Strength of Joints that Combine Bolts and Welds

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Most connections use a single fastening device (bolts, rivets, welds) to make the connection between adjoining members. However, it is sometimes necessary, or desirable, to combine different fasteners in a single connection. This is particularly true when making alterations to existing structures, and the usual situation is the need to add one fastener type to a joint that was made using another type. For example, in order to increase the capacity of an existing riveted joint, some rivets can be replaced by high-strength bolts. In another situation, fillet welds can be added to an existing bolted joint, again so that an increased load can be carried. Usually, it is necessary to do such alterations while the member is already loaded.

Joints that combine two (or more) types of fastening elements can take different forms. For example, a beam-to-column connection that uses a beam web shear tab welded to the column face and bolted tee-stubs that connect the beam flanges to the column is a combination bolted-welded joint. Such arrangements can be handled by simply considering the forces that must be transferred at each location. It is not as apparent, however, how to determine the capacity of a joint that uses bolts and welds, for example, that act in the same shear plane. This would be the case if an existing bolted truss member splice were to be reinforced with fillet welds. The discussion presented here will be limited to the particular case where the connectors act in the same shear plane and in which the connection elements are specifically high-strength bolts and fillet welds.

Statement of Problem

The capacity of each connector type in a shear-splice type of joint is reflected by its shear strength and shear deformation characteristics. When two or more types of fasteners share the load, it is clear that it is not satisfactory to simply combine the ultimate strength of each individual fastener type. Each of the different fastening elements has a different ductility and, in general, each reaches its ultimate strength at a different value of overall connection deformation. The characteristics of a bolt, transverse fillet weld, and longitudinal fillet weld shown in Figure 1 are representative. (It should be noted that the relative strength of the transverse and longitudinal welds is a function of their respective lengths. Similarly, the relative position of the bolt curve depends on the size of the bolt.)

In order to determine the ultimate strength of a combination joint, the load vs. deformation characteristics of each fastener type first must be established. The way in which these individual elements interact with each other and how much resistance each contributes to the connection can then be determined.

REVIEW OF PREVIOUS WORK

Research by Holtz and Kulak

A series of tests involving combination joints was performed by Holtz and Kulak (1970). In the only relevant portion, double lap shear splices using bolts and welds in the same shear plane were loaded in axial tension. Each connection contained either one or two 20 mm diameter A325 high-strength bolts in combination with fillet welds of 6 mm nominal leg size (E410 electrode). Nine specimens were tested, and there were three identical splices in each of the following groups:

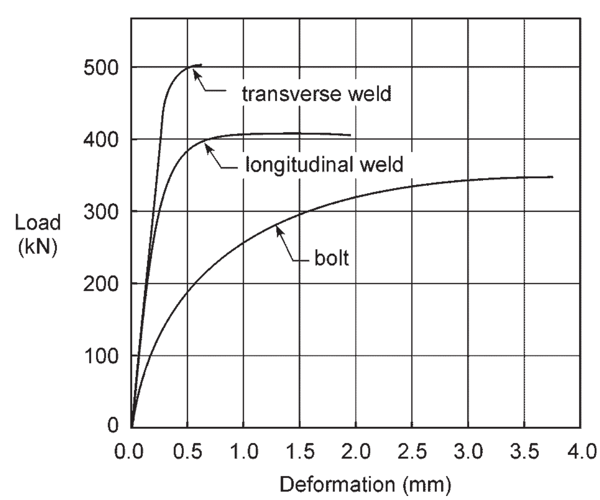

Fig. 1. Representative load vs. deformation characteristics.

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1. Longitudinal fillet welds combined with two high-strength bolts. No clearance was provided between the bolts and holes, i.e., so-called fitted bolts. Bolts were snug-tightened only.
2. Same as Case 1 except that clearance between bolts and holes was provided at the standard nominal value, 1.6 mm ($^1/_{16}$ in.), and the bolts were pretensioned.
3. Transverse fillet welds combined with one high-strength bolt. Clearance between bolt and hole was provided at the standard value, 1.6 mm. Bolts were pretensioned.

In the first series, the bolts were assumed to be in bearing from the beginning of the test (because of the so-called fitted condition), but in the other two series, the bearing condition at the start of the test was unknown.

Holtz and Kulak made predictions of the load vs. deformation behavior of their combination joints by using the Fisher model for component response (Fisher, 1965). The specific deformation vs. strength relationships they used for their bolts and welds were the same as those used by Crawford and Kulak (1971) and Butler, Pal, and Kulak (1972), respectively. (The Holtz and Kulak test bolts and the Crawford and Kulak bolts came from the same lot. The weld electrode specification was the same in both the Holtz and Kulak study and in the Butler et al. investigation.) Using these characteristics, the investigators calculated the resistance contributed by each fastening element for each notional increment of deformation of the combination joint. In the analysis, the deformation of the bolt was adjusted for those connections in which bolt hole clearance was present. By summing the calculated resistances of the individual fastening elements, a prediction of the ultimate resistance of the joint is produced.

