Strength of Shear Studs in Steel Deck on Composite Beams and Joists

W. SAMUEL EASTERLING, DAVID R. GIBBINGS and THOMAS M. MURRAY

INTRODUCTION

Composite beam or joist and slab systems typically provide the most efficient design alternative in steel frame construction, and indeed it is one of these systems that make steel an economically attractive alternative to concrete framed structures. Composite beam specification requirements and design aids are given in the American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD) Manual.[1] The LRFD composite beam design procedure results in designs that are typically 10-15 percent more economical than those obtained using the AISC allowable stress design (ASD) procedure. The efficiency of composite beam design using LRFD procedures has, in the authors' opinions, been the primary motivating factor for the use of the LRFD specification[2] to date.

The design strength and stiffness of composite beams depends on the shear connection behavior. The strength of the shear connectors may be reduced because of the influence of the steel deck geometry. An empirical expression for this reduction was developed by evaluating results of composite beam tests in which the deck ribs were oriented perpendicular to the steel beam.[3] A reduced stud strength is obtained by multiplying the stud reduction factor, SRF, by the nominal strength of a shear stud, Q_n. The expression for the nominal stud strength,[4] which has been incorporated in the AISC LRFD specification and is the basis for the tabular values given in the AISC ASD specification,[5] is given by:

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

where

A_{sc} = cross-sectional area of a stud shear connector
$f_c{}'$ = specified compressive strength of concrete
E_c = modulus of elasticity of concrete
F_u = minimum specified tensile stress of the stud shear connector

This equation was developed based on results from elemental push-out tests.[4] The stud reduction factor is given by:

$$SRF = \frac{0.85}{\sqrt{N_r}} \left(\frac{w_r}{h_r}\right)\left(\frac{H_s}{h_r} - 1.0\right) \le 1.0 \qquad (2)$$

where

N_r = number of studs in one rib at a beam intersection
W_r = average width of concrete rib
h_r = nominal rib height
H_s = length of shear stud after welding

This reduction factor applies to cases in which the deck ribs are perpendicular to the steel beam and is used in both the AISC LRFD and ASD specifications.

These equations, or similar forms, have been used in several design specifications, both in the United States and abroad. However, in recent years several researchers[6-11] have shown that Equation 2 is unconservative for certain configurations. The studies have considered numerous parameters, including depth of steel deck shear stud height, concrete unit weight, position of shear stud in the deck rib relative to the bottom flange stiffener, number of shear studs in a given deck rib, and the amount and position of reinforcement in the slab. The studies reported results from push-out tests alone[6,10,11] or a combination of push-out tests and beam tests.[7-9] A conclusion common to all of the studies is that a modified, or completely different, stud reduction factor is needed. Modified calculation procedures have been developed and reported in the recent research studies. However, none of the studies have reported reasons for the discrepancy between the experimental data and Equations 1 and 2.

The reason for the discrepancy between recent experimental results with those predicted using Equations 1 and 2 is not clear. However, it is clear that a significant base of data exists to substantiate the procedures.[3,12,13] A proper resolution of this dilemma will require careful consideration of all the data.

A review of the data reported by Grant, et al.,[3] along with related studies conducted by Henderson[12] and Klyce[13]

W. Samuel Easterling is assistant professor in the Charles E. Via, Jr. Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

David R. Gibbings is graduate research assistant in the Charles E. Via, Jr. Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, VA.

Thomas M. Murray is Montague-Betts professor of structural steel design in the Charles E. Via, Jr. Department of Civil Engineering, Virginia Polytechnic, Blacksburg, VA.

reveal two important characteristics that relate directly to the discrepancy. The majority, but not all, of the tests reported by Grant, et al. and all the tests reported by Henderson were detailed such that the studs were placed in pairs within a given rib. The single test reported by Klyce had two-thirds of the studs placed in pairs. Also, the deck used in the studies reported by Grant, et al. did not have a stiffener in the bottom flange. Both of these details make the position of the shear stud relative to the stiffener in the bottom flange of the deck, which is described in greater detail in the following paragraph, of less concern.

One of the important parameters identified in some of the recent studies was the position of the shear stud relative to the stiffener in the bottom flange of the deck. Most deck profiles manufactured in the United States have a stiffener in the middle of the bottom flange, thus making it necessary to weld shear studs off center. Tests have shown differences in shear stud strengths for the two choices. A stud placed on the side of the stiffener nearest the end of the span is in the "strong" position and one placed on the side of the stiffener nearest the location of maximum moment is in the "weak" position. A schematic of both strong and weak position stud locations is shown in Figure 1. The difference in strength is partly attributable to the differences in the amount of concrete between the stud and the web of the deck that is nearest to mid-span for the two positions. This detail will be considered further in subsequent sections of this paper.

A characteristic of partial composite beam design must be kept in mind when one evaluates results of beam tests and push-out tests. The relationship between the percentage of shear connection and the moment capacity is shown in Figure 2 for a W16×31 A36 section. The curves shown in Figure 2 were developed using the calculation procedure in the Commentary to the LRFD specification.[2] The nominal moment capacity, M_n, is shown normalized with respect to the fully composite moment, M_{fc}. The percent shear connection is given by $\sum Q_n/A_sF_y$, where $\sum Q_n$ is the sum of the shear connector strength between the points of maximum and zero moment, A_s is area of steel cross section, and F_y is yield stress of the steel cross section. Curves are shown for three values of $Y2$, which is the distance from the top of the steel section to the center of the effective concrete flange. Although the curves were generated for a W16×31, they are representative of a wide range of cross sections because of the normalization procedure. A value of M_n/M_{fc} of about 0.9 is obtained from a partial shear connection value of 0.7. This relation can be extended to evaluating test results, in that if a measured to predicted moment capacity of 0.9 is obtained, then the measured to predicted shear connector capacity is 0.7. Because of this relationship, one can argue that an accurate evaluation of the shear connector strength must be made using carefully controlled elemental push-out tests, as opposed to evaluating stud strengths using only beam tests. The sensitivity of the stud strength to various parameters is difficult to discern if the strength is back calculated from beam test results. The best approach is to use a combination of the two test configurations, with the push-out tests being used to evaluate a wide range of parameters and formulate strength relationships, and with the beam tests used as confirmatory tests.

The remaining sections of this paper describe a research project conducted at Virginia Tech to evaluate the strong vs. weak shear stud position issue.[14] Results from a series of four composite beam tests are presented. Additionally, the results from a series of push-out tests are described. The push-out tests were part of another research project conducted prior to the beam tests.[15] An analysis of the results is presented which compares the experimental beam strengths with calculated values based on Equations 1 and 2, as well as values based on the push-out tests.

