

## **CHAPTER 1**

### **LITERATURE REVIEW OF BRIDGE DECK SYSTEMS**

#### **1.1 INTRODUCTION**

This chapter presents a literature review of bridge deck system, which consists of the cast-in-place (CIP) deck system, stay-in-place (SIP) precast deck system with cast-in-place topping system, and full-depth precast concrete deck system. Also included are various types of shear key and concrete overlay.

As a conventional system, CIP deck has been widely used due to its ease of field adjustment to produce a smooth roadway profile. However, it is labor intensive and time-consuming. The deck experiences cracking shortly after construction due to its tendency to shorten relative to the beams, which results from the differential creep, shrinkage, and temperature gradients. The SIP precast concrete deck panel with CIP topping is more cost-effective than the CIP system. Also, construction time is reduced by eliminating field construction and removal of formwork. Full-depth precast deck panel system is very popular currently due to its advantages of high structural performance and fast construction.

#### **1.2 BRIDGE DECK SYSTEMS**

##### **1.2.1 Cast-in-place (CIP) Deck Panel System**

Full-depth cast-in-place (CIP) panel is the commonly used deck panel system in the United States. The CIP deck panel typically consists of four layers of steel, including main reinforcement in the transverse direction and secondary reinforcement

## CHAPTER 5

### FUTURE REFINEMENTS AND CONCLUSIONS ON NUDECK SYSTEM

#### 5.1 SYSTEM REFINEMENTS

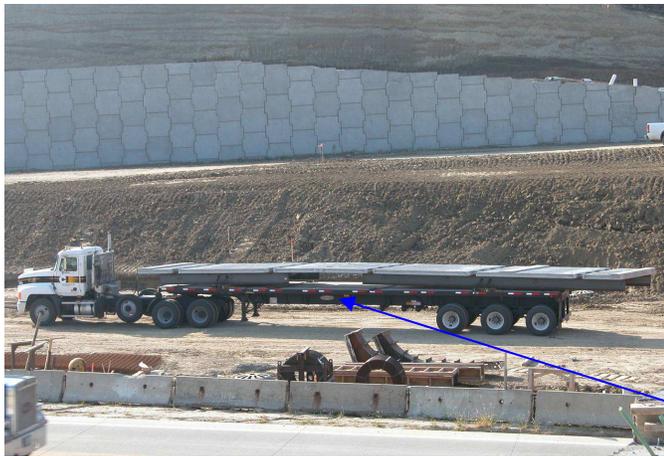
##### 5.1.1 Comments on the Current System

The Skyline Bridge NUDECK precast deck panel project was completed successfully. However, it is believed that the cost and time of construction can be significantly reduced. The experience from the Skyline Bridge was evaluated to investigate any potential refinements of the system. Based on what has been observed from the bridge construction, some comments are listed below.

1. The precast panels were shipped from Concrete Industries INC., in Lincoln to the bridge construction site at Omaha using the prefabricated steel frames with a 2% crown. There was an approximately 6 ft gap between the adjacent steel frames along the panel width direction (see [Figure 1](#)). Cracking was observed at the bottom of two end panels near the crown location, which might result from the unsymmetrical arrangement of pretensioning strands. However, the gap between the adjacent frames should have been minimized to reduce the moment in the panel. The current details at the crown location include a compressible material, namely, random oriented fiber, under the plastic rod. Since this material was not as compressible as expected, it worsened the situation especially for the end panels. Thus, the compressible material may deserve further investigation and be substituted in the future projects.
2. Along the crown location in each typical panel, there are three pockets and a 1/4 in. wide gap at the panel top, which requires grouting prior to shipping. For some reason, they were not grouted before the panels were shipped to the bridge construction site,

- which resulted in larger deformation than expected during handling and shipping. In addition, this gap may be widened to ensure it is completely grouted together with the pockets, which should provide higher rigidity for the panel.
3. The current shear key detailing includes a 1/2 in. gap at the bottom. Backer rod (or duct tape) need to be put between panels before concrete placement. Since there is a concern that the shear key bottom part may not be filled with concrete, the 1/2 in. gap can be widened to 2 or 3 in. Therefore, a higher shear capacity can be achieved. Meanwhile, more tolerance and flexibility can be provided for laying out the panels.
  4. The shear key at the panel outer edges or the channel locations can be produced with 6 in.-wide solid concrete so that no wood blocks (or duct tape, etc.) are needed prior to shear key grouting.
  5. The support system in the Skyline Bridge includes steel angles and cross ties over the steel beam top flange. It is suggested that plastic shims should be placed over the steel angle for minor adjustment of elevation, meanwhile, reducing the friction between the steel angle and deck panel at the time of post-tensioning. Alternatively, the steel angle can be greased, as what the contractor did for the Skyline Bridge.
  6. Although the location of the 1 1/4 in. diameter studs was determined to avoid any possible conflicts when the panels were placed, it still ended up with approximately 20 studs being cut. Since the contractor does not have the device to shoot the 1 1/4 in. diameter studs in field, each 1 1/4 in stud was replaced by 2-7/8 in. diameter studs. The 7/8 in. studs were arranged as close as possible to the original 1 1/4 in. stud line so that they may not interface with the post-tensioning strands.

7. The lifting points at the end panels were too close to the anchorage block, which ended up with pouring epoxy coating before the panels were post-tensioned. The location of lifting points should be arranged away from the anchorage block area.
8. The end of floor section in the Skyline Bridge plan is illustrated in [Figure 2](#). This detailing has two problems: 1) It requires the approach slab to be cast with a concrete mix different from that of the floor end. Since the approach slab and the floor end should be poured prior to the overlay placement, more construction steps have to be taken. 2) It requires grouting for the gap underneath the end panels before the NUDECK panels are placed. Since the contractors did not grout this small gap as requested, it was difficult to do the grouting afterwards. Eventually, the MASTERFLOW 816 cable grout was recommended by the UNL researchers. The cable grout can be used in tight area and it can be as flowable as water when placed. It can gain strength up to 8,000 – 10,000 psi over a period of several weeks. For future projects, a suggested modification is illustrated in [Figure 3](#). Note that the end floor area can use the same concrete mix as the approach slab, which allows them to be poured simultaneously. 1 in. (or 2 in.) thick asphalt is shown instead of the concrete overlay in the original design, which results in no need for finishing machine and curing. A vertical joint can be put in the floor end with bond break so that it allows the approach slab to rotate freely against the precast panels. A 6 in. bearing pad is placed to support the end panel, which provides a larger space to be filled with concrete rather than using the grouting as shown in the original plan.



Gap between adjacent frames

Figure 5.1-Prefabricated Steel Frame for Panel Shipping

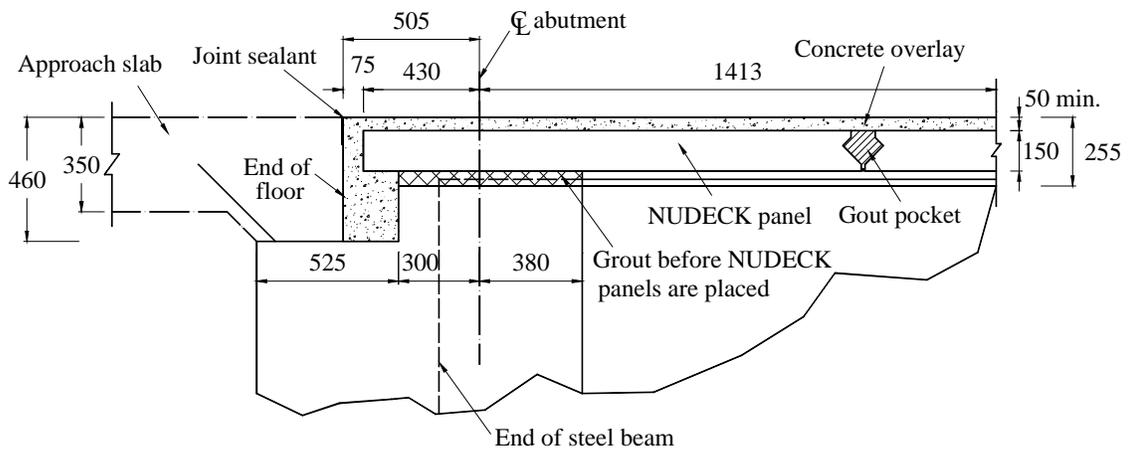
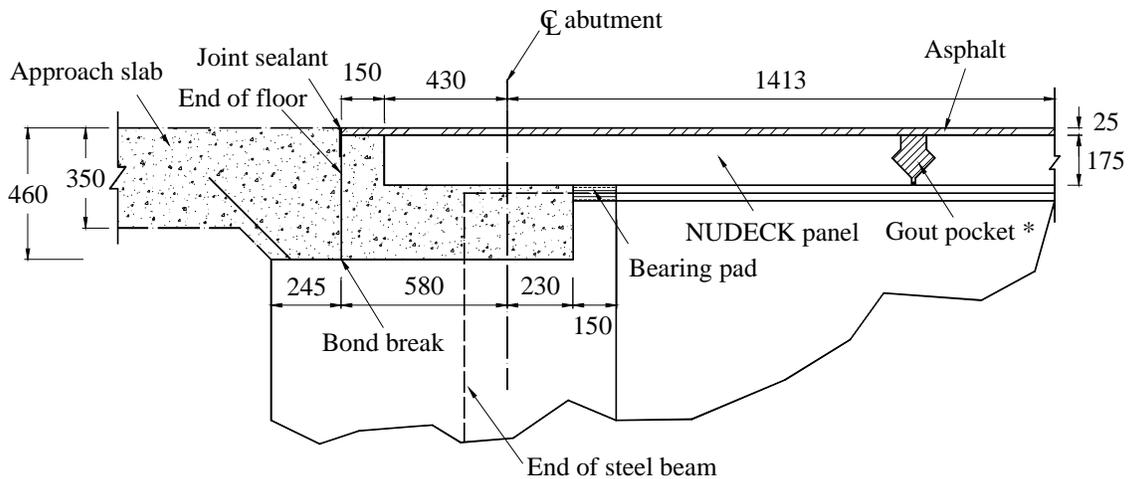


Figure 5.2-Section of End of Floor (see Skyline Bridge Plan 16/25)

Note: Metric unit is shown to be consistent with Skyline Bridge plan. (1 mm = 0.03937 in.)



Note: The shear key may be modified.

Figure 5.3-Suggested Section of End of Floor

Note: Metric unit is shown to be consistent with Figure 2. (1 mm = 0.03937 in.)

### 5.1.2 Possible Options for Improvement

Two possible options for improvement are illustrated in [Figure 4](#). They relate to simplification of precasting and post-tensioning, and elimination of the composite concrete overlay. The crown details can be simplified and the construction steps can also be reduced. [Figure 4](#) presents a bridge cross section with steel beam and precast deck panel. As shown in the plan view of the precast panel, the width of a typical panel is changed to 12 ft instead of 8 ft as previously specified in the NUDECK system. The 0.5 in. diameter, Grade 270 ksi pretensioned strands are spaced at 4 ft with groups of 4 strands instead of 2 strands at 2 ft spacing. The width of post-tensioning channel is widened from 12 in. to 14 in. and 12 strands are included instead of 16 strands used in the Skyline Bridge project, which results in more space for the strands to be pulled through. 4 x 4 – D10 x 10 wire mesh is provided instead of #5@12 in. for secondary reinforcement as well as for crack control. Railing dowel bars can be adopted in replacement of the galvanized female inserts if necessary. Option A uses 4-#10 rebar as the compression strut across the post-tensioning channel. The post-tensioning strands may be pulled along the channel by two layers with 6 strands at each layer. The steel tube (ID = 4.5” and OD = 5”) shown in Option B replaces the compression rebar. There are two 2.25 in. diameter holes at each tube allowing the post-tensioning strands to pass. A non-composite section with a 2 in. asphalt overlay is proposed herein to replace the Type K cement. A waterproofing membrane is provided to protect the reinforcement from corrosion. Using asphalt can reduce the time of construction as the concrete overlay normally requires 7 days curing. Also, asphalt can be easily removed if future replacement is needed. With the non-composite section, the precast deck thickness is changed from 6 in. to 7 in.

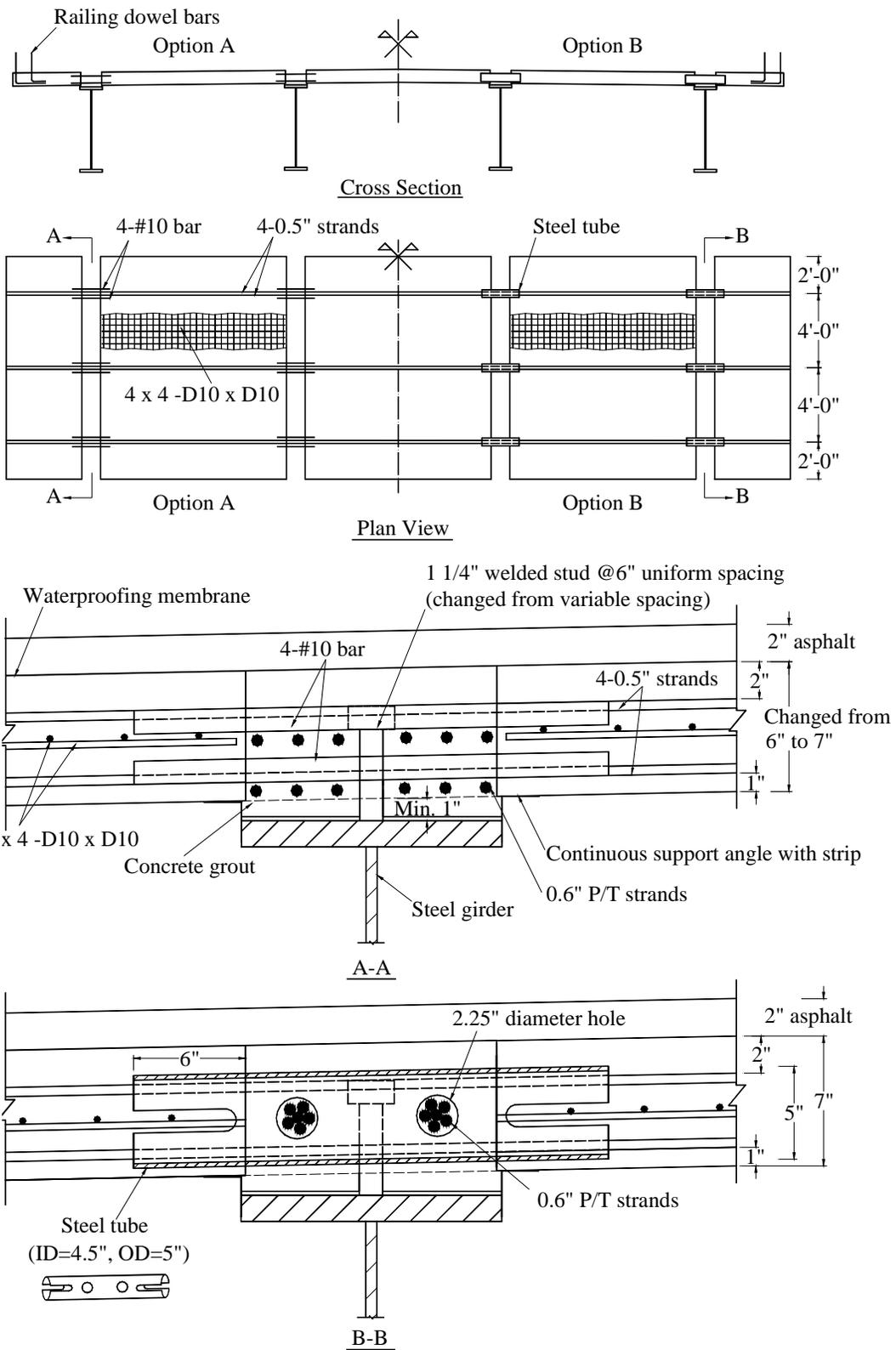


Figure 5.4-Two Options for Future Refinements.

### 5.1.3 System Refinements

A meeting was held among the UNL researchers, NDOR bridge engineers, precast producers, and contractors when the bridge deck panel construction was mostly completed. The possible simplifications were discussed in terms of panel production, handling, shipping, and construction. Some of the discussions are included below.

1. The typical panel width should be kept as 8 ft because a 12 ft-wide panel may introduce problems in terms of handling and shipping. The pretensioned strands can be changed from 2-0.5" strands at 2 ft spacing to 4-0.5" strands at 4 ft spacing. Accordingly, the compression rebar across the channel was replaced with 4#10 bars at 4 ft.

2. The width of post-tensioning channel can be constant as 14 in. for both concrete and steel beams. The amount of post-tensioning strands should be determined dependent on the bridge spans.

3. The 1 1/4 in. diameter stud will be kept for future project. To avoid any conflicts with the #7 bars across the channel, temporary nuts can be attached to the steel beam in the studs' location. After the panels are erected over the beam, the stud can be shot in the field. The feasibility of this suggestion is dependent on whether the added cost of nuts is less than substituting the 1 1/4 in. studs with 7/8 in. studs by the contractors in case of conflict.

4. Since Type K cement overlay adds a significant cost to the precast panel, the asphalt non-composite overlay and silica fume concrete were suggested in replacement of it. However, asphalt overlay has not been used very often in Nebraska for bridges. It is not recommended unless the time of construction is a factor of consideration. As indicated by the contractor, the cost of silica fume concrete is not much cheaper than

Type K cement. Furthermore, silica fume concrete is much harder to work with in comparison with Type K cement.

5. The contractor commented that pouring the post-tensioning channel and overlay in two steps was not a significant cost difference compared to pouring in a single step.

6. Erecting the panels by two front loaders is possible, although it may require drivers with some extra training to get familiar with the operation. In the case that the front loaders are used, the support system should be designed accordingly to carry the construction load.

7. The end panel may be produced with grouting holes so that grouting can be done for the gap between the end panel and the abutment cap.

## **5.2 CONCLUSIONS**

This report presents a full-depth precast prestressed bridge deck panel system. The precast panel is compressed in two directions, resulting in increased durability of the overall structure. The panel system is expected to last as long as the supporting steel or concrete beams. It can also be adopted for deck replacement with significant time savings. Some conclusions are presented below:

1. The panels are made of high performance precast prestressed concrete. Most of the creep, shrinkage, and temperature drop due to the cement hydration cycle occur before the deck is made composite with the rigid steel or concrete beams.
2. A continuous gap over the beam eliminates any potential problems related to quality of tendon grouting. Individual post-tensioning of strands results in ease of operation for the contractors.

3. The precast deck panel covers the full-width of the bridge, so there is no need for forming the overhangs.
4. The cast-in-place overlay allows for adjustments in roadway profile and provides an excellent riding surface.
5. The precast bridge deck panel system is cost competitive with cast-in-place concrete decks, while it is much faster to build and easy to maintain.

## CHAPTER 4

### NUDECK SYSTEM IMPLEMENTATION

#### 4.1 SYSTEM IMPLEMENTATION

##### 4.1.1 Implementation on Waterloo Northwest Bridge

After the stay-in-place (SIP) bridge deck panel system was developed by Badie and Tadros in 1997, NDOR assigned the Waterloo Northwest Bridge to be built using this system. The proposed bridge is located in Douglas County, Nebraska, on Highway No. US 275. It is a simple span structure of 128 ft. The cross section of the bridge consists of five girder lines spaced at 8.2 ft with a total width of 40.7 ft. This bridge was not built due to the high estimated cost of the panels from the local precast producers.

##### 4.1.2 Implementation on Skyline Bridge

The Skyline Bridge presents the first implementation of full-depth precast concrete NUDECK panel system (see [Figure 1](#)). It is located at 198<sup>th</sup> and West Dodge Road in Omaha, Nebraska. The bridge will carry Skyline Drive traffic over West Dodge Road (US 6 Expressway). Current average daily traffic is 1445 estimated to increase to 3110 vehicles by the year 2022. Construction of this bridge was completed in December of 2003. The bridge consists of two spans, 89 ft and 125 ft, a 25° skew, and 55 in. deep steel girders spaced at 10 ft-10 in. (see [Figure 2](#)). The precast bridge deck consists of twenty-six typical panels, with a width of 7.0 ft and a length identical to the bridge width, and two end panels, which house the post-tensioning anchorage block. The panel is 6 in.-thick with 2 in. Type K cement overlay. The concrete strength of the panel is 4,300 psi at release and 6,000 psi at service. The precast bridge deck panels are pretensioned

transversely and post-tensioned longitudinally. 16-0.6 in. strands are tensioned along each continuous 12 in.-channel over girder line.

Several new features were introduced to this project. The steel girders had 1.25 in. studs arranged in a single row over the steel girder web at a spacing of 6 in., rather than several rows of the smaller 3/4 in. or 7/8 in. studs. The bridge skew and the requirement for a sidewalk on one side made the deck panel geometry relatively challenging, especially that a requirement for a crown at the roadway centerline and a 2% cross slope was enforced.

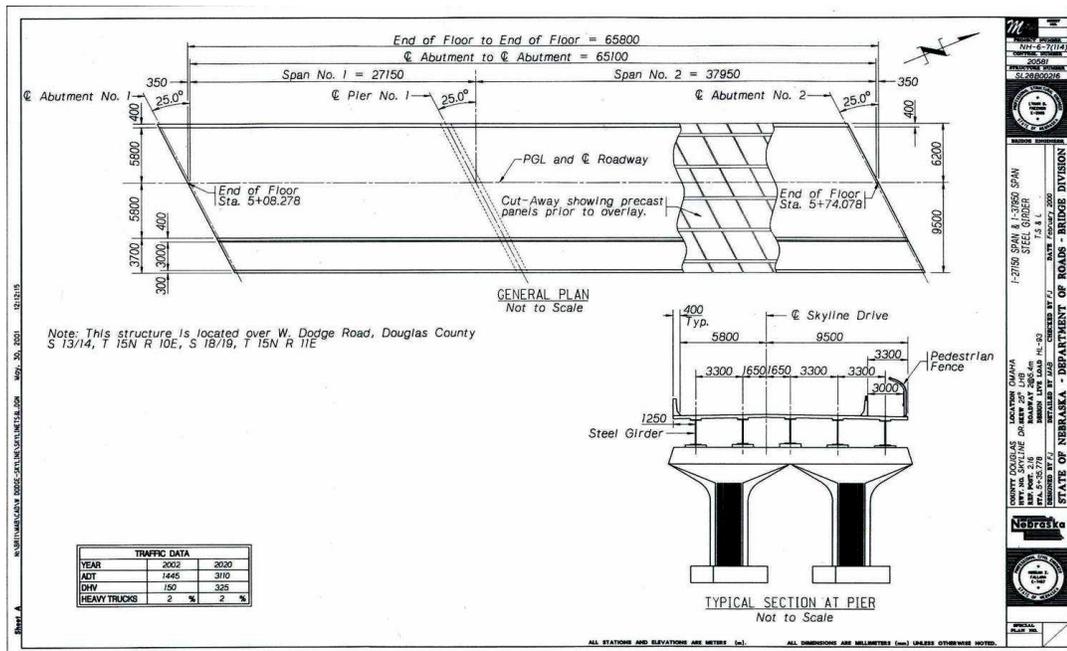


Figure 4.1-Implementation of NUDECK Bridge Deck Panel System



Figure 4.2-Skyline Bridge before Deck Erection

#### 4.2 ANALYSIS OF SKYLINE BRIDGE DECK

Service limit state analysis was performed to account for the post-tensioning. Some basic assumptions and notations are listed as follows:

1. Concrete strength at 28 days is 6,000 psi for both 6” precast deck panel and 1.5” CIP topping.
2. Beam bearing width is assumed as 12 inch.
3. The composite section of prefabricated steel plate beam and 6” precast deck panel is presented as composite section I; Composite section II refers to the full section including the steel beam, 6” deck panel, plus 1.5” CIP topping.
4. 2” topping load is considered to act on the composite section I.
5. At the positive moment area, the moment due to super-imposed dead load ( $M_{SID}$ ), the moment due to live load ( $M_{LL}$ ), and prestressing force change due to prestress losses applies to the composite section II. At the negative moment zone,  $M_{SID}$ ,

$M_{LL}$ , and prestressing force change are assumed to act on the composite section I in case that the 2" topping cracks.

The bearing face section is most critical in terms of service limit analysis. The detailed analysis results are presented in Appendix I. As shown in the table, the negative moment at the bearing face,  $x = 88.6$  ft of span 1, is -1944 kips-ft due to live load. The table gives the sum of stresses due to various force components for each section at service limit state. At the bearing face section, the total stress at the top fiber of composite section II (the top fiber of 1.5" topping) may be calculated as follows:

$$M_{SID} = 505.3 \text{ kips-ft}$$

$$0.8M_{LL} = 1944(0.8) = 1555.2 \text{ kips-ft}$$

The elastic section modulus of composite section II,  $S_{tc} = 6180.3 \text{ in.}^3$

$$f_{tc} \text{ (due to } M_{SID}) = M_{SID}/(NS_{tc}) = \frac{505.3(12)}{6(6180.3)} = 0.164 \text{ ksi (tensile)}$$

$$f_{tc} \text{ (due to } M_{LL}) = M_{SID}/(NS_{tc}) = \frac{1555.2(12)}{6(6180.3)} = 0.503 \text{ ksi (tensile)}$$

$$f_{tc} \text{ (due to prestressed force change)} = 0.07 \text{ ksi (tensile)}$$

$$f_{tc} \text{ (total)} = 0.737 \text{ ksi (tensile)} > f_{cr} = 0.581 \text{ ksi}$$

Therefore, the top fiber of the 1.5" topping at the bearing face section will crack. Conservatively, the properties of composite section I may be used for stress calculation, which results in a total stress of 0.25 ksi (tensile) at the top fiber of the 6" precast panel and less than  $f_{cr}$  value. Thus, no crack will occur in the precast deck panel and there is no need to worry about steel corrosion for this system. Note that most of the sections along the bridge spans are in compression at service limit state, which indicates the proposed system has the required high structural performance.

### 4.3 SKYLINE BRIDGE DECK PRODUCTION

The precast panels were produced in Concrete Industries INC., in Lincoln. [Figure 3](#) shows a typical precast deck panel reinforcement setup, which includes 4 pairs of 0.5 in. diameter pretensioning strands along the panel width (bridge transverse) direction. The conventional reinforcement consists of 8-#7 continuous bottom bars, 8-#7 short top bars across the open channel, and #5 rebar at 12” spacing as the secondary reinforcement. Also provided are the spirals at ends of pretensioning strands for confinement and for reduced demand of strand development length.

[Figure 4](#) presents the details to create the panel crown. Note that some minor modifications have been made in comparison with the demonstration panels to facilitate the panel production (see [Figure 5](#)). It normally takes one day to set up the deck panel reinforcement and the concrete is cast the next day. After the deck panel is poured and concrete hardens, it may be lifted out of the prestressing bed. The crown is then formed (see [Figures 6 and 7](#)). The panel was put on the steel supports which gave a 2% crown (see [Figure 8](#)). Afterwards, the top steel plate was removed and foam blocks were taken out. Once the top strands are cut, the panel will deflect following the supports’ elevation and 2% crown be formed accordingly. [Figure 9](#) illustrates the crown panels stacked up in the precast yard.



