Vertically Segmented Precast Concrete I-Girder

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ABSTRACT

For continuous span precast prestressed concrete spliced I-girder bridges, the critical location is generally at the pier due to large negative moments or large shear forces. Because of clearance requirements and lower structural forces in the positive moment zone, the optimum overall solution is often a haunched girder system where the standard prismatic girder size is deepened over the pier area to meet the relatively high forces. Also, in the negative moment zone, the bottom flange of the I-beam is much smaller than the deck slab available in the positive moment zone to resist the required compression force component of the applied flexure. Often standard I-beam shapes are produced in depths ranging up to 6 to 8 feet. Because of the need to use the standard sizes as repetitively as possible and to clear overhead obstructions during shipping, one solution is to have a separate precast haunch block and to attach it to the girder bottom flange to form a deeper section for the negative moment zone.

This paper provides a summary of extensive theoretical and experimental research on the feasibility of splicing of a haunch block onto a standard I-girder to form an efficient negative moment zone. The theory and design for the horizontal shear between the haunch block and the pier segment was verified with three types of specimens: small shear specimens, small connector pull-out specimens, and a large beam specimen, representing the pier zone of a continuous span bridge. Reinforcement details of the haunch block, the I-beam and the connection between them were evaluated for practicality and efficiency. A full-scale specimen, 68.5 ft long by 4 ft wide with a depth varying from 2.25 ft to 4.3 ft, was produced by a precast producer to investigate production and handling issues. The research has confirmed the tremendous potential of this novel system for I-girder spans up to 350 feet, without need for purchase of special forms for non-standard I-beam shapes.

Keywords: Bridge, Prestress, Spliced, I-Girder, Post-tensions, Segmental, Horizontal, Shear

INTRODUCTION AND BACKGROUND

Until a few decades ago, it was unfeasible to construct pre-stressed concrete bridges exceeding 155 feet (47 meters) or 70 tons since concrete bridge beams exceeding that height and/ or weight could not be made or shipped with the existing capacities of pre-cast concrete producers[1,2].

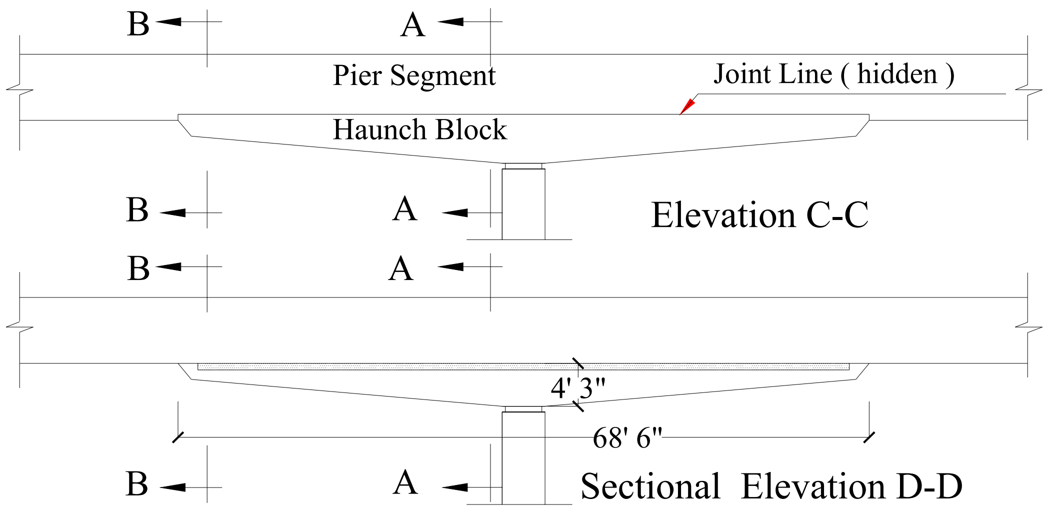
For very large spans, the critical location is generally at the pier due to large negative moments or large shear forces. The beam at the pier then needs to be deepened. One of the ways to deepen the pier segment is to have one pier segment with variable height. This results in a considerably heavier pier segment and a corresponding increase in production and transportation costs. However, this is only one of several options available to the designer[3,4].

