Design of Connectors in Web-Flange Beam or Girder Splices

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INTRODUCTION

Splices in beams and girders are often required when the lengths of members are limited by fabrication, transportation, or handling facilities available, or by the construction process. A commonly used bolted splice is shown in Fig. 1. Splice plates are lapped across the joint and bolted to the webs and the flanges of the beam in order to transfer the load. This type of splice is usually referred to as a web-flange splice.

Current design methods for the connectors in web-flange splices vary. For example, Fisher and Struik[1] recommend that the web splice be assumed to transfer all of the shear and that the flange splice be assumed to transfer all of the moment at the section. The bolt group on one side of the web splice is designed on the assumption that the shear force acts at the centroid of the bolt group on the opposite side of the splice. In a commonly used British design manual,[2] the same approach is recommended. Ballio and Mazzolani[3] present two alternative approaches for design of web-flange splices. In both approaches, the moment at the location of the splice is proportioned between the web splice and the flange splice. For the web splice, the first approach considers the shear force to act at the centroid of the opposite bolt group. This is similar to the recommendation of Fisher and Struik. In the second approach, the bolt group on one side of the web splice is designed assuming that the shear force acts at the centerline of the splice. Bresler, Lin and Scalzi,[4] Salmon and Johnson,[5] and Nethercot[6] also use this second approach, and they further recommend that the web splice be designed to transmit both the eccentric shear force and the portion of the moment that the web was designed to carry. However, Salmon and Johnson suggest that the effect of the eccentricity can be neglected except in cases where the shear and moment are high, and Bresler, et al. recommend neglecting the effect of the eccentricity when the eccentricity is much less than the depth of the web.

The validity of these web-flange splice design assumptions has not been substantiated on an analytical basis. Furthermore, very little experimental work has been carried out to verify any of the design approaches. As far as can be established, the work of Garrelts and Madsen in 1941 is the only experimental study that has been carried out to investigate the behaviour of riveted or bolted web-flange girder splices.[7] However, their data do not verify the exact distribution of the force in a web splice.

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The current AISC specifications[8,9] require that bolted beam or girder splices resist the most unfavorable combination of shear and moment at the location of the splice. However, they do not provide insight into how the eccentric effect of the shear force should be accounted for in the design of a web splice or how the moment at the section should be proportioned between the web splice and the flange splices.

This paper is a summary of the results of a study carried out to establish a rational design procedure for the connectors in a bolted web-flange beam or girder splice in which both the web and the flange material are spliced at the same location.

ANALYTICAL STUDY

A theoretical approach to predict the ultimate capacity of a web-flange beam or girder splice has recently been proposed by Kulak, et al.[10] It is a development similar to the method currently used to determine the ultimate strength of eccentrically loaded bolted connections,[8,9] that is, it is a rational approach that satisfies the equations of static equilibrium and uses the true shear load versus shear deformation response of the fasteners.

Figure 2(a) shows a simple beam that contains a bolted web-flange splice located in a region where both shear and moment are present. A free-body diagram taken by cutting the beam through one set of fasteners is shown in Fig. 2(b). The forces in these fasteners are assumed to rotate about an

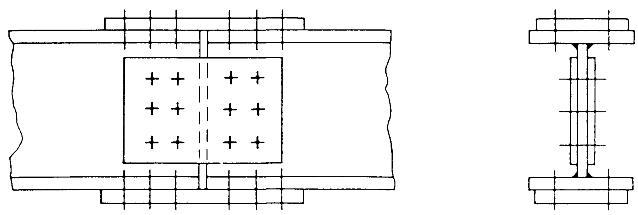


Fig. 1. Bolted web-flange girder splice.

