Moment Resisting Connections for Mixed Construction

NEIL M. HAWKINS, DENIS MITCHELL, AND CHARLES W. ROEDER

Structural steel and reinforced concrete are often combined in modern structures.[2] While such mixed construction offers a number of advantages, it also requires reliable connections between two very different materials. Headed metal studs are one method for connecting structural steel and concrete. Studs have been used for many years to provide shear connections between steel beams and concrete floor slabs.[4,5,8] More recently, in response to mixed and precast concrete construction needs, their range of application has been expanded to include connections such as those shown in Fig. 1. In such moment-resisting connections there is a combined axial force and shear loading on each stud. However, since the studs act as a group, the best distribution of the applied forces between individual studs for design purposes is not obvious. Information is needed on the factors affecting the strength and behavior of such connections and how best to design them.

This paper describes the results of a series of 22 tests[1,7] performed at the University of Washington on specimens of the type shown in Fig. 1. These specimens provide basic information relevant to two connection types found in practice. For mixed construction, the steel wide-flange section can be said to represent the end of a main member of a steel frame for which a moment resistant connection is required to a concrete column or wall. Alternatively, for either mixed or precast concrete construction,[6] the steel wide-flange simulates part of a corbel connection used to support another member. The tests covered a variety of moment, shear, and stud embedment conditions, and a widely varying range of connection behavior was obtained. The results of these tests are compared with the results of previous studies[3,4] and existing design recommendations.[5,6,8] Conclusions as to the effectiveness of these connections are presented and a recommended design procedure described.

Neil M. Hawkins is Professor and Chairman, Dept. of Civil Engineering, University of Washington, Seattle, Washington.
Denis Mitchell is Associate Professor, Dept. of Civil Engineering and Applied Mechanics, McGill University, Montreal, Quebec, Canada.
Charles W. Roeder is Assistant Professor, Dept. of Civil Engineering, University of Washington, Seattle, Washington.

BACKGROUND

The tensile and shear capacities of individual studs have been investigated by several writers.[3,4,8] In addition, individual studs have been tested under a number of combined loading conditions.[3] Design equations[5,6,9] have been developed for predicting the tensile and shear capacities of single studs and, based on those expressions, interaction relationships have been developed for predicting the ultimate capacities of individual studs under combined shear and tensile loads.

Shear and tensile tests on individual studs have shown that there are two possible modes of failure. If the stud is adequately embedded, failure occurs in the shank of the stud. The maximum capacity of the steel in that shank is developed. If the stud has an inadequate embedment length, then a pull-out cone develops in the concrete, as shown in Fig. 2. The strength of concrete determines the capacity of the stud.

For tensile loading and an adequate embedment length, the capacity equals the tensile strength of the steel in the shank of the stud. For an inadequate embedment length, the capacity equals the surface area of the pull-out cone, shown in Fig. 2, multiplied by the limiting tensile capacity for that surface. Design equations are based directly on those concepts.

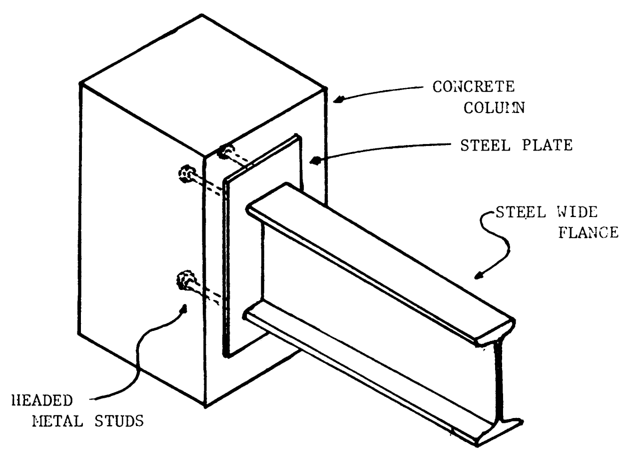

Fig. 1. Typical steel beam—concrete column connection

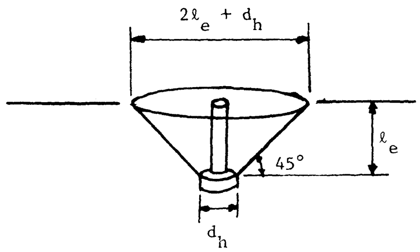

Fig. 2. Assumed geometry for concrete pull-out cone failure

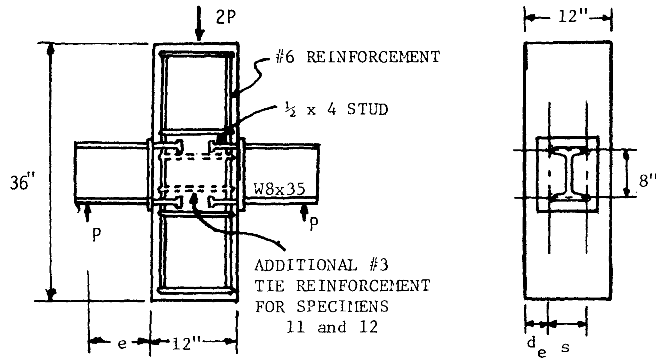

Fig. 3. Details of typical Group I specimen

For shear loading, the capacity for an inadequate embedment cannot be related directly to the characteristics of the pull-out cone. Tests[3,8] have shown that design expressions must vary with the concrete type and embedment conditions.

