CHALLENGES OF THE SEGMENTAL PRECAST CONCRETE OPTION FOR YANKTON BRIDGE

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ABSTRACT

The 1590 ft. bridge crossing the Missouri River connects Nebraska with South Dakota at the town of Yankton. The existing steel bridge is scheduled to be replaced with a new bridge. The Nebraska Department of Roads has already designed the steel plate girder option and is in the process of designing a precast concrete alternative for the letting.

The steel design consists of six spans, 235 ft. - 4 x 280 ft. - 235 ft., with the plate girders varying in depth from 6 ft at the midspans to 10 ft at the piers. The bridge width is 76 ft.-8 in. The proposed precast concrete alternative consists of segmental post-tensioned I-girders with similar depths and span arrangement to the steel option. There are many challenges associated with the precast concrete option, including analysis and detailing of the connections between the precast segments as well as between the pier segments and the piers due to construction loadings.

This paper presents the segmental precast design option for the Yankton bridge, discusses the challenges of this design, and offers viable solutions to some of these challenges.

Keywords: Segmental Bridges, Post-Tensioning, Precast Connections, Bridge Construction

INTRODUCTION

The Yankton bridge located on US Highway 81, crosses the Missouri river connecting Nebraska with the town of Yankton, South Dakota. The 1590 ft. existing steel bridge is scheduled to be replaced. The location of Yankton and a picture of the existing bridge are shown in Figure 1.

Fig. 1 Location and Photograph of Current Yankton Bridge

The Nebraska Department of Roads (NDOR) as already completed the design of the steel plate girder option for the new bridge. However, NDOR also desires to offer a precast concrete alternative for the letting and is currently in the process of designing such.

The proposed 1590 ft bridge consists of six spans with the end spans equaling 235 ft. and four interior spans equal to 280 ft. The four lane bridge has a roadway width of 74 ft.- 0 in. and total bridge width of 76 ft. - 8 in.. A total of 7 girder lines at 11 ft. – 4 in. spacing will be used. In addition, there is a 15 degree skew.

SEGMENTAL PRECAST CONCRETE OPTION

PRECAST SEGMENTS

The proposed concrete alternative will use segmental precast NU-I girders. The precast segments will consist of 150 ft. end span and midspan segments, 20 ft end block segments and 130 ft. pier segments, as shown in Figure 2. The construction sequence, starting from one end of the bridge, will begin by placing the first pier segment. The end span segment will then be installed between the first pier segment and the end block segment which will be installed on temporary supports. The next pier segment will then be placed and a span segment will be hung between the first two piers. This process will continue until the end span segment is installed between the last pier segment and the end block segment.

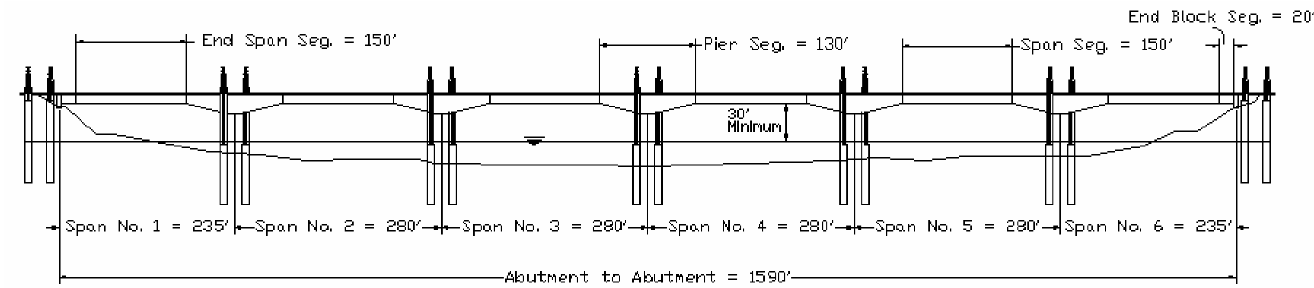

Fig. 2 General Elevation of Concrete Option for Yankton Bridge

The end span and midspan segments will be precast pre-tensioned NU 2000P I-Girders with constant dimensions as shown in Figure 3. The end span and midspan segments contain 46-0.6 in. diameter pre-tensioned strands. The pier segments are also NU 2000P I-Girders with the same dimensions, but varying in height from 78.7 in. to 144 in. at the centerline of the pier as shown in Figure 3 and 4. Due to height of the pier segments, pre-tensioning of the top flange would be very difficult. Therefore, the top flange of each pier segment will be post-tensioned at the precast plant with 24-0.6 in. strands located in two 3.31 in. ducts as shown in Figure 3.