instantaneous center, as shown in this figure. The location of the instantaneous center of rotation is identified[10] when the three equations of equilibrium are satisfied, namely:

$$\Sigma F_x = 0 \qquad (1)$$

$$\Sigma F_y = 0 \qquad (2)$$

$$\Sigma M = 0 \qquad (3)$$

Equation 1 is automatically satisfied because there are no external horizontal forces present. Equation 2 is satisfied when the sum of the vertical components of the bolt forces is equal to the shear acting at the section. For the member shown in Fig. 2, the result of taking the sum of the vertical forces to equal zero is:

$$\frac{Pb}{L} - \sum_{i=1}^{n} R_{iv} = 0 \qquad (4)$$

The result of taking the sum of the moments about the instantaneous center to equal zero is:

$$\frac{Pb}{L}(x + x_o + r_o) - F_f d - \sum_{i=1}^{n} R_i r_i = 0 \qquad (5)$$

Equation 5 can be rewritten as:

$$\frac{Pbx}{L} + \frac{Pb}{L}(x_o + r_o) - F_f d - \sum_{i=1}^{n} R_i r_i = 0 \qquad (6)$$

where

d = distance between the centroids of the top and bottom flange splice plates
F_f = force in the top or bottom flange bolts on one side of the splice
n = number of bolts on one side of the web splice
r_i = distance from the ith bolt to the instantaneous center of rotation
r_o = distance from the centroid of one bolt group to its instantaneous center of rotation
R_i = force in the ith bolt
R_{iv} = vertical component of the bolt force R_i
x_o = distance from the centerline of the splice to the centroid of the bolt group on one side of the splice.

Although this development started with a single concentrated load acting on a simply supported beam, it can be shown that the foregoing statements are true for any general loading case. Therefore, in order to present these equations in a general form, Pb/L will be replaced by V, the shear at the section, and Pbx/L will be replaced by M, the moment at the centerline of the splice. Using this notation, the equilibrium equations become:

$$\sum_{i=1}^{n} R_{iv} - V = 0 \qquad (7)$$

$$\sum_{i=1}^{n} R_i r_i + F_f d - [M + V(x_o + r_o)] = 0 \qquad (8)$$

Equation 7 identifies that it is the vertical components of the bolt forces that resist the transverse shear at the section. Equation 8 identifies how the moment transferred across the splice is shared between the bolts in the web splice and the bolts in the flange splices.

Althought Kulak, et al. recognized that it would be advantageous from a designer's point of view if a location of the eccentric shear force could be established that would yield results identical to those given by the equilibrium equations, they found no particular relationship between the eccentricity of the shear force and the center of gravity of either bolt group. However, a further examination of the equilibrium equations shows that the term $V(x_o + r_o)$ is the moment at the instantaneous center produced by the eccentric shear

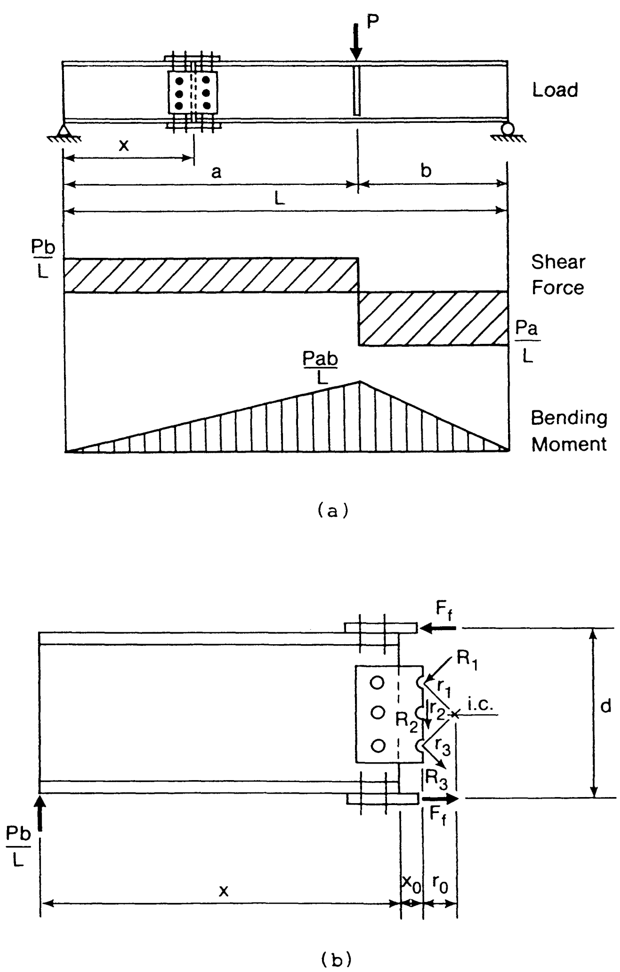


Fig. 2. Analytical model for a web-flange splice.

