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Extending Spacing of Horizontal Shear Connection Pockets from 24 to 48 inches for Steel Girders

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Introduction

Creating composite action between the precast deck panels and supporting girders has been one of the challenges that faced the design engineers in design of precast concrete bridge deck panel systems. Shear pockets over the girder lines have to be created in the panel to accommodate the shear connectors extending from the supporting girders. Also, the shear connectors have to be clustered in groups lined up with these pockets. The AASHTO LRFD Specifications[1] state that spacing between the shear connectors for steel girders should not exceed 24 inches.

For a bridge owner, it is advantageous to reduce the number and size of the shear pockets for the following reasons:

- To simplify and speed up the fabrication process of the panels. Forming of the shear pocket typically slows down the fabrication process of the panels and eventually raises the fabrication cost.
- To reduce the volume of the non-shrink grout used to fill the shear pockets, which results in reducing the cost of the deck panel system and increasing the construction speed, especially for over night deck replacement projects.
- To reduce the possibility of water leakage at the interface of the shear pocket and the grout filling it.
- To give the design engineer more flexibility in laying the transverse reinforcement of the panel.

This paper presents a summary of the experimental program that has been used by the researchers of the NCHRP 12-65 research project titled "Full-Depth, Precast-Concrete Bridge Deck Panel Systems"[2] to extend the maximum stud spacing to 48 inches. The paper also provides a summary of the history of the 24-in. stud spacing limit of LRFD Specifications, recent research activities regarding the fatigue and shear strength of stud clusters, and the benefits of using large size studs with precast concrete deck panel systems.

Background

(1) History of the Shear Connector Spacing Limit of LRFD Specifications:

The first composite concrete slab on steel I-beam bridges in the United States was constructed in the early to mid 1930s in Iowa. A composite bridge design example, prepared as part of a paper by Newmark and Siess[3] in accordance with the 3rd Edition of the American Association of State Highway Officials (AASHO) Standard Specifications for Highway Bridges (1941), states "*the spacing of the shear connectors shall be not more than 3 to 4 times the depth of the slab*". While this limit did not appear in the AASHO provisions, it appears to have been used as a convention or rule-of-thumb. Newmark and Siess[2] recognized in their text that while these connectors are generally only designed to transfer horizontal shear that they also play a dual role of preventing the separation of the beam and the slab.

The 24-in. maximum limit on shear connector spacing first appeared in the 4th Edition of the AASHO Standard Specifications for Highway Bridges in 1944. This requirement appears without commentary, which was typical of that era and the source of this change was not given. It may be attributed to the research work produced in the late 1930s and early 1940s in Germany. However these reports were all published in German and a free exchange of information was hardly present at that time.

A 1953 paper by Viest and Siess[4] contains a discussion of why mechanical connectors are needed. Their arguments include: (1) to prevent relative movement (either horizontal or vertical) between the beam and the slab during all loading levels up to ultimate and (2) to transfer horizontal shear from the slab to the beam. The discussion that supports these roles for shear connectors is primarily directed at insuring linear-elastic behavior of the composite system.

Viest and Siess returned to this subject in a 1954[5] paper that reports conclusions made from their experimental results and made design recommendations. It should be noted that these experiments where carried out using the channel-type shear connectors that were conventional at the time. Although they did not comment on the origin of the 24-in. maximum connector spacing in the AASHO provisions, the experimental results support retaining the limit. The testing considered connector spacing of 18 in. and 36 in. While the 18-in. spaced connectors performed as necessary, the 36-in. spaced connector specimens experienced some lift-off between connectors under load in the experiments. This result motivated the authors to recommend that *"the maximum spacing of channel shear connectors be not greater than four times the thickness of the slab, but in no case greater than 24 inches."*

Further investigation has revealed that when the headed stud shear connector became available to the steel bridge construction industry in the late 1950s, the steel industry people relied on Viest and Siess to help formulating the design provisions for these connectors that were eventually incorporated into the AASHTO specifications in the early 1960s. Based on their previous work,[4,5] Viest and Siess again recommended a limit of 24 in. maximum spacing for these provisions. This time frame also coincides with industry acceptance of precast/prestressed concrete girders as an alternative to steel girders for highway bridge construction.

The majority of research conducted in the 1970s-1990s continued to adopt the 24-in. limit. The research during this period concentrated on using the headed steel studs and investigating the following related issues: stud size and spacing, welding mechanism and quality control, and fatigue and design capacities. It is important to note that all of the research activities conducted between 1950 and early 1990s were for the single headed stud system that is used with cast-in-place slabs, where the studs are distributed across the span length and are not clustered in groups as the case when they are used with precast concrete deck panel systems.

(2) Recent Research Activities on Clustered Headed Studs:

Recently two research projects have investigated the fatigue and ultimate shear capacity of studs grouped in clusters. These attempts came as a result of the increasing interest in the United States for using precast concrete deck panels on bridges[6].

In 2003, the effect of number of studs per cluster on the shear strength was studied by Issa et al[7]. In this research quarter- and full-scale push-off specimens were made with various configurations of studs; two to four 7/8 in. diameter studs. The objective of the research was to investigate the feasibility of using Equation 6.10.10.4.3-1 of the LRFD Specifications[1], which is used for single studs, to be used for estimating the shear strength of a cluster of studs. The research concluded that the increase in ultimate strength of a cluster of studs was not linearly proportional to the number of studs. The research stated that for all specimens an initial slippage of about 0.02 in. was noticed before the studs started to initiate the composite action, and that shear failure was recorded at the stud base. The failure was accompanied with local cracking and crushing of the concrete close to the stud base. Once the concrete at the stud base crushed, the stud lost it's bearing support and started to act as a free cantilever, which finally lead to the shear failure at their base. The research reported that Equation 6.10.10.4.3-1 of the LRFD Specifications[1] overestimates the shear strength of stud clusters by as much as 22 percent. Also, the research reported that Equation 5.8.4.1-1 of the LRFD Specifications[1] correlates very well with the test results if used to estimate the shear strength of stud clusters. This conclusion was drawn based only on testing of push-off specimens and was not confirmed by any full-scale beam test. It is important to note that the effect of fatigue load on the shear strength was not investigated as the push-off specimens were tested directly for ultimate without exposing them to a fatigue load.

US Interstate 39/90, Door Greek Project, Wisconsin Department of Transportation used a precast deck panel system, where 48 in. spacing was used between clusters of 7/8 in. diameter studs. The decision of violating the maximum spacing limit given by the AASHTO LRFD Specifications was based on the experimental investigation conducted by Markowski et al[8], where a half-scale composite beam was tested. One-half of the beam length utilized 24 in. cluster spacing and the other half utilized 48 in. cluster spacing. The test results have shown that full composite action was achieved under full service load and no signs of stiffness deterioration were noticed after applying 2,000,000 cycles of repeated loading. The beam continued to show full composite action when it was overloaded beyond the service load level, however, the researcher could not test the beam at ultimate due to the limited capacity of the loading frame.

Careful studying of these attempts reveals that:

- Stud clusters may not be able to produce their shear strength as given by Equation 6.10.10.4.3-1 of the LRFD Specifications[1].
- Equation 5.8.4.1-1 of the LRFD Specifications[1], although it is provided in Article 5 of the specifications that covers design and analysis of concrete structures, correlates very well with the test results of push-off specimens, if used to estimate the shear strength of stud clusters.
- The effect of fatigue load on the shear strength was not investigated. In a real bridge, there is a fair chance that the studs will be exposed to a large number of live load cycles before the bridge is overloaded and the studs are loaded up to their maximum strength.
- Extending the maximum spacing to 48 in. has no negative effect on the fatigue capacity of clustered 7/8 in. studs.
- None of these attempts was able to investigate simultaneously the fatigue and ultimate capacity of clustered studs.
- Both attempts used ¾ and 7/8 in. diameter studs.