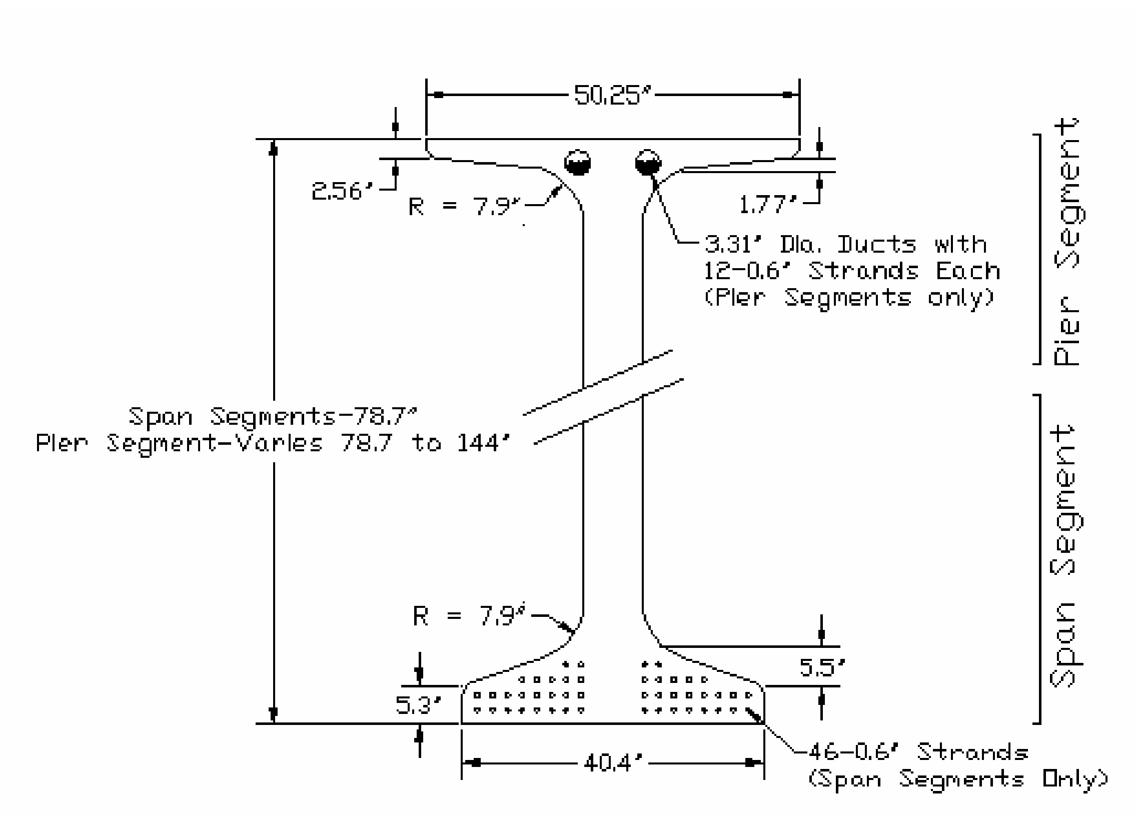

Fig. 3 Cross Section of Pier Segments and Midspan Segments

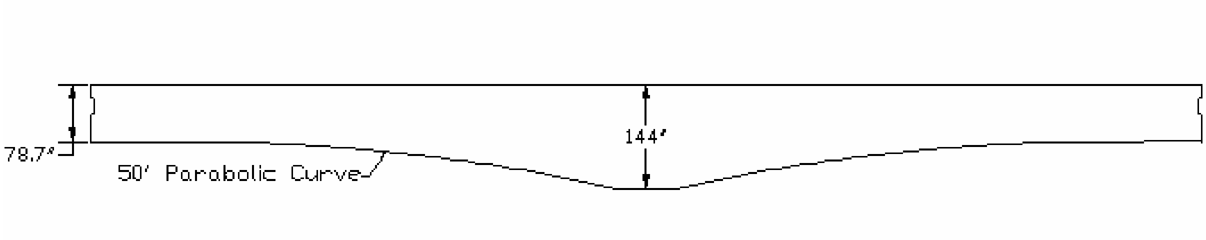

Fig. 4 General Elevation of Pier Segment

In order to use consistent dimensions and lengths for the both the midspan and end span segments as well as to reduce the weight of the precast pieces, a 20 ft. end segment was used. These segments will have the same dimensions as the span segments, shown in Figure 3, except near the abutment end where an end block is used to accommodate the anchorage hardware for the continuous post-tensioning. The concrete dimensions of the end block segment near the abutment end are shown in Figure 5.[1]

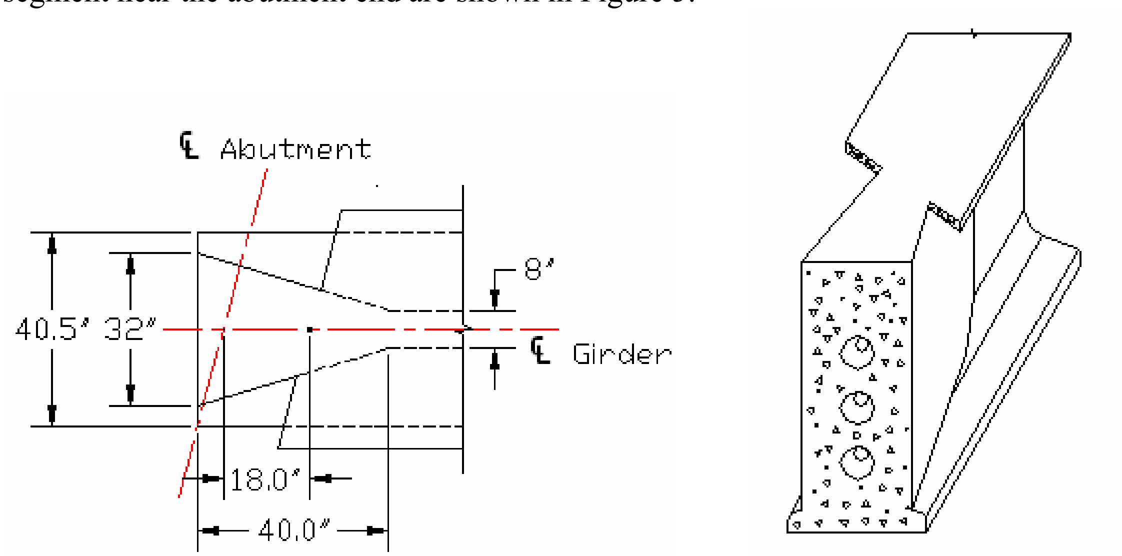

Fig. 5 Plan and General Elevation of End Block near Abutment End

CONTINUOUS POST-TENSIONING

In addition to the 46-0.6 in. pre-tensioned strands in the span segments and the 24-0.6 in. post-tensioned strands in the pier segments, continuous post-tensioning will run throughout the entire bridge length. The continuous post-tensioning will consist of 3 – 4.19 in. ducts each containing 15-0.6 in. strands. The centerline of tendon profile for the continuous post-tensioning is shown in Figure 6. The tendon profile was designed to produce no tensile stresses at the joints.

