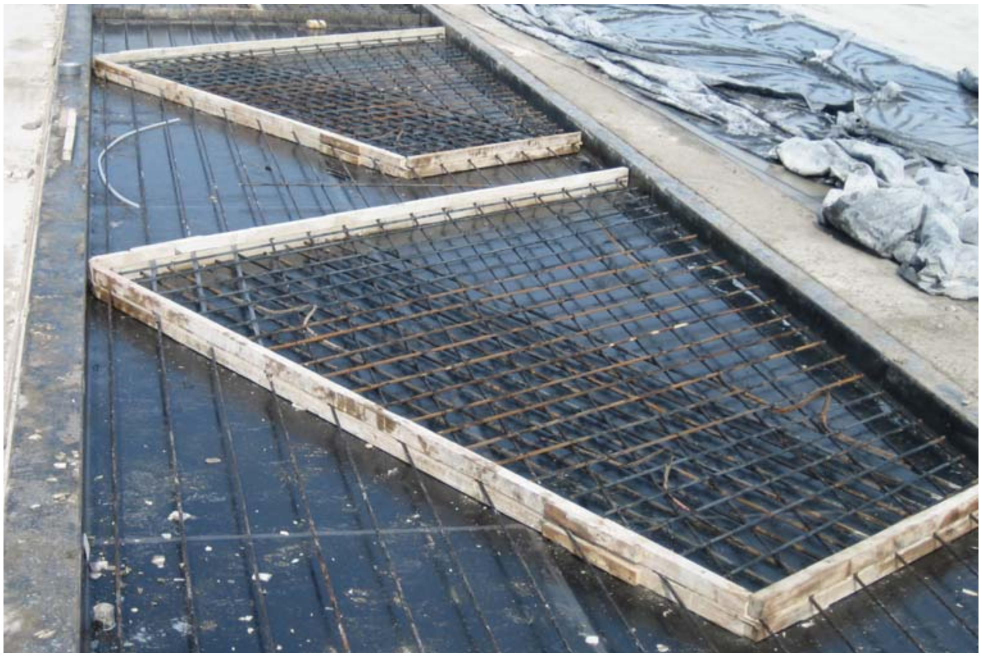


Figure 5.25: Two 30 degree skew panels ready for casting

5.3.2 Casting

Casting panels in the long lines common in prestressed concrete plants is a very quick process. A truck carrying several yards of fresh concrete from the on-site batch plant drove directly up to the casting location. The concrete was placed from the truck into the casting bed. The laborers then spread the concrete around the panel with shovels and dragged a vibrating screed across the top to create a level surface. The entire process took less than 4 minutes for both panels. Approximately 10 minutes later, a broom was used to create a roughened surface on the panels.

Typically during the casting process at this precast plant, the lines are flooded in locations where panels are completed to cure the panels. It is possible that flooding may tend to reduce the surface roughness of a "standard broom finish." The length of time exposed may also affect the curing process depending on ambient air temperatures.

Figure 5.26: Placing concrete in casting bed

Figure 5.27: Leveling concrete with shovels and vibrating screed

Figure 5.28: Casting complete

Figure 5.29: Broom finish

5.3.3 Assessment of Panel Construction

Four 30 degree skew panels were produced by a precast concrete plant. The fabrication processes were observed for two of the panels cast on the same day. The wooden formwork had already been constructed upon arrival, so the time required for fabrication is unknown. De-bonding the necessary strands and placing the additional longitudinal and transverse reinforcement took a matter of minutes. Placing and finishing the concrete was no different than standard rectangular panels and therefore required no additional time. Overall, the procedures for producing the trapezoidal panels went smoothly and quickly. The labor foreman gave the impression that the extra work required for trapezoidal panels, including custom formwork, de-bonding strands, and placing additional reinforcement beneath the strands, was not a problem and could be accomplished on a routine basis.

CHAPTER 6

Bridge Deck Construction

6.1 INTRODUCTION

With fabrication of the panels completed, the next phase of the project involved full-scale testing. Three panels fabricated at Ferguson Laboratory were tested along with two panels produced by the precast plant. The processes involving the precast panels used to construct the test specimen are described in this chapter.

