The Ultimate Strength of Unsymmetric Beam Bolted Splices

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ABSTRACT

A beam splice must be designed to transfer the shear and moment of the beam at the location of the splice. However, the actual force distribution between the web and flange splices is unknown. Currently, design methods for beam splices are conflicting with one another. These methods result in a different number of bolts, and are difficult to interpret, especially when designing unsymmetric splices. This paper presents a summary of an analytical and experimental research program performed on unsymmetric beam bolted splices. Simplified recommendations are presented for the force distribution between the web and flange splices. The validity of these recommendations is verified by the experimental program. To demonstrate the application of the design recommendations, three design examples are presented.

INTRODUCTION

The shipping length of steel beams could be limited by site and/or fabricator constraints. Beams longer than 125 to 150 ft are usually shipped in pieces, and then field spliced. Due to the difficulty of field welding, field splices are usually bolted. A typical bolted field splice is shown in Figure 1.

A beam splice must be designed to transfer the shear and moment of the beam at the location of the splice. Currently, conflicting design procedures are suggested for bolted splices. The American Institute for Steel Construction (AISC, 1994), in its LRFD *Manual of Steel Construction*, recommends that flange splices be designed for the total moment. It also recommends that web splices be designed for the shear, as well as moment, which is caused by the eccentricity of the shear. For symmetric web splices, AISC defines the eccentricity of the shear as the distance from the centerline of the splice to the centroid of the bolts on one side of the splice. On the other hand, The American Association of State Highway and Transportation Officials (AASHTO, 1996) requires that flange splices be designed for the portion of the design moment not resisted by the web. It also requires that web splices be designed for the shear, the moment caused by the eccentricity of the shear, and the portion of the design moment resisted by the web. AASHTO gives no guidance on how to determine the eccentricity of the shear, or how to apportion the moment between the web and flanges. Both AISC and AASHTO procedures have been used in the design of splices; however, the AASHTO procedure requires a larger number of bolts.

Because the force distribution between the web and flanges is open to interpretation, engineers face the task of determining the design forces for beam splices. Furthermore, engineers find it challenging to interpret either the AISC or AASHTO procedures, especially when designing unsymmetric splices. This paper presents a summary of an analytical and experimental research program performed on such splices. Design recommendations for the force distribution between the web and flange splices are presented based on the results of the research program. To demonstrate the application of the design recommendations, three design examples are presented.

ANALYTICAL PROGRAM

A research program performed on bolted splices of symmetric beams identified two cases for the force distribution between the web and flanges (Sheikh-Ibrahim and Frank, 1998). These cases correspond to beams with undeveloped and developed flanges, respectively. A beam is classified as having undeveloped flanges when the applied moment is smaller than, or equal to, the moment required to develop the flanges. Conversely, a beam is classified as having developed flanges when the applied moment is larger than

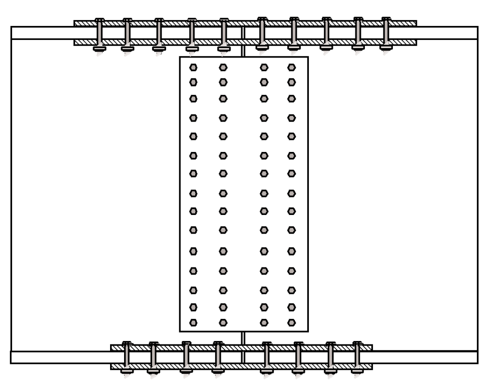

Fig. 1. Typical bolted field splice.

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the moment required to develop the flanges. For both cases, the moment required to develop the flanges is defined by Sheikh-Ibrahim and Frank (1998) as the axial resistance of the controlling flange times the distance between the centroids of the top and bottom flanges. The axial resistance of the controlling flange is defined as the smallest of the axial resistance of the compression and tension flanges. Furthermore, the axial resistance of the compression flange can be determined based on gross-section yielding, and the axial resistance of the tension flange can be determined based on gross-section yielding or net section fracture, whichever controls.

When the axial resistance of the flange splice plates is less than the axial resistance of the flanges, the force distribution between the web and flanges becomes dependent on the applied moment, and on the moment required to develop the flange splice plates. The moment required to develop the flange splice plates is defined by Sheikh-Ibrahim and Frank (1998) as the axial resistance of the controlling flange splice plate times the distance between the centroids of the top and bottom flange splice plates. The axial resistance of the controlling flange splice plate is the smaller of the axial resistance of the compression and tension flange splice plates. Furthermore, the axial resistance of the compression flange splice plate can be determined based on buckling or gross section yielding, and the axial resistance of the tension flange can be determined based on gross-section yielding or net section fracture, whichever controls. In this paper, the flange plate will refer to either the flange splice plate, or beam flange, whichever has the smaller axial resistance.

In the above-mentioned program, undeveloped flange plates were found capable of carrying the total moment through a tension-compression couple acting as axial forces at the centroids of the top and bottom flange plates. Undeveloped flange plates were also found capable of carrying a portion of the total shear, which could be conservatively ignored. Consequently, for such beams, the web splice can be designed for only the shear applied at the centerline of the splice. On the other hand, the web splice of beams with developed flange plates should be designed for the moment, which is not resisted by the flange plates, as well as the shear applied at the centerline of the splice.

In this paper, the two cases of force distribution, which were developed for symmetric beam splices (Sheikh-Ibrahim and Frank, 1998), are generalized and expanded into three cases to cover unsymmetric beam splices. The three are referred to as Case 1–Undeveloped Flange Plates, Case 2–Developed Smaller Flange Plate and Undeveloped Larger Flange Plate, and Case 3–Developed Flange Plates. The shear carried by the undeveloped flange plates is conservatively ignored to simplify the analysis of unsymmetric splices.

Case 1–Undeveloped Flange Plates

This case applies when the applied moment, M, is less than the moment required to develop the smaller flange plate, M_f. The moment required to develop the smaller flange plate is defined as the axial resistance of the smaller flange plate, P_{rf}, times the distance between the centroids of the top and bottom flange plates, $\bar{h}$.

The assumed force distribution for the case of unsymmetric beams with undeveloped flange plates is shown in Figure 2a. The applied moment is assumed to be carried by a tension-compression couple acting as axial forces at the centroids of the top and bottom flange plates. The web splice is assumed to carry the shear applied at the centerline of the splice. Therefore, the splice forces at the centerline of the splice can be calculated as follows:

$$P_f = P_F = M / \bar{h} \tag{1}$$

$$V_w = V \tag{2}$$

$$H_w = 0 \tag{3}$$

$$M_w = 0 \tag{4}$$

in which P_f and P_F are the axial forces in the smaller and larger flange plates, respectively; V is the applied shear; and V_w, H_w, and M_w are the vertical and horizontal shears, as well as moment resisted by the web, respectively.

Case 2–Developed Smaller Flange Plate and Undeveloped Larger Flange Plate: $M_f < M \leq M_F$

This case applies when the applied moment exceeds the moment required to develop the smaller flange plate, but is less than the moment required to develop the larger flange plate, M_F. The moment required to develop the larger flange plate is defined as the sum of moments (taken about the mid-height of the web) of the axial force resistance of the smaller flange plate, P_{rf}, and the axial force resistance of the larger flange plate, P_{rF}.

The assumed force distribution for the case of unsymmetric beams, with developed smaller flange plate and undeveloped larger flange plate, is shown in Figure 2b. The smaller flange plate is assumed to develop its axial resistance. The moment in excess of that required to develop the smaller flange plate is assumed to be resisted by the larger flange plate and web through a tension-compression couple, acting as axial forces at the centroid of the larger flange plate and mid-height of the web. In addition to the moment-induced axial force, the web is assumed to carry the shear applied at the centerline of the splice. Therefore, the splice forces at the centerline of the splice can be calculated as follows:

$$P_f = P_{rf} \tag{5}$$

$$P_F = P_f + (M - M_f) / h_F = P_{rf} + (M - P_{rf}\,\bar{h}) / h_F \tag{6}$$

$$V_w = V \tag{7}$$

$$H_w = P_F - P_f = (M - P_{rf}\,\bar{h}) / h_F \tag{8}$$

$$M_w = 0 \tag{9}$$

in which h_F is the vertical distance between the mid-height of the web and the centroid of the larger flange plate.

Case 3–Developed Flange Plates: $M > M_F$

This case applies when the applied moment exceeds the moment required to develop the larger flange plate. The assumed force distribution for unsymmetric beams with developed flange plates is shown in Figure 2c. At the centerline of the splice, both flange plates are assumed to develop their axial resistances. The web splice is assumed to carry the shear, unbalanced flange axial forces, and moment in excess of that required to develop the larger flange plate. Therefore, the splice forces at the centerline of the splice can be calculated as follows:

$$P_f = P_{rf} \tag{10}$$

$$P_F = P_{rF} \tag{11}$$

$$V_w = V \tag{12}$$

$$H_w = P_F - P_f \tag{13}$$

$$M_w = M - M_F = M - (P_{rf}\,h_f + P_{rF}\,h_F) \tag{14}$$

in which h_f is the vertical distance between the mid-height of the web and the centroid of the smaller flange plate.

EXPERIMENTAL PROGRAM

Full-Scale Splice Tests

To verify the validity of the analytical program, seven full-scale unsymmetric beam splices were tested. In these tests, the number of web bolts on each side of the splice ranged from two to fourteen, and the number of vertical bolt rows ranged from one to two. The ratio of the top flange plate area to bottom flange plate area ranged from 2.7 to ∞.