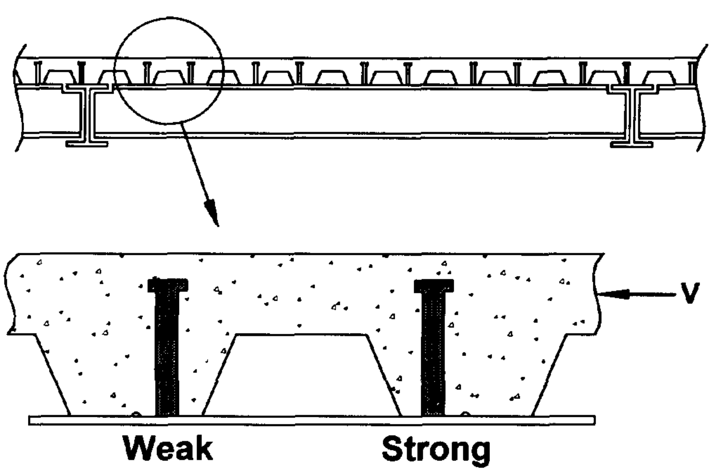

Fig. 1. Strong and weak position shear stud locations.

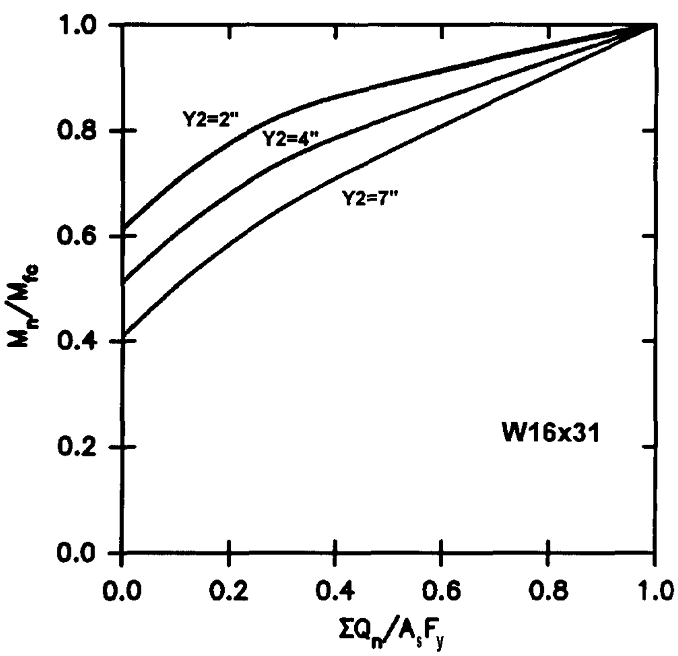

Fig. 2. Normalized moment versus percent shear connection.

STRENGTH AND STIFFNESS CALCULATION PROCEDURES

Test results were compared to calculated strength and stiffness values. The calculated shear stud strengths were determined using the LRFD Specification Equations 15-1 and 13-1 (Equations 1 and 2 in this paper). The flexural strength calculations were made using the equations given in the Commentary to the LRFD Specification. The elastic stiffness values were calculated using the lower bound moment of inertia defined in Part 4 of the LRFD Manual. Measured material properties were used in all calculations. The steel section properties that were measured (depth, flange thickness, flange width, and web thickness) were nearly identical to the tabular values given in Part 1 of the LRFD Manual. Therefore, tabulated cross-section properties for the steel shape were used in the calculations.

The flexural strength calculation procedure gives three equations for the nominal moment capacity, with the governing one determined based on the location of the plastic neutral axis (PNA). Yield stresses were determined separately for the web and flanges, thus the hybrid section idealization was used. All the specimens in this study were designed approximately 40 percent composite and the PNA was located in the web for all tests. The calculated moment capacity, M_c, using Equation C-I3-5,[2] is given by:

$$M_n = M_p - \left(\frac{C}{P_{yw}}\right)^2 M_{pw} + Ce \tag{3}$$

where

M_p = steel section plastic moment
C = compressive force in the concrete slab
P_{yw} = web yield force
M_{pw} = web plastic moment
e = distance from center of steel section to the center of the compressive stress block in the slab

The force C is given by:

$$C = \min \begin{cases} A_{sw}F_{yw} + 2A_{sf}F_{yf} \\ 0.85 f_c' A_c \\ \Sigma Q_n \end{cases} \tag{4}$$

where

A_{sw} = area of steel web
F_{yw} = yield stress of web steel
A_{sf} = area of steel flange
F_{yf} = yield stress of flange steel
A_c = area of concrete slab within effective width

The distance e is given by:

$$e = 0.5d + h_r + t_c - 0.5a \tag{5}$$

where

d = depth of steel section
t_c = slab thickness above the steel deck
a = depth of compression stress block

The lower bound moment of inertia was calculated using the moment of inertia of the steel beam plus an equivalent area of concrete, which is a function of the quantity of shear connection provided. The lower bound moment of inertia, I_{LB}, is given by

$$I_{LB} = I_x + A_s\left(Y_{ENA} - \frac{d}{2}\right)^2 + \left(\frac{\Sigma Q_n}{F_y}\right)(d + Y2 + Y_{ENA})^2 \tag{6}$$

where

I_x = moment of inertia about x-axis of structural steel section
Y_{ENA} = the distance from bottom of beam to elastic neutral axis (ENA) and is given by:

$$Y_{ENA} = \frac{\left[\frac{A_s d}{2} + \left(\frac{\Sigma Q_n}{F_y}\right)(d + Y2)\right]}{\left[A_s + \left(\frac{\Sigma Q_n}{F_y}\right)\right]} \tag{7}$$

TEST PROGRAM

Beam Test Specimens

The four composite beam tests were similarly constructed. Each specimen consisted of a single W16×31 A36 section with a composite slab attached. The span of each specimen was 30 ft and the total beam length was 32 ft because of a 1 ft cantilever at each end. The composite slab used for the beam tests was constructed using a 20 gage (0.036 in.), 3 in. deep, composite deck with a total of 6 in. of normal weight (145 pcf) concrete. The steel deck profile is shown in Figure 3. A single layer of welded wire fabric (WWF 6×6–W1.4×W1.4) was placed directly on the top of the deck. A total of 12 headed shear studs, ¾-in.×5 in. after welding, was used in each test. The studs were welded directly through the steel deck. The deck was placed with the ribs perpendicular to the beam span and the slab width was 81 in. A self-drilling screw was placed in each rib that did not have a shear stud in it, thus satisfying the requirement of having one fastener every 12 inches.[16] Deck seams were crimped (button-punched) twice on either side of centerline, resulting in an approximately 14-in. spacing. The only nominal difference in the specimens was the position of the shear studs. However, the material properties varied for each test.

All of the studs were placed in the strong position for Test 1 and the weak position for Test 2. In Tests 3 and 4 the stud positions were alternated, thus there were 3 in the strong position and 3 in the weak position along each half span. The stud nearest the support was placed in the strong position and the stud placement was alternated toward midspan. This resulted in a symmetric stud pattern in the two half-spans. (Test 4 was a repeat of the configuration used in Test 3 and was conducted due to the low concrete strengths obtained in Test 3.) The ribs in which shear studs were placed are shown in Figure 4. Note that all of the studs appear in the center of the deck ribs in Figure 4, however the studs were placed as described above.

