174 FT LONG GIRDER SETTING PRECAST RECORD IN NEBRASKA

Shane A. Hennessey, P.E., Tadros Associates LLC, Omaha, NE
Maher K. Tadros, Ph.D., P.E., University of Nebraska-Lincoln, Omaha, NE
Chuanbing Sun, Ph.D., Janssen & Spaans Engineering, INC., Indianapolis, IN

ABSTRACT

Platte River East Bridge in Nebraska was built using 174-foot precast concrete girders which are the longest girders ever used in Nebraska. The bridge has four girder lines with six spans (two spans at 135-feet and four at 175-feet). This paper discussed the bridge analysis and design, lifting, handling and shipping of the girders, and the bridge construction.

Keywords: High strength concrete; Case study; Deck weight continuity; High performance concrete

INTRODUCTION

Historically, concrete girders have not been able to compete depth for depth and span for span with steel plate girder, except when the concrete girders are post-tensioned. In order to match structure depth, other variables had to be adjusted, such as reducing girder spacing or reducing span lengths. As a result, concrete structures have not been cost competitive with steel in spans longer than about 150 feet. A major advantage steel has had over concrete is its ability to act continuous before placement of the deck slab. Steel structure may result in ease of construction due to its light weight. In addition, Hybrid steel plate girders can use higher strength steel in the high moment regions. Grade 70 weathering steel is increasingly being used in the state for flange plates over the piers. Designing the Platte River East Bridge discussed herein represents an attempt to incorporate advancements in precast/prestressed concrete prismatic beams to make beams continuous for deck weight, without having to resort to post-tensioning or to complicated precast concrete girder geometry.

Precast prestressed concrete I-girder bridges represent about one-third of the bridges built in the United States each year. They are generally constructed as simple span for their weight and the weight of the cast-in-place deck. Cast-in-place diaphragms and reinforcement in the deck render the superstructure continuous for superimposed dead loads and live loads. This system has served very well over the past three decades, especially in cold climate states where expansion joints over the piers create maintenance problems. However, the girders are made continuous for only about one-third of the total load and are thus not fully utilized in the negative moment zones. In addition, some of the bridges built using this type of continuity have experienced cracking due to positive time-dependent restraint moments at the piers, especially in highly prestressed girders. With a new continuity system developed by the local university, the girders are coupled over the pier using four 1-3/8” diameter, Grade 150 ksi threaded rods before the deck weight is applied. The threaded rod system resists the negative moments due to deck weight; therefore, the girders are made continuous for about two-thirds of the total loads. After the deck concrete has hardened, deck reinforcement, along with the high-strength threaded rods, resist the negative moments due to superimposed dead load and live load. Span capacities are improved by about 10 to 15 percent within a given girder size. More importantly, bridge performance is improved as the negative moments due to deck weight more than offset the positive restraint moments due to time-dependent effects. Reduction in positive restraint moments results in less cracking in the pier diaphragms.

SYSTEM SELECTION

Platte River East Bridge has a total length of 970’-0” (Fig. 1) from centerline to centerline of abutments. It is tangent with the bridge centerline (no skew) and has an overall bridge width of 46’-4” (Fig. 2). The roadway consists of a 44’-0” clear roadway and two 1’-2” wide rails. The bridge was designed using AASHTO LRFD Specifications, 2nd Edition.[1] Due to the size and cost of the bridge, Nebraska Department of Roads (NDOR) requested that alternate steel and concrete superstructure designs be completed. Providing alternate concrete and steel superstructure designs has been successful in increasing competition and ultimately reducing overall project cost.