The test set-up for the beam splices is shown in Figure 3. The test beam was a W40×167 simply supported A572 Gr. 50 steel beam, with an 18-ft span. The cross-section of the beam was made unsymmetric by cutting the bottom flange to a 7½-in. width. The beam was spliced 4 ft from the

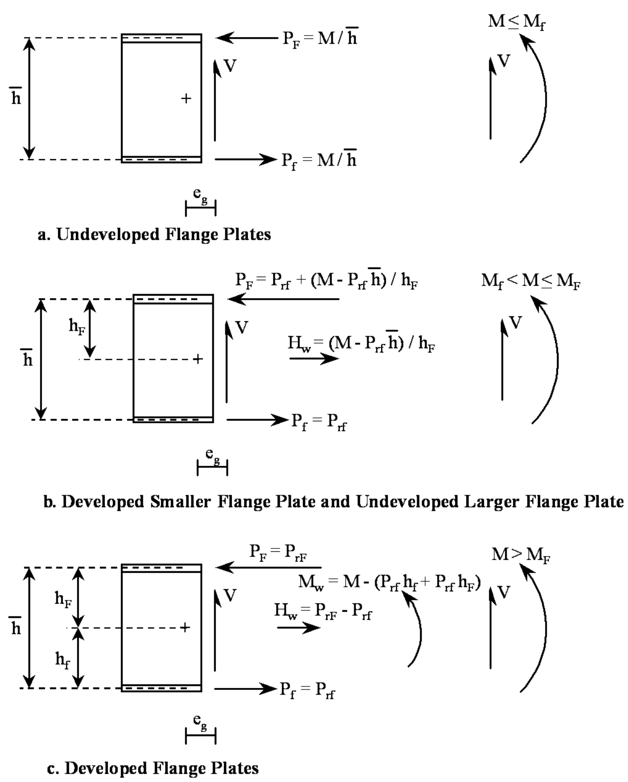

Fig. 2. The force distribution for unsymmetric beam splices at maximum load.

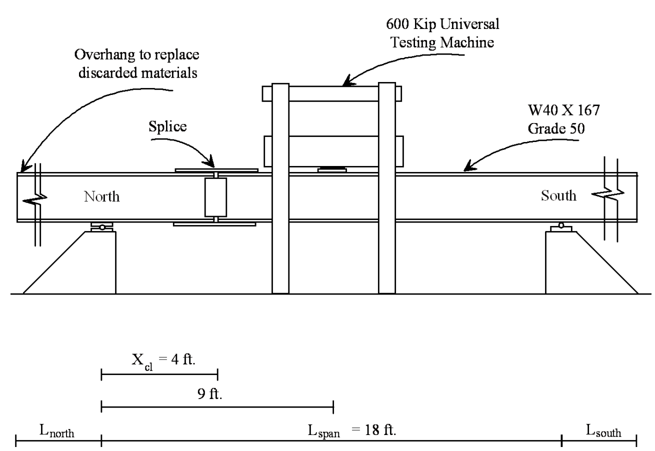

Fig. 3. Test set-up.

Table 1. Details of Splice Plates for the Full-Scale Specimens

Splice	$b_F \times t_F$ (in. × in.)	$b_f \times t_f$ (in. × in.)	$d_w \times t_w/2$ (in. × in.)
7u	10.03×0.503	-	28.94×0.403
8u	10.03×0.503	-	28.94×0.403
9u	10.03×0.503	7.51×0.246	28.94×0.403
10u	10.03×0.503	7.51×0.246	28.94×0.403
15u	10.03×0.503	7.51×0.246	28.94×0.403
17u	10.03×0.503	7.51×0.246	8.94×0.403
19u	10.03×0.503	7.51×0.246	28.94×0.403

Table 2. Distribution of Bolts and Geometrical Details for the Full-Scale Specimens

Splice	$n \times m$	b (in.)	D (in.)	e_g (in.)	G (in.)	$N_F \times R$	$N_f \times R$	E_f (in.)
7u	7 × 2	4.1875	3	3.75	½	10 × 2	-	-
8u	4 × 1	4.1875	-	2.25	½	10 × 2	-	-
9u	6 × 1	4.25	-	2.5	½	10 × 2	7 × 2	1.5
10u	6 × 1	4.25	-	2.5	½	10 × 2	6 × 2	4
15u	3 × 1	12.25	-	2.5	¾	10 × 2	7 × 2	4
17u	2 × 1	3	-	3	¾	10 × 2	6 × 2	9
19u	6 × 1	4.25	-	2.5	¾	10 × 2	7 × 2	4

north support, and the clearance gap between the two ends of the connected beam segments ranged from ½ in. to ¾ in.

Details of the splice plates, and distribution of bolts for all the specimens, are given in Figure 4, Table 1, and Table 2. The web splice plates consisted of two ⅜-in. A36 steel plates, one plate on each side of the web. The top flange splice plate consisted of a 10-in. × ½-in. A36 steel plate. The bottom flange splice plate for all but splices 7u and 8u consisted of a 7½-in. × ¼-in. A36 steel plate. For splices 7u and 8u, the bottom flange was not connected to exaggerate the unsymmetry of the splice, and to examine whether the web splice is capable of carrying the forces not resisted by the bottom flange. The flange splice plates were bolted on both sides using two longitudinal rows of ¾-in. diameter, ASTM

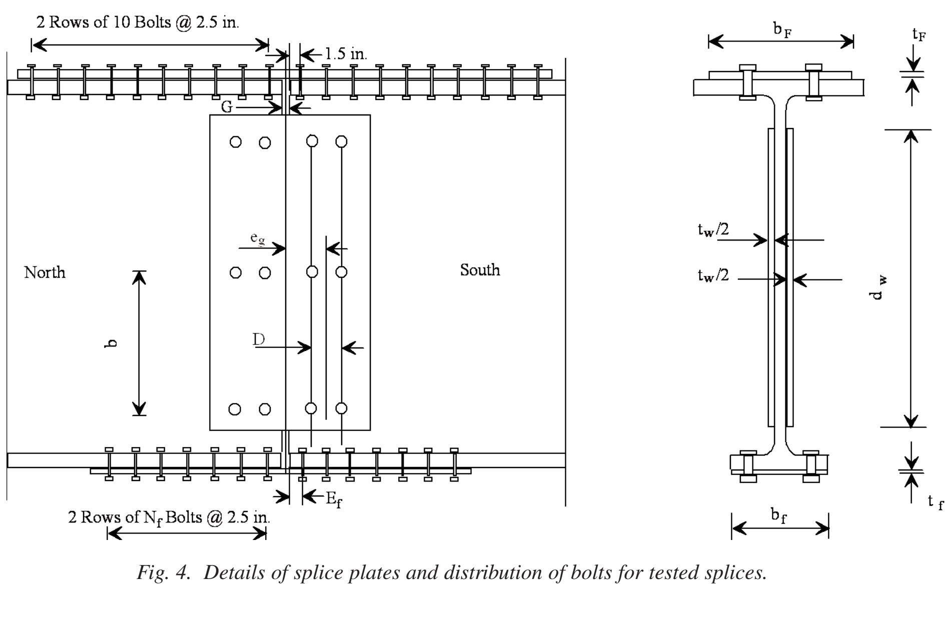

Fig. 4. Details of splice plates and distribution of bolts for tested splices.

A325 high-strength bolts, at 2½-in. longitudinal spacing. The top flange bolts were 2¾-in. long, ¾-in. diameter, ASTM A325 high-strength bolts. The web and bottom flange bolts were 2½-in. long, ¾-in. diameter, ASTM A325 high-strength bolts. The bolt holes in the beam were drilled, and those in the splice plates were punched to expedite the fabrication process. The top flange bolts were from one production lot. Also, the web and bottom flange bolts were from one production lot. All holes were ⅟₁₆ in. larger than the nominal diameter of bolts.

The full-scale specimens were designed to cover the three cases of force distribution, which were dependent on the axial resistance of the flange splice plates, as presented in this paper. When a splice strength was to be determined assuming a fixed moment-to-shear ratio, as was the case encountered in the design of the specimens, the case of force distribution was found sensitive to the type of eccentric shear analysis performed on the web bolts. For example, a splice designed as having undeveloped flange plates, when using elastic analysis of the web bolts, could be classified as one having developed flange plates when using nonlinear analysis. This is because the capacity of the splice when using nonlinear analysis is larger than that when using elastic analysis. Nonetheless, a decision on the type of analysis to be performed on the web bolts had to be made for the purpose of specimens sizing. For simplicity, and because it produces adequate strength estimates (Sheikh-Ibrahim, 1995), elastic analysis was used to design the web bolts.

All specimens were instrumented with strain gages in the flange splice plates and linear transducers at various locations. The transducers were provided on both sides of the joint, and at midspan of the beam, to measure vertical deflections at those locations. They were also provided to measure relative movements between the splice plates and the test beam.

A 600-kip universal testing machine, as shown in Figure 3, was used to apply a concentrated load to the test beam at midspan. Prior to slipping of the plates, the fully-pretensioned splices were cycled in the elastic range. The maximum load for each cycle was increased up to slipping of the web plates. After the web splice plates slipped and went into bearing, all specimens were loaded to failure.

Tensile Coupons

As was explained in the analytical program, the force distribution is a function of the applied moment and the smallest of the moment resistance of each of the flanges or flange splice plates. Because the flange splice plates used in the experimental program were smaller than the beam flanges, the force distribution in the experimental program was dependent on the axial resistance of each of the flange splice plates, and not on the axial resistance of the beam flanges. Therefore, to provide the best strength estimate for the full-scale specimens, tensile coupon tests were performed on the top and bottom flange splice plates.

The tensile coupons were machined to a 1½-in. width, and loaded in the 600-kip universal testing machine. For each coupon, three static yield values were recorded at several yield plateau locations, and averaged to obtain an estimate of the static yield. Each static yield value was recorded five minutes after stopping the loading head motion. Then the load was increased until fracturing of the coupon. The average yield and ultimate stresses were obtained by averaging the values from two different tested coupons.