Figure 4.3- Reinforcement Setup of Skyline Bridge Panel



Figure 4.4-Details of Crown Panel Forming



Figure 4.5-A Close View of Crown Forming Detail



Figure 4.6-Pouring the Precast Deck



Figure 4.7-Skyline Bridge Panel Handling



Figure 4.8-Forming Crown



Figure 4.9-Crown Panels Stacked Up in the Precast Yard

#### 4.4 SKYLINE BRIDGE PANEL HANDLING AND SHIPPING

To avoid any buckling of the #7 bars across the open channels, the lifting points were carefully determined. Figure 10 shows a possible lifting scheme considering the deck panel is subject to its self-weight of 1.02 kips/ft. Accordingly, the moment diagram is given in Figure 11, in which the bending moment values at the open channel locations are nearly zero. Special steel frames were made to ship the precast panels (see Figure 12). Note that it is desirable if the gap between adjacent steel frames can be minimized so that the corresponding positive moment in the panel can be reduced.

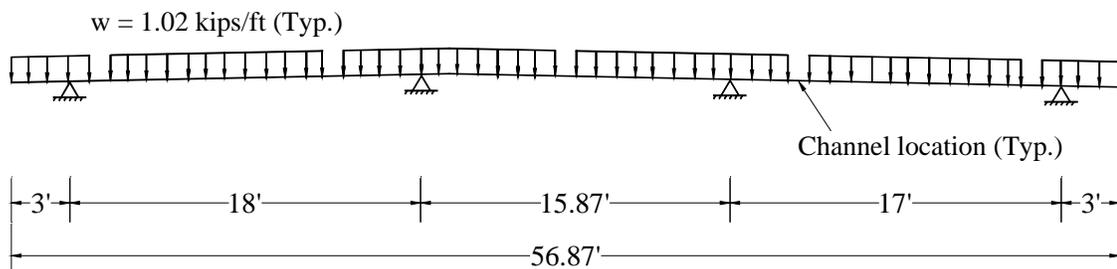


Figure 4.10-Panel Lifting Location Determination

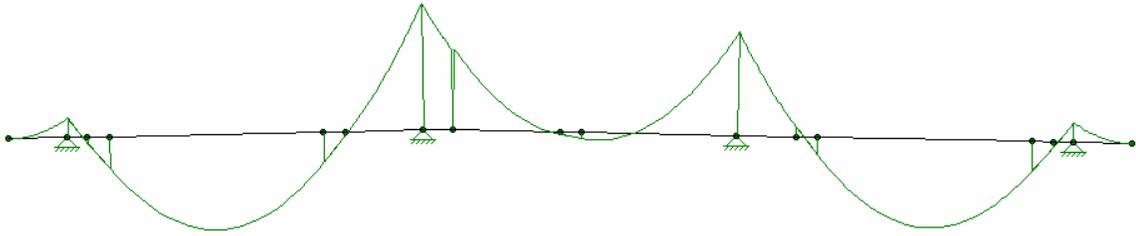


Figure 4.11-Moment Diagram due to Lifting



Figure 4.12-Skyline Bridge Panel Shipping

#### 4.5 SUPPORT SYSTEM

Prior to panel erection, an appropriate support system must be decided as to provide the required bridge elevation. Three possible support systems are presented herein.

1) Option A

As shown in [Figure 13](#), this support system consists of steel plate A (20"x6"x1/2"), steel angle B (8"x4"x1/4"), and stiffener C (4"x4"x1/4"). Steel plate A is put on plastic shims across the post-tensioning channel. Plastic shims can be placed over the steel girder top flange and may be adjusted to meet the demanded bridge profile. An 8 in.-long steel angle supports the adjacent panels at their corners along the channel direction. Stiffeners may be provided at the middle of the steel angle as needed.

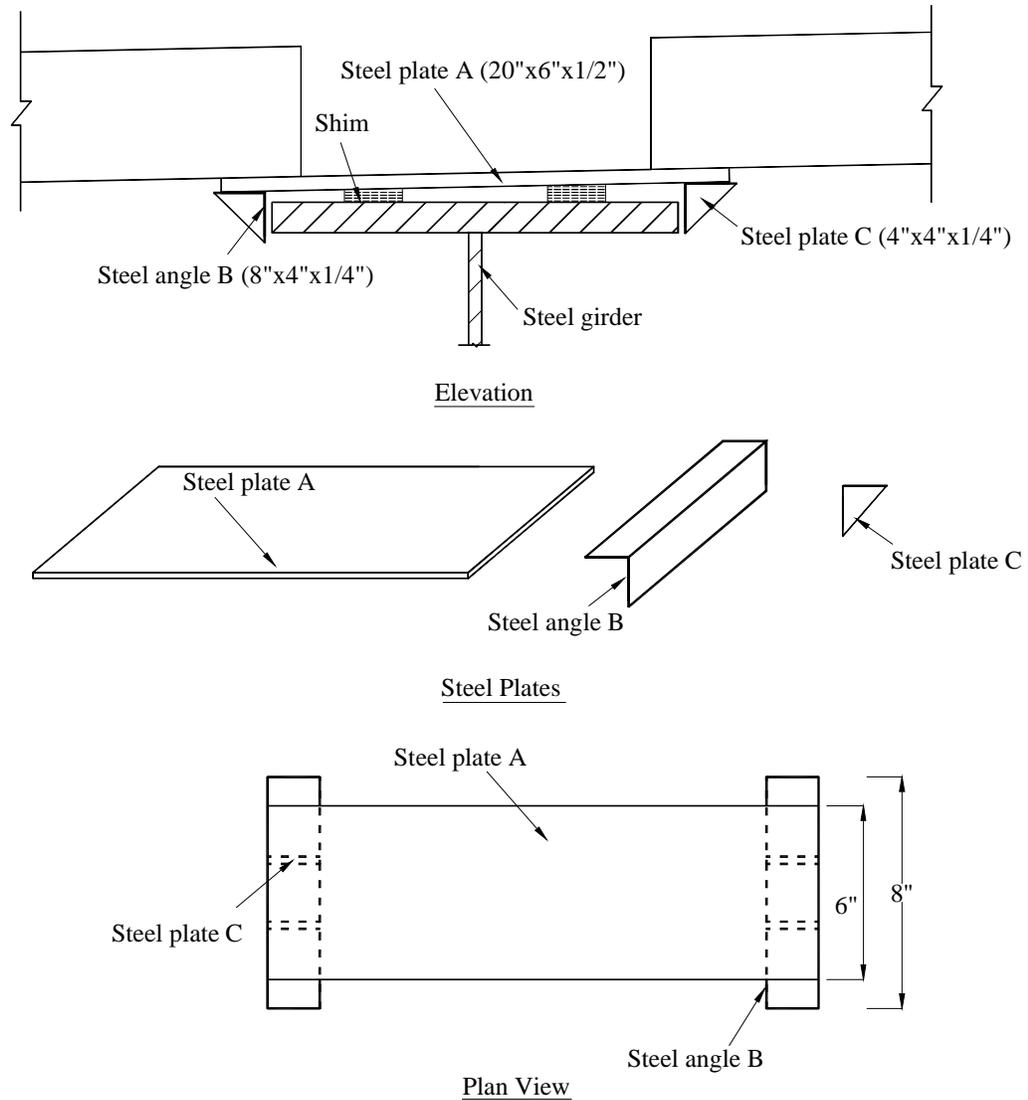


Figure 4.13-Option A Support System Details

2) Option B

Figure 14 illustrates a built-up T-section (or cut from WT 4x15.5) which carries the weight of adjacent panels. Similar to Option A, the panels are put over an 8 in.-wide T-section bottom flange at the corners along the channel. The T-section steel bottom flange may be cut to avoid any conflict with the 1 1/4 in. diameter studs. The height of T-section plate can go up to the bottom #7 bar.

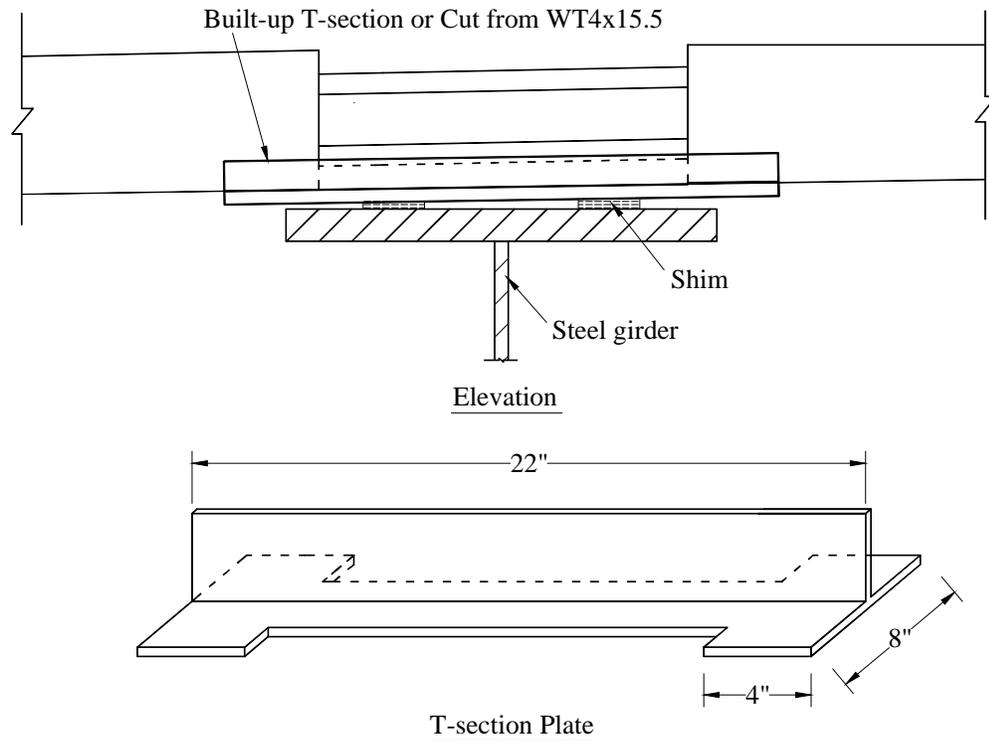


Figure 4.14-Option B Support System Details

3) Option C

Option C is the support system used in Skyline Bridge. It has continuous steel angels seated against the steel beam top flange. The interior beam has steel angles on both sides of its top flange while the exterior beam has steel angles only on its interior side. Separate strips cross the beam top flange and connect the steel angles (see Figure 15). The angels should be strong enough to carry the precast panel weight before the open channels are grouted.



Figure 4.15-Support System in Skyline Bridge

## 4.6 CONSTRUCTION OF SKYLINE BRIDGE DECK PANEL

### 4.6.1 Bridge Deck Post-tensioning Procedures

1. Pull the strands through the post-tensioning channel. Provide a minimum of 60 in.-long strand projection beyond the bridge floor at each end. Check that no strands are intersected or inter-wound.
2. Lodge the jaws in the “dead end” chucks with a light hammer. Use of excessive force may break the jaws.
3. Tension all the strand to 11.7 kips regardless of consequence. This force corresponds to a pressure of 1,500 psi on the pump gauge. Check again that no strands are intersected or inter-wound.
4. At the tension end, mark a point on each strand surface at a distance, say, 36 in., away from a fixed face, say the end anchorage. Call it Point A.
5. Based on the sequences given in the Appendix II, tension the strand to the final force of 38.9 kips (5,000 psi on the pressure gauge). Measure the displacement of Point A from 36 in. to (36 in. + X). Record X. The quantity X should be = 11.5 in. A tolerance

of  $\pm 0.5$  in. on the 11.5 average value is acceptable. If the value of X and the gauge pressure do not match the values shown, post-tensioning should stop and the engineer consulted.

6. If required by the owner, a check of the strand tension at the dead end may be performed by measuring the pressure required to begin a very small, say 0.25 in. elongation of the strand. That pressure must be equal to 5,000 psi less an allowance for friction, anchorage seating and slab shortening losses. A 10 percent loss is not unreasonable.

#### **4.6.2 Commentary of Post-tensioning Procedures**

1. In Step 3, 11.7 kips is 30% of the final tension force, 38.9 kips (173 KN). This initial tension of 11.7 kips is to take out the seating loss and the slag of the strand. A pressure of 1,500 psi on the pump gauge is determined based on the Load-Gauge Reading figure provided by Dywidag Systems International, INC. (see Appendix II). Similarly, a pressure of 5,000 psi on the pump gauge shown in Step 5 corresponds to the final force of 38.9 kips.
2. In Step 4, it is suggested that a point 36 in. away from a fixed face on the strand surface be marked. This 36 in. allows enough space for jack operation at the tension end.
3. In Step 5, the quantity of X is determined by

$$X = \frac{PL}{EA} = \frac{27.2(217)(12)}{28500(0.217)} = 11.5 \text{ in.}$$

where,

$$P = 38.9 - 11.7 = 27.2 \text{ kips}$$

L = length of the post-tensioning strand between the anchorage plates at the end panels,  
217 ft.

E = modulus of elasticity of strand, 28,500 ksi.

A = area of 0.6 in. diameter strand, 0.217 in.<sup>2</sup>

4. Note that the bridge deck panel shortening is determined as

$$\Delta = \frac{PL}{EA} = \frac{3112(217)(12)}{4696(3708.0)} = 0.5 \text{ in.}$$

where,

P = applied post-tensioning force to the deck panel cross section, which has totally five  
post-tensioning channels with 16 strands per channel, 16(5)(38.9) = 3112 kips

L = total length of the deck panel, 217 ft.

E = modulus of elasticity of deck concrete, which has a strength of 6,000 psi at 28 days.

$$E = 150^{1.5} (33) \sqrt{6000} / 1000 = 4,696 \text{ ksi.}$$

A = area of deck section, 52.35 ft-wide and 6 in.-thick. A = 51.5(12)(6) = 3708.0 in.<sup>2</sup>

Based on what was observed at the construction site, the panel shortened 7/16 in. at the  
89 ft span and 3/8 in. at the 125 ft span.

5. The post-tensioning sequences are determined such that symmetry is maintained  
during the post-tensioning operation (see Appendix II).

#### **4.6.3 Skyline Bridge Deck Panel Construction**

The precast panels were shipped from Lincoln to Omaha, which took a total of three  
days by three trucks. As shown in [Figures 16 \(a\) and \(b\)](#), the panel was erected by crane  
instead of the front loaders as initially specified by the contractor. Erecting each panel to

its required location took approximately fifteen minutes. For the firstly erected panels near the abutment, the #7 bars across the open channel conflicted with the 1 1/4 in. studs, which resulted in cutting those studs. 24-1 1/4 in. studs were removed and each large diameter stud was replaced with 2-7/8 in. studs. Figures 17-(a) to (c) shows the plan view of the bridge after several panels were erected. A typical panel at the channel location is illustrated in Figures 18 (a) and (b), in which the 1 1/4 in. studs, steel strips across the beam top flange, #7 bars, and 0.5 in. diameter pretensioning strands along the bridge transverse direction can be seen. Figure 19 presents the configuration of the shear key between panels. Once the backer rod and duct tape are put at the related locations, flowable concrete may be placed for the shear key (see Figure 20). Figure 21 illustrates the plan view of the Skyline Bridge before post-tensioning was applied. Afterwards, the post-tensioning strands may be pulled through the channels. Instead of using a special device such as the “Chinese Finger” to pull several strands simultaneously as proposed by UNL researchers, the contractor pulled each strand individually using a truck. Even though it was a little time-consuming, this procedure was completed without any problems (see Figure 22). The strands were anchored by one-time use chucks seated against the curved steel plate as shown in Figure 23. After making sure that no strands are intersected or inter-wound, post-tensioning can be applied following the given procedures (see Figure 24). Each strand was tensioned to a final force of 38.9 kips which was checked by both gauge reading and strand elongation (see Figure 25). As a result, the measured strand elongation matched very well with the expected value. Once the strands are post-tensioned, the precast panels become an integral unit supported by the steel angels. Afterwards, the post-tensioning channels were grouted to make the bridge deck

panels locked with the steel beams through composite action (see [Figures 26 and 27](#)). Type K non-shrinkage cement, shipped from Texas, was poured as the overlay (see [Figure 28](#)). Type K cement costs about \$1.7/ft<sup>2</sup> for this bridge, which is more expensive than silica fume concrete. However, according to the contractor, it is easier to work with than silica fume concrete.

The approach slab was poured prior to the placement of the concrete overlay and a joint was set between the approach slab and bridge floor end (see [Figure 29](#)). Galvanized female inserts were embedded in the curb, the top of which matched the top of overlay (see [Figure 30](#)). These inserts were set to connect the pedestrian fencing. [Figure 31](#) shows the barrier reinforcement setup and [Figure 32](#) illustrates the completed barrier. The bridge construction was finished in December 2003 (see [Figures 33 and 34](#)).



[Figure 4.16-\(a\)](#)-Erecting a Skyline Bridge Panel off the Truck



Figure 4.16-(b)-Erecting a Skyline Bridge Panel



Figure 4.17-(a)-Plan View of Skyline Bridge after Several Panels Placed



Figure 4.17-(b)-Plan View of Skyline Bridge Panels



Figure 4.17-(c)-Another Plan View of Skyline Bridge Panels



Figure 4.18-(a)-Plan View of a Typical Panel Channel

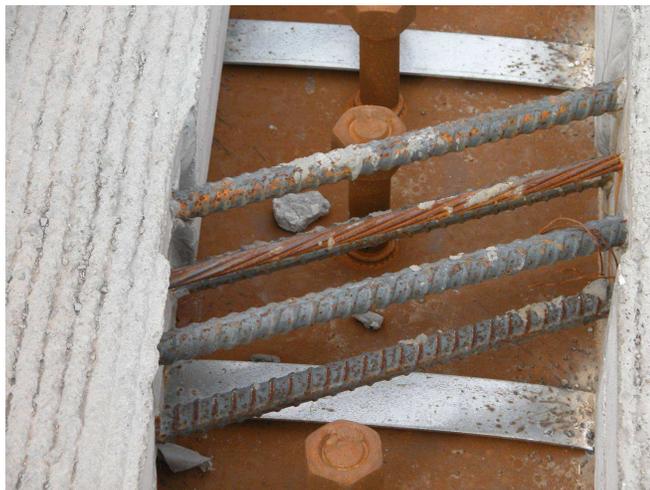


Figure 4.18-(b)-Channel Details before the Post-tensioning Strands are Pulled Through



Figure 4.19-Panel Shear Key Configuration



Figure 4.20-Curing the Shear Key



Figure 4.21-Plan View of Skyline Bridge before Post-tensioning



Figure 4.22-Post-tensioning Channel with Strands Pulled through



Figure 4.23-Strands Anchored with Chucks

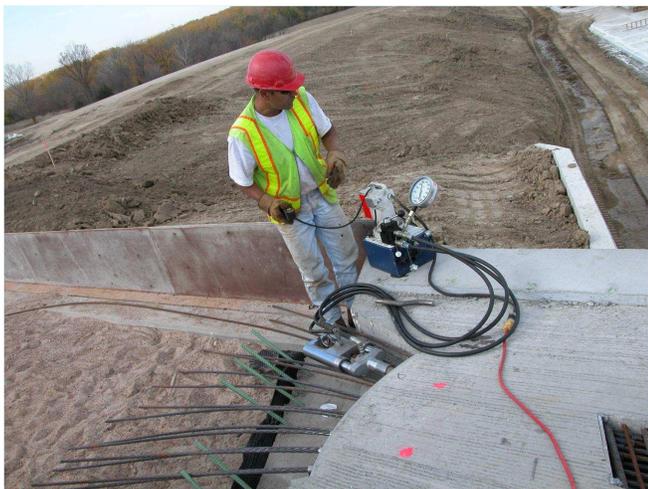


Figure 4.24-Post-tensioning Strands



Figure 4.25-Measuring Strand Elongation



Figure 4.26-Pouring Post-tensioning Channel



Figure 4.27-Curing Post-tensioning Channels



Figure 4.28-Curing Type K Cement Overlay



Figure 4.29-Joint between Approach Slab and Floor End



Figure 4.30-Galvanized Female Insert



Figure 4.31-Barrier Reinforcement



Figure 4.32-Completed Barrier



Figure 4.33-Plan View of Completed Skyline Bridge



Figure 4.34-Elevation of Completed Skyline Bridge

#### **4.7 COST ANALYSIS**

The bridge costs \$682,249, which converts to a unit cost of \$61.39/ft<sup>2</sup> (\$43.68/ft<sup>2</sup> for superstructure and \$17.71/ft<sup>2</sup> for substructure) based on Hawkins Construction Company.

The cost of the 6 in. precast deck panels is \$215,292, which converts to a unit cost of \$19.37/ft<sup>2</sup> (see details in Table 1). For comparison purpose, the bidding from other two contractors is given in Table 2.

Table 4.1-Detailed Project Cost Data from Hawkins Construction Company

Category	Item #	Item Description	Quantity	Unit	Unit Price (\$)	Amount (\$)	Unit Cost (\$/ft <sup>2</sup> )
Superstructure	1	NUDECK Panels	137.9	LS		215,292.0	19.37
	2	Class 47BD-30 Concrete for Bridge	16.8	m <sup>3</sup>	372.12	6,251.6	0.56
	3	Placing, Finishing, and Curing Type K Overlay	1,039.6	m <sup>2</sup>	13.87	14,419.3	1.30
	4	Concrete for Overlays-Type K	67.5	m <sup>3</sup>	281.95	19,031.6	1.71
	5	Steel Superstructure at Sta. 286+20	126,166.0	LS		209,662.0	18.87
	6	Pedestrian Barrier Rail	69.2	m	98.70	6,826.6	0.61
	7	Pedestrian Railing (Chain Link Type)	76.9	m	181.34	13,936.9	1.25
		Sub-total Cost (\$)					<b>485,420.0</b>
	Sub-total Unit Cost (\$/ft <sup>2</sup> )						<b>43.68</b>
Substructure	8	Abutment No. 1 Excavation	42.6	LS		2,542.0	0.23
	9	Abutment No. 2 Excavation	42.6	LS		2,542.0	0.23
	10	Pier No. 1 Excavation	40.6	LS		5,995.0	0.54
	11	Precompressed Polyurethane Foam Joint	34.6	m	98.99	3,423.1	0.31
	12	Fix Bearing	5.0	each	794.00	3,970.0	0.36
	13	Class 47BD-20 Concrete for Bridge	216.4	m <sup>3</sup>	299.60	64,833.4	5.83
	14	Structural Steel for Substructure	1,073.0	kg	2.75	2,950.8	0.27
	15	Concrete Slope Protection	38.5	m <sup>2</sup>	76.74	2,954.5	0.27
	16	Epoxy Coated Reinforcing Steel	18,656.0	kg	1.41	26,305.0	2.37
	17	Subsurface Drainage Matting	74.4	m <sup>2</sup>	12.18	906.2	0.08
	18	HP 310 mm x 79 kg Steel Piling	1,449.1	m	54.76	79,352.7	7.14
	19	38 mm Conduit in Bridge	80.4	m	13.12	1,054.2	0.09
	Sub-total Cost (\$)					<b>196,828.8</b>	
	Sub-total Unit Cost (\$/ft <sup>2</sup> )						<b>17.71</b>
Total Cost (\$)						<b>682,248.8</b>	
Total Unit Cost (\$/ft <sup>2</sup> )							<b>61.39</b>

Note: The total deck area = 11,112.98 ft<sup>2</sup>.

Table 4.2-Bidding Data from Other Two Contractors

Category	Item #	Item Description	Quantity	Unit	VRANA & SON CONSTRUCTION CO.			LUXA CONSTRUCTION COMPANY		
					Unit Price	Amount	Unit cost	Unit Price	Amount	Unit cost
Superstructure	1	NUDECK Panels	137.90	LS	244,050.00	244,050.00	21.96	261,586.50	261,586.50	23.54
	2	Class 47BD-30 Concrete for Bridge	16.80	m3	500.10	8,401.68	0.76	1,067.70	17,937.36	1.61
	3	Placing, Finishing, and Curing Type K Overlay	1,039.60	m2	22.70	23,598.92	2.12	37.37	38,849.85	3.50
	4	Concrete for Overlays-Type K	67.50	m3	335.90	22,673.25	2.04	589.18	39,769.65	3.58
	5	Steel Superstructure at Sta. 286+20	126,166.00	LS		238740	21.48		308565.3	27.77
	6	Pedestrian Barrier Rail	69.17	m	100.8	6971.832	0.63	136.67	9452.78055	0.85
	7	Pedestrian Railing (Chain Link Type)	76.86	m	175.8	13511.109	1.22	119.58	9190.3209	0.83
Sub-total Cost (\$)					557,946.79			685,351.76		
Sub-total Unit Cost (\$/ft2)					50.21			61.67		
Substructure	8	Abutment No. 1 Excavation	42.60	LS		2,970.00	0.27		8,541.60	0.77
	9	Abutment No. 2 Excavation	42.60	LS		2,970.00	0.27		8,541.60	0.77
	10	Pier No. 1 Excavation	40.60	LS		3,660.00	0.33		17,083.20	1.54
	11	Precompressed Polyurethane Foam Joint	34.58	m	130.00	4,495.40	0.40	125.99	4,356.73	0.39
	12	Fix Bearing	5.00	each	910.00	4,550.00	0.41	907.55	4,537.75	0.41
	13	Class 47BD-20 Concrete for Bridge	216.40	m3	387.60	83,876.64	7.55	774.08	167,510.91	15.07
	14	Structural Steel for Substructure	1,073.00	kg	4.20	4,506.60	0.41	4.80	5,150.40	0.46
	15	Concrete Slope Protection	38.50	m2	208.20	8,015.70	0.72	77.84	2,996.84	0.27
	16	Epoxy Coated Reinforcing Steel	18,656.00	kg	1.70	31,715.20	2.85	1.69	31,528.64	2.84
	17	Subsurface Drainage Matting	74.40	m2	23.50	1,748.40	0.16	43.78	3,257.23	0.29
	18	HP 310 mm x 79 kg Steel Piling	1,449.10	m	61.80	89,554.38	8.06	79.01	114,493.39	10.30
19	38 mm Conduit in Bridge	80.35	m	55.00	4,419.25	0.40	13.35	1,072.67	0.10	
Sub-total Cost (\$)					242,481.57			369,070.97		
Sub-total Unit Cost (\$/ft2)					21.82			33.21		
Total Cost (\$)					800,428.36			1,054,422.73		
Total Unit Cost (\$/ft2)					72.03			94.88		

## CHAPTER 3

### NUDECK SYSTEM DETAILING

#### 3.1 CROWNING THE PRECAST PANEL

##### 3.1.1 Creating the Crown Panel

Since the precast panel is transversely pretensioned, creating a 2% crown was one of the most challenging issues during the NUDECK system development. One possible way of achieving the required crown is to vary the thickness of concrete overlay based on the required profile. However, it will result in a bridge deck panel with various thickness, which is not acceptable by NDOR. Eventually, an innovative detail was created utilizing a plastic rod as a “hinge”, which enables the panel to rotate under its self-weight and form a crown.

Two precast panel specimens were produced to demonstrate the proposed crowning details. The first panel was made in Concrete Industries INC, Lincoln, as shown in [Figure 1](#). This panel is about 55 ft long, 8 ft wide, and 6 in. thick with a skew of 25°. [Figure 2](#) presents a closer view of panel at the crown location. To achieve a crown of 2%, an 8 ft-long, 1 3/4 in. diameter HVC plastic rod was put along the centerline of crown. The plastic rod is located 1 in. from its center to the panel top fiber. It works as a “hinge” allowing the panel to rotate about the plastic rod axis when the panel is lifted out of prestressing bed. Four 0.5 in. diameter pretensioned strands at the panel top layer pass through the center of plastic rod. Underneath the plastic rod is a steel plate, which has slots at the related locations allowing the bottom strands to pass through. As illustrated in [Figure 3](#), 4-#7 bars are discontinuous at the plastic rod and steel plate location. Note that

each pair of pretensioned strands at the top and bottom layers is tied vertically by wires to avoid the strands' splitting when the panel rotates along the plastic rod. The bottom strands are debonded approximately 12 in. at each side of plastic rod, which allows the bottom strands to shorten relative to the panel concrete. The steel plate is tapered along the panel thickness direction as shown in Figure 4. With slots at the bottom strand location, the steel plate can be removed when the panel is lifted out of prestressing bed. Figures 5 and 6 illustrate the panel being poured and seated in the prestressing bed, respectively. The crown was created when the panel was lifted out of prestressing bed (see Figure 7). Afterwards, it was put on a template with a 2% crown during handling and shipping.