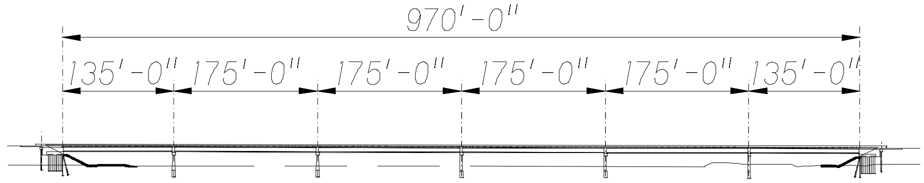


Fig. 1 Bridge Elevation View

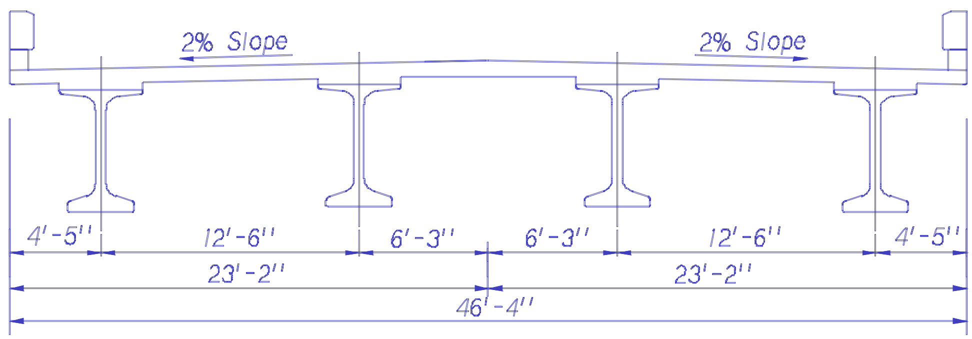


Fig. 2 Cross Section of the Bridge

During the initial system selection, the steel plate girder alternate was analyzed first. The girder depth, spacing and span lengths were determined. The steel hybrid plate girder sections were optimized based on input from the National Steel Bridge Alliance, local steel fabricators, and the state bridge staff. The resulting steel hybrid plate girder system used four girder lines with a spacing of 12'-6" and spans of 135' - 4 @ 175' - 135'. All structural steel for girder flanges, webs, and splice materials were Grade 50W, except for girder flanges over the piers, which used Grade 70W steel. Once the steel bridge system selection was complete, the process of selecting a competitive concrete section began.

The decision was made to match the steel hybrid plate girder structure span for span, depth for depth, and girder spacing for girder spacing. The only way to achieve these goals was to use the longest precast/prestressed concrete girders in the state's history. The girders also used high performance concrete. In addition, the concrete girders needed to be made continuous for deck weight, similar to the steel alternate. This was accomplished using the threaded rod continuity system developed by the local university and first used in the US-30/N-92 Clarks Viaduct (Figs. 3 & 4).

ANALYSIS AND DESIGN

The longest girder previously produced in the state was a 165'-0" NU 2000 I-girder (2000 mm or 78.7 in. deep). The concrete girders used for this bridge are standard NU 1800 I-girders (1800 mm or 70.9 in. deep, see Fig. 5). The longest girders will have a length of 174'-

0". The concrete used in the NU 1800 I-girders have a release strength of 5,500 psi and 56-day strengths of 9,500 psi. Four 1-3/8" diameter Grade 150 high strength threaded rods are embedded in the girder top flange for the threaded rod continuity connection.

