VALUE ENGINEERING ARBOR ROAD BRIDGE USING THE CURVED PRECAST CONCRETE U-GIRDERS

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ABSTRACT

Arbor Road Bridge over the highway I-80 is a horizontally curved two-span bridge (141'-10" + 135'-10") with a bridge width of 38'-4". The original design utilized the steel plate I-section girders. Due to the current high cost of steel materials, a curved concrete U-girder solution was proposed. The concrete bridge presents a total of four girder lines spaced at 9'-4" instead of the original five girder lines at 8'-0". Three stages of post-tensioning are included to carry the girder self-weight, the deck weight, and the superimposed loads, respectively. Conventionally reinforced precast deck panels with 1 ½" concrete overlay may allow to significantly cut the construction time and to reduce the inconvenience to the traveling public. Moreover, the precast deck panel system presents a unique approach to eliminate any necessary top bracing during deck placement. This paper discusses the project background, addresses the issues regarding the bridge analysis, design as well as the construction sequence, and makes a cost comparison between the steel bridge and the concrete alternate. This project is expected to be completed by the end of 2005.

Keywords:
Concrete Curved Bridge, Precast Deck Panel, Value Engineering.

INTRODUCTION

Precast curved bridge girders are usually made as a series of short straight segments, or chords, to approximate the theoretical arc. The simplest and most economical way to support a curved roadway is to use straight girders beneath a curved deck. When the offset between chord and arc is too large, however, the appearance may be unacceptable. If so, each individual girder can be made up with a number of chords to achieve the curved bridge geometry. Forms for the girders are made in straight segments with a small angle at the form joint.

Arbor Road Bridge is an overpass across the highway interstate I-80 in Lincoln, Nebraska. It has two spans, about 142 ft and 136 ft (see Fig. 1). As a horizontally curved bridge, it has a central angle of around 3 degrees. The overall bridge width is 38'-4". The bridge has a skew of 31 degrees. The original design used the steel plate I-section girders with a 7 ½" cast-in-place (CIP) deck slab. There are a total of 5 girder lines at a spacing of 8 ft (see Fig. 2). The steel girder section is 43 ½" deep at midspan and is increased to 67 ½" over pier. The intermediate diaphragms space at about 18.5 ft (see Fig. 3).

VALUE ENGINEERING

Due to the current high cost of steel material, a value engineering proposal with the use of precast concrete girders was submitted to the state by Tadros Associates and the awarded contractor. During the early stage of the value engineering, the standard NU (Nebraska University) I-girders were proposed to replace the steel plate girders. As shown in the girder layout (see Fig. 4), five short straight segments can be introduced to each span to achieve a desirable appearance.

Even though the NU I-girder option is technically feasible and believed to be cost-effective, only one local precast producer was available to bid the girders. To allow the suppliers from other states to participate in the bidding, the U-section concrete girder option was used instead in the final design.

