Value Engineering Arbor Road Bridge with Curved Precast Concrete Girders

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Arbor Road Bridge is an Interstate 80 overpass consisting of two spans that are about 142 ft (43.3 m) and 136 ft (41.5 m) long. It is a horizontally curved bridge with a skew of 31 degrees. The original design used Grade 50 (50 ksi [345 MPa]) weathering steel plate girders and haunched segments spliced over the pier. Due to the high cost of steel girders at the time the project was bid, a value engineering proposal using a precast concrete alternative was approved by the owner. This paper describes a number of innovative designs in this project, including the use of curved precast concrete girders and precast concrete deck panels. Also presented are the girder analysis and design procedures, a description of the production of the precast concrete girders and deck panels, the construction sequence, and a cost comparison between the concrete and steel alternatives.

A majority of curved bridges are made with steel girders due to the perception that a curved concrete girder would be too expensive or even impossible to construct. Precast concrete girders have been used on roadways with curved alignment in the past.[1] To be economical, this has generally been accomplished with straight concrete girders between supports and curved deck edges. However, when the offset between the chord girders and curving deck is too large, the bridge appearance may be objectionable and the overhang length may be too large to meet the American Association of State Highway and Transportation Officials (AASHTO) requirements for crash resistance.

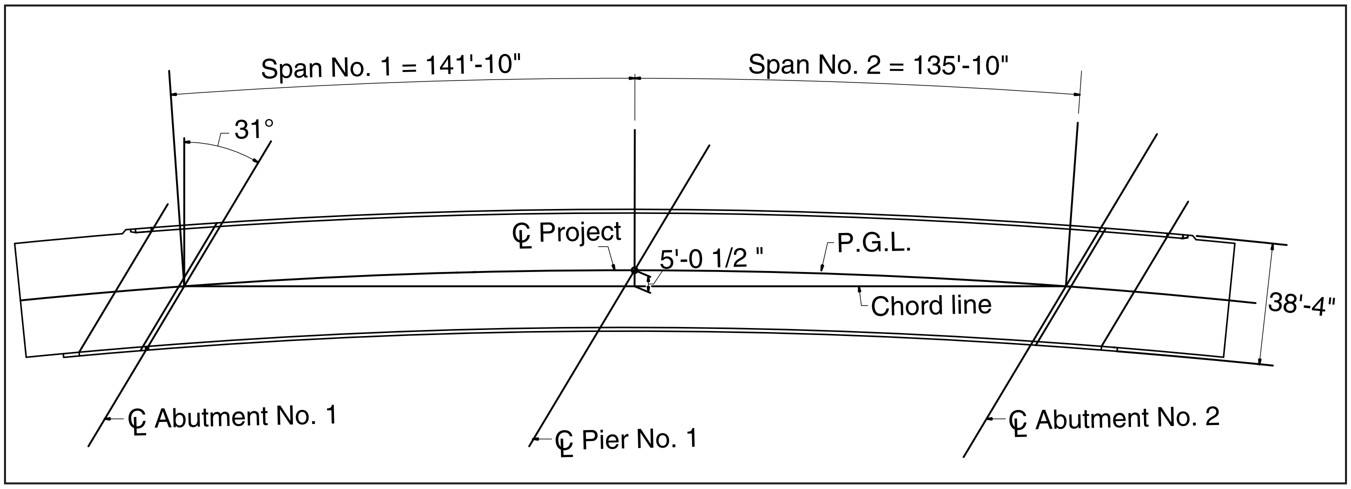

Fig. 1. Arbor Road Bridge general plan view showing span lengths. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; ' = ft; " = in.

To overcome these disadvantages, a "curved" concrete girder can be made with straight 40-ft-long (12.2-m) segments that have a "kink" between the adjacent segments. Due to standard casting bed lengths, a 40 ft (12.2 m) length is typical for precast concrete I-girders and other products. Thus, the available stock of forms used for straight pretensioned concrete I- and box girders can be used with slight, inexpensive modifications to create girders that have the appearance and function of curved girders. It was also possible to replace the high-performance steel plate girder design with a concrete girder design without increasing the superstructure depth.

This concept was the basis for the value engineering effort for the Arbor Road Bridge in Lincoln, Neb. The girders used on this bridge were originally conceived as NU1100 (43 in. [1100 mm] deep) I-girders (NU designates Nebraska University design). However, due to the special circumstances of this bridge and the contractor's uncertainty of whether both precasting companies in Nebraska (which own NU I-girder forms) would bid on the project, the contractor decided to use a generic U-shaped girder. In this paper, the authors present the details of the first implementation of the curved concrete girder bridge in Nebraska.

ORIGINAL DESIGN USING STEEL GIRDERS

Arbor Road Bridge is an Interstate 80 overpass in Lincoln. It has two spans, about 142 ft (43.3 m) and 136 ft (41.5 m) long (**Fig. 1**). As a horizontally curved bridge, it has a central angle of about 3 degrees. The overall bridge