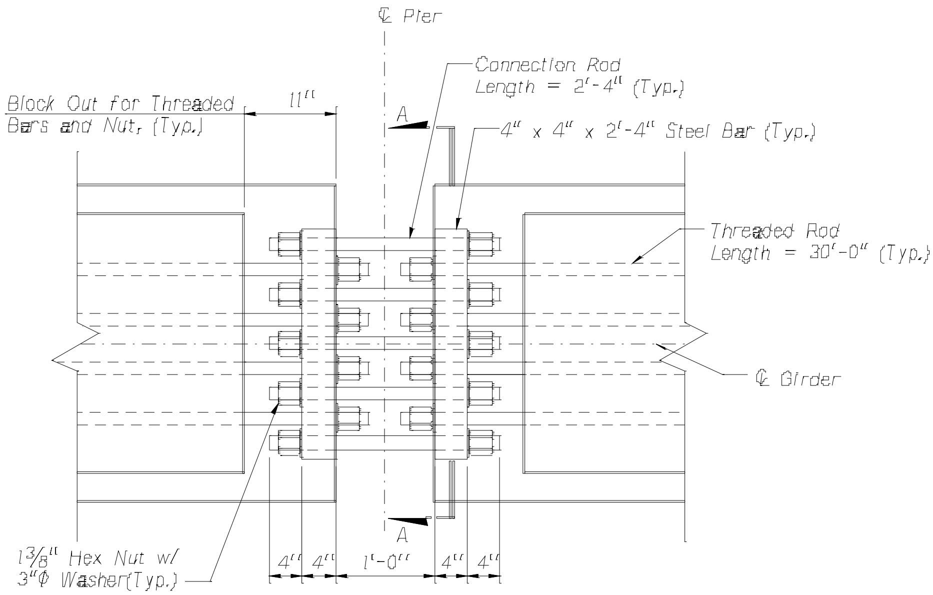


Fig. 3 Plan View of Threaded Rod Continuity Connection at the Piers

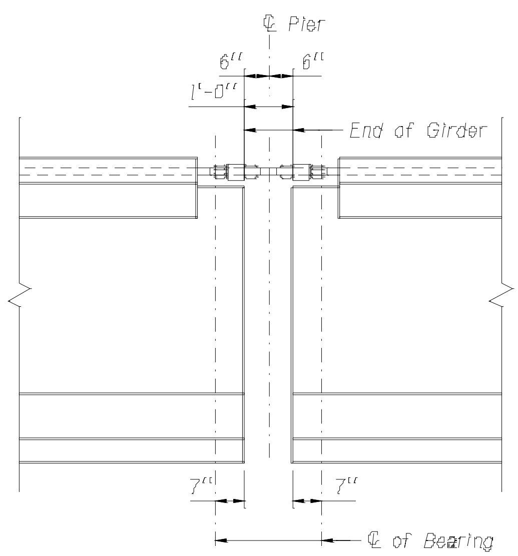


Fig. 4 Elevation View of Threaded Rod Continuity Connection at the Piers

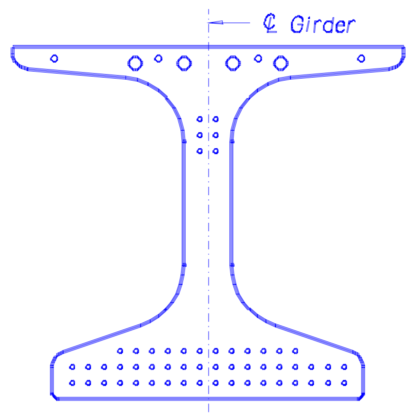


Fig. 5 Cross-Section of NU 1800 (1800 mm High) Girder with four 1-3/8" threaded rods

Due to the complicated design and analysis, typical bridge design software could not be used. Instead, the analysis was performed using Excel spreadsheets and RISA-2d. LEAP Software's CONSPAN LA was used to obtain camber data. The first step in girder design was to determine the number of pretensioned strands required based on the allowable tension stress at final. A spreadsheet was written that allowed the designers to quickly change the number of strands and see how it affected the girder tension stress. The spreadsheet incorporated AASHTO LRFD lump sum prestressed losses and transformed section analysis for stress computations. Once the number of strands was determined based on tension stresses, the concrete release strength was determined using strength design. The state has recently eliminated the allowable compressive stress check and instead adopted a strength design approach that utilizes strain compatibility at prestress release.

Once the number of strands was set for each span and the concrete release stress determined, the ultimate flexural capacity was determined using Excel spreadsheets. The spreadsheets were used for both ultimate positive and negative moment capacities. The spreadsheets utilized strain compatibility and Mast's Unified Approach.[3] Strain compatibility was used in order to account for the high strength threaded rods in addition to the mild reinforcement in the deck slab. Strain compatibility also allows for inclusion of the compression steel. The analysis indicated that the compression stress limits at the pier sections were exceeded but that strength (using Mast's Unified Approach) was acceptable. The analysis and design resulted in 42 - 0.6" diameter, Grade 270, low relaxation strands. Six of the strands were draped (2 strands in each of three rows).

