Development and Application of Large-Size Shear Studs to Steel Girder Bridges

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The majority of bridges in the United States built with steel girders are typically made composite with a concrete deck slab. Compared to noncomposite construction, composite construction provides many advantages, such as: (1) high span-to-depth ratio that allows longer spans to be covered without affecting the vertical clearance and (2) wider girder spacing that reduces the cost of the superstructure. Also, composite construction produces stiffer structures with less deflection and vibration under service loads compared to noncomposite construction. Over the years, it has been found that these advantages offset any incremental cost required to create the composite action between the deck slab and the steel girder.

Composite action is created by using shear connectors that are installed on the top surface of the girders and embedded in the deck slab. The shear connectors resist the horizontal shear that is caused by the superimposed loads at the girder-deck interface. Several types of shear connectors have been used: (1) flexible channel sections, (2) spiral shear connectors, and (3) headed steel stud shear connectors. Early applications of these types of shear connectors required manual welding, which was time consuming and labor intensive. In the 1960s, a semi-automatic arc-shielding procedure was developed for welding the headed steel studs. The procedure involves using a welding gun that holds the stud and has a trigger-activated circuit to initiate the weld (Chambers, 2001). The welding gun has a lifting mechanism to draw the stud away from the base material and initiate the welding arc. Over the past 30 years, this procedure underwent significant development that led to high quality welding. Also, due to the great technological advancement in construction machinery, the welding equipment (welding gun and power supply) has become available in compact sizes and reasonable prices that allow the majority of stud welders to weld the studs at a competitive price (Chambers, 2001). Research that has been conducted on composite construction using steel studs has proven the feasibility of this system to create full composite action between the concrete slab and the steel girder (Kakish, 1997; Ollgaard, Slutter and Fisher, 1971; Slutter and Driscoll, 1965). Today, the headed steel stud system is the common type of shear connector used on steel girders, not only in the United States, but also all over the world.

The steel studs are made of various sizes ranging from 6 mm (¼ in.) up to 22.2 mm (⅞ in.) in diameter. In bridges, the 19-mm-diameter (¾-in.-diameter) and 22.2-mm-diameter (⅞-in.-diameter) studs are typically used due to the heavy superimposed dead and live loads that exist on bridges. In high shear areas of steel girder bridges, as many as three 22.2-mm-diameter (⅞-in.-diameter) studs per row are used to satisfy design requirements, as shown in Figure 1.

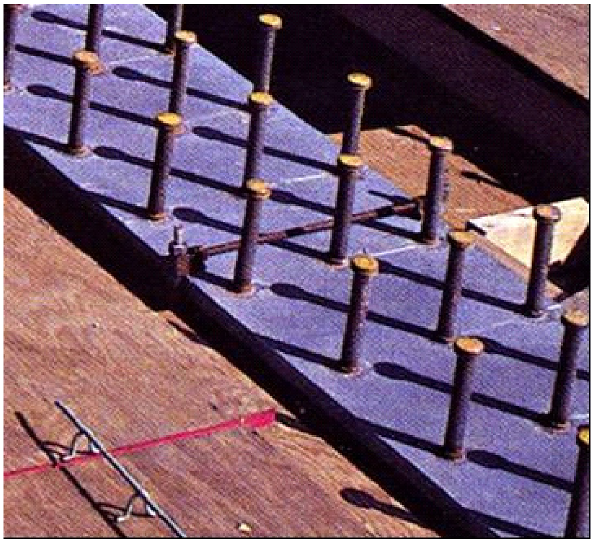

Fig. 1. Three 22.2-mm-diameter (⅞-in.-diameter) studs per row used on a steel beam.

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Table 1. Minimum Mechanical Properties of SAE 1018, Cold-Drawn Bars

Tensile strength, MPa (ksi)	440 (64.0)
Yield strength, MPa (ksi)	372 (54.0)
Elongation, %	15
Reduction in area, %	40
Brinell hardness	126

The relatively high number of studs has many disadvantages, such as: (1) long installation time; (2) little or no room is left on the top flange for the construction workers to walk, which raises safety concerns; and (3) difficult deck removal that may damage the studs as well as the girder top flange. For these reasons, a girder-to-deck connection that reduces the number of shear studs could be advantageous.

In 1997, a group of researchers started working on developing a large size 31.8-mm-diameter (1¼-in.-diameter) stud. The stud development was initiated on the NCHRP 12-41 research project titled "Rapid Replacement of Bridge Decks" (NCHRP, 1998) and then the stud was used on bridges in Nebraska on the Research Project No. PR-PL-1(035) P516 funded by the Nebraska Department of Roads (Badie and Tadros, 2000). Recently, the 31.8-mm-diameter (1¼-in.-diameter) stud has been considered for use with precast concrete panels made composite with steel girders in the ongoing NCHRP 12-65 research project titled "Full-Depth, Precast-Concrete Bridge Deck Panel Systems" (NCHRP, 2004).

This paper provides information on the dimensions, manufacturing, and welding processes, and quality control testing of the 31.8 mm (1¼ in.) stud. Also, the paper provides a discussion on the applicability of the current AASHTO specifications (AASHTO, 2002; AASHTO, 2004) and AISC *Specification for Structural Steel Buildings* (AISC, 2005) for design of the 31.8 mm (1¼ in.) stud.