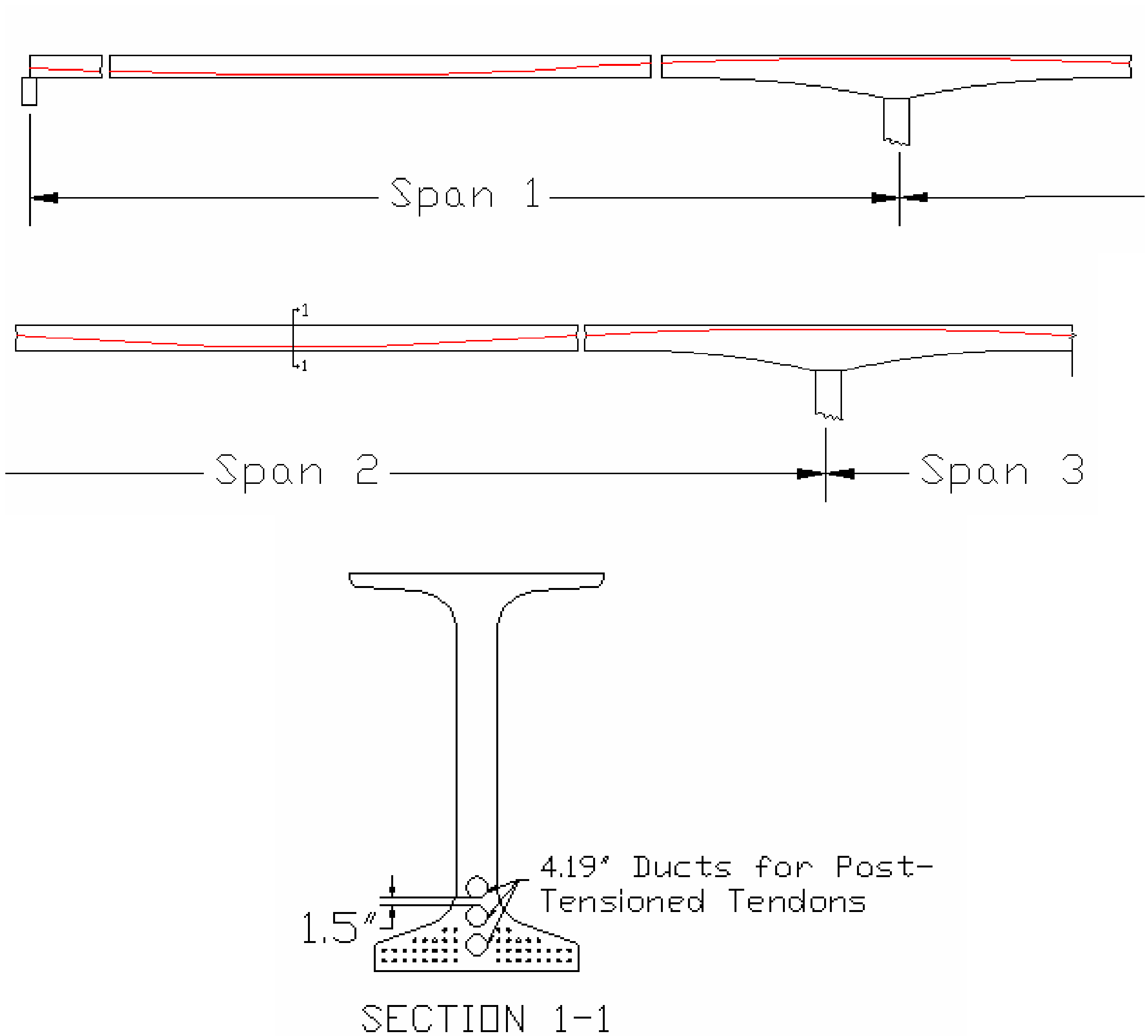

Fig. 6 Tendon Profile of Continuous Post-Tensioning

POST-TENSIONING ANCHORAGE DETAILS FOR PIER SEGMENT

As discussed previously, the pier segments are to contain 24-0.6 in. strands that will be post-tensioned at the precast plant. Therefore, at the ends, anchorage details were necessary to transfer the post-tensioning force. The anchorage details will consist of an anchor box which will be assembled by welding ½ in. plates for the bottom, top, sides and back. A 1½ in. face plate with holes drilled according to the strand pattern will be welded to the bottom and side plates and will bear against the top plate. Details of the welded anchor box are shown in Figures 7 -9.

The strands will be seated against the face plate on 2 in. spacing each way and will be collected into two-3.31 in. ducts over the length of the anchor box as shown in Figure 9.

Since the available weld length on the face plate is limited, the anchor box will be filled with grout, before tensioning the strands to reduce the stress on the welds. The strands will therefore need to be sheided inside the anchor box to allow for post-tensioning after the grout reaches the specified strength. The forces on the anchor box will be transferred to the girder through 16 headed studs and 12 - 3 ft. long - #6 DBA bars welded to the top, bottom and side plates. The strands will be post-tensioned in 3 stages using a mono-strand jack. Post-tensioning will be done in a predetermine sequence to reduce eccentricity, first tensioning the strands to 30 percent of the full force. Following the same sequence, the strands will then be post-tensioned to 100 percent of the full force. Lastly, the strands will be post-tensioned from the other side of the girder to ensure that the desired force has been reached.