Another unique feature of this bridge is the detailing of the pier diaphragms. Since the concrete girders are made continuous for deck weight, a partial height diaphragm must be in place prior to casting the deck slab in order to transfer the compression force from girder to girder. The tension force is transferred through the threaded rods in the top of the girder. The negative moments developed over the piers in the bridge were very large (14,000 kip-ft / girder line). The girders are acceptable according to strength design but required a girder concrete strength is 9,500 psi. The pier diaphragm concrete strength is typically 4,000 psi (same as the deck slab). In order to avoid having a large difference in concrete strength, the concrete strength of the diaphragm was increased to 6,000 psi. In addition, three #5 rectangular ties, about 2 ft tall, were placed between the girder bottom flanges to confine the compression strut that develops between the two adjacent girder bottom flanges (Figs. 6 & 7).

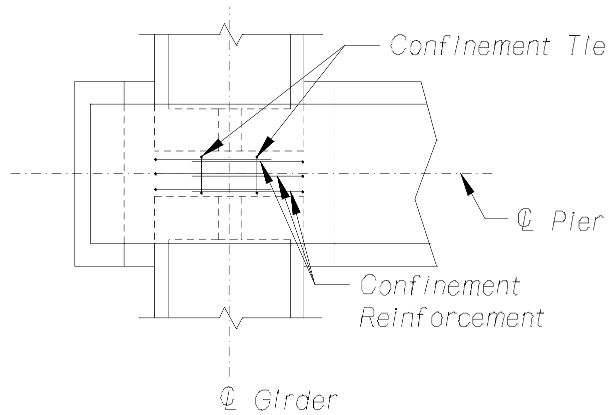


Fig. 6 Plan View Showing Confinement Reinforcement in Pier Diaphragms

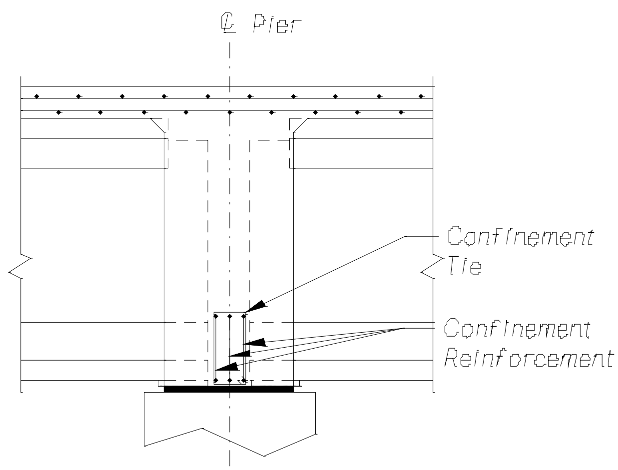


Fig. 7 Elevation View Showing Confinement Reinforcement in Pier Diaphragms

In addition to having alternate steel and concrete superstructures, the bridge also has alternate substructures. Both structures were designed with encased wall piers, but the deep foundation elements are either driven steel piling or drilled shaft with rock sockets. For this bridge, the original design consists of four 5'-6" diameter drilled shafts with 5'-0" rock sockets. The rock socket is drilled 34'-0" deep into the natural shale. The four shafts are spaced at 14'-0" on center. The alternate foundation consists of 15 - HP14x89 driven steel piling with centerline spacing of just over 3'-0" (Fig. 8). The piers do not include a pile or drilled shaft cap on the bottom of the wall section. The piling extends into the wall for the alternate and the wall sits on top of the drilled shafts for the original design.