Figure 3.1-Plan View of Crown Panel Reinforcement Setup



Figure 3.2-Panel Detailing at the Crown Location



Figure 3.3-Details of Crown Forming



Figure 3.4-Steel Plate underneath the Plastic Rod



Figure 3.5-Casting the Panel



Figure 3.6-Panel Seated in the Prestressing Bed



Figure 3.7-Lifting Panel out of the Prestressing Bed

This panel was successfully produced with a 2% crown. A meeting was held among the UNL researchers, precast producers, contractors, and NDOR bridge engineers to discuss any possible problems using the modified details. Some of the issues and concerns are listed as follows:

- 1) Some contractors proposed the question of erecting the precast panels by two CAT 950F front end loaders, which necessitates a high amount of positive reinforcement to

resist the construction loading. In the current detailing, only four pairs of pretensioned strands are continuous along the bridge transverse direction, which is inadequate to carry the loading due to the front end loaders.

2) After the bottom steel plate is removed from the panel, the section is pre-cracked at the bottom, which requires grouting after the crown is created. Since it is hard to do grouting underneath the panel, further simplification is needed.

3) Since the hinge locates at the top of the deck panel and the panel might rotate back and forth about the plastic rod when the live loading is on and off, some of the NDOR bridge designers were concerned about fatigue of the bottom strands.

### **3.1.2 Consideration of the Feedback**

As indicated by the local contractors, the CAT 950F wheel loader is a commonly used machine on many job sites. It can quickly be fitted with the required attachment and is easy to rent locally. Installing the precast panels require two wheel loaders. The total weight of each machine is 35,000 lbs and its width is 9 ft. About 70% to 80% of the machine weight goes to its front axle during panel erection. An analysis was performed to determine the required positive reinforcement to resist the construction loading.

Figure 8 shows a possible arrangement of loaders to determine the maximum positive moment. Considering two wheel loaders, the machine load =  $2(35) = 70$  kips. It is conservatively assumed that 80% of the machine weight goes to the front axle, which gives  $0.8(70) = 56$  kips. Note that each deck panel is installed by two loaders, which indicates the panel weight of 27 kips is distributed by four front wheels. Thus, the load

per front axel =  $(56 + 27)/4 = 20.75$  kips. This force is distributed to an effective tire width of 20 in., which equivalently applies a uniform load,  $w = 20.75/(20)*12 = 12.45$  kips/ft. As shown in [Figure 8](#), a simple-span beam, 9.84 ft long, is subject to this uniform load, which results in a maximum positive moment of  $20.75(1/2)9.84 - 20.75(14+10)/12 = 60.6$  ft-kips. Assuming a load factor of 1.75 for this vehicle load plus the deck weight it carries, the factored moment at ultimate is  $1.75(60.6) = 106.1$  ft-kips. The moment due to the deck weight underneath the loader is  $0.15(7)(6)/12(9.84)^2/8 = 6.4$  ft-kips. Consider a load factor of 1.5 for the deck self-weight, which gives a factored moment of  $1.5(6.4) = 9.6$  ft-kips. Thus, the total factored moment is  $106.1 + 9.6 = 115.7$  ft-kips.

Based on the analysis, 8#7 bars at the deck bottom were made continuous along the transverse direction to provide adequate flexural capacity and carry the wheel loaders. With 8#7 bar plus 4-0.5 in. diameter bottom strands, the flexural capacity is approximately 166 ft-kips.

In addition, the plastic rod was moved to the panel bottom according to the feedback from precast producers and the NDOR bridge engineers. Consequently, three pairs of A706 #7 bars were placed at the deck top and welded after the crown formation to maintain the panel's stability during handling and shipping.

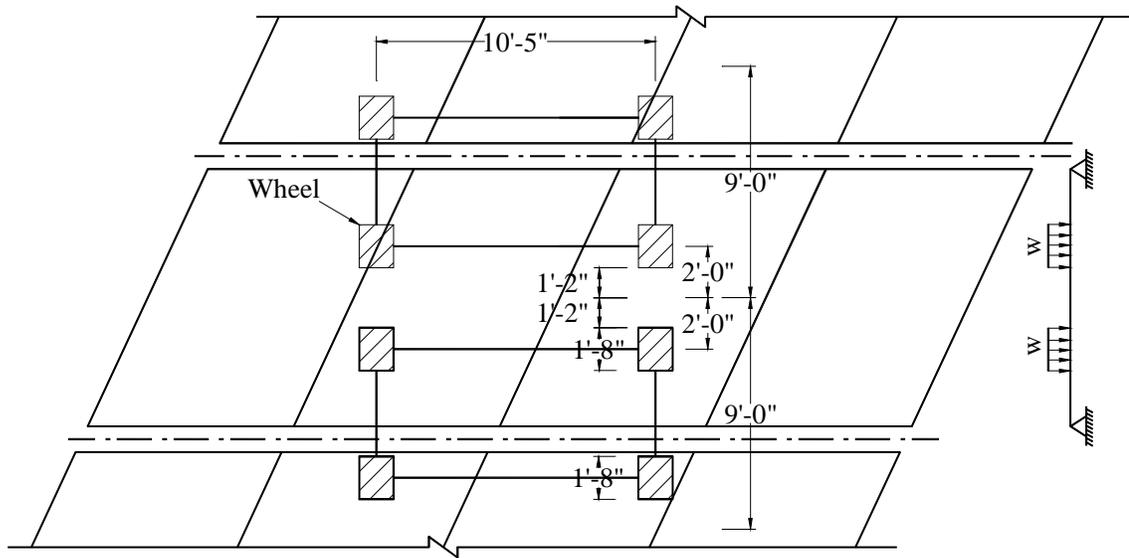


Figure 3.8-CAT 950F Wheel Loader Arrangement

### 3.1.3 Optimizing Crown Panel Details

After the feedback from various agencies was taken into account, a 10 ft x 8 ft x 6 in. crown panel specimen was produced at Rinker Materials to demonstrate the modified details. As shown in Figures 9 and 10, four pairs of 0.5" diameter, Grade 270 ksi strands at 2 ft spacing were arranged along the length direction. Three pairs A706 #7 bars were placed between the top strands with 1 in. clear cover to the panel top fiber. To avoid any concrete crushing under the bottom #7 rebar when the panel deflects to form the crown, a compressible random oriented fiber was put around the #7 bar (see Figures 11 and 12). Also shown is a foam block to allow cutting the top strands after the concrete is poured. Note that all bottom #7 bars pass through the plastic rod to ensure adequate flexural capacity. Figure 13 illustrates the panel after concrete casting in the precast yard. Once the panel is lifted out of the prestressing bed, it is placed on a wood template which has a 2% crown (see Figure 14). Afterward, the top steel plate was removed as shown in Figure

15. The four top 0.5 in. pretensioned strands were cut at the crown location, which allowed the panel to deflect following the given profile of the wood template (see [Figures 16 and 17](#)). Three pairs of A706 #7 bars were welded by steel plates to maintain the crown during handling and shipping (see [Figure 18](#)). Finally, the pockets were grouted at the crown location.

The crown forming details were updated to allow for the operation of construction vehicles over the precast panels. In comparison with the first crown panel made in Concrete Industries INC., the new detailing is expected to facilitate the panel production while providing better structural performance during panel handling and shipping.

The crown panel production procedure is summarized as follows: 1) prefabricate the panel; 2) remove the panel from the prestressing bed; 3) place the panel on a crown template or support; 4) remove the foam block (or steel tube) and the top steel plate; 5) cut the top strands; 6) weld the A706 #7 bars; and 7) grout the pockets.

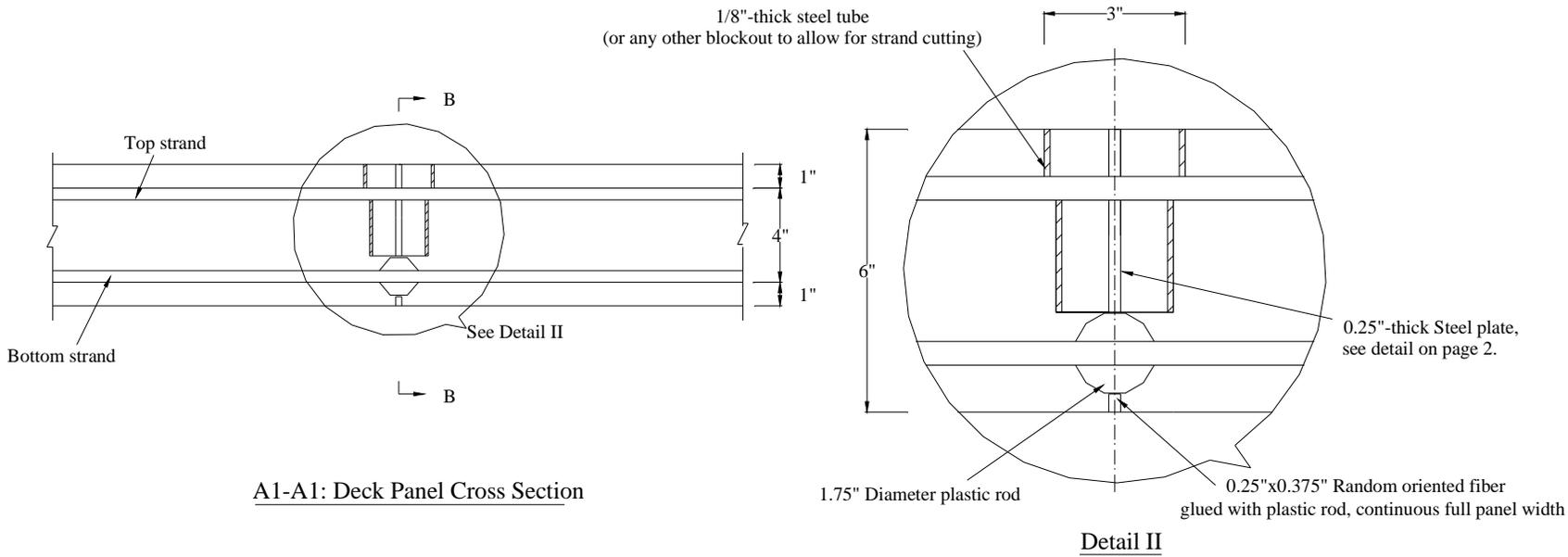
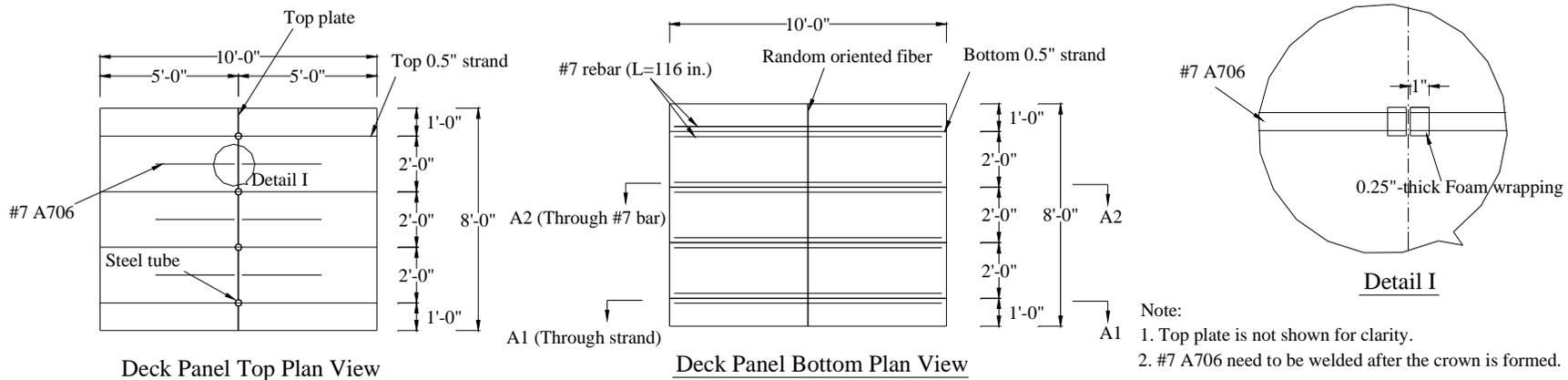


Figure 3.9-Production of a 10 ft x 8 ft Crown Panel Details 1

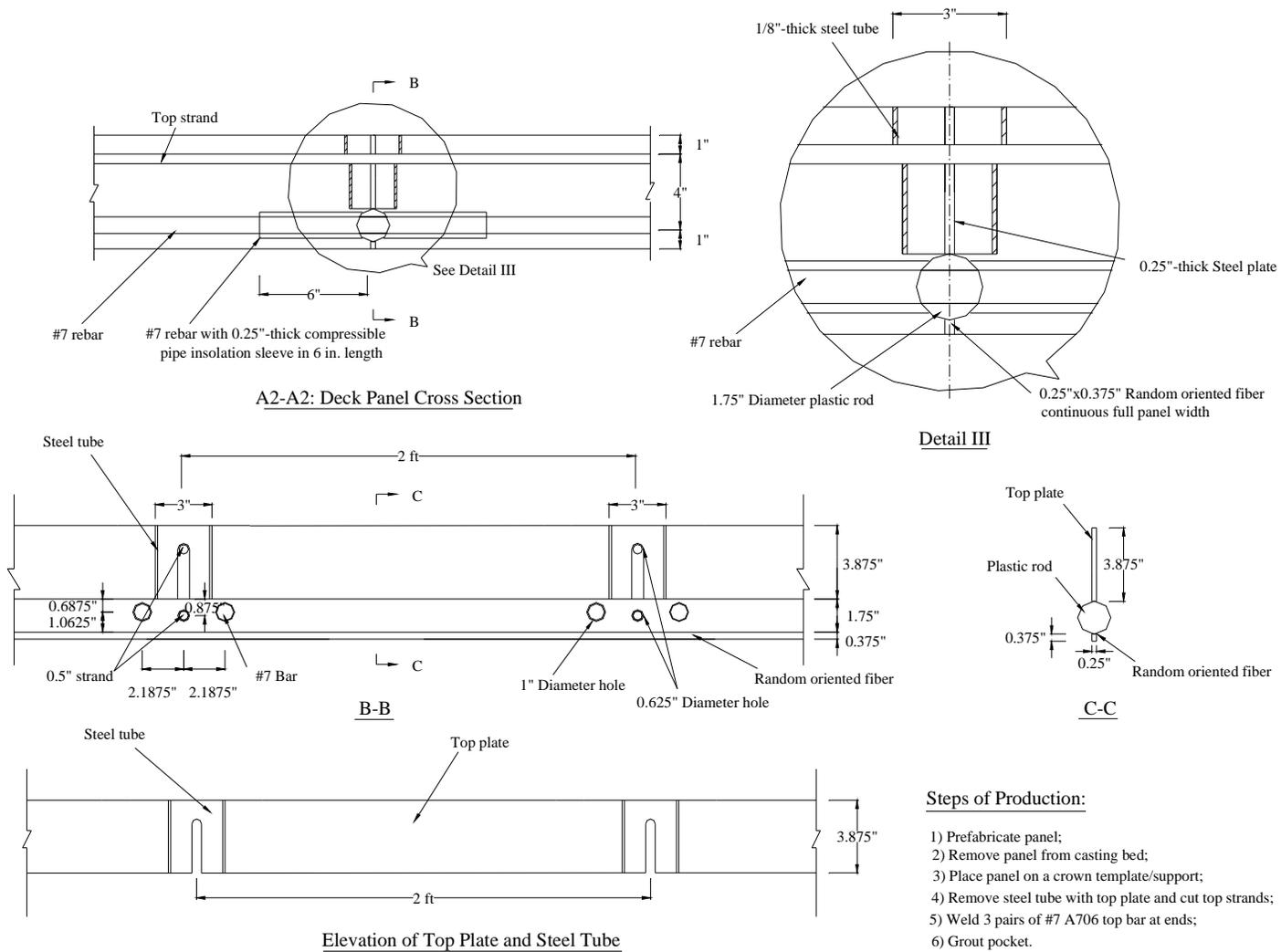
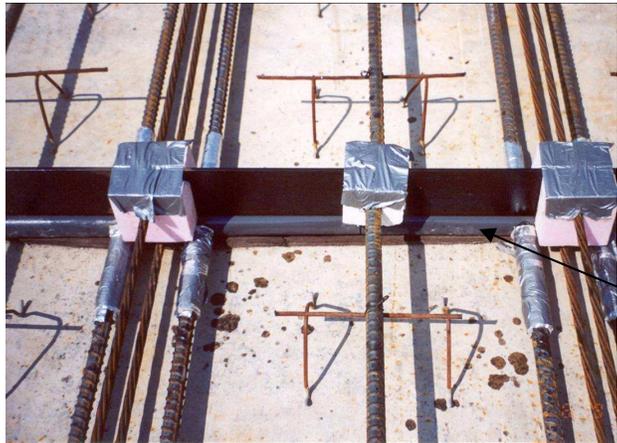


Figure 3.10-Production of a 10 ft x 8 ft Crown Panel Details 2



Plastic Rod

Figure 3.11-A Closer View of the Details at the Crown Location



Figure 3.12-Crown Forming Details

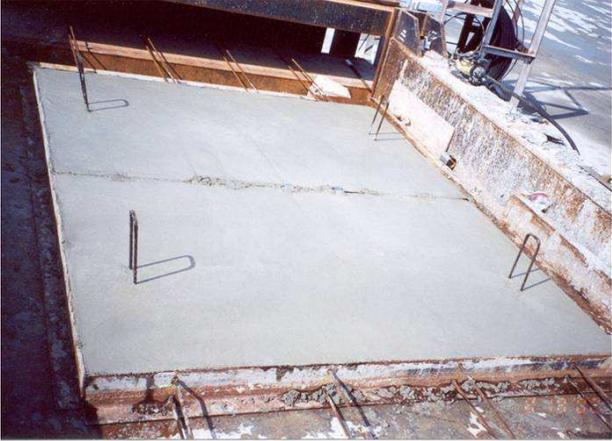


Figure 3.13-Pouring Concrete for the Panel



Figure 3.14-Panel Seated over a Wood Template



Figure 3.15-Removing the Top Steel Plate



Figure 3.16-Cutting the Top Pretensioned Strands



Figure 3.17-Forming a 2% Crown



Figure 3.18-Welding the A706 #7 Rebar

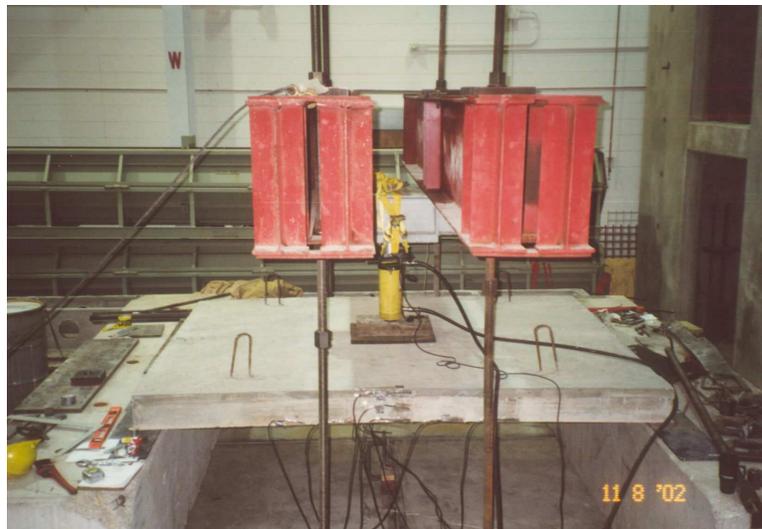


Figure 3.19-Grouting the Pockets at the Crown Location

### 3.1.4 Testing the Crown Panel Specimen

After the crown panel was made in Rinker Materials, it was shipped to the UNO Structures Lab for testing. A 2 in. concrete topping was cast to simulate the concrete

overlay of the actual bridge deck. This testing was intended to evaluate the structural performance of the crown panel using the modified details. As shown in [Figure 20](#), the panel was supported by two concrete blocks with a clear span of 9 ft between the centerline of bearing. Also shown are two steel beams seated against high strength threaded rods, which were tied with the concrete floor. The hydraulic jack was put on a steel plate at the panel center. Several strain gauges are attached to the various locations and the deflection can be measured by the testing setup. Concrete strength at 28 days was approximately 6,500 psi for the 6 in.-thick deck panel and 5,000 psi for the 2 in. overlay. [Figure 21](#) illustrates that the panel was tested to failure. The ultimate loading is approximately 55,000 lb, higher than estimated, at which the pretensioned strands had bond failure. The load-deflection diagram is shown in [Figure 22](#). Note that the panel deflected about 0.55 in. under the peak loading. Despite that the strands had bond failure, the modified hinge details exhibited excellent structural performance as expected, which was the main objective of the testing.



[Figure 3.20-Crown Panel Testing Setup](#)



Figure 3.21-Crown Panel Testing to Failure



Figure 3.22-Crown Panel Load-deflection Diagram

### 3.1.5 Alternative Crown Forming Scheme

The precast producers and some NDOR bridge designers proposed an alternative scheme to create the crown. It was suggested that the panels were produced as flat and there was no need to form the crown intentionally. Once the panels are placed on the beams, they may deflect under their self-weight following the elevation of supports,

which results in an automatic crown forming. This scheme sounds attractive since it removes the crown forming procedure and largely simplifies the panel production. An analysis was performed to verify the feasibility of this scheme.

A typical unit panel, 51.5 ft x 7 ft x 6 in., is considered to be placed on the angle support system (see [Figure 23](#)). The elevation of supports reflects a 2% crown along the panel transverse direction. Note that the panel sits on supports A and B, which creates a cantilever for the panel parts exterior to supports A and B. The partial panel for analysis, which locates the right side of support A, is illustrated in [Figure 23](#). The panel deflects due to self-weight and matches the supports' elevation. Section 1 refers to the section which has 2-0.5 in. diameter pretensioned strands and 4-#7 bars. Section 2 is the precast section, 7 ft x 6 inch. The section properties, area (A) and moment of inertia (I), of Sections 1 and 2 are given as follows:

$$\text{For Section 1, } A = 4[4(0.6) + 2(0.153)] = 10.82 \text{ in.}^2;$$

$$I = 4(4) \left[ \frac{\pi(0.875^4)}{64} + 0.6(1.563^2) \right] + 4(2) \left[ \frac{\pi(0.5^4)}{64} + 0.153(1.375^2) \right] = 26.244 \text{ in.}^4;$$

(see Chapter 2 for detailed dimension)

$$\text{For Section 2, } A = 7(12)6 = 504 \text{ in.}^2; I = \frac{84(6^3)}{12} = 1512 \text{ in.}^4. \text{ The uniform loading}$$

due to panel self-weight,  $w$ , is  $0.15(7)(6)/12 = 0.525$  kips/ft. Using the RISA program, the moment at section I-I is 150.97 ft-kips and the deflection at the free end is 11.2 inch.

$$\text{The stress of top fiber at Section I-I is determined as } -\frac{My_t}{I} = -\frac{151.0(12)3}{504} = -3.595 \text{ ksi}$$

(tension). The prestress force due to 8-0.5 in. pretensioned strands,  $P = 8(0.9)202.5(0.153) = 223.1$  kips assuming 10% prestress loss. The compression stress

due to prestress =  $\frac{P}{A} = \frac{223.1}{7(12)6} = 0.443$  ksi. As a result, the final stress at the top fiber of Section I-I =  $0.443 - 3.595 = -3.152$  ksi (tension). The tensile stress at this section is so high that concrete will definitely crack. Thus, this alternative crown panel concept is not feasible.

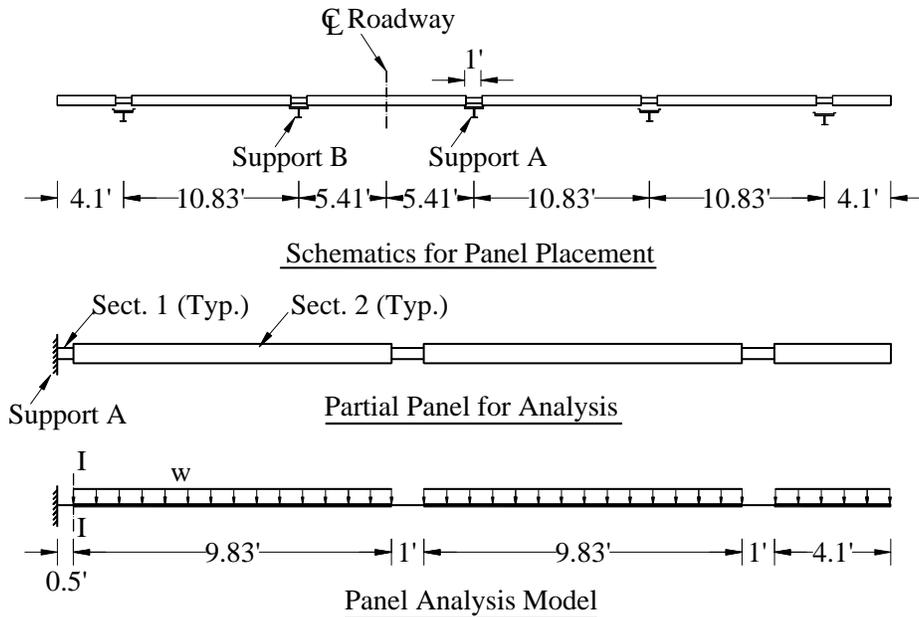


Figure 3.23-Alternative Crown Forming Scheme

## 3.2 POST-TENSIONING ANCHORAGE DETAILS

### 3.2.1 Considerations of the Anchorage Zone

The precast deck panel is only 6 in. thick while prestressed with 16-0.6” diameter strands longitudinally along each channel. Therefore, the anchorage zone is critical for design. Unique concrete post-tensioning anchorage blocks are proposed and illustrated in Figures 24 and 25. The post-tensioning concrete block would be left in place as part of the abutment back wall. The concrete block houses standard Freyssinet four-strand thin member anchors. It is creatively reinforced with stud rail reinforcement which has just recently been endorsed by ACI 318 Building Code for reinforcement of thin slabs

in punching shear. The anchorage reinforcement consists of  $\frac{3}{4}$  in. stud rail at 3 in. spacing and #4 rebar at 3 in. each way. The stud rail reinforcement is used to resist splitting due to post-tensioning. Based on a preliminary design, 6#6 rebar are included as the tensioning tie along the anchorage block end.

Alternatively, a structural steel anchorage block may be more economical in the Northeast and other parts of the country. It does not require Freyssinet anchors or precast concrete blocks. The steel anchorage block consists of several steel plates which form a frame transferring the post-tensioning force to the precast panel (see Figure 26). Despite a higher cost than the concrete block alternative, removal of the steel block for future reuse should be feasible so that the overall cost can be distributed.