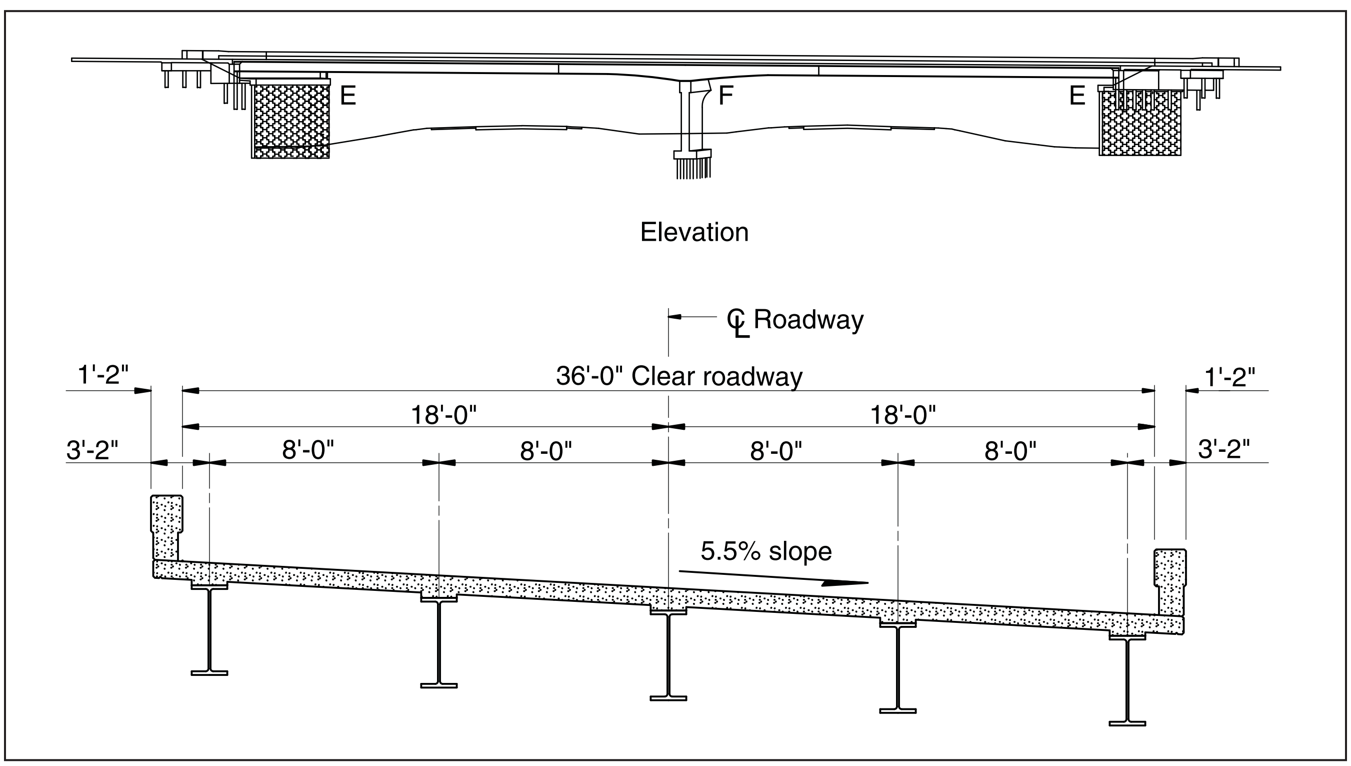

Fig. 2. Arbor Road Bridge elevation and cross section using the steel girders. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; ' = ft; " = in.

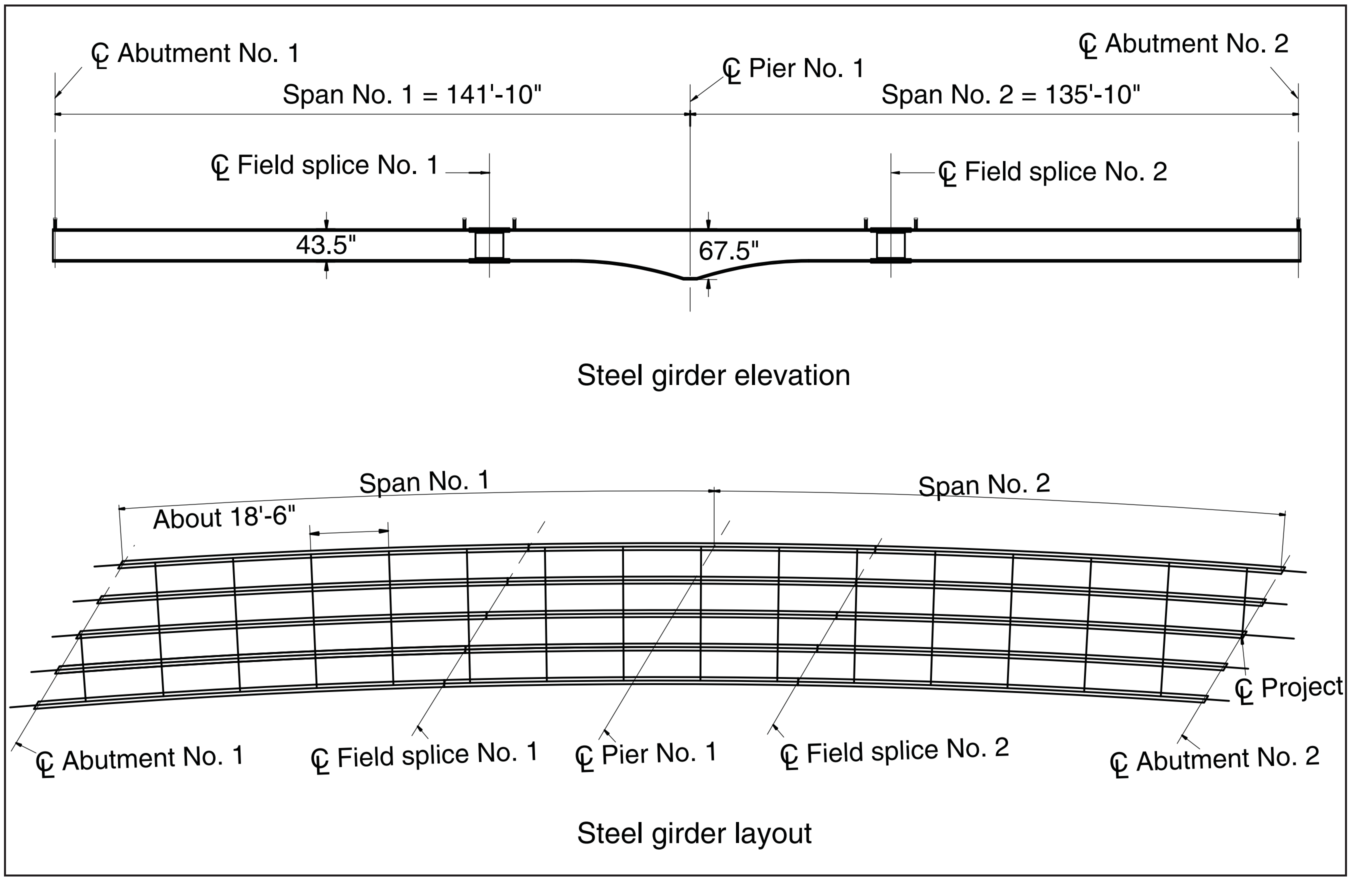

Fig. 3. Steel bridge girder elevation and framing plan with intermediate steel diaphragm placement at 18.5 ft (5.6 m). Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; ' = ft; " = in.

width is 38.33 ft (11.7 m), and it has a skew of 31 degrees. The original design used steel plate I-girders with a 7.5-in.-thick (191 mm) cast-in-place (CIP) concrete deck slab. Grade 50 (50 ksi [345 MPa]) weathering steel plate was specified for the plate girder to allow a girder depth of 43.5 in. (1105 mm), increasing to 67.5 in. (1715 mm) at the pier, to span the 142 ft (43.3 m) length.

The steel superstructure consisted of five girder lines spaced at 8 ft (2.4 m) (**Fig. 2**). Intermediate steel diaphragms were spaced at about 18.5 ft (5.6 m), as shown in the steel girder layout in **Fig. 3**. Steel diaphragms are considered primary elements in curved steel girder design. As a result of the high cost of steel at the time the project was bid, a value engineering proposal using a precast concrete alternative was submitted to the bridge owner, the Nebraska Department of Roads (NDOR), by the designer, Tadros Associates LLC of Omaha, Neb., on behalf of the contractor, A. M. Cohron & Sons Inc. of Atlantic, Iowa.

VALUE ENGINEERING USING CONCRETE ALTERNATIVE

Due to site constraints and in order to facilitate acceptance of the proposed curved concrete girder alternative, which was new to NDOR, the proposal stipulated that the substructure geometry and the total superstructure depth not be altered. The increase in pier and abutment vertical reactions due to use of the heavier concrete girders would be accommodated with the use of additional piling. The value engineering effort focused on the girder design.

Initially, the value engineering design involved the use of NU (Nebraska University) I-girders because it was—and still is—believed by the authors to be the most cost-effective solution. Five straight segments would have been used on each full-span-length girder. For all practical purposes, the girders would look curved. After the preliminary plans were sent to the two Nebraska precasters, only one precaster expressed interest in producing the curved concrete NU girders. Lack of interest by the other precaster was apparently due to the extremely high work load it had at that time. Even though the NU I-girder option was believed to be more cost-effective than the U-girder option, the contractor instructed the designer to use a generic girder shape to allow other, out-of-state producers to bid on the precast concrete products. The designer selected the U-girder shape due to its torsional stability during shipping and handling and its aesthetic appeal on the final bridge.