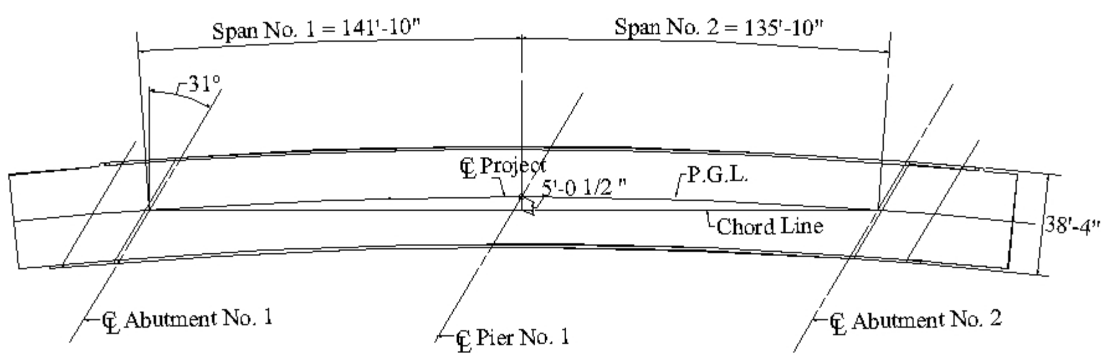
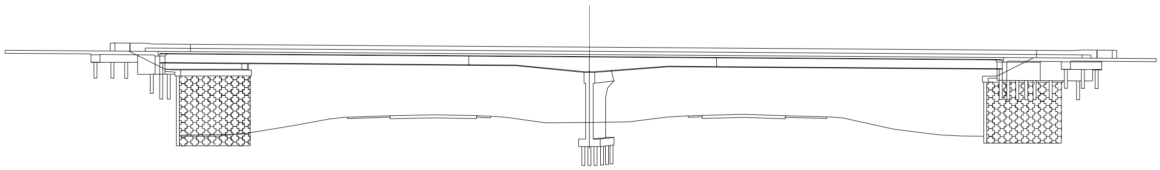
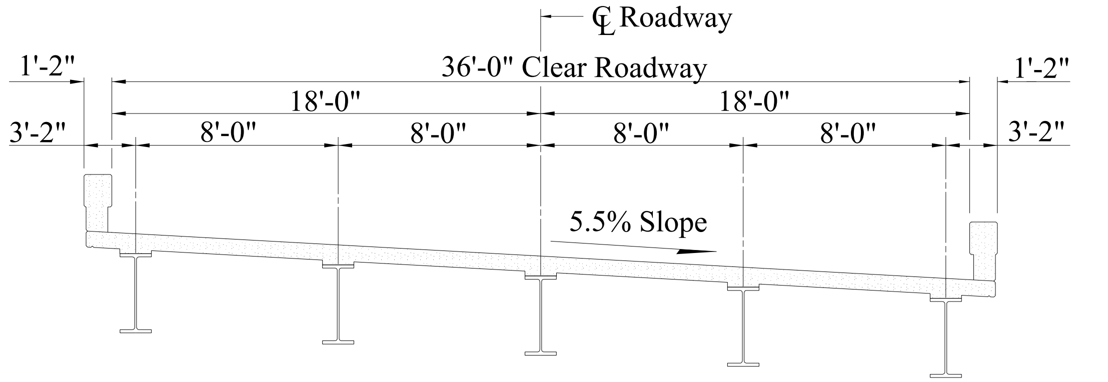