6.2 BRIDGE DECK SPECIMENS

Five bridge deck specimens were built and tested. Panel P45F60-2 was not used in constructing a bridge deck specimen because it was a redundant design and primarily fabricated to test cracking occurrence at release. The second set of 30 degree skew panels, P30P45-3 and P30-45-4, will be used for bridge deck test specimens in the future.

All bridge deck specimens were loaded with a single wheel load at the center of the skewed end. Additionally, the panels containing the parallel strand pattern were loaded a second time with a single wheel load at the center of the non-skewed end to test the strength of the partially prestressed panel end. Because the non-skewed end of the flared strand panel was fully prestressed, a second test was non necessary. To more accurately simulate field conditions for the non-skewed end tests, an additional 4 ft. by 9 ft. 6 in. rectangular panel was placed adjacent to the skewed panel with the topping slab placed over both panels. All rectangular panels were cast at a precast plant and delivered to Ferguson Laboratory. A summary of the bridge deck test specimens is provided in Table 6.1. The nomenclature used to identify the test specimens is as follows: 1) Positive moment (P), 2) 45 degree (45) or 30 degree (30) skew, 3) Prestressed panels detail (P), and 4) the specimen number (1-3).

Table 6.1: Summary of bridge deck test specimens

Specimen No.	Panel	Skew Angle	Adjacent Rectangular Panel
P45P1	P45F60-1	45°	No
P45P2	P45P60-1	45°	Yes
P45P3	P45P45-1	45°	Yes
P30P1	P30P45-1	30°	Yes
P30P2	P30P45-2	30°	Yes

To simulate bridge girders, each panel was supported on 12 in. by 12 in. concrete beams with a foam bedding strip between the concrete beams and the prestressed panel. The bedding strip for specimen P45P1 was conventional foam purchased from a hardware store. The remainder of the specimens used the same 40 psi foam bedding strip as TxDOT bridge construction. Because bearing conditions were critical, the maximum bedding strip width allowed by TxDOT of 2 in. was selected. To ensure good concrete flow beneath the overhanging edge of the panel, the bedding strips were cut 2 in. tall. In the specimens containing an adjacent rectangular panel, the panels were set with a 3/4 in. gap between them. Backer rod was then used to seal this gap. A schematic of the construction process for the bridge deck specimens is shown in Figure 6.1. More detailed descriptions of the test setups are given in Boswell (2008). In Figure 6.2, a photo of the construction of specimen P45P3 is shown.

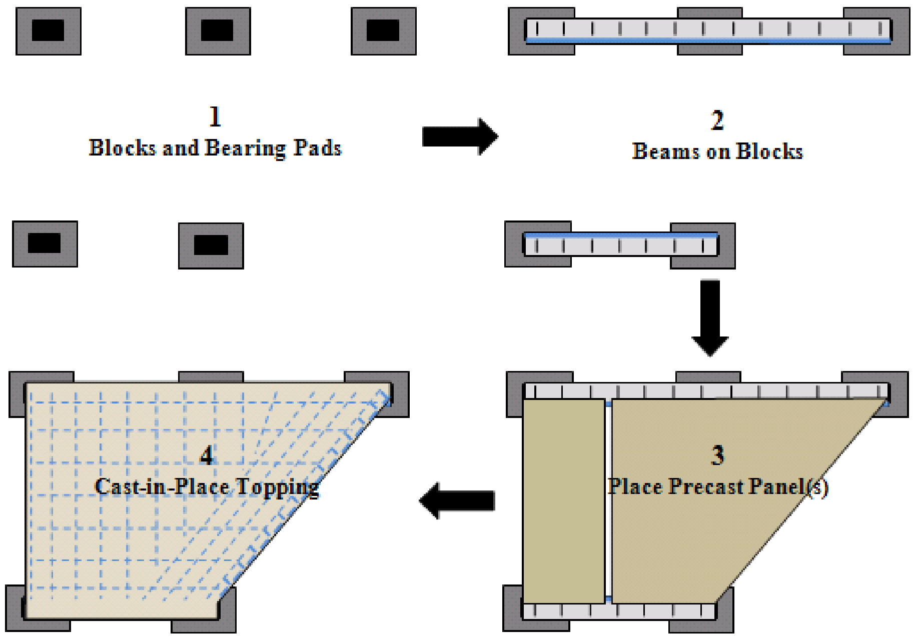


Figure 6.1: Schematic of construction of bridge deck test specimens (Boswell 2008)