For the first series, fitted bolts and longitudinal welds, the ratio of predicted ultimate load to test ultimate load was 1.21, and the standard deviation was 0.08. This poor, and non-conservative, result reflects the inadequacy of the assumption that the bolts contribute resistance right from the start of the test. For the tests in which bolts installed in holes of standard 1.6 mm clearance were combined with longitudinal welds, the ratio of predicted load-to-test load was 1.00, and the standard deviation was 0.02. In the last series, bolts in standard holes combined with transverse welds, the ratio was 0.93, and the standard deviation was 0.03. In these last two cases, the ratio of predicted ultimate load to test ultimate load is reasonable and is conservative.

Holtz and Kulak concluded that only a small amount of load was carried by the bolts at service loads (taken as about $^1/_3$ ultimate), that the effect of friction when pretensioned bolts are used cannot be predicted reliably, and that bolts do not work very well in combination with transverse welds because of the limited ductility of the latter.

Research by Jarosch and Bowman

Jarosch and Bowman (1986) performed a series of physical tests on combination joints similar to the tests performed by Holtz and Kulak (1970). All connections used standard hole clearance, and the bearing condition of the bolts at the start of the test was unknown. Six combination joints were tested, two in each of the following groups:

1. Two high-strength bolts in combination with longitudinal fillet welds.
2. Two high-strength bolts in combination with transverse fillet welds.
3. Two high-strength bolts in combination with both longitudinal and transverse fillet welds.

Jarosch and Bowman used values of fastener ultimate strength and ultimate deformation as obtained in calibration tests, but used the same regression coefficients for both bolts and welds as developed by Holtz and Kulak. Their predictions of test load were generally satisfactory. Combining all six specimens, the ratio of predicted ultimate load to the test load was 1.09, and the standard deviation was 0.07.

As had been suggested by Holtz and Kulak, Jarosch and Bowman also recommended that transverse welds should not be used in combination with high-strength bolts because their limited ductility does not permit the development of any significant shear load in the bolts.

Research by Manuel and Kulak

These researchers carried out a series of 24 tests of tension lap splices that had bolts and welds in the same shear plane (Manuel and Kulak, 2000). The parameters encompassed the following conditions:

1. Bolts pretensioned or snug-tight only.
2. Bolts combined with longitudinal fillet welds only, combined with transverse fillet welds only, or combined with both longitudinal and transverse fillet welds.
3. Position of the bolts relative to their holes established at the start of the test. Bolts were either in a location in which no slip was possible, "positive bearing," or in a location such that maximum slip could take place, "negative bearing."

There were two replicate tests in each category. For example, there were two tests of a specimen that contained four 20 mm ($^3/_4$-in.) diameter ASTM A325 bolts in positive bearing combined with 560 mm of longitudinal fillet weld of 6 mm nominal leg size deposited using E48018-1 filler metal.

In addition to these full-size specimen tests, ancillary tests were carried out to determine the load vs. deformation characteristics of the fastener components. Bolt shear tests used bolts from the same lot of bolts subsequently installed in the full-size pieces and plate taken from the same source as used in the full-size tests. Likewise, the plate used to make up the weld coupon test specimens was taken from the same stock of plate used to fabricate the full-size specimens. The weld electrodes used in the coupon tests came from the same stock as used subsequently to fabricate the full-size specimens.

Manuel and Kulak reported the following conclusions:

1. When transverse fillet welds are used in combination with high-strength bolts, the deformation at the time of weld fracture is such that almost no bolt shear strength is developed. (This was also observed by Holtz and Kulak and by Jarosch and Bowman.)

2. The term "slip-critical," used in the design of bolted joints, has no meaning when high-strength bolts are combined with fillet welds. When welds are present, there can be no slip that places the bolts uniquely into bearing. Once the welds fracture, the situation simply reverts to that of a bolted joint with no welds.

3. If the bolts in a combined bolted-welded joint are known to be in a condition of negative bearing, the ultimate strain of even longitudinal fillet welds will not be sufficient to mobilize any significant bolt shear strength.