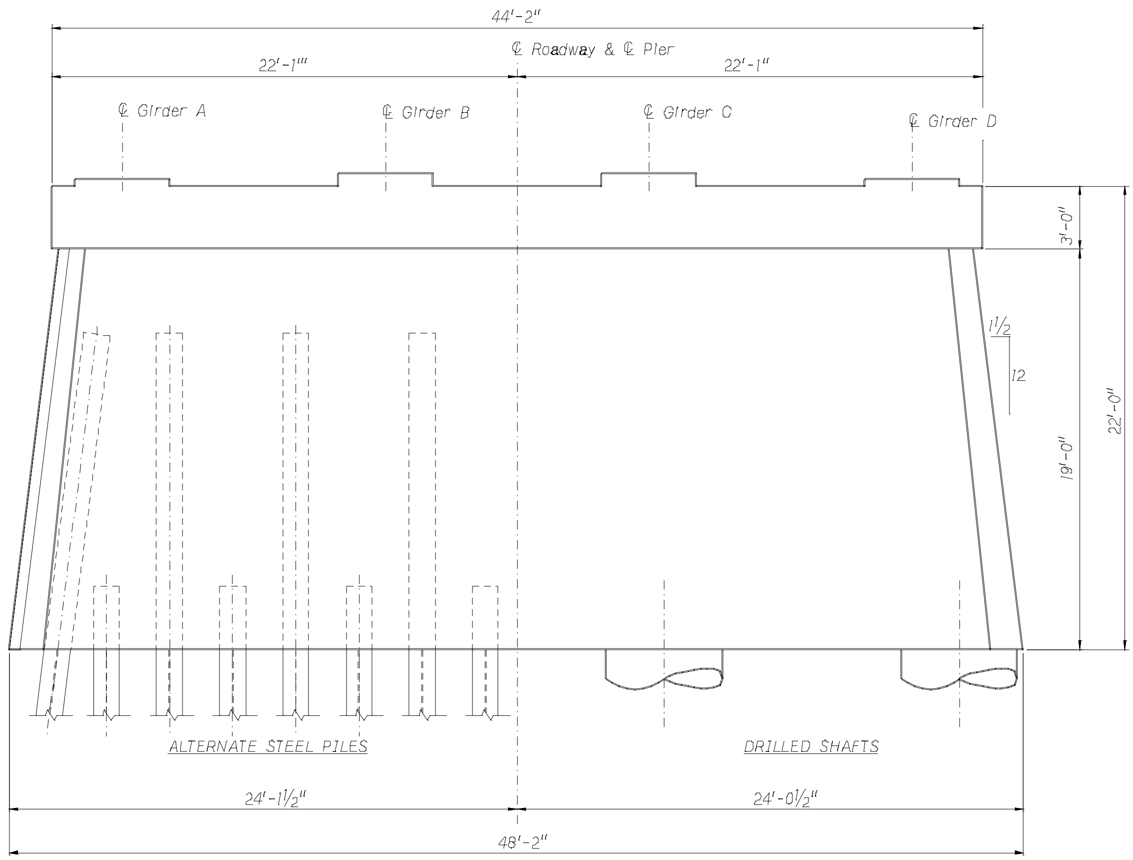


Fig. 8 Pier Elevation Showing Alternate Deep Foundation Elements

GIRDER PRODUCTION

Due to the significant girder length, it was found to be critical to control the girder sweep as required. It is specified in the PCI Bridge Design Manual that the girder sweep can not exceed 1/8" per 10' of the girder length. The obvious methods to control straightness of the girder are to assure that the forms are installed straight and that the prestressing strands are properly installed. Also, it is important that precast members are stored plumb. During restoration, the girder sweep should be monitored frequently as needed. If excessive sweep is observed, it can be corrected by leaning the members in the direction opposite to the sweep during storage.

Excessive girder sweep creates torsional effects within the girders under various loadings, particularly during deck placement. Once the deck is in place, it helps to transfer load to adjacent beams, thus limiting torsion in the beams. However, the deck does not provide adequate stiffness to prevent the girder bottom flange from straightening out when the girder deflects vertically. Analysis may be performed to determine the need of additional torsional reinforcement in the girder due to deck placement.