Individual Bolt Tests

All splices were designed to fail the critical web bolt in shear. Therefore, to provide the best estimate of the capacity of the splices, individual web bolts were tested in shear, in the tension splice shown in Figure 5. The bolts, bolt preload, plates, holes, and hole-making procedure in the individual bolt tests were identical to those used in the full-scale specimens. The individual bolts were subjected to double shear, with the threads excluded from only one shear plane, which matched the conditions of the bolts in the full-scale specimens. Since most of the full-scale specimens had fully-pretensioned bolts, and one specimen had hand-tight bolts, shear tests were performed on both fully-pretensioned and hand-tight individual bolts. Using the calibrated wrench method, the fully-pretensioned bolts were tightened by applying a preload of 32.9 kips, which exceeds the AISC requirements for bolt installation. The bolt preload was measured, and the installation procedure was calibrated, in a Skidmore-Wilhelm Bolt Tension Calibrator. The hand-tight bolts had no preload to make the effect of bolt preload on the connection strength pronounced, if any. The tension

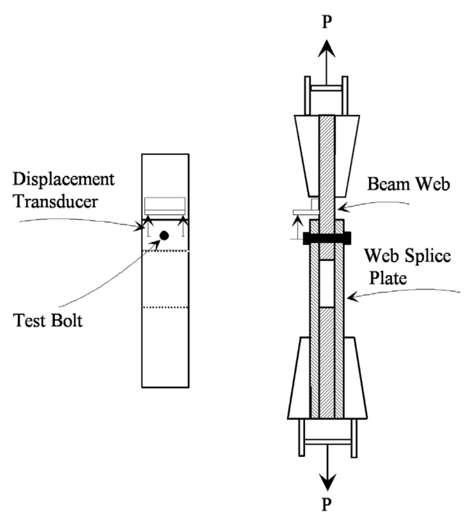

Fig. 5. Test set-up for an individual bolt in a double-shear tension splice.

splices were loaded in a 600-kip universal testing machine. The load was increased until fracturing of the bolt specimen.

DISCUSSION OF EXPERIMENTAL RESULTS

Tensile Coupons

The average static yield stress of the top and bottom flange splice coupons was 44.1 ksi and 43.9 ksi, respectively. Thus, both yield stresses were above the 36-ksi minimum yield stress, as specified by ASTM for A36 steel. Also, the average tensile strength of the top and bottom flange splice coupons was 68.5 ksi and 64.3 ksi, respectively. Thus, both ultimate stresses were above the 58 ksi minimum tensile strength specified by ASTM for A36 steel.

Individual Bolt Tests

All bolts fractured at the shear plane that passed through the threads. The shear strengths of the individual bolts, as well as maximum joint deformations, are shown in Table 3. Also, the average load-deformation responses of the individual bolt specimens are shown in Figure 6. The maximum joint deformations in these load-deformation curves represent the values at which the peak loads occurred. As shown in Table 3 and Figure 6, the average maximum joint deformation of 0.288 inches was constant, regardless of the bolt preload. On the other hand, the mean double-shear strength of the hand-tight and fully-pretensioned bolts, with the threads included in only one shear plane, was 65.3 kips and 74 kips, respectively. These values are 9 percent and 24 percent larger than the summation of the single shear values [(15.9+19.9)/(0.75×0.8) = 59.7 kips] specified in the AISC LRFD specifications, after excluding the 0.75 strength resistance factor (ϕ) and the 0.8 joint-length reduction factor. The latter factor was included in the specified design strength of one bolt to account for the decreasing average strength of all bolts in a joint as a function of the increasing joint length (RCSC, 1996).

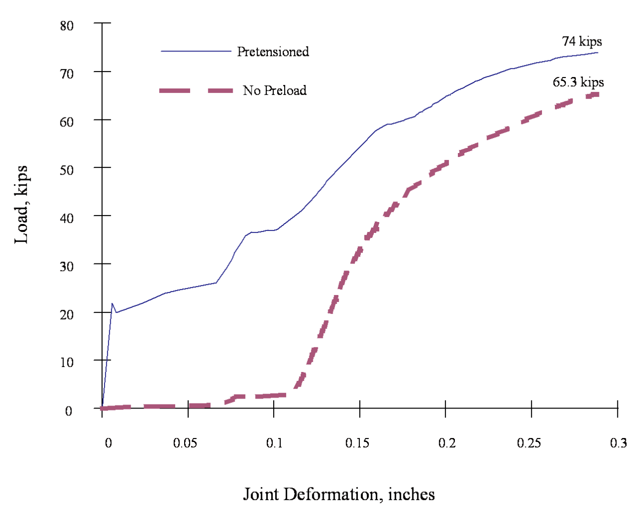

Fig. 6. Load versus joint deformation for a double-shear tension splice with one fully-pretensioned and hand-tight ¾-in. A325 bolt.

Table 3. The Results of Double-Shear Tension Splice Tests of ¾-in. A325 bolts (threads are included in one shear plane)

Specimen Number	Bolt Preload	Joint Deformation (in.)	Double-Shear Capacity (kips)
1	Yes	0.240	75.3
2	Yes	0.342	69.8
3	Yes	0.283	76.8
Average	Yes	0.288	74.0
1	No	0.331	67.3
2	No	0.280	65.1
3	No	0.275	63.2
4	No	0.267	65.4
Average	No	0.288	65.3

Due to the residual friction between the connected plates, the shear strength of the fully-pretensioned bolts was 13 percent larger than that of the hand-tight bolts. However, the bolts in a typical connection are snug tight as a minimum. Therefore, as reported by Wallaert and Fisher (1965), the difference between the shear strength of fully-pretensioned bolts and the shear strength of snug bolts is likely negligible.

Full-Scale Splice Tests

The maximum midspan loads, maximum splice centerline moments and shears, and failure modes for the full-scale specimens are shown in Table 4. The maximum splice-centerline moments and shears reported in Table 4 are adjusted to account for the self-weight of the beam. As predicted, all splices (except 7u and 9u) failed by shearing off the critical web bolt.

During the testing of splice 7u, the spliced beam failed in an elastic lateral-torsional-buckling mode. As shown in Table 5, the experimental maximum load of the beam exceeded the calculated maximum load obtained by using the recommended force distribution along with elastic analysis of the web bolts. However, this experimental load was 4.4 percent less than the calculated maximum load obtained by using the recommended force distribution along with nonlinear analysis of the web bolts. Therefore, the experimental maximum load of the beam was considered a lower bound for the splice maximum load.

Table 4. Test Results of Unsymmetric Beam Splices

Splice	P_{test} (kips)	M_{test} (in.-kips)	V_{test} (kips)	Failure Mode
7u	411	9,689	207	Elastic lateral torsional buckling of spliced beam
8u	208	4,814	105	Fracture of bottom critical web bolt
9u	380	8,979	191	Fracture of bottom four web bolts, and net section fracture of bottom flange splice plate
10u	388	9,198	195	Fracture of bottom critical web bolt
15u	301	7,182	151	Fracture of bottom critical web bolt
17u	147	3,507	74.5	Fracture of bottom critical web bolt
19u	340	8,153	171	Fracture of bottom critical web bolt

Table 5. The Calculated Versus Experimental Maximum Loads for Full-Scale Specimens

Splice	$P_{Elastic}$ (kips)	$P_{Nonlinear}$ (kips)	P_{Test} (kips)	$\text{Difference}_{Elastic}$ %	$\text{Difference}_{Nonlinear}$ %
7u	348	429	411	-15.3	4.4
8u	200	218	208	- 4.0	4.9
9u	295	330	380	-22.5	-13.2
10u	293	329	388	-24.4	-15.3
15u	249	268	301	-17.1	-10.8
17u	131	131	147	-11.3	-11.3
19u	279	309	340	-18.0	-9.1
|Avg.|	-	-	-	16.1	9.9

Splice 9u failed by shearing off the bottom four web bolts (including the critical web bolt), as well as fracturing of the bottom flange splice plate at its net section. Because the failure was sudden, it could not be concluded which component failed first.

One method of verifying the analytical approach is by comparing the experimental behavior with the analytical one. Three types of splice behavior were observed at failure. The type of behavior was found dependent on the magnitude of the applied moment, and the moments required to develop the smaller and larger flange plates. The first failure mode, which is shown in Figure 7, corresponded to the case of undeveloped flange plates. The beam formed a dominant shear kink at the splice location, and the web splice plates underwent double curvature. Thus, at the centerline of the splice, the web splice carried the shear, and the flange plates carried the total moment, as assumed in the analytical program. The larger flange plate carried not only the moment induced axial force, but also a portion of the total shear. This was evident by the formation of plastic hinges at the location of the first row of bolts (point of plate fixity) on

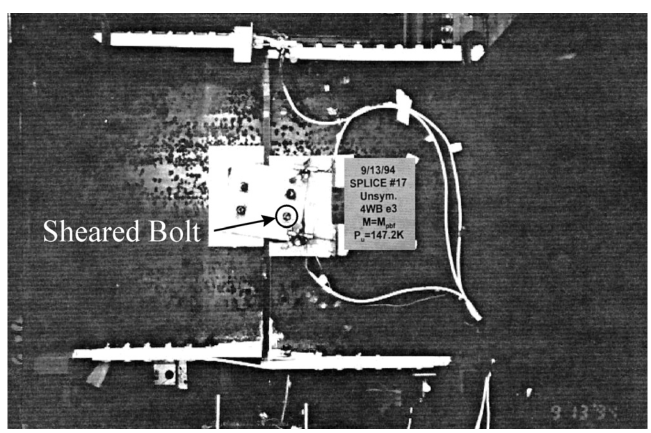

Fig. 7. Typical failed unsymmetric beam splice with undeveloped flange plates (splice 17u).

one side of the joint, and at the beam edge (point of maximum plate bending) on the other side of the joint.

The second failure mode, which is shown in Figure 8, corresponded to the case of a developed smaller flange plate and undeveloped larger flange plate. The beam formed a combination of flexural and shear kinks at failure. Thus, the web splice and larger flange plate carried the moment in excess of that required to develop the smaller flange plate.