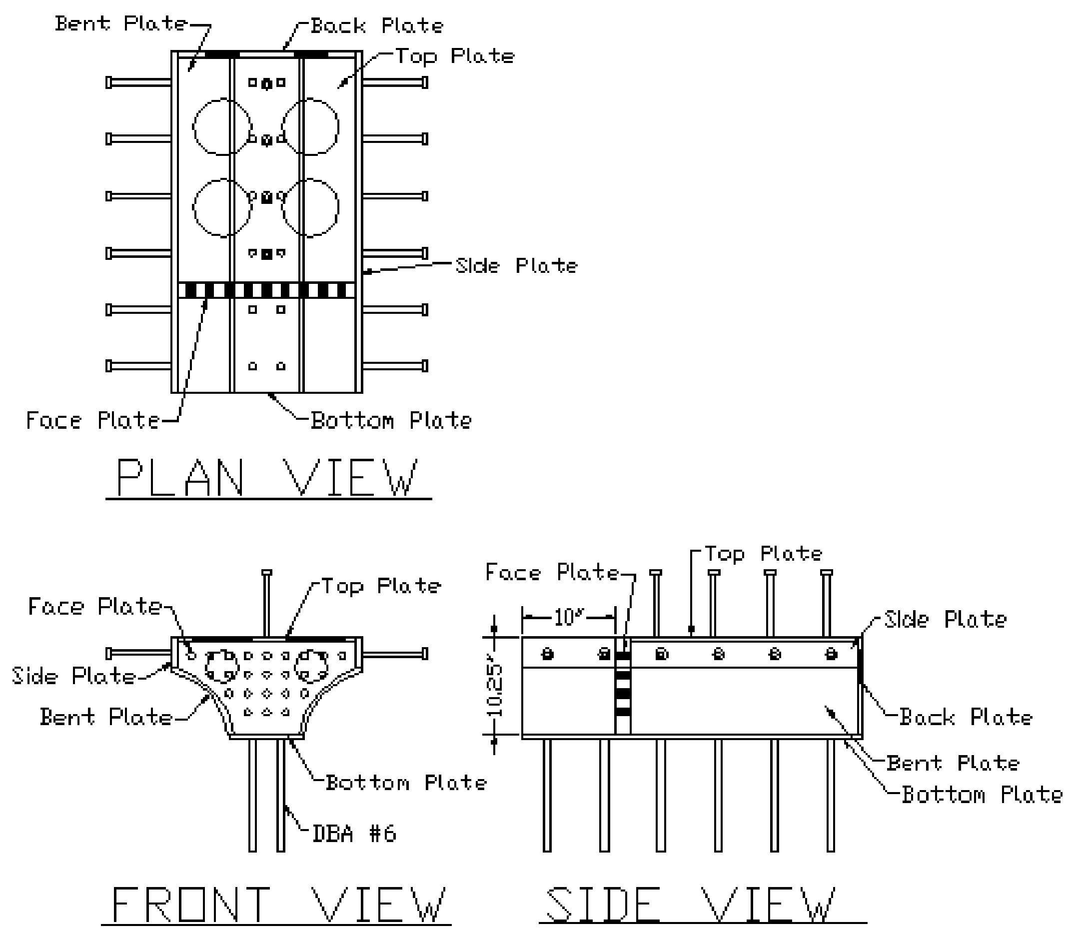

Fig. 7 Details of Welded Anchor Box

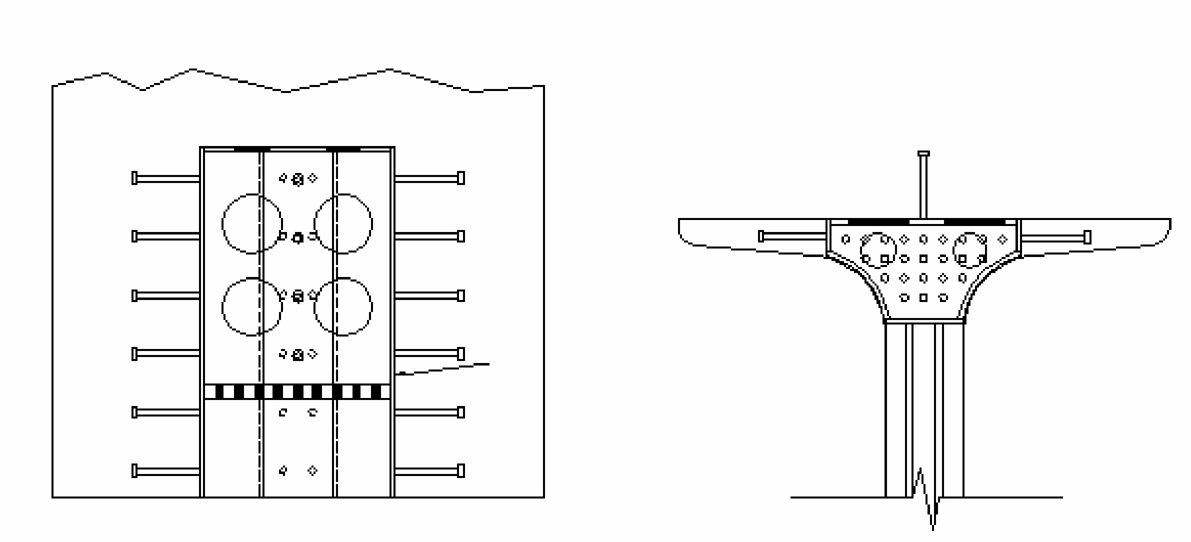

Fig. 8 Welded Anchor Box Placed Inside Pier Segment

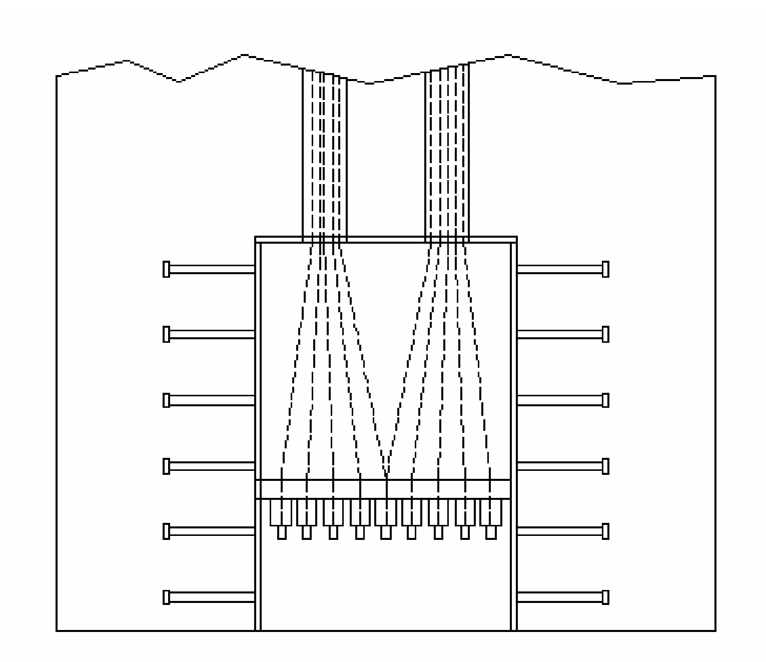

Fig 9. Post-Tensioned Strands in Welded Anchor Box

ERECTION CHALLENGES AND WET JOINT DESIGN

There are many challenges associated the erection of segmental concrete bridges. This is especially true in cases such as the Yankton bridge where temporary supports at the location of the field splices are not allowed. Therefore, field connections had to be designed between the precast segments to provide stability until the bridge is post-tensioned.

CAZALY HANGER

The first step was to transfer the shear force at the wet joint. It was decided to use a Cazaly hanger to accomplish this shear transfer. The cantilever bar will consist of two-2 in. steel plates welded to a ¾ in. steel plate in the middle. In addition, 14 - #6 bars will be used to anchor the hanger to girders. The Cazaly hangers will be embedded in the span segments, which will then hang on the pier segments. The hanger will sit on the bottom plate of the welded anchor box cast in the ends of the pier segment. The details of the Cazaly hanger are shown in Figure 10.