LARGE STUD MATERIAL AND GEOMETRY DEVELOPMENT

The size of the large stud was determined based on the following conditions: (1) cutting the number of the 22.2-mm-diameter (⅞-in.-diameter) studs by as much as 50%; (2) using material that is commercially available in the market; (3) using the same technique and equipment currently used for welding the 22.2 mm (⅞ in.) studs; and (4) maintaining a competitive price for the large stud compared to the 22.2-mm-diameter (⅞-in.-diameter) stud. To fulfill these conditions, the researchers conducted a search in cooperation with stud manufacturers to find the steel grade and raw material that could be used in producing the large stud. The study revealed that Society of Automotive Engineering (SAE) 1018 steel, cold-drawn bars, currently used for producing the 22.2 mm (⅞ in.) studs, is available in 31.8 mm (1¼ in.) diameter. The 31.8 mm (1¼ in.) diameter was chosen because a 31.8-mm-diameter (1¼-in.-diameter) circle has almost twice the cross-sectional area of a 22.2-mm-diameter (⅞-in.-diameter) circle, which will reduce the number of 22.2 mm (⅞ in.) studs by 50%. Table 1 gives a summary of the mechanical properties of SAE 1018 steel.

The study also revealed that the 31.8 mm (1¼ in.) stud can be produced as a headed stud, similar to the 19.1 mm (⅞ in.) studs, using hot forging or upsetting equipment, or as a headless stud by threading the top part of the stud and adding a hexagonal nut. The headless 31.8 mm (1¼ in.) stud with hexagonal nut was used in the early stages of development and the implementation projects due to the lack of forging equipment at the local stud manufacturers (Tadros and Baishya, 1998). Recently, the researchers on the NCHRP Project 12-65 (Badie and Tadros, 2004) have found several stud manufacturers that can produce 31.8 mm (1¼ in.) headed studs at a competitive price.

The stud head plays an important role in developing the full tensile capacity of the stud. When horizontal shear forces are applied at the interface, the deck slab starts to move vertically away from the steel beam applying upward vertical thrust on the bottom surface of the stud head and a tensile force in the stud stem. As a result, the stud head resists this force by applying high compressive stress on the concrete mass surrounding the stud. The compressive stress helps to confine the concrete around the stud's stem, protecting it from premature failure and helping the stud to develop its full tensile capacity. The effect of enhanced confinement has been recognized in some design specifications and building codes in the formulas used to determine development length of bars in tension. For example, see Equation 12-1 of the ACI 318-05 Building Code (ACI, 2005), where the effect of enhanced confinement is represented by the factor K_{tr} provided in the denominator.

To determine the head size of the 31.8 mm (1¼ in.) stud, the researchers studied the head-to-stem cross-sectional area of the 22.2 mm (⅞ in.) stud and determined that the area ratio was equal to 2.0. Because a 31.8 mm (1¼ in.) stud provides at least double the tensile capacity of a 22.2 mm (⅞ in.) stud, the diameter of the head of the 31.8 mm (1¼ in.) stud was determined by doubling the head-to-stem cross-sectional area ratio for the 22.2 mm (⅞ in.) stud. This resulted in a stud head diameter of 63.5 mm (2½ in.). The 13 mm (½ in.) thickness of the stud head was determined based on designing the head as a free cantilever subjected to bearing stresses. Figure 2 shows the headed and headless 31.8 mm (1¼ in.) studs. Regarding the length of the 31.8 mm (1¼ in.) stud, the researchers decided to follow the AASHTO and AISC specifications recommendations, where: (1) the stud head should pass the bottom layer of reinforcement of the concrete slab, (2) the ratio of the stud length after installation to its diameter should not be less than 4, and (3) the minimum concrete clear cover should be maintained on top of the stud head.

STUD WELDING TECHNIQUE

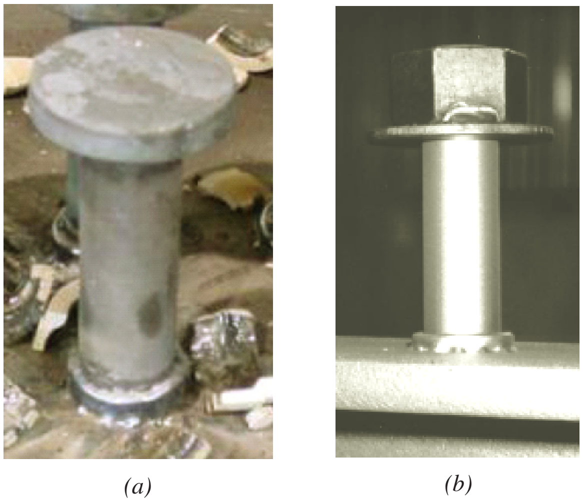
The researchers determined that the arc stud welding process that is currently used in welding the 19.1 mm (¾ in.) and the 22.2 mm (⅞ in.) studs could be used for the larger stud, because of its availability, productivity, and familiarity. During this welding process, a controlled electric arc is used to melt the base of the stud and a portion of the base metal. The stud is thrust automatically into the molten metal and a high-quality fusion weld is produced.

The "chuck" of the welding gun that grips the stud was modified to fit the large headed or headless stud, as shown in Figure 3. Many welding trials were conducted to determine the factors that may affect the welding quality. Three factors were the slope of the stud chamfer, amount of flux, and power supply. During early welding trials, it was evident that steeper chamfer and more flux than typically used with the 22.2 mm (⅞ in.) studs would facilitate the welding process and lead to high-quality welding. Thus, the 31.8 mm (1¼ in.) stud was provided with a steep chamfer and the amount of flux material was tripled compared to that used with the 22.2 mm (⅞ in.) studs. Because the 31.8 mm (1¼ in.) stud has a larger cross-sectional area than the 22.2 mm (⅞ in.) studs, it was expected that welding would require a power source with higher amperage. Welding trials showed that a power source of 2,400 minimum amperage, which is available from commercial vendors, would produce enough heat to melt the stud base and lead to good welding quality. Note that welding a 22.2 mm (⅞ in.) stud usually requires amperage in the range of 1,800–2,000. With these modifications, excellent welding quality was achieved. Figure 4 gives the final dimensions of the 31.8 mm (1¼ in.) stud.