Finally, the third failure mode, which is shown in Figure 9, corresponded to the case of developed flange plates. The beam formed a dominant flexural kink at failure. Thus, in addition to the vertical shear, the web splice carried the portion of the moment not resisted by the flange plates. Further, the smaller flange splice plate in specimens 9u and 10u, carrying a moment much larger than that required to develop the smaller flange splice plate, showed evidence of strain hardening. The bottom flange splice plate in both specimens necked down at the net section, and the bottom splice plate of specimen 9u eventually fractured at the net section. Plate fracturing did not occur in specimen 10u since the plate length between the first row of bolts on both sides of the splice was increased, thus causing the plate axial strain to decrease. Necking and/or fracturing of the bottom plate indicates that the plate carried a portion of the moment by strain hardening, which is not considered in the analysis presented in this paper. Consequently, it should be conservative to assume that the web carries the moment in excess of that required to develop the flanges.

Another method of verifying the analytical approach is to compare the calculated maximum loads with the experimental ones. The calculated maximum loads versus the experimental values for the full-scale specimens are shown in Table 5. Also, the normalized, load-deflection responses for all specimens are shown in Figures 10 and 11. The calculated maximum loads for the full-scale specimens are obtained by using an iterative procedure. Starting from an arbitrary, assumed shear capacity, the corresponding splice moment and force distribution are determined. Then, two methods of eccentric shear analysis are performed to determine the strength of the web bolts. Elastic analysis is first used to examine whether it can be used to approximate the strength. Also, nonlinear analysis using the measured load-deformation bolt response is used because it had been shown to produce the best strength estimates (Sheikh-Ibrahim, 1995). The nonlinear analysis is identical to the instantaneous center of rotation method (AISC, 1994), except that the empirical load-deformation response of one bolt is replaced with the measured one. After a new shear capacity of the connection is determined for each of the two previously mentioned analyses, a shear convergence check is performed. If a shear convergence is not achieved, the smaller of the initial and newly calculated shears is incremented by 0.1 percent, and the previous steps are repeated. For the rest of this paper, the calculated maximum load and moment of a splice obtained by using the recommended force distribution, along with elastic analysis of the web bolts will be referred to as the calculated elastic maximum load and moment. Also, the calculated maximum load and moment of a splice obtained by using the recommended force distribution, along with nonlinear analysis of the web bolts, will be referred to as the calculated nonlinear maximum load and moment.

The calculated elastic and nonlinear maximum loads for splice 17u were determined using Case 1 of force distribution. For splice 15u, the calculated elastic maximum load was determined using Case 2 of force distribution; whereas,

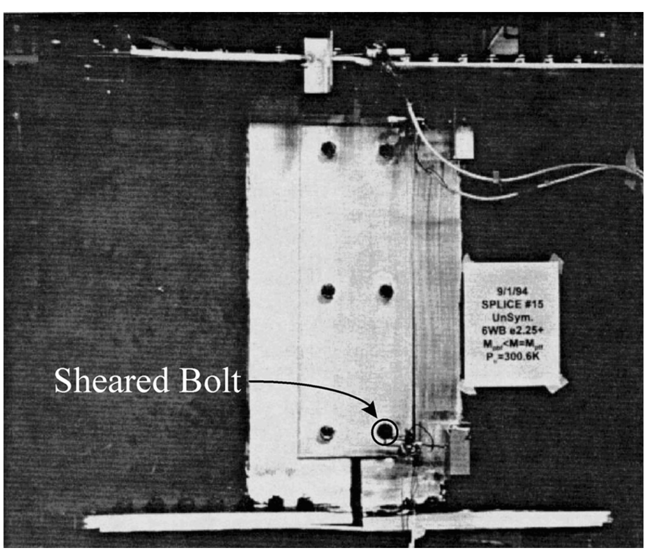

Fig. 8. Typical failed unsymmetric beam splice with developed smaller flange plate and undeveloped larger flange plate (splice 15u).

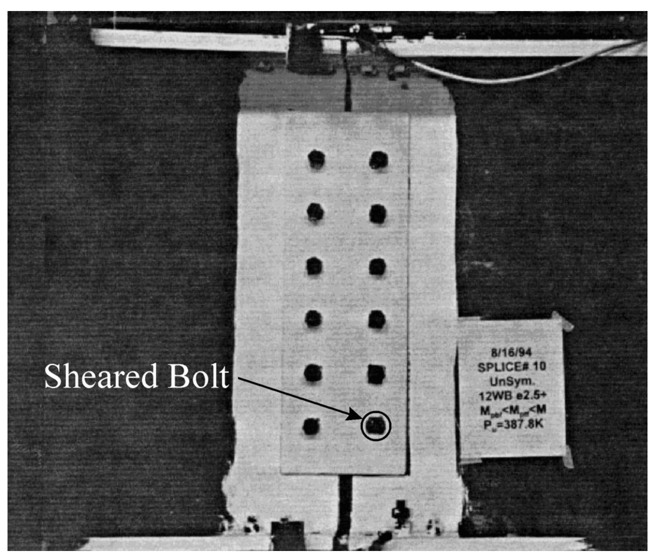

Fig. 9. Typical failed unsymmetric beam splice with developed flange plates (splice 10u).

the calculated nonlinear maximum load was determined using Case 3 of force distribution. Finally, the calculated elastic and nonlinear maximum loads for splices 7u, 8u, 9u, 10u, and 19u were determined using Case 3 of force distribution. The maximum difference between the experimental and calculated nonlinear splice capacities occurred for splices 9u and 10u because the bottom flange plate strain hardened and reached its tensile strength at the net section. Conversely, the smallest difference occurred for splices 7u and 8u because the effect of strain hardening was eliminated by not connecting the bottom flanges.

The calculated elastic maximum load was conservative for all splices. On the other hand, the calculated nonlinear maximum load was conservative for splices 9u, 10u, 15u, 17u, and 19u (having their bottom flange connected), and unconservative for splices 7u and 8u (having their bottom flanges not connected). Nonetheless, the unconservativeness for the latter splices was practically negligible. Further, since actual beams have both of their flanges spliced, the maximum load estimate for such splices is expected to be conservative, due to flange strain hardening.

The calculated elastic and nonlinear maximum loads agreed well with test results, as shown in Table 5, Figure 10, and Figure 11. The difference between the experimental and calculated nonlinear maximum loads ranged from +4.9 percent to -15.3 percent. In which, a positive value indicates an

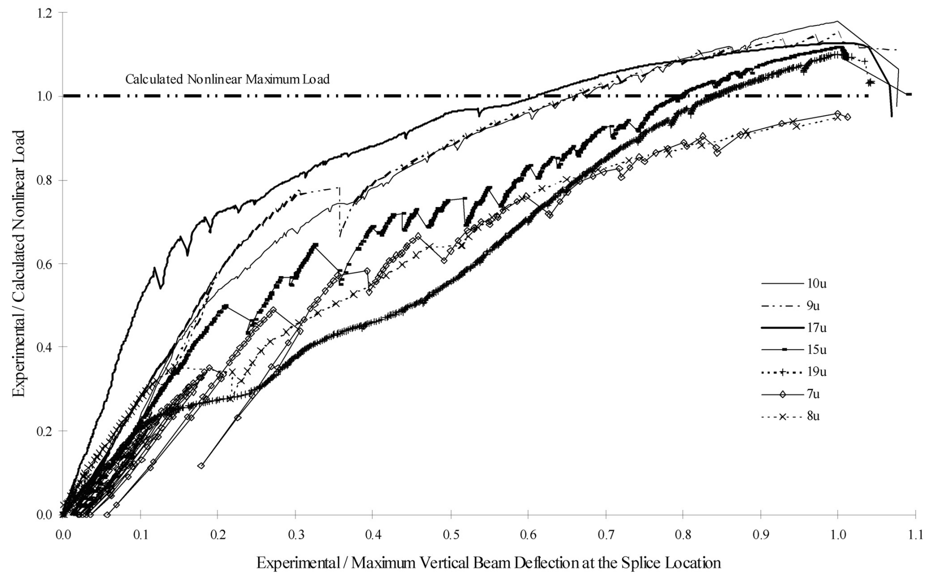

Fig. 10. Load-deflection response of spliced beam specimens normalized with respect to the calculated nonlinear maximum load as well as maximum vertical beam deflection at the splice location.

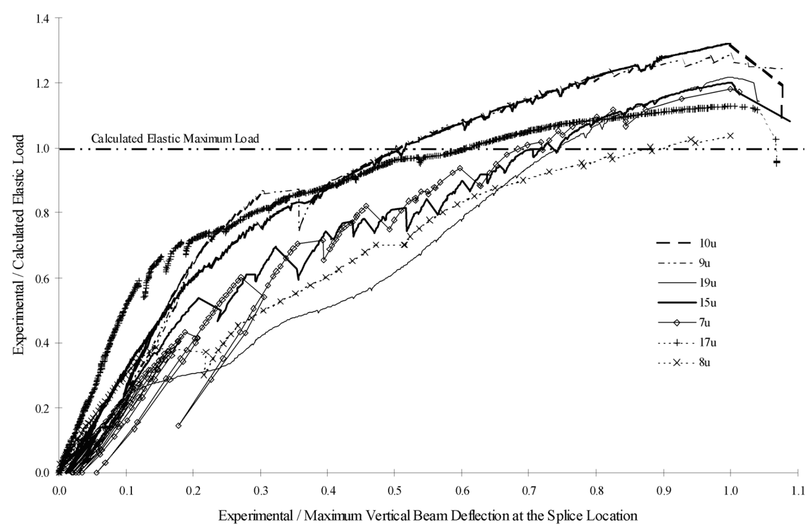

Fig. 11. Load-deflection response of spliced beam specimens normalized with respect to the calculated elastic maximum load as well as maximum vertical beam deflection at the splice location.