(3) Fatigue and Ultimate Capacities of Steel Studs per AASHTO LRFD Specifications[1]:

Fatigue Capacity: Equation 6.10.10.2-1 of the LRFD Specifications[1]

$$Z_r = \alpha d^2 \geq \frac{5.5}{2} d^2 \qquad (1)$$

$$\alpha \text{ (ksi)} = 34.5 - 4.28 \log (N) \qquad (2)$$

Where: Z_r = fatigue resistance force of shear connector (kips)

d = stud diameter (in) = 1.23 in

N = number of cycles

Ultimate capacity: Equation 6.10.10.4.3-1 of the AASHTO REFD Specifications[1]:

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \leq A_{sc} F_u \qquad (3)$$

where: Q_n = nominal capacity (kip)

A_{sc} = cross sectional area of 1¼ in. stud (in^2)

$f_c^{'}$ = compressive strength of the concrete surrounding the stud (ksi)

E_c = modulus of elasticity of the concrete surrounding the stud (ksi) = 5,645 ksi

F_u = ultimate tensile strength of the stud material (ksi) = 64 ksi

Equation 5.8.4.1-1 of the LRFD Specifications[1]:

This equation is derived based on the shear friction theory and is commonly used for the design of horizontal shear reinforcement for slab/concrete girder composite beams. However, the LRFD Specifications give values for c and μ if steel beams are used.

$$V_n = c\,A_{cv} + \mu\,A_{vf}\,f_y \qquad (4)$$

Where: c = cohesion factor, 0.025 ksi for concrete anchored to as-rolled structural steel by headed studs (LRFD Art. 5.8.4.2)

μ = friction factor, 0.7 for concrete anchored to as-rolled structural steel by headed studs (LRFD Art. 5.8.4.2)

A_{cv} = area of concrete engaged in shear transfer, plan area of the shear pocket

A_{vf} = area of shear reinforcement crossing the shear plane

f_y = yield strength of the horizontal shear reinforcement, 54.0 ksi for SAE1018 steel

(4) Large Size Steel Studs:

The 3/4 in. and 7/8 in. diameter studs are the most common sizes used in steel bridges due to the heavy superimposed dead and live loads exist on bridges. In high shear areas of steel girder bridges, as many as four 7/8 in. diameter studs per row are used to satisfy design requirements, as shown in Figure 1. The relatively high number of studs has many disadvantages especially when used with precast concrete deck panel systems. This is because large size shear pockets have to created that: (1) add restraints on distributing the transverse reinforcement of the panel and (2) consumes a large volume of non-shrink grout, which increases the total cost of the deck system.

In 1997, a group of researchers developed the 1¼ in. diameter stud, as shown in Figure 2. The stud development was initiated on the NCHRP 12-41 research project titled "Rapid replacement of Bridge Decks"[9] and then the stud was used on demonstration bridges in Nebraska[10,11,12]. Since one 1¼ in. stud replaces two 7/8 in. studs, use of the 1¼ in. studs with precast deck panels results in: (1) reducing the shear pocket size and the volume of the filling grout by as much as 45 percent and (2) increase the speed of construction by reducing the time required to weld the studs. For these reasons, the 1¼ in. diameter stud has been considered in the experimental investigation of the NCHRP

12-65 research project titled "Full-Depth, Precast-Concrete Bridge Deck Panel Systems"[2].

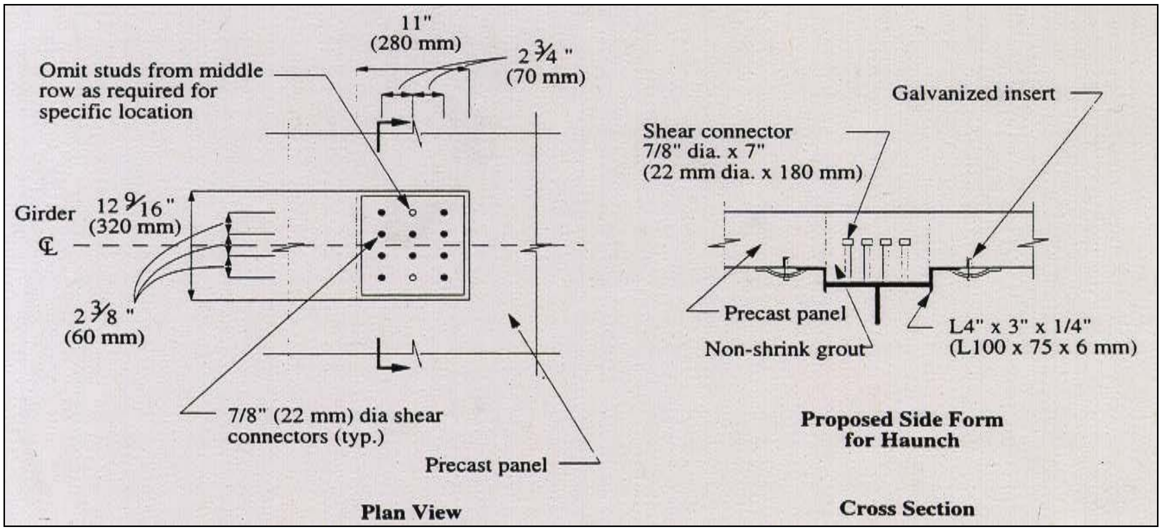


Figure 1. Details of the panel-to-girder connection used on Queen Elizabeth Way-Welland River Bridge, Ontario, Canada[2]

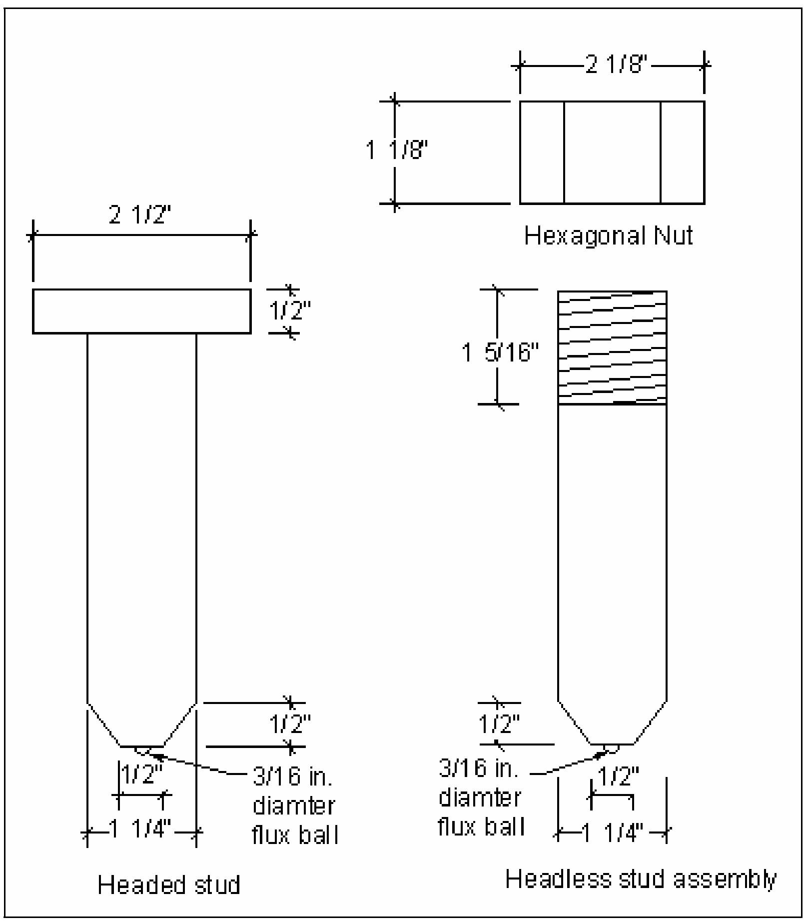


Figure 2. Dimensions of the 1¼ in. diameter stud

Experimental Program of the NCHRP 12-65[2]

The experimental program that has been envisioned in the NCHRP 12-65 to investigate the possibility of extending the stud cluster spacing to 48 in. consists of two groups of push-off specimens and full-scale beams. Only 1¼ in. diameter studs were used in all specimens.

