

Design and Detailing of Nebraska's First UHPC Ribbed Slab Decked I-Beam Bridges

Alec Stubbe, P.E. (corresponding author) – Bridge Design Team Leader, e.construct.USA, LLC, Omaha, NE, USA, Email: alec.stubbe@econstruct.us

Adam Sevenker, P.E. – Senior Bridge Engineer, e.construct.USA, LLC, Omaha, NE, USA, Email: asevenker@econstruct.us

Mostafa Abo El-Khier, Ph.D., P.E. – Structural Engineer, e.construct.USA, LLC, Omaha, NE, USA, Email: mostafa.aboelkhier@econstruct.us

Fouad Jaber, P.E. – Assistant Bridge Engineer, Nebraska Department of Transportation, Lincoln, NE, USA, Email: fouad.jaber@nebraska.gov

Kyle Zillig, P.E. – Assistant Bridge Engineer, Nebraska Department of Transportation, Lincoln, NE, USA, Email: kyle.zillig@nebraska.gov

Abstract

The state of Nebraska has embarked on an ambitious program to design a series of bridges using ultra-high-performance concrete. The features of design of four bridges being designed for NDOT are presented. The first bridge is the Belvidere North Bridge, in Belvidere, Nebraska. It consists of three spans, 75-85-75 ft (22.9-25.9-22.9 m). It is 42 ft-8 in. (13.0 m) wide. This paper covers design and detailing in the longitudinal direction and the transverse direction. It shows analysis for the closure pours to meet two conditions: permanent net compression across the joint between precast and CIP components using transverse post-tensioning, and full strength resistance with rebars embedded in the CIP joint. Design and detailing over the piers for continuity due to superimposed dead loads and live load will be discussed. Design and detailing of an open railing system for crash loading will be covered. Resistance of the top flange ribs to this extreme loading will be shown to be satisfactory without need for converting the overhang in this area to a solid slab. Two types of longitudinal prestressing will be covered. The first type is a segmental match-cast post-tensioning system. The second type is an innovative incrementally cast pretensioned system. Both systems are motivated by the desire to accommodate precasters in the US where mixers are limited to about 2-3 cubic yards (1.5-2.3 m³) per batch and where production of a full span product involves higher risk than production of short segments.

Keywords: UHPC, Post-tension, Ribbed Slab Decked I-Beam, Segmental Bridge, Match-Cast Joint, Accelerated Bridge Construction (ABC)

1. Bridge Superstructure Features

The state of Nebraska has embarked on an ambitious program to design a series of bridges using ultra-high-performance concrete (UHPC). The design is based on previously completed research by the University of Nebraska to create a cost effective UHPC mix (Mendonca et al. 2020). The non-proprietary mix only costs a fraction of prebagged proprietary mixes. It achieves very high tensile properties and adequately high compressive strength. It also achieves excellent durability. Using this mix along with a highly optimized decked I-beam cross section, shown in Figure 1, achieves a cost competitive UHPC superstructure system that features rapid construction and a highly durable deck. This paper covers the features of design of four bridges being designed for NDOT. The first bridge is the Belvidere North Bridge, in Belvidere, Nebraska which is the focus of this paper. The other three are overpasses over I-480 in Omaha, Nebraska. The Belvidere North Bridge is the first one to be designed with this system.