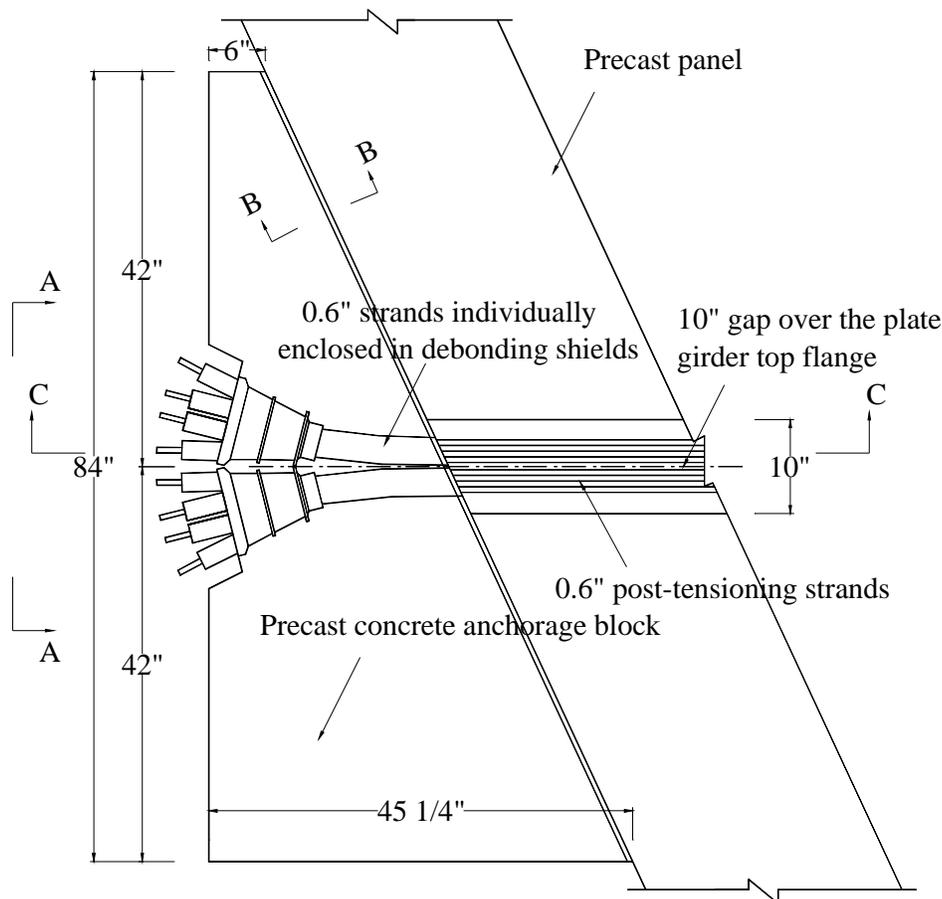


Figure 3.24-Plan View of Concrete Post-tensioning Anchorage Block

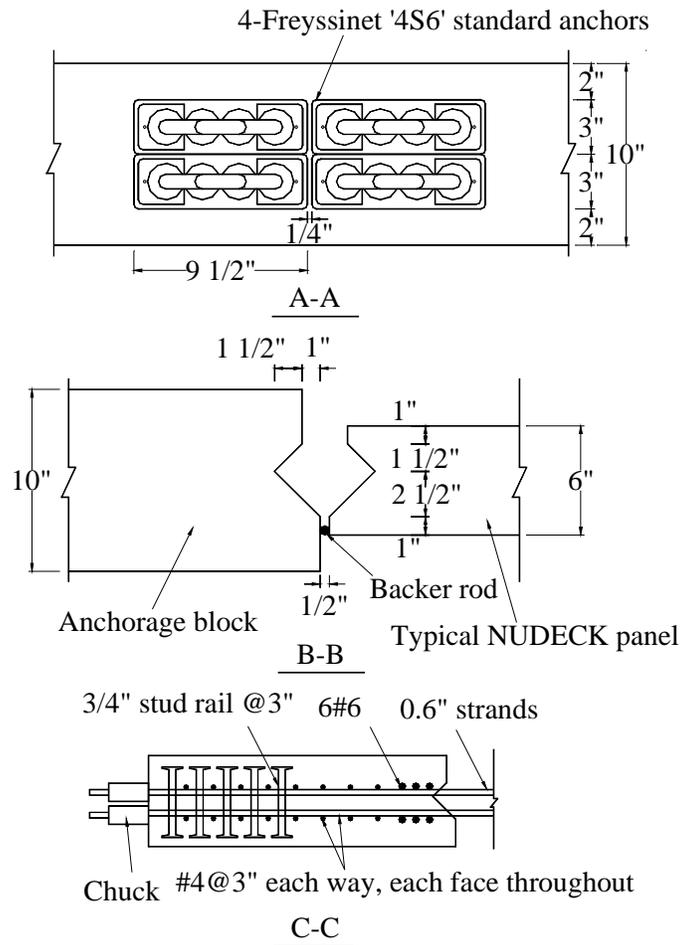


Figure 3.25-Concrete P/T Anchorage Block Sections at A-A, B-B, and C-C

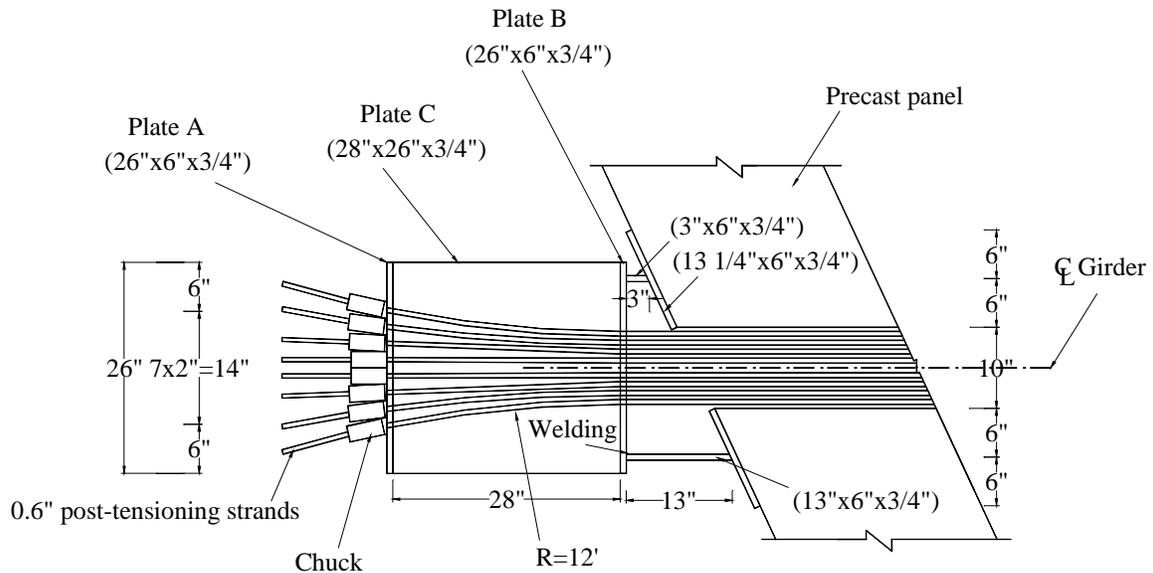


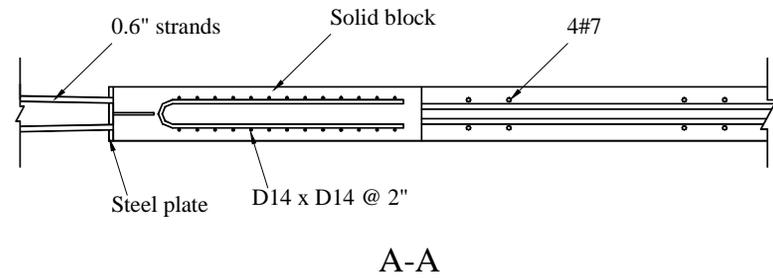
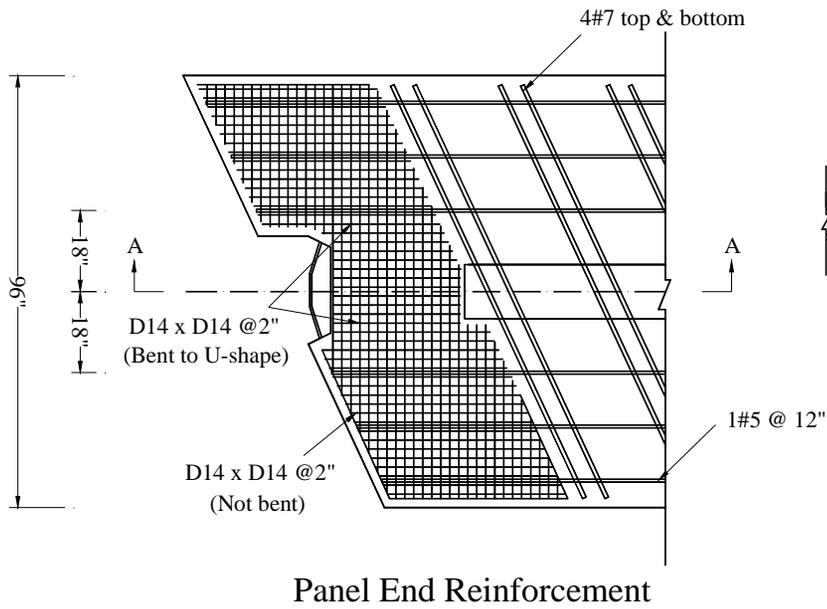
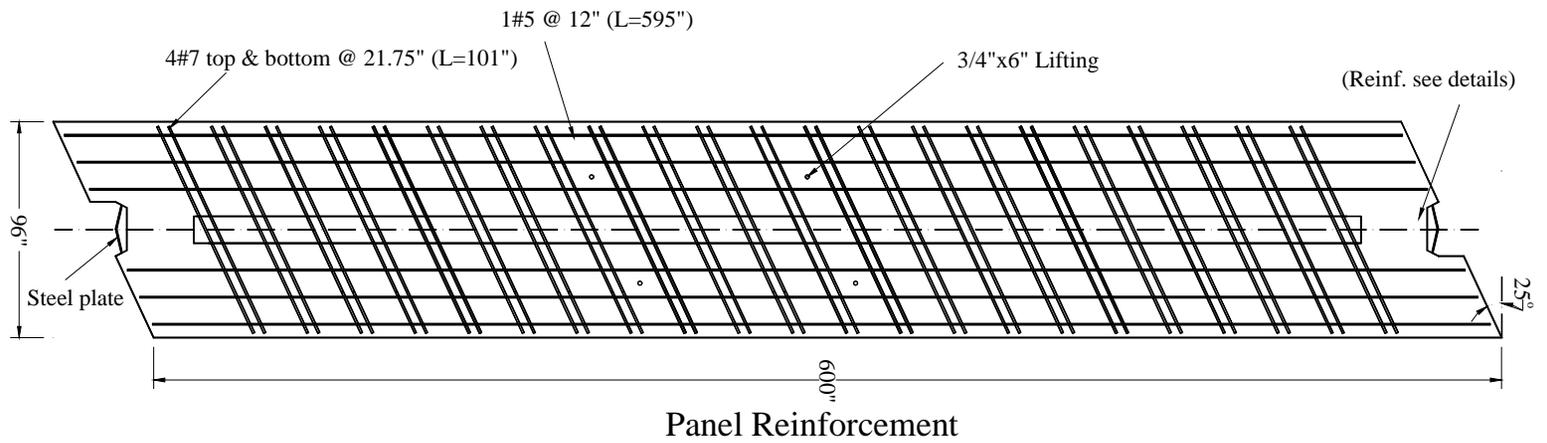
Figure 3.26-Plan View of Structural Steel P/T Anchorage Block Alternative

### 3.2.2 Post-tensioning Panel Demonstration at the UNO Structures Lab

Due to conflicts in laying out the anchorage block with the approach slab in the implementation bridge, NDOR bridge engineer recommended that the anchorage block be made integral with the end panel. With this solution, there is no need to cast or assemble the anchorage block at the bridge site, which greatly facilitates the bridge construction. Accordingly, the Skyline Bridge had 26 typical precast panels and 2 special end panels.

For demonstration purpose, a bridge deck panel specimen was made in the UNO structures lab. This specimen, 50 ft-long, 8 ft-wide, and 6 in.-thick, represents an 8-ft wide strip over a bridge girder line (see [Figure 27](#)). There is a 12 in. wide open channel in the middle of the panel for post-tensioning. No shear key was included in this specimen since it was made as an integral panel strip. The panel was reinforced by 4-#7 bars at 21.75 in. spacing transversely and #5 bars at 12 in. spacing longitudinally. Totally 16-0.6" strands were longitudinally post-tensioned. Also shown are the anchorage details at the panel end including welded reinforcement mesh, D14 x D14 @ 2 in. spacing, to resist the post-tensioning force. Note that one mesh was bent into a U-shape at the panel end to take the bursting force since the post-tensioning strands were not vertically straight along the longitudinal direction. The reinforcement mesh details are given in [Figure 28](#). A curved steel plate with an outside diameter of 48 in. was put at the panel end to provide adequate bearing due to post-tensioning. The curved plate was welded with a straight plate at its mid-thickness such that they could be embedded with the panel (see [Figure 29](#)). There were totally 16-11/16 in. diameter holes in the curved plate allowing the plastic shields to pass through.

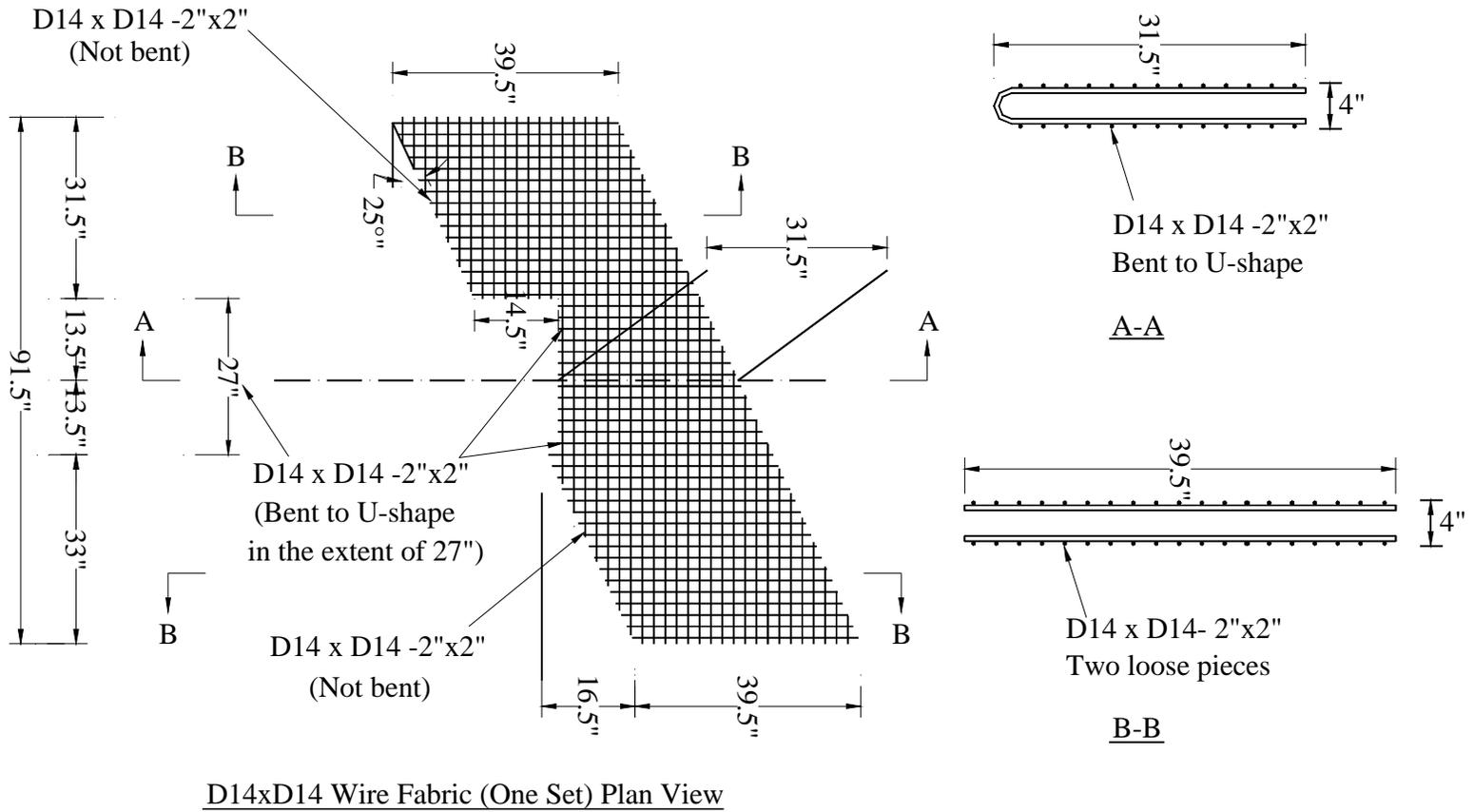
Figure 3.27-Post-tensioning Panel Specimen at UNO Structures Lab



Note:

1. Tubes for P/T strands will be used in the solid block.
2. The 0.6" strands inside the solid block and chucks are not shown for clarity.

Figure 3.28-Post-tensioning Panel Specimen Reinforcement Details



Note: This wire fabric is supposed to be put in the 6"-thick deck panel. Totally 2 sets are needed.

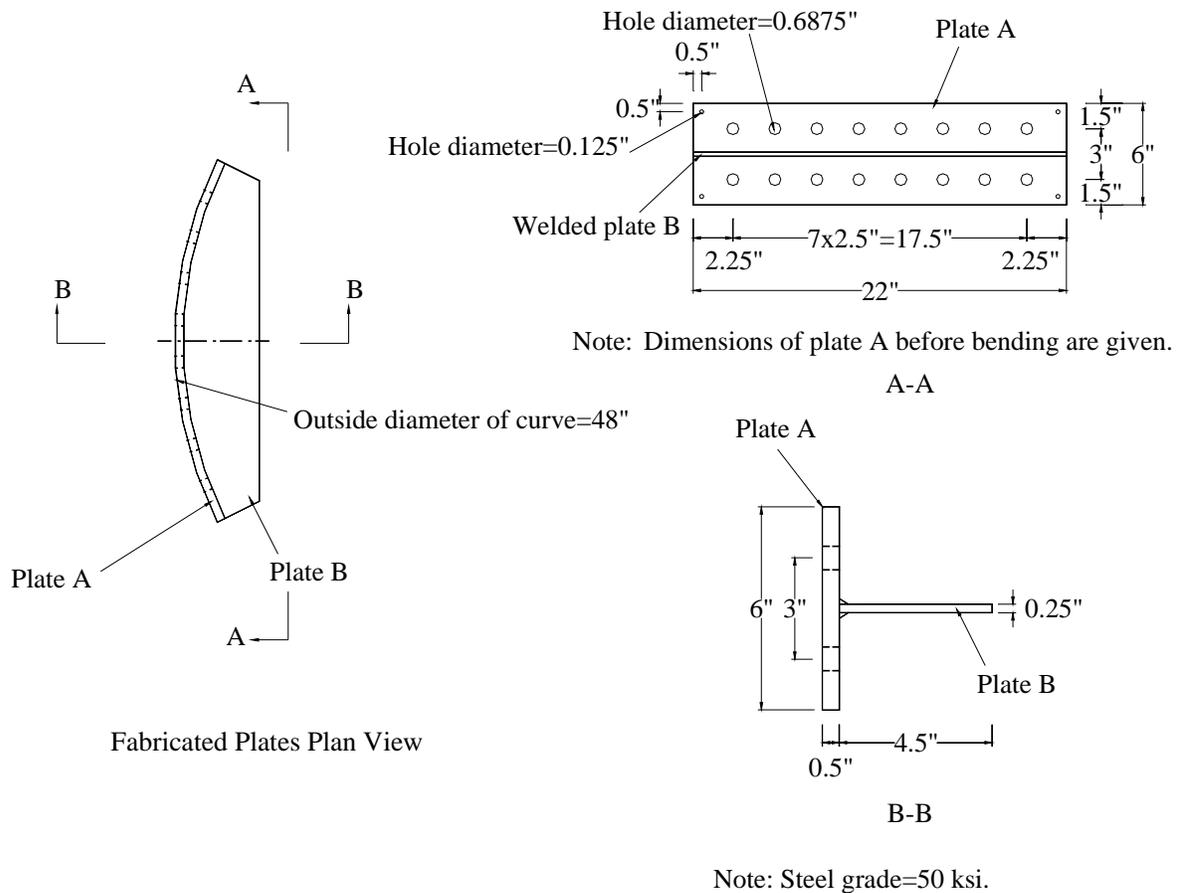


Figure 3.29-Post-tensioning Anchorage Steel Plates

Figure 30 shows the conventional reinforcement setup for the demonstration panel. Totally 16-0.6" strands were pulled through the open channel (see Figure 31). Some commercial devices such as the "Chinese Finger" can be utilized to facilitate this step (see Figure 32). Figure 33 shows the anchorage details including the curved steel plate and reinforcement mesh at the panel end. Note that #4 bar was used to replace the D14xD14 wire mesh in this panel for equivalency and convenience. The plan view of the panel before pouring concrete is illustrated in Figure 34. The strands projected about 4 ft beyond the panel end to provide adequate length for post-tensioning operation. After the concrete was poured and hardened, the panel was ready for post-tensioning (see Figures 35 and 36). Also shown are the reusable chucks used in this demonstration panel. A light-

weight mono-strand post-tensioning jack was utilized to tension the strands (see [Figure 37](#)). Each strand was tensioned to 30% of full tension to lock-in the dead end anchorage jaws. Two of the innermost strands were tensioned first. Afterwards, tensioning was applied from the center of the strand pattern outward in both horizontal directions to maintain the symmetry of P/T force. The strands were marked at the chuck edge after the initial tensioning to allow measuring strand elongation (see [Figure 38](#)). Eventually, the strands were tensioned to the required force by two means of measurement: (1) strand elongation, and (2) pressure gauge reading (see [Figures 39 and 40](#)). In addition, the panel shortened relative to the wood form as expected (see [Figure 41](#)).



[Figure 3.30-Conventional Reinforcement Setup in the Demonstration Panel](#)



Figure 3.31-Pulling the Post-tensioning Strands along the Channel



Figure 3.32-“Chinese Finger” for Pulling the Post-tensioning Strands



Figure 3.33-Anchorage Reinforcement Setup



Figure 3.34-Plan View of Panel before Pouring Concrete



Figure 3.35-Pouring Concrete for the Demonstration Panel



Figure 3.36- Demonstration Panel before Post-tensioning

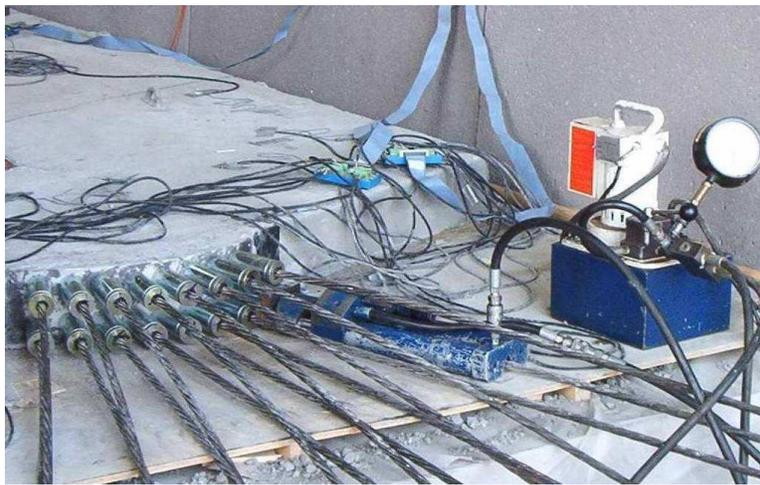


Figure 3.37-Initial Post-tensioning



Figure 3.38-Marking the Strands



Figure 3.39-Tensioning the Strand to the Required Level of Tension



Figure 3.40-Strand Elongation Measurement



Figure 3.41- Measurement of Panel Shortening



Figure 3.42-Panel End at the Tension End

Based on the experience of the demonstration panel, it showed that the post-tensioning procedure was practical and the post-tensioning device was easy to operate. However, some details may be modified to improve the system efficiency. Some of issues are listed as follows:

- 1) The holes in the curved steel plate need to be enlarged to allow for pulling the strands easily.
- 2) Since the anchorage details consist of steel mesh at a small spacing, the concrete needs to be flowable enough to achieve the desired pouring quality.
- 3) The anchorage details with rebar mesh (or welded wire reinforcement) worked as expected under the full post-tensioning force from 16 strands and no cracking was observed (see [Figure 42](#)). However, it would be more beneficial if the anchorage reinforcement was reduced or simplified.
- 4) Since the P/T strands were not straight at the panel end block, it was hard to arrange the plastic tubes exactly as designed without any intermediate plate or rebar.

- 5) The curved steel plate was welded with a straight plate at its mid-depth to achieve adequate embedment in the panel end. However, it helped bursting the panel due to post-tensioning, which was not desirable.

### **3.2.3 Analysis of the Anchorage Zone**

To optimize the anchorage block details, finite element analysis was performed to determine the stress distribution. [Figure 43](#) represents the FEA model of an end deck panel, which is assumed to be 12 ft wide, 8 ft long, and 6 in. thick. The 12 ft width reflects the maximum beam spacing allowed in Nebraska and most other states. A 12 in.-wide channel starts 3 ft away from the panel end, where the post-tensioning force is applied. The model included a solid element for the deck panel and each post-tensioning force was modeled as an equivalent area loading applied from the chuck to the end plate along the panel longitudinal (length) direction. [Figures 44, 45 and 46](#) illustrate the analysis results in terms of the panel stress along the longitudinal, transverse, and thickness directions, respectively. Note that the panel is subject to a high compressive stress at the end and the post-tensioning force is gradually spread out along the panel longitudinal direction. To avoid any concrete crushing at the end zone, the concrete strength of end panel may be increased or some reinforcement should be provided. At the open channel end area, concrete is in tension along the transverse direction, which necessitates some tensile reinforcement accordingly (see [Figure 45](#)).