Also, it was found that welding of the 31.8 mm (1¼ in.) stud resulted in a melted section of the top flange thickness deeper than that typically occurs with the 22.2 mm (⅞ in.) stud. Therefore, it is recommended to use the 31.8 mm (1¼ in.) stud on flanges with a 19 mm (¾ in.) minimum thickness. However, if the 31.8 mm (1¼ in.) studs are welded on one line directly above the beam vertical web, a 13 mm (½ in.) minimum flange thickness can be used.

QUALITY CONTROL

Bridge owners require testing of studs for quality assurance. Specifications in some Department of Transportation Bridge Manuals require that studs welded to the steel girders be bent at a 45° angle using a sledge hammer without failure at the weld. This procedure has been successfully used with the



(a) *(b)*

Fig. 2. General view of the 31.8 mm (1¼ in.) stud. (a) Headed stud with integral head. (b) Headless stud with hexagonal nut.

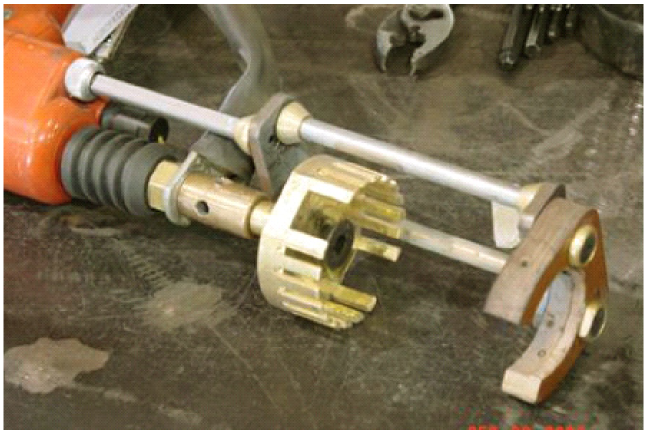


Fig. 3. Welding gun with the special chuck used with the 31.8 mm (1¼ in.) headed stud.

31.8 mm (1¼ in.) stud as shown in Figure 5. However, it was noticed that the top part of the stud was damaged during testing due to the large force that is needed to bend the stud.

The researchers developed a portable hydraulic jacking system that could be used in the shop or in the field for testing pairs of studs, as an alternative to the stud bending procedure. The device, shown in Figure 6, consists of two collars placed around two adjacent studs, a small hydraulic jack, and a top tie. The collar consists of two steel blocks tied together with four screws. By tightening the four screws, the collar is in full contact with the stud. The base of the collar is chamfered to accommodate the weld at the stud base. A hydraulic jack is placed between the collars to provide lateral shearing force at the stud base. The top tie, which consists of two hooks and a turnbuckle, is used to protect the studs from bending.

The quality control test is conducted by applying a horizontal force to cause a tension failure in the stud. The force is calculated by analyzing the studs with the top tie as a frame structure, where the studs are fixed at the base and hinged at the top. By equating the principal stresses at the stud base with the stud yield strength, a relationship between the applied force and the stud yield strength is derived. To protect the stud from damage during the quality control test, an appropriate factor of safety may be applied. This device has been used successfully in addition to the bending test on the demonstration bridge projects that will be covered later in this paper.

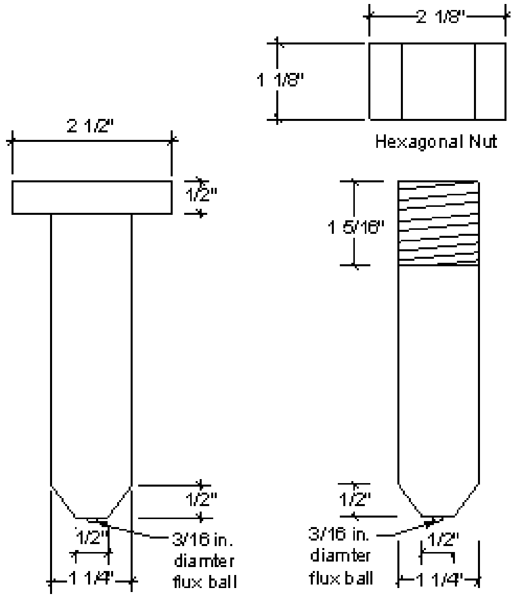
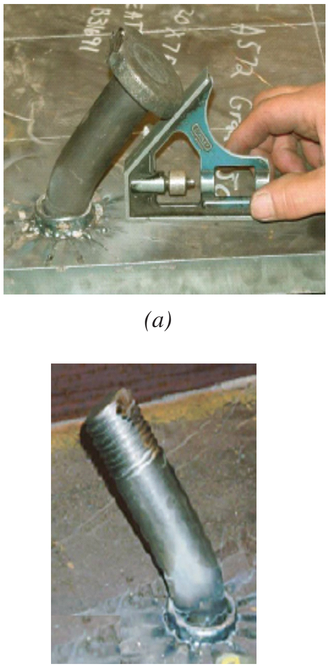


Fig. 4. Dimensions of the 31.8-mm-diameter (1¼-in.-diameter) stud.

AASHTO AND AISC DESIGN PROCEDURE FOR STEEL STUDS

The AASHTO *Bridge Design Specifications* (AASHTO, 2004) procedure for steel studs involves checking the stud ultimate and fatigue capacities, while the AISC *Specification for Structural Steel Buildings* (AISC, 2005) procedure involves checking only the ultimate capacity, as described in this section.



(a)

(b)

Fig. 5. Quality control test by bending the stud to 45°. (a) Headed stud. (b) Headless stud.

Fig. 6. Quality control test conducted by a portable hydraulic jacking system.