force. This leads to a solution to the problem which is identical to that for a bolt group loaded eccentrically by a force V located at a distance x_o from its center of gravity. In other words, for a web splice located at a point of contraflexure, the design of the bolt group on the one side of the splice can proceed on the same basis as that for a bolt group acting under a load equal to the shear at the splice and located at the centerline of the splice. Design aids are available for this case. In allowable stress design, the procedures and design aids presented in the 9th edition of the AISC *Manual* can be used;[8] if load and resistance factor design is the basis of the design, then the 1st edition of the LRFD *Manual* should be used.[9]

A small problem in nomenclature arises. The distance between the shear force and the centroid of the bolt group, called x_o in the development so far, is given different symbols in the two AISC manuals, and is referred to by yet other different symbols in other design aids.[11] However, most general references and textbooks[1,5,10] refer to this distance as the eccentricity e. In order to avoid confusion, that symbol will be adopted here. Therefore, Eq. 8 will be rewritten as

$$\sum_{i=1}^{n} R_i r_i + F_f d - [M + V(e + r_o)] = 0 \qquad (9)$$

Equations 7 and 9 are general and can be applied to bolted web-flange splices in both simple and continuous beams. For the special case of a beam in which only the web is spliced, but wherein both shear and moment are present, there are no forces transferred across the flanges. The equilibrium equations yield results identical to designing the bolt group on one side of the splice to resist the shear and moment at the centerline of the splice, applied to the bolt group as shown in Fig. 3. The ultimate capacity of the bolt group can then be determined using the ultimate strength method for analyzing eccentrically loaded connections.

For the case of a beam or girder in which both the web and the flanges are spliced, the designer will have to make an assumption regarding the portion of the moment at the location of the splice that the flange splices will be designed to resist. This assumption is then used in Eq. 9 to identify how the moment is shared between the web splice and the flange splices. Traditionally, the flange splices have been designed to resist either 100 percent of the moment at the centerline of the splice or the portion of the moment that the flanges in the beam or girder were designed to resist. Either one of these approaches may be used as long as the equilibrium statements (Eqs. 7 and 9) are satisfied.

(If the designer chooses to provide a flange splice connecton that is based on the assumption that the flanges carry 100 percent of the moment, then obviously the flange splice is designed conservatively. The concomitant question is whether the web splice at the same location designed only for shear, that is, neglecting the proportion of the total moment that must be present in the web, will be satisfactory.

There have not been any physical tests covering this case. However, design procedures for the beam itself include the assumption that an I-shaped cross section can attain the full plastic moment capacity even though the web is fully yielded in shear.[12] This is the result of the beneficial effect of strain-hardening of the flange material: the flanges pick up the relatively small contribution that the web makes to the total moment capacity. In the web and flange splice case, the flange splice plates should behave similarly to the flanges of any unspliced beam, that is, the flanges should be capable of carrying all of the moment. As far as the web bolts themselves are concerned, the rigid body movement of the beam relative to the splice means that the deformation imposed on any of the bolts due to moment will always be a maximum in the flange splice bolts. Thus, assigning all the moment to the flange splice plates and all the shear to the web splice plates should be a satisfactory situation.)