As a result of past research, there are reasonably reliable methods for predicting the strength of individual studs. However, information on the behavior of stud groups and methods for predicting their capacities are lacking. This research was intended to partially fill that void. The behavior of connections such as those shown in Fig. 1 depends on the distribution of the applied forces to the studs. That distribution varies with the shear force and bending moment acting at the concrete to steel interface as well as the embedment conditions for the stud. The authors are unaware of any previous research into the strength and behavior of connections of the type shown in Fig. 1.

EXPERIMENTS

Tests were conducted on two groups of 12 and 10 specimens, respectively. All studs in the Group I tests were 4 in. long, while those in the Group II tests were 6 in. long. According to the design expressions of Refs. 5, 6, and 9, the 4-in. embedment length used for the studs in Group I was more than adequate to prevent a pull-out failure. That was not the case; therefore, the Group II tests were made using studs with a larger embedment length.

Group I specimens had the form shown in Fig. 3. A pair of W8X35 beams were fully welded to a 0.5-in.-thick steel plate, which was connected to the reinforced concrete column with four 0.5-in.-diameter, 4-in.-long studs. The horizontal stud spacing, s, edge distance, d_e, and plate size were varied, as shown in Fig. 3 and Table 1. Those variations permitted investigation of the effect of overlap of the pull-out cones on the capacity of the connection and the mode of failure. Specimens 1, 2, 3, 9, and 10 were identical. They had a 6-in. stud spacing and 3-in. edge distance. The eccentricity of the loading on the beam, e, Fig. 3, was varied between 3 in. and 12 in. That variation produced a wide range of shear force and bending moment combinations. Specimens 11 and 12 were similar to specimens 1 through 10, except that additional stirrups were added to the column and within the depth of the steel beam, as shown by broken lines in Fig. 3. Pull-out failures of the tension studs occurred for several of the specimens 1 through 10; the additional ties were inserted in specimens 11 and 12 to determine if they would alter the mode of failure or significantly increase the pull-out capacity of the studs. Specimens 4, 5, 6, 7, 8, and 12 were all loaded with a 3-in. eccentricity. Specimens 4, 5, and 6 employed an 8-in. horizontal stud

Table 1. Properties of Test Specimens, Group I (Ref. 1)

Specimen	Eccentricity, e (in.)	d (in.)	Edge Distance, d_e (in.)	s (in.)	Plate Size (in.)	f'_c (ksi)	f_u (ksi)
1	3	12	3	6	8 x 10 x ½	5.4	80
2	6	12	3	6	8 x 10 x ½	6.9	80
3	12	12	3	6	8 x 10 x ½	5.9	80
4	3	14	2	8	10 x 10 x ½	4.9	80
5	3	12	3	8	10 x 10 x ½	5.0	80
6	3	16	4	8	10 x 10 x ½	6.35	80
7	3	14	4	6	8 x 10 x ½	5.1	80
8	3	12	4	4	6 x 10 x ½	4.9	80
9	4.5	12	3	6	8 x 10 x ½	5.6	80
10	9	12	3	6	8 x 10 x ½	4.4	80
11	6	12	3	6	8 x 10 x ½	4.3	80
12	3	12	3	6	8 x 10 x ½	4.3	80

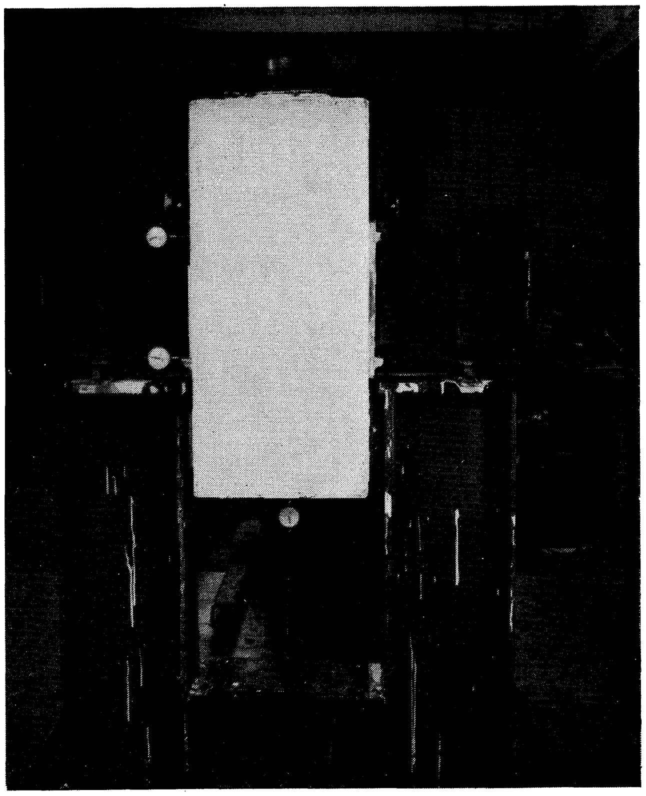

Fig. 4. Test set-up

spacing, with the distance between the center of the stud and the parallel edge of the column varied between 2 in. and 4 in. For specimens 5 and 6, the potential pull-out cone overlapped the edge of the column. Specimens 7 and 8 had

Table 2. Properties of Test Specimens, Group II (Ref. 7)

Specimen	Eccentricity, e (in.)	S_1 (in.)	Number of Compression Studs	Number of Tension Studs	f'_c (ksi)	f_u (ksi)
1	9	8	2	2	2.99	80.0
2	6	8	2	2	5.45	80.0
3	3	8	2	2	5.06	80.0
4	6	12	2	2	4.04	80.0
5	3	12	2	2	3.73	80.0
7	6	12	4	2	4.14	73
8	9	12	4	2	3.81	73
9	6	8	2	2	7.07	73
10	9	12	4	3	4.66	80.0

a 4-in. edge distance and a variable stud spacing. They were used to examine the effect of cone overlap. The loads applied, as shown in Fig. 3, were increased monotonically until failure occurred. A photograph of the test set-up is shown in Fig. 4.