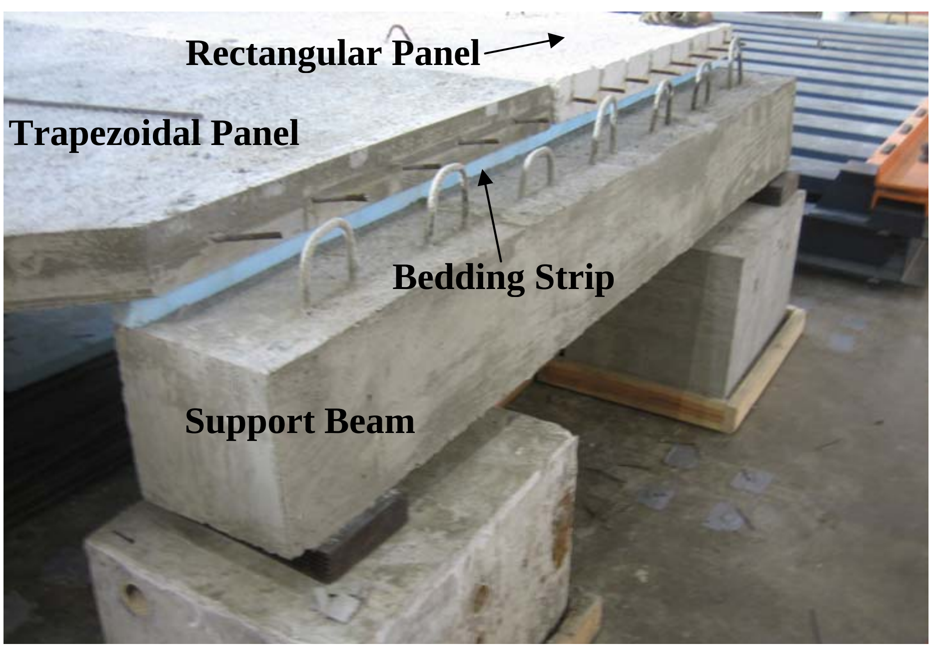


Figure 6.2: Construction of specimen P45P3

6.3 Bedding Strip Compression

6.3.1 Specimen P45P1

The conventional foam used for specimen P45P1 bedding strip had insufficient strength. Upon placement of the panel, the bedding strip supporting the short edge of the panel compressed more than 1.5 in. As a result, temporary supports were placed beneath the test setup near the short edge of the panel until the concrete topping slab had cured. The bedding strip on the long edge did not compress appreciably and did not require a temporary support.

6.3.2 Specimen P45P2

The test setups for specimens P45P2 and P45P3 both consisted of the trapezoidal panel with a rectangular panel adjacent to the non-skewed end. P45P2 used a trapezoidal panel with a 60 in. short edge length, which should have had a bearing pressure of 37 psi under an evenly distributed load. After placement of the topping slab for specimen P45P2, a large amount of compression in the bedding strip beneath the short edge of the trapezoidal panel was discovered. Directly under the obtuse-angled corner, the deflection was 3/4 in. as shown in Figure 6.3. This compression decreased linearly to the non-skewed end where the compression was approximately 3/8 in. However, beneath the rectangular panel next to the non-skewed end, the compression was roughly 1/16 in. This differential settlement, shown in Figure 6.4, indicates a much larger bearing pressure beneath the short edge of the trapezoidal panel. No noticeable compression was measured beneath the long edge of the trapezoidal panel.