In order to substantiate the analytical procedure just presented, an experimemtal program was established. Tests of large-size spliced members were carried out in order to establish the ultimate capacity of bolted web splices in which the number and arrangement of bolts varied. The experimental results were compared with analytical predictions. The web splice design approach which uses the assumption that the shear force acts at the centerline of the splice will subsequently be referred to as the "proposed method of analysis." The web splice design approach which uses the assumption that the shear force acts at the centroid of the opposite bolt group will subsequently be referred to as the "conventional method of analysis."

EXPERIMENTAL PROGRAM

In order to be able to predict the capacity of a group of

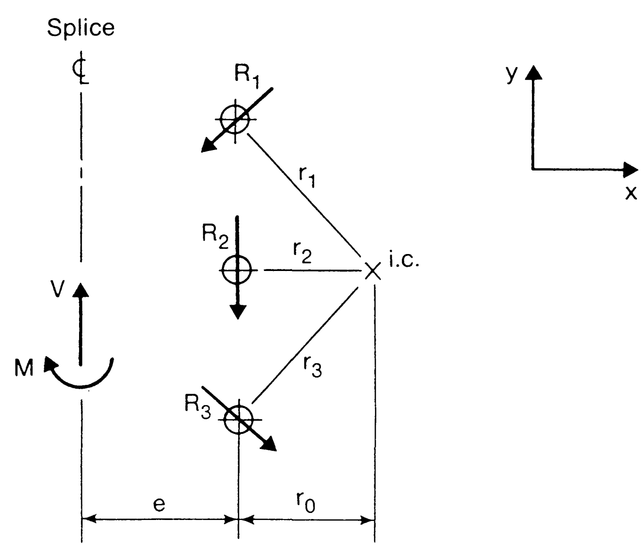


Fig. 3. Web splice bolt group design forces.

fasteners in the full-size test specimens, it is necessary first to know the load versus deformation response of a typical individual fastener. In this program, both a tension jig and a compression jig[10] were used, although, as will be noted subsequently, the tension test jig results are considered to more closely represent actual conditions. The specimens were detailed to conform as closely as possible to the details in each of the full-scale test configurations. In all cases, the steel plates used to make the jig were cut from the same material that was used to fabricate the full-size bolted web splice test specimens described subsequently. The test bolts were ¾ in. diameter ASTM A325 bolts, 3½ in. in length, of which 1⅜ in. was threaded. Any possible variation in the bolt properties was minimized by using bolts that were all from the same production lot. The bolts were tightened to the snug position.

The large-scale web splice test specimens were constructed by joining two steel beams together using two steel splice plates, one on either side of the web. These plates were lapped across the joint and bolted to the beam webs. The dimensions of the beams were chosen so that the beams would not yield before the bolt group in the web splice reached its ultimate capacity. The steel in the beams was required to meet CSA Specification G40.21-M 300W. The specified minimum yield strength of this steel is 300 MPa,[13] or about 44 ksi.

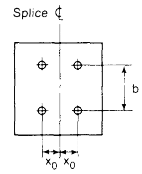
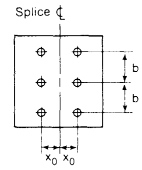
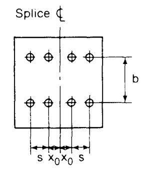
Six different splices were tested. In all cases, the thickness of the beam webs was ¾ in. The thickness of the splice plates used in specimens C1 through C4 (see Table 1) was also ¾ in. These splice plates were cut from the same plate that was used to make the beam webs. The thickness of the splice plates used in the remaining two specimens, C5 and C6, and was ½ in. The other dimensions of the splice plates were simply a reflection of the bolt patterns used.

Details of the geometry of the bolted web splices are provided in Table 1. In all of the full-scale tests, the connections used in ¾ in. diameter ASTM A325 bolts from the same lot as those used in the single bolt shear specimens. A ¾ in. drill bit was used to ensure that all of the bolts would be bearing against the web and the splice plates as soon as a load was applied. The initial slippage was minimized by the small clearance of the bolt holes. Although this represents an idealized condition, it prevents another variable (slippage) from being introduced into the experiment. It is felt that the idealized condition approximates the behavior of the bolts in a real connection as the connection reaches its ultimate capacity. The bolts were tightened to the snug condition.