Group II specimens had the form shown in Fig. 5 and the properties listed in Table 2. Wide-flange beams were welded to a 0.5-in.-thick steel plate to which metal studs were attached. All studs were 6 in. long with a 6-in. horizontal spacing center-to-center of the outer studs and a 3-in. edge distance between the outer studs and the parallel faces of the column. The wide flange was a W8X35 for specimens 1, 2, 3, and 9 and a W12X50 for specimens 4, 5, 6, 7, 8, and 10. The vertical stud spacing, S_1, was varied with the depth of the beam. The specimens were loaded similarly to Group I, with the eccentricity varied between 3 in. and 9 in. Specimens 7, 8, and 10 had two additional studs in the compression zone, located as shown by broken lines in Fig. 5. Specimen 10 had an additional stud in the tensile zone,

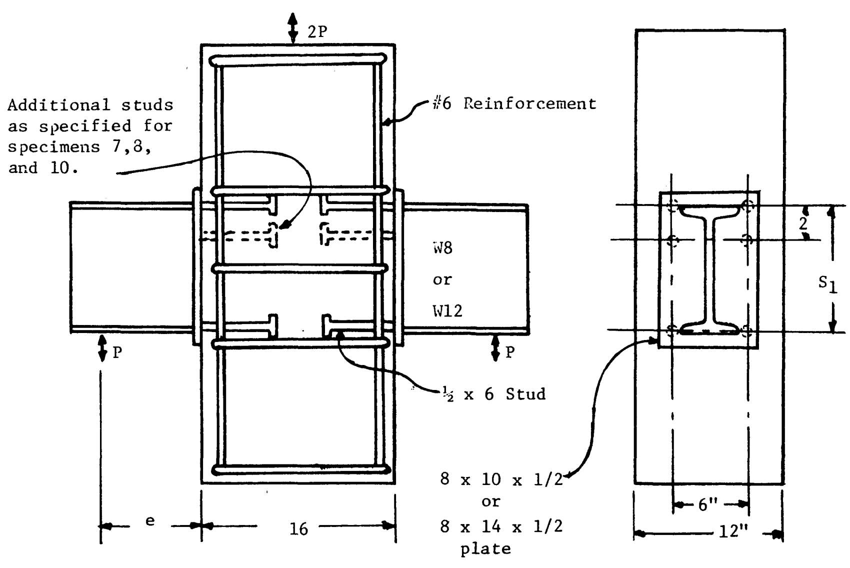

Fig. 5. Details of typical Group II specimen

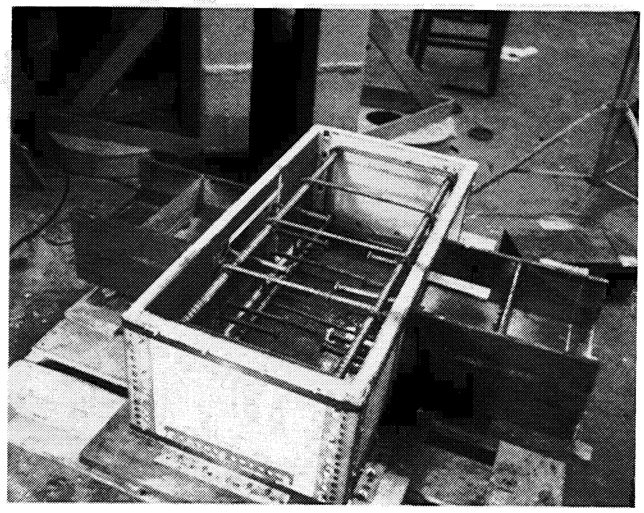

Fig. 6. Typical specimen prior to casting

centrally located between the two outer studs. These specimens were designed to investigate the effect of the added studs on the connection capacity. Specimen 9 had the same design as 1, 2, and 3, but it was reversed-cyclically loaded. Cycling was performed by loading the specimen monotonically to a desired load level, unloading, inverting the specimen and loading in the opposite direction until the desired load level was again attained. The objective was to evaluate the behavior of the connection under the type of loading likely during a severe earthquake.

A specimen prior to casting of the concrete is shown in Fig. 6. The steel stub beams with the studs attached were positioned through the formwork in such a manner that the end plate for the beam was not embedded into the face of the concrete column. Several 6 x 12-in. test cylinders were cast with each specimen and tested to failure on the same day as the specimen. The measured strengths ranged from 7,000 psi down to 3,000 psi. Values for each specimen are tabulated in Tables 1 and 2. The studs for the specimens were supplied and installed by the Seattle Office of the Nelson Division of the TRW Corporation. The tensile capacity of the studs, f_u , was taken as the average value obtained from tests on six studs supplied with each batch of stub beams. The corresponding capacities are listed in Table 1 and 2. The Grade 60 reinforcing bars for the column had the sizes and distribution shown in Figs. 3 and 5.

