The Ultimate Strength of Symmetric Beam Bolted Splices

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INTRODUCTION

Rolled beams or plate girders are often spliced for a variety of reasons, such as: the required full length may not be available from the mill, the designer may desire a change in the cross section of the beam, or the fabricator might find it economical to splice the beam at certain locations to reduce shipping lengths or lifting loads. Figure 1 shows a typical bolted beam splice.

A beam splice must transfer the shear and moment of the beam at the location of the splice. However, the exact distribution of shear and moment between the web and flange splices is not known. Figure 2 shows a free body diagram taken by cutting the splice through its centerline. Figure 2 shows that the flange splices carry tension or compression and possibly a portion of total shear. It also shows that the web splice carries predominantly the shear and possibly a portion of total moment at the splice.

Different design procedures are available for bolted beam splices. These different procedures result in a different number of web and flange bolts. For example, the AISC (American Institute of Steel Construction) in its *LRFD* (Load and Resistance Factor Design) *Manual of Steel Construction*[1] recommends that flange splices be designed for the total moment and web splices be designed for the shear and moment caused by the eccentricity of the shear. For symmetric splices, AISC defines the eccentricity as the distance from the centroid of the web bolts to the centerline of the splice. This eccentricity will be referred to from hereon as the geometric eccentricity, e_g. Unlike AISC, AASHTO[2] (American Association of State Highway and Transportation Officials) requires that web splices be designed for the shear, moment due to the eccentricity of the shear, and the portion of the design moment resisted by the web. Also, AASHTO requires that flange splices be designed for the portion of the design moment not resisted by the web. However, AASHTO gives no guidance on how to calculate the eccentricity of the shear or how to apportion the web and flange moments. The *Guide to Design Criteria for Bolted and Riveted Joints*,[3] referred to from hereon as the Guide, recommends that flange splices be designed for the total design moment and web splices be designed for the total shear that satisfies moment equilibrium at the web bolts center of rotation. As an alternative, the Guide recommends that web splices be designed for the shear and moment caused by the eccentricity of the shear. The Guide defines the eccentricity of the shear as the distance from the centroid of the bolts under consideration to the centroid of bolts on the other side of the joint, twice the geometric eccentricity recommended by AISC. Based on an experimental program performed on web splices located at the inflection point, Kulak and Green[4] recommend that bolt groups on each side of web splices located at the inflection point be designed for the total shear that acts at the centerline of the splice, and for the moment caused by the eccentricity of the shear. The eccentricity of the shear is the distance from the centerline of the splice to the centroid of the web bolts, same eccentricity given in Reference 1.

A limited number of experimental programs has been conducted to verify the previous design procedures. The only two experimental programs that can be found in the literature are the ones done by Garrelts and Madsen in 1941[5] and by Kulak and Green in 1990.[4] The first program was performed on riveted girder web splices where only the web was spliced.[5] At the centerline of the splice, the girder flanges were present but were not spliced. In that program, the splices were designed such that the girder failed before the splice. The

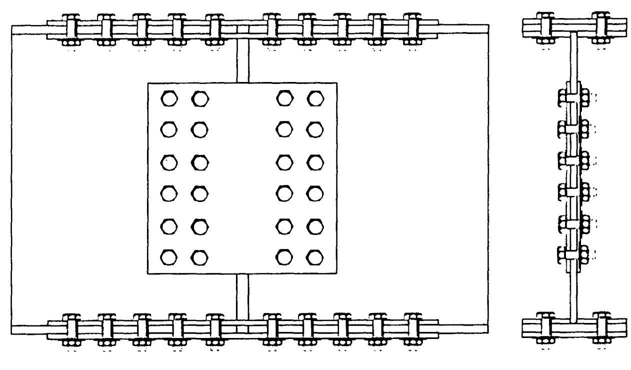

Fig. 1. Typical bolted beam splice.

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second experimental program was limited to bolted web splices located at the inflection point.[4]

In this paper, a summary of the analytical and experimental program performed by Sheikh-Ibrahim[6] on full-scale bolted beam splices will be presented. The tested splices were subjected to both moment and shear simultaneously. The program considered both symmetric and unsymmetric splices. It also considered both bearing-type and slip-critical splices. The paper presented herein will discuss the symmetric bearing-type splice tests and the analytical models developed to explain the test results.

ANALYTICAL PROGRAM

At the time the experimental program was conducted, it was not clear which of the available methods should be used to predict the strength of the tested specimens. After the first test was conducted, it was found that none of the available analytical methods predicted the strength of the splice with a reasonable accuracy. Therefore, it was felt necessary to develop analytical models to improve the predicted capacity of beam splices and to explain the experimental results. Thus, two analytical models for estimating the strength of symmetric beam splices at ultimate load were developed. The models are called the plastic model for splices with undeveloped flanges and the plastic model for splices with fully-developed flanges, respectively. As the name indicates, the plastic model for splices with undeveloped flanges estimates the capacity of beam splices in which the axial strength of the flange splices is not fully developed when subjected to the total applied moment. The plastic model for splices with fully-developed flanges estimates the capacity of beam splices in which the axial strength of the flange splices is fully developed.