Fig. 1-Arbor Road Bridge General Plan View

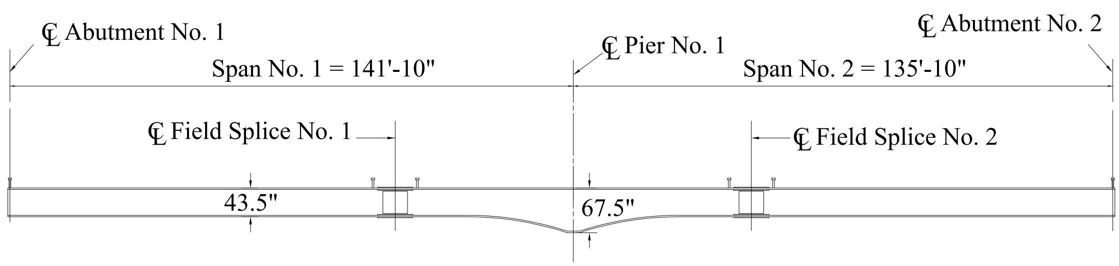


(a) Elevation

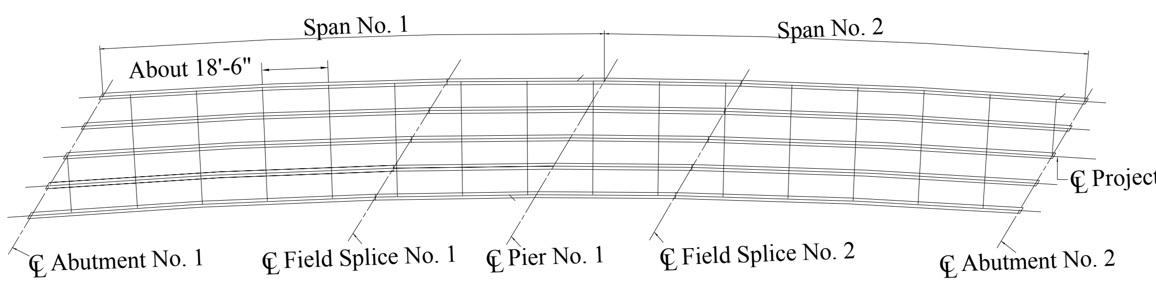


(b) Cross Section

Fig. 2-Arbor Road Bridge Elevation and Cross Section using the Steel Girders



(a) Steel Girder Elevation



(b) Steel Girder Layout

Fig. 3-Steel Plate Girders Elevation and Layout

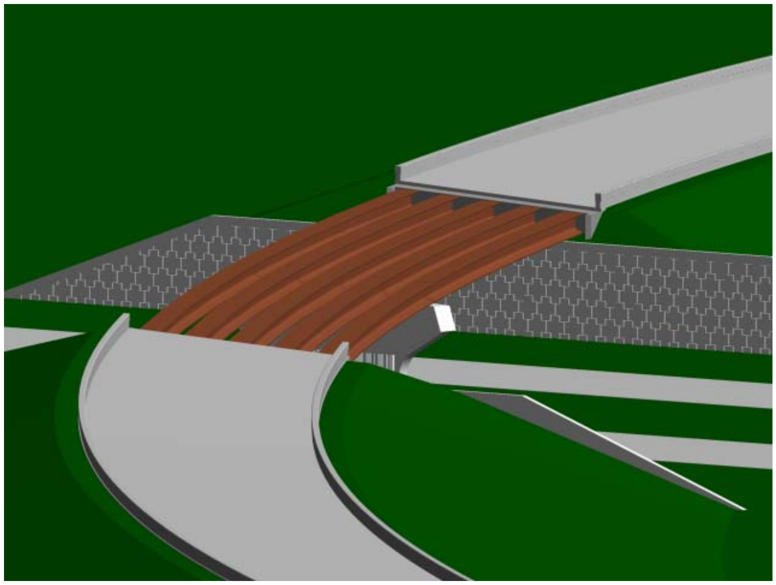


Fig. 4-Preliminary Design using the Concrete NU (Nebraska University) I-girders

PRECAST CONCRETE U-GIRDERS

Due to the time restraints, it is intended to maintain the substructure geometry the same as that in the original steel design. The increased weight of concrete superstructure can be accommodated by adding several piles. The concrete superstructure includes four girder lines spaced at 9'-4" instead of five girder lines in the steel design (see Fig. 5). The prismatic U-section is 45.8" tall at the left web and reduced to 43.5" at the right following a 5.5% cross slope. Note that 6" thick precast deck panels with 1 ½" concrete overlay are used in the design, which will be discussed below.

The precast U-girder at each span is made up with four straight segments, three of which are 40 ft long and the fourth segment over pier varies in length for different girders (see Fig. 6-(a)). Internal concrete diaphragms are provided at the joint between the adjacent segments. They are about 5 ½ in. wide and allows for a small angle change at the joint. Each girder also has an internal concrete diaphragm over the pier which provides the bearing for the external post-tensioning, as discussed shortly in this paper. At the abutment end, each girder has a solid block to accommodate the post-tensioning anchorage hardware. The U-girder bottom flange is typically 5" thick along the spans and is thickened at the pier end to meet the flexural strength requirements. The web thickness is 7 ½" at the top and increased to 8" at the bottom. As shown in the girder layout, the intermediate steel diaphragms are spaced at about 33 ft (see Fig. 6-(b)).