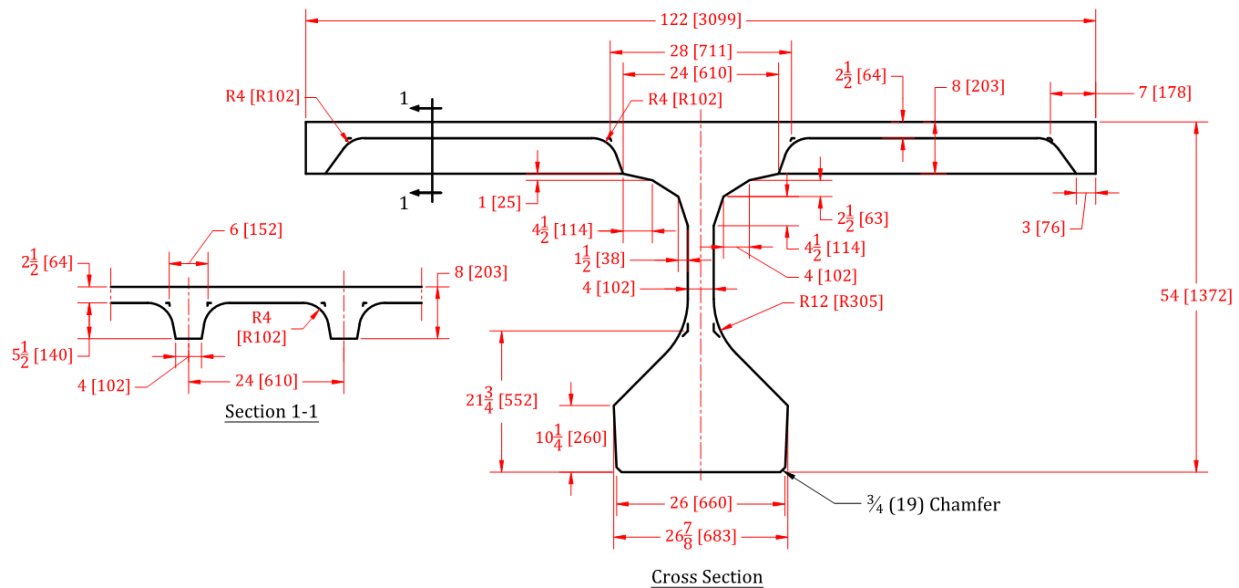


Figure 1. UHPC Ribbed Slab Deck I-Beam Cross-section Dimensions Used in Belvidere Bridge (Units: in (mm)).

The bridge is a part of a larger development in the town of Belvidere, about 75 miles southwest of Lincoln, Nebraska. The twin structure next to the proposed bridge dictated the framing arrangements for the subject bridge in order to maintain the flow of stream water under the bridges. It consists of three spans, 75-85-75 ft (22.9-25.9-22.9 m). It is 42 ft-8 in. (13.0 m) wide. Figure 2 shows the proposed cross section of the bridge. The deck will slope 2 percent for the full width. Only four girder lines, or a total of 12 girders are required for the bridge superstructure. Exterior girders are attached to open railing required to resist TL3 crash loading. A cast-in-place longitudinal UHPC joint is used to transversely connect the four girders in the cross section. As such, bars projecting from the beams and overlapped in the joint, would ensure full development in the 8-inch (203 mm) wide joints, due to the superior properties of UHPC.

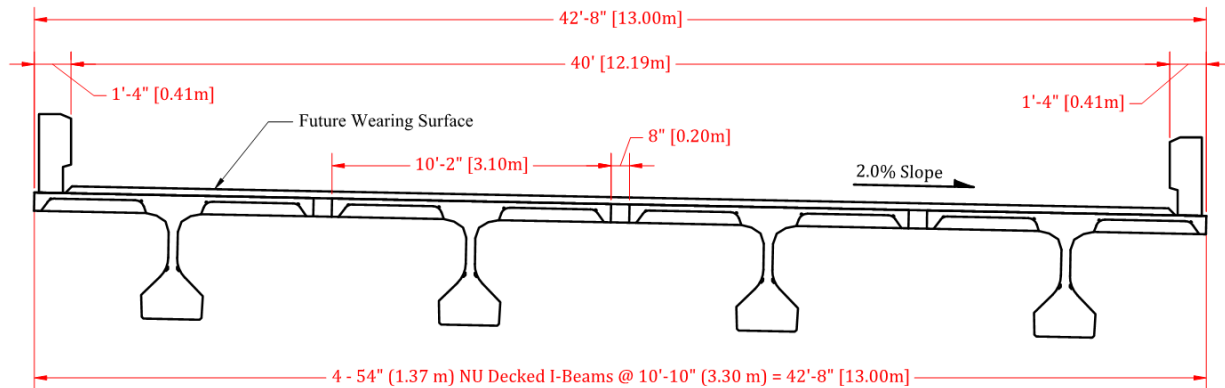


Figure 2. Belvidere Bridge Cross Section.

The three spans are made continuous over the piers using rebars extending from beam ends. The bars were designed to resist the negative moments generated by a 3 in. (76 mm) future asphalt wearing surface, 34" (0.86 m) open railing and live load. To maintain a full UHPC deck surface, a UHPC layer is cast in place over the pier and abutment diaphragms.