Manuel and Kulak modeled their physical test results using the load vs. deformation characteristics of the individual components of the joints and by making an estimate of the frictional resistance present. Regression analysis of the individual bolt shear tests and weld shear tests was used to develop the necessary relationships (Fisher, 1965). Once these expressions were obtained, the predictions for the physical tests were based on [a]:

$$R_{ult\ joint} = R_{friction} + R_{bolts} + R_{trans} + R_{long} \qquad (1)$$

where

$R_{ult\ joint}$	=	ultimate strength of combination joint, kN
$R_{friction}$	=	plate friction resistance, kN
R_{bolts}	=	bolt shear resistance, kN
R_{trans}	=	transverse weld shear resistance, kN
R_{long}	=	longitudinal weld shear resistance, kN

At any particular value of the joint deformation, each fastening element potentially will contribute to the load resistance, and the least ductile fastener component in a connection will govern the deformation at which the initial fracture occurs. In the case of the bolts, the condition of negative bearing or positive bearing was taken into account. As already noted, the contribution of bolts in negative bearing was taken as zero and the friction contribution when snug-tightened bolts were used also was neglected.

Using this approach, the following results were obtained by Manuel and Kulak:

1. Longitudinal welds + bolts (various conditions of bolt bearing and bolt pretension); predicted load/actual test load = 0.98 (8 cases)

2. Transverse welds + bolts (various conditions of bolt bearing and bolt pretension);predicted load/actual test load = 1.02 (6 cases)

3. Both longitudinal and transverse weld + bolts (various conditions of bolt bearing and bolt pretension);predicted load/actual test load = 1.09 (6 cases)

(Certain tests in the Manuel and Kulak study had instrumentation problems and were rejected for purposes of this summary analysis.)

The results of the analysis showed that, overall, the ultimate strength of a combined bolted-welded joint can be calculated with a high degree of accuracy when the characteristics of the individual components are known. However, it is unlikely that a designer will have the load vs. deformation fastener characteristics that are necessary to make the ultimate load prediction. Recognizing this, Manuel and Kulak used their results to provide simplified rules. (Because their tests included specimens with transverse welds, this case is included in the Manuel and Kulak model. However, they recommended that transverse fillet welds not be used in conjunction with bolts, i.e., the strength of such joints would be taken as the larger of the bolt strength or transverse weld strength alone.)

Frictional Forces

Friction between the plates at the time of joint ultimate load is quite variable, as would be expected. However, according to the Manuel and Kulak tests, a reasonable lower bound is

$$R_{friction} = 0.25 \times P_{slip} \qquad (2)$$

where

P_{slip} = the slip resistance of a bolted joint, kN

(It can be noted that frictional force will be present as the ultimate capacity of the combined bolted-welded joint is reached because some bolt pretension still exists at this stage. It is only later, at the time of bolt ultimate shear, that the bolt pretension has decreased to a negligible value.)

Transverse Weld Shear

When transverse welds are used in a combination joint, the shear resistance of the joint is the largest of the transverse weld shear strength or the bolt shear strength. For a given

[a] Although SI units are used in this paper, the concepts and conclusions are independent of the system of units used.

weld length, l, and given weld leg size, w, the contribution of the transverse weld component is taken as

$$R_{trans} = R_{ult\ trans} \times l \times w \tag{3}$$

where

$R_{ult\ trans}$ = ultimate unit shear resistance of the transverse weld, kN/mm/mm

Longitudinal Weld Shear

If only longitudinal welds are used in a combined bolted-welded joint, then resistance of the longitudinal welds is simply their ultimate shear strength. If transverse welds are also present in the connection, then the contribution of the longitudinal welds must be established at the deformation corresponding to the ultimate deformation of the transverse welds. The test data show that longitudinal welds contribute about 87 percent of their ultimate shear capacity when used in combination with transverse welds and high-strength bolts. For simplicity, this is rounded to 85 percent.

The equations for connection resistance contributed by longitudinal weld shear for a given weld length, l, and a given weld leg size, w, then are as follows:

- For combination joints with *only* longitudinal welds

$$R_{long} = R_{ult\ long} \times l \times w \tag{4}$$

- For combination joints with *both* longitudinal and transverse welds, the strength of the longitudinal weld is taken as

$$R_{long} = 0.85 \times R_{ult\ long} \times l \times w \tag{5}$$

Bolt Shear

The contribution from bolt shear will depend upon whether transverse welds are present and upon the bearing condition of the bolts at the start of loading. However, when bolts and transverse welds are combined, the Manuel and Kulak test program showed that the bolt shear contribution at the time of transverse weld fracture is negligible.

Connection resistance from bolt shear is available when the bolts are used in combination with longitudinal welds. In order to evaluate this resistance, the amount the joint must deform to put the bolts into bearing must first be evaluated. When the condition of positive bearing was explicitly introduced in the test specimens, it was found that the bolts contribute about 79 percent of their ultimate shear strength. This will be rounded down to 75 percent. In practice, however, it is unlikely that the bolts will be in such a favorable bearing condition. A reasonable assumption, which is based on both field and laboratory experience (Kulak, Fisher, and Struik, 1987) is that about one-half a hole clearance of slip is likely, i.e., 0.8 mm. Using this assumption, Manuel and Kulak estimated that the bolts contribute about 49 percent (rounded to 50 percent) of their ultimate shear strength.