The concrete slabs were formed using 6-in. cold-formed pour-stop material, resulting in three inches of cover on the 3-in. steel deck. A detail of the deck and slab is shown in Figure 5. After the concrete was placed, the slab was covered with plastic and cured for seven days. During this curing time the slab was kept moist. After seven days, the plastic and the pour-stop on the sides of the specimen were removed and the slab was allowed to cure for at least 21 additional days prior to testing. Concrete cylinders (4 in. × 8 in.) were cast at the same time as the concrete slab. The cylinders were kept adjacent to the slab, thus were covered with plastic and kept moist for the initial seven days.

Each specimen was partially supported during construction. Timber supports were used to prop the steel deck along the sides of the slab at the quarter points during concrete placement. This bracing prevented the slab from warping during the placement of the concrete and was not intended to shore the beam. The timber props were cut to allow for the deflection of the beam under the weight of the fresh concrete and were removed along with the pour-stop after seven days. Additional support was provided by concrete blocks placed under the four corners of each specimen to prevent rocking of the slab during construction and testing.

Beam Instrumentation

A standard instrumentation arrangement for strain, deflection, end rotation and slip measurement was used for all beam tests. All of the instruments were monitored using a computer controlled data acquisition system.

Eight strain gages were used to measure the strain through the beam cross-section at three different locations, resulting in a total of 24 strain gages per specimen. Two gages were placed at each of the following locations: the bottom of the top flange, the center of the web, the top of the bottom flange and the bottom of the bottom flange, as indicated in Figure 6. Gages were placed near one end support, at one quarter point and at the centerline.

Vertical deflections were measured at the centerline and the quarter points. Measurements were taken using linear wire transducers.

Slip measurements were made using potentiometers attached to the top flange of the beam. The potentiometers measured the relative movement between the top flange of the beam and a screw embedded in the concrete slab through a hole in the steel deck. A total of 12 potentiometers were used in each test, except Test 1, with one placed adjacent to each shear stud. Slip was not measured adjacent to the two studs nearest to midspan in Test 1. The slip measurement detail is shown in Figure 7.

End rotations were measured using two different techniques. Transducers were used to measure the upward deflection of the ends of the specimen and the support beam. The 1 ft overhang was assumed to rotate rigidly about the

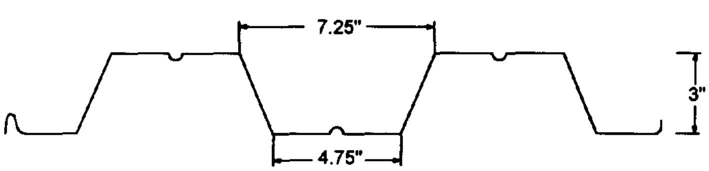

Fig. 3. Composite deck profile.

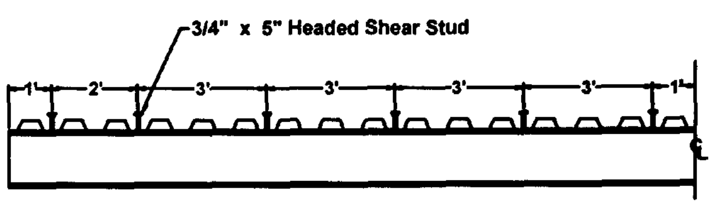

Fig. 4. Shear stud locations for composite beam specimens.

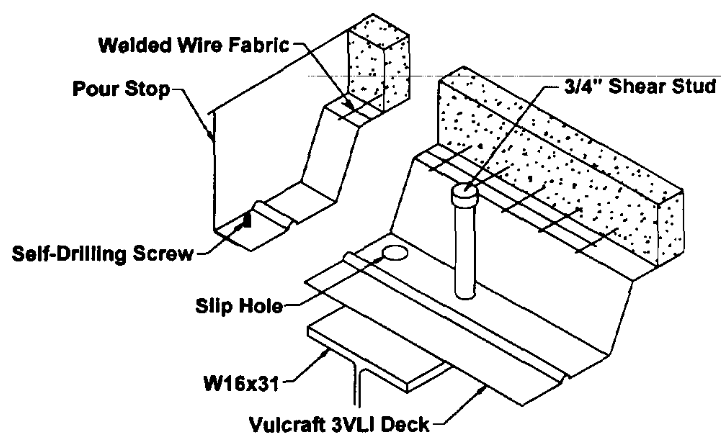

Fig. 5. Deck/slab detail.

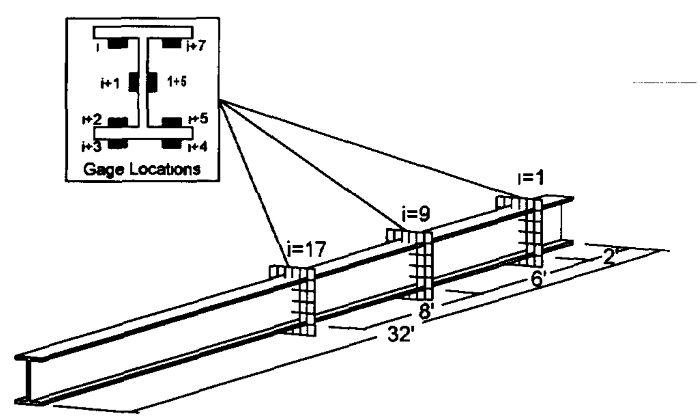

Fig. 6. Strain gage locations for composite beam specimens.

support, thus using the net upward deflection and the distance between the measurement and the support, the end rotation was calculated. Additionally, a digital level was used to measure the angle of the slab relative to horizontal, over the support, to the nearest 0.1 degrees.

In addition to the strain measurements already described, axial strain was measured in a select number of studs in Tests 2–4. This measurement was made using an innovative approach, adapted from bolt strain measurement techniques. However, due to problems with the gage installation technique, only a limited amount of usable data was obtained. For the benefit of those involved with similar research in the future, the instrumentation technique is presented here.

A cylindrical uniaxial strain gage, referred to as a bolt gage by the manufacturer, was inserted in the stud into a pre-drilled hole (approximately 0.1-in. diameter) after it had been welded to the beam. Lead wires were attached and electrical shrink tubing was placed over the lead wires to protect them during concrete placement. The end of the shrink tubing was embedded in a small amount of protective coating that was applied to the top of the stud. Subsequently the tubing was heated to conform to the general shape of the lead wire bundle. The lead wires were brought from the gage straight up through the concrete to prevent interference with the bonding between the concrete and the shear stud. A detail of the strain-gaged shear stud is shown in Figure 8.