As a result of the final design, the concrete alternative consisted of four girder lines spaced at 9.33 ft (2.8 m). A significant innovation of the structural system was that precast concrete deck panels were utilized to eliminate the extensive top bracing generally required on curved-girder bridges. Additionally, use of the precast concrete deck would significantly reduce the shrinkage normally seen in CIP concrete decks. The following sections include descriptions of the structural analysis, girder design, precast concrete deck panel system, construction sequence, and cost comparison between the steel and concrete alternatives. This bridge was completed in July 2006.

HORIZONTALLY CURVED PRECAST CONCRETE U-GIRDERS

The concrete alternative utilizes a prismatic U-girder section, which is 45.8 in. (1163 mm) deep at its left web and 43.5 in. (1105 mm) deep at its right web to match the 5.5% cross slope super-elevation (**Fig. 4**). The bottom flange of the U-girder is 5 in. (127 mm) thick for most of the girder length and is thickened to about 12 in. (300 mm) at the pier end to accommodate the negative moment requirements in that area. The girder web thickness is 7.75 in. (197 mm) at the top and is increased to 8.25 in. (210 mm) at the bottom to facilitate removal of the steel forms inside the U shape. Each precast concrete girder consists of four straight segments, three of which are the standard 40 ft (12.2 m) steel form length and the fourth of which, near the pier, varies in length for each of the girders, as shown in the U-section girder elevation in **Fig. 5**. Internal concrete diaphragms, about 5.5 in. (140 mm) wide, are placed at the kinks between the adjacent straight segments.

Although it is possible, and perhaps more cost effective, to use pretensioning on chorded, horizontally curved concrete girders, only post-tensioning (PT) is used due to lack of time for design, development, and setup in this value engineering project. Three stages of PT are used to resist the girder weight, the deck weight, and the superimposed dead and live loads. Each girder has a PT anchorage block at its abutment end. It also has a thinner concrete diaphragm at the pier end to provide anchorage for the external top PT required for girder handling. Concrete is placed in the 12-in.-wide (305 mm) joint between the girder ends over the pier. No continuous pier diaphragms between girders are used, eliminating the cost and time associated with the forming and facilitating PT in the field without excessive longitudinal restraint. As shown in the U-section girder layout, the intermediate steel diaphragms are spaced about 33 ft (10 m) apart (Fig. 5), which is much longer than required by the corresponding curved steel girder alternative.

BRIDGE ANALYSIS

The bridge was designed using the same criteria used for the original steel design: AASHTO *Standard Specifications for Highway Bridges* with HS-25 truck loading.[2] A three-dimensional model was developed considering the actual curvature of each girder line using RISA-3D software. The analysis considered the precast concrete girders, the deck-slab strip transversely spaced at about 5 ft (1.5 m), and the intermediate steel diaphragms. The bridge was modeled as fixed for torsion at the abutment and free for torsional movement over the pier. All girder lines are seated over the bearing pads at the girder joints without being fixed to the pier so that no torsional moment is transferred between the superstructure and the substructure.

During the grid analysis, bridge members were subjected to dead loads, PT loads, and live loads. Torsional moments in the girders and forces in the intermediate diaphragms were determined. Also determined were the bending moments and shear forces in the girders, which are comparable to the results, assuming straight girders with lengths equal to the longest developed lengths.

GIRDER DESIGN

Working Stress and Flexural Strength Design Methods

Working stress and flexural strength design methods were performed at the critical sections to determine the amount of PT required. The flexural strength design method employs the strain compatibility approach, which has existed in PCI's *Precast Prestressed Concrete Bridge Design Manual*[1] for almost 10 years and was just recently approved for inclusion in the *AASHTO LRFD Bridge Design Specifications*.[3] As a result, a typical girder has 36 unbonded monostrands, which are 0.6 in. (15 mm) in diameter and Grade 270 ksi (1862 MPa) tensile strength (**Fig. 6**), to carry the girder self-weight and to achieve the desired camber. Unbonded

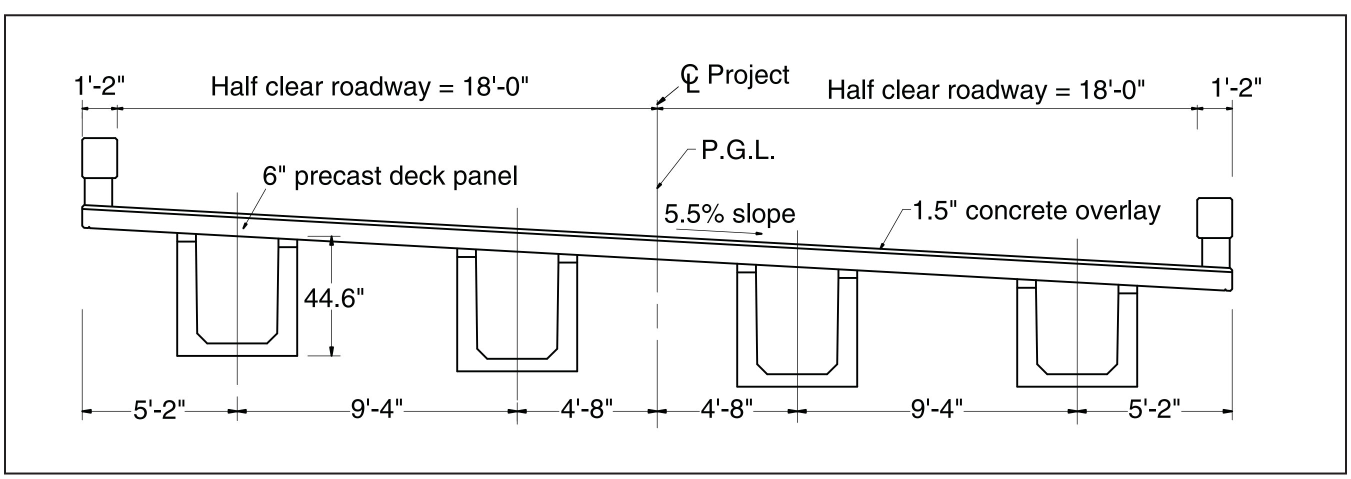

Fig. 4. Bridge cross section using the U-girders show a 44.6 in. (1130 mm) average depth of U-girder webs. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 ft = 0.3048 m; ' = ft; " = in.