THE PLASTIC MODEL FOR UNDEVELOPED FLANGES

In this model, the applied moment is assumed to be carried by a couple equal to the force in the flanges times the distance between the mid-thicknesses of the top and bottom flange splice plates; as given in Equation 1.

$$M = C \times h = T \times h \tag{1}$$

If the flange splice plates are not fully yielded when subjected to the total applied moment, the flanges are assumed to provide shear stiffness to the splice and hence contribute to the shear resistance of the splice. Thus at ultimate load, the flange splice plates are assumed to carry axial load, shear, and moment. The analytical plastic model for beam splices with undeveloped flanges is shown in Figure 3. In the model, the two girders on each side of the splice centerline are assumed to be infinitely rigid. The flange splice plates are depicted by the lines connecting M_{pc} points, and the web splice plates are depicted by the line connecting M_w points. The model assumes that two plastic hinges form in each of the flange splice plates. These plastic hinges are the M_{pc} points shown in Figure 3. The first plastic hinge is assumed to be located at the first row of flange bolts on one side of the splice because the flange splice plate lifts off of that location which coincides with the point of fixity. On the other side of the splice, the second plastic hinge is assumed to form at the edge of the beam flange since the splice plate is bent about that corner. The distance from the first row of bolts on one side of the flange splice to the beam edge on the other side of the splice will be referred to herein as the shear gap; L_f. Further, the web bolt groups on both sides of the joint are assumed to develop their eccentric shear capacities simultaneously. This means that an inflection point is assumed to be present at the centerline of the web splice and, more importantly, the eccentricity of the shear is constant and equal to one half the distance between the centroids of the two bolt groups. This eccentricity was defined earlier as the geometric eccentricity; e_g.

From the principle of virtual work, one can write:

$$V(\theta L_f) = 4M_{pc}\theta + 2M_w\left(\frac{\theta L_f}{2e_g}\right) \tag{2}$$

By rearranging Equation 2, one gets:

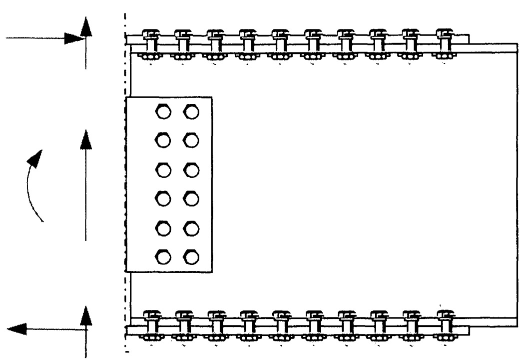

Fig. 2. A free body diagram taken by cutting the splice through its centerline.

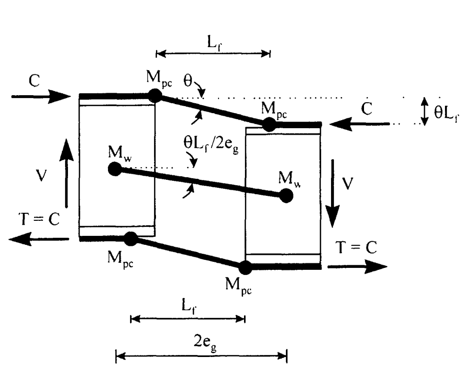

Fig. 3. The plastic model for beam splices with undeveloped flanges and equal shear gaps.

$$V = \frac{4M_{pc}}{L_f} + \frac{2M_w}{2e_g} \tag{3}$$

Figure 4 shows the idealized stress distribution in the bottom and top flange splice plates due to axial load and bending moment. From horizontal equilibrium of flange forces, one gets:

$$y_o = \frac{C}{F_{yf}b_f} = \frac{T}{F_{yf}b_f} \tag{4}$$

Defining y as:

$$y = \frac{t_f - y_o}{2} \tag{5}$$

and assuming that the ratio of axial load in the flanges to the total shear is constant and equal to α, one can write:

$$\alpha = \frac{C}{V} = \frac{T}{V} = \frac{M/h}{V} \tag{6}$$

Further, the plastic moment resistance of the cross section of each flange splice plate modified for the presence of axial load can be written as:

$$M_{pc} = F_{yf}b_f y(t_f - y) \tag{7}$$

By substituting Equations 4, 5, and 6 into Equation 7:

$$M_{pc} = F_{yf}b_f\left(\frac{t_f}{2} - \frac{\alpha V}{2F_{yf}b_f}\right)\left(t_f - \frac{t_f}{2} + \frac{\alpha V}{2F_{yf}b_f}\right) \tag{8}$$

and rearranging Equation 8, one gets:

$$M_{pc} = \frac{F_{yf}b_f}{4}\left[t_f^2 - \left(\frac{\alpha V}{F_{yf}b_f}\right)^2\right] \tag{9}$$

Thus:

$$M_{pc} = \left(\frac{b_f t_f^2}{4}\right)F_{yf} - \frac{\alpha^2V^2}{4F_{yf}b_f} \tag{10}$$

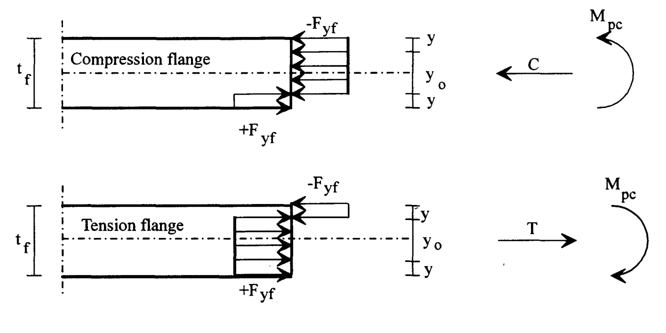

Fig. 4. The idealized stress distribution in the bottom and top flange splice plates for beam splice with undeveloped flanges.