The problems with the installation technique were attributed to the method used to insert the glue in the pre-drilled hole. The viscosity of the glue was such that the glue had to be worked into the hole using a blunt probe. Once the gage was inserted, it was worked back and forth to eliminate any air bubbles. A different technique, which utilizes a syringe to fill the hole from the bottom, has been used in other tests on composite members since the completion of the beam tests. The change in installation procedures appears to have corrected the problem.

Beam Load Apparatus and Test Procedure

A four-point loading system was used for all tests, with the loads spaced seven feet apart. The load was applied with a single hydraulic ram and distributed to the slab by a two-tier distribution system, as shown in Figure 9.

The load program was similar for all tests. An initial load, equal to approximately 15 percent of the calculated strength, was applied to seat the specimen and was then removed. The instrumentation was then re-initialized. Load increments were applied to the specimen until the load vs. centerline displacement response became non-linear. The specimen was then unloaded and then reloaded to the previous peak in three, approximately equal, increments. Displacement increments, based on the mid-span vertical deflection, were subsequently used to complete the test. The specimen was unloaded during the displacement controlled

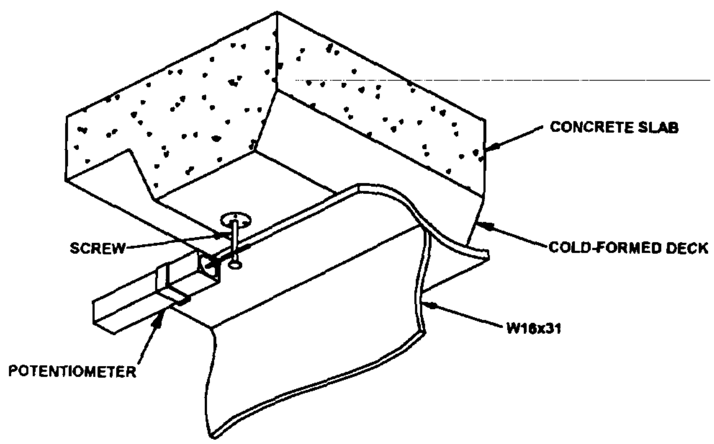

Fig. 7. Slip measurement detail.

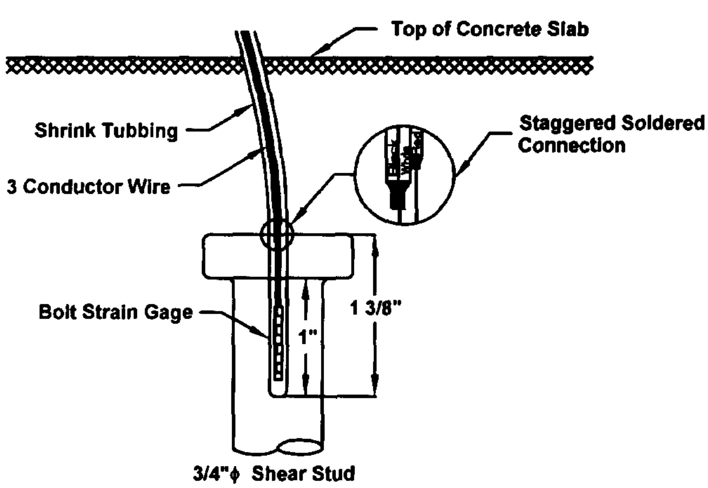

Fig. 8. Detail of strain gage in a shear stud.

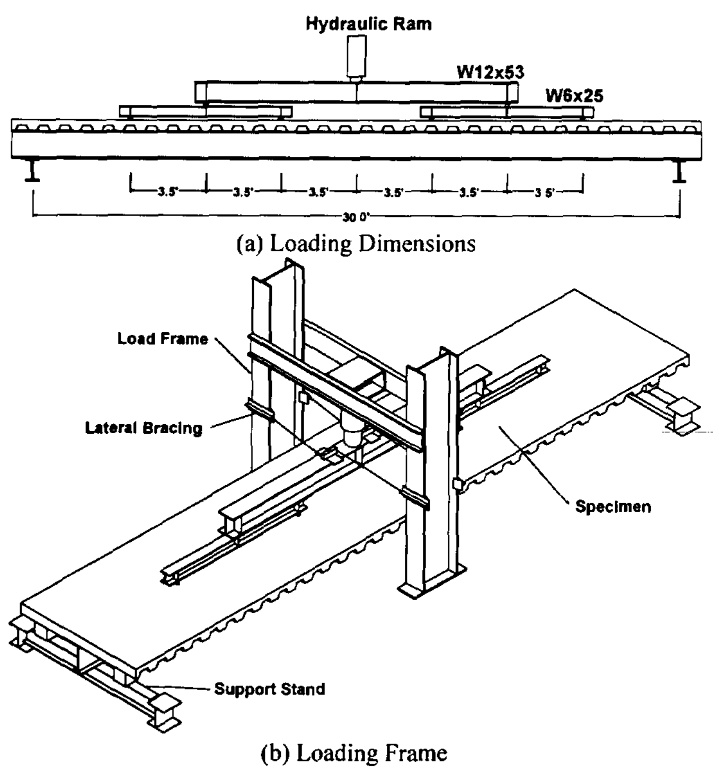

Fig. 9. Loading frame for composite beam specimens.

phase if it was necessary to adjust the loading apparatus.

Push-Out Test Specimens

A total of eight push-out specimens were fabricated, four with studs in the weak position and four with studs in the strong position. These tests were performed as part of another study reported by Sublett, et al.[15] The push-out tests were constructed using the same deck profile and shear stud size that were used in the beam tests. Each half of a push-out specimen was constructed by attaching a piece of 3-in. deep composite steel deck to a W5×11. The ribs of the deck were oriented perpendicular to the length of the WT section. One or two shear studs (¾-in. × 5 in. after welding) were welded through the deck to the structural tee. Two each of the strong and weak position groups had one stud per specimen half. The other two specimens in each group had two studs, spaced 12 inches apart along the length of the WT, on each specimen half. A normal weight concrete slab, 6-in. thick by 24-in. wide by 36-in., was cast on the deck. Welded wire fabric (WWF 6×6–W1.4×W1.4) was placed on top of the deck prior to casting the concrete. The specimens were covered and kept moist for seven days, at which time the forms were removed. Concrete test cylinders (4 in. × 8 in.) were cast along with the push-out specimens and cured in a similar manner.

After the slabs had cured, two halves were bolted through the stems of the structural tees to form a complete specimen. This manner of casting permitted the slabs to be cast horizontally and from the same batch of concrete. By doing this the concrete curing problems associated with either casting the specimens vertically or from different mixes were avoided. Overlapping the stems of the tees induced an eccentricity in the built-up steel section, as compared to using a rolled H-shape. The effect due to this eccentricity was deemed negligible.

Push-Out Test Instrumentation

A standard instrumentation arrangement for measurement of slip, shear load, and normal load was used for all tests. Slip between the steel deck and steel section was measured at two locations on each half of the push-out specimen using mechanical dial gages. The applied shear load was measured using a load cell that was part of the universal test machine. A normal force was applied to the slab, as described in the next section of the paper, and monitored using a electronic load cell.