(1) Push-off Specimens

Two groups of push-off specimens were fabricated and tested. Group #1 consisted of eight specimens tested for ultimate. Group #2 consisted of eight specimens exposed to 2,000,000 cycles of fatigue load and then tested for ultimate. Figures 3 to 6 show the details of the specimens. Figure 7 shows the specimens during fabrication and Figure 8 shows the test set up.

(2) Full-scale Beam Specimens

Two full-scale composite beams, 32-ft long each, were fabricated. The beams were identical except that the spacing between the stud clusters was 24 in. for the Beam #1 and 48 in. for the Beam #2. Each composite beam was made of 8-in. thick precast slab supported by a W18x119 steel beam. The slab and the steel beam were made composite using 64- 1¼ in. studs over the full span length. The studs on the Beam #1 were clustered in 16 groups spaced at 24 in., 4 studs per group. The studs on the Beam #2 were clustered in 8 groups spaced at 48 in., 8 studs per group. The spacing between the studs in each group was 3 in. in the longitudinal direction. Two studs per row spaced at 5 in. in the transverse direction were used. In each beam, the stud clusters on the south half of beam were confined with hollow structural steel tubes (HSS), and the stud cluster on the north half were confined with individual closed ties. The concrete slab of each beam was made of one precast panel, which was reinforced with two welded wire reinforcement (WWR) meshes. The top mesh was made of 6x6 in.-D10xD10, and the bottom mesh was made of 6x6 in.-D14xD14. This amount of reinforcement was provided in accordance with the minimum reinforcement requirements of the Empirical Design Method given in Article 9.7.2 of the AASHTO LRFD Specifications[1]. Figures 9 to 11 show the details of the full-scale beams. Figure 12 shows various steps of building the composite beams.

The beams were loaded with fatigue load for 2,000,000 cycles by applying one concentrated load at the midspan section of each beam as shown in Figure 13. This setup put all of the shear pockets under the same horizontal shear force. Finally, each half of each beam was tested as simply supported beam to ultimate, as shown in Figures 14 and 15.

The test results and conclusions of the experimental investigation and final recommendations were submitted to the project sponsor in summer 2006 and are expected to become publicly available by mid 2007.