The equations for connection resistance contributed by bolt shear for n bolts are then as follows:

- For combination joints consisting of bolts and transverse welds (with or without longitudinal welds)

$$R_{bolts} = 0 \tag{6}$$

It should be noted that, once the transverse weld has fractured in a joint that combines bolts and transverse welds, the joint is no longer a combination joint and the strength of the joint is now limited to that of the bolts. Thus, the strength of a joint that combines bolts and transverse weld is effectively the maximum of the transverse weld strength or the strength of the bolts.

- For combination joints with only longitudinal welds and with bolts known to be in a positive bearing condition

$$R_{bolts} = 0.75 \times n \times R_{ult\,bolt} \tag{7}$$

- For combination joints with only longitudinal welds and bolts in an indeterminate bearing condition (taken as field conditions)

$$R_{bolts} = 0.50 \times n \times R_{ult\,bolt} \tag{8}$$

In order to determine the shear resistance of the entire joint, the values of $R_{friction}$, R_{long}, R_{trans}, and R_{bolts} are substituted into Equation 1.

ULTIMATE STRENGTH CRITERIA

The design recommendations presented here are intended for prediction of the ultimate resistance of a connection. When examining the results of the full-scale tests, only the load resistance at first weld failure was considered. However, as already noted, the ultimate load does not necessarily correspond to the load at first weld failure. Since the objective is to predict the ultimate resistance, each of the different failure mechanisms must be considered. The resistance of the connection should be taken as the largest of these failure mechanism loads.

The need to examine individual components that make up the combined joint can be illustrated with an example. Consider a bolted joint, bolts non-pretensioned and assumed to be in the intermediate bearing condition, to which only a very small amount of longitudinal fillet weld is added. The combination joint will have a certain capacity calculated according to Equation 1. The weld fractures at a deformation that utilizes only 50 percent of the bolt shear capacity (Equation 8). However, after the weld has reached its ultimate deformation, the joint (now consisting of only the bolts) can continue to carry load up to 100 percent of the bolt shear capacity. The designer should then realize that adding only a small amount of fillet weld does nothing to increase the ultimate load of the joint and a revision of the bolt/weld proportions is necessary. Other illustrations of the need to consider the individual component strengths can be found in the section Illustrative Examples.

For the 20 Manuel and Kulak tests, the mean value of the ratio, *predicted ultimate load/test ultimate load* is 1.03, and the standard deviation is 0.06. The same approach was used to predict the results obtained by the other researchers. For the nine Holtz and Kulak (1970) tests, this ratio was 1.01, and the standard deviation was 0.05. For the six Jarosch and Bowman (1986) tests, the ratio was 0.98, and the standard deviation was 0.06.

NEW TESTS AT THE UNIVERSITY OF ALBERTA

Following the completion of the Manuel and Kulak research, it was realized that it would be desirable to conduct tests on combination welded-bolted joints that are in the "as-built" condition. Specifically, this means that the location of the bolts relative to the holes that contain them would not be controlled. This work has been reported by Sato (2000).

Nineteen full-size specimens as shown in Figure 2 were tested (Sato, 2000). All specimens were nominally the same, except for the bearing condition of the bolts, which was random. Because the transverse weld case had been shown by Manuel and Kulak (and others) to be inappropriate in a combination bolted-transverse welded joint, that weld configuration was not included in the program. The case of bolts combined with fillet welds that were both longitudinal and transverse also was not included. Steel grade of the connected material was not a variable in the tests nor was plate thickness. (These were not variables in any of the earlier tests either). It is considered that these factors will not affect the ultimate load results as long as (1) the steel grades used are compatible with the weld electrodes employed and (2) the plate thicknesses are reflective of thicknesses that would normally be employed with these fasteners in usual structural fabrication practice.

The fasteners used in the test joint shown in Figure 2 consisted of four 19 mm A325 bolts, placed in 20.6 mm holes, with 6 mm longitudinal welds deposited using E48018-1 welding electrode. The plate material used for the test specimens was ASTM A572 Grade 50 steel. The bolts in the test joints were pretensioned using the turn-of-nut method. In order to determine the pretension force, the turn-of-nut method was calibrated using bolts from the same lot as those used in the test joints. The mean pretension measured in this way was 183 kN per bolt. The longitudinal fillet welds were deposited after the bolts had been pretensioned. Table 1 presents a brief description of the weld sizes in the test specimens and the measured capacity. Separate tests on bolts in double shear and on longitudinal welds were performed to obtain the shear resistance of a bolt and the shear resistance of the welds. The static shear resistance of a bolt from this test program, taken as the average of three tests, is 326 kN. The static shear resistance of the welds was determined as 329 N/mm/mm. All tests were conducted under stroke control and quasi-static conditions, thereby allowing the load to stabilize at each displacement increment.