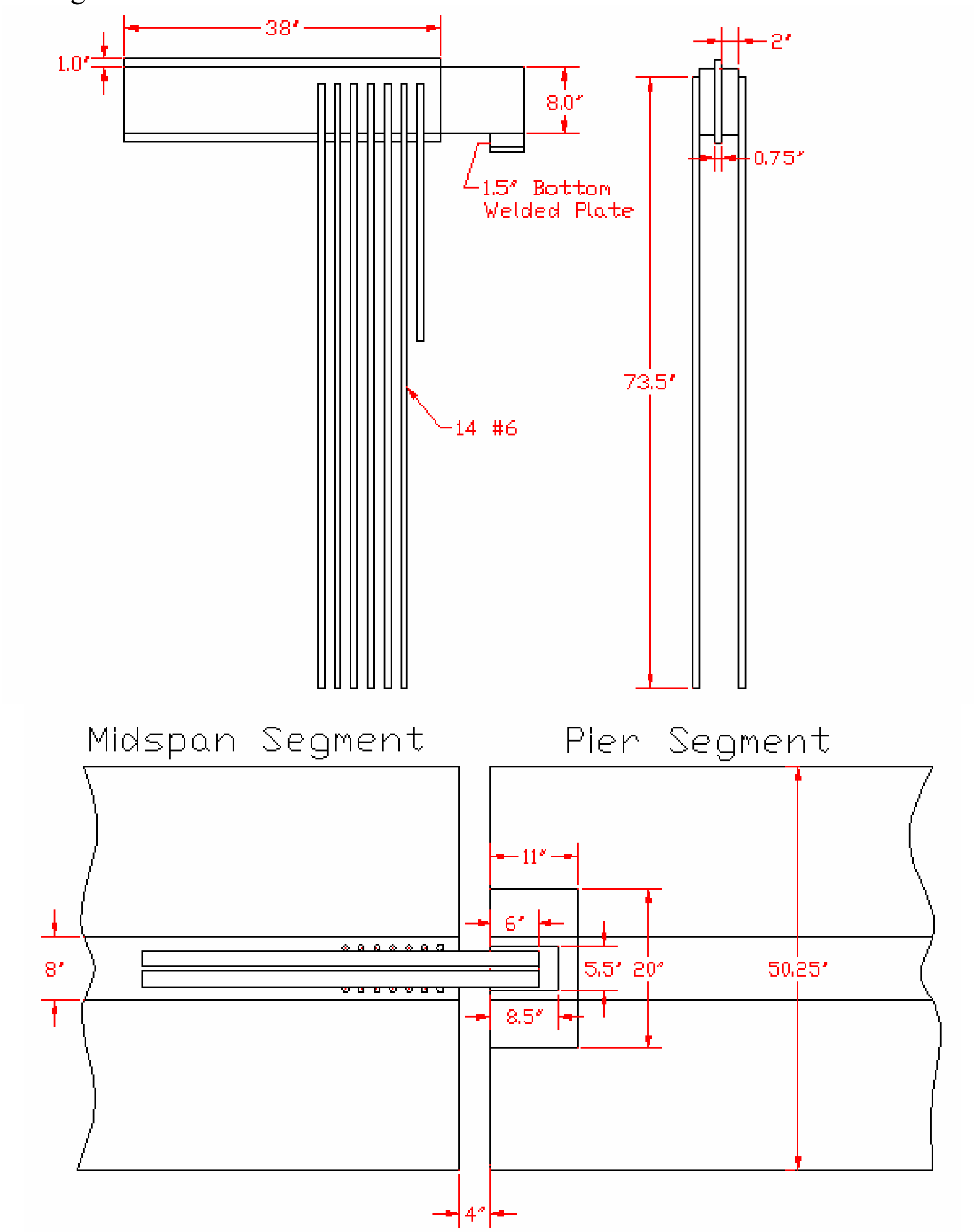

Fig. 10 Details of Cazaly Hanger Between Precast Segments

MOMENT CONNECTION

During analysis throughout the construction stages it was observed that the piers experienced very large torsional forces when one end of the pier segment was connected to a span segment and the other end was a free cantilever. Therefore, it was decided to use a moment connection to reduce the torsion on the end pier. The original concept for the wet joint was to provide an internal moment connection by embedding threaded rods in one segment and hardware to connect the threaded rods in adjacent segment. However, the extremely large tensile forces placed on the welds resulted in the choosing of an alternative system. After a detailed analysis was performed of several options it was decided to use an external moment connection with four – 1 ¾ in. 150 ksi threaded rods. The threaded rods will be provided by the precasters and will be returned to the precaster once the construction is completed. Two sets of threaded rods (8 total) will be used for each girder line. The threaded rods will be moved throughout the construction sequence to be placed at the joint next to pier that is not balanced by span segment on each end as shown in Figure 11.

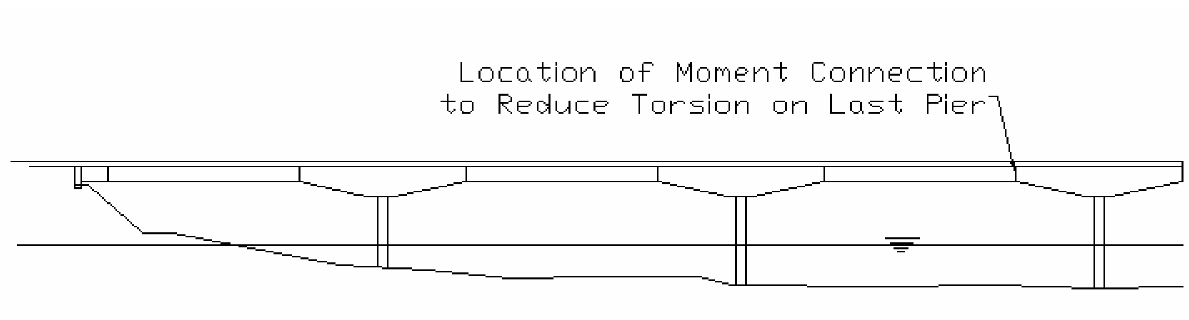

Fig. 11 Location of Moment Connection

In addition, due to the large compression force created by the moment connection, it is necessary to thicken the top flange of the girders by 4 in. near the ends. The strength of this connection was checked at several locations including the wet joint as well the sections at the ends of both the pier and span segments. The connection was found to be adequate at all sections with the critical section being at the end of the pier segment where there is a reduced positive moment capacity due to the 24 post-tensioned strands located at the top of the girder. Figure 12 shows the details of this moment connection.

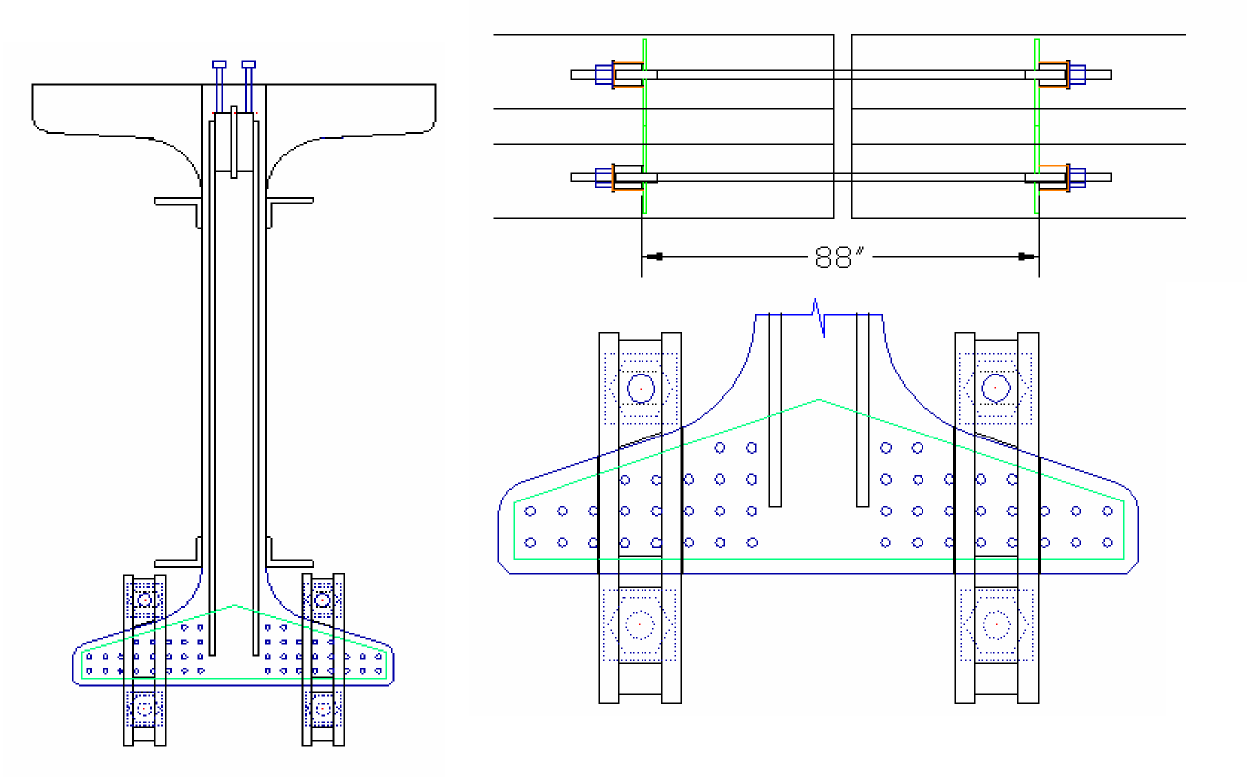

Fig. 12 Moment Connection Using External Threaded Rods

As shown in Figure 12, the girders must be precast with voids in the bottom flange to accommodate the anchorage assembly for the threaded rods. Two - 1¼" plates will be slid into the void between the strands. The threaded rod will be placed between the plates and be anchored using a bearing plate, washer and nut. The anchorage assembly will bear against a ¾ in. steel plate cast in the girder, with holes drilled at the location of the strands. In addition, two 7 ft. long L 4 x 6 x ½ sections will be bolted across the joint as close as possible to the top and bottom flanges to ensure that the span segments and pier segments are correctly aligned. Figure 13 shows the wet joint including the various components of the design.