Ultimate Capacity [AASHTO LRFD (AASHTO, 2004), Equation 6.10.10.4.3-1; AISC (AISC, 2005), Equation I2-12]

$$Q_n = 0.5A_{sc}\sqrt{f_c'E_c} \le A_{sc}F_u \tag{1}$$

where

Q_n = nominal ultimate stud shear resistance, N (kip)

A_{sc} = stud cross sectional area, mm^2 (in.^{2})

f_c' = specified 28-day compressive strength of concrete, MPa (ksi)

E_c = modulus of elasticity of concrete, MPa (ksi)

F_u = specified minimum tensile strength of a stud, MPa (ksi)

The stud ultimate capacity is compared against the factored shear force that is generated at the interface using loads applied after the composite section is formed. For highway bridges, these loads include weight of the overlay, weight of side barriers, utility loads, and live load. According to the AASHTO LRFD and AISC Specifications (AASHTO, 2004; AISC, 2005), the horizontal shear force demand at the interface for full composite action, between the zero-moment point and maximum positive moment point, P_p, is taken as the least of the maximum plastic force that can be generated by the concrete slab or the steel girder, as follows,

$$P_p = \text{least}\begin{Bmatrix} 0.85 f_c' b_s t_s \\ F_y A_{steelbeam} \end{Bmatrix} \tag{2}$$

where

f_c' = specified 28-day compressive strength of concrete, MPa (ksi)

b_s = effective width of the slab, mm (in.)

t_s = thickness of the slab, mm (in.)

F_y = yield strength of the steel beam, MPa (ksi)

$A_{steelbeam}$ = cross-sectional area of the steel beam, mm^2 (in.^{2})

Fatigue Capacity [AASHTO LRFD (AASHTO, 2004), Equation 6.10.10.2-1]

$$Z_r = \alpha d^2 \ge \frac{38}{2} d^2 \qquad \text{SI Units} \tag{3a}$$

$$Z_r = \alpha d^2 \ge \frac{5.5}{2} d^2 \qquad \text{English Units} \tag{3b}$$

where

Z_r = stud fatigue resistance, N (kips)

d = stud diameter, mm (in.)

α (MPa) = 238 – 29.4 Log(N) AASHTO LRFD (AASHTO, 2004), Equation 6.10.10.2-2

α (ksi) = 34.5 – 4.28 Log(N) AASHTO LRFD (AASHTO, 2004), Equation 6.10.10.2-2

N = number of cycles of fatigue loading

The stud fatigue capacity is compared against the fatigue demand due to the live load at the interface. For the majority of practical cases of concrete/steel composite bridge construction, the fatigue capacity of the stud controls the design and the resulting pitch of the studs.

AASHTO Maximum Spacing Limit

Article 6.10.10.1.2 of the AASHTO LRFD Specifications (AASHTO, 2004) states that center-to-center spacing of the studs should not exceed 610 mm (24.0 in.). Review of the literature has revealed that this limit was based on the research conducted by Viest and Siess (1954), where they reported relative separation between the slab and the steel beam when 915 mm (36.0 in.) spacing was used. Because this limit would limit some of the advantages of using 31.8 mm (1¼ in.) stud, the researchers of the ongoing NCHRP Project 12-65 have been investigating extending this limit.

EXPERIMENTAL PROGRAM

To investigate the applicability of the design procedure given by the current AASHTO bridge specifications (AASHTO, 2002, 2004), the research team conducted a comprehensive experimental program (Tadros and Baishya, 1998; Badie and Tadros, 2000; Badie, Tadros, Kakish, Splittgerber, and Baishya, 2002). The experimental program consisted of the following parts: (1) 20 push-off specimens for ultimate strength investigation; (2) 25 push-off specimens for fatigue resistance investigation; and (3) one full-scale beam test.

Ultimate Strength Investigation

Five groups, four specimens per group, of push-off specimens were used in the ultimate strength investigation. In all specimens, studs were welded on the top flange of a W-shape steel beam, and a cast-in-place concrete slab was then poured on top of the steel beam. Full discussion on the ultimate strength experimental investigation is given by Badie et al. (2002). Table 2 and Figures 7 and 8 give full details of the test specimens. The specimens were tested in a self-equilibrium frame, where the load was applied through hydraulic jacks. A linear variable differential transducer was installed on the side of the specimens to monitor the differential movement between the concrete slab and the steel beam.

Table 2. Ultimate Test Results of Push-Off Specimens

Group #	Stud Arrangement	Test Type	# of Specimens	Specimen Dimensions	Concrete Strength, MPa (ksi)	Q_{test} per Stud, kN (kip)	Q_{test}/A_{stud}, MPa (ksi)	$\frac{Q_{test}}{Q_{LRFD}}$	Ultimate Slip, mm (in.)
G1	4 rows, 2–⅞” studs/ row, all headed studs	Ultimate	4	See Fig. 7	32 (4.7)	127 (28.6)	328 (47.6)	97%	6.35 (0.250)
G2-a	4 rows, 1–1¼” stud/ row, all headed studs	Ultimate	2		32 (4.7)	242 (54.3)	305 (44.3)	69%	3.00 (0.120)
G2-b		2×10^6 cycles and ultimate	2		32 (4.7)	253 (56.9)	319 (46.4)	72%	1.78 (0.067)
G3-a	Same as G2, but studs are welded on steel brams used in Group 1 after removal of the ⅞” studs	Ultimate	2		38 (5.6)	231 (52.0)	292 (42.4)	66%	2.00 (0.080)
G3-b		2×10^6 cycles and ultimate	2		38 (5.6)	245 (55.2)	310 (45.0)	70%	1.62 (0.064)
G4-a	4 rows, 1–1¼” stud/ row, alternate headed/ headless studs	Ultimate	2		32 (4.7)	198 (44.5)	250 (36.3)	57%	2.09 (0.082)
G4-b		2×10^6 cycles and ultimate	2		32 (4.7)	207 (46.7)	263 (38.1)	59%	1.64 (0.065)
G5-a	Same as G4, no eccentricity and with transverse ties	Ultimate	2	See Fig. 8a & 8b	46 (6.8)	297 (66.8)	375 (54.4)	85%	1.70 (0.067)
G5-b	Same as G4, no eccentricity and with enhanced confinement	Ultimate	2	See Fig. 8a & 8c	46 (6.8)	349 (78.5)	441 (63.9)	100%	1.75 (0.069)