Push-Out Load Apparatus and Test Procedure

To prevent premature separation between the slab and steel deck, in a direction normal to the slab surface, a yoke device was placed on the specimen. This manner of loading simulated the gravity load placed on a slab in a composite beam/slab arrangement. A load cell and hydraulic ram were part of the yoke assembly. The specimen configuration with the yoke in place is shown in Figure 10.

Specimens were placed in a universal testing machine on an elastomeric bearing pad, which minimized the effects caused by any unevenness in the bottom of the specimen. Shear load was applied with the universal testing machine in load increments equal to approximately 10 percent of the expected specimen capacity. Displacement control was used once the load levels reached approximately 80 percent of the expected capacity.

Load normal to the slab surface was applied using the yoke assembly. The load was monitored using a load cell and controlled with a hydraulic hand pump and ram. The normal load was increased along with the applied shear load. The normal load was approximately 10 percent of the applied shear load throughout a test.

Material Tests

Standard material tests were conducted on the concrete and steel components. The concrete cylinders were tested to determine compressive strength on the days of the various beam and push-out tests. Tensile coupons (0.5 in. width, 2 in. gage) were cut and machined from both the web and one flange of each structural steel shape, as well as from flat widths of the steel deck profile. The ultimate tensile stress for

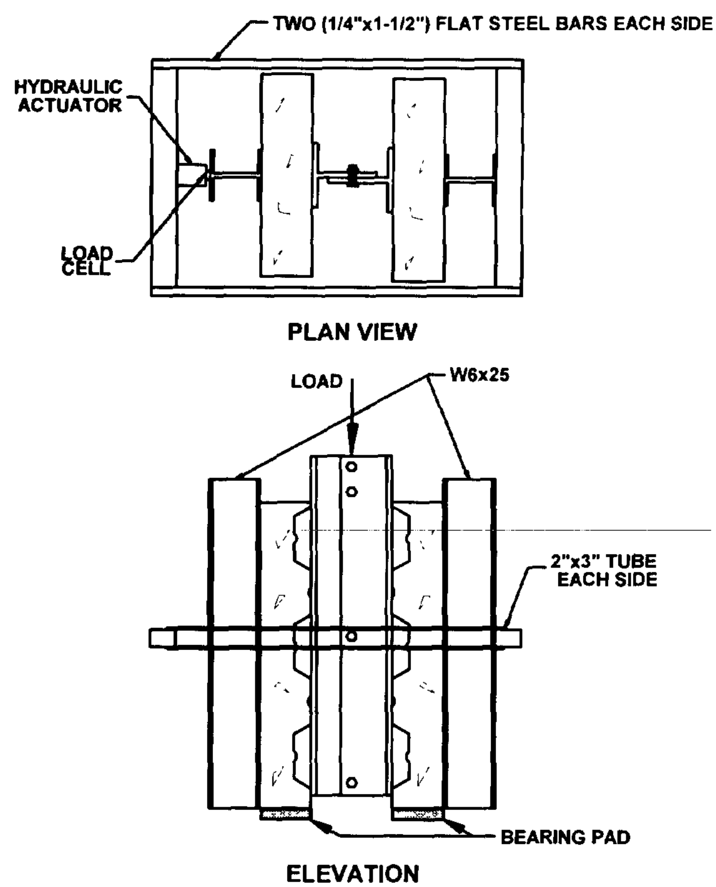

Fig. 10. Push-out specimen schematic.

the shear studs was reported by the manufacturer. Material properties are given in Table 1.

TEST RESULTS

Beam Test Results

The observed behavior was similar for all beam tests, but notable differences exist. A normalized moment versus deflection plot of the four tests is shown in Figure 11. The experimental moments, M_e, were normalized with calculated moment strengths using measured material properties and the procedure described previously. Note that the plots in Figure 11 include the non-composite load and corresponding deflection. The vertical mid-span deflection, Δ, was normalized with Δ_H. The deflection corresponding to the point where the elastic stiffness, calculated using the lower bound moment of inertia, intersects the calculated moment strength is defined as Δ_H.

As indicated in Figure 11, all tests exhibited a ductile response. The moment versus deflection response in Tests 1, 3, and 4 (strong and alternating stud position tests) remained elastic up to a normalized moment of approximately 0.6. Test 2 (weak stud position test) remained elastic up to a normalized moment of approximately 0.4.

The behavior of the shear studs was distinctly different for the strong and weak position studs. Strong position studs

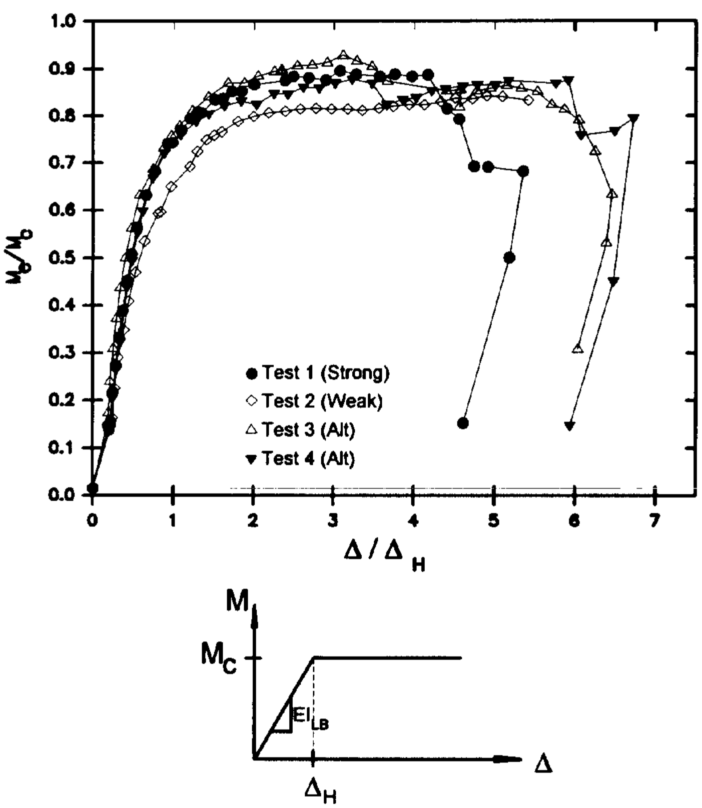

Fig. 11. Normalized midspan moment versus displacement for composite beam specimens.