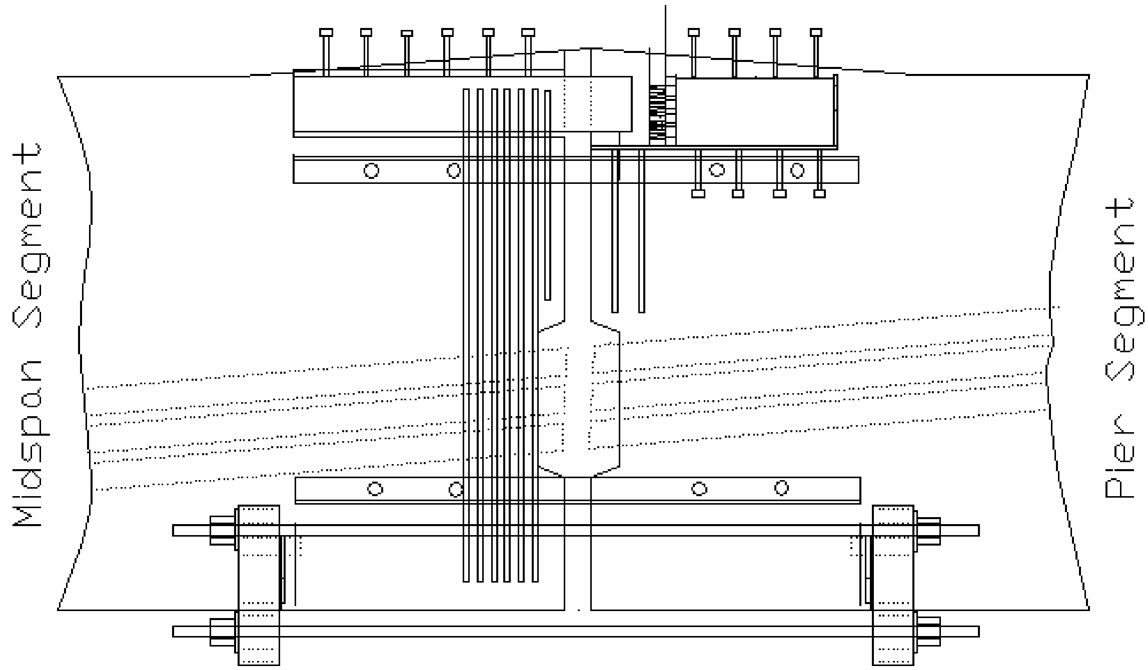

Fig. 13 Wet Joint Design

CONNECTION BETWEEN PIER SEGMENTS AND PIER

To provide additional stability to the bridge during construction stages, the pier segments will be connected to piers using 4 - 1¾ in. 150 ksi threaded rods. The threaded rods will be embedded in the piers and will tighten against the top flange of the pier segment. This connection, shown in Figure 14, between the pier segments and the pier will also help in reducing the moment at the wet joints.

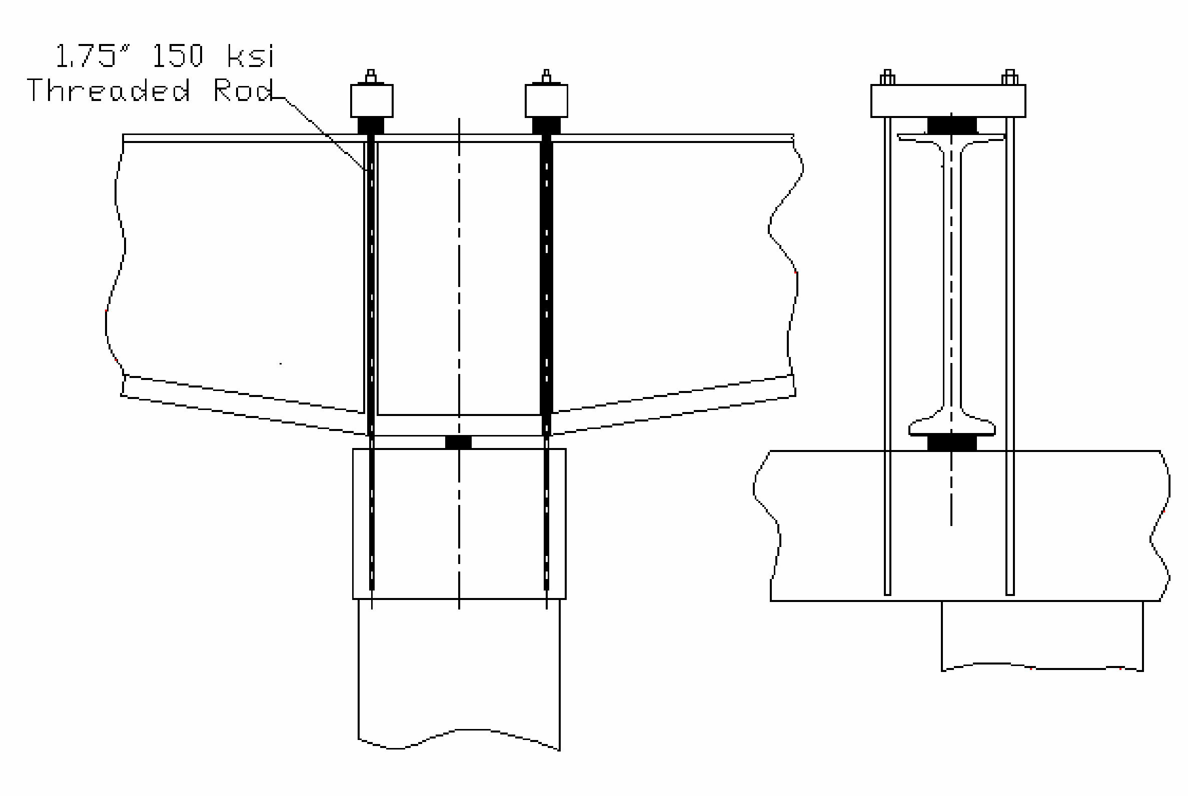

Fig. 14 Connection Between Pier and Pier Segment

PIER DESIGN

In order to be as competitive as possible with the steel plate girder option, it was desired to use a similar design for the piers and foundation. Therefore, it was hoped to use four columns and 8 ft. drilled shafts. However, during the design process it was seen that the concrete option produced loads that exceeded the capacity of an 8 ft drilled shaft. It was also observed that there was a significant difference in the load carried by the inside columns compared to the outside columns. Therefore, to determine the most cost-effective solution, a RISA model was created to investigate the effects of various design options including the size of the pier cap, the number of columns, the use of a concrete diaphragm as well as possibly post-tensioning the pier cap. It was hoped that a combination of these design options would result in a more equal distribution of loads and allow for the use an 8 ft. drilled shaft. However, after the analysis NDOR chose to select a four column pier with 10 ft drilled shafts for the foundation. The two inside columns will have a narrower spacing to produce a more equal load distribution. The conventionally reinforced pier cap will be 9 ft. wide and 8 ft deep. In addition, a 12 ft. x 1 ft. diaphragm will be used consisting of a steel K-truss encased in concrete. Figure 15 shows the elevation of the pier design.