This paper is the outcome of extensive laboratory research and a comprehensive literature review. It presents a cost-effective and aesthetically satisfactory alternative to the above option. Through the utilization of a non–prestressed haunched concrete block underneath the pier segment, a large number of relatively short, light girders, interconnected using post tensioned cables, are proven to result in longer-than-usual pre-stressed concrete bridge spans, as shown in Figure 1.

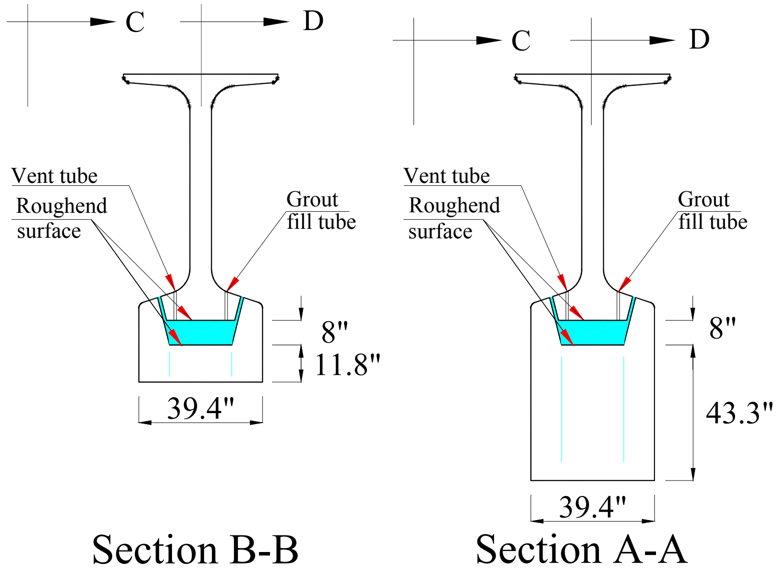
PROPOSED CONNECTION DETAILS

To connect the haunch-block with the pier segment, an 8 in. pocket is created between the two precast elements, which will be filled with a flowable concrete after installing the pier segment. Figure 1-A shows the elevation of the connection. Figure 1-B shows cross sections in the pier segment haunch block connection. Figure 1-C shows the horizontal shear reinforcement details[3].

A full-scale specimen was manufactured by two precast producers in Nebraska as shown in Figure 2. The purpose of manufacturing the specimen is to go through the production process to uncover any potential problems as well as for demonstration purposes.



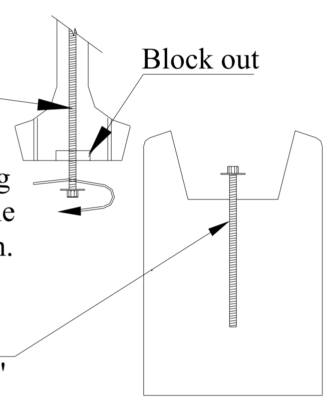
A- Elevation of Connected Haunch Block to the Pier Section



B- Concrete Cross-Sections

1.25" X 36" Lubricated coil threaded rod with welded hex nuts

to be turned before installing the pier segment such that the extension below soffit = 8 in.



1.25" threaded Rod @ 12", Staggered at 6"

C- Horizontal Shear Reinforcement Details

Figure 1 Proposed Connection Details

Figure 2 Full-Scale Specimen

DESIGN EXAMPLE

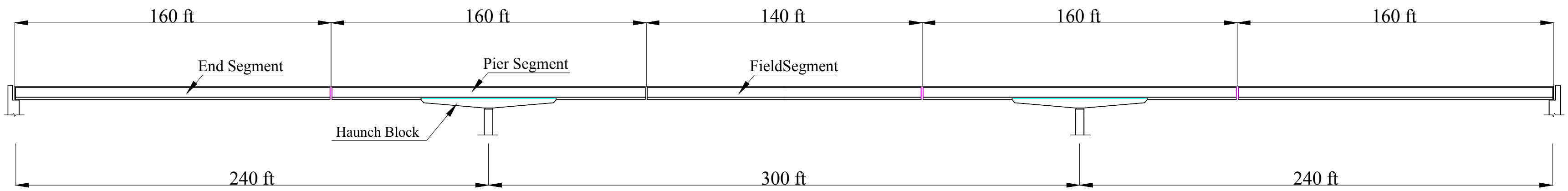
This design example demonstrates the design of a non skew bridge with three spans (240ft-300ft-240 ft) using five NU2000 beams, and two haunch blocks in the girder line, and post-tensioning, as shown in Figure 4. This example illustrates the design of a typical interior beam at the critical sections in ultimate positive flexure, ultimate negative flexure, LRFD shear, and service III at the positive moment cross section due to prestress, dead and live loading. The superstructure consists of five girder lines spaced at 9'-8" centers, as shown in Figure 3. The compressive strength of the precast beams is 10 ksi and of the CIP slab is 4 ksi. Beams are designed to act compositely with the 8-in., cast-in-place concrete slab to resist all superimposed dead loads, live loads and impact. An additional ½ in. wearing course is considered an integral part of the 8-in. slab. The design is in accordance with LRFD Specifications[5].