Table 1. Material Properties for Composite Beam Specimens

Test	Flange F_{yf} (ksi)	Flange F_{uf} (ksi)	Web F_{yw} (ksi)	Web F_{uw} (ksi)	Slab f_c' (ksi)
1	42.0	68.8	47.0	71.9	4.81
2	41.9	70.4	45.4	73.8	3.20
3	42.5	70.1	47.0	75.7	2.28
4	43.6	63.4	49.1	62.9	4.99
Shear Studs: F_u = 64.8 ksi					
Steel Deck: F_y = 40.3 ksi F_u = 53.6 ksi					

exhibited failure by developing concrete shear cones or by shearing off in the shank. Weak position studs exhibited failure by punching through the deck rib without developing a significant shear cone in the concrete or shearing in the stud shank. In Tests 1, 3 and 4, one or two of the strong position studs closest to one of the specimen supports sheared off in the shank. However, the weak position stud between the two strong position studs in Tests 3 and 4 did not shear off, but punched through the deck web and remained attached to the beam.

Push-Out Test Results

An average strength of 13.55 kips per stud was obtained from the four push-out tests in which the studs were in the weak position. The concrete compressive strength was similar for each of the tests, with an average for the four tests of 4.27 ksi. There was no significant difference between the strengths (load per stud) obtained from the tests with one stud per specimen half and the tests with two studs per specimen half. In all of the weak position tests, failure occurred by the studs punching through the adjacent web of the steel deck. A small wedge of concrete between the stud and the deck web was crushed or broken out in each of the tests. The deck was noticeably bulged out adjacent to the stud prior to reaching the maximum applied shear load. This behavior was an indication that the load was being primarily resisted by the deck.

An average of 18.82 kips per stud was obtained from the three push-out tests in which the studs were in the strong position. The average concrete strength was 4.57 ksi. The results for the fourth specimen were inexplicably low and are not included in the average. The decision to omit this test was based on the other three tests plus an additional 11 tests, similarly constructed, that were part of a proprietary study in which double angle sections were used as the base members instead of structural tees. There was no significant difference between the strengths (load per stud) obtained from the test with one stud per specimen half and the tests with two studs per specimen half. In all of the strong position tests, the strength was limited by the development of a failure surface

Table 2. Experimental and Calculated Results											
Test	**Q_c (kips)**	**Q_{po} (kips)**	**Q_{cb} (kips)**	**M_c (ft-kips)**	**M_{po} (ft-kips)**	**M_e (ft-kips)**	**Q_{cb}/Q_c**	**Q_{po}/Q_c**	**Q_{cb}/Q_{po}**	**M_e/M_c**	**M_e/M_{po}**
1 (str.)	28.7	19.3	18.8	344	303	304	0.66	0.67	0.97	0.88	1.00
2 (weak)	22.6	13.6	13.4	316	274	273	0.59	0.60	0.99	0.87	1.00
3 (alt.)	17.5	13.3	14.5	297	277	283	0.83	0.76	1.09	0.95	1.02
4 (alt.)	28.7	16.6	17.0	354	301	303	0.59	0.58	1.02	0.86	1.01

All values based on measured material properties

Q_c = calculated stud strength using Equations 1 and 2.

Q_{po} = calculated stud strength using Equation 8 and concrete strength from beam test for strong position studs and a constant value of 13.55 kips for the weak position studs.

Q_{cb} = calculated stud strength using Equation 3 with M_e in place of M_n.

M_c = calculated moment strength using Equation 3 and Q_c.

M_{po} = calculated moment strength using Equation 3 and Q_{po}.

M_e = maximum applied experimental moment including weight of specimen, load beams, and applied ram load.

in the concrete. None of the shear studs exhibited a shear failure in the shank.

The response of the studs in the weak position, in terms of load versus slip, was more ductile than that of the studs in the strong position. This difference is attributed to the way in which the load appeared to be resisted, based on the observed failure modes. The failure mode for the strong position tests was brittle; concrete shear, and the failure mode for the weak position tests was more ductile; bearing and eventual tearing of the steel deck web. A typical plot of load versus slip behavior for strong and weak position shear studs is illustrated in Figure 12.

ANALYSIS OF RESULTS

The results of the beam and push-out tests were compared with calculated values. Several comparisons have been made and are presented in this section. The calculated moment values were based on the expressions described previously in this paper, using measured material properties and values of shear connector strength that were calculated using the LRFD specification or taken from normalized push-out test results. Shear connector strength was also back calculated using the experimental moment values obtained from the beam tests. The results of each of these calculations and comparisons are given in Table 2.

The values Q_c given in Table 2 are calculated stud strengths. These were determined using Equations 1 and 2 with measured material properties. Stud strengths Q_{cb}, were back-calculated using the experimental moment from the beam tests, measured material properties and the calculation procedure described previously.

Because the shear studs in the weak position, in both the push-out and beam tests, failed by punching through the web of the deck it was hypothesized that their strength was not primarily a function of concrete strength. Rather, the stud strength is primarily a function of the steel deck strength (i.e., the yield stress of the steel deck). Certainly some interaction between the concrete and the deck occurred, but the dominant component was the steel deck. Based on this hypothesis, the weak position push-out test strengths were averaged and used for all the weak position stud strengths in the calculations for the beam tests. No adjustment was made to account for variable concrete strengths.

The strength of the shear studs in the strong position was taken as a function of the concrete strength. The strong

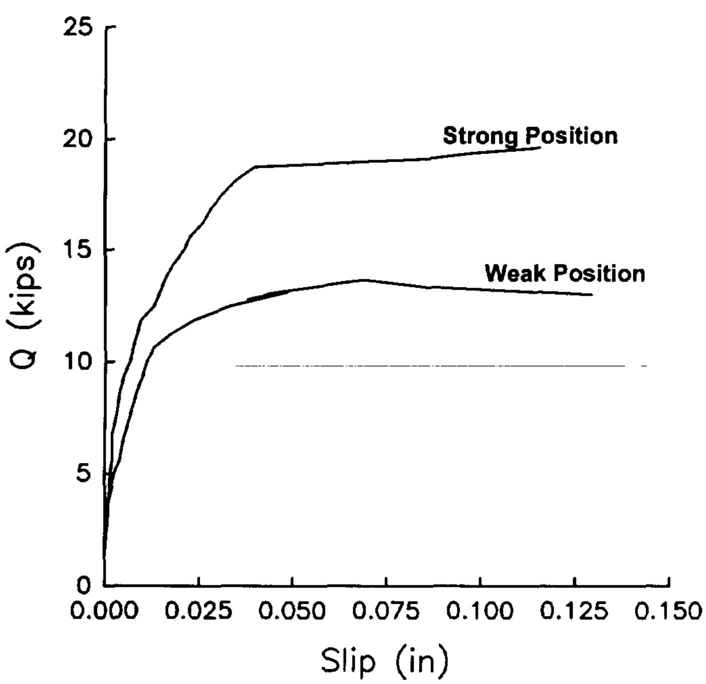

Fig. 12. Load vs. slip for strong and weak position shear studs for push-out tests.

position stud strengths in the beam tests were calculated by normalizing the push-out test results with the concrete strengths as given by:

$$Q_{po} = 18.82\text{kips}\sqrt{\frac{f_c'}{4.57 \text{ ksi}}} \qquad (8)$$

where f_c' is the concrete compressive strength for the composite beam test, 18.82 kips is the average stud strength from the push-out tests, and 4.57 ksi is the concrete compressive strength from the push-out tests. The Q_{po} values represent stud strengths for the beam tests based on push-out test results.