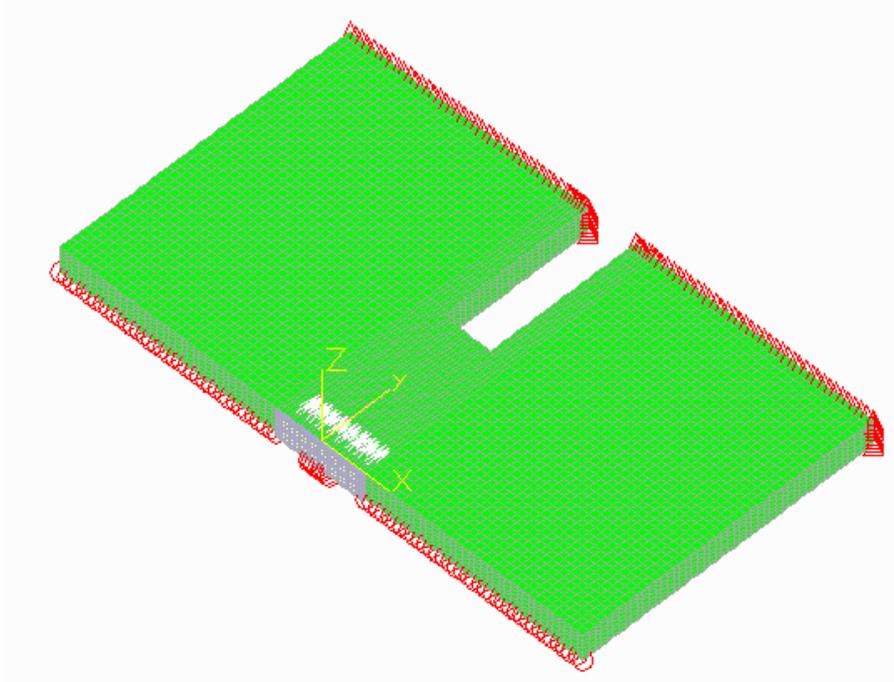


Figure 3.43-FEA Model for Bridge Deck Panel

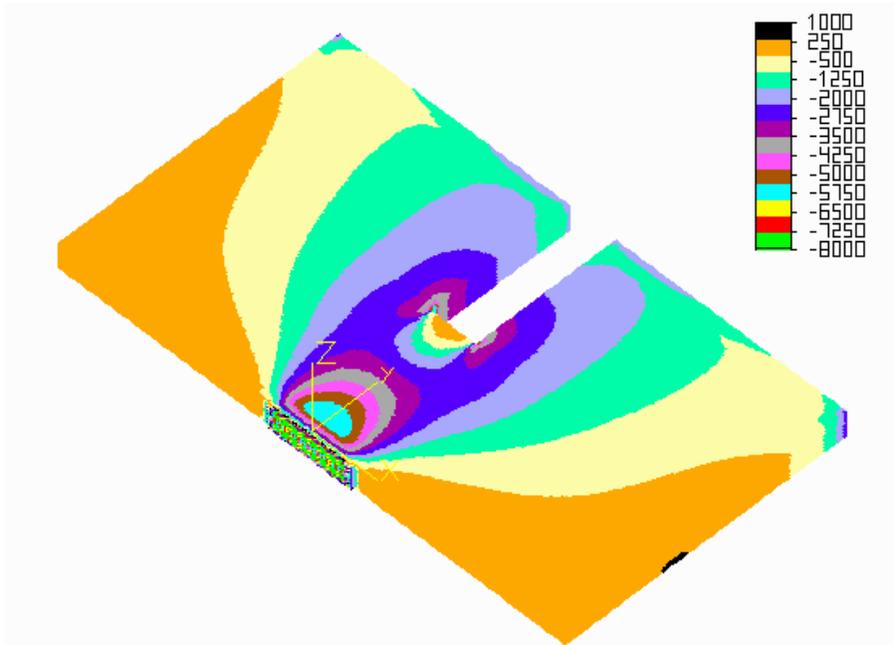


Figure 3.44-Panel Stress along the Deck Longitudinal Direction  
Note: tension stress is positive; unit is in psi.

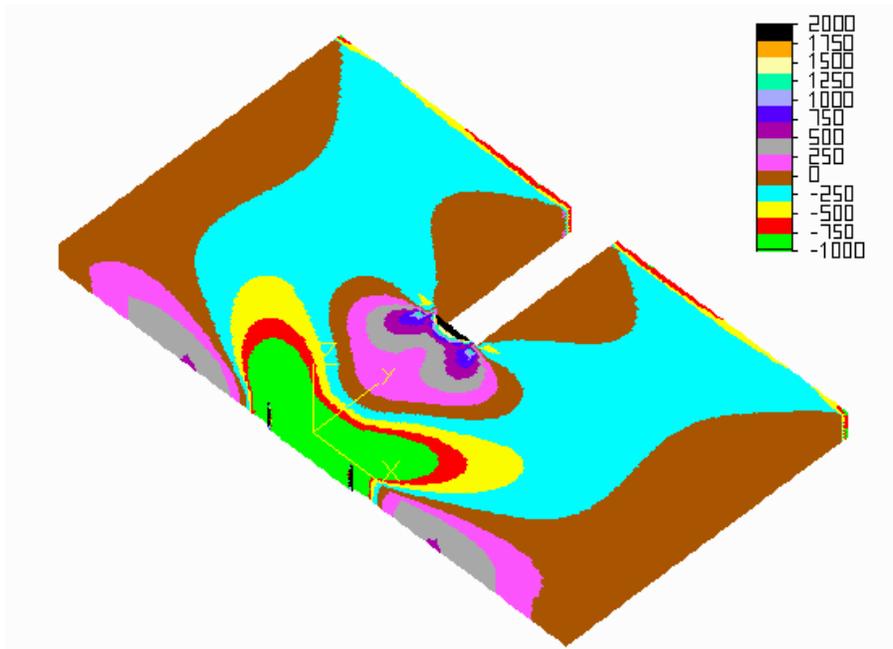


Figure 3.45-Panel Stress along the Deck Transverse Direction

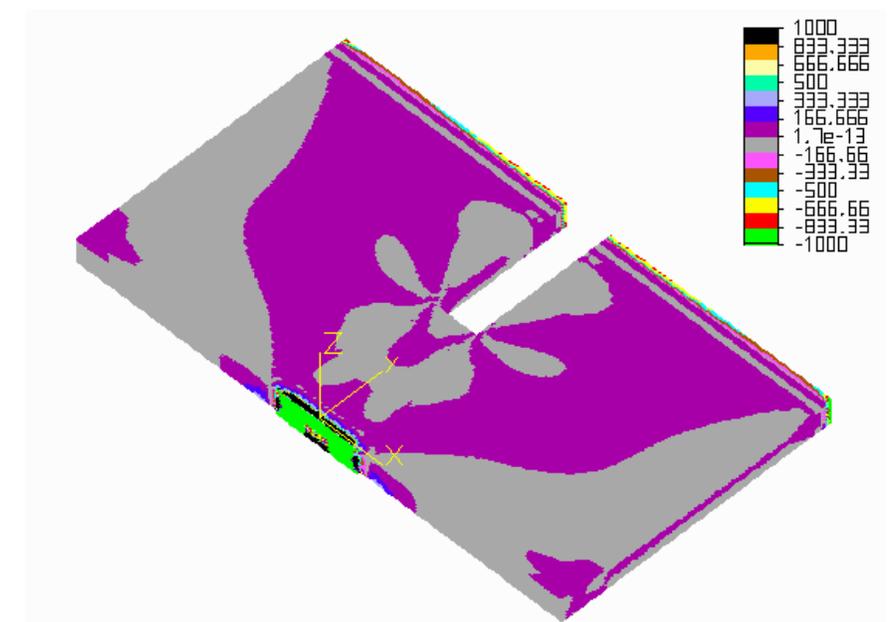


Figure 3.46-Panel Stress along the Deck Thickness Direction

### 3.2.4 Modified Anchorage Details

According to the finite element analysis and the experience from the demonstration panel, two options of modified details were proposed: one was the steel plate assembly

which consisted of 4 pieces of steel plates welded by bars; the other was the steel tube assembly which was composed of 16 steel tubes connected with 2 prefabricated steel plates.

#### **3.2.4.1 Steel Plate Assembly**

The steel plate assembly includes 4 pieces of prefabricated steel plates at a spacing of 1 ft (see [Figures 47, 48 and 49](#)). 4-#4 bars are welded across the plates to form a frame with 2 bars at top and 2 at the bottom. The plates are arranged at 1 ft spacing so that the strand profile can satisfy the minimum curvature requirement in the LRFD Specifications. Each steel plate has 16-1 in. diameter holes allowing the plastic shields to pass through (see [Figure 48](#)). Before pouring the end panel, temporary post-tensioning strands may be left in the plastic shields to avoid any possible blockage during panel construction.

As shown in [Figure 50](#), the curved steel plate has an outside diameter of 4 ft and a dimension of 22 in. x 6 in. x 0.75 in. before bending. With the 1 in. diameter holes, 16 washers (O.D. = 2 in., I.D. = 0.75 in.) may be put between the chucks and curved plate for adequate bearing, if necessary. Plate B, 0.25 in. x 6 in. x 12 in., is embedded at the post-tensioning channel end (see [Figures 49 and 50](#)). Plates C and D, 0.25 in.-thick intermediate plates, may contribute to maintain the required strand profile. According to the local steel manufactures, i.e., Drake-Williams Steel Inc., the cost of the 4 steel plates is approximately \$200.00. The manufacture indicated that the plate thickness barely influenced the cost. In addition, A36 steel costs about 2 cents less than Grade 50 steel per pound, which is insignificant in this case if Grade 50 steel is used.

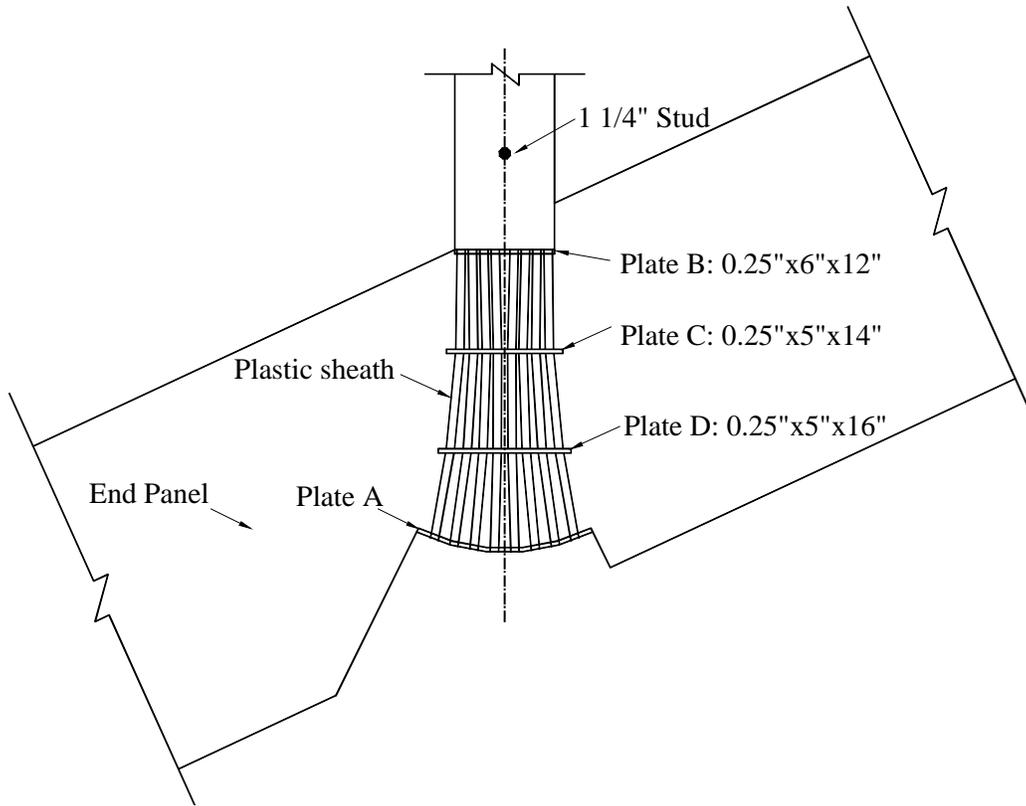


Figure 3.47-Plan View of End Panel with Steel Plate Assembly

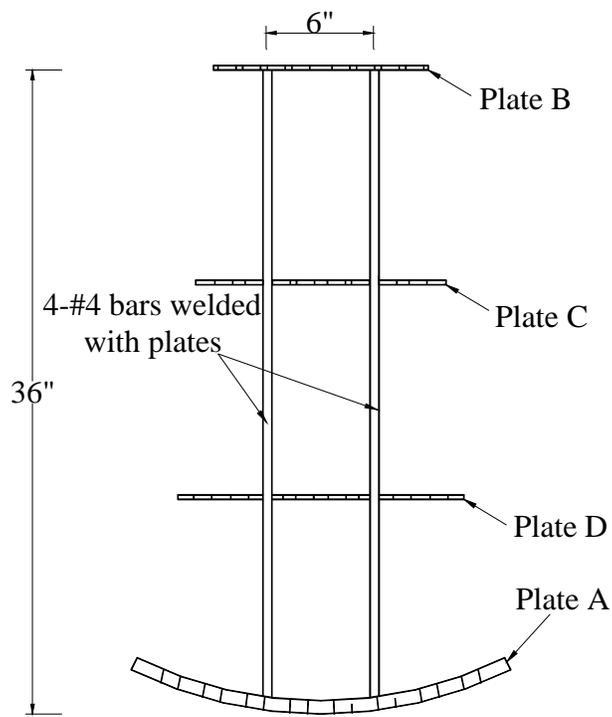


Figure 3.48-Plan View of Steel Plate Assembly

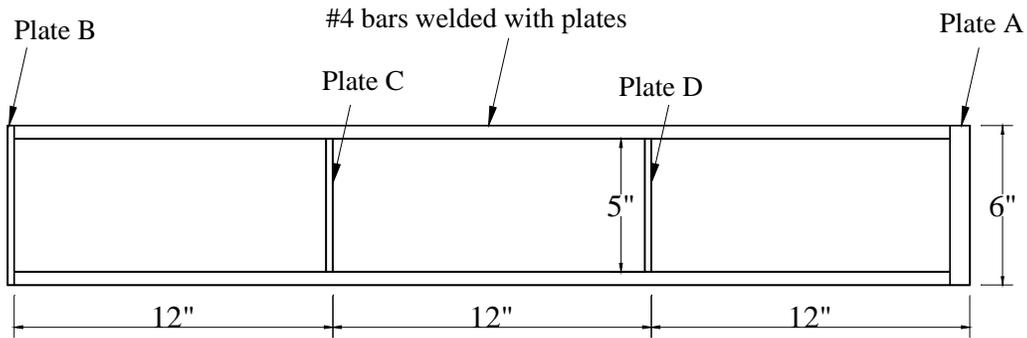


Figure 3.49-Elevation of Steel Plate Assembly

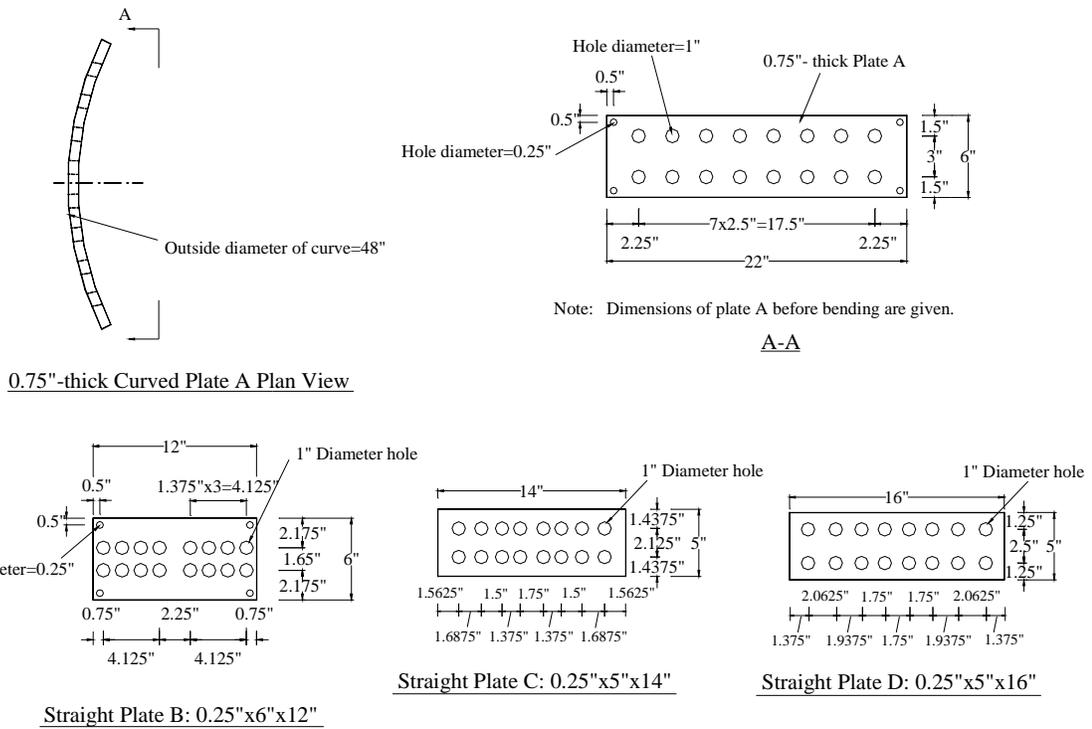


Figure 3.50-Prefabricated Steel Plates Details

### 3.2.4.2 Steel Tube Assembly

The alternative anchorage detail, steel tube assembly, consists of two steel plates welded with 16 steel tubes, which may be prefabricated as a whole piece and embedded in the end panel (see Figure 51). The tubes, 1 in. outside diameter with 0.1 in. thickness, are bent to provide the strand profile. According to the local steel manufacture, i.e.,

Drake-Williams Steel Inc., the total cost of this anchorage detail is approximate \$600. Since it is time-consuming to bend the steel tubes and weld them with the steel plates as requested, the cost is higher than that of the steel plate assembly.

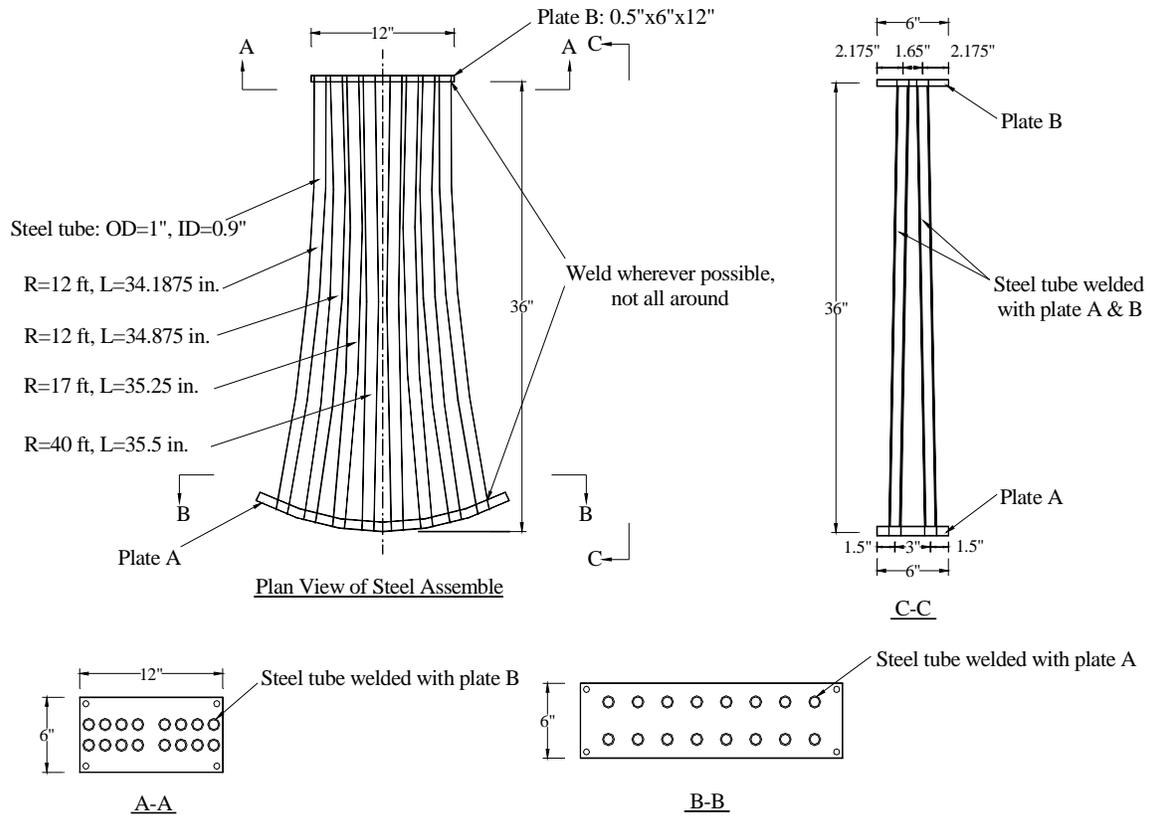


Figure 3.51-Steel Tube Assembly Details

### 3.2.4.3 Steel Plate Assembly Specimen

Since the cost of steel plate assembly was more acceptable than the steel tube assembly, it was selected for demonstration at the UNO Structures Lab. The post-tensioning panel introduced in 10.2.2 was sawed 3 ft away from the dead end after the post-tensioning force was released (see Figure 52). The steel plate assembly was then placed including four steel plates welded with 4-#4 bars (see Figure 53). Note that two layers of 16 total

plastic tubes pass through the steel plates. Also shown were 4-#4 bars placed near the post-tensioning channel end and the panel end zone. The reinforcement at the channel end was used to resist the tensile stress and prevent the concrete from cracking. The rebar placed around the panel end zone should contribute to spread out the high compressive stress longitudinally and avoid any concrete crushing. After the panel end block was poured and concrete hardened, 16-0.6 in. diameter strands were fully tensioned. As a result, the modified anchorage detail using the steel plate assembly provided adequate strength to resist the post-tensioning force. No crushing or obvious cracking was found during the post-tensioning process. From what has been observed, several issues are listed below.

1) Since there was not enough concrete cover provided for the top 2-#4 bars which were welded to the steel plates, two fine cracks were found along the rebar. It happened about 8 hours after the end block was poured. As a modification, the #4 bars may be welded to the sides of intermediate plates instead of top and bottom. Also, the height of intermediate plates can be reduced to 4 in. rather than 5 in. so that enough concrete cover may be provided.

2) Two 1/4"-thick intermediate steel plates helped to maintain the demanded strand profile. Also, they, together with the rebar, contributed to take the splitting force due to post-tensioning.

3) It was difficult to pull the strands through the plastic tubes by hand since the distance between adjacent plates is only 1 ft. Either a larger plate spacing should be adopted or less steel plates be included, say, three plates instead of four, unless a strand-pulling device is utilized.

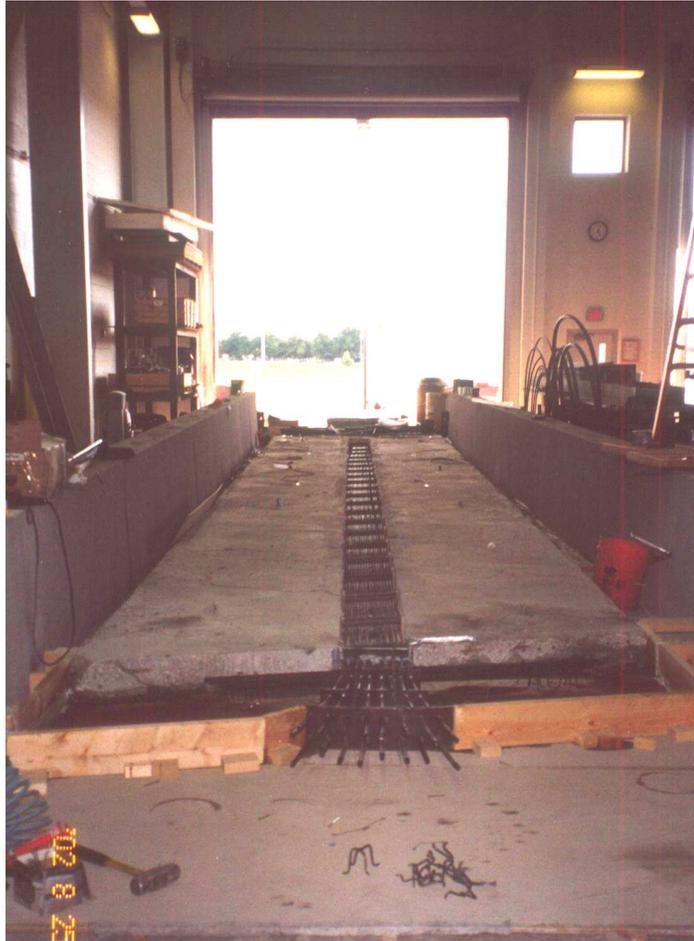


Figure 3.52-Steel Plate Assembly Placed at the Panel End



Figure 3.53-Plan View of Steel Plate Assembly

## CHAPTER 2

### NUDECK SYSTEM DEVELOPMENT

#### 2.1 BACKGROUND

The large majority of short to medium span bridge systems in the United States are the composite beam/slab systems. The beam is generally precast prestressed concrete or structural steel I-beam. Cast-in-place (CIP) reinforced concrete deck system, as the most commonly used system, has the primary advantages such as field adjustment to produce a smooth roadway profile. But CIP bridge deck slabs experience cracking shortly after construction due to its tendency to shorten relative to the beams, which is resulted from temperature gradients and differential creep and shrinkage. They generally require major repair or replacement work in only 15 to 25 years, while the beams generally last much longer. In addition, nearly one-half of about 600,000 highway bridges in the United States are in need of replacement or rehabilitation. Since a large number of these bridges carry heavy traffic, it is necessary to remove and replace the deck in a short period.

Numerous attempts have been made to correct this weakness, with varying degrees of success. One of the earliest FHWA “High Performance Concrete (HPC)” Showcase Program bridges was the 120<sup>th</sup> and Giles Road Bridge in Sarpy County, Nebraska, which was opened to traffic in 1997. The deck was made of 8,000 psi HPC concrete. Strict curing specifications were placed, which included misting, spraying of a curing compound and placement of wet burlap for seven days. Upon inspection of the deck several weeks after completion of the bridge, significant cracking was observed at the bottom surface of the deck. There have been similar observations on other bridges, even with use of Type K expansive cement and shrinkage compensating admixtures.

Additional drawbacks of CIP concrete decks are low speed of construction and the need for strict field quality control.

There are currently two types of precast prestressed deck panel systems. The more common one utilizes 3 to 5 in. stay-in-place pretensioned panels that span between beam edges and are totally separated over beam top flanges. They house the positive transverse moment reinforcement, which is generally pretensioning strand. Negative moment reinforcement is provided in a CIP composite topping. This system sometimes experiences reflective cracking over panel edges. In addition, experiments in the NCHRP 12-41 project have confirmed that lack of anchor of the transverse strand reinforcement in individual panels into the beam supports reduces arching action and the system's load capacity compared to full-bridge-width CIP or precast panel systems. Another major drawback of this system is the need for conventional forming and construction of overhangs.

The second precast concrete deck system, often used in replacement projects, is full-depth, full-width precast panels that are about 8 to 10 ft long. These panels are conventionally reinforced in the transverse direction and post-tensioned along traffic. They have been placed primarily on steel plate beam bridges with the horizontal shear stud connectors clustered in pockets at about 2 ft spacing. The pockets are grouted after the panels are post-tensioned. This system does not have the flexibility to be used with precast prestressed beams because the horizontal shear reinforcement cannot be conveniently clustered in pockets. In addition, pockets and post-tensioning ducts require grouting. The system generally has no transverse prestressing, and is thus subject to cracking under service conditions.

It appears, therefore, that a need exists for a cost-effective concrete deck system that has the following characteristics: (a) Precast concrete installed after most of the creep, shrinkage and heat of hydration has taken place; (b) Prestressed concrete in which the level of prestressing is high enough to result in zero residual tension at service conditions, both in the longitudinal and transverse directions; (c) A panel system that allows for simple construction and creation of composite action with the beams, especially concrete beams; and (d) A panel system that allows for the prestressing to be introduced before composite action takes place, so that the much stiffer beams do not attract most of the prestressing.

## **2.2 RESEARCH SIGNIFICANCE**

Deck is connected with a basic component of most bridges built in the U.S., regardless of whether the supporting elements are steel or concrete beams, arches or trusses. On the other hand, decks are the major source of bridge deterioration and deficiency. Deck replacement must be done quickly to avoid loss of revenue to the traveling public. In particular, the proposed bridge deck system can result in a significant reduction in the construction time of deck slabs. The deck obtained is compressed in two directions, resulting in a significant reduction in operation and maintenance costs and in greater durability of the overall structure. The space between beam flanges is fully covered with deck panels, thus protecting workers against accidental falling. Rapid construction and reduced maintenance diminish the probability of workers' injury.