GIRDER LIFTING, HANDLING AND SHIPPING

As the longest girder ever produced in the state, the stability of the 174 ft-long girder was investigated during lifting, handling and shipping. Girder lifting generally appears to be more critical than shipping since the girder is hanged in the air and it is inclined to roll about the axis through the lifting points. The current AASHTO Bridge Design Specifications do not have specific provisions on how to perform the slender girder stability analysis. A valuable reference is the Bow River Bridge in Alberta, Canada, which includes 211 ft-long NU I-girders. A steel truss was provided at the girder top flange to achieve the stability during lifting and shipping. Considering the extra cost related to the steel truss, a stability analysis was performed for this bridge to determine the need of a truss. The analysis refers to the special publication by Mast at the PCI JOURNAL. The simplest method to increase the lateral stability is to move the lifting devices in from the girder ends. However, long overhang results in high negative moment, which accordingly increases the required concrete strength at release. Another effective way of contributing to the girder stability during lifting is to heighten the lifting points above the girder as much as possible. As a result, a unique lifting device was developed for this bridge (see Fig. 10). It consists of two 8’-0” (length) x 6” (width) x 5/8” (thick) vertical steel plates and one 1’-6” x 2 11/16” x 1/2” plate welded between the vertical plates above the girder top flange. As noted in Fig. 10, one or two vertical shear wires (shear welded wire reinforcement) may be removed to accommodate the vertical steel plates. At the bottom of the vertical plates, 3-#8 bars run through the plates to hold them with the girder. A 3” diameter hole is drilled near the top of each vertical plate for lifting. The center of lifting devices is 16 ft away from the girder end. Fig. 11 shows the lifting device prior to the concrete placement and Fig. 12 illustrates the girder being stored in the precast yard.

Precast girders were shipped to the bridge site by steerable trailers. Prior to shipping, the required rotational stiffness of the vehicle was determined following Mast’s method. Meanwhile, the following steps were recommended to the shipping company on how to measure the rotational stiffness of the vehicle:

1) Choose a concrete girder, preferably weighing over 100 kips (100, 000 lbs). As shown in Fig. 14, “W” represents the beam weight.
2) Place the concrete girder on the vehicle as shown in Fig. 14. Note that the centerline of the concrete girder is aligned with the centerline of the front jeep and it offsets the centerline of the rear jeep by a distance “e”. Secure the girder end at the front jeep. Choose a maximum eccentricity “e” without causing the girder to roll over or damage the girder or the equipment. For example, use “e” = 12 in. The crane may be hooked to the girder for convenience and safety consideration until Step 6 is completed.
3) Measure the deflection shown as “Δ” and the distance “L” in Fig. 14 at the rear jeep.
4) Determine the rotational angle, $\theta = \frac{\Delta}{L}$ (“Δ” and “L” in inches, “θ” in radian)

5) Determine the rotational stiffness of the vehicle (including both front and rear jeeps) using the following formula: $K_{\theta} = \frac{We}{\theta}$ ("W" in kips, "e" in inch, "K_{θ}" in kip-in per radian)
6) Repeat Step 2 to 5 by moving the girder end at the rear jeep only to achieve a negative eccentricity "e", which means the centerline of the girder is still aligned with the front jeep, but it offsets the centerline of the rear jeep by "e" at the opposite direction as shown in Fig. 14, which allows for a double check for this measurement.
7) Determine the average rotational stiffness obtained from Steps 5 and 6.

As a result, it was found that the measured vehicle rotational stiffness is significantly higher than what is required. All girders were shipped without any problems (see Fig. 15).

Fig. 9 Shipping a 211 ft-long girder in Bow River Bridge

BRIDGE CONSTRUCTION

Once all girders are installed over the substructure, the threaded rods projecting outside of the girder ends can be connected (see Figs. 16 and 17). The bottom of the pier diaphragm is poured prior to the deck placement (see Fig. 18). The bridge was completed late 2005 (see Fig. 19).