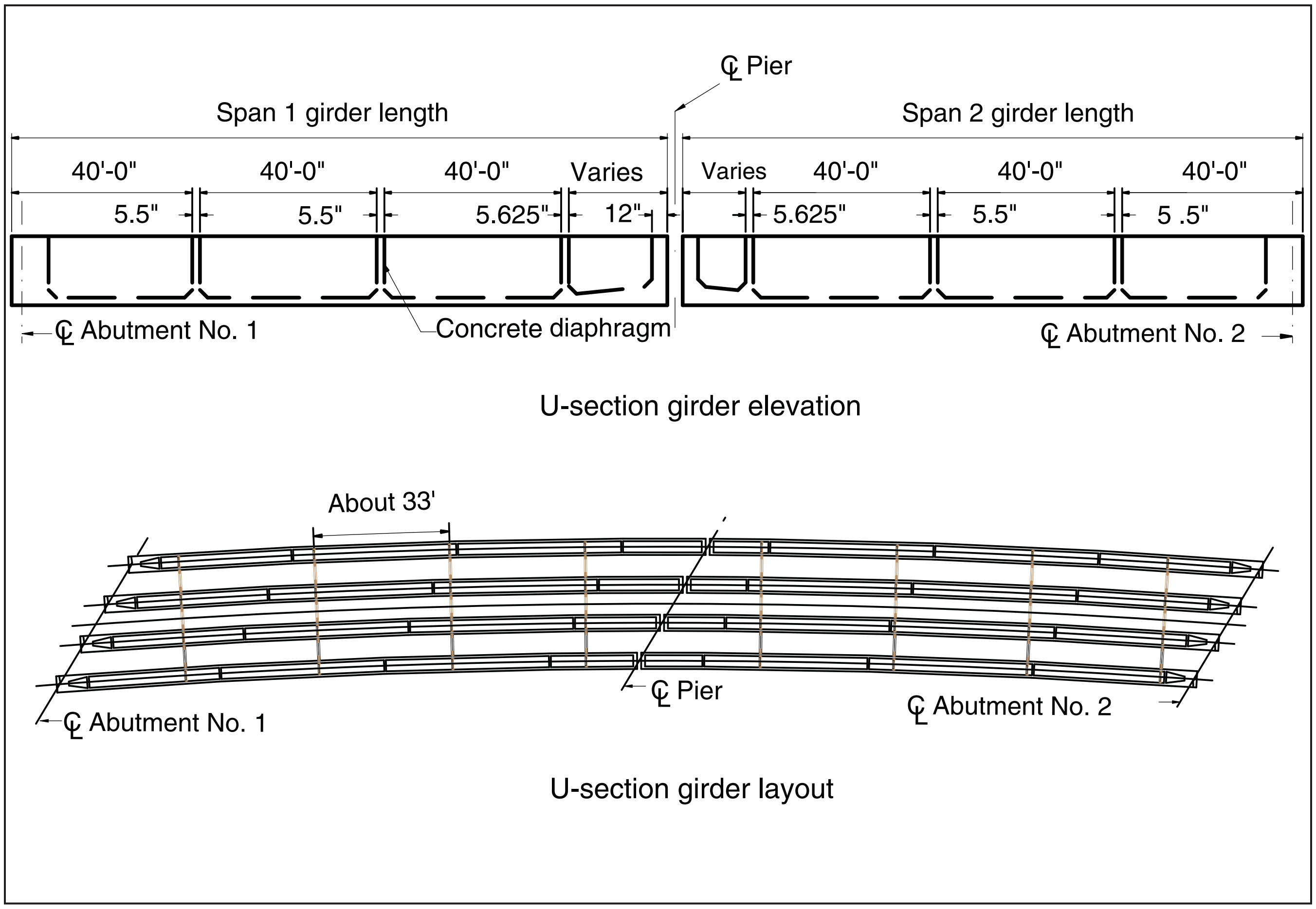

Fig. 5. U-girder elevation and framing plan. Note: 1 ft = 0.3048 m; 1 in. = 25.4 mm; ' = ft; " = in.

monostrands were utilized to avoid the need for grouting and anchorage at the first PT stage. The typical girder also has 4 PT tendons with 15 strands of 0.6 in. (15 mm) diameter at each tendon to carry additional loads (**Fig. 7**). Two of the four tendons are tensioned prior to the deck panel installation, and the other two are tensioned after the deck overlay is placed. The required girder concrete strength is 10,000 psi (70 MPa) at 56 days.

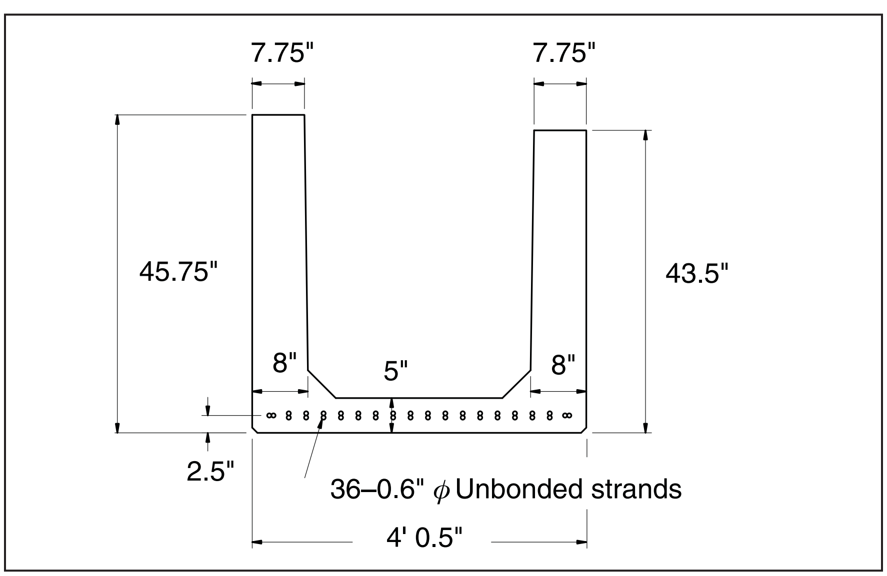

Fig. 6. Concrete girder section with unbonded monostrands. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; ' = ft; " = in.

Design for Prestress at Jacking

The strength design method was utilized to determine the required concrete strength at each PT stage, according to the NDOR policy established in May 2001.[4] At the first-stage PT of the monostrands, the required concrete strength is essentially controlled by the conditions at the time of girder lifting. Four lifting inserts are specified in the webs at each end. Due to the curvature of the girder, the center of the lifting inserts at each end was about 30 ft (9.1 m) from the girder end. The location of the lifting inserts ensures that the resultant lifting force and the center of gravity of the girder all pass through a straight line so that no twisting develops during lifting. Due to the long girder overhang at the time of lifting, four 0.6-in.-diameter (15 mm) external PT strands are temporarily used at the section top to resist the resulting negative moment. In addition, 10 pairs of

No. 5 bars (16M), 30 ft (9.1 m) long, are placed in the lifting area to control cracking. As a result, the required concrete strength at the first-stage PT is only 5900 psi (41 MPa) according to strength design. Similarly, the required concrete strengths are 8000 psi (55 MPa) and 10,000 psi (69 MPa) for the second- and the third-stage PT, respectively.

Shear and Torsion Design

The shear and torsion reinforcement is composed entirely of 75 ksi (517 MPa) welded wire reinforcement (WWR). D31 WWR (equivalent to No. 5 [16M] reinforcing bars) is used at the abutment, and D20 (equivalent to No. 4 [13M] reinforcing bars) is used for the remainder of the girder length. These are shown as WWR1 to WWR5 in **Fig. 8**. WWR6 and WWR7 are placed at the girder bottom flange for torsional resistance. WWR8 is placed at the girder ends to take the bursting force due to the 36 unbonded PT monostrands.