This report presents an innovative full-depth precast/prestressed concrete bridge deck system that enables fast efficient construction, yields superior performance in service, and reduces long-term maintenance and replacement costs.

## 2.3 DEMONSTRATION PANELS

During the early-stage of full-depth NUDECK panel system development, four precast panels were produced for demonstration purpose in 2001. Two basic types of precast panel systems were presented: one is the precast prestressed deck panel (see [Figures 1 and 2](#)) and the other is the precast conventionally reinforced deck panel (see [Figures 3 and 4](#)).

[Figure 1](#) shows two 4 ft long by 46'-6" wide by 6 in. thick panels on display at the Concrete Industries INC., in Lincoln, Nebraska. [Figure 2](#) illustrates the details of the precast prestressed panel, which is reinforced with 0.5 in. diameter, Grade 270 ksi low relaxation strands along the panel transverse direction. Also shown are the adjustable bolts to achieve the required panel elevation. The plan view of a conventionally reinforced panel is illustrated in [Figure 3](#). The transverse joint between panels may be spliced by spirals (see Badie, 1997). [Figure 4](#) shows the details of the precast panel over a concrete block. Note that a steel plate was embedded at the block top surface to simulate the steel beam top flange.

The panels shown in [Figures 1 and 3](#) were designed for a beam spacing of 12 ft, which was the maximum spacing allowed in Nebraska and many other states, and for a skew angle of 30°, which is near the limits for skewed panels in this system. For larger skews, right angle panels may be more convenient to use. The panels were produced by Rinker Materials in Bellevue, Nebraska, and shipped to Concrete Industries INC., Lincoln, Nebraska. This provided a full-scale demonstration of the handling, shipping, and installation ability of these panels, as most casual observers would tend to believe

that they are too flexible to handle and ship. These demonstration panels showed that the precast deck system was easy to work with.

According to a discussion among UNL researchers, precast producers, and NDOR bridge designers, the precast panel system was further simplified to improve its ease of production and structural efficiency.

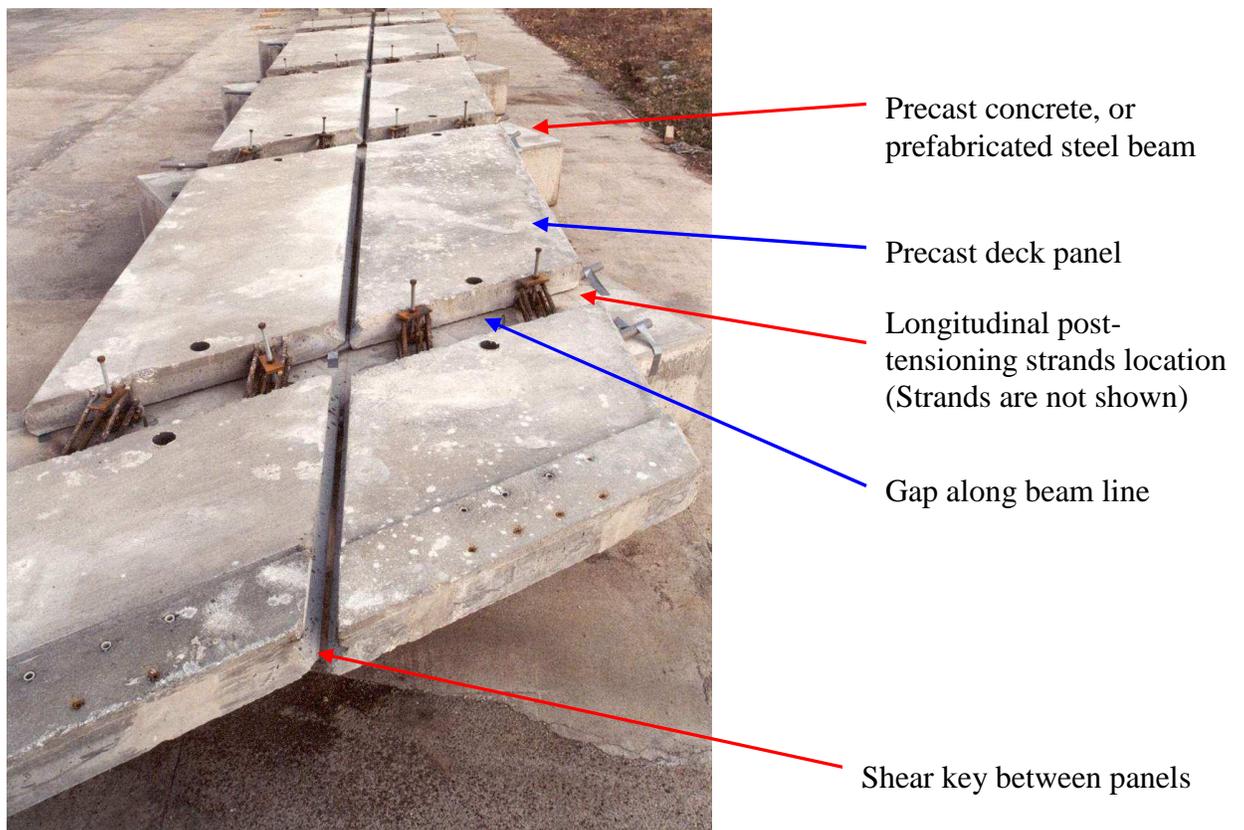


Figure 2.1-Plan View of Precast Prestressed Panel for Demonstration

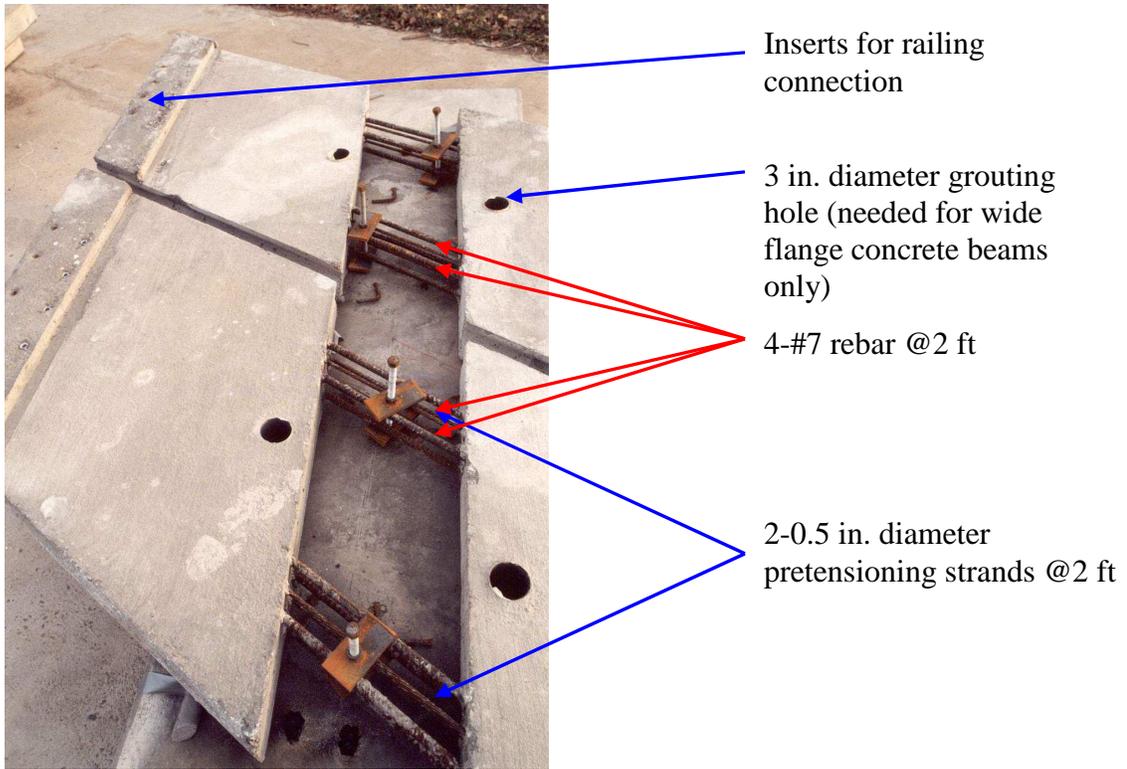


Figure 2.2-Details of Precast Prestressed Panel for Demonstration



Figure 2.3-Plan View of a Precast Conventionally Reinforced Panel

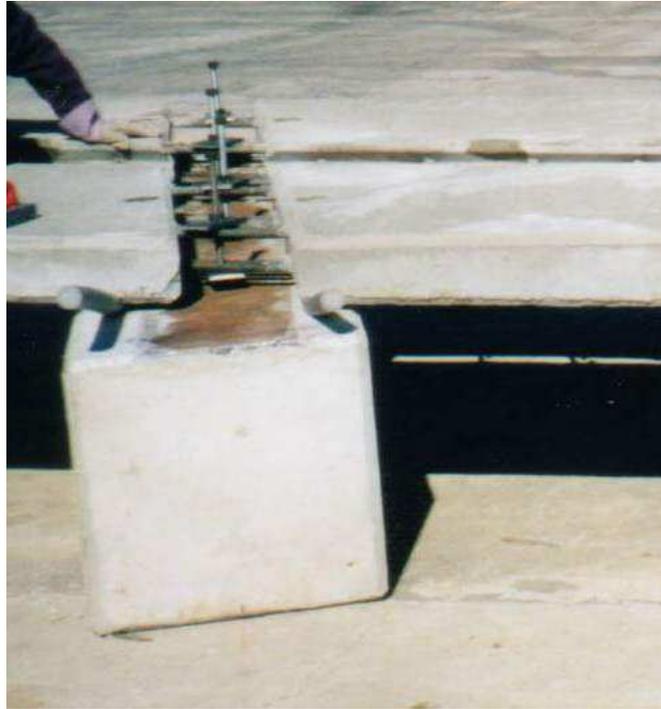


Figure 2.4-Details of Precast Conventionally Reinforced Panel over Concrete Girder

## 2.4 Description of NUDECK Panel System

Figure 5 shows the modified NUDECK panel system which was implemented in the Skyline Bridge in Omaha, NE. The panels are 8 ft long (along traffic) and as wide as the bridge, i.e. 51.5 ft. The plan view and cross section of a typical NUDECK panel are shown in Figures 6, 7 and 8. Note that each panel is reinforced with #5 bars at 12 in. spacing along the beam line direction. The transverse reinforcement is pairs of 0.5 in. diameter, Grade 270 ksi low relaxation strands at 24 in. spacing. The two strands in the pair are spaced vertically to allow 1 in. concrete clear cover. The effective prestressing stress (after accounting for time dependent losses) amounts to about 350 psi. Considering arching effects and strand continuity, theory has shown this quantity to be adequate for beam spacing up to 12 ft. To assure a short transfer length of prestressing, an innovative detail was used. High strength wire spiral was placed around each pair at the end 2 ft.

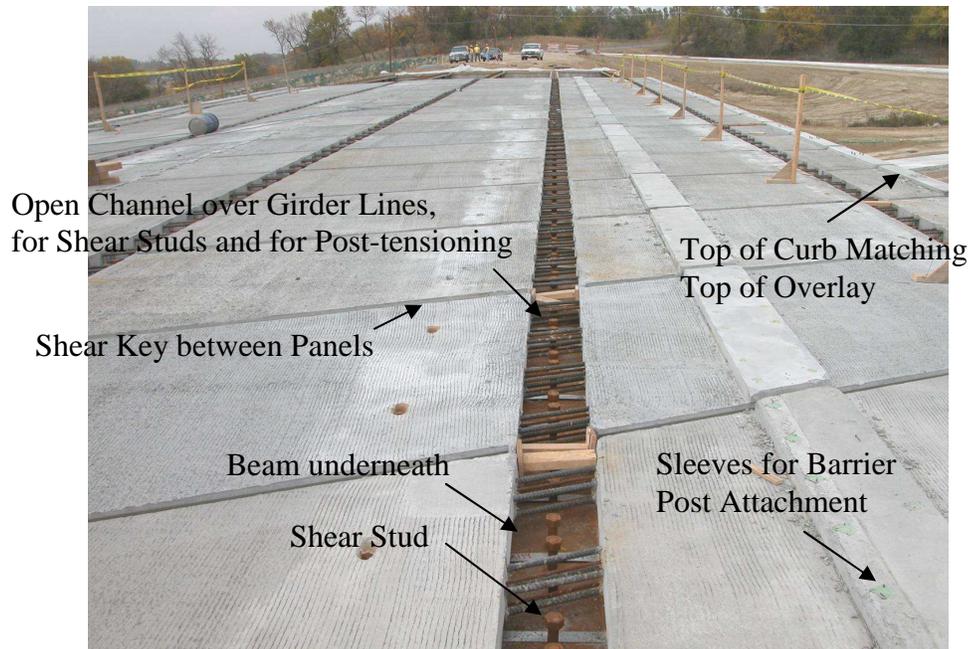


Figure 2.5-Plan View of Typical Precast Prestressed NUDECK Panels

One of the primary innovations of this system is the fully open gap in the panel over each of the beam lines. To preserve the tension in the continuing strands and, thus the pre-compression in the concrete, the absent concrete strip in the gap is substituted with 4#7 bars at the location of each pair of strands. These bars can be viewed as “prestressed rebar”.

Within the space in any given gap, the tension in each pair of strands is equal to the compression in each set of four bars. In the solid concrete between beam line gaps, the tension in the strands is equal to the compression in the concrete. The bar size is determined by its ability to resist buckling during prestress release and bending during handling and erection. The value of a totally open gap is to avoid conflicts with the composite action studs, and more importantly to greatly simplify post-tensioning in the longitudinal direction.

Large diameter studs are placed at the centerline of the beam in one row at 6 in. spacing. NCHRP 12-41 project and additional funded research by NDOR included

extensive ultimate and fatigue studies on 1 ¼ in. diameter studs. It has been demonstrated that each 1 ¼ in. stud is equivalent to about two 7/8 in. diameter studs. A bridge has already been constructed with this new size of stud and has performed equally to conventional design. Using these large studs greatly reduces fabrication costs of steel plate girders, improves safety in the field, and speeds up total construction.

The hanging supporting system, or plastic shim stacks, can be placed at the corners of the panels at each beam line (see Figure 9). The height of support or shim stack is expected to be provided by the designer. The designer should calculate the support or shim heights after accounting for deflection due to panel weights and additional loads. After the panels are set in place on the support or shims, further minor profile adjustments can be accommodated in the topping thickness.

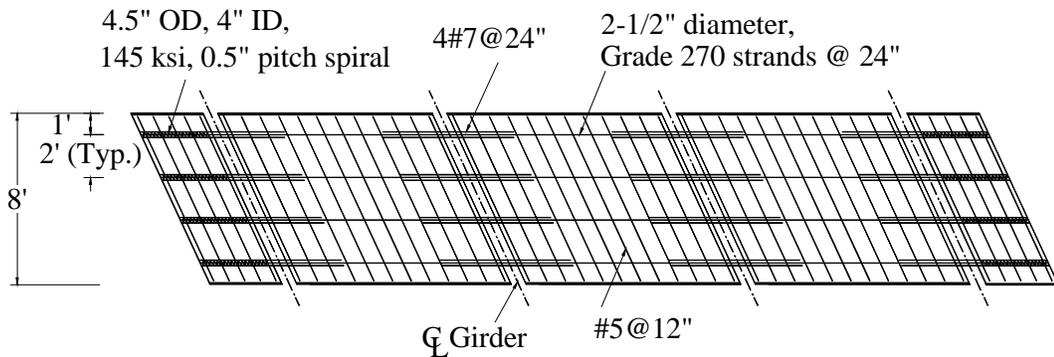


Figure 2.6-Plan View of a Typical Prestressed NUDECK Panel

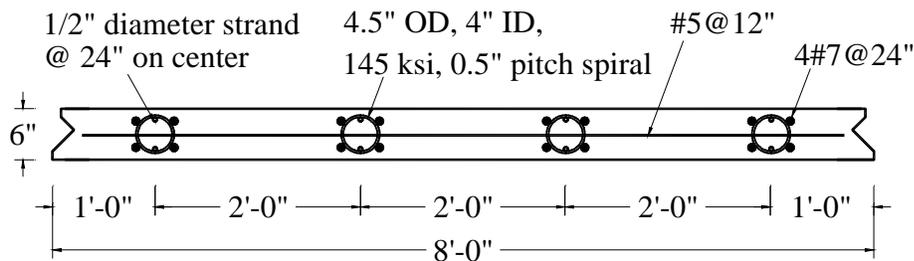


Figure 2.7-Cross Section of a Typical Prestressed NUDECK Panel

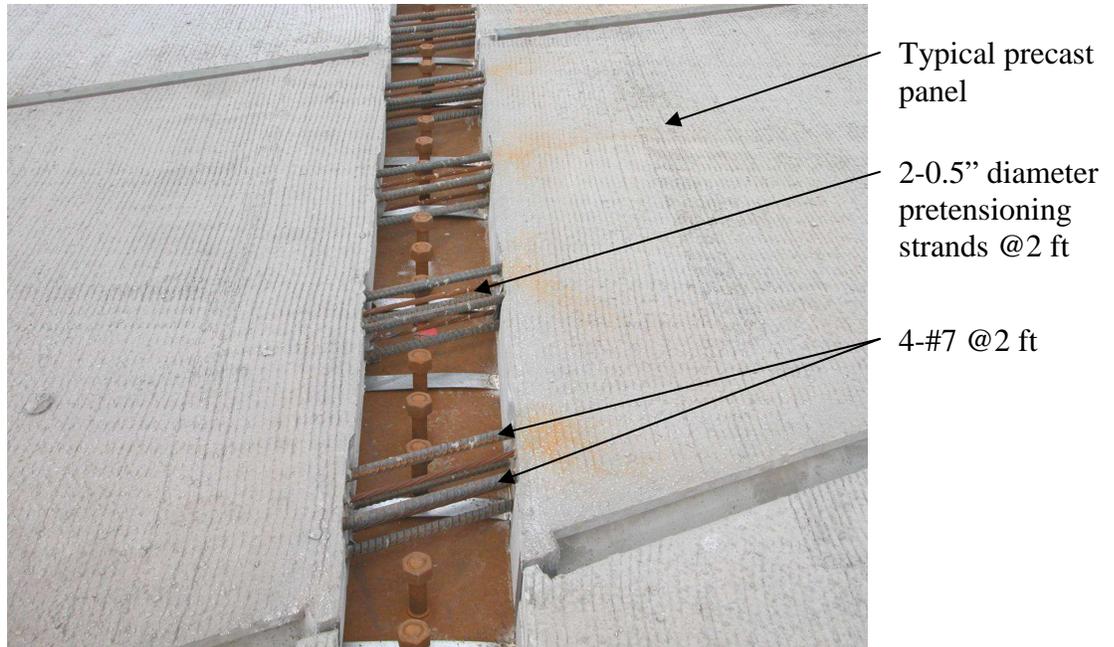


Figure 2.8-Precast Prestressed NUDECK Panel Details

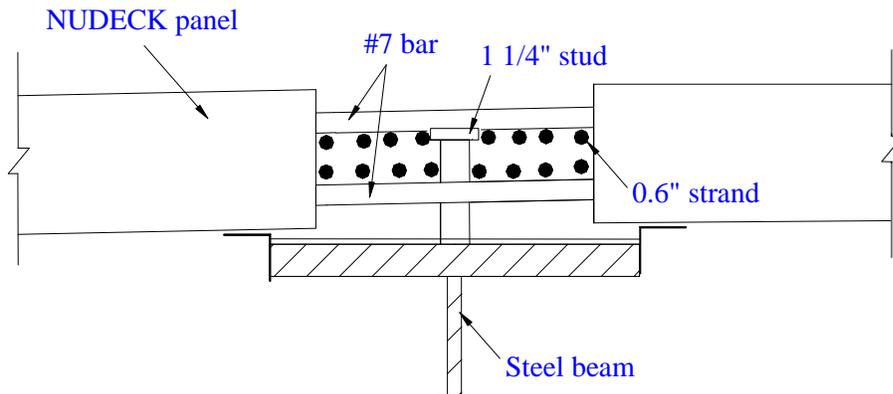


Figure 2.9-Arrangement of Post-tensioning Strands

Grouting of transverse shear key joints between panels is done using a high performance concrete flowable grout. There is no need to use shrinkage-compensating admixtures in this grout as the joints will have in-service residual compression due to longitudinal post-tensioning. The grout strength should match that of the panels, a minimum of 5,000 psi.

Post-tensioning is done using a unique process. 16-0.6 in. diameter strands are threaded between the pairs of strands and groups of four bars. The gap over the beams can be viewed as an open channel post-tensioning “duct, or sheathing.” The criticism in recent years of a lack of quality grouting of post-tensioning ducts is totally eliminated with this system. The tensioning of the strands is done individually using a light weight mono-strand jack. Post-tensioning is done after the transverse joints attain adequate strength (about 1,500 psi) and before composite action is effected. This puts the post-tensioning force fully in the deck rather than inefficiently sharing it with the much stiffer beams. This system is therefore highly resistant to transverse cracking, which has been a major problem in recent years. The longitudinal channel over the beams and the 2-in. composite overlay are filled with Type K, non-shrinkage, cement. Grout in the channel, similar to grout in P/T ducts, is not precompressed.

A number of innovations are introduced with the NUDECK system. They include the following: The panels are made of high performance precast prestressed concrete. The concrete is precompressed in two directions such that the residual stress in service is compression and cracking is avoided. Most of the creep, shrinkage, and temperature change due to hydration take place before the panel is connected with the rigid underlying beams, eliminating a major source of cracking. The continuous gap over beam lines assures simple, high quality post-tensioning and eliminates the question about the quality of tendon grouting. Individual post-tensioning of strands allows most contractors, even in areas not familiar with post-tensioning, to do the post-tensioning work with local crews. The precasters also would have the capability and the option to include post-tensioning as part of supplying the panels. The proposed prestressed deck panel system

covers the entire width of a bridge, which eliminates the necessity of forming for the overhangs. All materials used in the production of the deck panels and the other construction steps are non-proprietary and readily available. This makes the system cost competitive with cast-in-place concrete decks, while it is much more rapid to build and durable to maintain. Use of large diameter studs reduces the required number of studs. Thus, economy of fabricated steel beams is improved and, more importantly, worker safety is enhanced. Finally, the cast-in-place overlay allows for adjustments in roadway profile. It provides an excellent riding surface, and large cover for the reinforcement.

## **2.5 POSSIBLE CONSTRUCTION STEPS**

The possible NUDECK panel construction procedures are listed as follows:

1. Determine the elevations of the support system and install them.
2. Set the panels sequentially from one end, starting with the special anchorage end panel, then the typical panels, then the special end panel. Use two CAT 950F Front End Loaders or crane to install each panel.
3. Grout the shear keys between the panels. Make sure the ends are fully sealed to prevent accidental leakage into open channels.
4. Install post-tensioning strands.
5. Fill open channels and place overlay concrete.

## CHAPTER 6

### NUMERICAL EXAMPLE OF NUDECK PANEL

#### 6.1 STANDARD DETAILS OF NUDECK PRECAST PANEL SYSTEM

The standard details of the NUDECK precast deck panel are repeated in this chapter for design engineers to facilitate their deck design. Presented herein are the reinforcement setup of a typical panel, the shear key details, and the longitudinal details over the girder lines (see Figures 1 to 4).

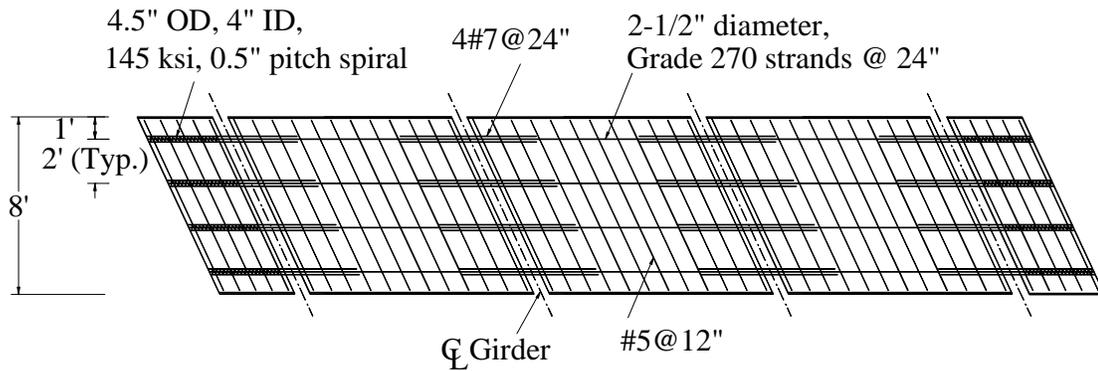


Figure 6.1-Plan View of a Prestressed NUDECK Panel

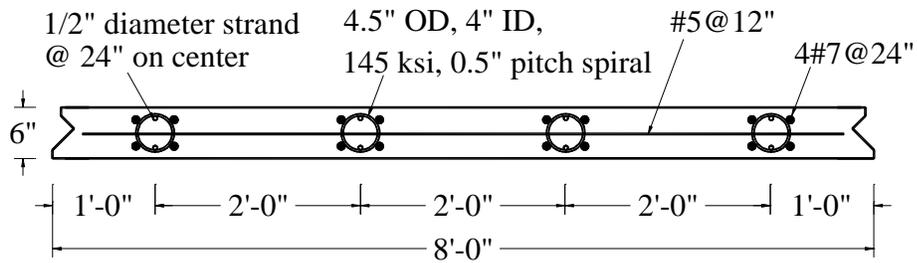


Figure 6.2-Cross Section of a Typical Prestressed NUDECK Panel

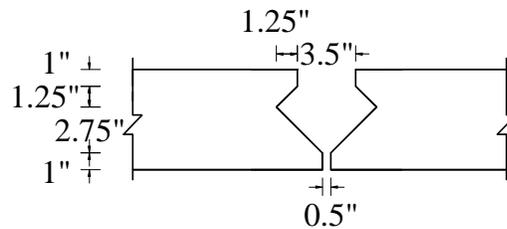


Figure 6.3-Transverse Joint Configuration

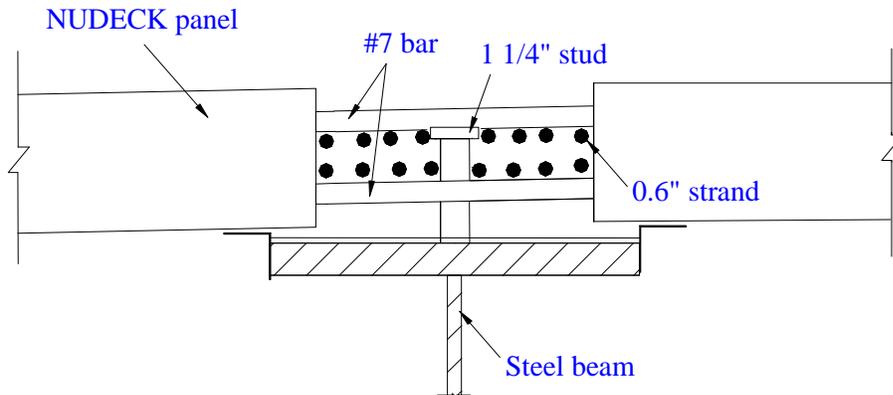


Figure 6.4-Arrangement of Post-tensioning Strands over Girder

## 6.2 DESIGN ASSUMPTIONS

The NUDECK precast deck panel is design based on the LRFD Bridge Design Specifications using the approximate design method. The following assumptions are made during the development of this system.