Substituting Equation 10 into Equation 3:

$$V = \frac{b_f t_f^2 F_{yf}}{L_f} - \frac{\alpha^2V^2}{F_{yf}b_f L_f} + \frac{2M_w}{2e_g} \tag{11}$$

Substituting Equation 6 into Equation 11:

$$V = \frac{b_f t_f^2 F_{yf}}{L_f} - \frac{\left(\frac{M/h}{V}\right)^2 V^2}{F_{yf}b_f L_f} + \frac{V_w e_g}{e_g} \tag{12}$$

Defining M_f, the moment resistance of the flange splice plates:

$$M_f = \left[(b_f t_f)F_{yf}\right]h \tag{13}$$

Also, defining $\overline{M}_f$, the plastic moment resistance of the cross section of each flange splice plate in the absence of axial load, as follows:

$$\overline{M}_f = \left(\frac{b_f t_f^2}{4}\right)F_{yf} \tag{14}$$

By substituting Equations 13 and 14 into Equation 12, one gets:

$$V = \frac{4\overline{M}_f}{L_f}\left[1 - \left(\frac{M}{M_f}\right)^2\right] + V_w \tag{15}$$

Equation 15 represents the moment-shear interaction for a beam splice with undeveloped flanges. This interaction equation shows that the total shear is distributed between the flange and web splice plates. The first and second term of the equation represents the shear resisted by the flanges and web, respectively. Equation 15 shows that the portion of the shear carried by the flanges ranges from zero for fully yielded flanges ($M = M_f$) to a maximum value for a splice located at the inflection point ($M = 0$). It further shows that the shear carried by the flanges is a function of:

1. Gap clearance in the beam. The shear in the flanges decreases as the gap increases.
2. End distance of the flange bolts. The shear in the flanges decreases as the end distance of the flange bolts increases.
3. Level of flange axial force required for moment = flange axial force divided by flange area. The shear in the flanges decreases as the level of flange axial force increases.

Since the moment can be written as a function of the applied shear, Equation 15 can be further developed by substituting Equations 6, 13, and 14 into Equation 15:

$$\left(\frac{\alpha^2}{b_f F_{yf} L_f}\right)V^2 + V - \left(\frac{b_f t_f^2 F_{yf}}{L_f} + V_w\right) = 0 \tag{16}$$

Equation 16 is a second order equation of V that has a feasible solution as follows:

$$V = \frac{b_f F_{yf} L_f}{2\alpha^2}\left[-1+\sqrt{1+\left(\frac{4\alpha^2}{b_f F_{yf} L_f}\right)\left(\frac{b_f t_f^2 F_{yf}}{L_f}+V_w\right)}\,\right] \qquad (17)$$

Equation 17 provides a closed form solution for the shear capacity of bolted beam splices with undeveloped flanges. The limit on this equation is:

$$M \leq M_f \qquad (18)$$

Since the splices tested in this program had different top and bottom shear gaps, Equation 17 was rederived to represent that case. The analytical plastic model for estimating the shear strength of beam splices with undeveloped flanges and unequal shear gaps is shown in Figure 5. For that case, it can be verified that the total shear capacity becomes:

$$V = \frac{\left[-1+\sqrt{1+\left(\frac{2\alpha^2}{b_f F_{yf}}\right)\left(\frac{1}{L_f}+\frac{1}{L_F}\right)\left[\frac{b_f t_f^2 F_{yf}}{2}\left(\frac{1}{L_f}+\frac{1}{L_F}\right)+V_w\right]}\,\right]}{\frac{\alpha^2}{b_f F_{yf}}\left(\frac{1}{L_f}+\frac{1}{L_F}\right)} \qquad (19)$$

THE PLASTIC MODEL FOR FULLY-DEVELOPED FLANGES

The plastic model for fully-developed flanges is simply a limiting case of the plastic model for undeveloped flanges. After the flanges are fully developed (yielded axially), the flange splices are assumed to develop the axial resistance of the beam flanges and the web splice is assumed to carry the moment not resisted by the flanges at the centerline of the splice. The total shear is assumed to be carried by the web splice; no shear is carried by the flanges. Therefore, the moment resistance of the splice can be written as:

$$M = [(b_f t_f) F_{yf}]\, h + M_w \qquad (20)$$

Further, the shear resistance of the splice can be written as:

$$V = V_w \qquad (21)$$

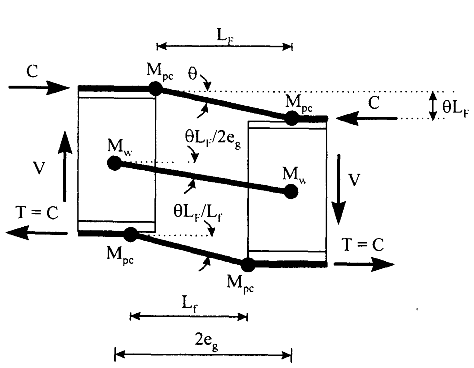

Fig. 5. The plastic model for beam splices with undeveloped flanges and unequal shear gaps.

EXPERIMENTAL PROGRAM

All beam splice tests reported in this paper were designed to fail the critical web bolt in shear. The critical web bolt is the bolt which lies the farthest from the web bolts center of rotation. Therefore, to be able to estimate the capacity of the tested beam splices, shear tests on individual web bolts were performed. The web splice bolts were ⅝-in. diameter ASTM A325 high strength bolts. The web bolts used to manufacture the splices were from two different production lots and individual bolts from each production lot were tested. The threads in the individual bolt tests were excluded from the shear planes, as were the threads in the beam splice tests. The individual web bolts were tested in single shear in the special compression jig shown in Figure 6. The jig consists of two relatively rigid plates placed in two jaws. The bottom jaw is fixed to the testing machine while the top jaw is free to move downwards with the loading head of the testing machine. When the top jaw moves downwards, it forces one of the two rigid plates to move downwards; thus, applying a shearing force to the bolt. All bolts tested in the compression jig were tightened to the snug position.