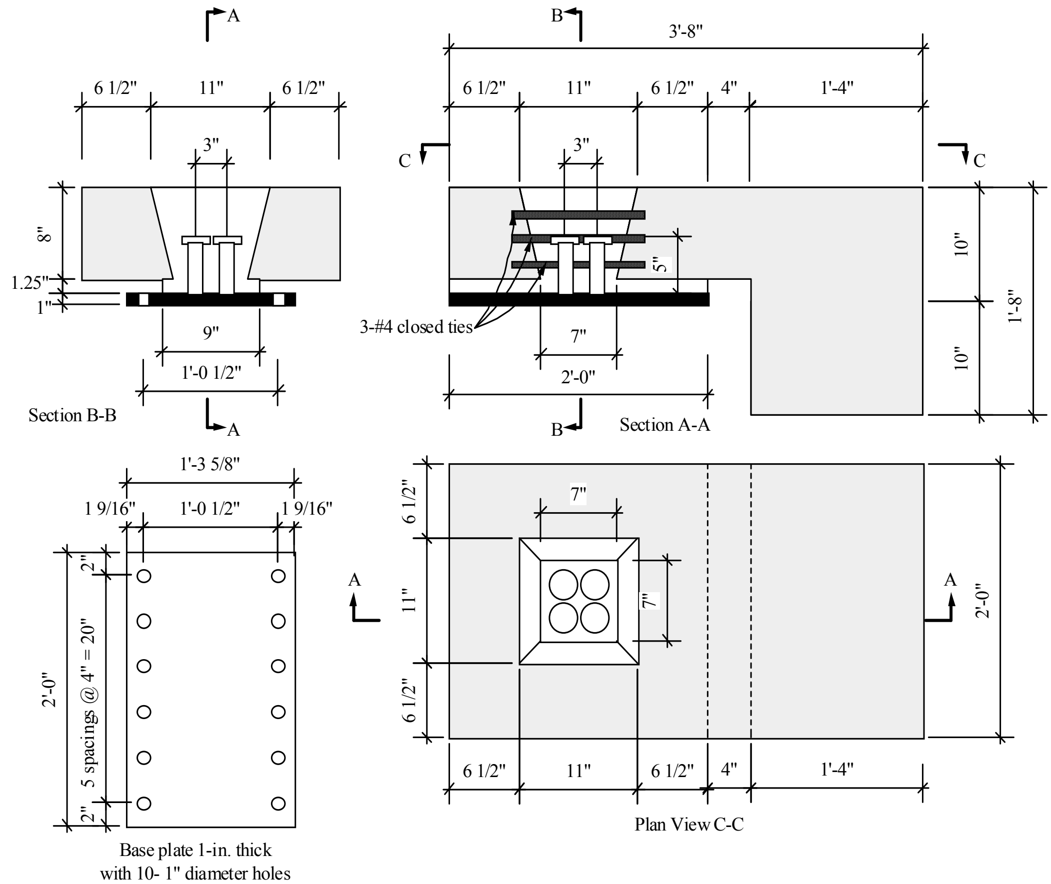


Figure 3. Concrete dimensions of the four-stud, closed-ties confinement specimen

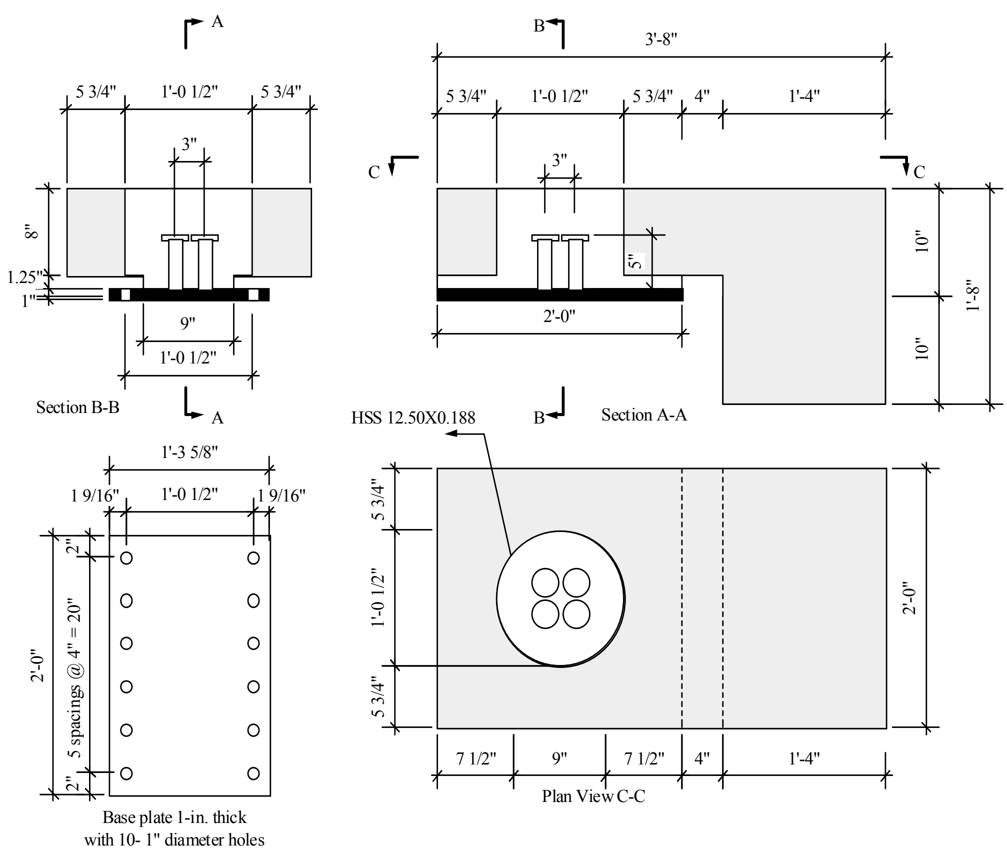


Figure 4. Concrete Dimensions of the four-stud, steel-tube confinement specimen