The 22 specimens were tested to failure in the U-frame shown in Fig. 4. Load was applied through a swivel head to the top of the concrete column and reacted through rollers extending across the width of the bottom flange of the stub beams. Because of the symmetry of the test specimen and the load application, the shear force on each connection was one-half the total load and the moment equaled the shear force multiplied by distance from the column face to the supporting roller. That distance is termed the eccentricity.

Dial gages were mounted at the top and bottom flanges of the beams to measure rotations and horizontal displacements of the beam with respect to the column. Another gage was clamped to the U-frame to measure the center line deflection of the specimen. Two other gages were used to measure the relative vertical slip between the steel plate and the adjacent concrete column. In combination, those gages permitted measurement of total deflections of the column relative to its supports and also permitted determination of how much of that deflection was caused by slip, by rotation of the end plate of the beam relative to the column and by beam bending. The tension studs of specimens 1 through 8 of Group I also had electrical resistance strain gages mounted on them prior to casting of the concrete column.

THEORETICAL STRENGTH CALCULATIONS

One of the objectives of these tests was to evaluate the effectiveness of alternate design procedures. Therefore, connection strengths were predicted by several procedures, and those predictions compared to the test results.

The *tensile capacity* of individual studs was determined using the pull-out core concept shown in Fig. 2. The ultimate tensile capacity for a concrete failure, P_{uc} ,[6] was taken as:

$$P_{uc} = \phi_c k 4 \sqrt{f'_c}\, A_o \quad \text{(psi)} \qquad (1)$$

with the limitation that P_{uc} could not exceed the ultimate tensile capacity of the steel taken as:

$$P_{uc} \leq \phi_s A_s f_u \qquad (2)$$

The strength reduction factors for the steel and concrete, ϕ_s and ϕ_c , were taken as 0.9 and 0.85, respectively. The terms f'_c and f_u are the ultimate strengths of the concrete and the steel, A_o and A_s are the areas of the shear cone surface and the cross-sectional area of the steel stud, and k is a factor which depends on concrete type (k = 1.0 for normal weight concrete). The cone surface area, A_o , was reduced according to the procedures specified in Ref. 9 whenever the cone overlapped another cone or the edge of the concrete.

The *ultimate shear capacity*, V_{uc} , of an individual stud[9] was taken as:

$$V_{uc} = \phi_c\, 6.66(10^{-3})\, A_s f'_c{}^{0.33}\, E_c{}^{0.44} \quad \text{(psi)} \qquad (3)$$

with the restriction that V_{uc} could not exceed the ultimate tensile capacity of the stud, $\phi_s A_s f_u$ computed from Eq. (2). The term E_c is the modulus of elasticity of the concrete.

Studs which are loaded with combined shear, V_u , and tension, P_u , had their ultimate loads checked by the interaction expression:[3]

$$\left(\frac{P_u}{P_{uc}}\right)^{5/3} + \left(\frac{V_u}{V_{uc}}\right)^{5/3} \leq 1.0 \qquad (4)$$

Table 3. Test Results, Group I

Specimen	Failure Mode	Eccentricity (in.)	Ultimate Shear, V_{test} (kips)	Ultimate Moment, eV_{test} (kip-in.)	Maximum Plate Slip (in.)	Maximum Deflection (in.)	Rigid Plate Method V_{test}/V_u	Plastic Method V_{test}/V_u
1	Stud	3	54.3	163	0.055	0.26	1.26	1.13
2	Concrete	6	35.5	213	0.020	0.14	1.17	0.98
3	Concrete	12	19.4	233	0.030	0.215	1.09	0.99
4	Stud	3	44.3	133	0.074	0.210	1.03	0.92
5	Stud	3	53.3	160	0.040	0.210	1.24	1.11
6	Stud	3	54.0	162	0.045	0.240	1.26	1.12
7	Stud	3	54.0	162	0.020	0.240	1.26	1.12
8	Stud	3	50.7	152	0.051	0.210	1.19	1.06
9	Concrete	4.5	43.9	198	0.050	0.092	1.22	1.04
10	Concrete	9	27.0	243	0.020	0.140	1.20	1.04
11	Concrete	6	35.0	210	0.029	0.130	1.18	0.98
12	Stud	3	52.6	158	0.048	0.160	1.26	1.10

Table 4. Test Results, Group II

Specimen	Failure Mode	Eccentricity (in.)	Ultimate Shear, V_{test} (kips)	Ultimate Moment, eV_{test} (kip-in.)	Maximum Plate Slip (in.)	Maximum Deflection (in.)	Rigid Plate Method V_{test}/V_u	Plastic Plate Method V_{test}/V_u
1	Stud	9	32.5	292	0.125	0.32	1.41	1.17
2	Stud	6	42.0	252	0.15	0.25	1.30	1.09
3	Stud	3	51.1	153	0.18	0.23	1.15	1.04
4	Stud	6	49.2	295	0.22	0.28	1.27	1.10
5	Stud	9	42.0	378	0.19	0.32	1.35	1.13
6	Stud	3	50.9	153	0.11	0.145	1.35	1.02
7	Stud	6	60.0	360	0.13	0.23	1.41	1.09
8	Stud	9	49.0	441	0.13	0.28	1.54	1.31
9	Stud	7	34.2	305	C.L.	C.L.	C.L.	C.L.
10	Concrete	9	61.0	549	—	0.23	1.42	1.25

Note: C.L. = Cyclic Loading

While Eqs. (1) through (4) provide a useful method for predicting the strength of an individual stud, they are not sufficient to determine the strength of the moment-resisting connection shown in Fig. 1. In a connection, the shear force must be distributed among the studs in some manner, and the moment will produce a tensile force distribution on the studs. The strength of the connection will be very dependent upon the distribution of these forces. There are no specific existing guidelines for predicting this distribution, but there are two basic methods that might logically be used. In this paper, those two methods are termed the *rigid plate* and *plastic distribution* methods.