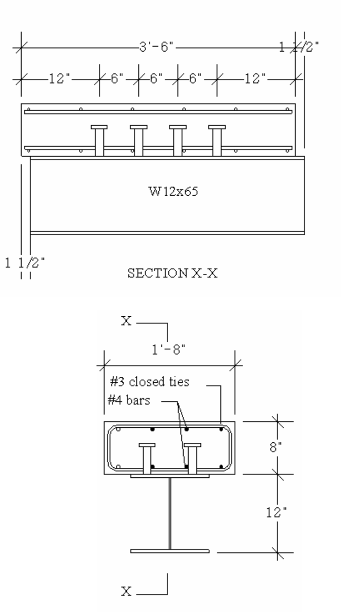


Fig. 7. Ultimate test specimen, Group 1.

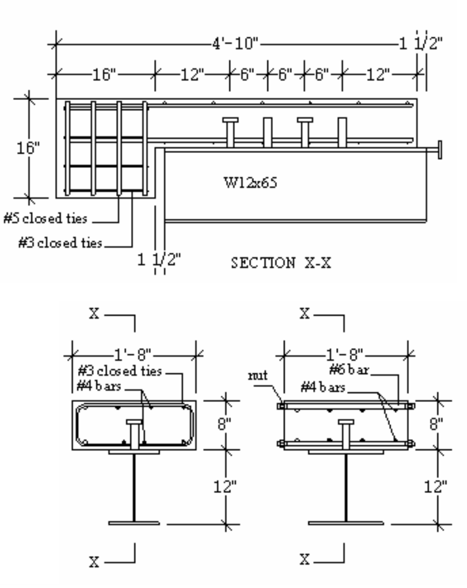


Fig. 8. Ultimate test specimen, Group 5.

Test Results and Discussion

Table 2 summarizes the test results; where the ultimate force per stud and the corresponding slippage are given. In addition, Table 2 gives the ratio between the stud capacity observed from the test and that determined by Equation 6.10.10.4.3-1 of the AASHTO LRFD Specifications (AASHTO, 2004). For all specimens, the stud tensile capacity controlled the stud capacity in Equation 6.10.10.4.3-1, which was 130.8 kN (29.4 kips) for the 22.2 mm (⅞ in.) studs and 349.2 kN (78.5 kips) for the 31.8 mm (1¼ in.) studs. A similar mode of failure was observed in most of the specimens, in which failure occurred by shear at the base of the stud weld. Test results of the ultimate strength investigation can be summarized as follows:

1. The 31.8 mm (1¼ in.) studs can achieve their ultimate capacity, as demonstrated by the AASHTO LRFD equation (AASHTO, 2004), and shown by the results of Group G5-a. Studs in Group G5-a achieved 85% of their tensile capacity after eliminating the eccentricity in the test setup. By eliminating the eccentricity and providing adequate anchorage of the transverse reinforcement, Group G5-b, the studs achieved full tensile capacity. This shows the importance of the impact of the confinement produced by the transverse reinforcement on the capacity of the studs. Slutter and Driscoll (1965) reported the same finding when they compared the results of flexural beams and push-out specimens. A high level of concrete confinement is usually achieved in bridge decks by using continuous top and bottom transverse reinforcement over the girder lines. This issue is well recognized by the empirical deck design of the AASHTO LRFD specifications, which mandates the use of continuous top and bottom transverse reinforcement over the girder lines.

2. Although the 31.8 mm (1¼ in.) studs in Groups G2 and G3 did not achieve the ultimate capacity demonstrated by the AASHTO LRFD equation (AASHTO, 2004), they achieved a load capacity two times that of the 22.2 mm (⅞ in.) studs in Group 1. Because the 31.8 mm (1¼ in.) stud has almost twice the cross-sectional area of the 22.2 mm (⅞ in.) stud, one 31.8 mm (1¼ in.) stud can conservatively replace two 22.2 mm (⅞ in.) studs.

3. A comparison between test results of Groups G2 and G3 showed that residual stresses due to removal and replacement of the 22.2 mm (⅞ in.) studs with 31.8 mm (1¼ in.) studs appeared to have no detrimental effect on the capacity of the large stud.

4. The specimens made with 31.8 mm (1¼ in.) studs showed about 30% less slippage at failure than the specimens made with the 22.2 mm (⅞ in.) studs.

5. Results of Groups G2, G3, and G4 showed that two million cycles of cyclic loading caused no reduction in the strength of the 31.8 mm (1¼ in.) studs.

6. Replacing 50% of the headed studs with headless studs, Group G2 versus Group G4, resulted in a reduction of the group capacity of 17%. Use of headless studs may appear to be disadvantageous due to the anticipated relatively large number of studs needed for full composite action. However, ease of long-term deck removal and replacement may compensate for the increased number of headless studs used. The AASHTO Specifications (AASHTO, 2002, 2004) do not provide any guidance on this issue.

Fatigue Resistance Investigation

Based on the experience gained from the ultimate push-off test, an L-shaped specimen similar to that of Group G5-b of the fatigue experimental investigation was used in the fatigue push-off tests. Twenty-five specimens were prepared, 11 specimens with 22.2 mm (⅞ in.) headed studs and 14 specimens with 31.8 mm (1¼ in.) headed studs. Each specimen had two studs welded on a 12.7-mm-thick (1.0-in.-thick) steel plate. The studs were welded on one line at 152.4 mm (6 in.) along the longitudinal axis of the specimen. The minimum applied fatigue load was equivalent to 34.5 MPa (5 ksi) direct shear stress on the stud, representing the shear stress caused by the superimposed dead loads, in other words, barrier and future wearing surface weight. The fatigue stress range on the stud, S_r, varied from 70 to 172 MPa (10 to 25 ksi), representing the shear stress caused by the live load effects. Details of the specimens and the fatigue resistance experimental investigation are given by Badie et al. (2002). The results of the fatigue resistance investigation can be summarized as follows:

1. The test results are shown in Figure 9, where the relationship between the number of cycles and the stress range, S_r, for the 31.8 mm (1¼ in.) and 22.2 mm (⅞ in.) studs is given.