Prestress Force

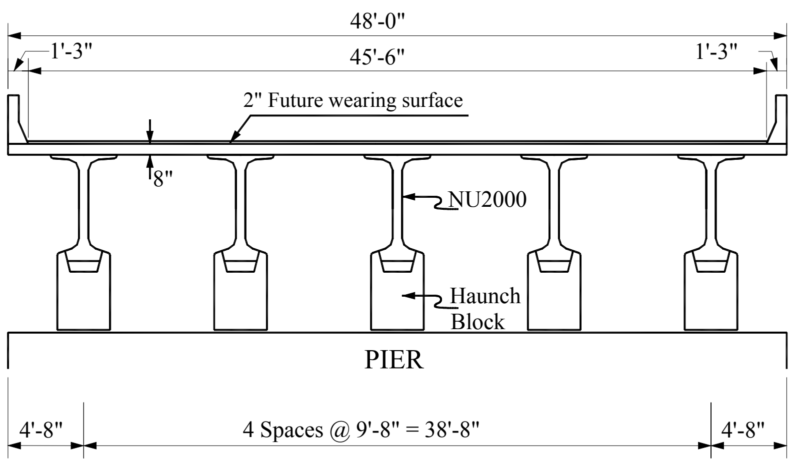
Post-tensioning is applied at only one stage after casting the wet joint between segments. Three 3.75 in., diameter ducts are used in the calculations. Each duct contains 15-0.6 strands. The post-tensioning profile is shown in Figure 4. The pre-tensioning is 46-0.6 strands only in the field segments.

Shear Forces and Bending Moments

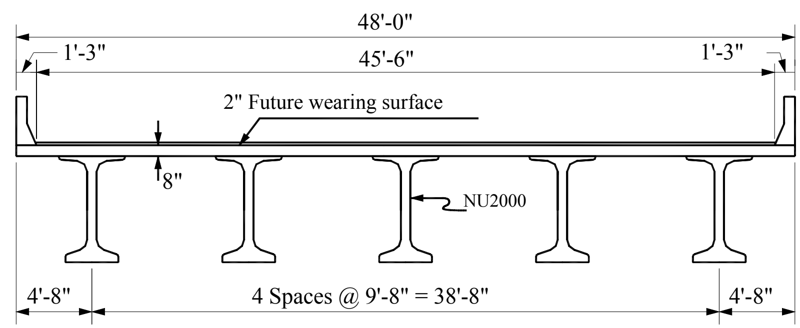
The shear forces and the bending moments due to prestress, dead and live loading are shown in Table 1. The live load distribution factors are calculated based on the LRFD equations without the span upper limit of 240 ft. These distribution factors are calculated based on 10 ft girder spacing and an average span length of 270 ft.