Tensile coupon tests were performed on the flange splice plates since they were assumed in the analytical models to contribute to the strength of the beam splices. All tensile coupons were machined to a 1½-in. width. In the linear elastic range of the coupon's material, the rate of loading was approximately 0.1 in. per minute. After reaching the yield plateau, the rate of loading increased to 0.2 in. per minute. For all coupon tests, three static yield values were recorded at several yield plateau locations and averaged to obtain the static yield. Each static yield load was recorded five minutes after stopping the loading head motion. Then, the load was increased monotonically until fracturing of the coupon. After fracturing of each coupon, the elongated length was measured. Then, the percent elongation was calculated based on the original 8-in. gage length. The yield and ultimate stresses were obtained by averaging the values from two different coupons tested.

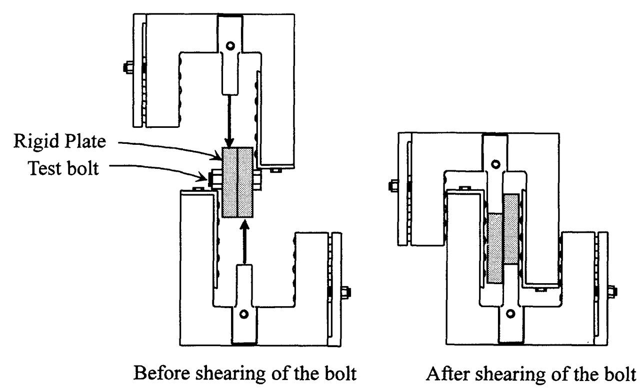

Fig. 6. Test set-up for individual bolts in a single-shear compression splice.

Splice	$b_f \times t_f$ in. × in.	$d_w \times t_w/2$ in. × in.	n × m	b in.	e_g in.	L_F in.	L_f in.
1s	8.072×0.495	12×0.489	2×1	6	4.5	3	6
2s	8.072×0.495	12×0.489	2×1	6	4.5	3	3
3s	8.072×0.495	12×0.489	2×1	2.5	5.5	4	3
4s	8.072×0.495	18×0.489	2×1	15	4.0	3	3
5s	4×0.495	18×0.489	3×1	5	4.0	3	3
6w	0×0	12×0.489	2×1	8	7.5	—	—

Table 1. Details of Splice Plates and Web Bolts Distribution

The test set-up for the beam splices is shown in Figure 7. The test beam was a W24×55 simply supported A36 steel beam with a 16-ft. span. The beam was reinforced by two 8-in.×½-in. A572 grade 50 cover plates to ensure that the beam would not fail before the critical web bolt reaches its shear strength. The beam was spliced at the quarter points near each end. This helped in testing two splices simultaneously and provided a 4-ft. moment to shear ratio.

Six different full-scale symmetric splices were tested. Figure 8 and Table 1 show details of splice plates and web bolts distribution for the tested splices. The flange splice plates were a continuation of the cover plates used to reinforce the test beam. The 8-in. flange splice plate width was kept constant for splices 1, 2, 3, and 4. For splice 5, the flange splice plates were cut at the splice to a width of 4 in. to force the web splice to transfer the moment in excess of the flange splice plates reduced moment resistance. For splice 6, the flange splice plates were removed to force the web splice to transfer the total moment. The flange splice plates for the first splice tested were welded on the south side and bolted on the north side using five longitudinal rows of two ¾-in. diameter A325 high strength bolts at 2½-in. longitudinal spacing. For the rest of the splices, except for splice 6, the flange splices were welded on both sides of the splice. The web splice plates consisted of two ½-in. thick A36 steel plates, one plate on each side of the web. The web bolts for specimens 1 through 4 were from the first production lot. The web bolts for specimens 5 and 6 were from the second production lot. The bolt holes in the beam were drilled and those in the splice plates were punched to expedite the fabrication process. All holes were ¹⁄₁₆-in. larger than the nominal diameter of bolts, as usually done in practice. All bolts were installed using a spud wrench; no evaluation of bolt tension was done. The clearance gap between the two ends of the connected beam segments was 3 inches for all splices. This large clearance was used to introduce a large shear eccentricity to the web bolts.

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

At the time of conducting this testing program, the behavior and force distribution among the web bolts was better understood than that of a beam splice. Consequently, the main focus in all of the tests was to fail the critical web bolt in shear. After failing the critical web bolt in shear, the web and flange forces could then be calculated as the forces required to

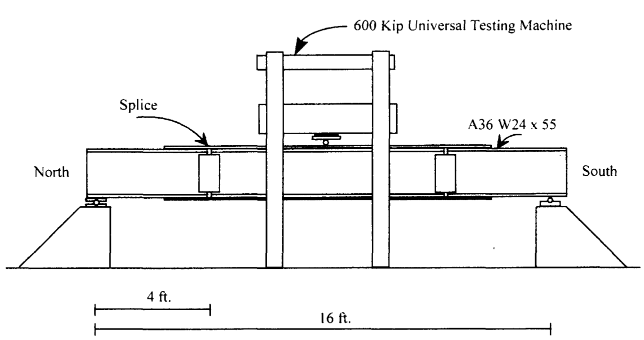

Fig. 7. Test set-up for symmetric beam bolted splices.

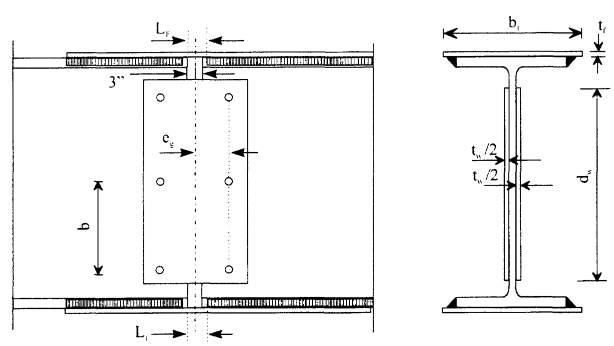

Fig. 8. Details of splice plates and web bolts distribution for tested splices.