The rigid plate method assumes rigid studs connected to a rigid end plate, and is somewhat analagous to methods used for the elastic design of bolt groups. Since the studs and plate are assumed to be rigid, the shear force is distributed uniformly among all studs. The bending moment induces a tensile force in the studs in accordance with the reinforced concrete beam model shown in Fig. 7. Failure of the section is assumed to occur when any one of the tensile studs reaches its full capacity under the combined loading as defined by Eq. (4). That procedure is utilized in a design example shown in Ref. 5.

The plastic distribution method recognizes that the connection is not rigid, and that considerable plastic and elastic redistribution of forces can occur. The tensile forces are computed from the applied moment, using the beam model shown in Fig. 7. Initially, the shear is equally distributed among the studs in the compression zone of the connection. If the tensile studs reach their full tensile capacity before the compression studs reach their full shear capacity, then failure is assumed to occur when the tensile studs reach their full capacity. However, if the compressive studs reach their full shear capacity first, then a plastic redistribution of the excess shear force is assumed. The excess shear force is distributed equally among the tensile studs until all tensile studs reach their full combined load capacity as defined by Eq. (4). The strength of each connection was computed, using both the rigid plate and plastic distribution methods and the measured strengths of the studs and the concrete. The measured strengths are compared to the predicted results in Tables 3 and 4. Calculations for the typical specimen 1 of Group I are shown in the Appendix to this paper.

The methods utilized here for predicting the strength are conservative. The ϕ factors incorporated into Eqs. (1), (2),

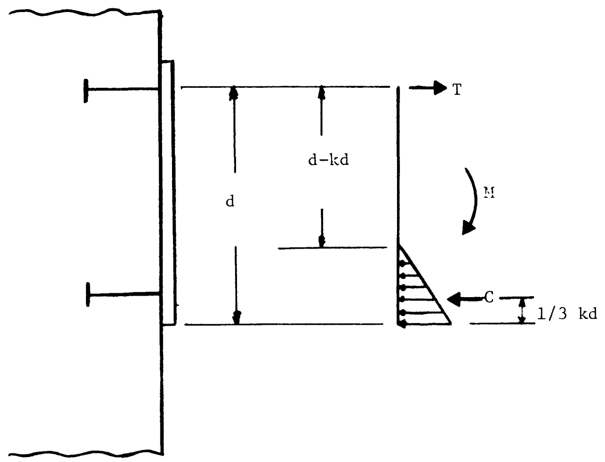

Fig. 7. Assumed stress distribution at the interface between steel plate and column

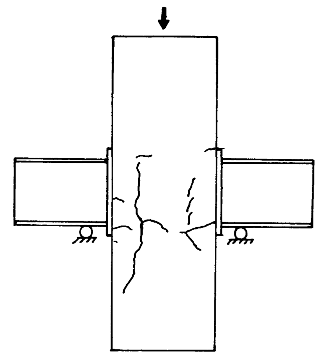

Fig. 8. Sketch of the concrete cracking of specimen 1 of Group I at failure

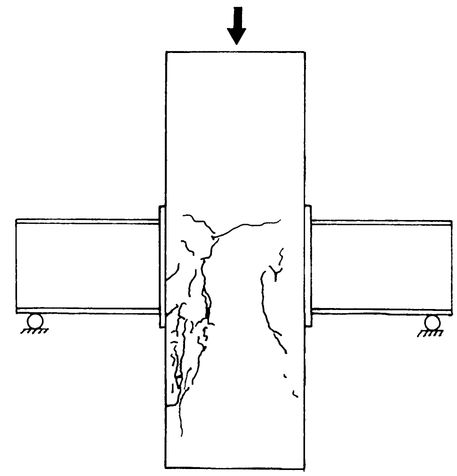

Fig. 9. Sketch of the concrete cracking of specimen 3 of Group I at failure

and (3) should result in lower bound predictions to the test data. The assumption that the concrete behaves elastically up to collapse, according to the diagram shown in Fig. 7, is likely to result in an underestimation of the internal lever arm for bending moments for some of the specimens.

TEST RESULTS—GROUP I

The results of the tests in Group I are summarized in Table 3. Specimens 1, 2, 3, 9, and 10 were identical, except that the eccentricity of the applied loading was varied from 3 in. to 12 in. Shown in Fig. 10 are the shear force-rotation curves for these tests where the rotation is taken as the deflection on the center line of the column divided by the eccentricity. That measure of rotation was used rather than the center line deflection, because the center line deflection is not a reliable measure of the ductility demand for a connection in which the steel beam represents the end of a main member of a steel frame. Geometry would require that in that case the center line deflection increase with the span length of the steel beam. When the specimen is considered to represent a corbel connection, then the center line deflection is a reliable measure of the ductility. The ultimate shear capacity of the connection, V_{test} , decreased as the eccentricity increased. The failure was a relatively ductile stud shearing failure for a 3-in. eccentricity. However, for 4.5-in. and higher eccentricities, a more brittle stud pull-out failure occurred. A dramatic loss in connection ductility with increasing eccentricities can be noted in Fig. 10. After the studs yielded in shear, there was an abrupt pull-out type failure for the 4.5-in. eccentricity specimen. However, for the larger eccentricity specimens, the pull-out type failure was less abrupt and there was a more gradual deterioration in capacity with increasing deformations. The relative vertical slip between the plate on the end of the beam and the adjacent concrete was measured during these tests, and much larger slips were noted for the low eccentricities. That result also indicates more ductile shear behavior for those specimens. Considerable cracking in the concrete was noted as apparent from the sketches of specimens 1 and 3 at failure in Figs. 8 and 9. The amount of cracking increased as the eccentricity of the loading increased. For all specimens with pull-out cone-type failures, collapse occurred shortly after vertical cracking developed within the column and immediately behind the head of the studs.