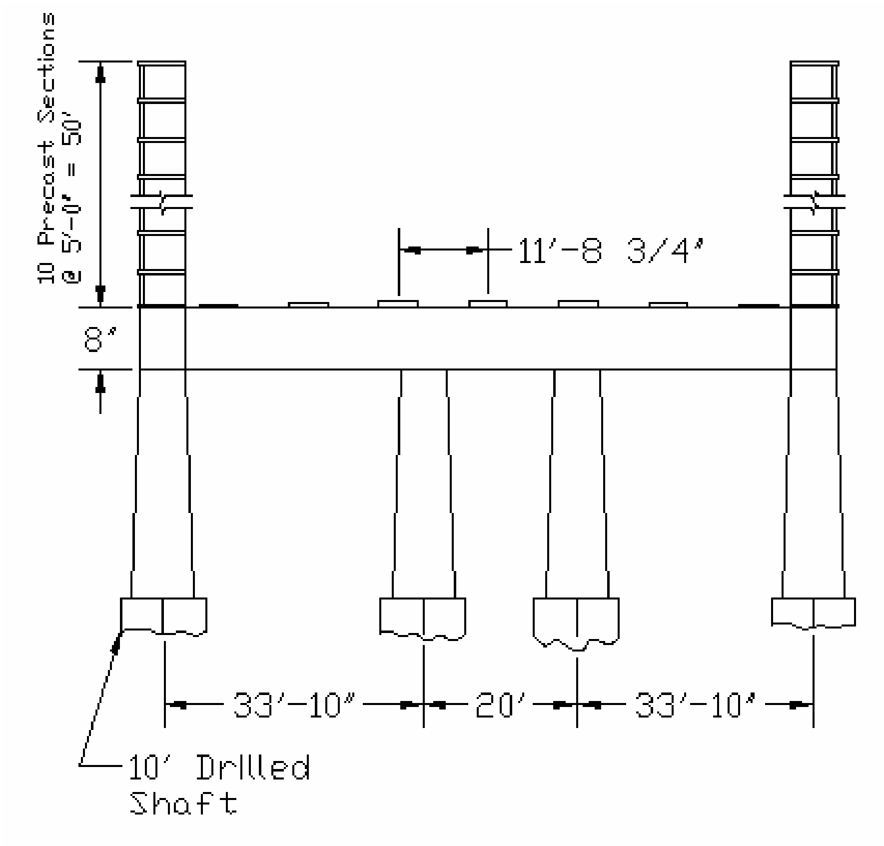

Fig. 15 Pier Elevation

TEST GIRDER

Due to the innovative and unique aspects of the design, particularly for the field connection between the precast segments, it was decided to construct a test girder to assess the constructability and performance of the design. Fabrication of the test girder was a cooperative effort by the Prestressed Concrete Association of Nebraska (PCAN). A 30 ft. girder representing the pier segment was cast along with a 6 ft. girder to represent the midspan segment. The girder had a constant height of 78.7 in., except for near the joint where an increased flange thickness of 4 in. was used. The following are pictures taken of the test specimens.