The set-up used to test the bolted web splice specimens is shown in Fig. 4. Two independently controlled hydraulic jacks were used to apply vertical loads to the specimen, and the loads applied to the specimen were measured using electronic load cells placed at the load and reaction points.

Because the tests were designed so that only the bolted web splice transferred the forces across the joint, a gap was

Table 1.
Details of Test Specimens

Specimen	Bolt Group	x_o (in.)	b (in.)	s (in.)
C1		1¼	4	—
C2		1¼	3⅛	—
C3		2	8⅝	—
C4		2	3½	—
C5		2	4¾	2⅜
C6		2¾	8⅝	2¾

left between the beams to ensure that the ends of the beams did not touch each other at any time during the tests. Electrical resistance strain gauges were placed along the beam flanges to monitor the bending moment in the specimens. Displacement transducers were positioned to measure the vertical deflections of the beams and the horizontal and vertical movements of the splice plates relative to the beam webs.

For specimens C1 through C5, the east and west jacks were used to apply equal loads; thus, the centerline of each splice was at the theoretical location of zero moment. In the case of specimen C6, the load applied to the east jack was twice

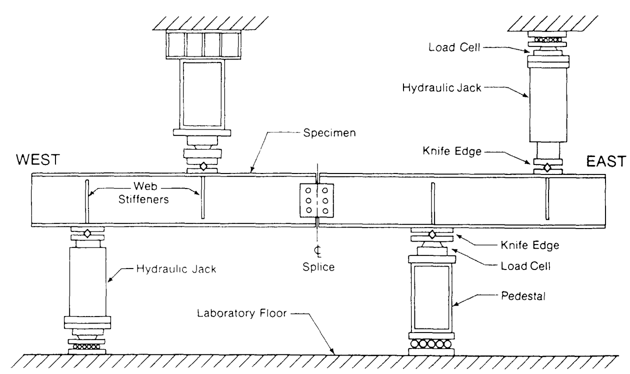


Fig. 4. Schematic diagram of test set-up.

as great as that applied to the west jack, resulting in a moment at the location of the splice that was greater than zero.

Load cell, strain gauge and transducer readings were electronically obtained and recorded at each load step. Testing was stopped when the ultimate load was reached. When testing had been completed, the angles of deformation of the bolt holes in the web were measured with respect to the horizontal axis.

EXPERIMENTAL RESULTS AND DISCUSSION

A typical load versus deformation response of single bolts tested in double shear (tension jig) is shown in Fig. 5. There is no well-defined yield point, and the relationship between load and deformation deviates from linearity as the load increases. The mean ultimate load and deformation of each series of specimens are presented in Table 2.

The mean ultimate capacity of the single bolt shear specimens in series A is approximately equal to the mean ultimate capacity of the specimens in series B. Because all of the bolts were from the same lot, little variation in the ultimate shear capacity was expected using the same type of test jig. The average deformation at the ultimate load is higher for the specimens in series B than for those specimens in series A. This difference can be attributed mainly to the