Due to the unavoidable presence of cold joints at the closure pours and over the piers and abutments, it was decided to use severe corrosion resistant reinforcing bars, according to ASTM A1035 CS. These bars have a minimum yield strength of 100 ksi (690 MPa) and ultimate strength of 150 ksi (1034 MPa). The bar's high strength was utilized for the negative moments over the piers in the longitudinal direction.

Belvidere North Bridge was designed to be produced using two possible options. Option 1 assumes the conventional precast pretensioned system which is the most common system for conventional concrete in the US. However, it poses a significant challenge at present as each girder would require nearly 26 cubic yards (19.9 m³) of concrete. Current mixers used by precasters can only mix about 2-3 cubic yards (1.5-2.3 m³) of UHPC per batch. This would require 10 batches per girder and pose a significant risk of forming horizontal cold joints in various layers being placed in the forms.

The second option, which is the focus of this paper, utilizes segmental match-cast girders. Each segment is about 20 ft (6.1 m) long and requires less than 7 cubic yards (5.4 m³). It is anticipated that the precaster will batch two or three batches, place them in a ready-mix concrete truck, continue to agitate the concrete in the drum, and add admixtures as needed to maintain flowability until the contents are placed in the forms. A specially designed bulkhead will help form shear keys in the leading end of the segment. The bulkhead and side forms are then advanced to the end for the next segment and concrete is placed for that segment. The resulting cold joint between segments are designed using the same zero tension considerations as for segmental box girder bridges. Due to the high tensile capacity of UHPC, it was determined, both theoretically using finite element analysis and experimentally, that no enlarged end blocks for post-tensioning anchorages are required. Therefore, the same cross section is used throughout the full length of the girder.

2. UHPC Specifications

Three different UHPC special provisions were created to represent precast post-tensioned segmental girders, precast pretensioned girders, and cast-in-place UHPC at jobsite. All provisions have the following minimum requirements of mechanical properties at 56 days, with specimens prepared and tested according to the general requirements of ASTM C1856: (1) compressive strength = 17.40 ksi (120 MPa), according to ASTM C39, tested using 3 in. by 6 in. cylinders; (2) tensile properties, according to ASTM C1609, 4 in. (102 mm) by 4 in. (102 mm) by 14 in. (356 mm) prisms: (a) cracking strength = 1.5 ksi (10.3 MPa), (b) peak strength = 2.0 ksi (13.8 MPa), (c) peak-to-cracking ratio ≥ 1.25 and (d) residual stress at span/150 to cracking ratio ≥ 0.75 . Requirements (c) and (d) represent minimum criteria for strain hardening and for ductility. They are needed, particularly for shear design to ensure that fibers in the mix can be used to contribute to shear capacity without having to have a minimum amount of stirrups (conventional shear reinforcing bars). Flow of fresh concrete is from 8 to 11 in. (203 to 279 mm). The minimum precast concrete strength at prestress is 10 ksi (69 MPa). The measured shrinkage according to ASTM C157 (using 4 in. (102 mm) by 4 in. (102 mm) by 14 in. (356 mm) prisms) at 28 days must not exceed 300 microstrain. To achieve that, a shrinkage reducing admixture is recommended to be used. Also, UHPC shall be placed continuously without interruption into the forms for any given product. Interruption for any time has the potential to cause cold joints which may not have fibers crossing them. Accordingly, interruption of concrete placement for any length of time is not permitted and can be a cause for rejection of the product.

3. Girder Segmentation

The following figure shows the location of the match-cast construction vertical joints for the girders in both spans, 75 ft (22.9 m) and 85 ft (25.9 m).