Equation 8 was used to calculate the values for Q_{po} in the Test 1, and the constant value reported in the push-out results section was used for Test 2. The strong and weak position values were averaged in determining the values for Q_{po} in Tests 3 and 4.

Three values of moment are shown in Table 2, M_c, M_{po}, and M_e. The first, M_c, was calculated using Q_c, M_{po} was calculated using Q_{po}, and M_e represents the maximum experimental moment from the beam tests. Various ratios of stud strengths and moment strengths are also given in Table 2.

Two trends are clearly indicated by the results in Table 2. One of these is that the stud strengths predicted by Equations 1 and 2 do not compare favorably to the values from the push-out tests or the beam tests. This is indicated by the ratios Q_{cb} / Q_c and Q_{po} / Q_c. The second trend that is evident is that the results from the push-out tests and beam tests compare very well, as indicated by the ratio Q_{cb} / Q_{po}.

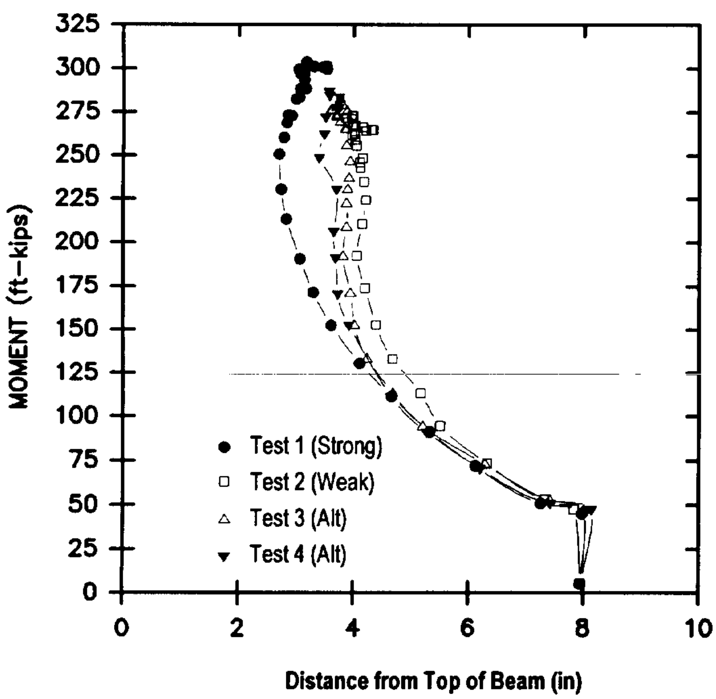

Fig. 13. Applied moment versus position of neutral axis for composite beam specimens.

Table 3.
Experimental and Calculated Neutral Axis Positions

Test	PNA_e (in.)	PNA_c (in.)	PNA_{po} (in.)	PNA_{cb} (in.)
1	2.7	0.78	3.13	3.15
2	4.2	2.32	4.56	4.61
3	3.9	3.88	4.64	4.57
4	3.6	0.88	3.85	3.76
All values of PNA are measured from top of steel section.				

Additionally, while a comparison between strong and weak position shear stud strengths indicates some difference, the more pronounced and significant difference is between the predicted values and the beam and push-out test results. The ratios Q_{cb} / Q_c or Q_{po} / Q_c indicate the strong position values are approximately 70 percent of the predicted and the weak position values are approximately 60 percent of predicted.

The sensitivity of the moment strength to the shear stud strength is also illustrated in the results. Values of experimental to calculated shear stud strengths varied between 0.59 and 0.83, while the experimental to calculated moment values, indicated by M_e / M_c, varied between 0.85 and 0.94. The relationship between shear connection and moment strength is illustrated for the W16×31 used in this study by the normalized moment versus shear connection relationship in Figure 2. Although as previously indicated, this relationship is generally presented in the context of partial composite design, it can also be used to consider the reduction in moment strength due to a reduction in shear connector strength.

The strain data collected from the beam tests also indicate the difference between strong, weak, and alternating position studs. The relationship between the position of neutral axis and the applied moment is illustrated in Figure 13. A linear regression analysis was performed using the eight strain readings located at midspan in the steel section to determine the neutral axis location. As noted in Figure 13, the strong position studs resulted in the neutral axis being higher in the steel than for the weak or alternating tests. Further, the position for the alternating tests fell between the strong and weak values.

Using Figure 13, the plastic neutral axis position can be established by visually locating the point at which the slope of line is approximately vertical. These values are given in Table 3. Also shown in Table 3 are calculated values of the plastic neutral axis based on Q_c, Q_{po}, and Q_{cb}. Note that the calculated values using either Q_{po} or Q_{cb} correspond more closely to the experimental values than do the positions calculated using Q_c, in all but Test 3.

DESIGN IMPLICATIONS

The implications of the study described here, as well as previous studies, on composite beam design merit consideration at this point. Based on the test results presented in the previous sections, it is evident that Equation 2 is not conservative in all cases. Specifically, if single shear studs are used, as opposed to pairs of studs, the equation over-predicts the strength of the stud. Based on a review of previous studies[3,12,13] the authors believe that Equation 1 is conservative for designs in which two studs per rib are utilized. No general modifications to the form of the equation are proposed at this time. Until such modifications are formulated, the following recommendations are offered:

1. The stud reduction factor should not exceed 0.75 for cases in which there is one stud in a rib.
2. Detail all single studs in the strong position. The implementation of this detail requires coordination between the structural engineer and the stud contractor to effectively relay the objective of the detail.
3. Use 50 percent composite action as a minimum, i.e., keep $\Sigma Q_n/A_sF_y$ greater than or equal to 0.50. This will minimize the adverse effect of under-strength studs on the design moment strength, as reflected by the trend of the curves in Figure 2.

The result of implementing the above recommendations is an increase in the number of shear studs for designs utilizing one stud per rib. This will obviously result in a small increase in the cost, however the percentage increase in the in-place cost of the composite beam for these situations will be minor. Certainly in view of the questions that have been raised regarding the strength of the studs, the increase is warranted.

A consideration in future composite beam studies and modifications to the specification procedures should be the application of a strength reduction factor, ϕ, to the shear studs. In the current AISC LRFD specification[2] a single strength reduction factor is applied to the nominal moment strength for the composite beam system, which includes the variable effects of the shear connectors. However, the flexural strength of the beam and the shear strength needed at the steel concrete interface are associated with different modes of behavior and limit states and therefore merit separate consideration. If this approach were pursued, one would expect that the value of ϕ for the flexural limit state may increase above the present value of 0.85, thus making more efficient use of the steel shape which is the dominant component in the cost of the composite beam. At the same time the variability that exists in the shear stud strength would be reflected in a ϕ value for shear studs.