Table 6. The Calculated Maximum Moments Normalized With Respect to the Calculated Moment Resistances of the Small and Large Flange Plates

Splice	$M_{elastic}/M_f$	$M_{elastic}/M_F$	$M_{nonlinear}/M_f$	$M_{nonlinear}/M_F$
7u	∞	1.88	∞	2.33
8u	∞	0.96	∞	1.05
9u	2.19	1.17	2.46	1.31
10u	2.19	1.17	2.46	1.31
15u	1.88	1.00	2.03	1.08
17u	0.98	0.52	0.98	0.52
19u	2.12	1.13	2.34	1.25

unconservative prediction, and a negative value indicates a conservative prediction. The difference between the experimental and the calculated elastic maximum loads ranged from -4.0 percent to -24.4 percent. The average difference between the experimental and the calculated nonlinear maximum loads was 9.9 percent; while that between the experimental and the calculated elastic maximum loads was 16.1 percent. Therefore, the nonlinear maximum load estimates agree best with the test results. However, the nonlinear load-deformation responses for all commonly used bolts and plates are not available, and costly to obtain. On the other hand, the elastic maximum load estimates are adequate and there is no need to perform a nonlinear analysis on the web bolts.

The validity of the recommended force distribution was verified for a wide range of maximum moments. As shown in Table 6, the calculated elastic maximum moments normalized with respect to the moment required to develop the smaller flange plate ranged from 0.98 (splice 17u) to ∞ (splices 7u and 8u in which the bottom flange was not connected). Also, the calculated maximum moment normalized with respect to the moment required to develop the larger flange plate ranged from 0.52 (splice 17u) to 1.88 (splice 7u) when using elastic analysis, and from 0.52 to 2.33 when using nonlinear analysis.

The Effect of the Web Bolt Preload on the Strength of Beam Splices

To examine the effect of the web bolt preload on the strength of beam splices, specimen 10u was replicated in specimen 19u. However, the web bolts in specimen 19u were hand-tight. The normalized load-deflection responses for both specimens are shown in Figure 12. At the splice location, the beam with hand-tight web bolts underwent approximately 55 percent more vertical deflection than did the one with fully pretensioned bolts. Also, the maximum load for the splice with hand-tight web bolts was 12 percent less than that for the fully pretensioned splice. However, since the shear strength of an individual bolt was 12 percent less than that of a fully pretensioned bolt, it is believed that the reduction in splice strength was mainly due to the lower shear strength of hand-tight bolts.

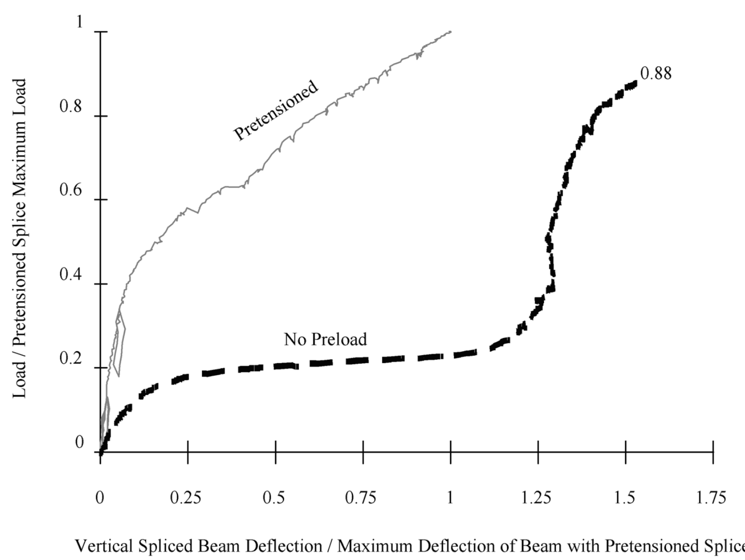

Fig. 12. Normalized load-deflection response of spliced beam specimens with fully-pretensioned and hand-tight web bolts.

DISCUSSION OF ANALYTICAL METHODS

Three cases of force distribution between the web and flanges of unsymmetric beams were presented. Generally, a splice can be designed according to any of the three cases. For example, even though the flanges might be able to resist the total moment, an engineer could design the web for a portion of the total moment, as required by AASHTO. Due to the effect of the additional eccentric effect on the web bolts, this will increase the number of bolts significantly. Therefore, to produce cost-effective splices, and to simplify construction, the AISC approach should be followed.

Given that an engineer chooses to follow the AASHTO approach, the web must be designed for any unbalanced flange axial forces to satisfy force equilibrium. Also, to avoid premature flange failure, the authors of this paper recommend that flange splices be designed for the smaller of the total applied moment, or the moment required to develop the flanges. As an alternate to developing the full moment in the flanges, deformation compatibility must be enforced, as explained by Sheikh-Ibrahim and Frank (1998). However, this is a lengthy, uneconomical procedure. On the other hand, the AISC approach is simple and economical. The only two modifications that need to be made to the AISC approach are for the cases when the moment exceeds that required to develop either flange. For both cases, the smaller flange splice should be designed to develop the axial resistance of that flange. The web and larger flange splices should be designed for the moment in excess of that required to develop the smaller flange, as explained earlier in Case 2 and Case 3 of force distribution. Finally, the web splice should be designed for the shear, moment due to the eccentricity of the shear, moment in excess of that required to develop the larger flange, and any unbalanced flange axial forces. For symmetric web splices, the eccentricity of the shear is defined as the distance from the centerline of the splice to the centroid of the web bolts under consideration.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the analytical and testing program presented in this paper, the following conclusions and recommendations are made to reduce the total number of bolts, prevent premature flange failure, and to simplify the design process:

- When the maximum design moment is less than that required to develop the axial resistance of the smaller flange, the AISC-LRFD method is in agreement with the test results, and should be used to obtain cost-effective splices. The flange splices should be designed to carry the total moment through a tension-compression couple acting at the centroids of the top and bottom flanges. The web splice should be designed for the shear and moment due to the eccentricity of the shear.
- When the maximum design moment exceeds that required to develop the axial resistance of the smaller flange, but is less than that required to develop the axial resistance of the larger flange, the web splice should be designed for the shear including the eccentric effect, and any unbalanced flange axial forces applied at mid-height of the web. The smaller flange splice should be designed to develop the axial resistance of the smaller beam flange. The larger flange splice should be designed for the axial resistance of the smaller beam flange, as well as the axial force caused by the moment in excess of that required to develop the smaller flange. This axial force is calculated as the force caused by a tension-compression couple acting at the centroid of the larger flange and mid-height of the web.
- When the maximum design moment exceeds that required to develop the axial resistance of the larger flange, the web splice should be designed for the shear including the eccentric effect, any unbalanced flange axial forces applied at mid-height of the web, and moment in excess of that required to develop the larger flange. Each flange splice should be designed to develop the axial resistance of the corresponding beam flange.
- For symmetric web splices, the eccentricity of the shear is defined as the distance from the splice centerline to the centroid of the web bolts under consideration.
- Elastic analysis of the web bolts is adequate and there is no need to perform nonlinear analysis on the bolts.
- Beam splices with hand-tight web bolts (no preload) undergo more deformation and have less capacity than those with fully pretensioned web bolts. The reduction in strength is mainly due to the lower shear strength of individual hand-tight bolts. Therefore, beam splice bolts should preferably be fully pretensioned unless large deformation is of no concern to the engineer.

AISC-LRFD DESIGN EXAMPLES

Design Example 1: (Undeveloped Flanges)

Design a bolted splice for an A572 Gr. 50 beam. The beam has a 14-in.×¾-in. top flange, 14-in.×1½-in. bottom flange, and 70-in.×⅞-in. web. Use ⅞-in. diameter A325-X bolts and A572 Gr. 50 plates. The factored shear and moment at the centerline of the splice are V_u = 850 kips and M_u = -25,000 in.-kips.

Solution:

Flange Splice Design:

Determine the axial flange force assuming the flanges are undeveloped (this assumption will be checked later):

$$P_{uf} = P_{uF} = \frac{M_u}{\bar{h}} = \frac{M_u}{h + \frac{t_f}{2} + \frac{t_F}{2}} = \frac{25{,}000}{70 + \frac{0.75}{2} + \frac{1.5}{2}} = 351.5 \text{ kips}$$

Determine the axial design strength of the smaller flange (top flange in tension): Based on tension yielding of flange:

$$\begin{aligned}\phi R_n &= \phi F_y A_g = 0.9 \times 50 \times (14 \times 0.75) \\ &= 0.9 \times 50 \times 10.5 = 472.5 \text{ kips}\end{aligned}$$

Based on tension rupture of flange (assuming four rows of bolts):

$$\begin{aligned}\phi R_n &= \phi F_u A_n = 0.75 \times 65 \times \left[10.5 - 4\left(\tfrac{7}{8} + \tfrac{1}{8}\right) \times 0.75\right] \\ &= 0.75 \times 65 \times 7.5 = 365.6 \text{ kips}\end{aligned}$$

Determine whether the flanges are developed or undeveloped:

$$P_{uf} = 351.5 \text{ kips} < \phi R_n = 365.6 \text{ kips}$$

Therefore, the flanges are undeveloped and are capable of resisting the factored design moment, as shown in Figure 13a.

Determine the number of ⅞-in. flange bolts required for shear:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{351.5}{27.1} = 13 \text{ bolts}$$

where ϕr_n=27.1 kips is the single shear design strength as given in Table 8-11 (AISC, 1994).

Determine the number of ⅞-in. flange bolts required for material bearing on the beam flange using PL ¾ in. × 14 in. × 24½ in., L_e = 1½ in. > 1.5d, and s = 3 in.:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{351.5}{102 \times 0.75} = 4.6 \approx 5 \text{ bolts}$$

where ϕr_n=102 kip/in. thickness as given in Table 8-13 (AISC, 1994).

Therefore, bolt shear is more critical than material bearing. Try four rows of four bolts at 3-in. spacing, 5-in. center gage, and using 1½-in. end and edge distances.