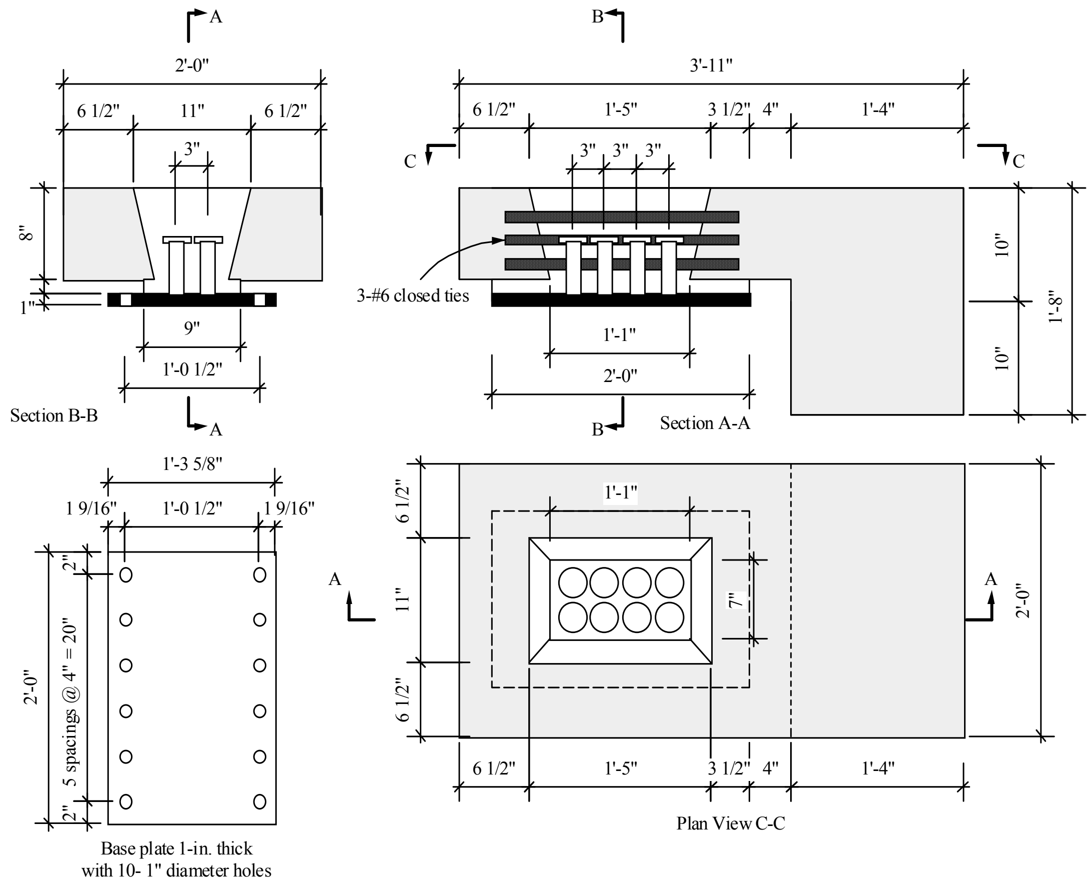


Figure 5. Concrete Dimensions of the eight-stud, closed-ties confinement specimen

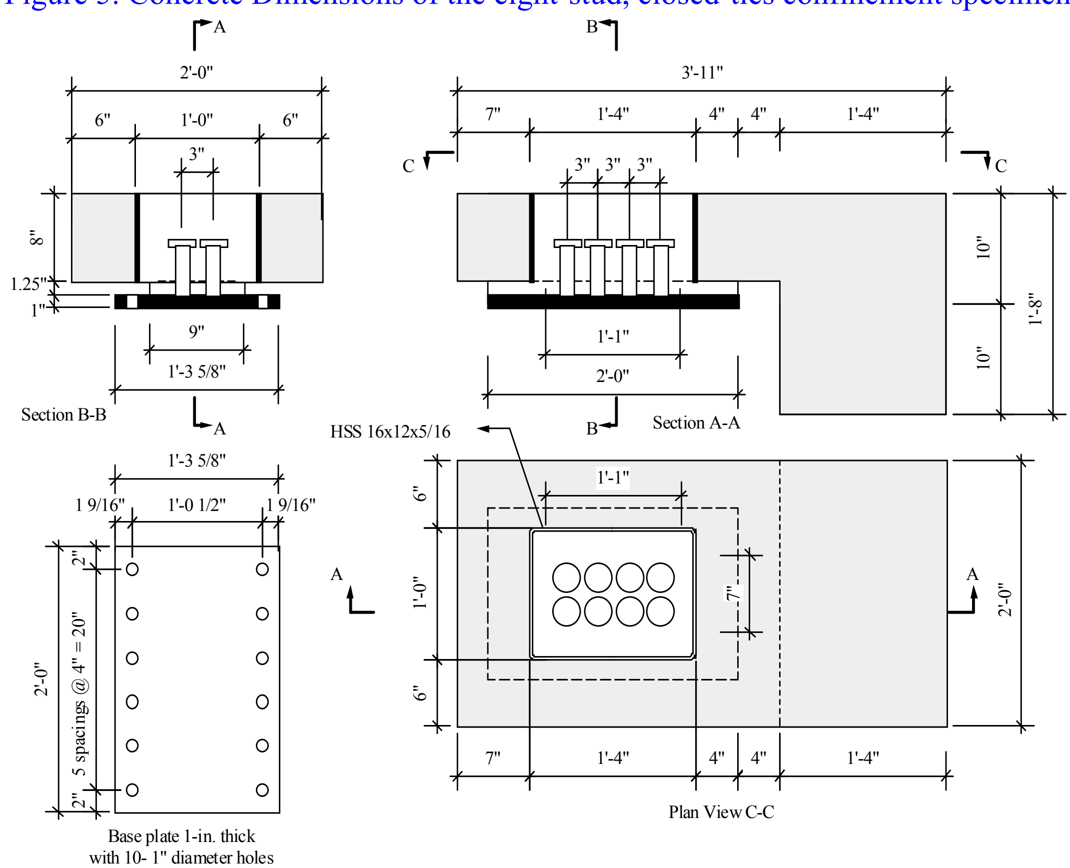
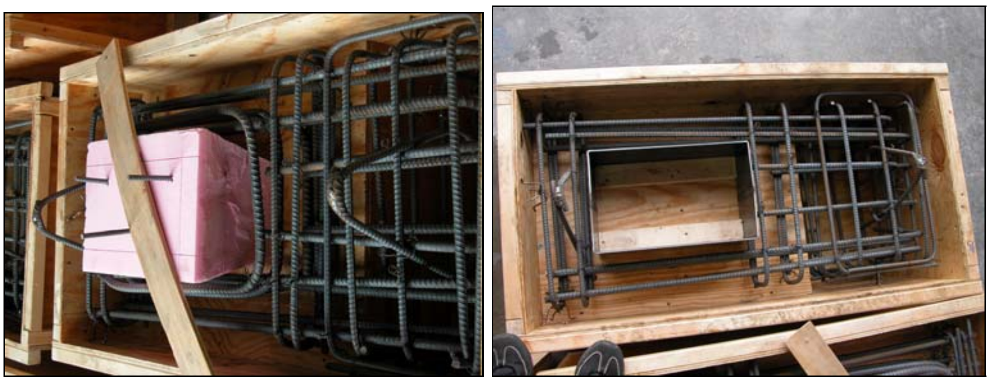