- Concrete:
  - Concrete strength at release,  $f'_{ci} = 4,000$  psi, concrete strength at 28 days,  $f'_c = 6,000$  psi
- Steel:
  - Low relaxation prestressing strand, 0.5 in. or 0.6 in. diameter, Grade 270 ksi, and Grade 60 ksi rebar
- Maximum girder spacing is 12 ft.
- Maximum deck overhang is 4.5 ft.
- The dimension of a typical precast panel is full-bridge width, 8 ft-long, and 6 in.-thick. The precast panel is made composite with a 2 in. CIP concrete overlay.
- Skew angle is up to 25 degree.

- The weight of each barrier is assumed as 0.4 kips per unit length. A 2 in.-thick future wearing surface is included in the analysis.
- The panel can be erected by crane or the CAT 950F wheel loaders with a weight up to 35,000 lbs. During lifting of deck panels, approximately 70% to 80% of the machine weight goes to the front machine axle.

In addition, the amount of post-tensioning strands may be determined based on bridge spans to satisfy requirements at the service and ultimate limit state. The post-tensioning anchorage details using either the steel plate assembly or steel tube assembly may be adopted.

### **6.3 SKYLINE BRIDGE DECK DESIGN BY THE APPROXIMATE METHOD**

The Skyline Bridge precast deck analysis is presented using the approximate design method based on the LRFD Bridge Design Specifications. It considers a unit width of strip supported by the girders underneath. This strip is designed as a continuous beam and the maximum positive and negative moment apply to all the sections. Considering the Skyline Bridge, [Figures 5 and 6](#) present the moment diagrams due to one-truck and two-truck loading. Note that both the impact and the presence factors are included. The values shown are those prior to division by an equivalent strip width (E. S. W.). [Figures 7 and 8](#) show the moment due to deck weight plus barrier and future wearing surface, respectively, which are also determined based on one-foot unit width strip. [Tables 1 and 2](#) give the positive and negative moment due to live load plus impact on a unit width basis. The sections A through D represent the positive moment sections and sections 1 through 5 refer to sections at the supports. Listed in [Tables 3 and 4](#) are the total positive and negative moments due to various loads at the related critical sections. The maximum

positive and negative moment are 13.11 ft-kips and 15.84 ft-kips, respectively. The flexural capacity of the 7.5” thick bridge deck is 30 ft-kips considering a unit length section reinforced with 1-0.5” diameter pretensioned strand plus 1-#6 rebar. In terms of analysis at Service III, the stress at the deck section bottom due to all loading can be determined as follows:

$$f_b = \frac{P}{A} - \frac{M_{deck}}{S_b} - \frac{M_{barrier} + M_{ws} + 0.8(LL + IM)}{S_{bc}}$$

Where  $P = 0.9(202.5)0.153 = 27.9$  kips

$$A = 12(6) = 72 \text{ in.}^2$$

$$S_b = \frac{12(6^2)}{6} = 72 \text{ in.}^3$$

$$S_{bc} = \frac{12(7.5^2)}{6} = 112.5 \text{ in.}^3$$

At the critical positive moment section, section A,  $M_{deck} = 0.55$  ft-kips,  $M_{barrier} = -0.51$  ft-kips,  $M_{ws} = 0.1$  ft-kips, and  $M_{LL+IM} = 7.37$  ft-kips. Thus,

$$f_b = \frac{27.9}{72} - \frac{0.55(12)}{72} - \frac{(-0.51)(12) + 0.1(12) + 0.8(7.37)12}{112.5} = 0.388 - 0.092 - 0.585$$

= -0.289 ksi (tensile). This satisfies with the tensile stress limit of  $0.19\sqrt{f'_c} = 0.19\sqrt{6} = 0.465$  ksi.

At the critical negative moment section, section 1,  $M_{deck} = 0.9$  ft-kips,  $M_{barrier} = 1.1$  ft-kips,  $M_{ws} = 0.19$  ft-kips, and  $M_{LL+IM} = 7.46$  ft-kips. Thus, at the top fiber of the precast deck panel section,

$$f_t = \frac{P}{A} - \frac{M_{deck}}{S_t} - \frac{(M_{barrier} + M_{ws} + 0.8(LL + IM))y_t}{I_{bc}}$$

where,  $S_t = \frac{12(6^2)}{6} = 72 \text{ in.}^3$

$I_{bc} = \frac{12(7.5^3)}{12} = 421.9 \text{ in.}^4$

$y_t = 6 - \frac{6+1.5}{2} = 2.25 \text{ in.}$

$f_t = \frac{27.9}{12} - \frac{0.9(12)}{72} - \frac{(1.1+0.19+0.8(7.16))(12)2.25}{421.9} = 0.388 - 0.15 - 0.449$   
 $= -0.211 \text{ ksi (tensile). It satisfies with the tensile stress limit of}$

$0.19\sqrt{f_c} = 0.19\sqrt{6} = 0.465 \text{ ksi.}$

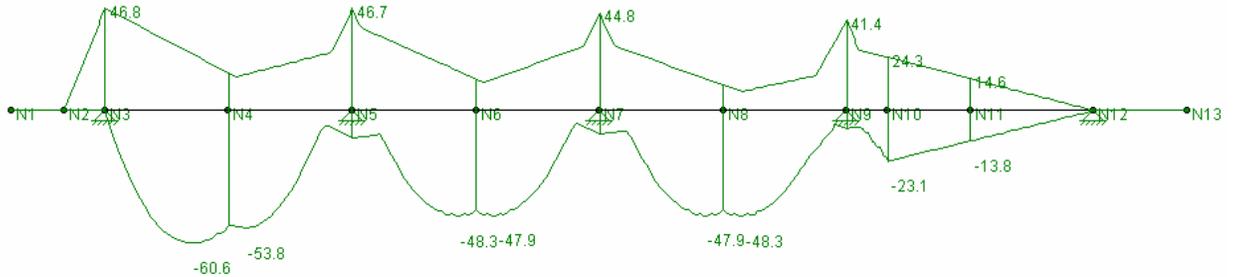


Figure 6.5-Moment due to One-truck Loading (with IM and Presence Factor)

Note: 1) Moment is in ft-kips; 2) The moment values shown are that prior to division by an equivalent strip width (E. S. W.).

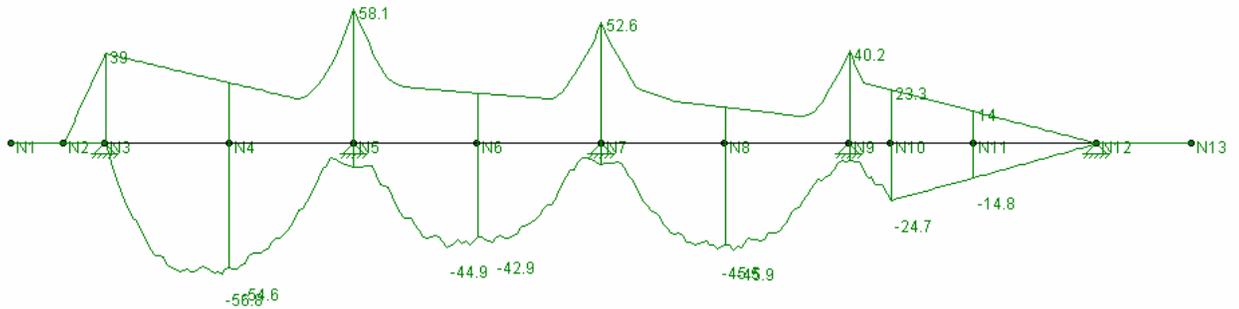


Figure 6.6-Moment due to Two-truck Loading (with IM and Presence Factor)

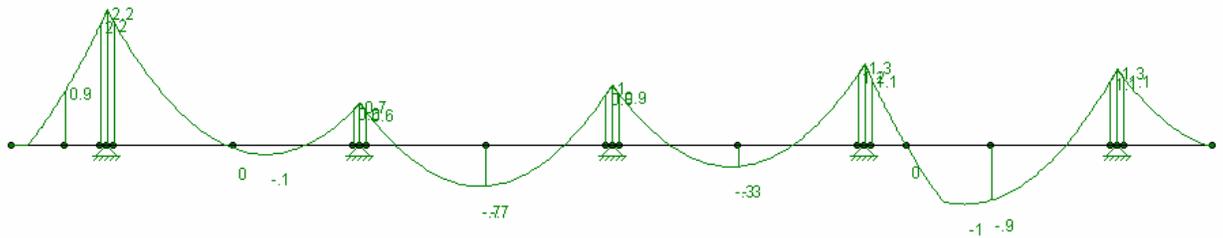


Figure 6.7-Moment due to Deck Weight plus Barrier Weight

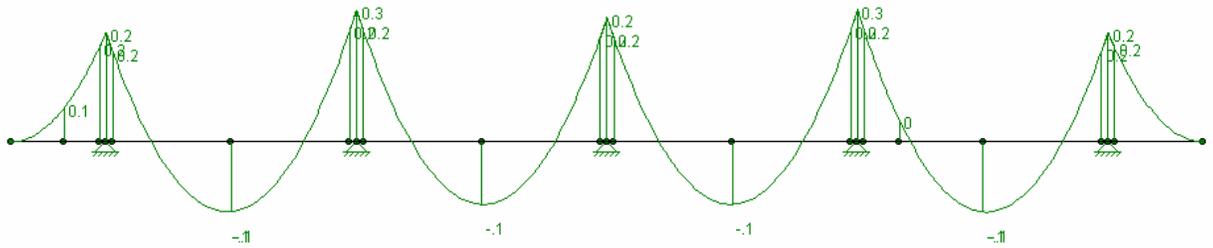


Figure 6.8-Moment due to Future Wearing Surface Weight

Table 6.1-Positive Moment due to LL+IM

Location	Positive Moment due to LL+IM (E. S. W. = 8.217 ft)	
	LL+IM	(LL+IM)/E. S. W.
	A	60.6
B	48.3	5.88
C	48.3	5.88
D	14.8	1.80

Table 6.2-Negative Moment due to LL+IM

Location	Negative Moment due to LL+IM			
	Overhang (E. S. W. = 5.278 ft)		Interior (E. S. W. = 6.750 ft)	
	LL+IM	(LL+IM)/E. S. W.	LL+IM	(LL+IM)/E. S. W.
1	39.37	7.46		
2			50.83	7.53
3			45.52	6.74
4			36.4	5.39
5			0.95	0.14

Table 6.3-Total Positive Moment

Location	Positive Moment			Mu
	Deck+Barrier	WS	(LL+IM)/E. S. W.	
A	0.04	0.1	7.37	<b>13.11</b>
B	0.66	0.1	5.88	11.26
C	0.34	0.1	5.88	10.86
D	0.89	0.1	1.80	4.41

Table 6.4-Total Negative Moment

Location	Negative Moment						Mu
	Overhang			Interior			
	Deck+Barrier	WS	(LL+IM)/E. S. W.	Deck+Barrier	WS	(LL+IM)/E. S. W.	
1	2	0.19	7.46				<b>15.84</b>
2				0.59	0.22	7.53	14.25
3				0.85	0.22	6.74	13.19
4				1.18	0.22	5.39	11.24
5				1.11	0.19	0.14	1.92

Also performed is the analysis due to construction loading (refer to Chapter 3.1.2). The total factored moment is 115.7 ft-kips considering two loader wheels arranged in the middle of the precast panel, which converts to 16.5 ft-kips per unit length. This flexural moment is larger than the moment due to HL-93 loading, i.e., the construction loading controls the ultimate design. The flexural capacity of the 6" thick precast deck panel is 25 ft-kips considering a unit length section reinforced with 1-0.5" diameter pretensioned strand plus 1-#6 rebar. In terms of analysis at Service III, the stress at the deck section bottom due to the construction loading can be determined as follows:

$$f_b = \frac{P}{A} - \frac{M_{DL} + 0.8M_{construction}}{S_b}$$

Where  $P = 0.9(202.5)0.153 = 27.9$  kips

$$A = 12(6) = 72 \text{ in.}^2$$

$$S_b = \frac{12(6^2)}{6} = 72 \text{ in.}^3$$

$M_{DL}$  = the moment due to the deck self-weight underneath the loaders =  $6.4/7 = 0.91$  ft-kips

$M_{construction}$  = the maximum moment due to the loaders plus the panel it erects =  $60.6/7 = 8.66$  ft-kips.

$$\text{Thus, } f_b = \frac{27.9}{72} - \frac{0.91(12) + 0.8(8.66)12}{72} = 0.388 - 1.306 = -0.918 \text{ ksi.}$$

This indicates the panel cracks under this construction loading. However, the arrangement of loader wheels is very conservative and only reasonable for the ultimate design. Normally there should be only one loader, i.e., either one wheel or two wheels of the same loader (5 ft spacing), acting on the panel between the adjacent beams. Also, it is

very conservative to determine the moment from a simply-supported beam model since there are 4-#7 bars and 2-0.5” diameter strands across the open channel.

Rebuild the model using the section properties given in the Chapter 3.1.5. Note that the section over the gap has 2-0.5 in. diameter pretensioned strands and 4-#7 bars per two feet. The section properties, area (A) and moment of inertia (I) per unit length, are given as follows:

$$A = [4(0.6) + 2(0.153)] / 2 = 1.353 \text{ in.}^2, \text{ and}$$

$$I = \frac{1}{2}(4) \left[ \frac{\pi(0.875^4)}{64} + 0.6(1.563^2) \right] + \frac{1}{2}(2) \left[ \frac{\pi(0.5^4)}{64} + 0.153(1.375^2) \right] = 3.281 \text{ in.}^4. \text{ The}$$

precast panel section, 1 ft x 6 inch, has the properties of  $A = (12)6 = 72 \text{ in.}^2$ ; and  $I = 216 \text{ in.}^4$ .

Considering the distributed loading from one front wheel is 12.45 kips/ft, the maximum moment due to these two loading cases can be determined as 35 ft-kips, which corresponds to a moment of 5.0 ft-kips per unit length.

$$f_b = \frac{27.9}{72} - \frac{0.91(12) + 0.8(5.0)12}{72} = 0.388 - 0.818 = -0.430 \text{ ksi, which is within the}$$

tensile stress limit.

in the traffic direction.

Constructing formwork is one of the most expensive and time-consuming duties for CIP bridge deck construction. Approximately 55% ~ 60% of the bridge deck cost is in the labor and materials related to formwork. About 20% of the overall time of the project is spent on forming. Wood forming is commonly used to form a flat soffit between the supporting beams. Also, there are a number of reusable and adjustable form accessories available in the market. Some of the manufactures producing new forming systems are listed as follows: Borg Adjustable Joint Hanger Co., Minnetonka, MN; Economy Forms Corp. (EFCO), Des Moines, IA; Modern Bridge Forming Company, St. Louis, MO; Symons Corp., Des Plaines, IL.

This system provides a smooth riding surface for the traffic. However, field casting and curing concrete necessitates a good field quality control, which might be impacted by harsh weather conditions. Also, longitudinal and transverse cracking are regularly reported in CIP decks (Rogalla, 1995). The volumetric change due to shrinkage, creep, thermal stresses, and chemical incompatibility of concrete components results in transverse cracking. Longitudinal cracking results from flexural stresses. Cracking causes rapid deck deterioration because of corrosion of the steel reinforcement. Therefore, CIP decks normally last only 20 to 30 years and requires replacement while the bridge girders are still in service. Even though some state agencies allow the use of epoxy coated welded wire, there are some problems encountered with this type of reinforcement, such as splicing, especially with large bar size, and the effect of epoxy coating on development and anchorage.

### **1.2.2 Precast Concrete Stay-in-place (SIP) Deck Panel System**

Precast concrete SIP deck panel system has been widely used in the United States. A survey conducted by the PCI committee in 1982 showed that 21 states used this system regularly, i.e., Kansas, Missouri, and Virginia. This system has the advantage of cost-effectiveness in comparison with the CIP system. Also, it reduces construction time by eliminating field construction and removal of formwork.

Precast prestressed panels work as stay-in-place formwork spanning the girders. Although this system avoids field forming between girders, the formwork for deck overhang is usually still required. Once the panels are erected over the girders, the top layer of reinforcement is placed and a cast-in-place concrete topping is poured. The top surface of the panels is normally roughened to provide mechanical bonding with the CIP topping slab. Thus, the panels are made composite with the concrete topping after the CIP concrete hardens. The SIP panels house the positive reinforcement to resist the moment due to its self-weight and CIP topping. The topping slab houses the negative moment reinforcement to carry the moment due to superimposed loads over the girders. The precast panels are 2.5 in. to 3.5 in. thick, and prestressed at their centerlines with 3/8 in. or 1/2 in. strands. They are typically 4 ft to 8 ft wide. The thickness of CIP topping varies from 3.5 in. to 6 in., contingent on girder spacing.

The SIP deck panel system proposed by Badie and Tadros (1997) consists of a 4.5 in. thick SIP precast continuous panel and a 4.5 in. thick CIP concrete topping. [Figure 1](#) shows the cross section of the deck system, on a typical 44 ft wide bridge. [Figure 2](#) illustrates the plan view of the precast panel. The panel covers the entire bridge width and varies from 4 ft to 12 ft in length depending on the transportation situation and

lifting equipment available in the field. Along the girder line, there is a full-length gap of 8 in. wide to accommodate the shear connectors. The panel is pretensioned from end to end with 16-0.5 in. diameter, Grade 270 ksi strands. The strands are provided in two layers and uniformly spaced at 12 in. as shown in Section A-A of [Figure 3](#). In order to maintain the 8 in. gap over the girderline and to transmit the prestressing force at the gap at the time of releasing the strands, 28-#6 reinforcing bars are arranged in two layers, shown in Section C-C of [Figure 3](#). These bars have an 18 in. embedment length to transfer the compression force over the gap. Shear keys and reinforced pockets are provided to maintain continuity in the longitudinal direction between the adjacent precast panels. Section 1-1 in [Figure 4](#) illustrates the dimension of the proposed shear key. Reinforced pockets are spaced at 2 ft on center. Also shown in [Figure 4](#) is the pocket configuration. The panel is reinforced longitudinally with #4 bars spaced at 2 ft at the location of the pockets. The #4 bars provide the lateral distribution of concentrated live loads, shrinkage reinforcement, and longitudinal connection between precast panels. An innovative confinement detail is used to provide the tension development for the #4 bars. The splice consists of a loose 9 in. #4 bar and a spiral of 3 in. O.D., 1 in. pitch, and 0.25 in. wire diameter (see [Figure 5](#)). This splice detail was evaluated with tension specimen and found to produce the full bar yield strength of 60 ksi (see Yehia et al., 1997). A bolt-leveling device was developed to level the panels by turning the bolt up and down when the panels set over the supporting girders.

This panel system can reduce construction time by 3% to 20% in comparison with

the full-depth CIP deck system (see Badie, 1997). Also, it shows substantial elimination of the most significant problem encountered in the conventional CIP deck system, which is the transverse cracking due to concrete shrinkage. This system provides rapid construction as field forming is eliminated because the SIP panel can cover the full width of the bridge.

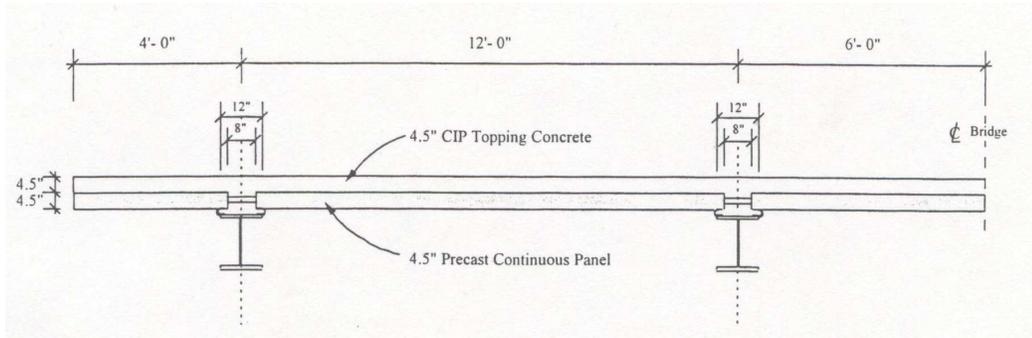


Figure 1.1-Cross Section of a SIP Bridge Deck

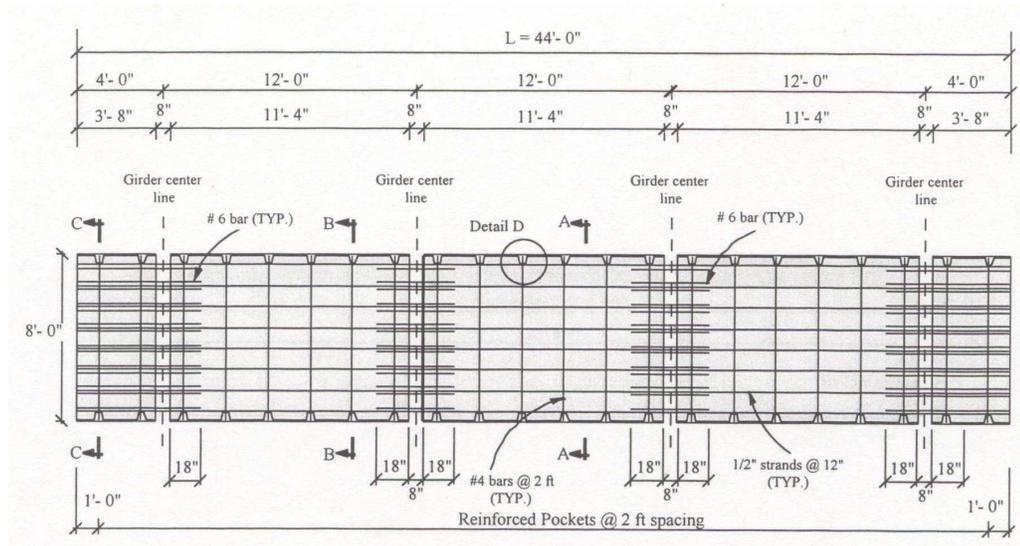


Figure 1.2-Plan View of a SIP Panel

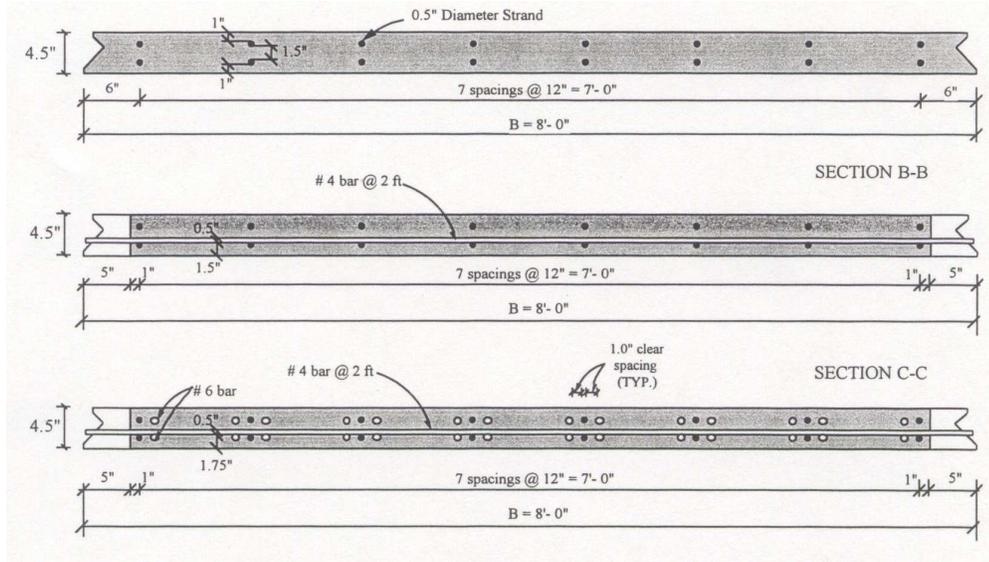


Figure 1.3-Cross Sections at A-A, B-B, and C-C

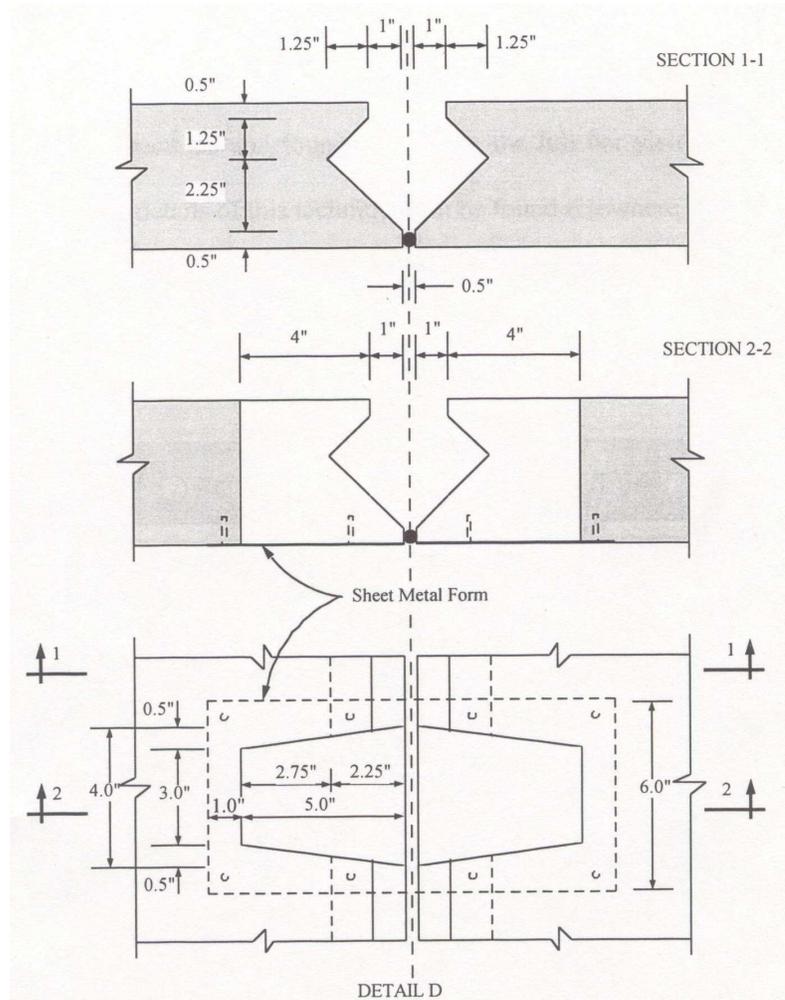


Figure 1.4-Transverse Joint between the Precast Panels

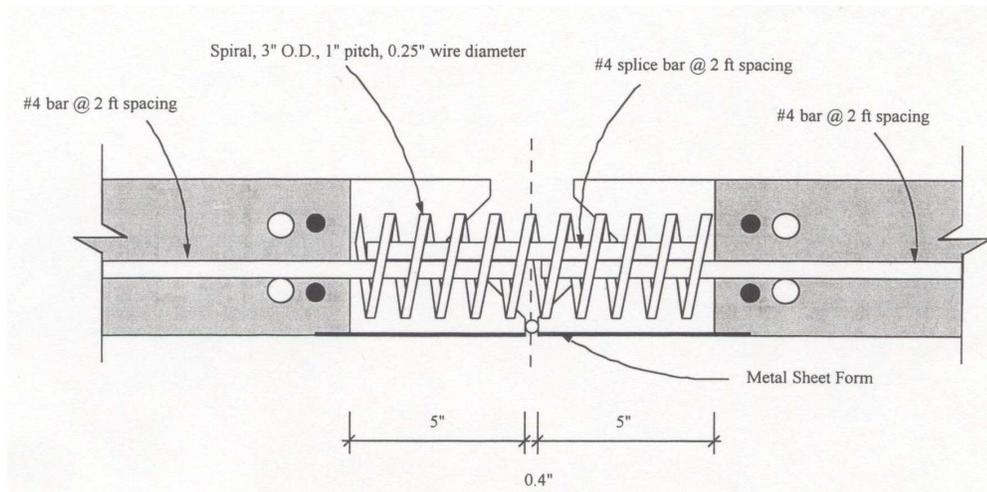


Figure 1.5-Reinforced Pocket Details

### 1.2.3 Precast Deck Panel System

The precast deck panel system has become increasingly popular due to the demand for high performance structural components and fast construction. Full-depth precast system may be conventionally reinforced, transversely prestressed, or transversely prestressed and longitudinally post-tensioned. It is desirable to have longitudinal post-tensioning to control transverse cracking at the panel joints due to service loads. Transverse prestressing can be included so that the cross section dimensions of the panel can be reduced. This system has two main advantages: 1) high durability; and 2) fast construction. The concrete of precast panel is plant controlled which normally has a better quality than that of the CIP deck. The deck concrete is generally strong when it begins to interact with the beams. Also, the shrinkage, creep, and temperature drop in the cement hydration cycle occur before the deck is made composite with the rigid steel or concrete girders, which eliminates a major source of cracking. Some of the newly developed precast deck panel systems are studied and listed herein.