Check tension yielding of flange plate:

$$\begin{aligned}\phi R_n &= \phi F_y A_g \\ &= 0.9 \times 50 \times 0.75 \times 14 = 472.5 \text{ kips} > 351.5 \text{ kips} \quad \text{o.k.}\end{aligned}$$

Check tension rupture of flange plate:

$$\begin{aligned}\phi R_n &= \phi_t F_u A_n = 0.75 \times 65 \times \left[10.5 - 4\left(\tfrac{7}{8} + \tfrac{1}{8}\right) \times 0.75\right] \\ &= 365.6 \text{ kips} > 351.5 \text{ kips} \quad \text{o.k.}\end{aligned}$$

Check block shear rupture of tension flange and flange plate:
There are four cases for which block shear must be checked. The first case involves the rupture of the two blocks outside the interior two rows of bolt holes in the flange plate. The second case involves the rupture of the block inside the two exterior rows of bolt holes. The third case involves the rupture of one block which is bounded by one of the exterior two rows of bolt holes in the flange plate and the edge of the flange plate farthest from the row of bolt holes under consideration. Finally, the fourth case involves the rupture of two blocks of the beam flange located outside the interior two rows of bolt holes. One can show that for this example,

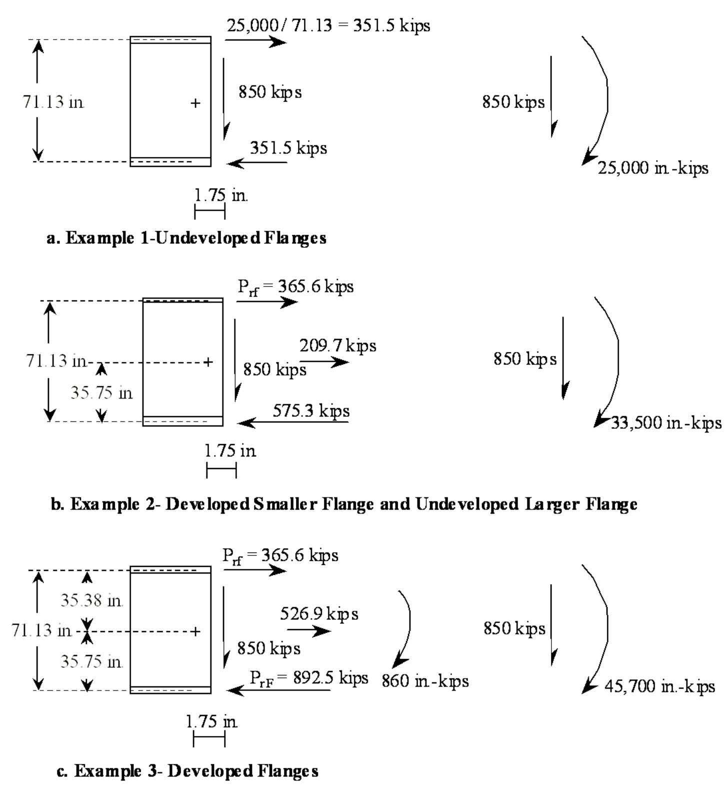

Fig. 13. The force distribution for the three design examples.

the block shear rupture strength of the third case controls and is determined as follows:

$$0.6F_uA_{nv} = 0.6\times65\times\left[1.5+3\times3-3.5\times\left(\tfrac{7}{8}+\tfrac{1}{8}\right)\right]\times0.75$$
$$= 204.8 \text{ kips}$$
$$F_uA_{nt} = 65\times\left[12.5-3.5\times\left(\tfrac{7}{8}+\tfrac{1}{8}\right)\right]\times0.75 = 438.8 \text{ kips}$$

Since $0.6F_uA_{nv} < F_uA_{nt}$, use:

$$\phi R_n = \phi\left[F_uA_{nt}+0.6F_yA_{gv}\right]$$
$$\phi R_n = 0.75\left[439+0.6\times50\times10.5\times0.75\right]$$
$$= 506.3 \text{ kips} \;>\; 351.5 \text{ kips} \quad \text{o.k.}$$

Check the design compressive strength of flange plate:
Determine the design compressive strength of the flange plate assuming $K = 0.65$ and $l = 3.5$ in. (1½-in. edge distance on each side of the splice and ½-in. clearance gap between spliced beam segments):

$$\frac{K\ell}{r} = \frac{0.65\times3.5}{\sqrt{\dfrac{\left(14\times0.75^3\right)/12}{\left(14\times0.75\right)}}} = 10.5$$

From AISC-LRFD Specifications Table 3-36 with $K\ell/r = 10.5$, $\phi F_{cr} = 42.2$ ksi and the design compressive strength of the flange plate is:

$$\phi R_n = \phi F_{cr}A_g = 42.2\times\left(14\times0.75\right)$$
$$= 443.1 > 351.5 \text{ kips} \quad \text{o.k.}$$

Use PL ¾ in. × 14 in. × 24½ in. and four rows of four ⅞-in. A325-X bolts at 3-in. spacing, 5-in. center gage, and 1½-in. end and edge distances.

Web Splice Design:
Try 2-PL 5⁄16 in. × 6½ in. × 63 in. and one row of sixteen ⅞-in. A325-X bolts at 4-in. spacing on each side of the splice.

Determine the controlling design strength per one bolt:

$$\phi r_n = \text{minimum of}\begin{Bmatrix}\text{double shear strength of one bolt}\\ \text{bearing strength at one hole edge}\end{Bmatrix}$$
$$= \begin{Bmatrix}\phi r_n = 54.1 \text{ kips}\\ \phi r_n = 102\times0.625 = 63.8 \text{ kips}\end{Bmatrix}$$

Therefore, bolt shear controls the design of the web bolts and $\phi r_n = 54.1$ kips.

Check the design strength of web bolts using the elastic vector method and assuming $e = 1.75$ in. (1½-in. edge distance and ¼-in. which is half of the clearance gap between spliced beam segments):

Direct shear force per bolt:

$$r_1 = \frac{V_u}{n} = \frac{850}{16} = 53.1 \text{ kips}$$

Additional shear force on critical bolt due to eccentricity:

$$r_2 = \frac{V_ueC_y}{I_p} = \frac{850\times1.75\times30}{5{,}440} = \frac{1{,}488\times30}{5{,}440} = 8.2 \text{ kips}$$

where I_p can be calculated as:

$$I_p = \frac{mn}{12}\left[s^2\left(n^2-1\right)+g^2\left(m^2-1\right)\right]$$
$$= \frac{16}{12}\left[4^2\left(16^2-1\right)+0\right] = 5{,}440$$

in which m and n are the number of bolts in a horizontal and vertical row, respectively; s and g are the vertical and horizontal spacing of the web bolts, respectively.

Resultant shear force on critical bolt:

$$r_u = \sqrt{r_1^2+r_2^2} = \sqrt{53.1^2+8.2^2}$$
$$= 53.7 \text{ kips} \;<\; \phi r_n = 54.1 \text{ kips} \quad \text{o.k.}$$

Check flexural yielding of web plates:

$$\phi M_n = \phi F_yS_x$$
$$\phi M_n = 0.9\times50\times\frac{0.625\times63^2}{6}$$
$$= 18{,}605 \text{ in.-kips} \;>\; V_ue = 1{,}488 \text{ in.-kips} \quad \text{o.k.}$$

Check flexural rupture of web plates:

$$\phi M_n = \phi F_uS_{net}$$
$$\text{where } S_{net} = \frac{t}{6}\left[d^2-\frac{s^2n\left(n^2-1\right)\left(d_b+0.125\right)}{d}\right]$$

as specified in Table 12-1 (AISC, 1994).

$$S_{net} = \frac{0.625}{6}\left[63^2-\frac{4^2\times16\left(16^2-1\right)\left(\tfrac{7}{8}+0.125\right)}{63}\right] = 305.5 \text{ in.}^3$$
$$\phi M_n = 0.75\times65\times305.5$$
$$= 14{,}893 \text{ in.-kips} > 1{,}488 \text{ in.-kips} \quad \text{o.k.}$$

Check shear yielding of web plates:

$$\phi R_n = \phi\left(0.6F_yA_g\right)$$
$$\phi R_n = 0.9\times0.6\times50\times\left(0.625\times63\right)$$
$$= 1{,}063 \text{ kips} \;>\; V_u = 850 \text{ kips} \quad \text{o.k.}$$

Check shear rupture of web plates:

$$\phi R_n = \phi(0.6F_u A_n)$$
$$\phi R_n = 0.75 \times 0.6 \times 65 \times 0.625 \times (63 - 16 \times 1)$$
$$= 859 \text{ kips} > V_u = 850 \text{ kips} \quad \text{o.k.}$$

Check block shear rupture of web plates:

$$0.6F_u A_{nv} = 0.6 \times 65 \times \left(1.5 + 15 \times 4 - 15.5 \times \left(\tfrac{7}{8} + \tfrac{1}{8}\right)\right) \times 0.625$$
$$= 1{,}121 \text{ kips}$$
$$F_u A_{nt} = 65 \times \left(1.5 - 0.5 \times \left(\tfrac{7}{8} + \tfrac{1}{8}\right)\right) \times 0.625 = 40.6 \text{ kips}$$

Since $0.6F_uA_{nv} > F_uA_{nt}$, use:

$$\phi R_n = \phi\left[0.6F_u A_{nv} + F_y A_{gt}\right]$$
$$\phi R_n = 0.75\left[1{,}121 + 50 \times 1.5 \times 0.625\right]$$
$$= 876 \text{ kips} > 850 \text{ kips o.k.}$$

Use 2-PL 5⁄16 in. × 6½ in. × 63 in. and one row of sixteen 7⁄8-in. A325-X bolts at 4-in. spacing on each side of the splice.

Design Example 2: (Undeveloped Larger Flange and Developed Smaller Flange)

Redesign the bolted splice in Example 1 to resist a factored splice moment of M_u = -33,500 in.-kips.