Lot Number	Specimen Number	Single-Shear Capacity kips	Estimated Double-Shear Capacity, kips	Test / Spec.
1	1	26.70	53.40	1.16
1	2	25.95	51.90	1.13
1	3	26.10	52.20	1.13
1	**Average**	**26.25**	**52.50**	**1.14**
2	1	27.10	54.20	1.18
2	2	28.20	56.40	1.23
2	3	27.00	54.00	1.17
2	4	26.55	53.10	1.15
2	5	27.85	55.70	1.21
2	6	26.85	53.70	1.17
2	**Average**	**27.26**	**54.52**	**1.19**

Table 2. Shear Resistance of Individual Web Bolts

provide equilibrium of the internal and external forces, as was discussed in the two plastic models.

The 600-kip universal testing machine was used to apply a concentrated load to the test beam at midspan. All splices were loaded monotonically to failure with a varied load increment based on the observed load versus splice vertical deflection plot. The static load was recorded at several loading stages prior to failure. When a splice on one end of the beam failed, by shearing off the critical web bolt, the failed splice was reinforced to enable failure of the splice on the other end of the beam.

DISCUSSION OF EXPERIMENTAL RESULTS

Individual Bolt Tests

Table 2 shows the single and estimated double shear resistance of individual web bolts from each production lot. The double shear resistance of the bolts was assumed twice that of single shear. The mean tested single shear strength of the bolts from the first and second lot was 26.25 kips and 27.26 kips, respectively. Thus, the two shear strengths were practically identical. Further, the previous shear strengths are, respectively, 14 percent and 19 percent larger than the 23 kips value specified in the AISC LRFD specifications after eliminating the built-in 0.75 strength reduction factor and the 0.8 joint-length reduction factor. The latter factor was built into the design strength of one bolt to accommodate the decreasing average strength of all bolts in a joint as a function of the increasing joint length.[7]

One could argue that the shear strength of an individual bolt tested in a tension splice is lower than that of a bolt tested in a compression jig. However as will be shown later, the failure mode of all beam splices presented in this paper was by shearing off of the critical web bolt in the compression zone. Therefore, it is believed that the strength of the tested beam splices is best estimated by using the shear strength of an individual bolt tested in a compression jig.

Tensile Coupons of Flange Splice Plates

The mean static yield stress of 52.7 ksi was above the 50 ksi minimum yield stress as specified by ASTM for A572 grade 50 steel. Also, the mean tensile stress of 74.4 ksi was above the 65 ksi minimum tensile stress as specified by ASTM for A572 grade 50 steel. Both coupons were ductile and failed with a mean axial elongation of 24.6 percent.

Beam Splice Tests

Table 3 shows the midspan failure load, failure moment at the centerline of the splice, and failure mode for the tested splices. Due to symmetry of the tested splices about the neutral axis of the beam, all splices had two critical web bolts. However, all splices, except for splice 2 where loading was discontinued, failed by shearing off the top critical web bolt. From strain measurements taken in this program and those taken by Garrelts and Madsen,[5] it was observed that the compressive stress in the top flange splice plate was always less than the tensile stress in the bottom flange splice plate. This could be due to the reduced axial stiffness of the compression flange which is caused by the P-Δ effect. The reduced stiffness caused the top flange splice to carry a smaller axial load than that of the bottom one. Thus to maintain horizontal equilibrium of forces, the difference between the flange forces was forced to be carried by the web bolts in the compression zone causing the top critical web bolt to fail first.

One method of verifying the analytical approach is to compare the actual failure modes of the tested beam splices with the ones assumed in the analytical models. In general, two failure modes were observed from the tested splices. These failure modes were dependent on whether the flange splice plates were axially developed or undeveloped.

Figure 9 shows the first observed failure mode which corresponds to a symmetric beam splice with undeveloped flanges. At failure, the beam formed a dominantly shear kink at the splice location. The web underwent a double curvature and an inflection point was formed at the centerline of the web splice. Also, plastic hinges were formed in each flange splice plate at the location of the end flange bolt on one side of the joint and the beam edge on the other side of the joint. The formation of plastic hinges in the flange splice plates on both sides of the joint indicates that the flanges carried a portion of the total shear. That the flanges carry a portion of the total shear was also verified by Garrelts and Madsen.[5] Thus, the observed failure mode of the tested beam splices with undeveloped flanges agrees with that assumed in the plastic model for splices with undeveloped flanges.

Figure 10 shows the second observed failure mode which corresponds to a symmetric beam splice with fully-developed flanges. At failure, the beam formed a dominantly flexural kink at the centerline of the splice. The web splice plates underwent a single curvature indicating that the web carried a portion of the applied moment. Thus, the observed failure mode of the tested beam splices with fully-developed flanges agrees with that assumed in the plastic model for splices with fully-developed flanges.

After slipping and going into bearing, the splice plates acted independently of the beam; the strain profile in the beam was not coincident with that in the splice plates. This agrees with the finding of Garrelts and Madsen.[5] Therefore, a beam splice should not be thought of as a cross-section at one point. Rather, the splice should be thought of as a mini-structure acting independently from the beam, as was done in the plastic model presented earlier. Consequently, treating the splice as a cross-section having a linear-elastic stress distribution does not necessarily represent the true distribution of stresses at ultimate load.