The Manuel and Kulak model that was recommended for bolts in the "indeterminate" bearing condition, i.e., Equation 8, was used for the bolt shear contribution to the total capacity. Because pretensioned bolts were used in this study, a friction component is added in accordance with

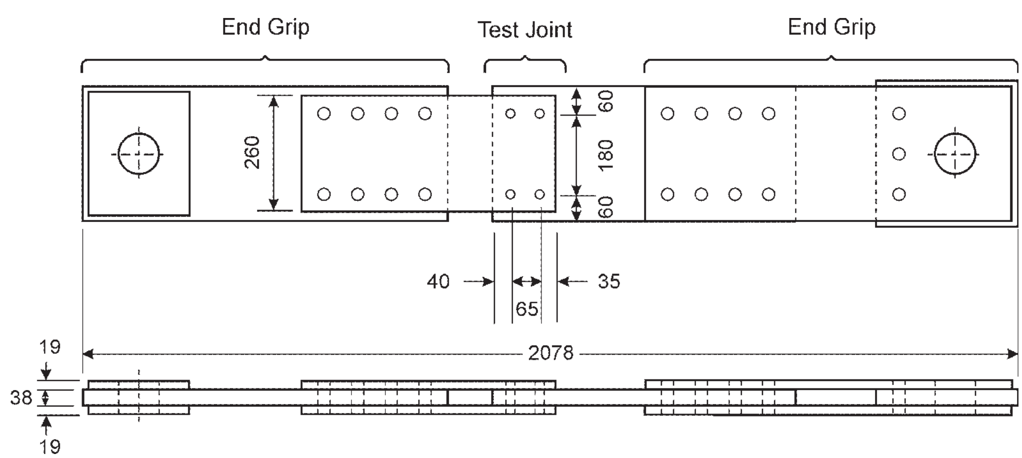

Fig. 2. Typical test specimen from Sato (2000).

Table 1. Test Results from Sato (2000)

Test Specimen	Test Capacity (kN)	Weld Geometry Size (mm)	Weld Geometry Length (mm)	Predicted Capacity Eq. (2) + Eq. (4) + Eq. (8) (kN)	Predicted/Test
1	1905	5.3	569	1765	0.93
2	1975	5.5	586	1833	0.93
3	1897	5.5	572	1808	0.95
4	1878	6.0	573	1904	1.01
5	1806	6.1	575	1927	1.07
6	1818	5.9	575	1889	1.04
7	1891	5.8	565	1851	0.98
8	1860	6.0	555	1868	1.00
9	1864	5.9	571	1881	1.01
10	1987	5.7	566	1834	0.92
11	1820	5.7	582	1864	1.02
12	1919	5.8	581	1881	0.98
13	1817	5.8	590	1899	1.04
14	1838	6.6	566	2002	1.09
15	1735	5.9	568	1875	1.08
16	1766	6.3	572	1958	1.11
17	1807	6.2	575	1946	1.08
18	1746	6.0	575	1908	1.09
19	1712	6.1	575	1927	1.13

Unit weld strength = 329 N/mm/mm; unit bolt shear resistance = 326 kN/bolt (double shear capacity).
Bolt pretension = 183 kN/bolt

Equation 2. The faying surfaces were clean mill scale, and a coefficient of friction equal to 0.33 was assumed for the calculation of the slip resistance. The capacity of the welds was predicted using Equation 4. The predicted capacity for each of the 19 test specimens is presented in Table 1.

The mean predicted-to-test ratio for the 19 Sato tests is 1.02 and the standard deviation is 0.06. For these tests, it is clear that the simplified model proposed by Manuel and Kulak gives excellent correlation between test ultimate load and predicted ultimate load.

REVIEW OF LRFD DESIGN RULES

Combination Joints with Bearing-Type High-Strength Bolts

The *Load and Resistance Factor Design Specification for Structural Steel Buildings* (LRFD Specification) (AISC, 1999) stipulates that when bearing-type high-strength bolts and welds are used together in new work, the welds must be proportioned to carry the entire force in the connection (Section J1.9). Thus, the presence of even a small amount of weld added to an existing bolted joint means that the weld capacity controls the design and the bolts are deemed ineffective. A rationale is provided in the Commentary to this clause, where it is stated that slip will take place prior to ultimate load and "the weld will carry an indeterminately larger share of the load."