### 1.2.3.1 Full-depth Precast Deck Panel System by Yamane and Tadros

The precast panel system proposed by Yamane and Tadros, et al. (1998) is shown in Figure 6. This system includes precast prestressed concrete panels, welded headless studs, welded threaded studs, grout filled shear keys, leveling bolts, and threaded bars for post-tensioning. The panels are transversely pretensioned and longitudinally post-tensioned. Pretensioned strands are arranged in two layers and the eccentricity is minimized since the panel is subject to both positive and negative moments. Longitudinal post-tensioning has typically been provided at or near the mid-depth of panels and the post-tensioning tendons are located above the top flanges of girders. Blockouts are provided for anchorage and couplers at both transverse edges of the panels.

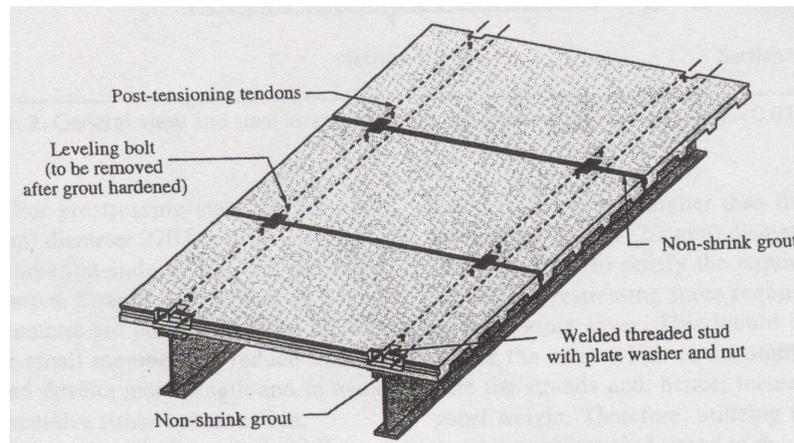


Figure 1.6-Precast Panel System by Yamane and Tadros

### 1.2.3.2 EFFIDECK System by Fort Miller Co., INC

EFFIDECK System, developed by Fort Miller Co., INC, is a lightweight deck system consisting of a 5 in. reinforced concrete slab supported by closely spaced structural steel members cast compositely with the deck. The typical EFFIDECK panel is 10 ft wide. Steel members rest directly on stringers or floor beams to which

they are bolted (see [Figure 7](#)). 3/4 in. diameter studs are welded to the top surface of the tube beams so that composite action between the concrete slab and the support tube beams can be maintained (see [Figure 8](#)). The composite action between the panel and the supporting stringers is provided by blockouts in the deck, which is along the girder lines. As shown in [Figure 9](#), EFFIDECK voids out concrete between tubes where it is not needed. This makes this deck system light and as strong as it needs to be. Notice how the deck is haunched down to the bottom of the tubes at stringers or floor beams as illustrated in [Figure 9](#). [Figure 10](#) shows an EFFIDECK Panel for Stewart Ave. in Ithaca, NY, which consists of a 5 in. deck supported by 8 in. deep tubes. Stud shear connectors and threaded stud hold downs are attached directly to the stringer or floor beams from the top of the deck (see [Figure 11](#)). Panel-to-panel connectors are attached to adjacent steel tubes, also from the top of the deck. All pockets are filled with non-shrink grout to achieve full composite action. Steel shims between tubes and supporting floor beams constitute a steel-on-steel load path (see [Figure 12](#)). Non-grouted EFFIDECK panels provided a work platform for the crane to set new panels, as shown in [Figure 13](#).

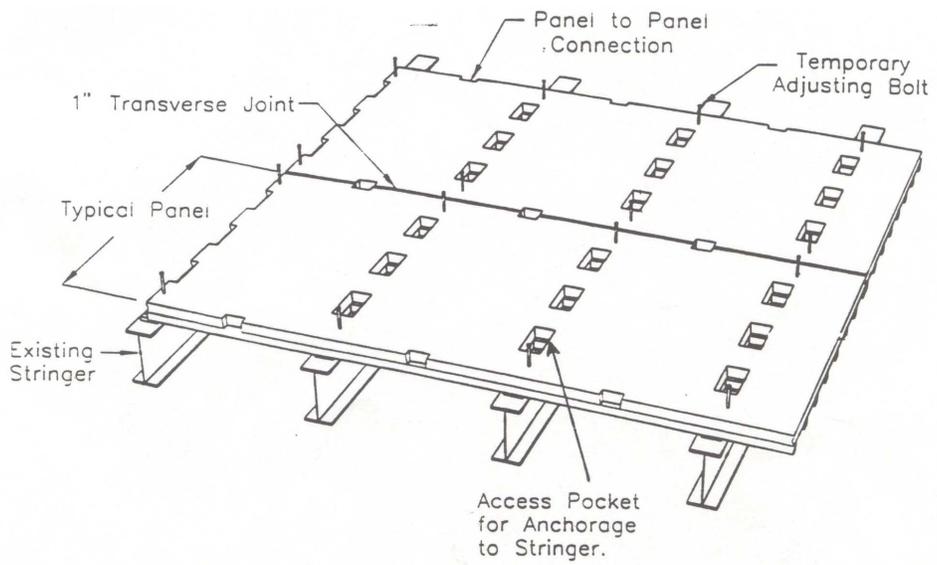


Figure 1.7-EFFI-DECK System by Fort Miller Co., INC



Figure 1.8-EFFIDECK Panel Plan View



Figure 1.9-A View of EFFIDECK Panel from Bottom



Figure 1.10-EFFIDECK Panel for Stewart Ave. in Ithaca, NY

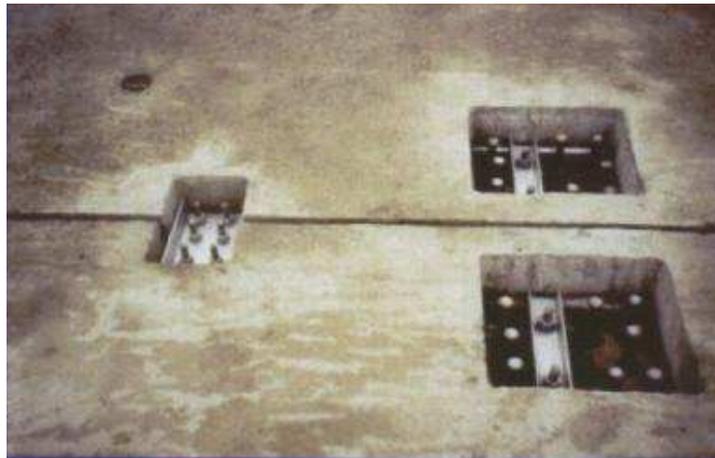


Figure 1.11-EFFIDECK Panel-to-Panel Connection and Shear Connector Details



Figure 1.12-EFFIDECK Panel Shimming



Figure 1.13-Crowned EFFIDECK Panel Placed on New Steel Stringers



Figure 1.14-Non-grouted EFFIDECK Panel Carrying the Crane

### 1.2.3.3 Full-depth Precast Bridge Deck Panel by Issa, et al.

The full-depth bridge deck panel system proposed by Issa, et al. (2000) is shown in [Figure 15](#). The proposed deck system utilizes stage construction to maintain traffic both ways. The deck panels can be either precast or precast prestressed, and post-tensioned along the longitudinal direction to provide continuity and secure tightness of the joints between the adjacent precast elements. The panels are connected to the existing steel stringers or concrete girders through shear pockets to provide composite action. The shear pockets are designed so that the configuration

represents a tapered form, i.e., the top of the pocket is wider. The bottom of the tapered shear pocket is controlled by the width of the beam flange. Furthermore, the corners on the pockets must be beveled to eliminate stress concentration.

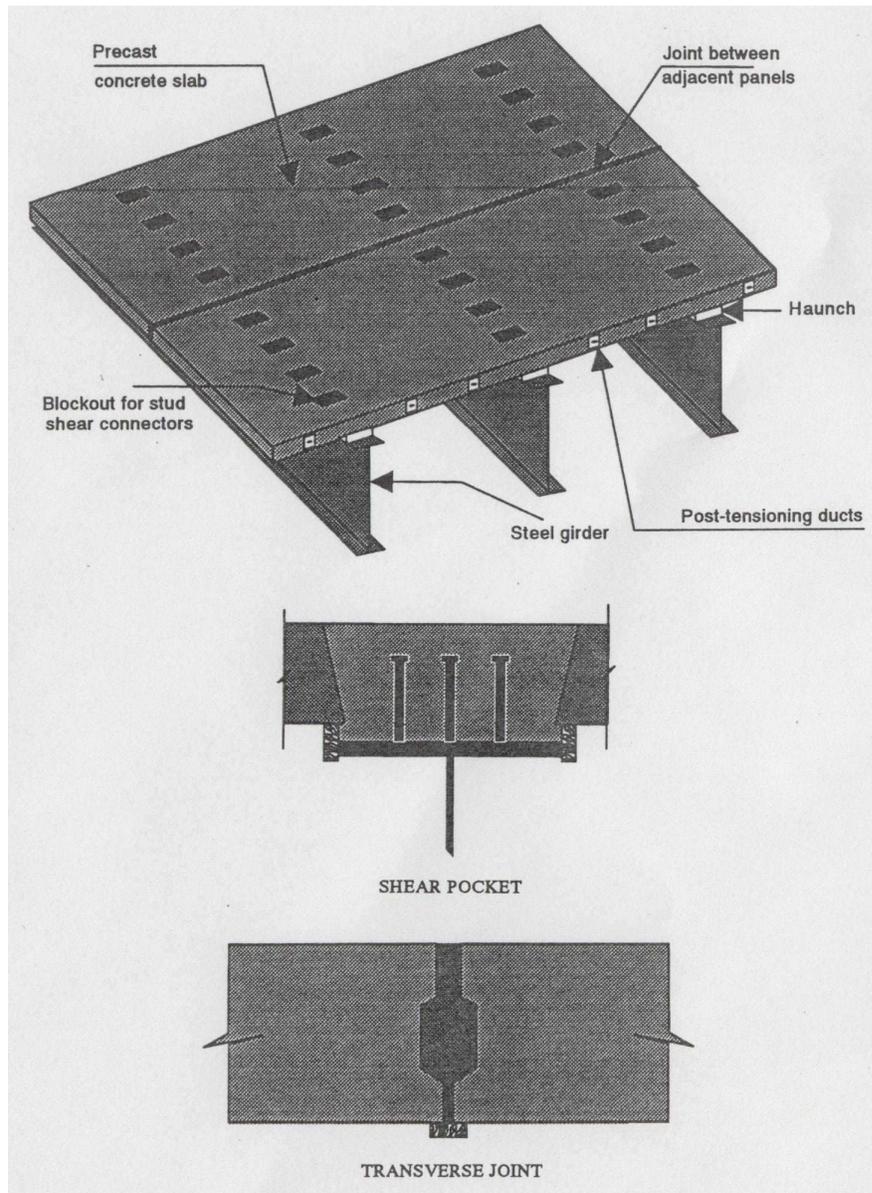


Figure 1.15-Full-depth Precast Bridge Deck Panel by Issa, et al.

#### 1.2.3.4 Full-depth Precast Bridge Deck Panel Used in Other States

The Connecticut DOT undertook rehabilitation of approximately 1640 bridges at an estimated cost of \$1.6 billion. Many of the bridges involve complete deck

replacements requiring complicated stage erection sequences and occasional bridge closure during construction. In an attempt to expedite the construction process, a design using a precast concrete deck slab was incorporated for one of the bridges, Connecticut Bridge 03200. It is a six-span bridge with a total length of 700 ft consisting of straight composite plate girders running on tangents from pier to pier. The structure is located on a horizontally compound curve requiring various degrees of deck superelevation. To account for the curvature, each slab was designed as a trapezoid. One end of the slab would be 8 ft wide and the other slightly less, depending on the curvature. Figure 16 shows a typical section of precast slabs of this bridge. The panels were designed as full bridge width, 26.7 ft, and 8 in. in depth. The shear connector blockouts were rectangular, 18 in. x 5 in. at the top and tapered from top to bottom. The spacing of the blockouts was 2 ft on centers for each slab. Three 7/8 in. welded stud shear connectors were placed in each blockout.

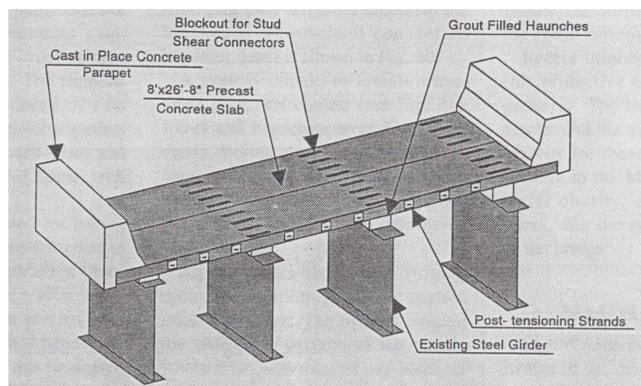


Figure 1.16-Typical Section of Precast Slabs of Bridge 03200 by Connecticut DOT

### 1.3 SHEAR KEY

#### 1.3.1 Shear Key Proposed by UNL Researchers

It is required that the transverse joint can transfer live loads and prevent water

leakage. The transverse joint configuration proposed by Yamane and Tadros (1998) is illustrated in Figure 17. A clear spacing of 0.4 in. is provided between panels for production and construction tolerances. Rapid-set non-shrinkage grout material, Set 45, made at Master Builders, Inc., was recommended based on the study performed by Gulyas. The simulated axle load consisted of four concentrated loads in accordance with AASHTO Specifications and was applied to three locations (see Figure 18). Location 1 was adjacent to a transverse joint, Location 2 was centered between transverse joints, and Location 3 was at the edge of a precast panel. A 2 million-cycle fatigue loading was applied to Location 1. A water pool was provided at the transverse joint to check for water leakage during the fatigue loading. A monotonic ultimate load was applied at Location 2. At Location 3, only service loads were applied to check stress in the panel. The test results showed that stresses for the loaded side and the unloaded side of the transverse joint were about the same for maximum positive and negative moment zones, which indicates that the transverse joint detail effectively transfer loads from one panel to an adjacent panel. In addition, there were no cracks or water leakage under the fatigue loading. Therefore, the transverse joint with the given configuration exhibits satisfactory performance under service load and fatigue load.

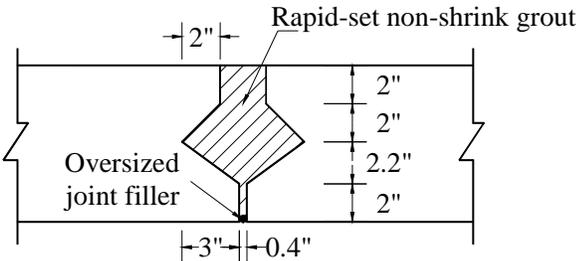


Figure 1.17-Transverse Joint Configuration

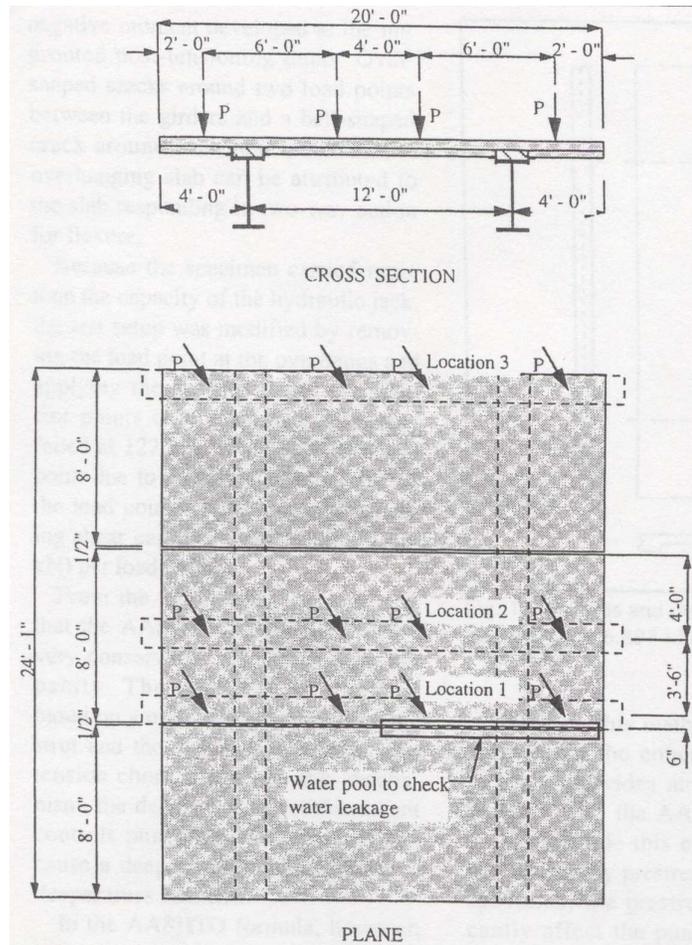


Figure 1.18-Loading Locations

### 1.3.2 Shear Key Investigation by Issa, et al. (1995)

The investigation by Issa, et al. concluded that the transverse joints between the precast slabs should be female-to-female (shear key) and have a minimum nominal width of  $1 \frac{1}{4}$  in. at the top and  $\frac{1}{2}$  in. at the bottom. Longitudinal post-tensioning was also recommended to secure tightness of the joints. Issa, et al. also did a case study of shear keys used in the U. S. Some of them are listed as follows:

In the Connecticut Bridge 03200, a standard shear key configuration filled with high strength, non-shrink grout was chosen for the transverse joint (see Figure 19). Longitudinal post-tensioning was designed to provide continuity. The strands were

pulled through plastic ducts that were spliced at each transverse joint through small blockouts. An arbitrary stress of 150 psi was chosen for simple spans and was increased to 300 psi in the three-span continuous portion of the bridge to account for the significant composite dead loads and live loads stresses.

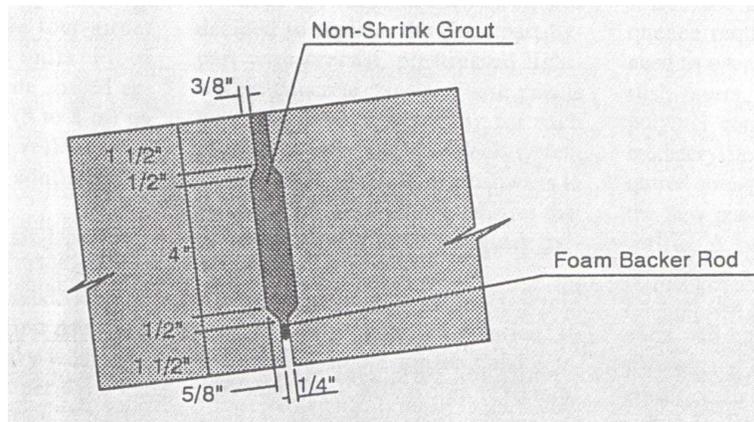


Figure 1.19-Standard Shear Key Configuration of Bridge 03200 by Connecticut DOT

In 1987, Maine DOT had a deck replacement project on the Deer Isle-Sedgwick Bridge. This bridge consists of nine spans: four at 65 ft, one at 484 ft, one at 1080 ft, one at 484 ft, and two at 65 ft. It has a total width of 23.5 ft center to center of the suspended girders. A precast concrete slab alternative was adopted by the contractors for redecking process. The lightweight precast deck panels were designed to cover a half width of the bridge to maintain traffic flow during construction. The panels were 6 ½ in. thick, 9 ft 11 in. wide and of variable length depending on the spacing of the suspended girders. A typical female-to-female transverse joint was chosen (see [Figure 20](#)). Joints were filled with epoxy mortar after the shear connectors and plate connections were welded. No prestressing was applied to the slabs. Epoxy coated reinforcing steel was used. All the panels had a ¼ in. epoxy waterproofing overlay applied prior to erection. The overlay covered the entire top surface of the panels

within 6 in. of any blockout for shear connectors or shear key. After the shear keys and blockouts were filled, the epoxy waterproofing overlay was placed over these areas. However, recent inspection has revealed that this protective coating does not work properly. The transverse joints have cracks resulting in leakage in the panel. The reasons for these adverse results, according to the Maine DOT, are material quality, construction procedures, and the substantial movement of the bridge.

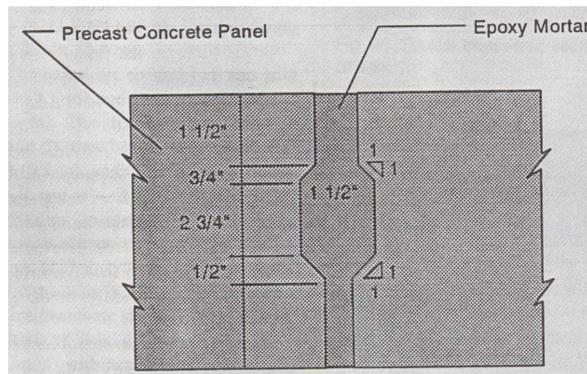


Figure 1.20-Typical Transverse Joint of Deer Isle-Sedgwick Bridge by Maine DOT

The Seneca Bridge funded by Illinois DOT was built in 1932. It consists of 13 spans. Spans 1 through 5 and 10 through 13 are approach spans, while Spans 6 through 9 are interior truss spans. The four truss spans, along with the approach spans, had the existing concrete deck removed and replaced with a 6 1/2 in. precast, prestressed deck slab. All precast slabs were match set, with the replacement being performed in sections. A male-to-female type joint was used between the deck panels (see Figure 21).

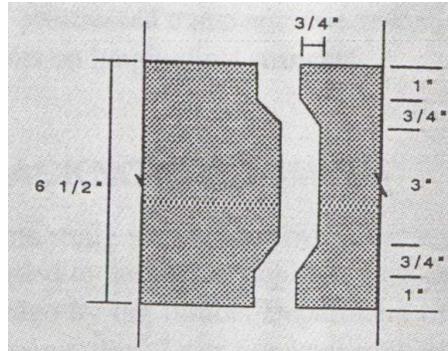


Figure 1.21-Typical Joint between Adjacent Precast Panels of Seneca Bridge

### 1.3.3 Shear Key Grouting Material

The most commonly used joint materials in civil infrastructure, Set 45, Set 45 hot weather (HW), Set Grout, and Polymer concrete were investigated by Issa, et al. (2003). A total of 36 full-scale specimens were tested to evaluate the performance of grouting materials subjected to vertical shear, direct tension, and flexure. Based on the observed results, it was concluded that the shear, tensile, and flexural strengths of polymer concrete grout are the highest among all types of material studied. Polymer concrete is the least permeable and it also performs best in terms of shrinkage. However, polymer concrete is very expensive and requires careful application, including thorough surface preparation and adequate mixing. Issa, et al. recommended the use of Set Grout in transverse deck joints due to its ease of use and satisfactory performance. In special cases where the joint is subjected to excessive stresses or quick resumption of traffic is critical, the proper application of the polymer concrete is recommended.

### 1.4 TYPES OF OVERLAY

Several types of overlays are currently used in bridge deck rehabilitation projects (see references by Issa et al.). They include latex modified concrete (LMC), EP-5

(epoxy), silica fume, etc. EP-5 is normally used along with aggregates of the same size, and applied as an overlay in the range of 1 3/4 to 2 1/4 in thick. Silica fume overlay requires a minimum thickness of 1 1/4 inch. This type of overlay is beneficial and more effective than latex modified concrete. Silica fume is less expensive but more sensitive to temperature change than LMC.

#### **1.4.1 Epoxy Concrete Overlay**

Prior to placing the epoxy concrete, the entire deck surface should be cleaned by shotblasting or other specified cleaning methods. This is done to remove deteriorated asphaltic material, oils, dirt, rubber, curing compounds, paint carbonation, laitance, weak surface mortar, and other potentially detrimental materials that may interfere with the bonding or curing of the overlay. The epoxy mixture is uniformly applied to the surface of the bridge deck with a squeegee or paint roller. The temperature of the bridge deck must be above 60°F (16°C) at the time of application. The dry aggregate is then applied to completely cover the epoxy mixture within 5 minutes. Type EP-5 is a low modulus patching, sealing, and overlay adhesive with an elongation of at least 10 percent. Brooming is not performed until the epoxy resin has cured sufficiently to prevent tearing.

#### **1.4.2 Silica Fume Concrete**

Silica fume concrete is a very fine material consisting primarily of noncrystalline pozzolanic silica produced by electric arc furnaces as a by-product of the production of metallic silicon or ferrosilicon alloys. It is also known as condensed silica fume or microsilica. When the overlay is placed on newly cast concrete with a surface that is

clean and free of curing compound or other chemicals, light sandblasting or shot blasting is required to remove the laitance. The overlay should be replaced until the new concrete has attained at least 90 percent of its design strength. The surface of the base concrete should be wetted at least one hour before placement of the overlay. The cleaned and wetted surface is covered with a plastic to prevent contamination prior to placement. Silica fume concrete is brushed on the surface and excess aggregate is discarded just prior to placement. The minimum thickness of the bridge deck overlay must be not less than 1 1/4 in. Silica fume concrete does not bleed as much as normal concrete and is more susceptible to plastic shrinkage cracking.

#### **1.4.3 Latex Modified Concrete**

Latex modified concrete (LMC) achieves a compressive strength of 3,000 to 3,500 psi within 2 to 3 days, which is sufficient to allow traffic on the overlay. For applications requiring high-early strength, Type III cement can be used. The smaller particles associated with Type III cement react quickly in LMC and allow LMC overlays to be open to traffic in 24 hours, with no reduction of the ultimate properties of the concrete. The tensile strength of the LMC bond exceeds 100 psi after one-day cure. The normal curing procedure for LMC is one day of moist cure followed by air drying for the remainder of curing time. The low permeability of LMC contributes to the impermeability of the cured concrete and mortar. By resisting infiltration of moisture and gases, LMC produces concrete with a modulus of elasticity that is 15 percent lower than comparable conventional concrete, i.e., LMC is not as brittle as conventional concrete.