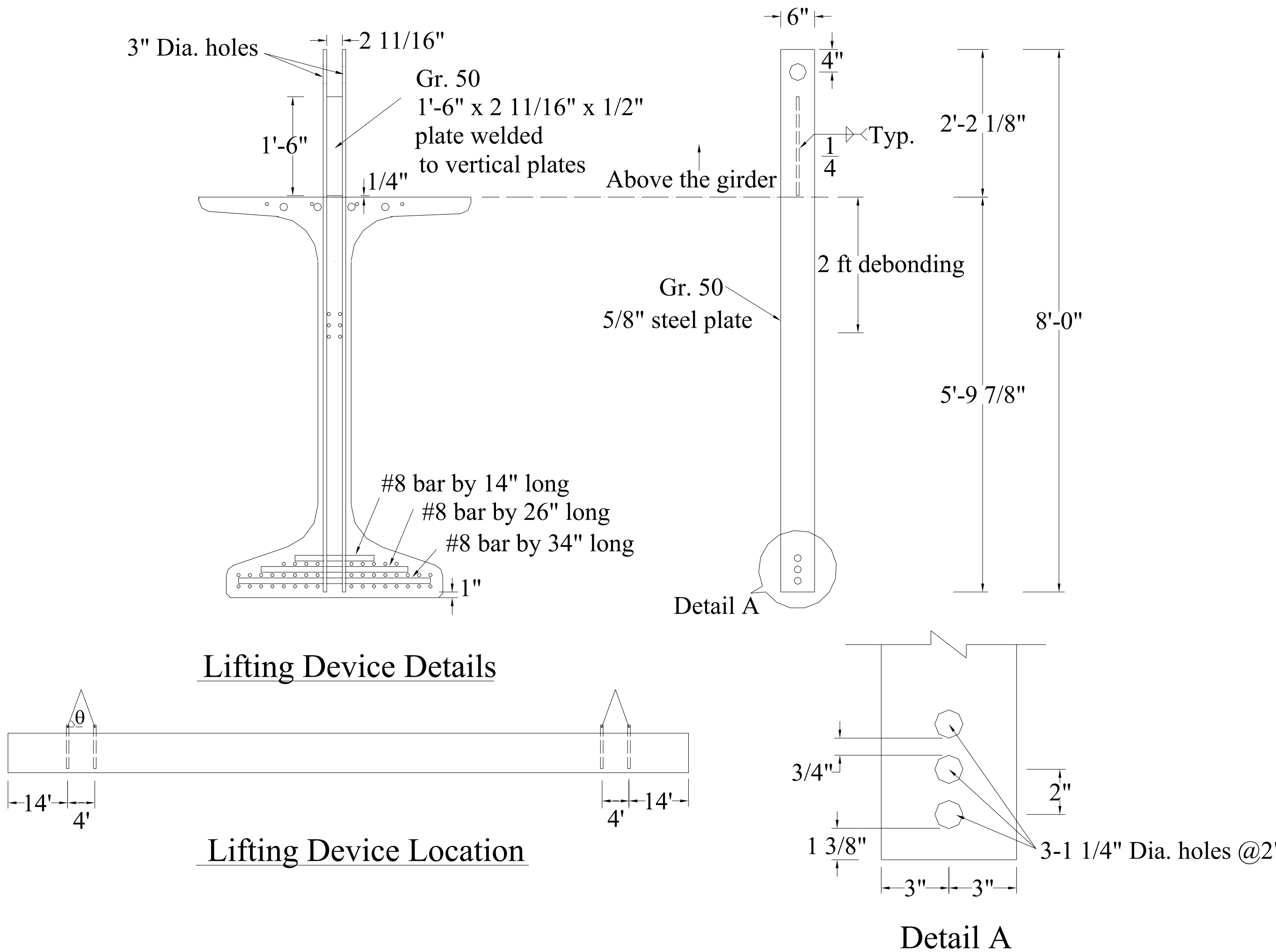
CONCLUSIONS

This bridge design sets a record for the longest precast/prestressed concrete girders used in the state to date. In the past, when alternate concrete and steel superstructure designs were performed on a structure, the steel alternate often has fewer spans or wider girder spacing than the concrete. With the concrete alternate design on this project, it seems that designers can first optimize a steel design and then match spans, depth, and girder spacing with a concrete design.