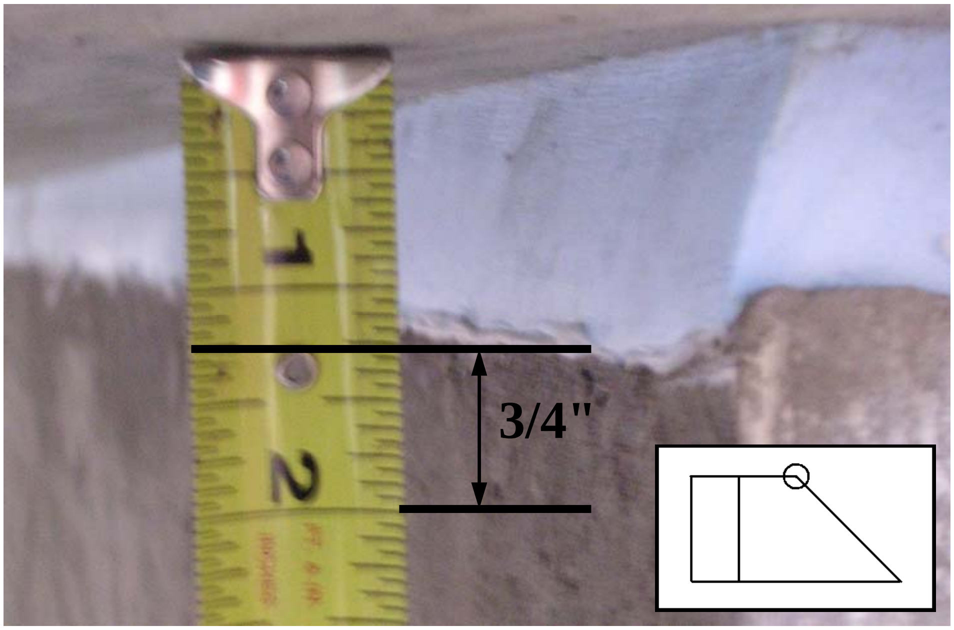


Figure 6.3: Bedding strip compression beneath obtuse corner of specimen P45P2

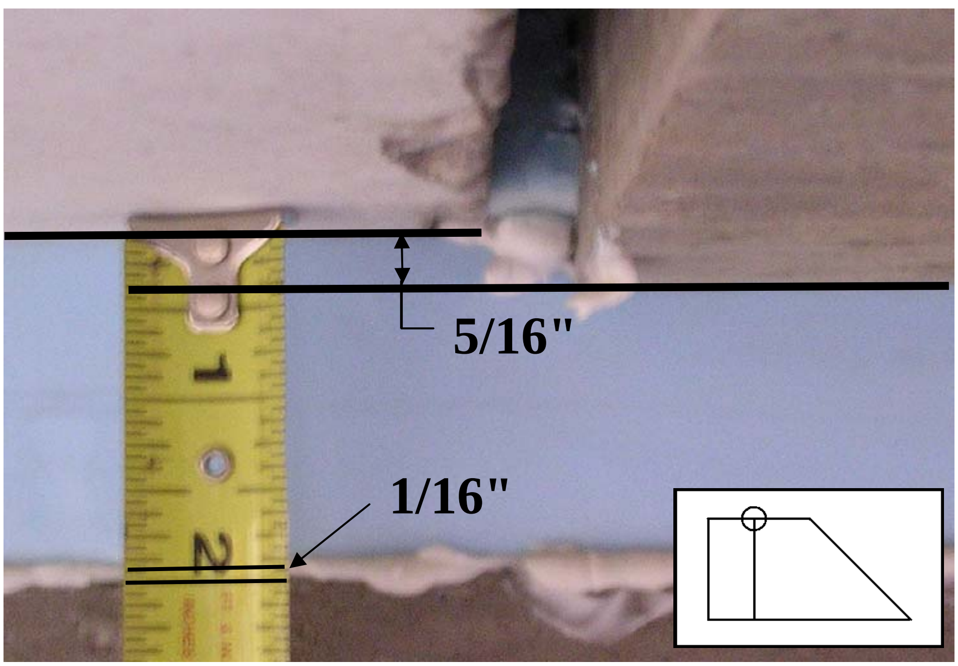


Figure 6.4: Differential compression in bedding strip at joint between trapezoidal and rectangular panels in specimen P45P2

6.3.3 Specimen P45P3

A trapezoidal panel with a 45 in. short edge length was used in specimen P45P3, which should result in a higher bearing pressure, almost 50 psi, with an evenly distributed load. However, the bedding strip only compressed approximately 3/8 in. uniformly across the length of the short edge after the topping slab was cast. The compression of the bedding strip beneath the obtuse angled corner of specimen P45P3 is shown in Figure 6.5. Similar to construction of the specimen P45P2, there was a differential compression of 1/4 in. between the non-skewed end of the trapezoidal panel and the adjacent rectangular panel (Figure 6.4). Compression of the bedding strip supporting the rectangular panel as well as the long edge of the trapezoidal panel was about at 1/16 in. on average.