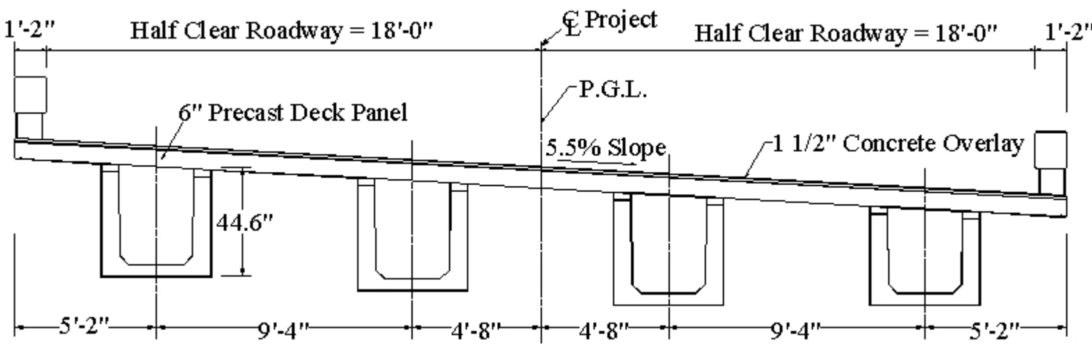
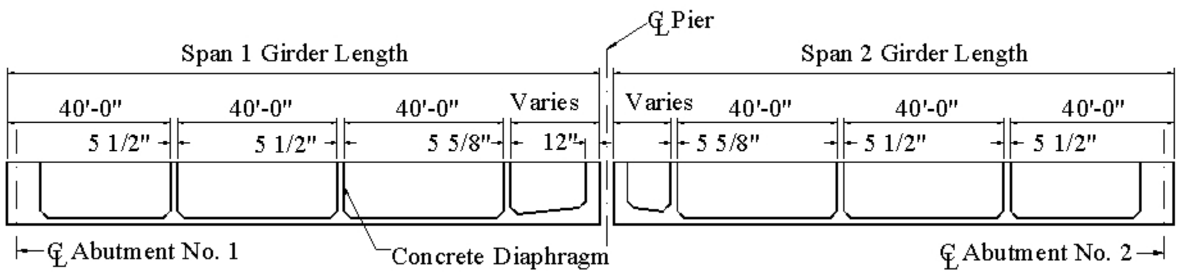
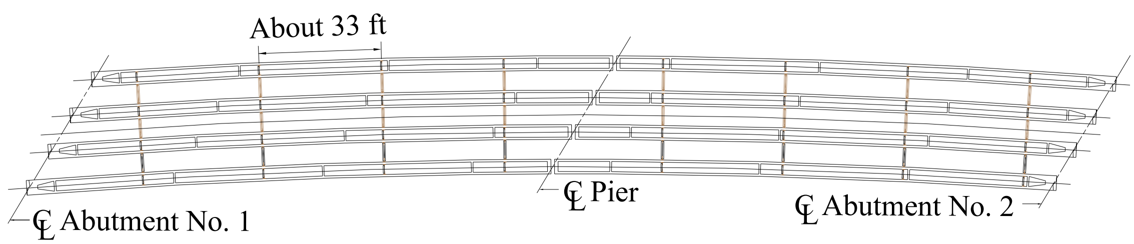


Fig. 5-Bridge Cross Section using the U-section Girders



(a) U-section Girder Elevation



(b) U-section Girder Layout
Fig. 6-U-section Girder Elevation and Layout

GIRDER ANALYSIS

The bridge was designed following the AASHTO Standard Bridge Design Specifications. Since the central angle for each span is only around 3 degrees, the bending moment and shear force in the girders were calculated using the LEAP software, CONSPAN, considering the longest girder line. The bridge was designed to carry the HS25 live load. The torsional moment in the girders and the reactions of the intermediate diaphragms was determined through the grid analysis by the program RISA 3D.

The grid analysis model includes the precast girders, the deck slab along the bridge transverse direction, and the intermediate steel diaphragms (see Fig. 7). The bridge was modeled as fixed for torsion at the abutment and free for torsion over pier. The girders are seated over the bearing pads without being fixed to the pier so that no torsional moment is transferred to it. In this model, several loading cases are applied, including the deck weight, the post-tensioning, and the superimposed loads, to get the reactions in the girder and the intermediate diaphragms. The bending moment and shear force from the grid analysis are very close to the results obtained from the CONSPAN.