Bridge Elevation



Bridge Cross Section at Pier



Bridge Cross Section at Mid-Span

Figure 3 Design Example Elevation and Cross-Sections

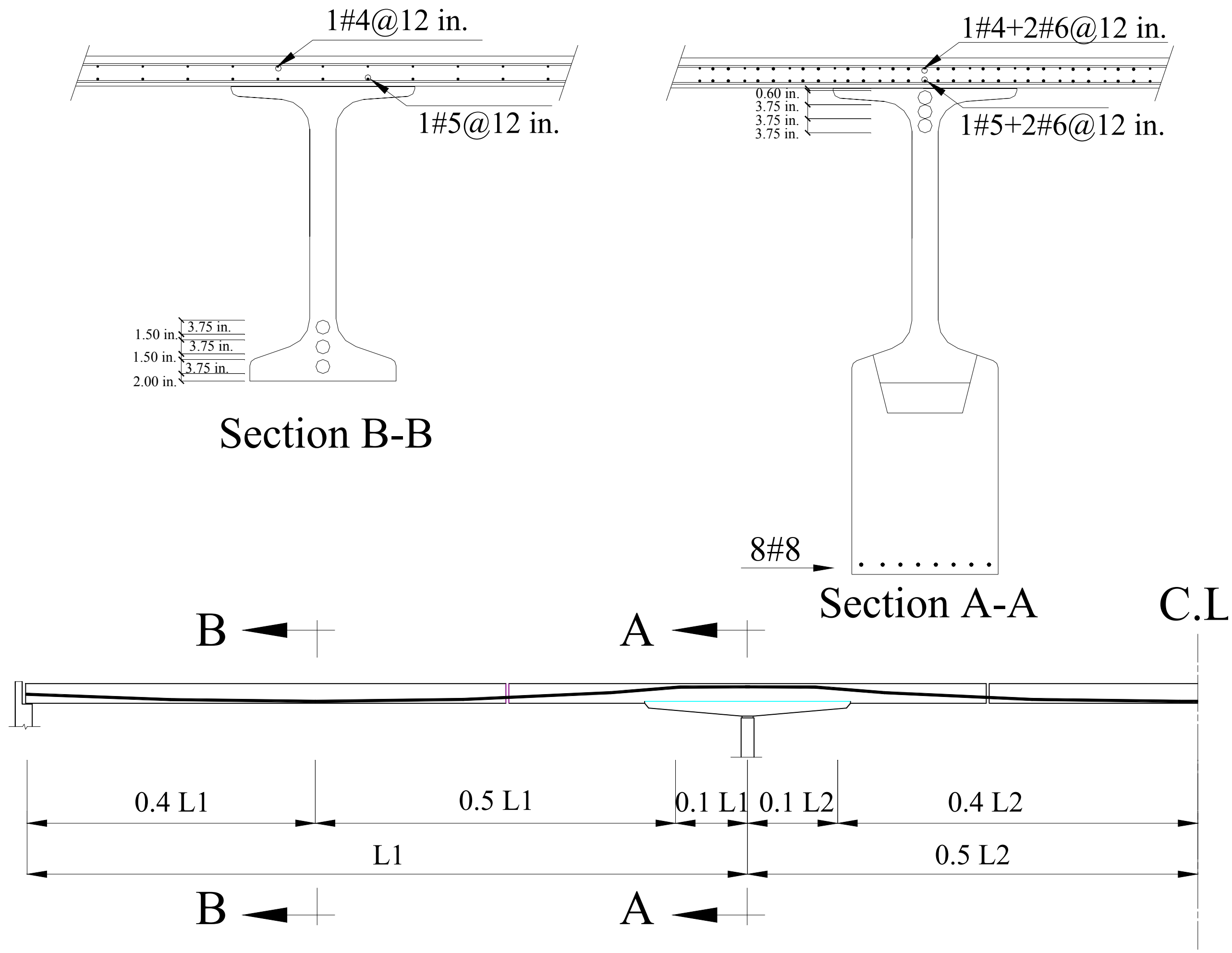


Figure 4 Design Example Post-Tensioning Profile

Table 1 Unfactored Shear Force and Bending Moments for a Typical Interior Girder

	Positive Bending Moment at 0.4 L (k-ft)	Negative Bending Moment at Haunch Block End (k-ft)	Negative Bending Moment at Pier C.L. (k-ft)	Shear Force at 13 ft from the Pier C.L.* (kips)
Girder Weight	3,552.1	-4,290.5	-9504.1	164.9
Deck Slab	3,248.2	-3,949.5	-8079.7	128.4
Wearing Surface	866.2	-1,053.2	-2154.6	34.3
Barrier	415.8	-505.5	-1034.2	16.4
Live Load	5,401.3	-4,842.8	-7329.7	181.6
Post-Tensioning[6] Total Moment	-3,528.4	6,093.3	11,743.8	------
Post-Tensioning Secondary Effect[6]	318.6	687.0	797.3	0.0

* The shear force critical section is located at 13 ft from the pier center line at the second span

Capacities of the Critical Sections

The stress at the NU I-beam bottom flange at 0.4 L_1 from the first span is -0.08 ksi tension due to service III. The LRFD allowable tensile stress is 0.6 ksi[3].