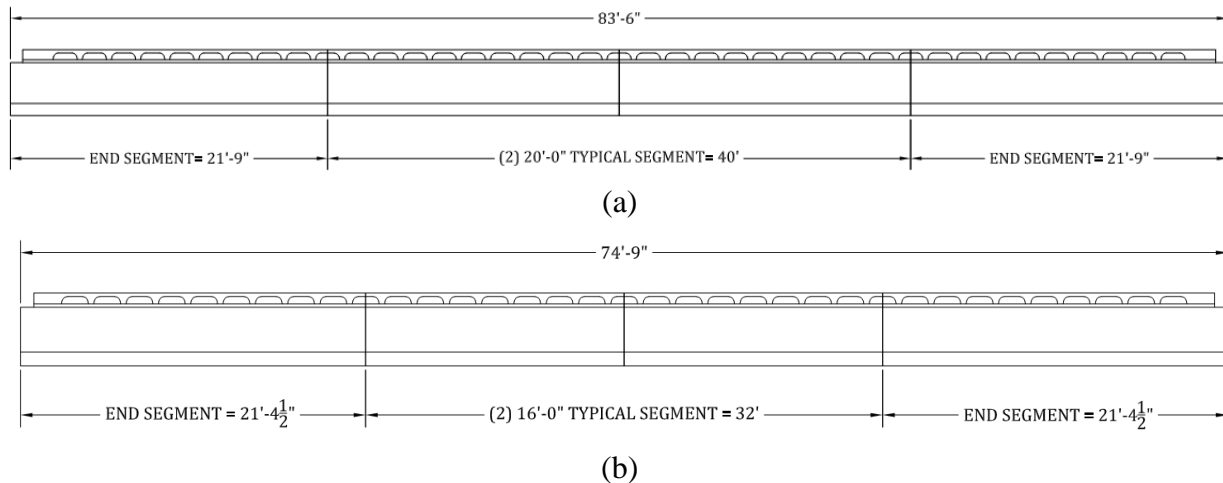


Figure 3. Girder Segmentation for (a) 85 ft (25.9 m) span and (b) 75 ft (22.9 m) span

4. Longitudinal Direction Design

The design is accomplished in accordance with the *AASHTO LFRD Design Specifications 9th Edition* and *Nebraska Bridge Office Policies and Procedures (BOPP) Manual 2016* supplemented with recommendations of the *structural design Guide "Appendix E" in PCI-UHPC Final Report*

Publication type: Full paper

Paper No: 38

for both options; post-tension and pretension. The Nebraska Department of Transportation (NDOT) requires a live load factor of 2.0, rather than the 1.75 in AASHTO. According to AASHTO LRFD Table 4.6.2.2.1-1, the Decked I-beam is considered as Type “j” bridge. An equivalent slab thickness, with the same inertia of the transverse ribbed slab, is obtained to allow for using the AASHTO LRFD live load distribution factor tables. PGSuper, a commercial software, was used to obtain the unfactored/factored shear forces and bending moments.

4.1. Required Prestress

No tension stresses are allowed at the match-cast vertical joints according to AASHTO LRFD Article 5.9.2.3. For the post-tensioned option 22-0.6” (15.2 mm) diameter bottom bonded strands in a 4.50 in. (114 mm) diameter duct and 7-0.6” (15.2 mm) diameter top unbonded strands in a 2.50 in. (64 mm) diameter duct are required. The design is controlled by the service stress limits. The following figure shows the UHPC DIB cross-section with the required post-tensioning tendons. The top unbonded strands are mainly for camber control. The same number of strands are used for the pretension option.

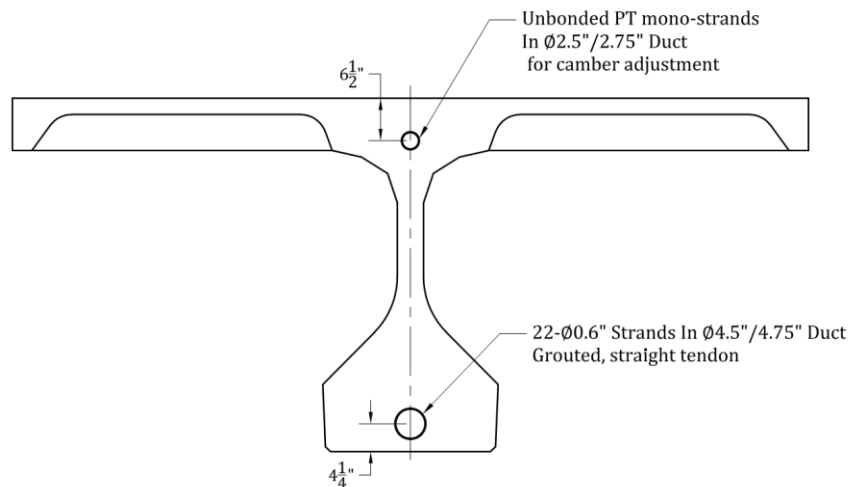


Figure 4. Required Prestress for Post-tensioned Option (1 in. = 25.4 mm).