This bridge uses a unique threaded rod continuity connection which was firstly used on the US-30/N-92 Clarks Viaduct. One of the main benefits of the threaded rod continuity connection is its simplicity. The connection creates an inexpensive and simple way to splice concrete girders, making them continuous for deck weight. In addition, an innovative lifting device has been developed for handling long precast girders over 170 ft.

REFERENCES

1. AASHTO, LRFD Bridge Design Specifications, Third Edition and Interims 2004 and 2005, American Association for State Highway and Transportation Officials, Washington, D.C.
2. Bridge Office Policies and Procedures (BOPP) Manual, Nebraska Department of Roads (NDOR), Lincoln, NE, 2001.
3. PCI Bridge Design Manual, Precast/Prestressed Concrete Institute, Chicago, IL, 1997



Notes:

1. It may require that one or two vertical shear wires be removed to place the lifting inserts.
2. Apply form oil or equivalent to debond the vertical plates as shown above.
3. Angle θ shoud be larger than 38 degree for handling.

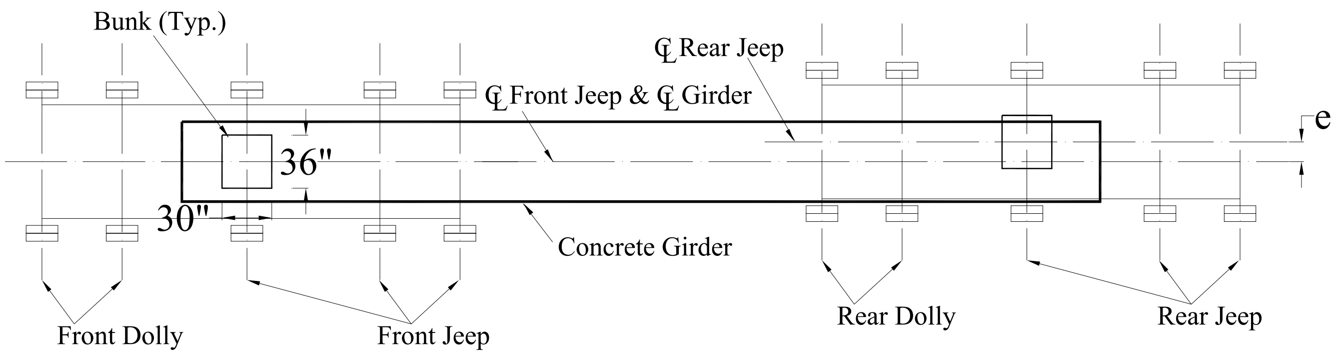
Fig. 10 Lifting Device Details for the 174 ft-long Precast Girder


Fig. 11 (a) Girder Lifting Device Elevation

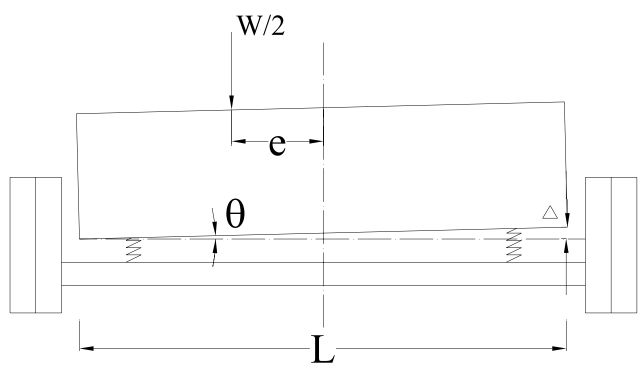

Fig. 11 (b) Lifting Device Closer View


Fig. 12 Girder Stored in the Precast Yard

Fig. 13 Girder Handling in the Precast Yard



(a) Plan View



(b) Cross Section

Fig. 14 Measurement of Vehicle Rotational Stiffness

Fig. 15 Girder Shipping to the Bridge Site

Fig. 16 (a) Girder Installation

Fig. 16 (b) Girder Installation

Fig. 16 (c) Girder Installation

Fig. 17 Threaded Rod Continuity Details over the Pier


Fig. 18 Deck Placement


Fig. 19 Completed Bridge