2. To compare the test results with the predicted capacities of the AASHTO LRFD Specifications, a regression analysis of the test results was conducted and yielded the following results:

   For the 31.8 mm (1¼ in.) stud (note that $\alpha = (\pi/4)\,S_r$)

   $$\alpha\,(\text{MPa}) = 278.8 - 31.4\,\text{Log}(N)\quad\text{(SI units)} \tag{4a}$$

   $$\alpha\,(\text{ksi}) = 40.44 - 4.56\,\text{Log}(N)\quad\text{(English units)} \tag{4b}$$

   For the 22.2 mm (⅞ in.) stud

   $$\alpha\,(\text{MPa}) = 277.0 - 32.1\,\text{Log}(N)\quad\text{(SI units)} \tag{5a}$$

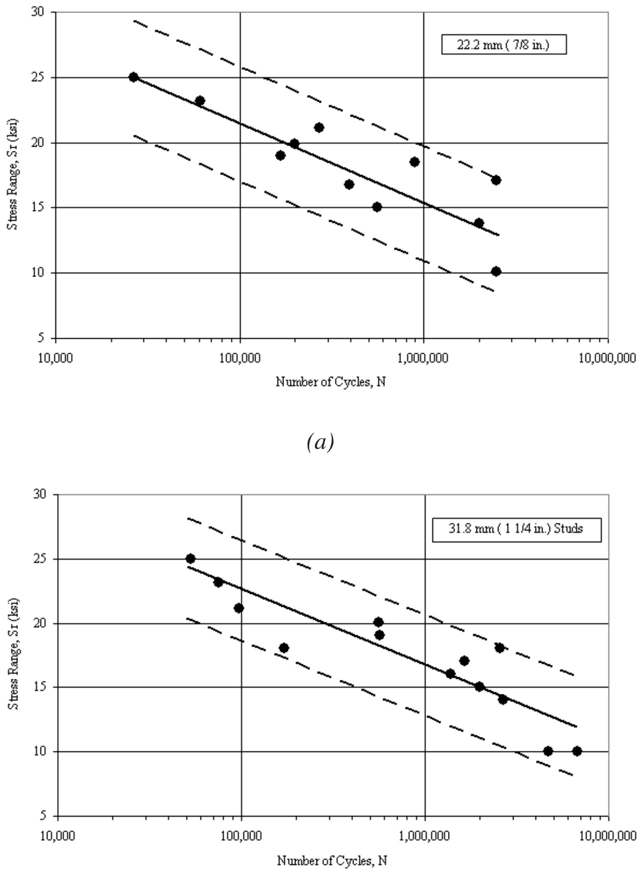
$$\alpha \text{ (ksi)} = 40.17 - 4.65 \text{ Log(N)} \quad \text{(English units)} \qquad (5b)$$

3. Figure 10 gives a comparison of the α-value between the AASHTO specifications (AASHTO, 2002, 2004), and the current testing program (in other words, Equations 4 and 5). From the comparison, the following conclusions can be drawn:

   - It is conservative to use the α-values given by the AASHTO Specifications (AASHTO, 2002, 2004) to calculate the allowable range of horizontal shear force for the 31.8 mm (1¼ in.) and the 22.2 mm (⅞ in.) studs.

   - Designers are encouraged to use Equations 4 and 5 developed in this research. Using these equations will reduce the amount of studs by about 30%, which will reduce the initial cost of a bridge as well as the cost of future deck removal.

   - If Equation 4 is used, it is expected that one row of 31.8 mm (1¼ in.) studs over the girder-web location, spaced at 150 mm (6 in.) or more, will be adequate to maintain full composite action for the majority of bridges. This will ease deck replacement in the future, and will increase the safety of the construction workers.



(a)

(b)

Fig. 9. Fatigue test results (solid line = regression analysis, dashed lines = twice the standard error of estimate).

Please note that in all of the ultimate and fatigue specimens, a relatively small load was applied to every specimen before testing in order to break the bond between the concrete slab and the steel beam.

Full-Scale Beam Test

A full-scale beam was tested to evaluate the performance of 31.8 mm (1¼ in.) studs in a flexural beam test. A 12.2-m-long (40-ft-long) W36×160 rolled section, with alternate headed and headless 31.8 mm (1¼ in.) shear studs spaced at 152.4 mm (6 in.), was used in the test. A 1.22-m-wide (4-ft-wide) and 203-mm-thick (8-in.-thick) concrete deck was placed on the top of the girder. The beam was tested in fatigue for 4.8 million cycles under AASHTO HS-25 truck loading positioned at the critical shear section, as specified by AASHTO Specifications. The truckload was then moved to the midspan section and run for another 4.8 million cycles. Fatigue testing showed that the steel beam acted compositely with the deck even after it was exposed to 9.6 million cycles. No loss of composite action between the concrete deck and the steel beam, and no distress in the concrete deck due to the use of 31.8 mm (1¼ in.) studs was observed. More information on the full-scale beam test can be found in Badie et al. (2002).