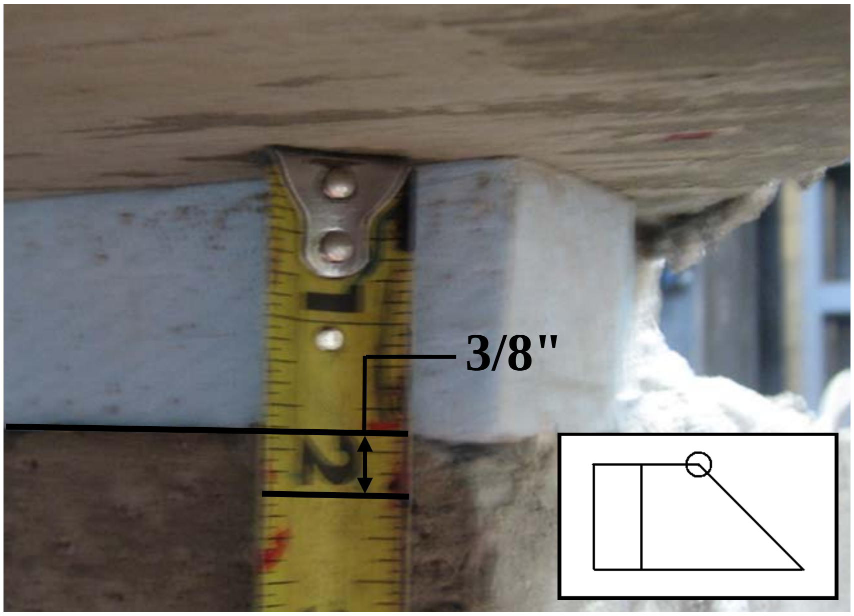


Figure 6.5: Bedding strip compression beneath obtuse corner specimen P45P3

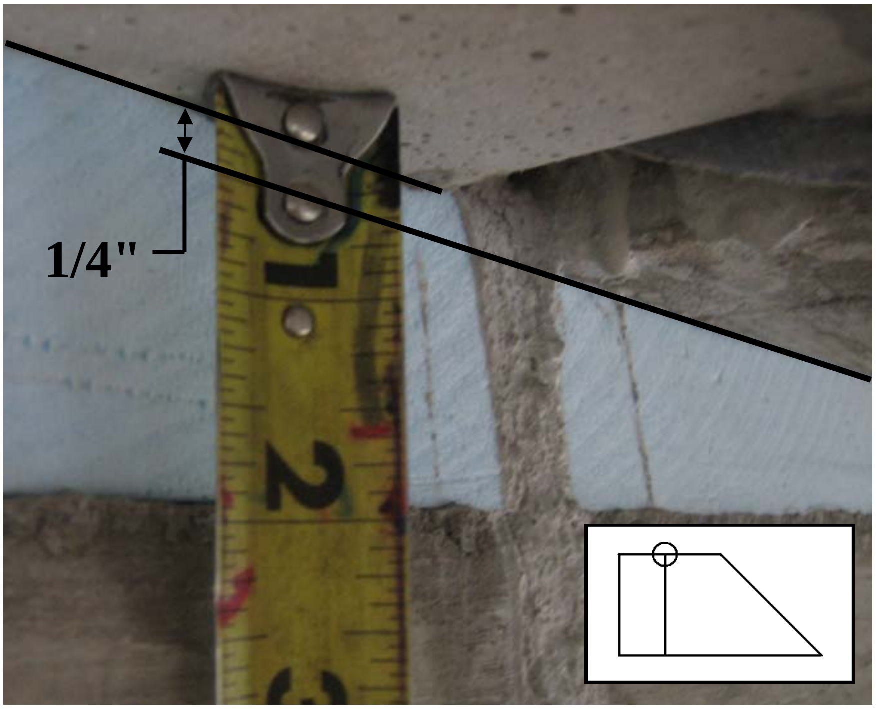


Figure 6.6: Differential compression in bedding strip at joint between trapezoidal and rectangular panels in specimen P45P3