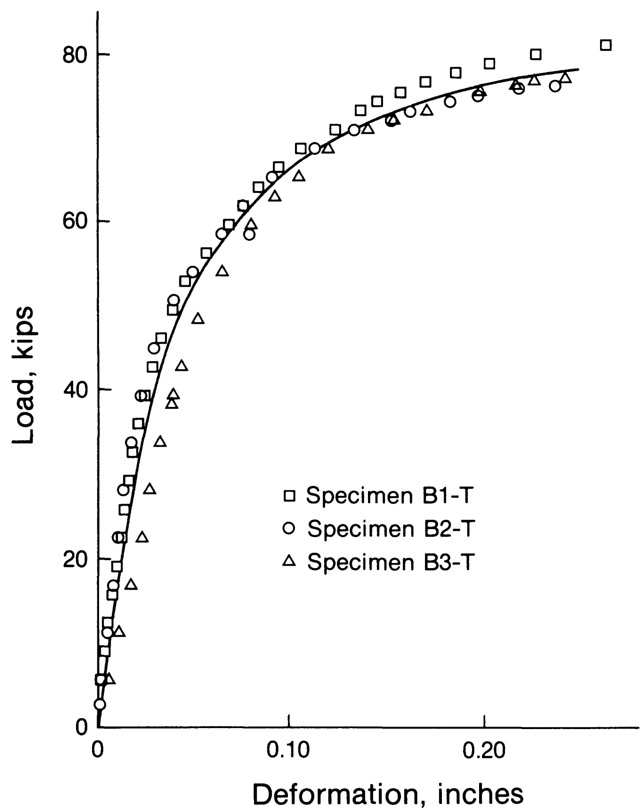


Fig. 5. Typical single bolt shear load v. deformation response.

Table 2. Single Bolt Shear Test Results

	Average Ultimate Load (kips)	Average Ultimate Deformation (in.)
Series A		
Compression Jig	82.7	0.19
Tension Jig	74.9	0.21
Series B		
Compression Jig	83.0	0.24
Tension Jig	77.3	0.25

differences in the thicknesses of the steel plates used in the two series and not to variation in bolt or steel properties.

An experimental study carried out by Wallaert and Fisher[14] showed that the shear strength of high-strength bolts contained in plates loaded in compression is approximately 10 percent higher than the shear strength of high strength bolts contained in plates loaded in tension. The lower shear strengths of the bolts tested in tension jigs were attributed to a tensile force in the bolts resulting from prying action of the outside plates.

The mean ultimate load for series A tension jig specimens is 90 percent of the mean ultimate load for series A compression jig specimens. The mean ultimate load for series B tension jig specimens is 93 percent of the mean ultimate load for series B compression jig specimens. Thus, these results are consistent with the work of Wallaert and Fisher.

Considerable permanent deformation of the web steel occurred around the bolt holes as a result of yielding of the steel under bearing action of the bolts. Typically, a bolt hole was deformed by approximately $\frac{1}{16}$ in. in the direction of the bolt force.

One of the ways in which the validity of an analytical method for the design of the bolts in a beam web splice can be evaluated is to compare the observed direction of the bolt force with a predicted direction. The experimental (measured) and theoretical angles of deformation of the web bolt holes, measured with respect to a horizontal axis, are compared in Table 3. The predicted direction of the bolt force is available directly from the analysis of the connection strength. In Table 3, the theoretical angles of deformation were calculated using both the conventional method of analysis and the method of analysis developed earlier in this paper, the proposed method. Clearly, the best agreement between the experimental and theoretical angles of deformation results using the method of analysis developed in this paper. Thus, the measured angles of deformation support the proposed method of analysis which considers the shear force to act at the centerline of the splice.

Failure of the bolts in the web splice specimens occurred shortly after the maximum loads were reached. The experimental and predicted ultimate shear capacities of the bolted web splice specimens are listed in Table 4. The predicted

Table 3. Bolt Force Directions				
		Predicted Angle (degrees)		
Specimen	Bolt	Proposed Method	Conventional Method	Test Angle (degrees)
*C*1	1	57	38	60
	2	57	38	60
	3	57	38	60
	4	57	38	55
*C*2	1	51	27	59
	2	51	27	47
	3	51	27	37
	4	51	27	59
*C*3	1	65	48	72
	2	65	48	72
	3	65	48	67
	4	65	48	65
*C*4	1	43	22	43
	2	90	90	87
	3	43	22	40
	4	43	22	43
	5	90	90	87
	6	43	22	45
*C*5	1	34	2	25
	2	34	2	25
	3	65	46	53
	4	65	46	55
	5	65	46	55
	6	65	46	53
	7	34	2	28
	8	34	2	35
*C*6	1	—	—	—
	2	—	—	—
	3	—	—	—
	4	—	—	—
	5	34	—	35
	6	34	—	28
	7	2	—	0
	8	2	—	0