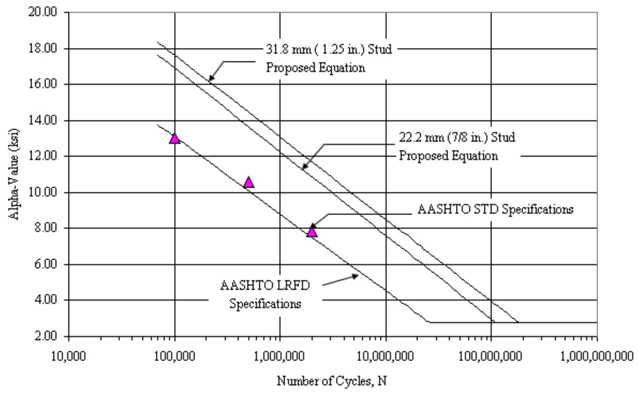


Fig. 10. Comparison of α-values.

DEMONSTRATION PROJECT

The 31.8 mm (1¼ in.) stud was used on two projects in Nebraska. The first project was a three-span continuous bridge in western Nebraska, on Highway 71 in Gering South, Nebraska, consisting of three continuous spans of 13.7, 18.28, and 13.7 m (45, 60, and 45 ft). The cross section of the bridge had five W30×99 rolled steel beams spaced at 2.67 m (8 ft-9 in.) made composite with a 190-mm-thick (7.5-in.-thick) cast-in-place slab. Headless 31.8 mm (1¼ in.) studs with hexagonal nuts were used on the south span, one stud per row welded directly over the girder web, with spacing from 177 to 254 mm (7 to 10 in.). A 22.2 mm (⅞ in.) stud was used on the center and north span, three studs per row at spacing from 254 to 407 mm (10 to 16 in.).

TSA Manufacturing, Omaha, Nebraska, produced the 31.8 mm (1¼ in.) studs, and the steel fabricator, Capital Contractors, welded them in its shop. An electric source of 2,500 A was used in welding the studs. Welding of the studs on the steel beams proceeded at a rate of 40 seconds per stud without any problems. The quality control test previously described was conducted at three locations on each girder. Even though a factor of safety of 1.0 was used in the quality-control test, no welding failure was observed in the quality control test.

Because this was the first time the 31.8 mm (1¼ in.) stud was used in bridges, and because the fatigue investigation was under way when the bridge was being constructed, the Nebraska Department of Roads (NDOR) bridge designers decided to add a washer to the stud assembly. The washer was 4.8 mm (3⁄16 in.) thick, with an outside diameter of 76 mm (3 in.), and was tack welded to the stud head. It was added to the stud assembly to ensure sufficient confinement of the concrete around the stud. However, upon completion of the fatigue resistance investigation, it was evident that providing continuous top and bottom reinforcement in the deck slab can provide adequate confinement to the concrete. Thus, the researchers have advised NDOR not to add the washer to the stud in future projects. Figure 11 shows the top view of the deck slab before casting the concrete.

The bridge construction was completed in the fall of 1999. The researchers and NDOR designers took deflection measurements of the bridge using a three-axle dump truck. Deflection measurements were taken at the maximum positive moment section of the center girder of the exterior spans. Both exterior spans showed the same amount of deflection, 3 mm (0.12 in.). Continuous visual inspection of the bridge deck has shown no cracks or distress on the south span where the large diameter stud was used.

The second project is the Skyline Bridge, Omaha, Nebraska (Fallaha, Sun, Lafferty, and Tadros, 2004). The bridge carries the Skyline Drive traffic over the US 6 Expressway. The bridge has two unequal spans 27150 and 37950 mm (89 and 124.5 ft) and the superstructure has five steel plate girders spaced at 3300 mm (10 ft-10 in.) made composite with a full depth precast concrete deck system called the NUDECK (Badie, Baishya, and Tadros, 1999). The headless 31.8 mm (1¼ in.) stud with a hexagonal nut was used on the entire bridge to create the composite action. Continuous open channels were created in the precast concrete panel over the girder lines to provide space for the shear connectors, as shown in Figure 12. The studs were provided by Nelson Stud Welding, Inc. Because this project was constructed after the researchers finalized the fatigue investigation, NDOR decided not to add washers to the stud head.

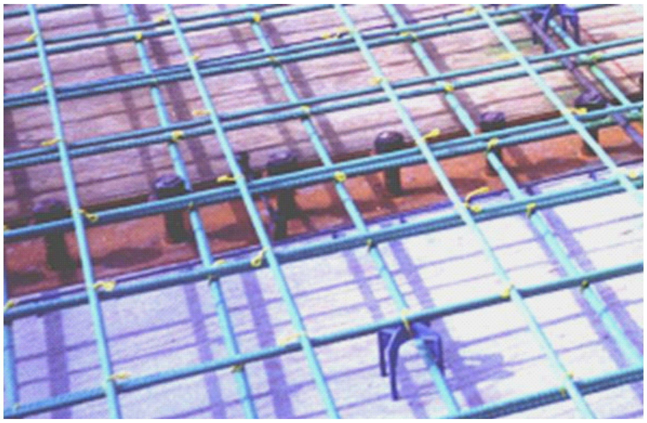


Fig. 11. Top view of the deck slab before casting concrete.

Fig. 12. Large diameter studs used on the Skyline Bridge, Omaha, NE.

The large studs were provided on one line over the web of the steel girder at a uniform spacing of 152 mm (6 in.). Use of the 31.8 mm (1¼ in.) studs allowed the designer to provide all the required longitudinal post-tensioning strands in the open channel rather than providing ducts in the precast panel to house them. This simplification, significantly helped in reducing the cost of the precast panels and gave the contractor high flexibility for grouting the open channels.

The studs were welded by the steel fabricator at the steel shop at a rate of 40 to 50 seconds per stud. The quality control test was conducted by bending the stud to 45° and using the hydraulic jacking device. Both tests have shown that the welds were of high quality. The studs were arranged so that they did not interfere with the transverse reinforcement of the precast panel passing over the girder lines.

The continuous open channels were filled with Type-K nonshrink cement mortar and cured in-place using wet burlap. Construction of the deck was completed early in 2004 and the bridge was open to traffic by March 2004. The bridge has been under continuous monitoring for almost one year, where deflection measurements have been within the planned design limits. Routinely, visual inspection of the deck has shown no separation between the deck and the steel girders nor any signs of cracks or distress.