4.2. Negative Moment Reinforcement

The negative moment design was conducted at the face of the support which has a construction joint, so the fiber contribution is not considered. Eight No. 8 (#25) ASTM A1035 CS Grade 100 bars are added in the top bulb of the DIB to resist the negative moment. The longitudinal bars in the cast-in-place UHPC joint are not considered in the flexural capacity. The power formula for the ASTM A1035 Grade 100 utilizes the ultimate strength of 150 ksi. Additional #5@8 in. (#16@203 mm) bars are added along the construction joint to control the cracks at the negative moment section considering exposure condition 1.

4.3. Shear Reinforcement

The critical shear sections are designed based on the PCI-UHPC Final Report Appendix E. The inclination angle “ θ ” for UHPC is usually below 30° in positive moment zones and it increases in

negative moment zones. The angle depends on the tension strain in the main resisting tension reinforcement at each zone. For the 75 ft (22.9 m) span, two different critical shear sections were investigated. The first section is near the abutment where the moment is positive and no shear reinforcement is needed. The second section is near the pier where the moment is negative and shear reinforcement is needed for about 11 ft (3.35 m) from the girder end. For the 85 ft (25.9 m) span, shear reinforcement is provided at both ends of the beam for 11 ft (3.35 m) to cover the negative moment zone area. The shear reinforcement was design based on using Grade 60 steel, however, ASTM A1035 CS Grade 100 bars are used. The shear reinforcement varies from #5@7" (#16@178 mm) to #5@12" (#16@305 mm) for the end 11 ft (3.35 m) of the beam.

4.4. Vertical Shear Design at Match-Cast Joint

According to AASHTO, Article 5.12.5.4.2, it is recommended to use multiple small-amplitude shear keys on the web over approximately 75% of the section depth instead of larger single-element keys. The main shear keys at the UHPC DIB web are shown in Figure 5. The shear keys in the top and bottom flanges are not expected to transfer major shear forces and are considered alignment keys. At ultimate limit state, the top flange may not be available in the negative moment zone for resisting interface shear. Thus, the bottom flange area is only considered for this contribution since it is expected to be in compression as shown in Figure 6. Also, the joint will be filled with epoxy which is not considered in the design.

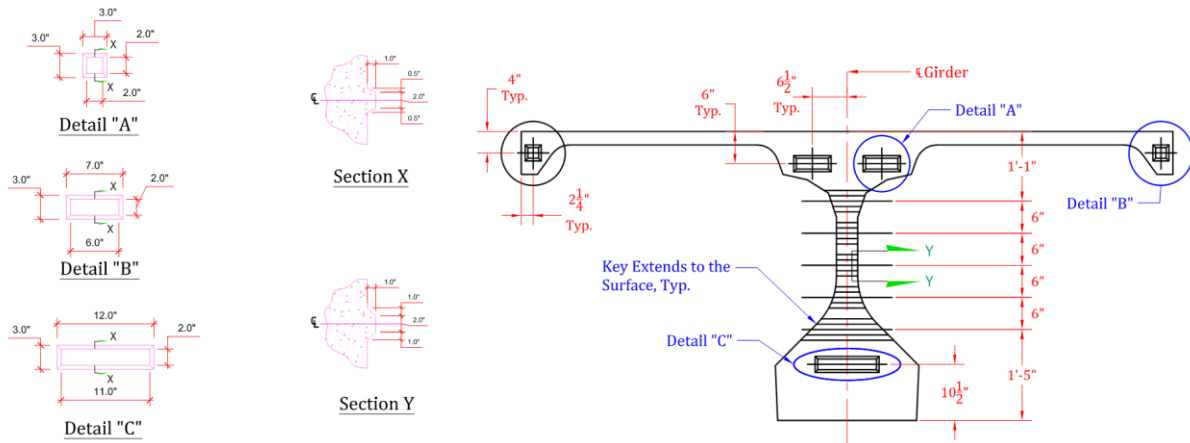


Figure 5. Shear Keys at Match-Cast Vertical Construction Joint (1 in. = 25.4 mm)