Table 4. Comparison of Web Splice Test Results with Predictions Based on the Proposed Method of Analysis and Tension Jig Test Results				
	Predicted Shear Capacity	Test Shear Capacity	Test	
Specimen	kips	kips	Predicted	% Error*
*C*1	126.1	123.9	0.98	+1.8
*C*2	116.9	116.5	1.00	0.4
*C*3	136.2	128.1	0.94	+6.3
*C*4	182.1	176.0	0.97	+3.4
*C*5	185.2	179.4	0.97	+3.3
*C*6	89.9	82.5	0.92	+9.0

* The test value is taken as the true value.

shear capacities were calculated using the proposed method of analysis, that is, with the shear force considered to be acting at the centerline of the splice. The response to shear load obtained from the single bolt tension jig tests was used in the analysis. The ultimate shear capacity test-to-predicted ratio varies from 0.92 to 1.00. The mean value of the ratio for the six tests is 0.96. These ratios indicate that this method of analysis yields results that are consistent, but slightly unconservative. The factors that may cause the results to be unconservative will be examined subsequently.

The experimental ultimate shear capacities were also compared to predictions made using compression jig test results.[15] In this case the test-to-predicted ratio ranged from 0.85 to 0.90.

The prediction of the ultimate shear capacities of web splice specimens Cl through C5 is based on the assumption that the moment at the centerline of the splice is equal to zero. Under idealized conditions, the bolt groups on either side of the web splices in these specimens would fail simultaneously. Even small differences between the idealized and experimental test set-up will result in one bolt group failing before the other one. In the tests carried out in this study, the east bolt group failed first in all of the specimens. However, visual inspection of the west bolts in specimens Cl through C5 after the tests indicated that the west bolt groups had almost reached their ultimate loads.

The test set-up permitted free horizontal translation at the locations of the east jack, east reaction, and west jack. Horizontal movement occurred both as a result of bending of the beams and from deformation of the splice components, but this movement was not symmetrical about the centerline of the splice because the west reaction was fixed. Although these horizontal movements were relatively small, they were large enough to cause the location of zero moment not to coincide with the centerline of the splice. Because the east bolt group was always the first to fail, it is concluded that the horizontal translations resulted in the location of zero moment being slightly west of the centerline of the splice in specimens Cl through C5. A moment at the centerline of the splice would contribute to an unconservative prediction of the shear capacity of a web splice if the analysis was carried out on the assumption that the moment at the centerline of the splice was equal to zero.

Crawford and Kulak[16] suggested that one reason connections which are subjected to eccentrically applied loads are not able to reach their theoretical ultimate capacities is because, in a full-scale test, the direction of the force on each bolt changes as the instantaneous center of rotation moves, whereas the direction of the force and corresponding deformation never changes in the single bolt calibration test. In the test program reported in this paper, specimens Cl, C2, and C3 contained two bolts in a single line on either side of the splice. For this bolt arrangement, the theoretical location of the instantaneous center of rotation does not change as the magnitude of the eccentrically applied load increases.

Consequently, there is no change in the direction of the force on each bolt. For the bolt arrangements used in specimens C4 and C5, the theoretical location of the instantaneous center moved a relatively small distance when the applied load was within the range where most of the deformation occurred. This resulted in only minor changes in the direction of the forces acting on the bolts. Therefore, movement of the instantaneous center during the loading history probably was only a small contributor to the difference between theoretical predictions and test results in this program.

Tests carried out in this study show that the shear strength of high-strength bolts in plates loaded in compression is greater than the shear strength of high-strength bolts in plates loaded in tension. This effect should result in the upper east and lower west bolts in the web splices failing at a lower load than was assumed in the connection ultimate strength analysis because these bolts are located in tension regions of the beam webs. This reduced shear capacity of the bolts in the tension regions can contribute significantly to decreasing the web splice shear capacities as predicted on the basis of the response to shear load of the single bolt compression jig specimens.