Figure 6. Concrete Dimensions of the eight-stud, steel-tube confinement specimen

Four-stud specimens



Eight-stud specimens

Figure 7. Fabrication of the push-off specimens

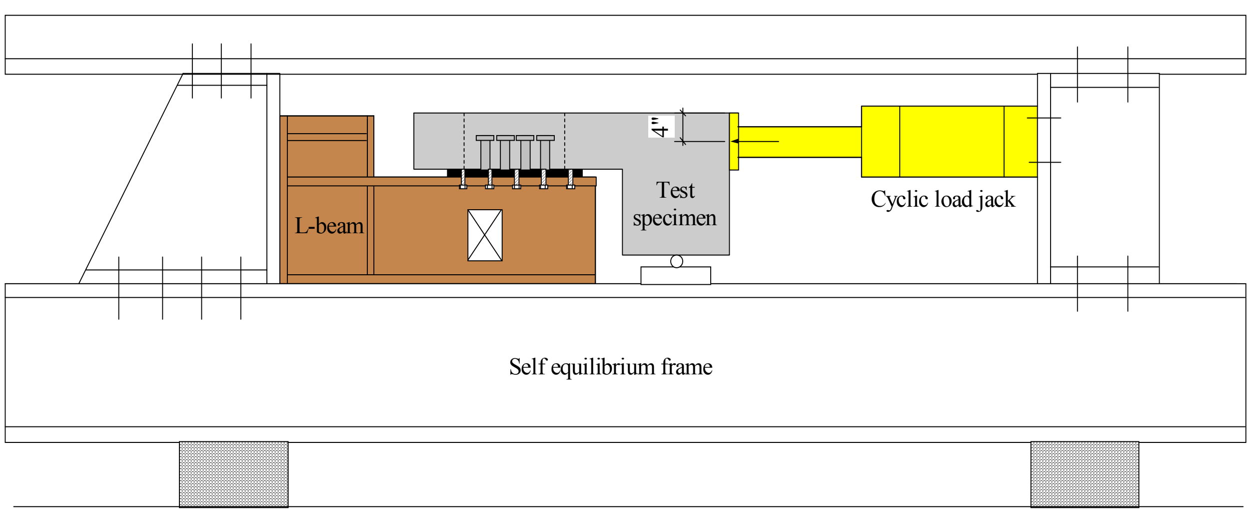


Figure 8. Test setup of the push-off specimens

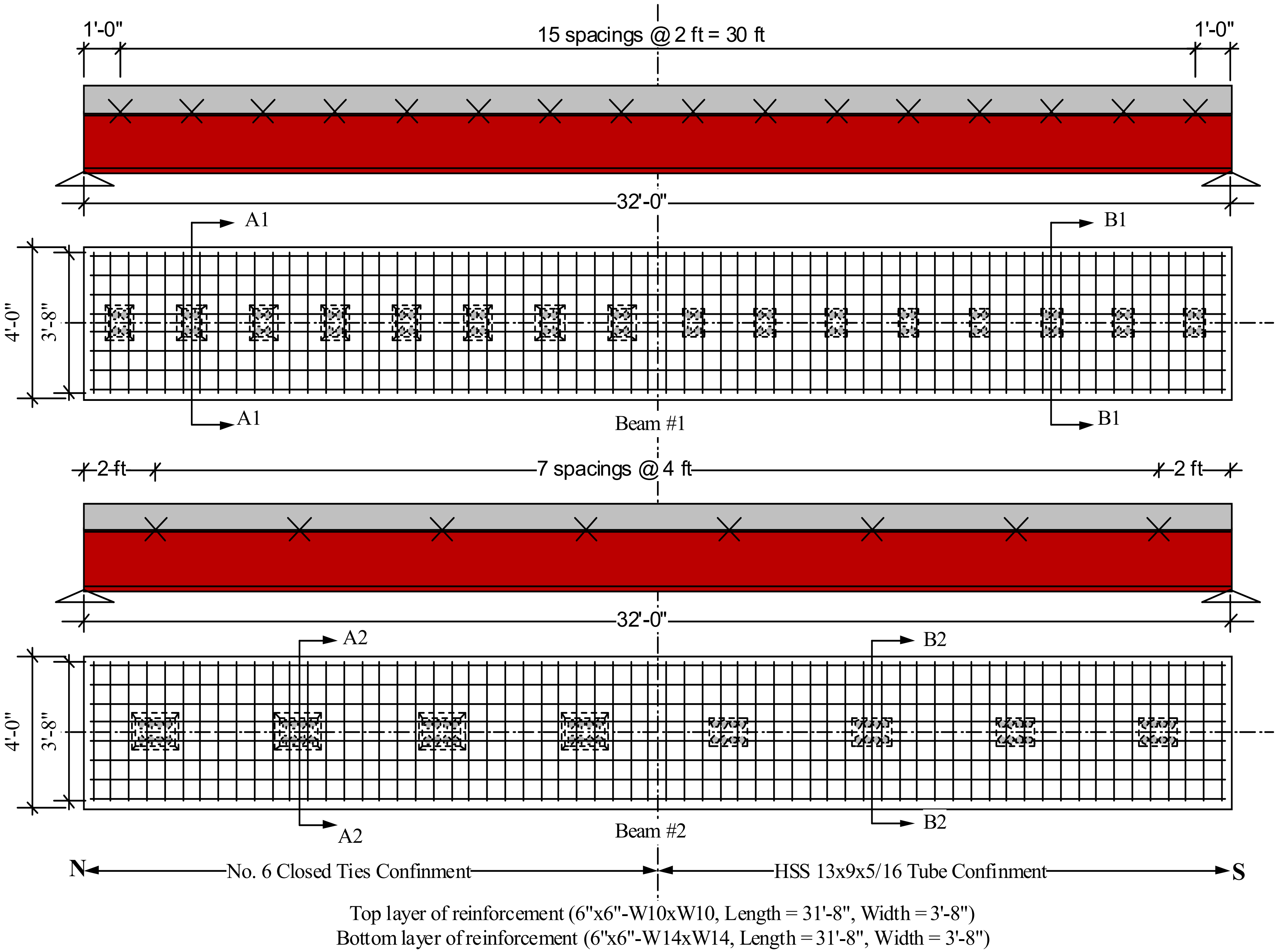