The ultimate load of a combined bolted-welded joint (e.g., longitudinal welds) will be that which exists at the time that fracture of the welds takes place, assuming that one of the component strengths alone does not govern. According to the Manuel and Kulak study, the load in the bolts at this time will be about 50 percent of their ultimate shear strength (Equation 8). The position taken in the Commentary that slip will take place in the combination bolted-welded joint prior to ultimate load is not correct: slip of the bolted portion of the combined joint physically cannot take place prior to fracture of the welds[b]. The LRFD rule for this case is not rational, and the experimental results show that both the bolts and the weld carry load in this condition.

With respect to existing structures, the Commentary states that it can be assumed that any slip that might take place in an existing bolted joint has already happened prior to the addition of welds. The advice then is to use welds to resist all forces other than the dead load that exists at the time of the alteration. The authors consider that this advice does not reflect the real condition of a bolted-welded joint. If some slip has taken place, which is a reasonable supposition, then the bolts will provide shear earlier in the loading process than they otherwise would. In other words, slip that exists in the bolted joint before the welds are added is a favorable condition insofar as the ultimate strength of the combination joint. This is further illustrated in the following section.

Combination Joints with Pretensioned High-Strength Bolts

The LRFD Specification allows the load in a slip-critical connection to be shared between pretensioned high-strength bolts and welds, taken at the factored load level[c]. This applies to new work. No advice is given as to how to establish the load carried by the welds, but the reasonable presumption is that it is intended that the factored resistance of the weld and the slip resistance (taken at the factored load level) be additive.

The advice given in the Commentary on this type of combination says that the welds should be placed after the bolts have been pretensioned because, "If the weld is placed first, angular distortion from the heat of the weld might prevent the faying action required for development of the slip-critical force." This is not correct; the slip resistance of the bolted joint is independent of the amount of area between the faying surfaces. As long as there is some area, which is a physical necessity for proper pretensioning of the bolts (RCSC, 2000), then the slip resistance will be developed.

The research has shown that the ultimate load of a combination bolted-welded joint that uses pretensioned bolts reflects the following components: shear in the bolts (Equation 8), shear in the welds (Equation 4), and a portion of the theoretical slip load (Equation 2). It is not the sum of the slip load and the weld ultimate resistance, as stated in the LRFD Specification.

With respect to existing structures where the bolts were pretensioned, the LRFD Specification says that the bolts should be assumed to carry the loads present at the time of the alteration and the added welds proportioned to carry the additional loads.

Combination Joints—Other Cases

The research did not specifically treat cases in which load existed on the bolted connection at the time that welds were added. However, the information obtained from these tests means that various other cases can be addressed. Consider the following conditions.

1. *Welds added to bearing-type bolted joint, bolts assumed to be already in bearing.* A conservative assumption is that the additional loads are shared by the bolts and the welds as a combination joint in accordance with Equation 1. In fact, the portion of the additional load carried by the bolts will be larger than that indicated by Equation 1 because the deformations imposed on the bolt no longer start from zero. Figure 3a shows a hypothetical case when the loads start from zero, i.e., new work. The ultimate load of the combined bolted-welded joint is reached at a joint deformation of 10 units. This limit corresponds to the deformation that causes fracture of the weld. At this time, 10 units of joint deformation, there is a shear load in the bolts as indicated, and the total ultimate load that can be carried is the sum of the weld ultimate load and the bolt load corresponding to 10 units of

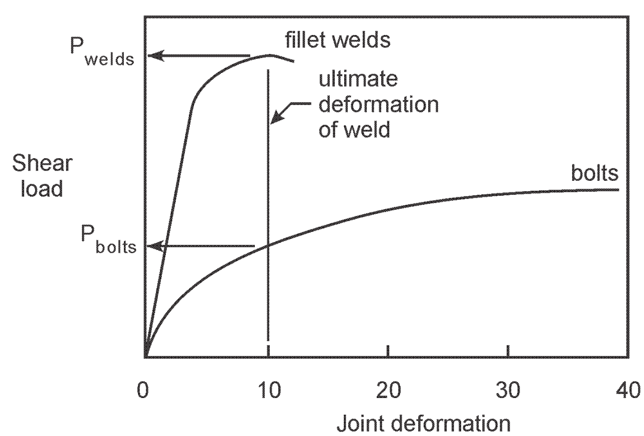

Fig. 3a. Superposition of bolt and weld characteristics—no initial load.

[b] Deformation of the component parts-connected material, bolts, and welds-necessarily must take place, but this is not the same thing as slip into bearing.