The flexural and shear stud limit states are treated independently in other limit states design specifications.[17,18] The nominal strengths, as well as the stud reduction factors, vary between the three specifications. A graphical comparison of the three specifications for the 3-in. deep composite deck shown in Figure 3 is given in Figure 14. The differences illustrated in Figure 14 in part reflect the uncertainty that exists at the present time regarding shear connector strength.

SUMMARY AND CONCLUSIONS

Results were described for a recent study conducted at Virginia Tech in which a series of push-out tests and

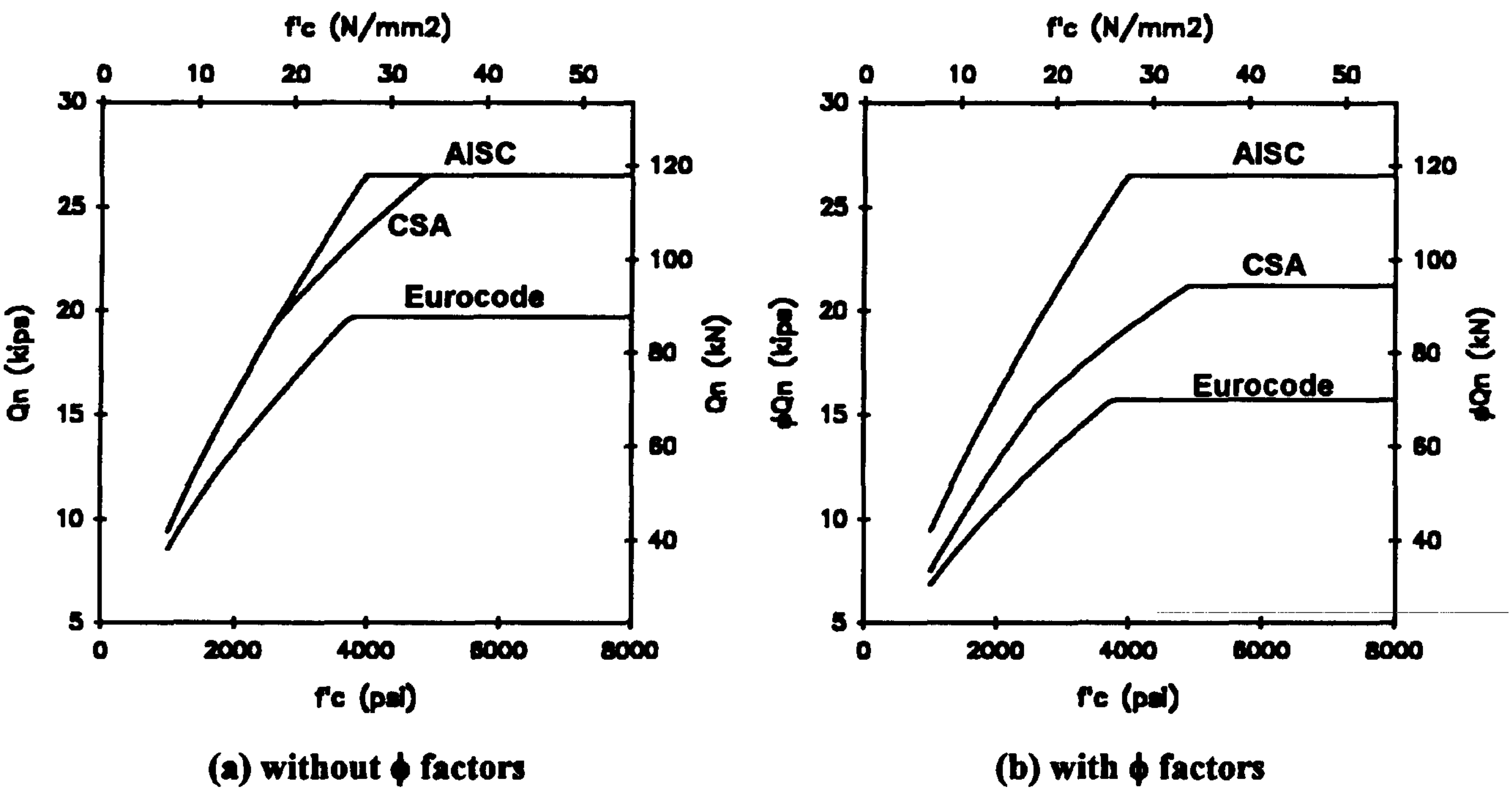

Fig. 14. Shear strength comparison for AISC, CSA, and Eurocode specifications.

composite beam tests were conducted. The results were consistent with other recent studies reported in the literature, in that the strength of shear studs placed in the ribs of steel deck oriented transverse to the beam span, calculated using Equation 2, were higher than measured values. Review of the test data used to develop Equation 2 indicated that the majority of the tests were conducted with the shear studs placed in pairs. Equation 2, when combined with Equation 1, accurately reflects the stud strength for these cases.

Specific modifications to Equation 2 were not proposed, as further evaluation of existing procedures is required. The hypothesis regarding the influence of the steel deck material properties on the stud strength must be evaluated at the same time and perhaps included as a modification to one of the existing methods. This hypothesis, while not conclusively verified, was supported by the results of the Virginia Tech research program.

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NOMENCLATURE

A_c = area of concrete slab within effective width
A_s = area of steel cross section
A_{sc} = cross sectional area of a stud shear connector
A_{sf} = area of steel flange
A_{sw} = area of steel web
a = depth of compression stress block
C = compressive force in concrete slab
d = depth of steel section
E_c = modulus of elasticity of concrete
e = distance from center of steel section to the center of the compressive stress block in the slab

F_u = minimum specified tensile stress of stud shear connector

F_y = yield stress of steel cross section

F_{yf} = yield stress of steel web

F_{yw} = yield stress of steel web

f_c' = specified compressive strength of concrete

H_s = length of shear stud after welding

h_r = nominal rib height

I_{LB} = lower bound moment of inertia

I_x = moment of inertia about x-axis of structural steel section

M_c = moment strength calculated using Q_c

M_e = maximum experimental moment

M_{fc} = fully composite moment strength

M_n = nominal moment strength

M_p = steel section plastic moment strength

M_{po} = moment strength calculated using Q_{po}

M_{pw} = web plastic moment

N_r = number of studs in one rib at a beam intersection

P_{yw} = web yield force

Q_c = calculated stud strength using Equations 1 and 2

Q_{cb} = stud strength calculated using M_e and Equation 3.

Q_{po} = stud strength calculated using push-out test results

Q_n = nominal strength of a shear stud

t_c = slab thickness above the steel deck

w_r = average width of concrete rib

Y_{con} = distance from top of steel beam to top of concrete

Y_{ENA} = distance from bottom of beam to elastic neutral axis

$Y2$ = $Y_{con} – a/2$

ΣQ_n = sum of strengths of shear connectors