Figure 9. Arrangement of the stud clusters of Beam #1 and #2

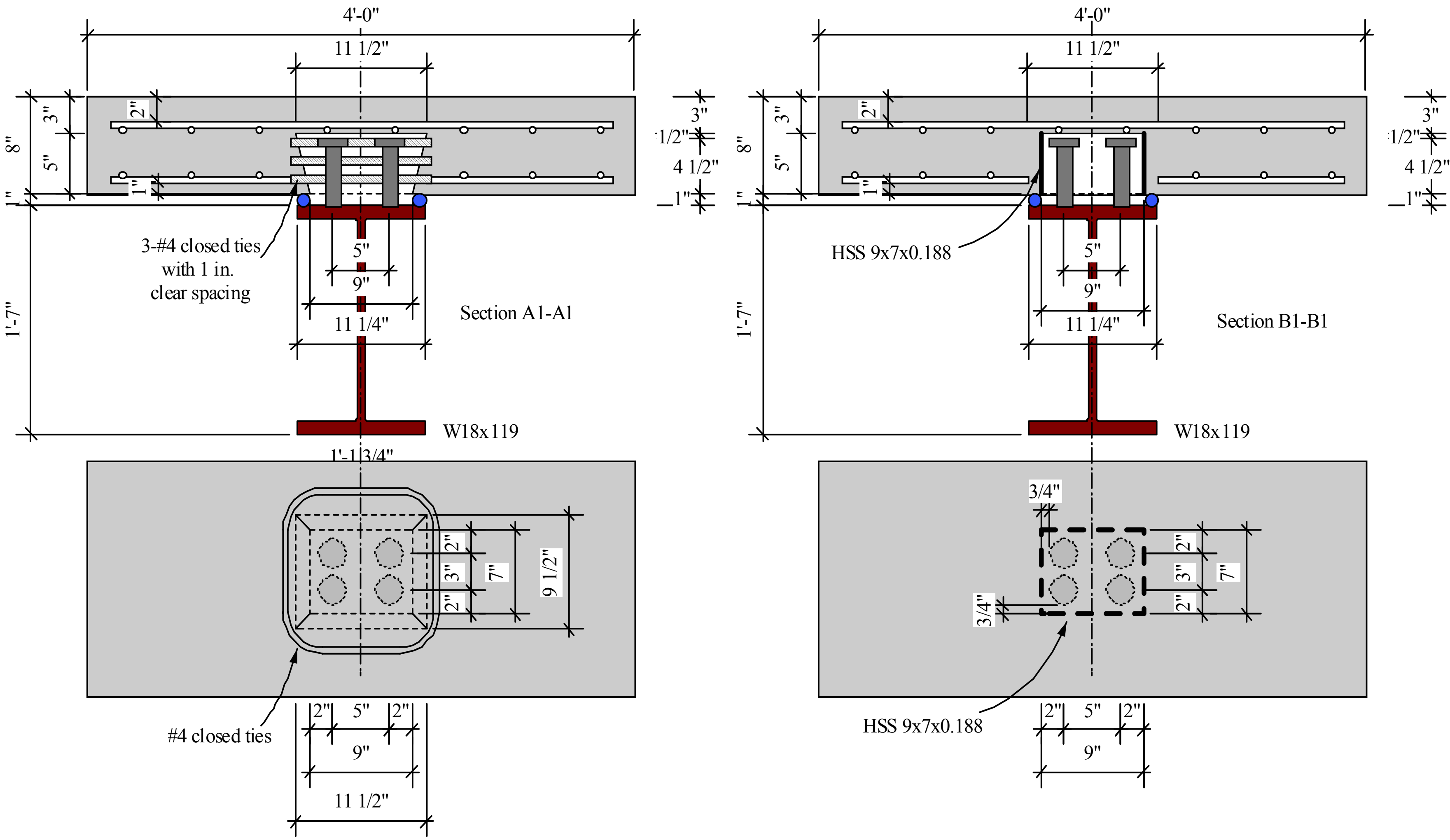


Figure 10. Sections A1-A1 and B1-B1

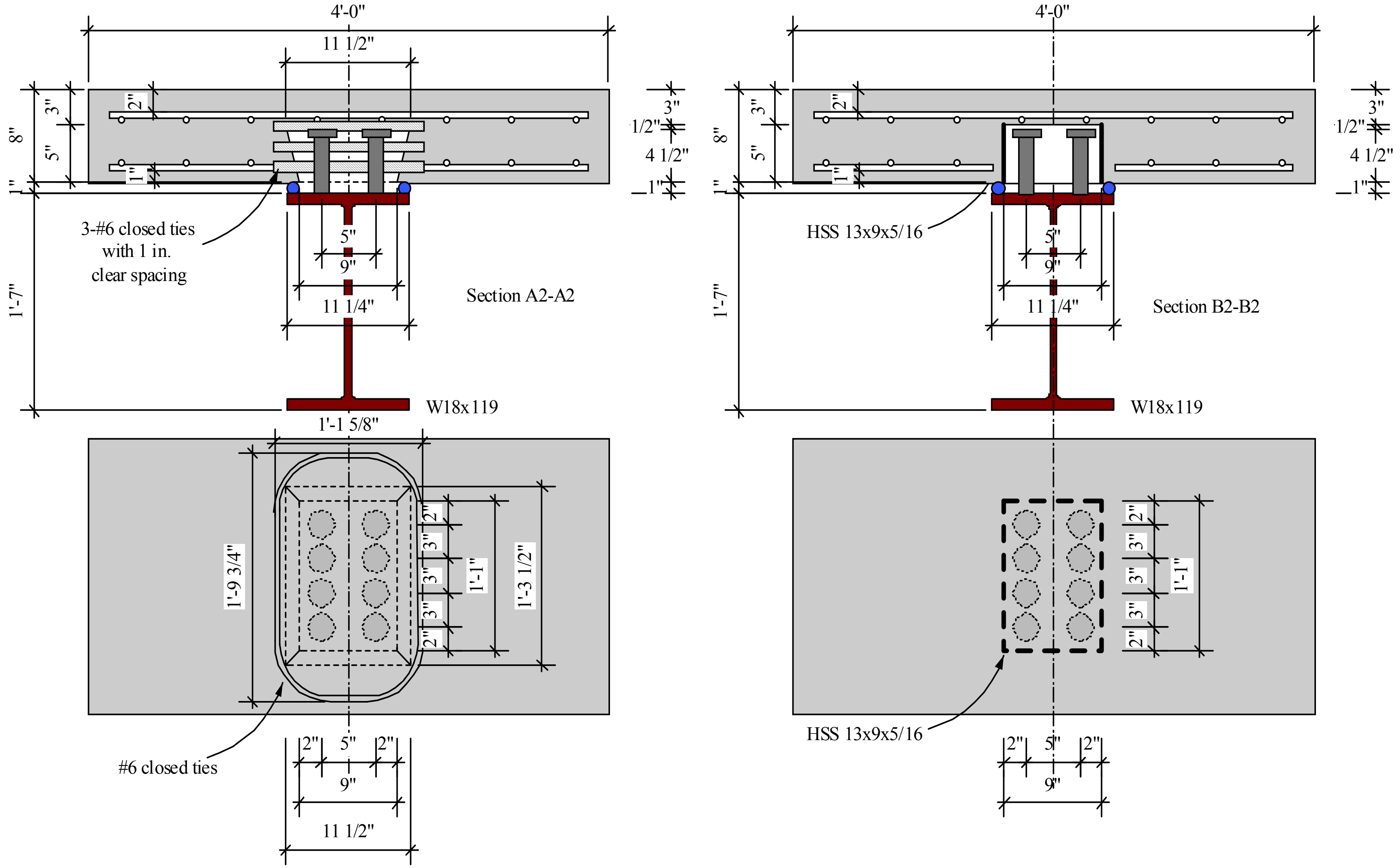


Figure 11. Sections A2-A2 and B2-B2

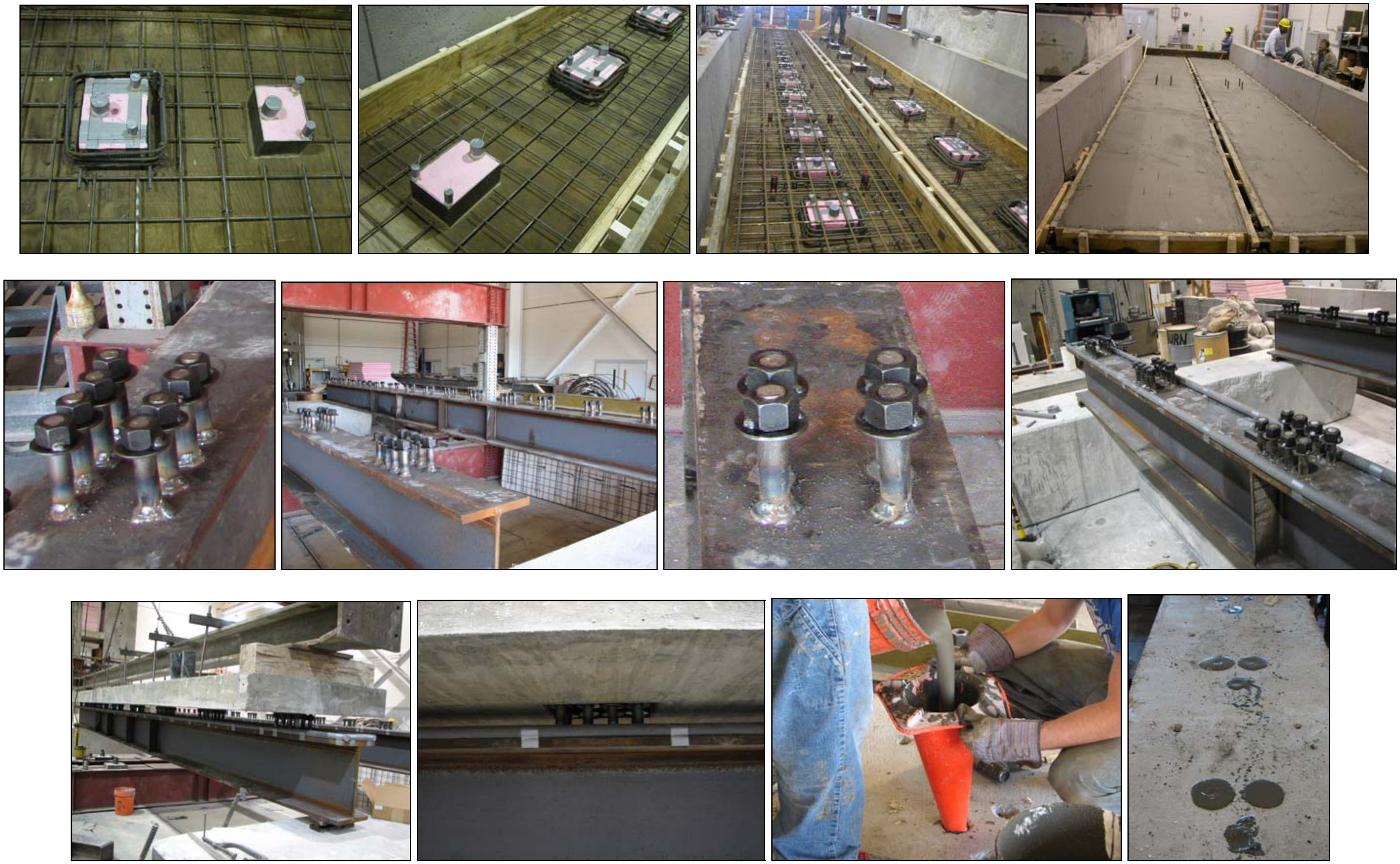


Figure 12. Building the composite beams