The strength limit state design is summarized in Table 2. The horizontal shear is 82 klf at the pier centerline.

Table 2 Critical Sections Shear Force and Bending Moments Capacities

	Positive Bending Moment at 0.4 L (k-ft)	Negative Bending Moment at Haunch Block End (k-ft)	Negative Bending Moment at Pier C.L. (k-ft)	Shear Force at 13 ft from the Pier C.L. (kips)
LRFD Due to Factored Load[5]	19,771.6	-20,986.6	-39,331.3	756.4
Section Capacity ($Ø M_n$ & $Ø V_n$)[5,7]	23,700.2	-21,368.0	-39,952.5	1,428.4*

* The maximum shear capacity is calculated based on the equation $\phi V_n = 0.9 f_c^{'} b_v d_v$

EXPERIMENTAL INVESTIGATIONS

Three types of tests were done to verify the composite action between the two precast pieces, the haunch block and the I-girder, and to estimate the capacity of the proposed horizontal shear reinforcement details between these two precast pieces. The first type was push-off tests, which were done on two groups of reinforcement. The other two types were the pull out tests and the full scale test[3].

Push-Off Tests

Seven push-off specimens in two groups with different heights and different numbers of rods were used. The first group tested a 1-1/2 in. (38.1 mm) diameter mild steel coil bolt and included two specimens. The second group tested a 1-1/4 in. (31.75 mm) diameter hard steel coil treaded rod and included five specimens.

Each specimen had two precast pieces. An 8 in. (200mm) pocket was created between these two precast pieces, containing part of the horizontal shear reinforcement. The pocket was cast with flowable concrete afterward. The geometry of that pocket had the same dimensions as those proposed in the real system. The first specimen in the first group had double the pocket size [16 in. (400 mm)], and its horizontal shear reinforcement was not staggered.

The results of the push-off tests demonstrated the ability of the proposed connection details between the hunch block and the pier segment to resist the 82 k/f horizontal shear force at the interface, calculated in the previous example.

The Pull-Out Test

The NU I-beam bottom flange with was simulated by a pull-out specimen. A 7 in. (175 mm) concrete stem also simulated the web of the post-tensioned NU I-beam. The objective of this test was to estimate the maximum pull-out force using the new system-- the lubricated coil rod[3].

The test results of the pull-out using 1-1/4” lubricated coil rods were satisfactory and larger than 82 k/ft, which is required in the design example.

Full-Scale Test

The specimen consisted of two precast pieces connected together by a horizontal concrete joint. The two precast pieces were an I-beam and a haunch block, similar to the real system except that the I-beam was an Iowa type A instead of an NU I-girder. The haunch block was located at the top of the I-beam. The specimen was simply supported from both ends and was loaded at its midpoint, as shown in the longitudinal section in Figure 5. The midpoint