Another method of verifying the analytical approach is to compare the actual failure loads of the tested beam splices with the ones obtained from the analytical models. Table 4 shows the theoretical and experimental results for splices 1 through 6. The theoretical results were obtained using the plastic model for either undeveloped or fully-developed flanges, as appropriate. The theoretical shear capacities of the web bolts were obtained using the elastic method for eccentrically loaded groups of bolts. One could argue that the nonlinear analysis represents the actual behavior of eccentrically loaded bolted connections better than does the elastic analysis. However, elastic analysis of the web bolts was used because it was shown by Sheikh-Ibrahim[6] that the elastic method yields reasonable results when compared with test results. Also, Sheikh-Ibrahim[6] showed that the difference between the elastic and nonlinear results gets smaller as the number of bolts decreases and that both methods yield practically identical results for connections with two bolts. Thus,

Table 3.
Test Results of Symmetric Beam Splices

Splice	$P_{test} = 2V_{test}$ kips	M_{test} in.-kips	Failure Mode
1s	142	3,408	Fracture of top critical web bolt
2s	142+	3,408+	Did not continue loading to failure
3s	90	2,160	Fracture of top critical web bolt and top flange weld
4s	190	4,560	Fracture of top critical web bolt
5s	124	2,976	Fracture of top critical web bolt and top flange weld
6w	15	360	Fracture of top critical web bolt

since the number of web bolts in the tests reported in this paper ranged from two to three bolts on each side of the splice, it was felt that elastic analysis of the web bolts should yield satisfactory results.

The capacities of splices 1 through 4 were calculated using the plastic model for undeveloped flanges. For these splices, the theoretical axial load in the flanges ranged from 45 percent (for specimen 3) to 93 percent (for specimen 4) of their axial yield loads. The theoretical portion of the total shear carried by the flanges ranged from 51 percent (for splice 3) to 5 percent (for splice 4). The test shear capacities for splices 1, 2, and 3 significantly exceeded the theoretical shear capacities of the web bolts based on the geometric eccentricity. The test shear capacities for splices 1, 2, and 3 were 22 percent, 22 percent, and 92 percent larger than the theoretical shear capacities of the web bolts, respectively. This verifies

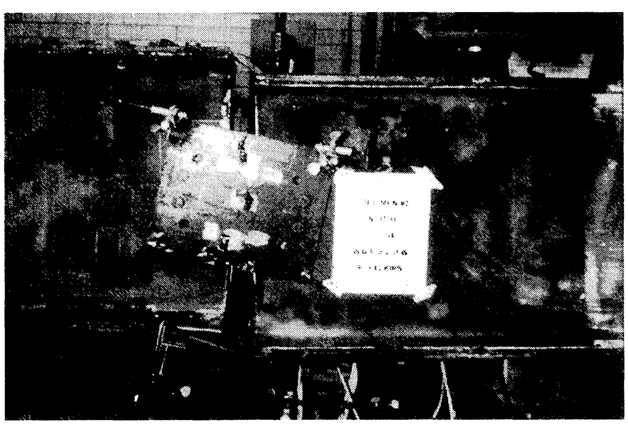

Fig. 9. The failure mode for a bolted beam splice with undeveloped flanges (specimen 3s).

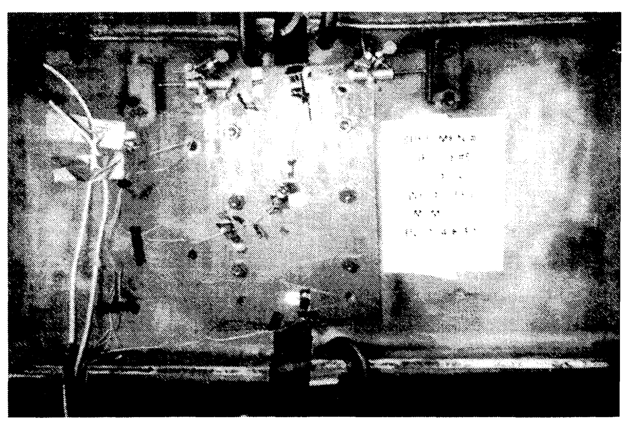

Fig. 10. The failure mode for a bolted beam splice with fully-developed flanges (specimen 5s).

Table 4. Theoretical Versus Test Results of Symmetric Beam Splices						
Splice #	1s	2s	3s	4s	5s	6w
$2V_w$ kips	116.5	116.5	46.5	185.3	116.2	15.7
$2V_f$ kips	27.8	34.1	48.5	10.0	0.0	0.0
$V_w / (V_w + V_f)$	0.81	0.77	0.49	0.95	1.00	1.00
V_w / V_{test}	0.82	0.82	0.52	0.98	0.94	1.05
$V_f / (V_w + V_f)$	0.19	0.23	0.51	0.05	0.00	0.00
M_{theory} / M_f	0.69	0.72	0.45	0.93	1.11	∞
M_{test} / M_f	0.68	0.68	0.43	0.91	1.19	∞
P_{theory} kips	144.3	150.6	95.0	195.3	116.2	15.7
P_{test} kips	142	142+	90	190	124	15
Difference %	1.6	—	5.6	2.8	–6.3	4.7

that undeveloped flange splice plates carry a portion of the total shear not resisted by the web. The test shear capacity for splice 4, where the theoretical axial load in the flanges was 93 percent of their axial yield loads, was only 2 percent larger than the theoretical shear capacity of the web bolts. This verifies that the shear carried by undeveloped flange splice plates decreases as the ratio of axial load to the strength of the flange splice plates increases. Thus, when not fully yielded, the flange splices carry not only the total moment but also a portion of the total shear due to their lateral stiffness. Further, the shear carried by the flanges increases as the ratio of applied axial load to yield load in the flanges decreases. Therefore, for splices with undeveloped flanges, assuming that the web splice carries the total shear should yield conservative results.