The horizontal shear reinforcement (used to ensure girder-deck composite action) consists of U-shaped bars projecting out of the girders. A typical U-shaped bar group consists of three No. 5 (16M) U-shaped bars. The U-shaped bar groups are spaced at approximately 2.5 ft (0.8 m), which is slightly larger than the maximum allowable spacing of 2 ft (0.6 m) specified in the *AASHTO LRFD Bridge Design Specifications*. This design is based on research conducted at George Washington University in Washington, D.C., and at the University of Nebraska, which showed that a spacing of up to 4 ft (1.2 m) results in satisfactory performance. The use of 2.5 ft (0.8 m) spacing on this bridge was required for modular placement of the pockets in the precast concrete deck panels.

PT End Block and Anchorage Details

Figure 9 shows the plan view of the PT anchorage block at the abutment. This bridge uses a PT end block modified from the standard block of NU I-girders, which has been used as a standard detail by NDOR.[4] As shown in Fig. 9, the abutment end block consists of a solid section about 3 ft (0.9 m) long and a 4.5-ft-long (1.4 m) transition block to the typical girder section. Four layers of WWR1 are included to control vertical splitting due to the tendons. Also, reinforcement WBC1 consists of No. 5 (16M) bars at a narrow 2 in. (51 mm) spacing to control horizontal cracking (**Fig. 10**). Also shown in Fig. 10 are the spirals surrounding the unbonded strands at the section bottom that confine the concrete to achieve adequate bearing strength.[5,6] The U-shaped bars at the section top contribute to the avoidance of possible splitting due to PT of the four external monostrands.

Girder Camber and Deflection

In the original design using the steel alternative, the vertical clearance at the

LESSONS LEARNED FROM THE POST-CONSTRUCTION MEETING

After completing the project and opening the Arbor Road Bridge to traffic, a meeting was held at the Nebraska Department of Roads (NDOR) District One office to discuss the experience with this novel project and lessons learned. Those present at the meeting included representatives of the precaster, contractor, designer, owner, and university. Following is a summary of the outcome of the meeting.

The project took longer than anticipated and longer than what the original steel girder bridge would have taken, contrary to expectations. Several factors contributed to delays, including bad weather, slow fabrication and installation of new curved girders, multiple concrete placements in the abutment, multiple concrete placements in the deck/railing system, and, most important, multistage field post-tensioning (PT). Future projects should try to minimize the number of field construction steps and perform as much of the PT in the precasting facility as possible.

Despite the delays, the project resulted in being more cost effective than steel, signaling a promising future for curved, precast concrete girder bridges.

It would take several projects of a similar nature to reach an adequate production and installation experience and "steady state," or typical, construction conditions, whereby more realistic time and cost estimates could be determined.

The contractor's past experience with precast, prestressed straight girders has been excellent. The contractor recommended that a curved I-beam, rather than the U-shape, be considered in future projects with the expectation that installation of all I-girders for each span would require one night's construction—rather than an installation rate of two girders per night for this system.

Multistage placement of concrete in the abutments slowed construction. This was the designer's way of keeping the girders stable against twisting, free to move during PT, and locked in place after PT. The designer indicated at the meeting that these tasks could have been achieved with a single abutment concrete placement and slightly different details. Removal of field PT would greatly assist in that task.

The precast concrete deck system used on the project involved too many construction steps and did not have adequate built-in tolerances. The contractor recommended simplifying it or using a cast-in-place concrete deck for future curved girder projects.

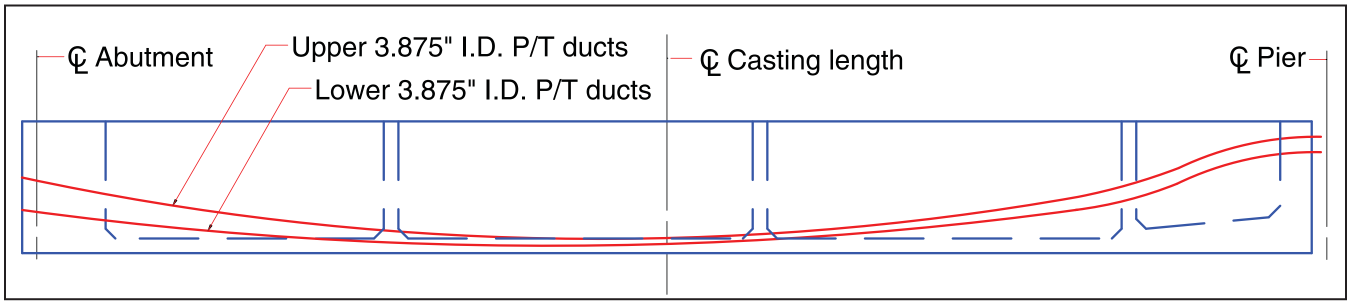

Fig. 7. Post-tensioning tendon layout. Note: 1 in. = 25.4 mm; ' = ft, " = in.

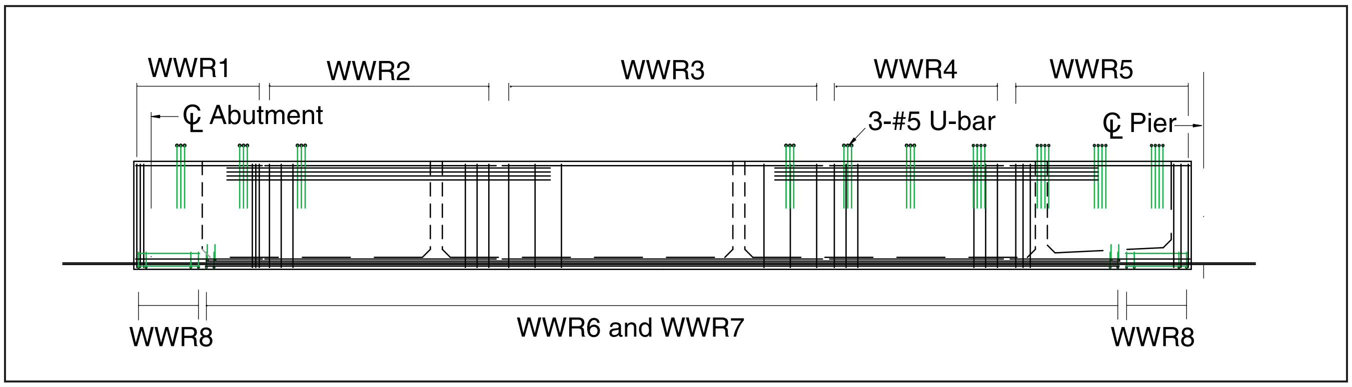

Fig. 8. Concrete girder welded wire reinforcement (WWR). Note: No. 5 bar = 16M; # = No.

critical section over Interstate 80 is exactly 16 ft (4.9 m), which is the minimum required clearance. Unlike the steel girders, which can be fabricated with an accurate pre-camber, the camber of the PT concrete girders can vary significantly from what is predicted. Therefore, the camber and deflection of the bridge was thoroughly investigated to ensure that the vertical requirement was satisfied.