Solution:

Top Flange Splice Design:

Determine the axial flange force assuming the flanges are undeveloped (this assumption will be checked later):

$$P_{uf} = \frac{M_u}{h + \frac{t_f}{2} + \frac{t_F}{2}} = \frac{33{,}500}{70 + \frac{0.75}{2} + \frac{1.5}{2}} = 471 \text{ kips}$$

Check the axial design strength of the smaller tension flange (determined in Example 1):

Based on tension yielding of flange:

$$\phi R_n = 472.5 \text{ kips} > 471 \text{ kips} \quad \text{o.k.}$$

Based on tension rupture of flange:

$$\phi R_n = 365.6 \text{ kips} < 471 \text{ kips} \quad \text{N.G.}$$

Therefore, the smaller flange is developed. Determine the smaller flange design force as the axial design strength of that flange:

$$P_{uf} = 365.6 \text{ kips}$$

Determine the number of 7⁄8-in. top flange bolts required for shear:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{365.6}{27.1} = 13.5 \approx 14 \text{ bolts}$$

Determine the number of 7⁄8-in. top flange bolts required for material bearing on the beam flange using PL ¾ in. × 14 in. × 24½ in., L_e = 1½-in. > 1.5d, and s = 3 in.:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{365.6}{102 \times 0.75} = 4.8 \approx 5 \text{ bolts}$$

Therefore, bolt shear is more critical than material bearing. Try four rows of four bolts at 3-in. spacing, 5-in. center gage, and using 1½-in. end and edge distances.

Check tension yielding of top flange plate (determined in Example 1):

$$\phi R_n = 472.5 \text{ kips} > P_{uf} = 365.6 \text{ kips} \quad \text{o.k.}$$

Check tension rupture of top flange plate (determined in Example 1):

$$\phi R_n = 365.6 \text{ kips} \equiv P_{uf} = 365.6 \text{ kips} \quad \text{o.k.}$$

Check block shear rupture of top tension flange and flange plate (determined in Example 1):

$$\phi R_n = 506.3 \text{ kips} > P_{uf} = 365.6 \text{ kips} \quad \text{o.k.}$$

Therefore for the top flange splice, use PL ¾ in. × 14 in. × 24½ in. and four rows of four bolts at 3-in. spacing, 5-in. center gage, and 1½-in. end and edge distances.

Bottom Flange Splice Design:

Determine the larger flange design force assuming developed smaller flange and undeveloped larger flange (this assumption will be checked later):

$$P_{uF} = P_{rf} + \frac{M_u - P_{rf}\bar{h}}{h_F}$$
$$= 365.6 + \frac{33{,}500 - 365.6 \times (0.375 + 70 + 0.75)}{35 + 0.75}$$
$$= 575.3 \text{ kips}$$

Determine the axial design strength of the larger flange (bottom flange in compression):

$$\phi R_n = \phi F_{cr} A_g = 0.85 \times 50 \times (14 \times 1.5)$$
$$= 0.85 \times 50 \times 21 = 892.5 \text{ kips} > 575.3 \text{ kips}$$

Therefore, the assumption that the larger flange is undeveloped is correct, and the force distribution at the centerline of the splice is as shown in Figure 13b.

Determine the number of ⅞-in. bottom flange bolts required for shear:

$$n_{\min} = \frac{P_{uF}}{\phi r_n} = \frac{575.3}{27.1} = 21.2 \approx 22 \text{ bolts}$$

Determine the number of ⅞-in. top flange bolts required for material bearing on the beam bottom flange plate using PL 1 in. × 14 in. × 36½ in., L_e =1½ in. > 1.5d, and s = 3 in.:

$$n_{\min} = \frac{P_{uF}}{\phi r_n} = \frac{575.3}{102 \times 1} = 5.6 \approx 6 \text{ bolts}$$

Therefore, bolt shear is more critical than material bearing. Try four rows of six bolts at 3-in. spacing, 5-in. center gage, and using 1½-in. end and edge distances.

Check the design compressive strength of the bottom flange plate:
Determine the design compressive strength of the bottom flange plate assuming K = 0.65 and l =3.5 in. (1½-in. edge distance on each side of the splice and ½-in. clearance gap between spliced beam segments):

$$\frac{K\ell}{r} = \frac{0.65 \times 3.5}{\sqrt{\dfrac{\left(14 \times 1^3\right)/12}{\left(14 \times 1\right)}}} = 7.9$$

From AISC LRFD Specifications Table 3-36 with $K\ell / r$ = 7.9, ϕF_{cr} = 42.3 ksi and the design compressive strength of the flange plate is:

$$\phi R_n = \phi F_{cr} A_g = 42.3 \times \left(14 \times 1\right) = 592.2 > 575.3 \text{ kips} \quad \text{o.k.}$$

Use PL 1 in. × 14 in. × 36½ in. and four rows of six ⅞-in. A325-X bolts at 3-in. spacing, 5-in. center gage, and 1½-in. end and edge distances.

Web Splice Design:
Determine the horizontal force to be resisted by the web plates at the mid-depth of the web:

$$H_{uw} = P_{uF} - P_{uf} = 892.5 - 365.6 = 526.9 \text{ kips}$$

Try 2-PL ⅜ in. × 6½ in. × 59 in. and one row of seventeen ⅞-in. A325-X bolts at 3½-in. spacing on each side of the splice.

Determine the controlling design strength per one bolt:

$$\phi r_n = \text{minimum of} \begin{Bmatrix} \text{double shear strength of one bolt} \\ \text{bearing strength at one hole edge} \end{Bmatrix}$$

$$= \text{minimum of} \begin{Bmatrix} \phi r_n = 54.1 \text{ kips} \\ \phi r_n = 102 \times 0.75 = 76.5 \text{ kips} \end{Bmatrix}$$

Therefore, bolt shear controls the design of the web bolts and ϕr_n = 54.1 kips.

Check the design strength of web bolts using the elastic vector method and assuming e = 1.75 in. (1½-in. edge distance and ¼-in. which is half of the clearance gap between spliced beam segments):

Direct vertical shear force per bolt:

$$r_1 = \frac{V_u}{n} = \frac{850}{17} = 50 \text{ kips}$$

Direct horizontal shear force per bolt:

$$r_2 = \frac{H_{uw}}{n} = \frac{526.9}{21} = 25.1 \text{ kips}$$

Additional horizontal shear force on critical bolt due to eccentricity:

$$r_3 = \frac{V_u e C_y}{I_p} = \frac{850 \times 1.75 \times 28}{\frac{17}{12}\left[3.5^2\left(17^2 - 1\right)\right]} = \frac{1{,}488 \times 28}{4{,}998} = 8.3 \text{ kips}$$

Resultant shear force on critical bolt:

$$r_u = \sqrt{r_1^2 + \left(r_2 + r_3\right)^2} = \sqrt{50^2 + \left(12.3 + 8.3\right)^2}$$
$$= 54.1 \text{ kips} \equiv \phi r_n = 54.1 \text{ kips} \quad \text{o.k.}$$

Check flexural yielding of web plates:

$$\phi M_n = \left(\phi F_y - \frac{H_{uw}}{A_{gw}}\right) S_x$$

$$\phi M_n = \left(0.9 \times 50 - \frac{209.7}{0.75 \times 59}\right) \times \frac{0.75 \times 59^2}{6}$$
$$= 17{,}519 \text{ in.-kips} > 1{,}488 \text{ in.-kips} \quad \text{o.k.}$$

Check flexural rupture of web plates:

$$\phi M_n = \left(\phi F_u - \frac{H_{uw}}{A_{nw}}\right) S_{net}$$

$$\text{where } S_{net} = \frac{0.75}{6}\left[59^2 - \frac{3.5^2 \times 17\left(17^2 - 1\right)\left(\frac{7}{8} + 0.125\right)}{59}\right]$$
$$= 308.1 \text{ in.}^3$$

as specified in Table 12-1 (AISC, 1994).

$$\phi M_n = \left(0.75 \times 65 - \frac{209.7}{0.75 \times \left(59 - 17 \times 1\right)}\right) \times 308.1$$
$$= 12{,}969 \text{ in.-kips} > 1{,}488 \text{ in.-kips} \quad \text{o.k.}$$

Check shear yielding of web plates:

$$\begin{aligned}\phi R_n &= 0.9\times 0.6\times 50\times\left(0.75\times 59\right)\\ &= 1{,}195 \text{ kips} > V_u = 850 \text{ kips o.k.}\end{aligned}$$

Check shear rupture of web plates:

$$\begin{aligned}\phi R_n &= 0.75\times 0.6\times 65\times 0.75\times\left(59-17\times 1\right)\\ &= 921.4 \text{ kips} > V_u = 850 \text{ kips o.k.}\end{aligned}$$

Check block shear rupture of web plates:

$$\begin{aligned}0.6F_uA_{nv} &= 0.6\times 65\times\left(1.5+16\times 3.5-16.5\times\left(\tfrac{7}{8}+\tfrac{1}{8}\right)\right)\times 0.75\\ &= 1{,}199 \text{ kips}\\ F_uA_{nt} &= 65\times\left(1.5-0.5\times\left(\tfrac{7}{8}+\tfrac{1}{8}\right)\right)\times 0.75\\ &= 48.8 \text{ kips}\end{aligned}$$

Since $0.6F_uA_{nv} > F_uA_{nt}$, use:

$$\begin{aligned}\phi R_n &= 0.75\left[1{,}199+50\times 1.5\times 0.75\right]\\ &= 941.4 \text{ kips} > 850 \text{ kips o.k.}\end{aligned}$$

Use 2-PL ⅜ in. × 6½ in. × 59 in. and one row of seventeen ⅞-in. A325-X bolts at 3½-in. spacing on each side of the splice.

Design Example 3 (Developed Flanges)

Redesign the bolted splice in Example 1 to resist a factored splice moment of M_u = –45,700 in.-kips.