The capacities of splices 5 and 6 were calculated using the plastic model for fully-developed flanges. For these splices, the portion of the total shear theoretically carried by the web was 100 percent. Also for these splices, the portion of the total moment theoretically carried by the web ranged from 11 percent (for splice 5) to 100 percent (for splice 6 where the flange splice plate were removed). The test failure moment for splice 5 was 19 percent larger than the moment resistance of the flanges. This could mean that either the flange splice plates carried the additional moment by strain hardening, or that the flange splice plates carried a portion of the additional moment by strain hardening and the web splice carried the remaining portion of the total moment not resisted by the flange splice plates. In splice 6, the flanges were not connected to investigate the ability of the web to carry the moment not resisting by the flanges. The failure moment for splice 6 was carried entirely by the web. Therefore, one can conclude that for a splice with fully developed flanges, assuming that the web splice carries the moment not resisted by the flanges should yield satisfactory results.

Table 4 shows that the predicted failure loads when using the plastic model for beam splices with both undeveloped and fully developed flanges combined with elastic analysis of the web bolts agreed well with test results. The difference between the predicted and experimental failure loads ranged from –6.3 percent to +5.6 percent with an absolute average value of 4.2 percent. The difference was unconservative for the splices with two web bolts (splices 1, 2, 3, 4, and 6) and conservative for the splice with three web bolts (splice 5). It is believed that the unconservativeness of the predicted capacity for the case of two web bolts is due to the connection's excessive rotation at failure as explained by Sheikh-Ibrahim.[6] The excessive rotation makes the applied load inclined with respect to the bolts original geometry, thus, reducing the connection capacity.

Discussion of Analysis Method

So far, two force distributions and behaviors were presented and verified for beam splices. It was shown that both the force distribution and type of behavior are dependent on whether the flanges are developed or undeveloped. However in general, it should be mentioned that the designer is in full control of the design. For example, the flanges alone might be able to resist the total moment; however, the designer might opt to under-design the flange splices and force the web to transfer the residual moment not resisted by the flange splices; as required by AASHTO. Conversely, the designer might opt to design the flange splices for the total design moment and the web splice for shear only; as recommended by AISC. Provided that either the AISC or AASHTO approach can be used, the designer needs to make a judgment on the approach to be used. For the sake of discussion, let us assume that the web is infinitely strong. Thus, one can go to the extreme case of not connecting the flanges and developing the full moment in the web splice. This will eliminate the flange bolts. However, the number of web bolts added to the web will be significantly larger than the number of bolts eliminated from the flanges. This is due to two reasons. The first reason is that the web bolts, unlike the flange bolts, are subjected to a moment due to the eccentricity of the shear which reduces their efficiency. The second reason is that the flange bolts, unlike the web bolts, are extremely effective in resisting the moment since they lie at the extreme beam fibers. Thus, if possible, designing the flanges for the total moment will yield a larger number of flange bolts, fewer web bolts, and most importantly a smaller total number of splice bolts. Hence, the AISC approach as opposed to the AASHTO approach will lead to material and labor cost savings as well as simplify construction.

Further, tests by Garrelts and Madsen[5] had shown that for splices designed using the AASHTO approach, there was an increase in flange stress over the theoretical value due to the splicing action. This caused the girder flanges to fail at a slightly lower load than anticipated. Based on unsymmetric splice tests performed by Sheikh-Ibrahim,[6] the authors of this paper believe that the flanges are more likely to strain harden before the web bolts can develop their resistance at ultimate load. Strain hardening of the flanges reduces the web splice design forces; however, the designer needs to be cautious of premature flange failure, such as compression flange buckling or tension flange net section fracture. Therefore, given that the engineer chooses to follow the AASHTO approach, the authors believe that the flange splices still need to be designed for the smaller of the total applied moment or the moment resistance of the flanges to avoid premature flange failure. As an alternate to developing the full moment in the flanges, the designer needs to enforce deformation compatibility. This can be done by assuming a linear strain distribution at the vertical cut that passes through the critical web bolt. Then given that the critical web bolt deformation at failure is known, the strain in the tension flange can be calculated. This strain can then be checked against either the strain at the beginning of the strain hardening region or that at fracture of the splice material. This is a rather lengthy, complicated, and yet uneconomical procedure. On the contrary, the AISC approach is simple yet economical. The only modification that needs to be done to the AISC approach is for the case when the flanges alone are not capable of resisting the total moment. For that case, the flange splices can be designed to develop the axial resistance of the beam flanges. The web splice can be designed for the shear, moment due to the eccentricity of the shear, and moment in excess of the moment resistance of the flanges.

SUMMARY AND CONCLUSIONS

Based on the results of the analytical and testing program of six full-scale symmetric bolted beam splices presented in this paper and on the tests available in the literature, the following conclusions are made:

- Flange splices, if not fully yielded, carry a portion of the total shear. This shear is a function of the ratio of the applied moment to moment capacity of the flange splices, gap clearance in the beam, and end distance of the flange bolts. To simplify the design process, the shear carried by the flange splices can be conservatively ignored and the web splice can be assumed to carry the total shear.
- AISC recommends that flange splices be designed for the total moment and web splices be designed for the shear as well as the moment caused by the eccentricity of the shear. On the other hand, AASHTO requires that web splices be designed for the shear, moment due to the eccentricity of the shear, and the portion of the design moment resisted by the web. Also, AASHTO requires that flange splices be designed for the portion of the design moment not resisted by the web. Either the AASHTO or AISC approach can be used to apportion the moment between the web and flange splices. However, the AISC approach is easier and more economical than the AASHTO approach.
- If the AASHTO approach is used, one of the following two methods must be used to avoid premature flange failure:
  1. The flanges need to be designed for the smaller of either the total moment or the moment resistance of the flanges
  2. Given that the critical web bolt deformation at failure is known and assuming a linear strain distribution at the vertical cut that passes through the critical web bolt, the strain in the tension flange can be checked against either the strain at the beginning of the strain hardening region or that at fracture of the splice material
- When the flanges alone are not able to resist the total moment, the AISC approach needs to be modified. For that case, flange splices can be designed to develop the axial strength of the beam flanges. The axial strength of the tension flange can be calculated based on gross section yielding or net section fracture, whichever controls. The axial strength of the compression flange can be calculated based on gross section yielding. The web splice can be designed for the shear, moment due to the eccentricity of the shear, and moment not resisted by the flanges.