Specimens 11 and 12 were identical to specimens 2 and 1, respectively, except for a closer spacing of the ties in the vicinity of the tension studs. The additional ties did not cause any noticeable increase in the strength or ductility of the connection over that observed for specimens 1 and 2, and there was no change in the mode of failure. However, specimens 11 and 12 had lower ultimate strength values for the concrete than specimens 1 and 2.

Specimens 4, 5, and 6 had stud pull-out cones which were designed to intersect with the edge of the test specimen. Specimen 4 had a 2-in. edge distance. Its ultimate capacity was approximately 18% lower than that of specimens 1, 5, and 6, which had either 3-in. or 4-in. edge distances.

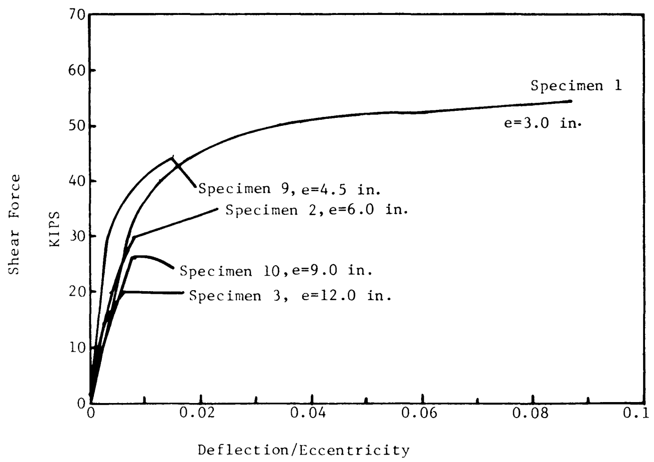

Fig. 10. Force-deflection curves for Group I specimens

However, its mode of failure, stud shearing, was the same as that for the other three specimens. Specimens 4, 5, and 6 also had larger horizontal spacings between the center lines of the studs than specimen 1. That change had no noticeable effect upon the strength or ductility of the connection. However, specimens 7 and 8 had a constant 4-in. edge distance, with a variable horizontal spacing between studs. Specimen 7 with the 6-in. stud spacing had approximately 6% lower capacity than specimen 6 with the 8-in. spacing. Similarly, specimen 8 with a 4-in. stud spacing had a capacity of 7% lower than specimen 7. Specimens 1 and 6 had similar capacities, despite the increase in the edge distance from 3 in. for specimen 1 to 4 in. for specimen 6 and a greater concrete strength for specimen 6 than for specimen 1. These comparisons indicate that the strength of the stud connection is influenced by overlap of the shear cone with the edge of the concrete. However, the overlap of one cone with another cone causes a smaller reduction in the strength of the connection.

The connection behavior was generally more ductile for small eccentricities. All specimens with an eccentricity greater than 3 in. failed in the concrete, although only specimen 11 was not found to be fully embedded according to Eq. (1) for a ϕ_c equal to 0.85. This result was interpreted as indicating that more ductile behavior could probably be obtained for higher eccentricities if larger embedment lengths were provided for the tensile studs.

The rigid plate method of analysis predicted strengths that were conservative estimates of the measured values. Ratios of measured to predicted strengths ranged from 1.03 to 1.26 and averaged 1.20. Ratios tended to increase with decreasing eccentricities. The plastic distribution method predicted strengths that agreed more closely with test results. Ratios of measured to predicted strengths ranged from 0.92 to 1.13 and averaged 1.05. Again, ratios tended to increase with decreasing eccentricities. For low eccentricities, the plastic distribution method predicted considerable redistribution of forces between studs due to shear yielding of the compression studs. For high eccentricities, the plastic distribution method predicted a brittle tension stud failure before the compression studs attained their full shear capacity. The test results and ductility observations agree with this prediction. At low eccentricities, large shear slip-deflections and ductility were obtained, but at high eccentricities the slip and ductility were substantially reduced.

Strain gages were mounted on the tensile stud of specimens 1 through 8. Those gages clearly indicated yielding of all studs except those for specimen 7. Apart from verifying whether or not yielding occurred, the strain gage measurements were not very useful, because several of the gages failed shortly after initiation of yielding in the studs and there were obviously considerable local bending strains in the studs.

TEST RESULTS—GROUP II

Because of the brittle stud pull-out failures noted in some specimens of Group I, the studs for Group II were embedded 2 in. deeper into the concrete than those of Group I. The results of the tests of Group II are summarized in Table 4. Specimens 1, 2, and 3 were of identical design, but they were tested at eccentricities of 9, 6 and 3 in. The shear capacity of the connection again decreased significantly with increasing eccentricity, but for these specimens with deeply embedded studs, failure at all eccentricities was ductile, and caused by fracture in the stud shank. Those three specimens were equivalent to specimens 1, 2, and 10 of Group I. The ductility and maximum strength were significantly greater for the specimens with 6-in. and 9-in. eccentricities in Group II than in Group I. The increase in strength was 18% and 20%, respectively, despite the concrete strength for the Group II specimens being considerably less than the concrete strength of the corresponding Group I specimens.