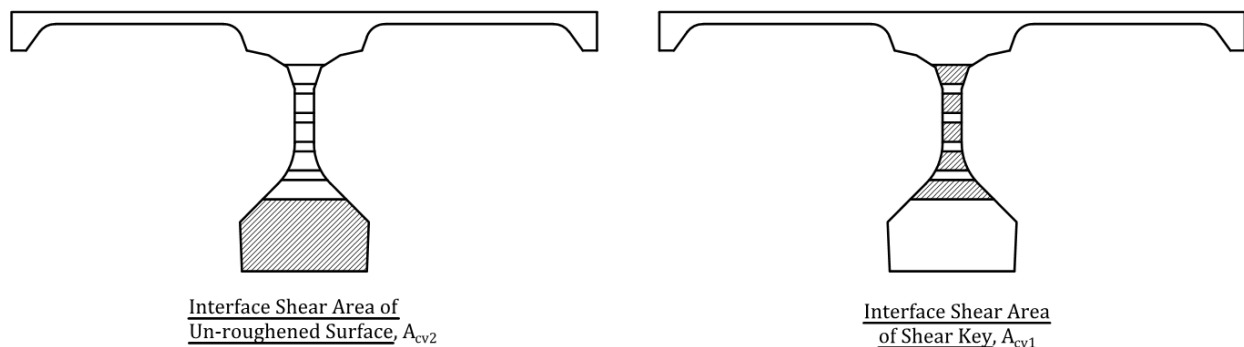


Figure 6. Contributed Interface Shear Areas in Resisting Vertical Shear at Construction Joint

5. Transverse Direction Design

A grid analysis using RISA 3D was conducted to obtain the maximum positive and negative moments at the interior transverse ribs due to one line of truck load. The area between two adjacent DIB stems is modeled using frame elements representing the transverse ribs and the longitudinal cast-in-place UHPC joint as shown in Figure 7. The 85 ft (25.9 m) span was used in the analysis. The moment continuity over the DIB stems and over the piers is simulated by fixed supports.

Loads acting on the transverse ribs are (1) Rib and CIP UHPC joint weights: the transverse ribs act as cantilever beams fixed at the DIB stem centerline and (2) SIDL and LL after the CIP UHPC joint has hardened and the transverse ribs are continuous. No. 6 (#19) ASTM A1035 CS bars, one in the top and one in the bottom of each transverse rib are spliced to the adjacent rib within the CIP UHPC joint and alone are adequate to resist the applied load. However, to ensure the top of the longitudinal joint is under compression, a 0.60 in. (15.2 mm) diameter unbonded strand in a 2 in. (51 mm) diameter duct is used in every other pair of adjacent ribs. Additionally, four No. 6 (#19) ASTM A1035 CS bars, two in the top and two in the bottom are placed along the length of the bridge in the CIP longitudinal UHPC joint.

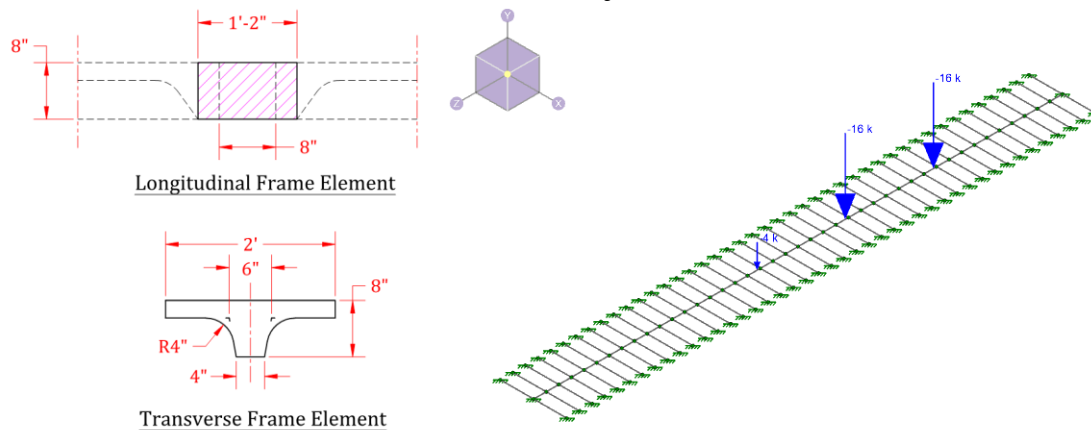


Figure 7. Grid Analysis for Transverse Ribs using RISA 3D Model.