A rigorous time-dependent analysis was performed considering the time line of the construction provided by the contractor. The designer used the creep coefficient equations proposed by Tadros et al. in the National Cooperative Highway Research Program (NCHRP) Report 496,[7] which was recently incorporated into the 2005 Interim Revisions to the *AASHTO LRFD Bridge Design Specifications*. In addition, a number of methods were utilized to increase the camber and possibly maximize the vertical clearance. One way is to release the external top strands once the girders are stored in the precasting yard. That is, the external top strands are tensioned only prior to PT of the bottom monostrands and re-tensioned during girder handling and shipping. By doing this, the downward deflection associated with the creep effect due to the top strands is completely eliminated. In addition, the height of the lowest girder line, which is the exterior girder,

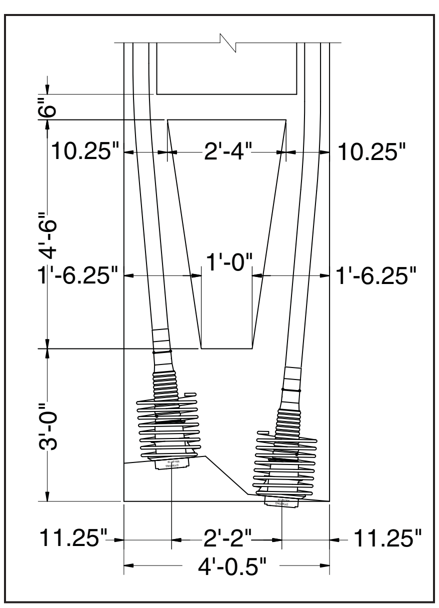

Fig. 9. Plan view of post-tensioning anchorage block at the abutment. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; ' = ft; " = in.

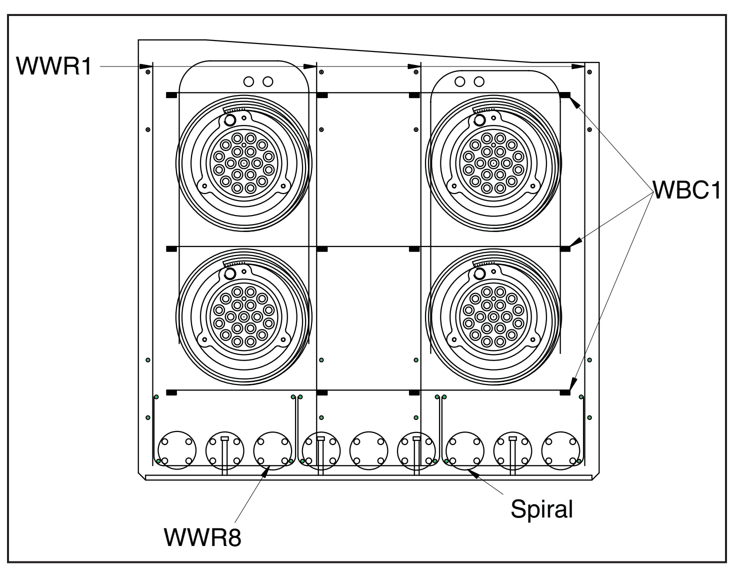

Fig. 10. Post-tensioning anchorage block reinforcing details.

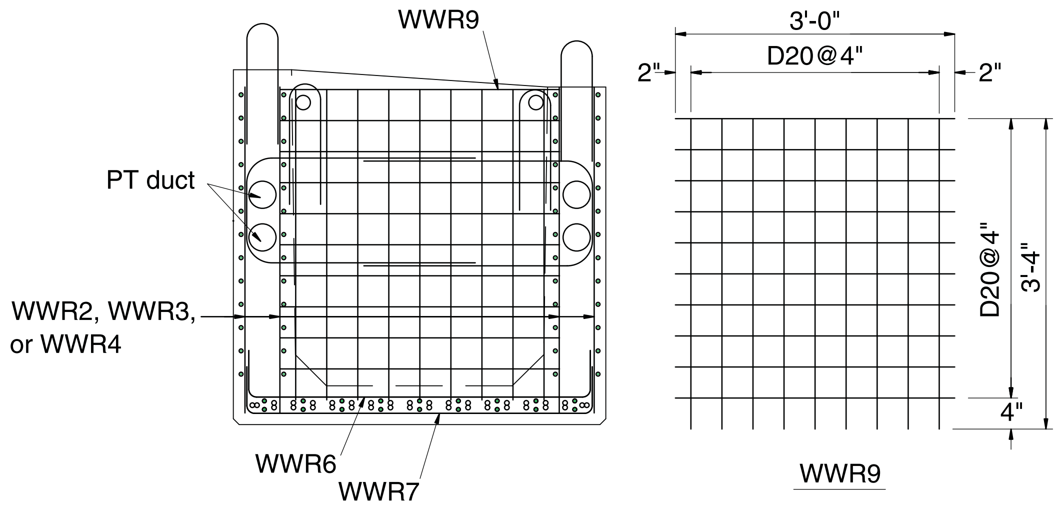

Fig. 11. Reinforcement in the internal concrete diaphragm. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; ' = ft; '' = in.

was reduced 4 in. (102 mm) to ensure that the minimum vertical clearance requirement was met.

Internal Concrete Diaphragm

A WWR consisting of D20 by D20 at 4 in. (102 mm) spacing is used in the internal concrete diaphragms (**Fig. 11**). The U-shaped bars oriented horizontally near the girder webs contribute to resisting the pull-out force during PT at the kinks. The vertically oriented U-shaped bars are included to avoid possible bursting due to the external top strands.

Girder Joint Details over the Pier

As discussed previously, there is no pier diaphragm included in this bridge for ease of construction. Only a 12-in.-wide (305 mm) joint is placed between adjacent girder ends over the pier. The specified strength of the concrete in the cast-in-place joint is 6000 psi (41 MPa) at 28 days. As a result of the analysis using the flexural strength design method over the pier, 6000 psi (41 MPa) concrete is insufficient to achieve the required flexural capacity. Instead of using an impractical high strength for cast-in-place girder joint concrete, confinement by WWR is considered, thus allowing for a more reasonable ultimate concrete strength. Because the *AASHTO LRFD Bridge Design Specifications* do not include provisions on confined concrete analysis, the research findings by Lambert and Tabsh[8] and Saatcioglu and Razvi[9] on confinement were implemented in the design. As a result, WWR consisting of D31 by D31 at 4 in. (102 mm) spacing is used at the joint section bottom to confine the concrete and to gain the required concrete strength for ultimate flexural design (**Fig. 12**).

Temporary Supports during Girder Installation

Typical precast concrete girder installation requires two bearing pads at each end of adjacent girders over the pier. To reduce the amount of bearing, temporary supports are included during girder installation (Fig. 12). Girder support brackets consist of steel channels supporting the girders and high-strength threaded rods anchored to the pier wall, which not only allow for possible adjustment of girder elevations during girder installation but also provide the girder with significant lateral stability. These support devices will remain until the bridge deck is placed.