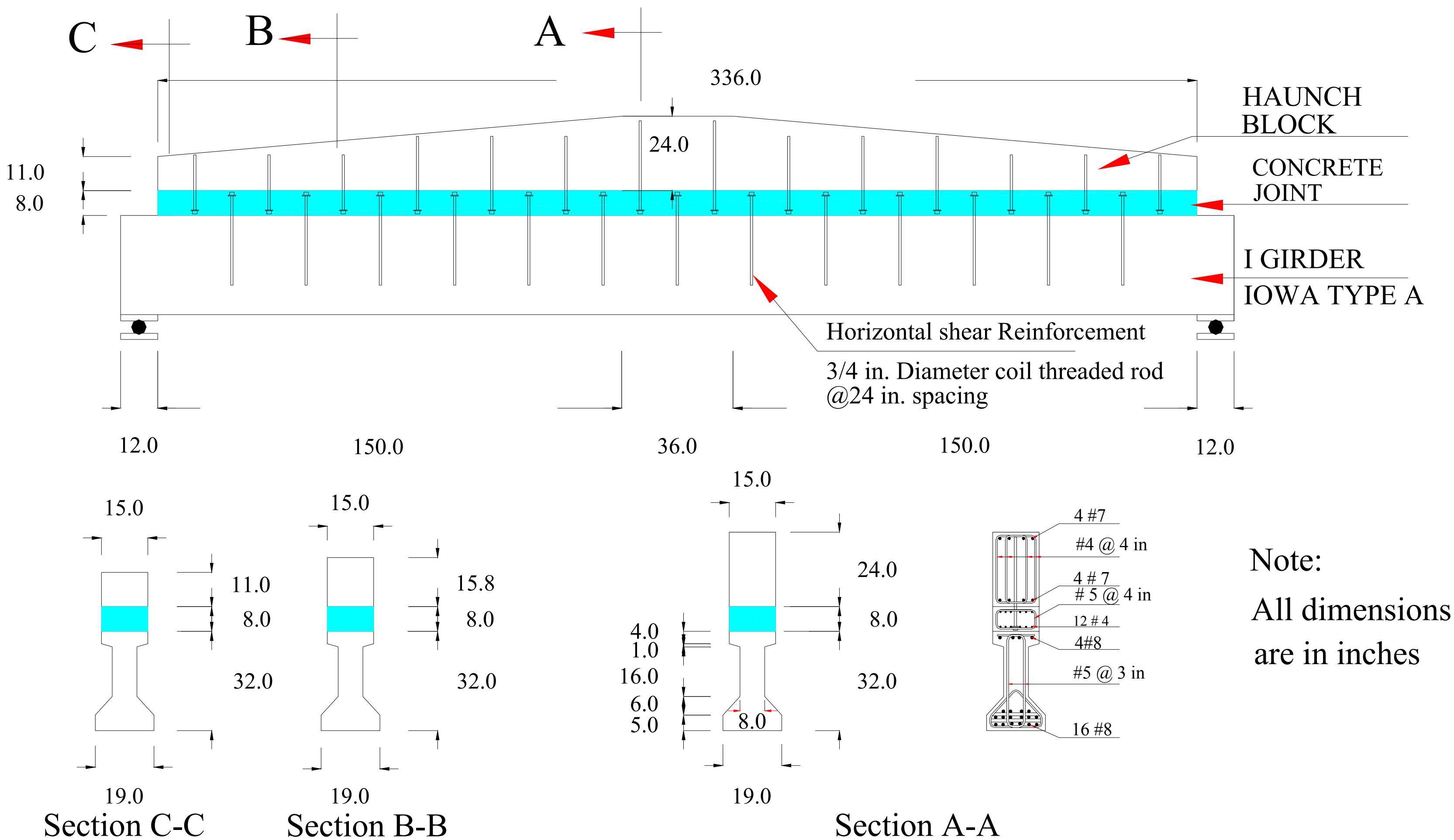


Figure 5 Full Scale-Test Concrete Dimensions and Reinforcement Details

simulated the pier reaction and the two end-supports simulated the two field segments reactions[3].

The results of the full scale test were satisfactory**.** It is recommended to use Loov's Patnaik's (1994) equation[8], since it gives closer failure loads than the LRFD equation to the obtained test results..

Discussion

This paper introduces a well-researched, cost-effective, and aesthetically appealing revolution in extending precast concrete bridge spans. Through extensive theoretical and experimental research, it has been shown for the first time that connecting a haunch block to a standard I-girder in the pier segment results in extending the spans of prestressed concrete bridges to up to 350 ft, while meeting the limitations of shipping and handling capacities. This approach efficiently substitutes for a customized, deepened pier segment while optimizing the negative and the positive moment capacities.

The presented innovative horizontal shear reinforcement uses 1-1/4 in. lubricated coil threaded rods, inserted in the form before casting the concrete, and then later turned to protrude 8 in. into the pocket. The reinforcement allows for utilization of standard forms, which enhances the cost efficiency of the proposed system.

The proposed system was progressively tested using seven push-off specimens, a pull-out specimen, and a full-scale specimen. Based on the experimental results Loov's Patnaik's (1994) equation is recommended over AASHTO LRFD, as the latter was found to be unnecessarily conservative.

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