6. Overhang Ribs Design

The open railing is designed for MASH Test Level TL-3 which has a lateral impact force of 71 kip (316 kN) at a height of 19.5 in (0.50 m) above bridge top surface. The open railing posts are centered between the ribs which results in each post being carried by two ribs. A 3D analysis using RISA 3D, similar to the model described in the previous section, was conducted to obtain the maximum moment at the transverse ribs. It was assumed that the influence zone is four rail bays on either side of the impact point. Two #6 (#19) ASTM A1035 bars are required at the top of the overhang ribs. The top bars are hooked at the edge of the flange to ensure full development.

7. Reinforcement Details

The following figure shows a 3D view of the required tendons and reinforcement for a typical interior segment and typical exterior segment.

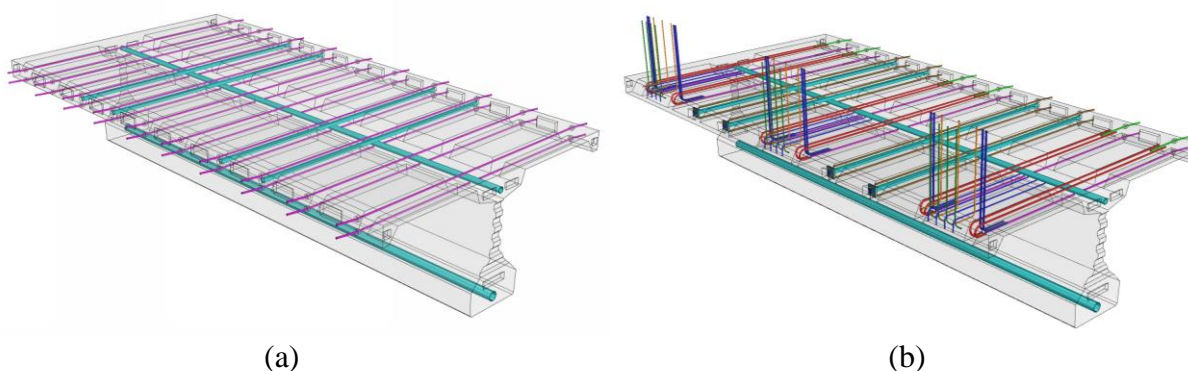


Figure 8. 3D Views for (a) Typical Interior Segment and (b) Typical Exterior Segment

8. Conclusion

Design with UHPC is still evolving as more research is completed and more projects get implemented. The authors believe this system to be highly optimized, yet conservative. Using a post-tensioned segmental match-cast system will reduce the risk taken by the precasters to produce long decked I-beam girders. The longitudinal CIP UHPC joint with reinforcement alone is adequate, however, transverse post-tensioning is provided to ensure the joint remains in compression. Shear reinforcement is minimal and only used for a limited distance at the negative moment area. Utilizing corrosion resistant ASTM A1035 CS Grade 100 bars with the power formula allows for a reduction in the number of bars required for both longitudinal continuity and in the transverse ribs, especially at the flange connection with the open railing. At vertical match-cast construction joints, the shear keys at the DIB beam web ensures the transfer of the vertical forces across the joint.

9. References

- AASHTO. 2020. AASHTO LRFD Bridge Design Specifications, 9th Edition, American Association of State Highway and Transportation Officials, Washington, DC.
- Mendonca, F., Abo El-Khier, M., Morcous, G. and Hu, J., “Feasibility Study of Development of Ultra-High Performance Concrete (UHPC) for Highway Bridge Applications in Nebraska.” No. SPR-P1 (18) M072, Nebraska Department of Transportation (NDOT), 2020.
- Nebraska Department of Transportation (NDOT): Bridge Office Policies and Procedures (BOPP) Manual. Nebraska, USA (2016)
- PCI (Precast/Prestressed Concrete Institute). 2021. “Implementation of ultra-high performance concrete in long-span precast pretensioned elements for concrete buildings and bridges.” Tadros, MK, and Lawler, J (Principal Investigators) and Mostafa Abo El-Khier, M, Gee, D, Klein, G., Kurt, A., Lucier, G. and Wagner, E. (Key Contributors), 754 pp.
- Precast/Prestressed Concrete Institute (PCI) TR-9-22 (2022) “Guidelines for the Use of Ultra-High-Performance Concrete (UHPC) in Precast and Prestressed Concrete”, PCI Concrete Materials Technology Committee, Chicago, IL.