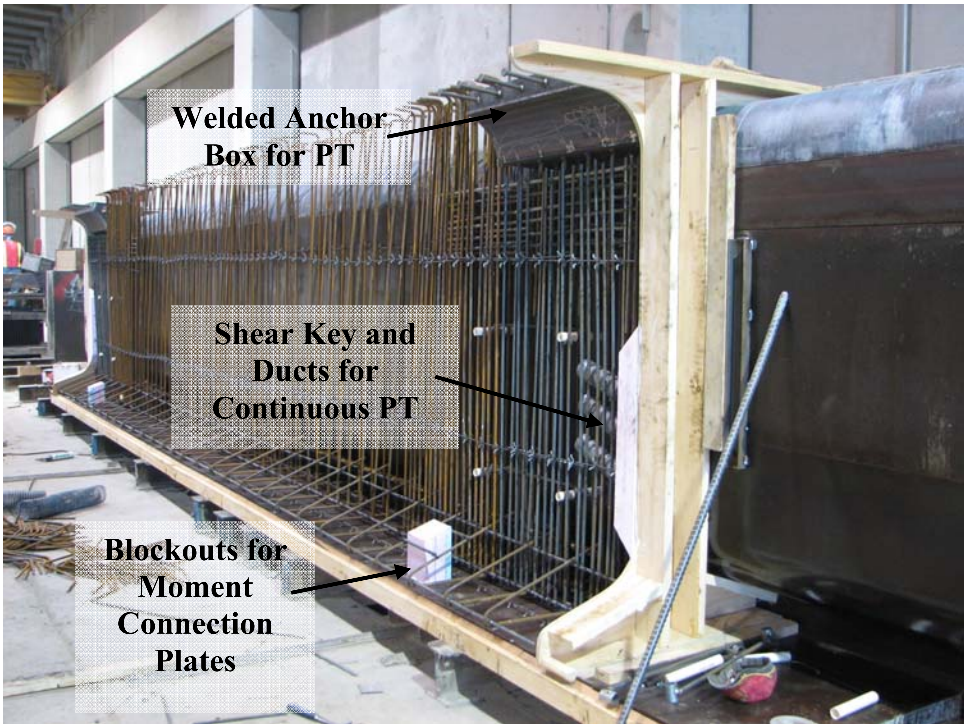

Fig. 16 Test Girder Before Casting

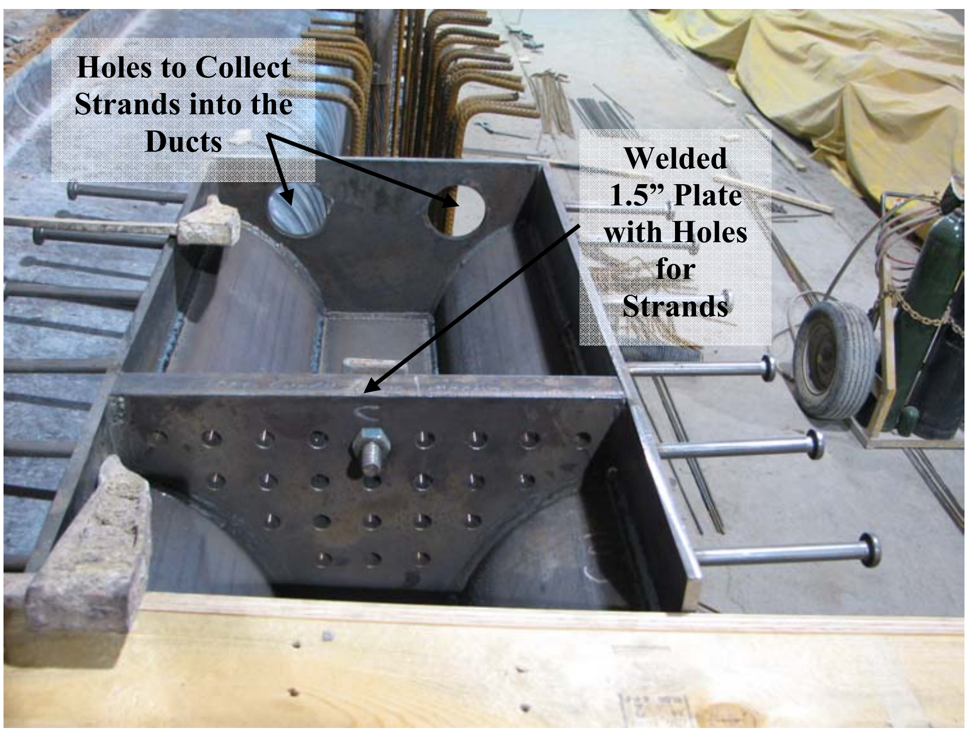

Fig. 17 Welded Anchor Box for Post-Tensioning of Pier Segment

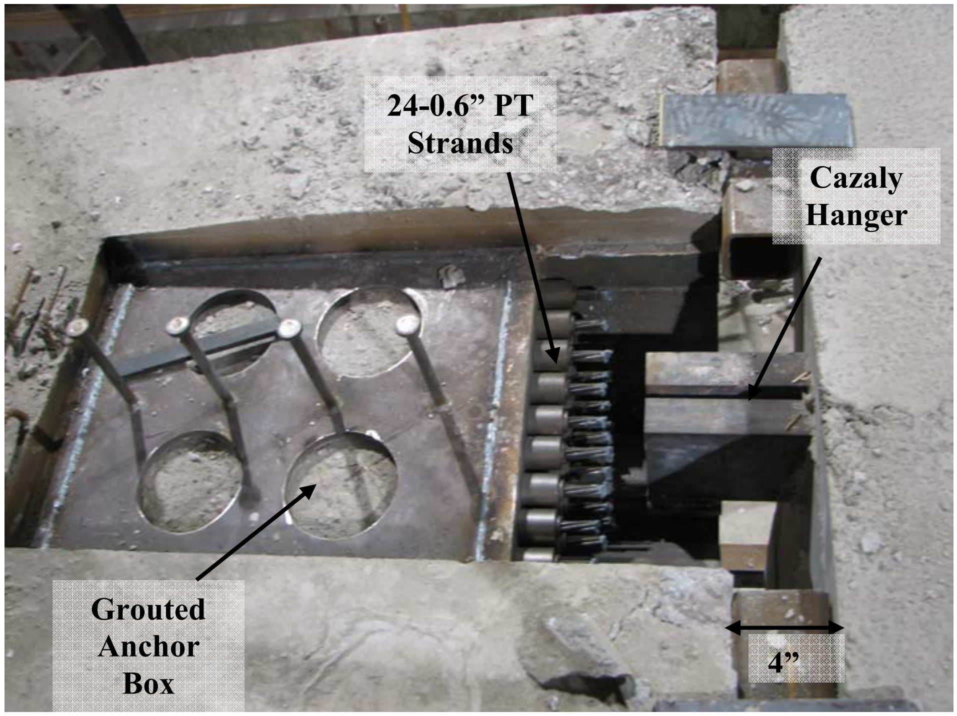

Fig. 18 Joint Connecting Pier Segment and Span Segment after Casting

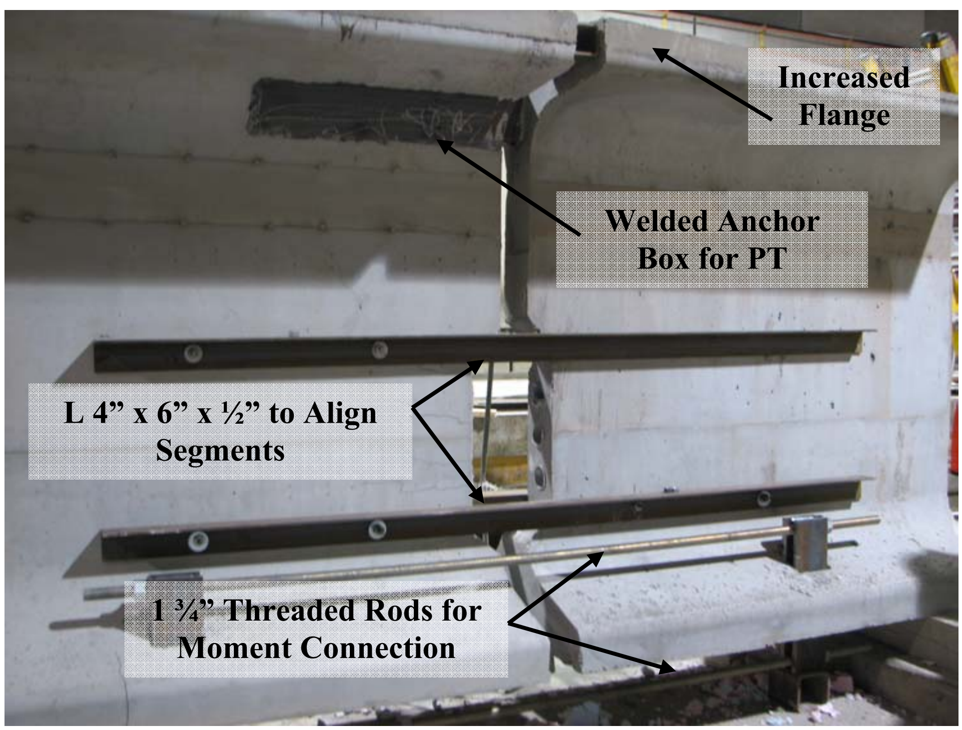

Fig. 19 Moment Connection Between Pier Segment (left) and Span Segment (Right)

CONCLUSIONS

Based on the work completed so far, it is felt by the authors that the segmental precast concrete option for the Yankton bridge offers a cost effective solution. The use of the Cazaly hanger and external moment connection will result in significant savings compared to traditional erection methods such as a structural steel strong back. In addition, the successfulness of the test girder has verified the feasibility and performance of many of the unique aspects of the design. The innovative design would provide designers and contractors with valuable knowledge and experience for future advancements in segmental precast bridges hopefully leading to more bridges of this type.

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