Specimens 4, 5, and 6 of Group II were similar to specimens 1, 2, and 3 of Group II, except that the vertical stud spacing was increased from 8 in. to 12 in. The shear capacity of the connection was increased 15% and 29% for the two larger eccentricities, but no change was observed for the 3-in. eccentricity. That result can be explained by noting that increasing the vertical stud spacing significantly increases the moment capacity of the connection, because the internal lever arm is increased and the resulting stud tensile force is reduced. However, the basic shear capacity of the connection is not changed. Lower eccentricities are dominated by shear effects, and higher eccentricities by moment effects. Thus, the capacity of the connection which is loaded with a large eccentricity increases with increasing vertical spacing for the studs.

Specimens 7 and 8 of Group II were identical to specimens 4 and 5 of Group I, except that two additional compression studs were added to the Group II specimens. This increased the shear capacity 22% and 17% for the 6-in. and

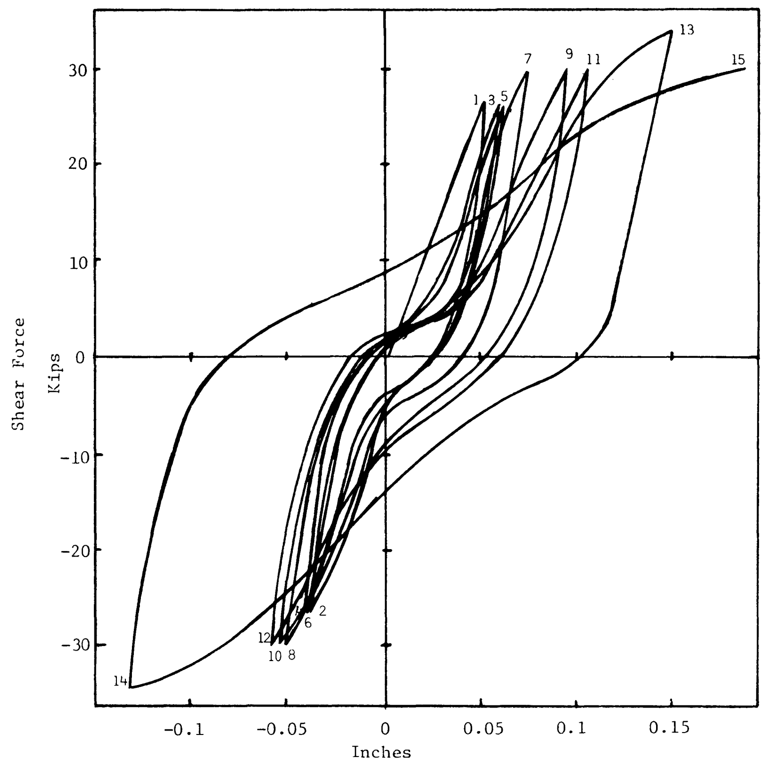

Fig. 11. Cyclic force-deflection relationship for specimen 9 Group II

9-in. eccentricities, respectively. This would not appear to be an efficient usage of studs, since the total number of studs was increased by 50%. The result was consistent with the other test results. Larger eccentricities produce large moments and tensile forces in the studs, but additional compressive studs have only a secondary effect upon the moment capacity of the connection. Specimen 10 had four compression studs, but it also had three studs in the tensile zone. It was tested at a 9-in. eccentricity and it increased the capacity of the connection 45% over that of specimen 5. This is consistent with other test results, since it verifies that the capacity of connections with large moments is best increased by adding tensile studs. Specimen 10 was the only specimen of Group II to fail by a cone pull-out failure. The additional tension stud caused considerable overlap of the potential pull-out cones in the tension zone.

Specimen 9 was of the same design as specimen 2 except that it was tested under cyclic loading. The cyclic loading produce a 19% reduction in the ultimate capacity of the connection. The eccentricity was 6 in., and the first three cycles were complete load reversals to 70% of the estimated capacity of the connection. This was followed by three additional cycles at 80% of the estimated capacity and one and one-half cycles at 90% of that capacity. A shear failure of the studs occurred during the final loading sequence. The cyclic loading resulted in the hysteretic behavior, shown in Fig. 11. The hysteretic curves are pinched, S-shaped loops, typical of systems that deteriorate under cyclic loading.

For the specimens of Group II, the rigid plate predictions were excessively conservative. Ratios of measured to predicted strengths ranged from 1.15 up to 1.54 and averaged 1.36. The plastic distribution method produced more realistic predictions of the connection strength. Further, the plastic method was always conservative for the Group II specimens, where the studs were embedded far enough into the concrete to prevent pull-out failures. Ratios of measured to predicted strengths ranged from 1.02 up to 1.31 and averaged 1.13. This result indicates that the plastic redistribution method is a suitable method for designing these composite connections, but that the fully embedded condition should be determined by a more conservative method than Eq. (1) with ϕ_c equal to 0.85. It is suggested that until further data are available, the stud lengths required for full concrete embedment in the presence of prying actions should be taken as 50% greater than those calculated from Eq. (1).