6.3.4 Specimens P30P1 and P30P2

The 30 degree skew panels were delivered to Ferguson Laboratory on a flatbed truck. They were transported to their respective test setups with the overhead crane. Both panels had 45 in. short edge lengths which should produce less than 40 psi bearing pressure with and evenly distributed load. During construction of the cast-in-place topping slabs of either P30P1 or P30P2, there was no appreciable compression of the bedding strip at any location.

6.4 Assessment of Construction of Bridge Deck Specimens

The precast panels were placed on concrete beams with a foam bedding strip between the concrete beams and the prestressed panels. Formwork was erected along with expansion joint armor prior to placing the reinforcing mat and casting the topping slab. Several observations were made throughout the construction procedures.

Placing both the rectangular and trapezoidal panels required roughly the same amount of time. Small areas exist where conventional formwork may still be required along the end of the slab, namely over the girders and between the expansion joint armor and prestressed panel. Because the expansion joint armor is supported by the girders, removing the formwork does not affect the armor bearing.

During the construction of all three 45 degree skew bridge deck specimens, significant compression of the bedding strip was observed beneath the short edge of the trapezoidal panel. It was also noticed that when the bedding strip compressed, the slab thickness increased, adding to the weight and bearing pressure on the foam bedding strip.

Upon further investigation, it was discovered that the conventional foam used in specimen P45P1 had a compressive strength of 25 psi. Due to the magnitude of compression beneath the short edge of the panel in specimen P45P1, the bearing pressures substantially exceed 25 psi.

Simple bearing tests conducted on samples of 40 psi bedding strip foam verified the compressive strength. However, a linear relationship was found between load and compression which resulted in roughly 1/2 in. compression at a bearing pressure of 40 psi. Once the 40 psi pressure was exceeded, significant compression occurred without much additional load. Therefore, the bearing pressures beneath the short edge of the trapezoidal panels in specimens P45P2 may not have exceeded 40 psi. However, the 3/4 in. compression observed beneath the obtuse-angled corner in specimen P45P2 most likely surpassed 40 psi.

When bedding strip compression was discussed with TxDOT bridge engineers, it was deemed tolerable as long as the minimum bedding strip requirements were still met.

CHAPTER 7

Summary, Conclusions, and Recommendations

7.1 SUMMARY

Prestressed concrete panels have been used by the bridge construction industry in the state of Texas for many years to increase speed and improve safety and economy. The panels serve as stay-in-place formwork and become a part of a composite deck after a topping slab is cast. At expansion joint locations, however, TxDOT currently uses the IBTS detail rather than prestressed panels. This cast-in-place detail requires temporary formwork and slows construction processes. Prestressed panels are used at the expansion joints of 0 degree skew bridge decks in a new detail developed under TxDOT research project 0-4418. With new detail, a more economical alternative to the current IBTS detail is provided.

The primary goals of this research project were to evaluate the feasibility of producing trapezoidal-shaped prestressed concrete panels as well as address construction related issues so that use of the new panel detail can be extended to include skewed expansion joints. One research objective was to devise a fabrication method that could accommodate a wide range of skew angles but maintain the economy of mass production. Another research objective involved construction of full-scale test specimens with trapezoidal panels so that any construction related problems could be observed.

A total of eight trapezoidal panels were fabricated using two primary prestressing layouts. Four were produced in Ferguson Laboratory and four were produced at a precast plant. A flared prestressing pattern with a 45 degree skew angle and a 60 in. short edge length were used in panel P45F60-1. Additional hairpin bursting reinforcement was also included in P45F60-1 and the strands were released when the concrete reached a 5,000 psi compressive strength. The same geometry and prestressing pattern from P45F60-1 was used in panel P45F60-2, but the additional bursting reinforcement was removed and the compressive strength of concrete at release was reduced to 4,000 psi. The same geometry with a 45 degree skew angle and 60 in. short edge length was continued in panel P45P60-1, but a parallel strand pattern with the strands parallel to the skewed end of the panel was used. Additionally, mild reinforcing bars were placed parallel to the non-skewed end of the panel. The parallel prestressing pattern and 45 degree skew angle were maintained in panel P45P45-1, but the short edge length was reduced to 45 in. Mild reinforcing bars were also placed parallel to the non-skewed panel end. The parallel prestressing strand pattern was used in panels P30P45-1, 2, 3 and 4 with a 30 degree skew and 45 in. short edge length. Mild reinforcing bars placed parallel to the non-skew end were also used. All panels were 4 in. thick and 9 ft. 6 in. wide.