[c] The LRFD rules provide expressions for slip resistance starting either at the specified load level or at the factored load level. Although the equations are adjusted such that the result is more-or-less the same, the LRFD rule on combined bolted-welded joints says that the starting point is to be taken as the factored load.

joint deformation. Figure 3b illustrates what happens when there is an initial load on the joint and it is carried only by the bolts. Suppose this load causes 4 units of bolt shear deformation. The figure illustrates the condition for the situation when additional load is applied to the joint. The joint deformation now must reach 14 units in order that the weld (the critical element) can reach its component failure deformation of 10 units. The total load carried by the combined bolted-welded joint is larger than that predicted by Equation 1 because the bolt shear load has increased.

2. *Welds added to a slip-critical bolted joint in which it is assumed that no slip has occurred.* Addition of welds means that slip cannot take place and the condition is that of a combination bolted-welded joint. The weld must be proportioned so that it has a capacity at least equal to that corresponding to the slip load of the original configuration. As a result, the combined bolted-welded joint is the same as those reported herein and Equation 1 can be applied.

ILLUSTRATIVE EXAMPLES

Example 1

A truss has been designed using snug-tightened high-strength bolts. After completion of the design, an error was discovered—a joint was identified that will not carry its intended factored load of 900 kN. This connection, which consists of four 20 mm dia. A325 bolts, double shear, no threads in the shear plane, has a factored resistance of 780 kN. Can transverse fillet welds of 6 mm leg size, E480XX electrodes, be added in order to obtain the desired capacity?

Using the LRFD Specification, when adding weld to new work in a bearing-type connection it is required that the weld transfer the total factored load of 900 kN. For a weld of this leg size and electrode grade, a length of 982 mm is required[d].

According to Equation 1, the actual capacity of a joint that consists of four 20 mm dia. A325 bolts in double shear plus 982 mm of 6 mm E480 transverse fillet weld is limited to the factored resistance of the welds, i.e., 900 kN. (Because transverse welds are used in combination with the bolts, there is no shear contribution from the bolts when considering the bolt-weld combination. Because the bolts are only snug-tightened, there is no plate friction component for this case when Equation 1 is used.)

In this example, the same result is obtained whether using the LRFD Specification or the prediction obtained using the concepts presented in this paper.

Example 2

Repeat Example 1, except that the fillet welds will be placed longitudinally with respect to the direction of the force.

According to the LRFD Specification, nothing has changed. The added welds must transfer the entire force of 900 kN and the length of fillet weld required is still 982 mm.

The strength of the combined bolted-welded joint can be calculated according to Equation 1. (The appropriate resistance factor must be included in the calculations for the relevant component contributions). The applicable components of Equation 1 are plate friction, bolt shear, and longitudinal weld shear capacity. The plate friction component is zero for this case of snug-tightened bolts. The bolt contribution is (using Equation 8) equal to (0.50 × 780 kN) = 390 kN. (The value 780 already contains the resistance factor.) The factored resistance of the 982 mm longitudinal fillet weld was identified in Example 1 as 900 kN. Thus, the actual factored capacity of this bolted-welded joint is 390 + 900 = 1290 kN.

Since the actual factored capacity of the joint (1290 kN) is well in excess of the force that must be transferred (900 kN), the designer could adjust the length of fillet weld so as

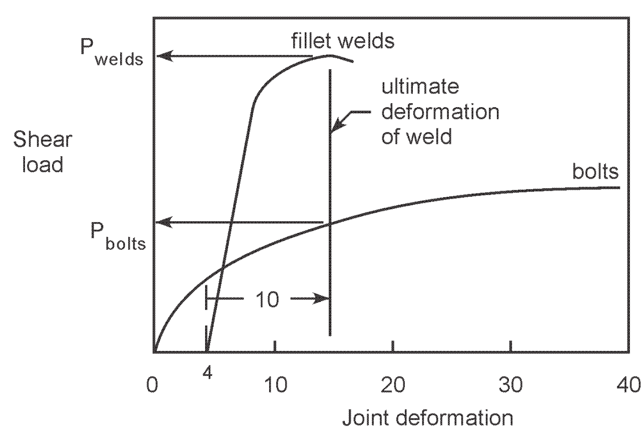

Fig. 3b. Superposition of bolt and weld characteristics—initial load present.

[d] If the designer wishes to take advantage of LRFD Specification Appendix J2.4, this length can be reduced because the weld is transverse to the direction of the force to be transferred. In any case, the weld length calculated might present a practical problem: there may not be enough room in the connection layout to place this much weld. A larger fillet size could be explored.