COST ANALYSIS OF THE LARGE DIAMETER STUD

Highway 71 Bridge, Gering South, Nebraska

Original plans of the bridge included 135—22.2 mm (⅞ in.) studs per girder on the south exterior span at a price of $0.71 per stud. This price does not include the cost of stud welding. Thus, material cost of the 22.2 mm (⅞ in.) studs for this span = 135 × 0.71 = $95.85 per girder, or 95.85 × 5 girders = $479.25 per span.

Replacing the 22.2 mm (⅞ in.) studs with the 31.8 mm (1¼ in.) studs resulted in using 67—31.8 mm (1¼ in.) studs per girder. Cost of the new stud was as follows: $3.45 for the 31.8 mm (1¼ in.) headless stud and $1.05 for the heavy hexagonal nut. Therefore, total material cost of the 31.8 mm (1¼ in.) studs on the south span = ($3.45 + $1.05) × 67 = $301.50 per girder = $301.50 × 5 girders = $1,507.50 per span. Thus, incremental materials cost = $1,507.50 – $479.25 = $1,028.25 and $1,028.25 / (41.34 ft × 45 ft) = $0.55 per ft^2 ($5.92 per m^2).

Skyline Bridge, Omaha, Nebraska

The price of the 31.8 mm (1¼ in.) headless studs at the time of fabricating the steel girders was $5.50 per stud, plus $1.45 per nut for each stud, while the price of the 22.2 mm (⅞ in.) studs at that time was $1.29 per stud. These figures do not include the cost of stud welding. As a result, using the 31.8 mm (1¼ in.) studs resulted in an incremental cost of about $ 0.50 per square foot of the total deck area.

Using the 31.8 mm (1¼ in.) studs on the demonstration project resulted in additional material cost for the following reasons:

1. In both projects, no competition for supplying this item was allowed within the provisions of this demonstration project.

2. NDOR decided to use the same type of studs produced and tested in the NCHRP 12-41. This decision led to using the headless stud with a heavy nut, which was more expensive than cold-formed, or even hot-forged headed studs.

3. Fatigue capacity of the studs was determined using the α-values given in the AASHTO Specifications, which are shown by the fatigue investigation to be conservative. If the new α-values developed by the researchers were used, this would have resulted in reduction of the number of the 31.8 mm (1¼ in.) studs by about 30%.

4. The relatively small quantity ordered on a short notice for the projects resulted in a higher price than for a large order.

5. The savings in labor due to the reduction in the number of welded studs is not considered in the preceding analysis.

6. Savings in time and labor associated with future deck removal is not considered.

Recent Cost Analysis of the 31.8 mm (1¼ in.) Studs Conducted in the NCHRP 12-65

It was clear from the cost analysis conducted for the demonstration projects that the 31.8 mm (1¼ in.) stud was significantly overpriced. In the NCHRP 12-65 (Badie and Tadros, 2004), the researchers asked three stud manufacturers to provide the price of the 31.8 mm (1¼ in.) headed stud based on a minimum guaranteed order per year. The manufacturers were also asked to identify the minimum number of studs for each unit price level and the preferred way for manufacturing the stud head. Two manufacturers used cold forming to produce the stud head and the third manufacturer used hot forging. Table 3 shows a summary of the prices quoted by the three companies.

The study reveals that the stud price can be as low as $2.19 per stud, which is less than the cost of two ⅞ in. studs (which is currently $1.30 per stud) not including the additional fabrication savings and the other benefits mentioned earlier. Also, with the increasing interest from many highway agencies in using the 31.8 mm (1¼ in.) stud, more manufacturers will be willing to produce the stud at a more competitive price.

Table 3. Cost Analysis of the 31.8 mm (1¼ in.) Stud Conducted in the NCHRP Project 12-65

Number of Studs	750	50,000	100,000	250,000	500,000	1,000,000	Head Forming Method
St. Louis Screw & Bolt	–	–	$2.57	–	$2.35	$2.20	Cold forming
TSA Manufacturing, Inc.	–	–	$3.06	$2.19	–	–	Cold forming
Master Bolt	$3.98	$2.76	$2.19	–	–	–	Hot forging

CONCLUSIONS

This paper presents the development and recent applications of 31.8 mm (1¼ in.) studs to steel girder bridges. The new studs have double the cross-sectional area of the 22.2 mm (⅞ in.) studs, resulting in a reduction of the number of studs needed to achieve full composite action with the concrete deck by 50%.

The 31.8 mm (1¼ in.) stud can be produced using 31.8 mm (1¼ in.) SAE 1018 rod that is commercially available. Current welding and quality inspection practices and equipment used for the 19.1 mm (¾ in.) and 22.2 mm (⅞ in.) studs can be used in welding and testing the proposed stud.

Use of the 31.8 mm (1¼ in.) studs has many advantages: (1) increase of fabrication and construction speed, (2) ease of deck construction, (3) ease of deck removal, and reduction of the possibility of damage to studs and girder top flange during that removal, and (4) enhancement of the safety of field personnel during construction because more space on the top flange is available for walking.

Experimental investigation of the 31.8 mm (1¼ in.) stud showed that the stud fatigue and ultimate capacities can be conservatively determined using current AASHTO bridge specifications to achieve full-composite action. The designers are encouraged to use the α-values developed in the present research. This would reduce the number of studs by about 30%.

Cost analysis studies show that using the 31.8 mm (1¼ in.) stud would result in a relatively small incremental cost if a small quantity for individual projects is ordered with no promise for repeat business. If a minimum annual order is guaranteed, it would allow manufacturers to acquire the equipment necessary for mass production and would drive the material cost to below the level of smaller studs. In addition, savings in labor, construction time, and future deck replacement would result in additional economic and safety benefits.

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