Solution:

Top Flange Splice Design:

Determine the smaller flange design force as the axial design strength of that flange (assuming that the smaller flange is developed):

$$P_{uf} = 365.6 \text{ kips}$$

As determined in Example 2, use PL ¾ in. × 14 in.× 24½ in. and four rows of four bolts at 3-in. spacing, 5-in. center gage, and 1½-in. end and edge distances.

Bottom Flange Splice Design:

Determine the larger flange design force assuming undeveloped larger flange (this assumption will be checked later):

$$\begin{aligned}P_{uF} &= P_{rf}+\frac{M_u-P_{rf}\bar{h}}{h_F}\\ &= 365.6+\frac{45{,}700-365.6\times(0.375+70+0.75)}{\left(35+0.75\right)}\\ &= 916.6 \text{ kips}\end{aligned}$$

Determine the axial design compression strength of the larger flange (determined in Example 2):

$$\phi R_n = 892.5 \text{ kips} < 916.6 \text{ kips}$$

Therefore, both flanges are developed, and the force distribution at the centerline of the splice is as shown in Figure 13c.

Determine the larger flange design force as the axial design strength of that flange:

$$P_{uf} = 892.5 \text{ kips}$$

Determine the number of ⅞-in. bottom flange bolts required for shear:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{892.5}{27.1} = 32.9 \approx 33 \text{ bolts}$$

Determine the number of ⅞-in. bottom flange bolts required for material bearing on the beam flange plate using PL 1½ in. × 14 in. × 54½ in., L_e =1½ in. >1.5d, and s = 3 in.:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{892.5}{102\times 1.5} = 5.8 \approx 6 \text{ bolts}$$

Therefore, bolt shear is more critical than material bearing. Try four rows of nine bolts at 3-in. spacing, 5-in. center gage, and using 1½-in. end and edge distances.

Check the design compressive strength of the bottom flange plate:

Check the design compressive strength of the bottom flange plate assuming K = 0.65 and l = 3.5 in. (1½-in. edge distance on each side of the splice and ½-in. clearance gap between spliced beam segments):

$$\frac{K\ell}{r} = \frac{0.65\times 3.5}{\sqrt{\dfrac{\left(14\times 1.5^3\right)/12}{\left(14\times 1.5\right)}}} = 5.3$$

From AISC-LRFD Specifications Table 3-36 with $K\ell/r$ = 5.3, ϕF_{cr} = 42.4 ksi and the design compressive strength of the flange plate is:

$$\begin{aligned}\phi R_n &= \phi F_{cr}A_g = 42.4\times\left(14\times 1.5\right)\\ &= 890.4 \approx P_{uF} = 892.5 \text{ kips o.k.}\end{aligned}$$

Use PL 1½ in. × 14 in. × 54½ in. and four rows of nine ⅞-in. A325-X bolts at 3-in. spacing, 5-in. center gage, and 1½-in. end and edge distances.

Web Splice Design:
Determine the horizontal force to be resisted by the web plates at the mid-depth of the web:

$$H_{uw} = P_{uF} - P_{uf} = 892.5 - 365.6 = 526.9 \text{ kips}$$

Determine the moment to be resisted by the web plates at the centerline of the splice:

$$M_{uw} = M_u - \left(P_{rf} h_f + P_{rF} h_F\right)$$
$$= 45,700 - (365.6 \times 35.38 + 892.5 \times 35.75) = 860 \text{ in.-kips}$$

Try 2-PL ⅜ in. × 6½ in. × 63 in. and one row of twenty-one ⅞-in. A325-X bolts at 3-in. spacing on each side of the splice.

Determine the controlling design strength per one bolt:

$$\phi r_n = \text{minimum of} \begin{Bmatrix} \text{double shear strength of one bolt} \\ \text{bearing strength at one hole edge} \end{Bmatrix}$$
$$= \text{minimum of} \begin{Bmatrix} \phi r_n = 54.1 \text{ kips} \\ \phi r_n = 102 \times 0.75 = 76.5 \text{ kips} \end{Bmatrix}$$

Therefore, bolt shear controls the design of the web bolts and $\phi r_n = 54.1$ kips.

Check the design strength of web bolts using the elastic vector method and assuming $e = 1.75$ in. (1½-in. edge distance plus ¼-in. which is half of the clearance gap between spliced beam segments):

Direct vertical shear force per bolt:

$$r_1 = \frac{V_u}{n} = \frac{850}{21} = 40.5 \text{ kips}$$

Direct horizontal shear force per bolt:

$$r_2 = \frac{H_{uw}}{n} = \frac{526.9}{21} = 25.1 \text{ kips}$$

Additional horizontal shear force on critical bolt due to eccentricity:

$$r_3 = \frac{\left(V_u e + M_{uw}\right) C_y}{I_p} = \frac{(1,488 + 860) \times 30}{\frac{21}{12}\left[3^2\left(21^2 - 1\right)\right]}$$
$$= \frac{2,348 \times 30}{6,930} = 10.2 \text{ kips}$$

Resultant shear force on critical bolt:

$$r_u = \sqrt{r_1^2 + \left(r_2 + r_3\right)^2} = \sqrt{40.5^2 + (25.1 + 10.2)^2}$$
$$= 53.7 \text{ kips} < \phi r_n = 54.1 \text{ kips} \quad \text{o.k.}$$

Check flexural yielding of web plates:

$$\phi M_n = \left(\phi F_y - \frac{H_{uw}}{A_{gw}}\right) S_x$$
$$\phi M_n = \left(0.9 \times 50 - \frac{526.9}{0.75 \times 63}\right) \times \frac{0.75 \times 63^2}{6}$$
$$= 16,793 \text{ in.-kips} > 2,348 \text{ in.-kips} \quad \text{o.k.}$$

Check flexural rupture of web plates:

$$\phi M_n = \left(\phi F_u - \frac{H_{uw}}{A_{nw}}\right) S_{net}$$
$$\text{where } S_{net} = \frac{0.75}{6}\left[63^2 - \frac{3^2 \times 21\left(21^2 - 1\right)\left(\tfrac{7}{8} + 0.125\right)}{63}\right]$$

= 331 in.[3] as specified in Table 12-1 (AISC, 1994).

$$\phi M_n = \left(0.75 \times 65 - \frac{526.9}{0.75 \times (63 - 21 \times 1)}\right) \times 331$$
$$= 10,600 \text{ in.-kips} > 2,348 \text{ in.-kips} \quad \text{o.k.}$$

Check shear yielding of web plates:

$$\phi R_n = 0.9 \times 0.6 \times 50 \times (0.75 \times 63)$$
$$= 1,276 \text{ kips} > V_u = 850 \text{ kips} \quad \text{o.k.}$$

Check shear rupture of web plates:

$$\phi R_n = 0.75 \times 0.6 \times 65 \times 0.75 \times (63 - 21 \times 1)$$
$$= 921.4 \text{ kips} > V_u = 850 \text{ kips} \quad \text{o.k.}$$

Check block shear rupture of web plates:

$$0.6 F_u A_{nv} = 0.6 \times 65 \times \left(1.5 + 20 \times 3 - 20.5 \times \left(\tfrac{7}{8} + \tfrac{1}{8}\right)\right) \times 0.75$$
$$= 1,199 \text{ kips}$$
$$F_u A_{nt} = 65 \times \left(1.5 - 0.5 \times \left(\tfrac{7}{8} + \tfrac{1}{8}\right)\right) \times 0.75$$
$$= 48.8 \text{ kips}$$

Since $0.6F_uA_{nv} > F_uA_{nt}$, use:

$$\phi R_n = 0.75[1,199 + 50 \times 1.5 \times 0.75]$$
$$= 941.4 \text{ kips} > 850 \text{ kips} \quad \text{o.k.}$$

Use 2-PL ⅜ in. × 6½ in. × 63 in. and one row of twenty-one ⅞-in. A325-X bolts at 3-in. spacing on each side of the splice.

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NOMENCLATURE

b = vertical spacing of the web bolts

b_f, t_f = width and thickness of the smaller flange plate, respectively

b_F, t_F = width and thickness of the larger flange plate, respectively

D = horizontal spacing of the web bolts

e_g = distance from centerline of splice to centroid of web bolts on the side of the joint under consideration

E_f = edge distance for the bottom flange edge bolt at the south side of splice

Difference = difference between the experimental and calculated maximum loads. A positive value indicates an unconservative maximum load estimate. A negative error indicates a conservative maximum load estimate

G = gap clearance between the two spliced beams

$\bar{h}$ = depth between the centroid of the larger flange plate and that of the smaller flange plate

h_f, h_F = vertical distance between the mid-height of the web and the centroid of the smaller and larger flange plate, respectively

h = clear distance between the flanges

H_w = horizontal force resisted by the web splice; difference between the axial forces in the larger and smaller flange plates

m = number of web bolts in a horizontal row

M = moment applied at the centerline of the splice

M_f = moment required to develop the smaller flange plate

M_F = moment required to develop the larger flange plate

M_{test} = experimental maximum moment at the centerline of the splice

M_w = theoretical moment resisted by the web splice at the centerline of the splice

n = number of web bolts in a vertical row

$N_f \times R$ = number of bottom flange bolts per longitudinal row × number of longitudinal rows

$N_F \times R$ = number of top flange bolts per longitudinal row × number of longitudinal rows

$P_{elastic}$ = calculated maximum load based on the recommended force distribution along with elastic analysis of the web bolts

P_f = axial force in the smaller flange plate

P_F = axial force in the larger flange plate

$P_{nonlinear}$ = calculated maximum load based on the recommended force distribution along with nonlinear analysis of the web bolts

P_{rf} = axial resistance of the smaller flange plate

P_{rF} = axial resistance of the larger flange plate

P_{test} = experimental maximum load

t_f, t_F = thickness of the smaller and larger flange plate, respectively

V = shear applied at the centerline of the splice

V_{test} = experimental maximum shear at the centerline of the splice

V_w = theoretical shear resisted by the web splice at the centerline of the splice

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