The predicted shear capacities of the web splices were also calculated using the assumption that the shear force acts at the centroid of the opposite bolt group. The response to shear load of the tension jig specimens was used in the analysis. The results, given in Table 5, show that the ratio of the ultimate test shear capacity to the predicted value ranges from 1.16 to 1.54 (mean value 1.39) using this conventional method of analysis. These test-to-predicted ratios indicate that the conventional method of analysis yields results that are both quite conservative and inconsistent, and that are considerably less accurate than were obtained using the method of analysis developed in this paper.

It should be noted that the conventional method of analysis is usually used together with the assumption that the web splice is designed to transfer only shear. Because the splice in specimen C6 was located where both shear and moment were present, it has not been included in Table 5.

The comparisons between the experimental (test) results and the proposed and conventional methods of analysis which have been reported herein show clearly that the proposed method of analysis is validated and that the conventional method of analysis is conservative. A further examination has been made by considering hypothetical cases. This examination, which is reported in full elsewhere,[15] included a total of 126 connections. The number of bolts per row ranged from 2 to 12, the number of rows of bolts on either side of the splice ranged from one to four, and various bolt spacing and eccentricities were included. For the range of the examination, the ratio of predicted load by the conventional method to predicted load by the proposed method was observed to vary from 1.03 to 1.84, with a mean value of 1.37. To the extent that these data are representative, it indicates that a considerable savings can be made in the design of web splices if the shear force is assumed to act at the centerline of the splice rather than at the centroid of the opposite bolt group.

Table 5.
Comparison of Web Splice Test Results with Predictions Based on the Conventional Method of Analysis and Tension Jig Test Results

Specimen	Predicted Shear Capacity kips	Test Shear Capacity kips	Test / Predicted	% Error*
*C*1	92.2	123.9	1.34	−25.6
*C*2	79.4	116.4	1.47	−31.9
*C*3	110.8	128.1	1.16	−13.5
*C*4	122.8	176.0	1.43	−30.3
*C*5	116.7	179.4	1.54	−35.0

* The test value is taken as the true value.

CONCLUSIONS

Analysis of the test results leads to the following conclusions:

1. The ultimate shear strength of high strength bolts tested in plates that are loaded in tension is up to 10 percent less than when the plates are loaded in compression. The reduction in ultimate strength is attributed to an increase in the axial force in the bolt.
2. The measured angles of deformation of the web bolt holes support the predictions of the bolt force directions obtained using the assumption that the shear force acts at the centerline of a web splice.
3. Analyzing the bolt group on one side of a web using the assumption that the shear force acts at the centroid of the opposite bolt group yields results that are inconsistent and conservative when compared with the test results.
4. Analyzing the bolt group on one side of a web splice on the basis that the shear fore acts at the centerline of the splice leads to good agreement between theory and experiment. When the tension jig single bolt response was used in the model, the test to predicted ratio ranged from 0.92 to 1.00 for the specimens tested in this program. This comparison includes the case in which both shear and moment were present at the splice location as well as the more usual case in which only shear is present.

DESIGN RECOMMENDATION

1. The actual response to shear of a single bolt tested in a tension jig provides a lower bound when used in the solution for the ultimate load of a bolt group. Therefore, it is recommended that eccentrically loaded bolted connections be designed using this load response rather than the currently used compression jig shear response.
2. The bolts in a beam web splice located at the point of contraflexure can be proportioned by considering the bolt group on either side of the splice to be acted upon by

a shear force located at the centerline of the splice. Design aids available for the design of eccentrically loaded connections can be used in this case.

3. When both shear and moment are present and only the web is spliced, the ultimate strength of the bolts in the splice can be established using the equilibrium equations outlined herein. The bolt group must be proportioned to carry both the moment present at the splice and a shear force located at the centerline of the splice.
4. When both shear and moment are present and both the flanges and the web are spliced, the equilibrium equations outlined herein can be used first to assign the proportion of moment to be carried by each component. The bolts in each component can then be designed using the method described herein.

ACKNOWLEDGMENTS

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