PRECAST CONCRETE DECK PANEL

This bridge uses 6-in.-thick (150 mm) precast concrete deck panels with a 1.5-in.-thick (38 mm) concrete overlay. Compared with a conventional cast-in-place concrete deck slab, the precast concrete deck panels are normally pro-

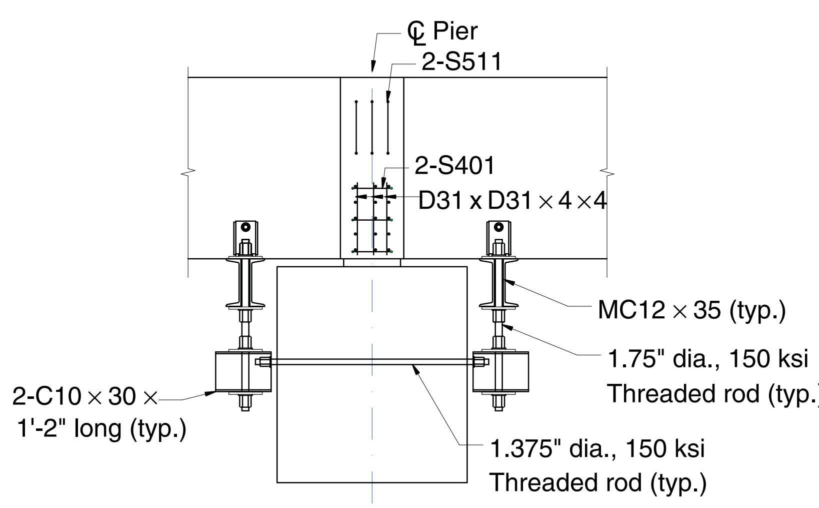

Fig. 12. Girder joint details over the pier. Note: 1 in. = 25.4 mm; 1 ft = 0.3048 m; ' = ft; '' = in.

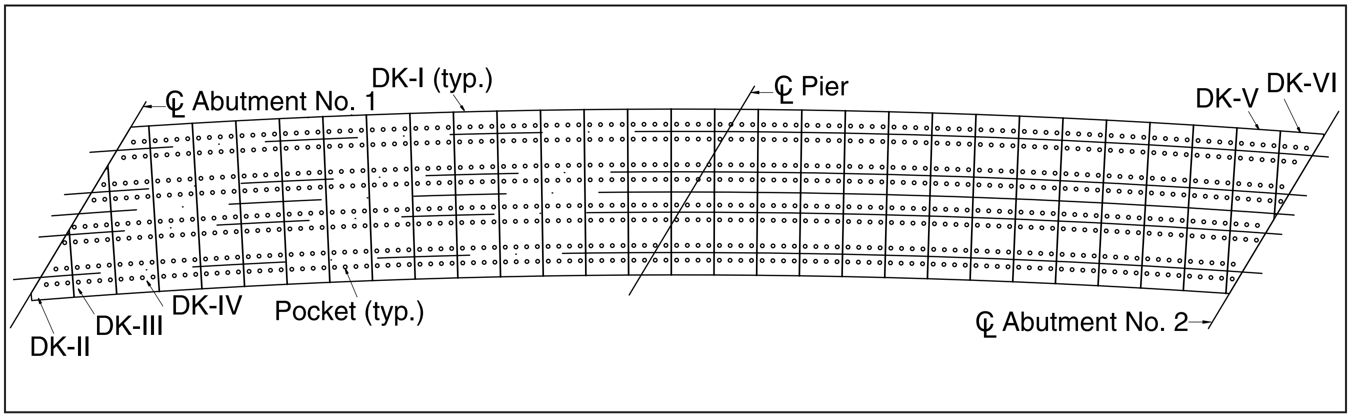

Fig. 13. Precast concrete deck panel layout.

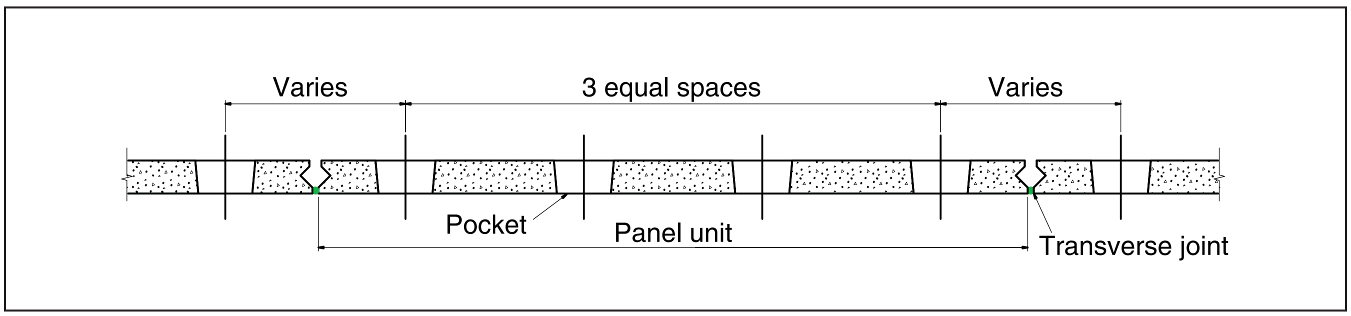

Fig. 14. Precast concrete deck panel cross section along the girder line.

duced with better quality control. Also, the majority of shrinkage, temperature drop due to the cement hydration cycle, and creep has occurred before the precast concrete panels are made composite with the girders. In addition, the deck is PT with the girders in the third stage. Therefore, the precast concrete deck panels are expected to last as long as the precast concrete girders. Another possible benefit of using precast concrete panels is fast construction.

The primary reason for using the precast concrete panels in curved bridge construction, however, is to eliminate the extensive lateral top bracing mostly required to prevent twisting of the girders during deck placement. In this bridge, the original steel girder design did not provide any additional top lateral bracing due to the small bridge curvature, but intermediate steel diaphragms were provided at a relatively close spacing of about 18.5 ft (5.6 m). Use of the precast concrete deck panel systems allows an increase in the steel diaphragm spacing to about 33 ft (10 m).

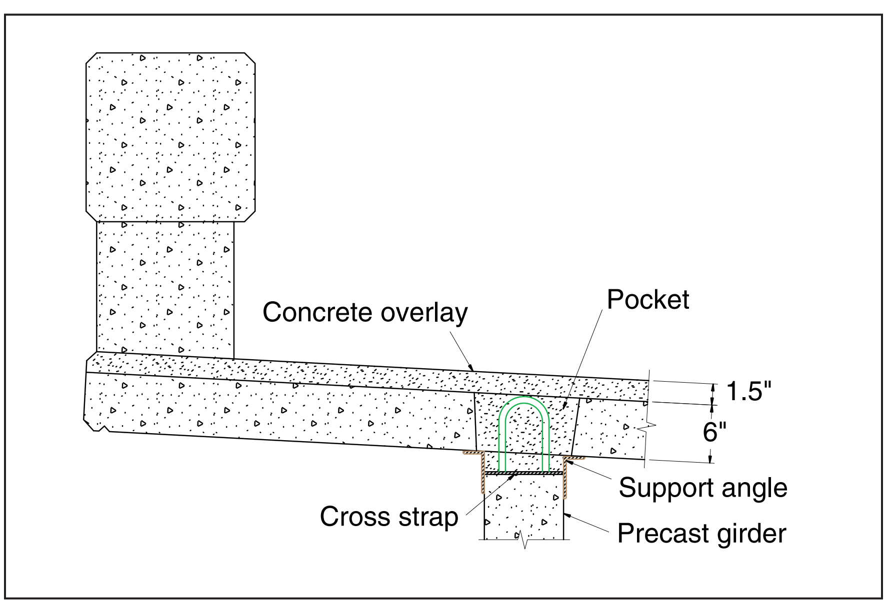

Fig. 15. Connection details between the precast concrete deck panel and the girder. Note: 1 in. = 25.4 mm; " = in.