Five of the eight panels were used for single bay, full-scale bridge deck test specimens. In four of the bridge deck test specimens, an additional rectangular panel produced at a precast plant was placed adjacent to the non-skewed end of the panel. Each panel was set onto bedding strips glued to concrete beams before a mat of deck reinforcing steel was tied and topping slab placed.

7.2 CONCLUSIONS

Based on the experiences and observations during production of the trapezoidal panels, the construction of full-scale test specimens, and discussions with TxDOT Bridge Division representatives, the following conclusions can be drawn regarding the practicality and constructibility of trapezoidal prestressed panels.

- Flared Prestressing Pattern
    - Custom formwork is required for each different panel geometry.

- The spacing of prestressing strands would be different for each combination of skew angle and panel width. This would require casting beds to be capable of shifting bearing locations to accommodate variations in strand spacing. Moreover, since each strand layout is unique to the panel geometry, standard drawings could not be utilized.
- Non-parallel prestressing strands create awkward bearing conditions, particularly on the short edge of the panel where strands converge on one another after exiting the face of the panel.
- No cracking was observed in highly stressed regions after strand release at the TxDOT specified release strength for panels of 4,000 psi despite lack of bursting reinforcement.
- Trapezoidal panels using a flared prestressing pattern cannot be mass produced on a long-line casting bed.

* Parallel Prestressing Pattern
    - Custom formwork is required for each different panel geometry.
    - Placing reinforcing bars beneath the prestressing strands is simple and requires minimal time.
    - Standard bulkheads could be used since strand spacing remains constant for all panel geometries.
    - Standard spacing of prestressing strands parallel to the skewed end allows implementation to any panel geometry.
    - Due to lack of required development length, some strands may require de-bonding or omission, which creates a region with partially prestressed concrete.

- No cracking was observed at interface of prestressed and non-prestressed regions where strands were de-bonded or omitted.
- Trapezoidal panels using a parallel prestressing strand pattern can be mass produced on a long-line casting bed. However, panels with large widths and skew angles would require casting beds wider than the 8 ft. standard to be built.

- Construction
  - Setting trapezoidal panels on support beams and aligning the skewed end is not difficult.
  - Compression of the bedding strip under high bearing pressures is not a concern to TxDOT engineers as long as minimum clearance requirements are still met.
  - The expansion joint armor is supported on the girders; therefore, removal of conventional formwork does not affect the placement of the armor.
  - Minimal formwork would still be required at the ends of the girders and between the expansion joint armor and precast panel.
  - Panel fit-up errors require a standard solution. One potential solution is custom cutting panels on site with a concrete saw.

7.3 RECOMMENDATIONS

The results of the research project demonstrate that producing trapezoidal prestressed panels can be economical while accommodating a wide range of geometries. Due to the complexity of the flared prestressing bearings, the variability of strand spacing, and the inability to use a long-line casting bed, the parallel prestressing pattern is the suggested design alternative. The parallel prestressing strand pattern allows current fabrication techniques and procedures to be utilized thereby maintaining the efficiency of the precast industry.

The research project also showed that trapezoidal prestressed panels can be used as stay-in-place formwork at skewed expansion joints. The compression in the bedding strip was deemed acceptable when using a 45 degree, 9 ft. 6 in. wide panel with a 45 in. short edge length and 40 psi bedding strip. Further research is needed to determine minimum bearing lengths for smaller panels and/or stronger bedding strip. Other construction issues, such as panel fit-up and formwork over the girders, should be discussed with bridge construction contractors to determine capabilities and solutions.

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Vita

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