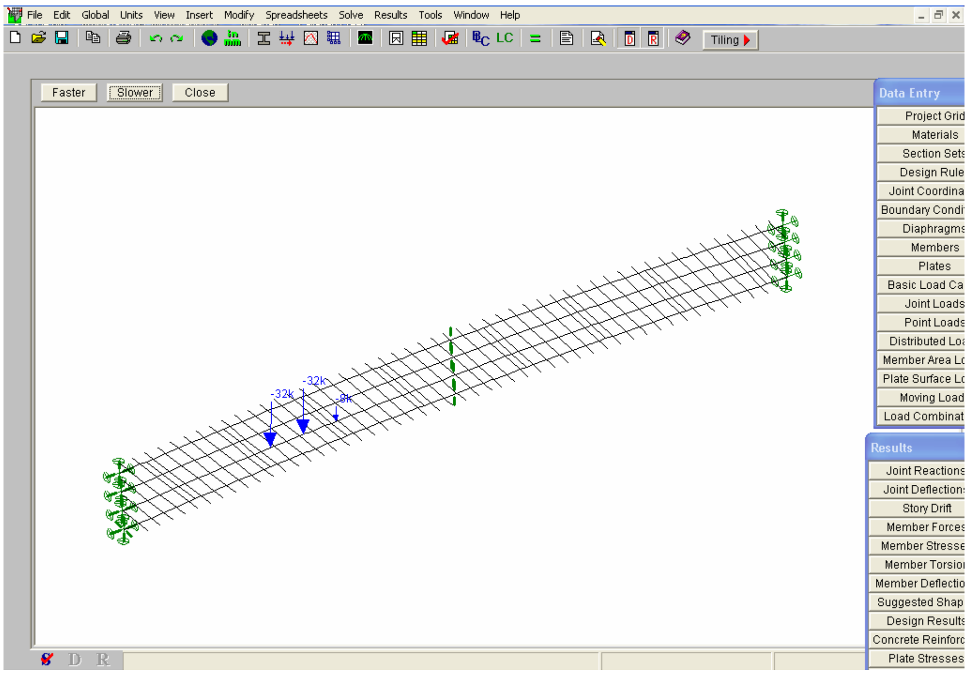


Fig. 7- Grid Analysis Model

GIRDER DESIGN

The working stress and the flexural strength design were performed to determine the amount of required post-tensioning (P/T) strands/tendons. The girder flexural strength was determined through the strain compatibility approach described in the PCI Bridge Design Manual[1]. As a result, three stages of post-tensioning are included and the post-tensioning steel includes 36-0.6" diameter, Grade 270 ksi unbonded mono-strands (stage I P/T, see Fig. 8) and 4 tendons with 15-0.6" diameter, Grade 270 ksi strands at each tendon (stage II & III P/T, see Fig. 9). The 36 unbonded strands are included to carry the girder self-weight and allow for an acceptable camber. Four post-tensioning tendons are used for the deck weight and the superimposed loads. The second stage of post-tensioning is applied before the deck panels are erected. After the installation of the panels and placement of concrete overlay, the third stage post-tensioning, which applies to both the girder and the deck sections, is provided to carry the superimposed loads,. The concrete strength at 28 days is 10,000 psi.

The strength design method was utilized to determine the required concrete strength at release, which is controlled by the loading case at girder lifting. By assuming 4 lifting points along each girder web, i.e., 8 points for each girder, 4-0.6" diameter, Grade 270 ksi external post-tensioning strands are included at the section top to achieve an acceptable concrete strength at release, which is 5,600 psi. Due to the curvature of the girder, the center of lifting points per girder side is about 30 ft away from the girder end so that the centerline of lifting points can pass through the c.g. of the girder.

The girder camber and deflection were calculated considering the creep effect using a construction schedule by the contractor. With the given bridge profile, it is a concern whether the minimum clearance requirement underneath the overpass, i.e., 16 ft, can be met. Since the camber and deflection calculations of post-tensioned concrete girders may be impacted by a number of uncertain factors, the actual camber could vary significantly from what is predicated. Several methods were considered to increase the camber and ensure the minimum clearance criterion will be met. One way is to release the external top strands when the girders are stored in the precast yard, and only being re-tensioned for girder handling and shipping, which eliminates the portion of downward deflection of the top strands due to the creep effect. Also, it was decided to shallow the lowest girder (exterior girder) by 4" so that the minimum clearance may not be a potential problem.

The vertical shear and the torsion effect were combined for consideration to determine the required shear reinforcement following the AASHTO LRFD Bridge Design Specifications. Welded wire meshes are used as the vertical shear reinforcement, including D31 mesh at the abutment and D20 at the remaining locations, shown as WWF1 to WWF5 in Fig. 10. Welded wire mesh 6 and 7 are placed at the girder bottom flange for torsional resistance. Welded wire mesh 8 at the girder ends is designed to take the bursting force due to 36 unbonded P/T strands. Fig. 11 shows the post-tensioning anchorage details at the abutment. Also shown are the spirals which confine the concrete to achieve adequate bearing strength for the 36 unbonded strands. Steel mesh WBC1 is given to resist the bursting force due to the P/T tendons.

At the internal concrete diaphragm, Fig. 12 shows the post-tensioning ducts, PVC sleeves allowing the top external strands to pass through, and the U-bars inside of the girder webs to resist the pullout force due to the external strands (see Fig. 12). In addition, the concrete diaphragm is reinforced by a steel mesh of D20 by D20.

As mentioned above, the bridge is designed to be free for torsion at the pier. When the girders are installed, they are seated over temporary steel beams, which are anchored to the pier by high strength threaded rods. After the second post-tensioning is applied, the temporary anchoring devices will be removed. No pier diaphragms are included for ease of construction.