SUMMARY AND CONCLUSIONS

These 22 tests indicate that headed metal studs can be used to connect steel beams to concrete columns and shear walls. Such connections can transfer both shear force and moment, but their ability to resist moment is severely limited by the tensile capacity of the studs and their vertical spacing. Such connections are ductile when the shear force is high and the moment is low. High moments produce high tensile forces in some of the studs, and a relatively brittle failure may occur if the tension studs are not deeply embedded.

The plastic distribution method described in this paper provides the best method for predicting the ultimate design capacity of such connections. However, if the ratio of the design moment to shear on the connection exceeds half the vertical spacing of the studs, it is recommended that ϕ_c be taken as 0.6 or less in predicting the fully embedded condition with Eqs. (1) and (2). The likelihood of brittle failure is then reduced without making the design overly conservative.

Severe cyclic loading reduces the ultimate strength of such connections and produces a deteriorating stiffness. It is recommended that when such connections are subjected to severe cyclic loads, they be designed conservatively until studied further. These tests also indicate that the best method for increasing connection bending moments and shear forces, is to add tensile studs.

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NOMENCLATURE

- A_o = area of shear cone surface
- A_s = cross-sectional area of steel stud
- b = overall width of steel plate
- d = depth from extreme compression edge of plate to center line of tension studs
- d_e = distance from center line of outer studs to parallel face of column
- d_h = diameter of stud head
- e = eccentricity of loading on steel beam
- E_c = modulus of elasticity of concrete
- f'_c = concrete cylinder compressive strength
- f_u = tensile strength of stud steel
- k = factor dependent on concrete type (k = 1.0 for normal weight concrete)
- kd = depth of compression zone for assumed stress distribution at steel plate to column interface
- l_e = embedment depth for stud from face of concrete to outer face of stud head
- P_u = ultimate tensile capacity of stud for combined tensile and shear loading
- P_{uc} = ultimate tensile capacity of stud for tensile loading only
- s = horizontal center to center spacing of studs
- s_1 = vertical spacing center to center of outer compressive and tensile studs
- V_{test} = measured ultimate shear capacity
- V_u = ultimate shear capacity of stud for combined shear and tensile loading
- V_{uc} = ultimate shear capacity of stud for shear loading only
- ϕ_c = strength reduction factor for concrete
- ϕ_s = strength reduction factor for steel

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APPENDIX—EXAMPLE CALCULATIONS

The following example calculations were performed for specimen 1 of Group I. The concrete and steel for this specimen had ultimate strengths of 5.4 ksi and 80 ksi, respectively. The shear cones for these studs overlapped other shear cones and the edge of the concrete. Despite this overlap, the studs were found to be fully embedded. The ultimate tensile capacity of the steel, predicted by Eq. (2), was less than the tensile capacity of the concrete predicted by Eq. (1). Thus, the tensile capacity of the stud for tensile loading only, Eq. (2), is:

$$P_{uc} = 14.1 \text{ kips}$$

The modulus of elasticity of the concrete was computed by

$$E_c = 57000\sqrt{f'_c} = 4.19 \times 10^6 \text{ psi}$$

The shear capacity of the stud for shear loading only, determined from Eq. (3), was found to exceed the capacity for full embedment. Thus, the shear capacity was taken as that for full embedment:

$$V_{uc} = 14.1 \text{ kips}$$

The specimen was loaded with a 3-in. eccentricity, and thus the moment applied to the studs was $3V$, kip-in. The linear elastic assumption of composite beam behavior shown in Fig. 7 gives

$$k = \sqrt{\frac{2E_sA_s}{E_cbd} + \left(\frac{E_sA_s}{E_cbd}\right)^2} - \frac{E_sA_s}{E_cbd} = 0.239$$

and the tensile force in the studs, T,

$$T = \frac{M}{(d - kd/3)} = 0.362V.$$

Rigid Plate Method—The rigid plate method assumes that the shear force is uniformly distributed to all studs, and the tensile force is distributed to the tensile studs in accordance with Fig. 7. The tensile and shear capacity are equal, and so failure will occur in the tensile stud in accordance with the interaction curve.

$$\left(\frac{0.181\ V_u}{14.1}\right)^{5/3} + \left(\frac{0.25\ V_u}{14.1}\right)^{5/2} \leq 1.0$$

This equation predicts a strength of

$$V_u = 42.8 \text{ kips}$$

The ultimate capacity obtained in the tests, V_{test}, was 54.3 kips, and thus the ratio of actual strength to predicted strength is

$$\frac{V_{test}}{V_u} = 1.26$$

Plastic Redistribution Method—The plastic redistribution method assumes that the shear is initially uniformly distributed among the compression studs and the tensile forces are distributed to the tensile studs in accordance with Fig. 7. If one of the tensile studs attains its full capacity before the compressive studs reach their full shear capacity, the connection fails. If the compressive studs reach their full shear capacity first, plastic redistribution of the additional shear force will occur until the tensile studs reach their maximum capacity under combined shear and tension. Specimen 1 of Group I had two tensile studs and two compressive studs and an eccentricity of 3 in. Thus, the compressive studs will attain their full capacity (28.2 kips) first and the interaction curve for tensile stud failure is

$$\left(\frac{0.181\ V_u}{14.1}\right)^{5/3} + \left(\frac{0.5V_u - 14.1}{14.1}\right)^{5/3} \leq 1.0$$

This interaction curve can be solved by trial and error and

$$V_u = 48 \text{ kips}$$

Thus, the ratio of the actual strength to predicted strength is

$$\frac{V_{test}}{V_u} = 1.13$$