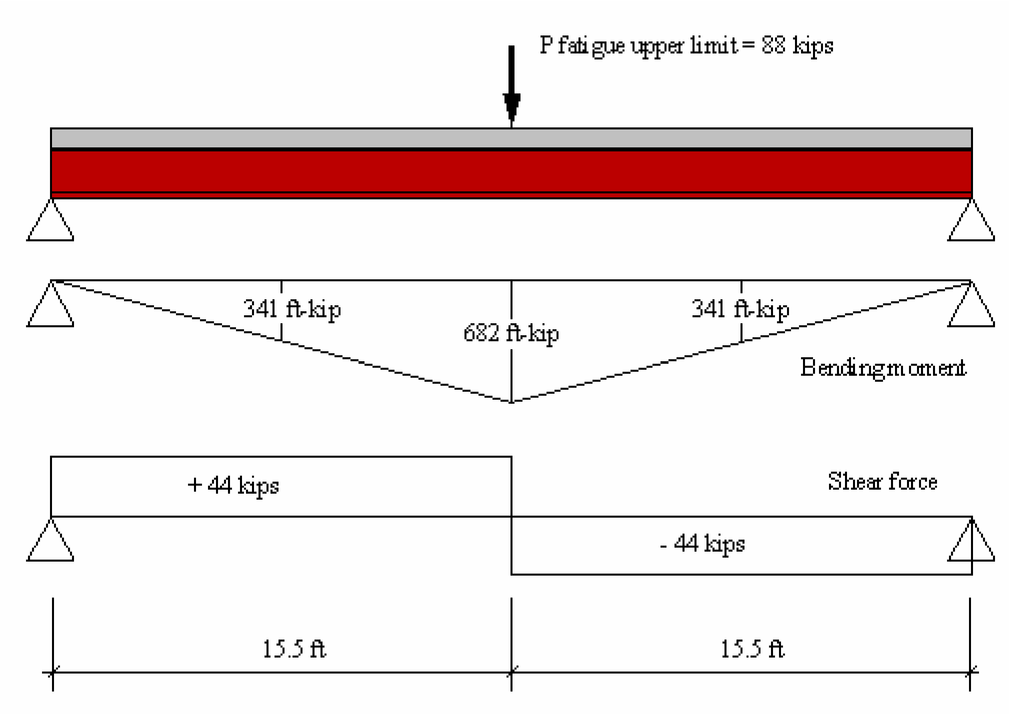


Figure 13. Fatigue test setup

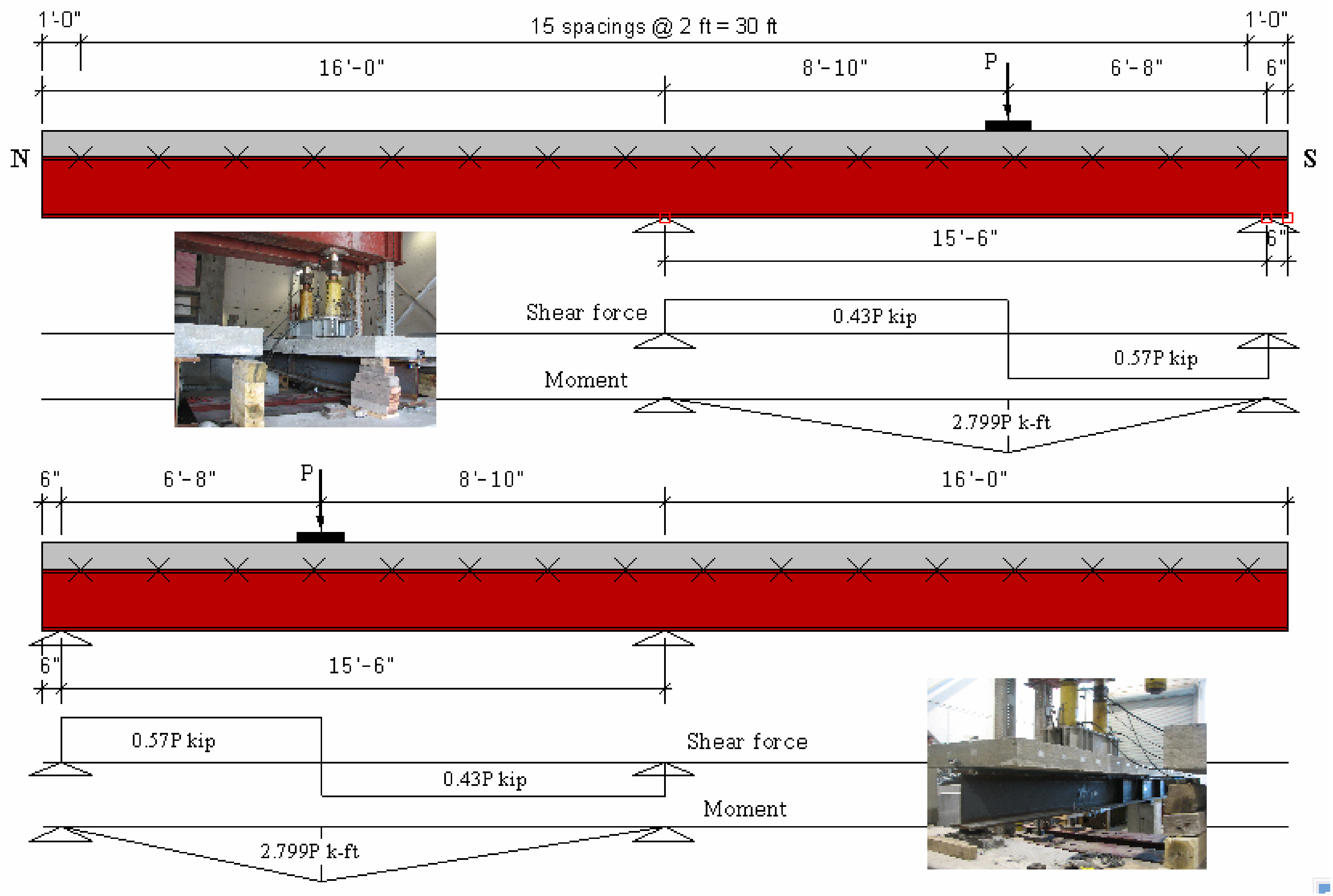


Figure 14. Ultimate test arrangement of Beam #1

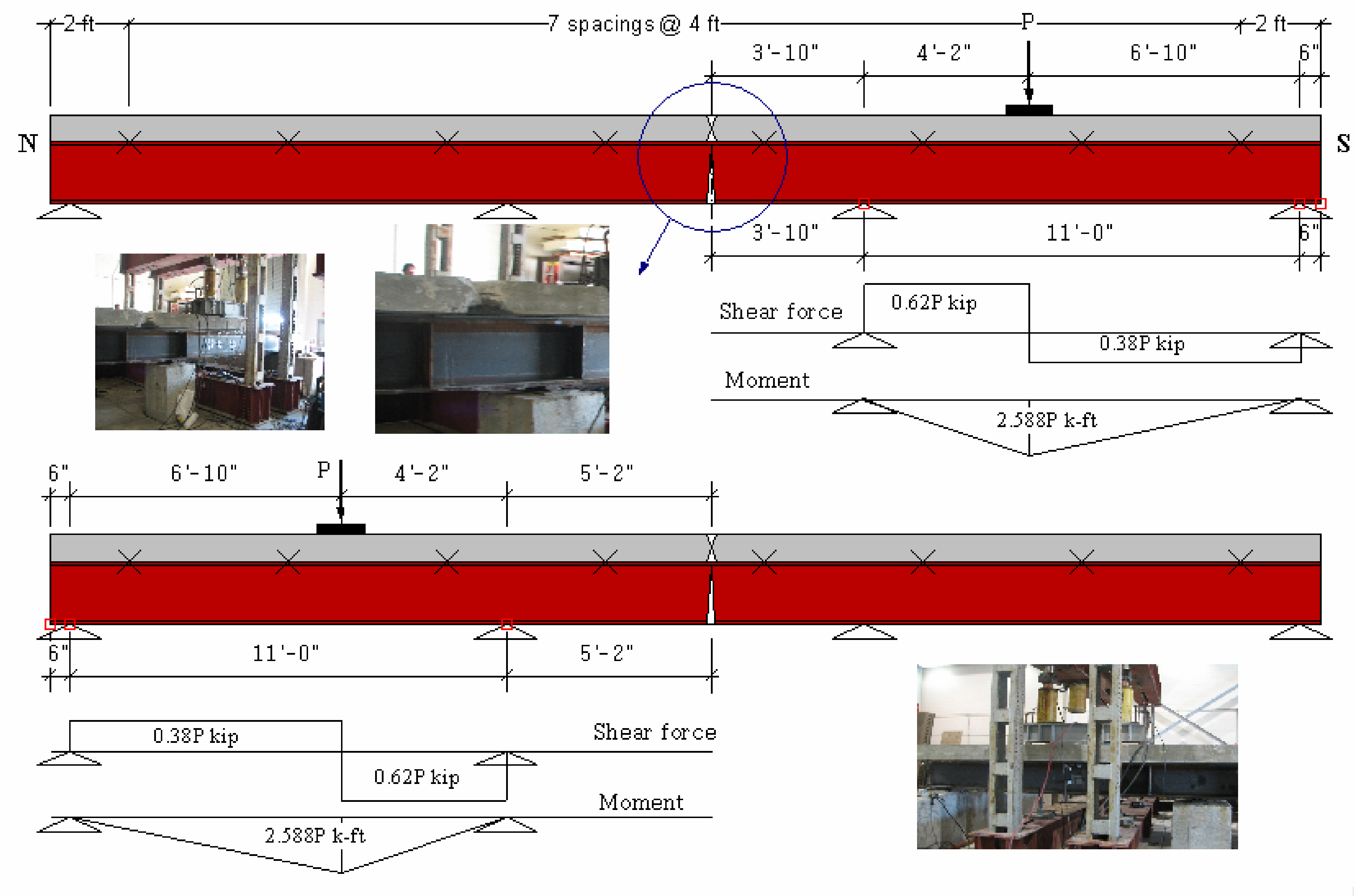


Figure 15. Ultimate test arrangement of Beam #2

Acknowledgments

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