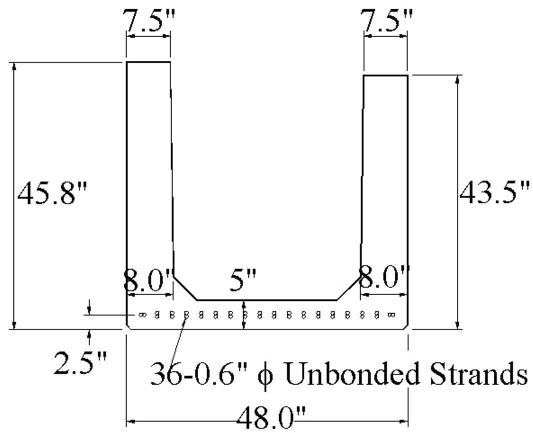


Fig. 8-Concrete Girder Section with Unbonded Strands

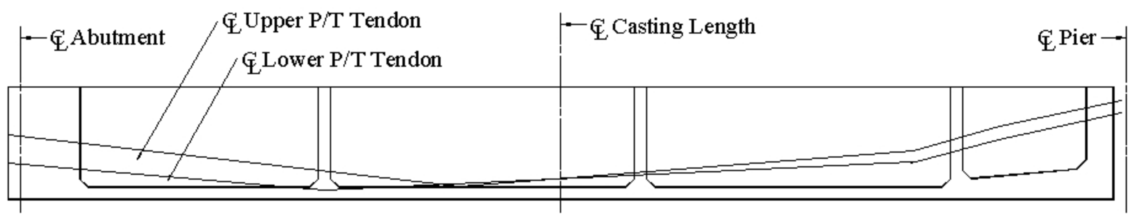


Fig. 9-Post-tensioning Tendons Layout

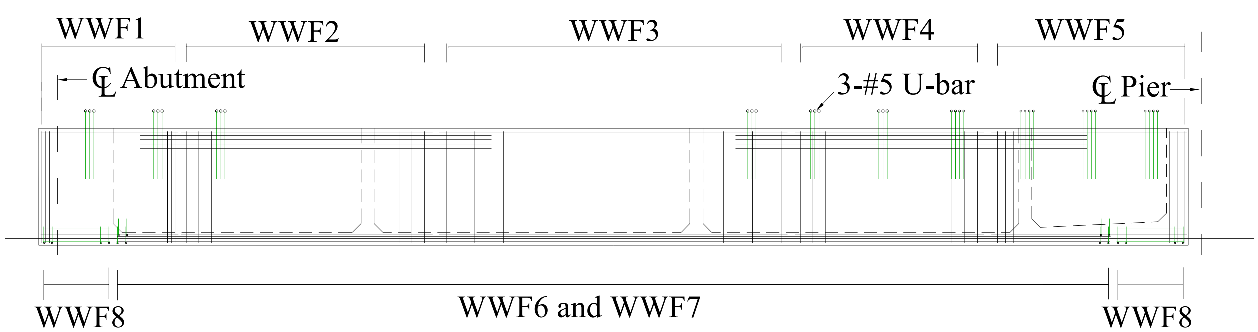


Fig. 10-Concrete Girder Conventional Reinforcement

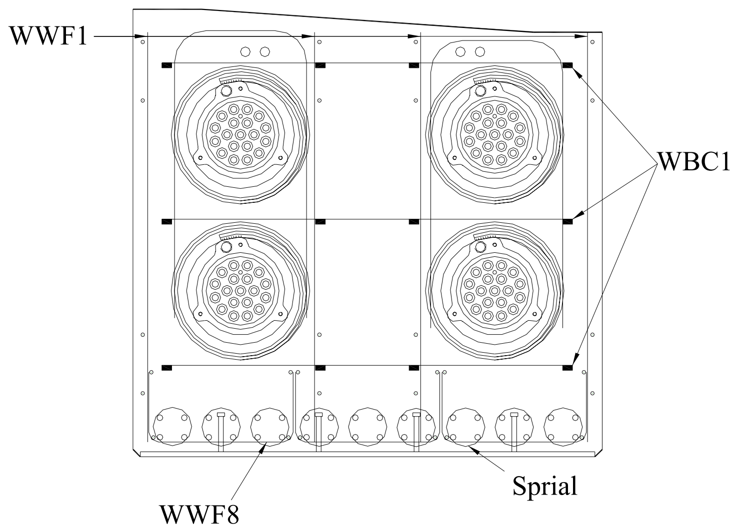


Fig. 11-Post-tensioning Anchorage Details

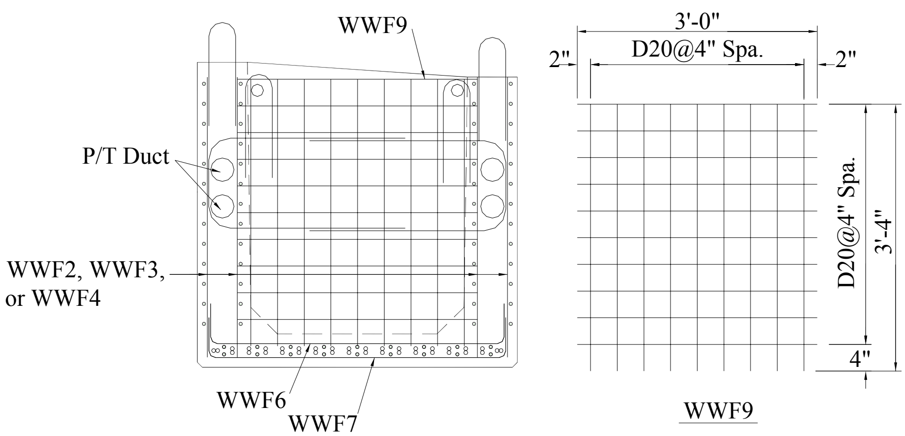


Fig. 12-Concrete Diaphragm Reinforcement

PRECAST DECK PANEL

Instead of using the CIP deck slab, 6"-thick precast deck panels are included in the design with 1 1/2" concrete overlay. Compared with the conventional CIP deck slab, the precast deck panels are normally produced under a better quality control. Also, most of the shrinkage, temperature drop due to the cement hydration cycle, and the creep has occurred before the precast panels are made composite with the girders. Therefore, the precast deck panels are expected to last as long as the precast girders. Additionally, it allows for fast construction. The primary reason to use the precast panels in this project instead is to eliminate any necessary lateral top bracing in the girders during deck placement. For curved steel bridge construction, the lateral top bracing, such as Z-bracing truss, are mostly included along the spans to avoid any local bucking or failure during the deck placement. Even though the concrete girder is much stiffer than the steel plate girder, the stability during the deck placement is still a concern, especially for the open girder sections with thin webs as in this project. After the deck slab hardens, the