This bridge includes a total of 30 precast concrete panels, 25 of which are typical units, as shown in the panel layout (**Fig. 13**). A typical panel is 38 ft 4 in. (11.7 m) long, about 10 ft (3 m) wide, and 6 in. (150 mm) thick. Each panel has pockets spaced at approximately 2.5 ft (0.8 m) along the girder line to house the U-shaped bars for the composite action between the girder and the panel. The pockets are 10 in. (255 mm) in diameter at the top and are tapered to 9 in. (230 mm) at the bottom. They are filled with non-shrinkage grout immediately after each panel is installed. Once the grout gains 2000 psi (14 MPa) of strength, the next panel can be placed. The panels are installed starting at the midspan section and moving on to the abutment and pier ends.

Figure 14 shows a typical cross section of the precast concrete panel. The panels are conventionally reinforced with No. 5 (16M) bars at 9 in. (230 mm) spacing. Prior to the panel installation, light gauge angles with cross straps are placed on the girder webs to support the panels (**Fig. 15**). This support system has been successfully used in the NU precast concrete deck panel system in the Skyline Bridge in Omaha.

GIRDER AND DECK PANEL PRODUCTION

The precast concrete girders were produced at Concrete Industries Inc. using self-consolidating concrete. **Figure 16** illustrates the girder reinforcement, which includes the bottom unbonded PT monostrands, vertical shear WWR, and PT anchorage reinforcement near the abutment end. Spiral reinforcement is used to confine the monostrands near the girder ends. Once the girder concrete is placed and achieves the specified concrete strength for the first-stage PT, the top external strands are tensioned, followed by tensioning of the bottom monostrands. **Figure 17** illustrates the precast concrete girder in the precasting yard. It also shows the top external strands, the horizontal shear U-shaped bars, lifting inserts, and the PT ducts. The precast concrete deck panels were produced by the contractor at the bridge site.

CONSTRUCTION SEQUENCE

This suggested construction sequence was followed:

1. Construct the substructure. Note that steel beams are connected with the brackets at the pier for torsional restraint during the installation of precast concrete girders and deck panels, which allows for one bearing pad at each girder line over the pier (**Fig. 18**).
2. Erect the girders span by span (**Fig. 19**). Anchor the girders to the abutment and pier. Detension the top external strands after the girders are installed.
3. Install the internal and intermediate diaphragms (**Fig. 20**).

Fig. 16. Girder reinforcement, elevation view.

Fig. 17. Precast concrete girder stored in the precasting yard.

Fig. 18. Constructed pier.

Fig. 19. Girder installation.

Fig. 20. Installation of intermediate steel diaphragms.

Fig. 21. Post-tensioning lower tendons.

4. Place the closure concrete in the gap between the precast concrete girders over the pier.
5. Thread all PT tendons through the ducts.
6. Tension the lower PT tendons when the closure concrete achieves a compressive strength of 4000 psi (28 MPa) (**Fig. 21**).
7. Grout the lower PT ducts, and fill the vents.
8. Install the light gauge support angles on the girder webs and adjust them to support the precast concrete panels at the correct elevations.
9. Erect the precast concrete panels. Panel pockets shall be grouted prior to the installation of the next panel (**Fig. 22**).
10. Remove the anchoring devices over the pier to allow the girders to sit on the bearing pad.
11. Place the concrete overlay and the panel transverse joints (**Fig. 23**).
12. Tension the upper PT tendons. Grout the upper PT ducts and fill the vents.
13. Place the concrete in the abutment diaphragm.
14. Cast the rail. **Figure 24** shows the completed bridge.

COST COMPARISON

The original steel plate girders cost about 40% more than the projected cost of the precast concrete girders. For the precast concrete girders, 30% of the cost was related to PT. The cast-in-place concrete deck slab used in the original steel design cost approximately the same as the cost projected for the precast concrete deck panels and the concrete overlay. In terms of the overall cost, the concrete alternative was projected to cost about 25% less than the original steel design. The actual cost of the Arbor Road Bridge was higher than the projected costs, but still lower than that of the steel alternative. Some of the additional expenses encountered were due to inexperience with this new system and unanticipated winter weather that delayed construction.

CONCLUSIONS

Value engineering of the Arbor Road Bridge using precast concrete girders involves a significant number of innovative designs. The uniqueness of this bridge is summarized as follows:

- Horizontal curved precast concrete girders: The precast concrete girders in this project are horizontally curved, which is the first implementation of such a bridge type in the state of Nebraska and one of very few in the United States.
- High-strength concrete girders: The precast girders use self-consolidating concrete with a design compressive strength of 10,000 psi (69 MPa) at 56 days. The high-strength concrete girders are expected to improve the overall structural performance and durability of the bridge structure.
- Efficient use of prestressing: This bridge presents a use of various types of prestressing (including internal and external) and bonded and unbonded PT strands. It includes three stages of PT for the girder self-weight, the deck weight, and the superimposed dead and live loads. The unbonded monostrands are used as the first-stage PT together with the top external strands for girder handling. The second-stage PT is applied to the precast concrete girder prior to deck panel installation. Once the panels act compositely with the girders and the overlay is placed, the third-stage PT is applied, which allows the deck panels to be in compression under service loads.
- High-performance precast concrete deck panels: Use of precast concrete deck panels eliminates the need for lateral top bracing or largely reduces the number of intermediate diaphragms required for curved bridge construction, which is a significant innovation of the total system. The precast concrete deck panels allow for better quality control during production, rapid construction, and improvement of structural durability.
- State-of-the-art analysis and design: The bridge analysis involves a 3-D grid analysis on the overall structure and a rigorous time-de-


Fig. 22. Precast concrete panel installation.


Fig. 23. Placement of overlay concrete.


Fig. 24. Completed bridge.

pendent analysis to account for creep effects. The girder design utilizes the strength design method for prestress at release, the strain compatibility approach for the flexural strength design, and the newly developed prestress loss solutions on high-strength concrete girders. The latest research findings on the concrete confinement and the maximum spacing of horizontal shear reinforcement have also been incorporated into the bridge design.

- Efficient and economical overall design: The bridge presents an efficient design with a span-to-depth ratio of 33, resulting in significant savings, not only including the direct cost savings over the original steel alternative, but also economies in future maintenance.
- Competition with curved steel bridges: This project shows that it is technically and economically feasible for concrete superstructures to compete with curved steel bridges.

As a winner of the 2006 PCI Design Award for Best Bridge with Spans Greater than 135 ft, the project garnered these comments from the jury: "A truly innovative structure, it utilizes several precast concrete elements and demonstrates a unique application of precast, prestressed girders on a curved alignment."

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