Table 2. Strength Calculations for Example 3

Item	Failure mode	Strength Equation	Calculated Strength (kN)
1	Bolt shear	$R_{bolts} = R_{ult\ bolts}$	780
2	Long. weld	$R_{long} = R_{ult\ long} \times l \times w$	625
3	Trans. weld	$R_{trans} = R_{ult\ trans} \times l \times w$	275
4	Welds	$R_{ult\ joint} = R_{ult\ trans} + 0.85\ R_{ult\ long}$	806
5	Long. weld + bolt shear	$R_{ult\ joint} = 0.5\ R_{ult\ bolts} + R_{ult\ long}$	1015

to reduce the capacity. (If the weld length is reduced to 557 mm, the actual factored capacity will be exactly 900 kN.) It is also noted that the calculated capacity of the combined bolted-welded joint is larger than the capacity of either of the components that make up the joint and 1290 kN is therefore the governing capacity.

Example 3

Repeat Example 2 except that the fillet welds will be placed as follows: 300 mm length will be in the transverse direction and the remaining weld length will be in the longitudinal direction.

Again according to the LRFD Specification nothing has changed. The welds must transfer the entire force of 900 kN, and the length of fillet weld required is still 982 mm. (Thus, the amount of longitudinal fillet weld is 982 − 300 = 682 mm.)

The calculations for this bolted-welded joint are summarized in Table 2. Items 1 to 3 tabulate the ultimate strength of the individual elements that make up the joint. As identified in the previous examples, the plate friction component is again zero.

The possible failure scenarios that must be investigated for this joint, in addition to the resistance of the individual components, are described as Items 4 and 5.

Item 4 is the strength reached at the time of rupture of the transverse weld, at which time 85 percent of the strength of the longitudinal welds is mobilized. Since the bolt shear contribution at the time the transverse weld ruptures is assumed to be zero, the governing equation for this case includes only the strength of the longitudinal and transverse fillet welds.

After the transverse weld has fractured, the joint consists of the longitudinal welds and the bolts. The strength of this combination is given as Item 5. For this case, the strength equation consists of the ultimate strength of the longitudinal welds plus 50 percent of the ultimate strength of the bolts.

The largest capacity of any of the cases described, either the individual component strengths or the combined bolted-welded strength cases, is the controlling resistance. Hence, the strength of this joint is governed by Item 5, i.e., 1015 kN.

In this example, the transverse weld does not contribute to the capacity of this bolted-welded joint and is simply redundant. It is worth noting that tabulation of combined bolted-welded solutions is advantageous because it helps to ensure that all possibilities have been examined.

Example 4

A certain slip-critical connection consists of four pretensioned 20 mm dia. A325 bolts, two shear planes, no threads in shear plane, in standard holes. The factored load to be carried is 550 kN. After completion of the structure, a check was made of the slip resistance. The check is made at the factored load level according to the requirements of LRFD J3.8a, i.e., $r_{str} = 1.13\ \mu\ T_b\ N_s$. For a slip coefficient $\mu = 0.33$, the value of the force that can be transmitted in friction by these bolts is calculated as 424 kN. Since this is less than the force that must be carried, 550 kN, the joint is deficient under the LRFD rules. Add 6 mm longitudinal fillet welds, using E480 electrodes, to strengthen the connection.

In accordance with the LRFD rules (Sect. J1.9), the bolts are assumed to carry the loads present at the time of the addition of the welds and the welds need to be proportioned to carry the additional load. After consideration of the situation, the designer has determined that the load at the time of the upgrade is 350 kN. Thus, the welds must carry 550-350 = 200 kN. A calculation will show that 218 mm of 6 mm fillet, E480 electrodes, is required. Thus, the combination bolted-welded joint consists of the four pretensioned A325 bolts and 218 mm of 6 mm fillet weld.

The capacity of the combination joint according to Equation 1 is:

$$R_{friction} = 0.25 \times slip\ \ load = 0.25 \times 424\ \text{ kN} = 106\ \text{ kN}$$

$$R_{ult\ long} = 0.916\ \text{ kN}/\text{mm} \times 218\ \text{ mm} = 200\ \text{ kN}$$

$$R_{bolts} = 0.50 \times 780\ \text{ kN} = 390\ \text{ kN}$$

or, a total factored resistance of 106 + 200 + 390 = 696 kN

In this example, the predicted factored resistance is considerably larger than that demanded, and an adjustment to the weld length could be made.

SUMMARY AND CONCLUSIONS

The strength of joints that combine bolts and fillet welds that act in the same shear plane has been examined. The study has shown that the rules provided in the AISC LRFD Specification do not adequately reflect the actual strength of this type of joint. Using test results from the University of Alberta and others, expressions are presented for the capacity of the various elements that make up a bolted-welded joint. These expressions enable the capacity of a combined bolted-welded joint to be predicted with a reasonable degree of accuracy.

The examination showed that transverse fillet welds do not work well in combination with bolts because of the limited ductility of the former.

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