torsional stiffness of the section is dramatically increased and, therefore, becomes significantly stable for torsional effect. Since the top bracing truss is included only for a temporary loading due to the deck weight, the precast deck panels can be utilized such that they can be made composite with the girders soon after the installation.

This bridge includes a total of 30 precast panels, 25 of which are typical panels as shown in the layout (see Fig. 13). The typical panels are 38’-4” long (full-bridge width), about 10 ft wide and 6” thick. 3-#5 U-bars at each girder web are provided at about 2 ft spacing for the composite action between the panels and the girders. In each panel, there are pockets spaced at approximate 2 ft along the girder line. The pockets are 10” diameter at the bottom and tapered to 9” at the top. They are grouted with non-shrinkage grout immediately after each panel is installed. After the pockets gain a strength of 2,000 psi, the next panel is allowed to be placed. The panels are installed individually starting from the midspan to the abutment/pier. The panel at the midspan is erected and grouted first to achieve the composite action at this critical location.

A cross section of the precast panel is shown in Fig. 14. The panels are conventionally reinforced with #5 bars at a 9” spacing. Before the panels are installed, the light gage support angles with cross straps are placed on the girder webs and adjusted appropriately to support the panels (see Fig. 15). This support system has been successfully used in the NU precast deck panels in Skyline Bridge[4].

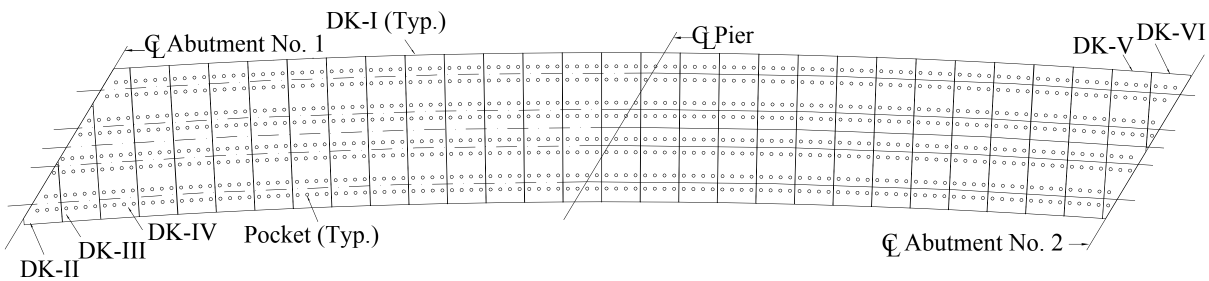


Fig. 13-Precast Deck Panel Layout

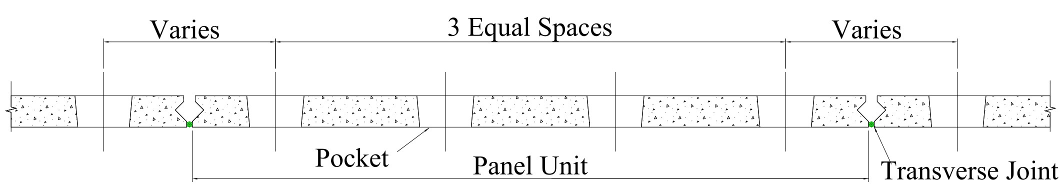


Fig. 14-Precast Deck Panel Cross Section

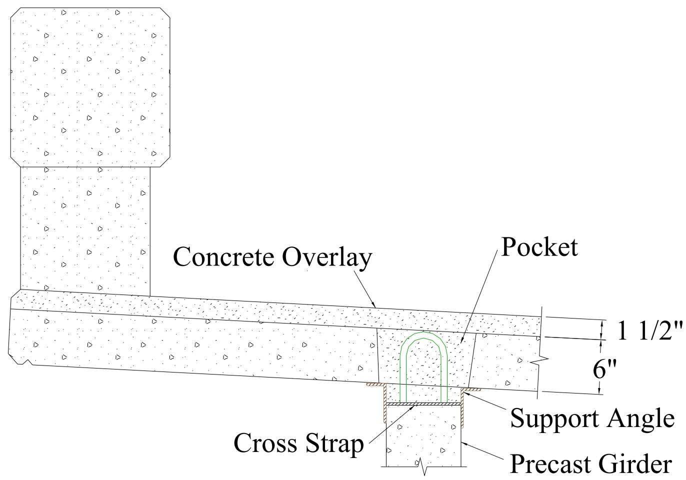


Fig. 15-Precast Deck Panel-Girder Connection Details

CONSTRUCTION SEQUENCE

The following construction sequence is recommended:

1. Construct the substructure.
2. Erect the girders span by span. Anchor the girders to the abutment and pier. De-tension the top external strands after the girders are installed.
3. Install the internal and intermediate diaphragms.
4. Cast the closure pour (the gap between the precast girders) over the pier.
5. Thread all tendons through the ducts.
6. Tension the lower tendons when the closure pour achieves a strength of 6,000 psi.
7. Grout the lower ducts and fill the vents.
8. Install the light gage support angles on the girder webs and adjust them to support the precast panels at the correct elevations.
9. Erect the precast panels. Panel pockets shall be grouted prior to the installation of next panel.
10. Remove the anchoring devices over pier to allow the girders to sit on the bearing pad.
11. Pour the concrete overlay and the panel transverse joints.
12. Tension the upper tendons. Grout the upper ducts and fill the vents.
13. Cast the Rail.
14. Pour the abutment diaphragm.

COST COMPARISON

The steel plate girders cost a total of about $500,000, which is 30% more than that of the concrete girders. For the concrete girders, 30 percent of the cost goes to the post-tensioning. The cost of cast-in-place bridge deck slab and rail in the original steel design is approximately $150,000, which is close to the cost of the precast deck panels with the concrete overlay, and rail. The precast deck panels will be produced by the contractor at the site. In terms of the total cost, the original steel design is about 25 percent higher the concrete alternate, which results in an estimated gross saving of $133,000.

CONCLUSIONS

This paper presents a value engineering project using the curved concrete U-girders to substitute for the steel plate girders, which results in a reasonable cost saving and a better overall structural performance. The precast deck panel solution not only eliminates any necessary lateral top bracing, but also significantly reduces the construction time. This project indicates that it is technically and economically feasible for the concrete superstructures to compete with the curved steel bridges.

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