DESIGN RECOMMENDATIONS

The following design recommendations are made to reduce the total number of bolts and to prevent premature flange failure:

- If possible, flange splices should be designed to carry the required flexural strength through a tension-compression couple acting as axial forces at the centroids of the top and bottom flanges. Web splices should be designed for the required shear strength and for the moment caused by the eccentricity of the shear. For splices symmetric about the centerline of the splice, the eccentricity of the shear is the distance from the centerline of the splice to the centroid of the web bolts on one side of the splice.
- If the flanges alone can not carry the required flexural strength, flange splices should be designed to develop the axial design strength of the beam flanges. The axial design strength of the tension flange can be calculated based on gross section yielding or net section fracture, whichever controls. The axial design strength of the compression flange can be calculated based on gross section yielding. The web splice should be designed for the required shear strength, moment due to the eccentricity of the shear, and moment not resisted by the flanges.

AISC LRFD DESIGN EXAMPLES

Design Example 1: (Undeveloped Flanges)

Design a bolted splice for an A572 Gr. 50 W18×50 beam. 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 = 65 kips and M_u = 2,400 in.-kips.

Solution:

Flange Splice Design:

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

$$P_{uf} = \frac{M_u}{d - t_f} = \frac{2{,}400}{17.99 - 0.57} = 137.8 \text{ kips}$$

Determine the axial design strength of the compression flange:

$$\phi R_n = \phi F_{cr} A_g = 0.85 \times 50 \times (7.495 \times 0.57)$$
$$= 0.85 \times 50 \times 4.27 = 181.5 \text{ kips}$$

Determine the axial design strength of the tension flange:

Based on tension yielding of flange:

$$\phi R_n = \phi F_y A_g = 0.9 \times 50 \times 4.27 = 192.2 \text{ kips}$$

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

$$\phi R_n = \phi F_u A_n = 0.75 \times 65 \times [4.27 - (2(7/8 + 1/8) \times 0.57)]$$
$$= 0.75 \times 65 \times 3.13 = 152.6 \text{ kips}$$

Determine whether the flanges are developed or undeveloped:

$$P_{uf} = 137.8 \text{ kips} < \phi R_n = 152.6 \text{ kips}$$

Therefore, the flanges are undeveloped and are capable of resisting the factored design moment.

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

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{137.8}{27.1} = 5.1 \approx 6 \text{ bolts}$$

where ϕr_n = 27.1 kips is the single shear design strength as given in Table 8-11.[1]

Determine the number of ⅞-in. flange bolts required for material bearing on beam flange, using PL⅝×7¼×18½-in., L_e = 1½ in. >1.5d, and s = 3 in.:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{137.8}{102 \times 0.57} = 2.4 \approx 4 \text{ bolts}$$

where ϕr_n = 102 kip/in. thickness as given in Table 8-13.[1]

Therefore, bolt shear is more critical than material bearing. Try two rows of three bolts at 3-in. spacing and 4¼-in. gage.

Check tension yielding of flange plate:

$$\phi R_n = \phi F_y A_g = 0.9 \times 50 \times 0.625 \times 7.25$$
$$= 203.9 \text{ kips} > 137.8 \text{ kips} \quad \textbf{o.k.}$$

Check tension rupture of flange plate:

$$\phi R_n = \phi_t F_u A_n = 0.75 \times 65 \times [7.25 - 2(7/8 + 1/8)] \times 0.625$$
$$= 160.0 \text{ kips} > 137.8 \text{ kips} \quad \textbf{o.k.}$$

Check block shear rupture of tension 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 two rows of bolt holes in the flange plate. The second case involves the rupture of the block inside the two rows of bolt holes. The third case involves the rupture of one block which is bounded by one of the 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 two rows of bolt holes. One can show that for this example, the block shear rupture strength of the fourth case controls and is determined as follows:

$$0.6F_u A_{nv} = 2 \times [0.6 \times 65 \times (1.5 + (2 \times 3)$$
$$- 2.5 \times (7/8 + 1/8)) \times 0.57] = 222.3 \text{ kips}$$

$$F_u A_{nt} = 2 \times [65 \times (1.6225 - 0.5 \times (7/8 + 1/8)) \times 0.57]$$
$$= 83.2 \text{ kips}$$

Since $0.6F_u A_{nv} > F_u A_{nt}$, use:

$$\phi R_n = \phi[0.6F_u A_{nv} + F_y A_{gt}]$$

$$\phi R_n = 0.75[222.3 + 50 \times (2 \times 1.6225 \times 0.57)]$$
$$= 236.1 \text{ kips} > 137.8 \text{ kips} \quad \textbf{o.k.}$$

Check the design compressive strength of flange plate:

Check 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):

$$\frac{Kl}{r} = \frac{0.65 \times 3.5}{\sqrt{\dfrac{(7.25 \times 0.625^3)/12}{(7.25 \times 0.625)}}} = 12.61$$

From AISC-LRFD Specifications Table 3-50 with

$$\frac{Kl}{r} = 12.61, \ \phi F_{cr} = 42 \text{ ksi}$$

and the design compressive strength of the flange plate is:

$$\phi R_n = \phi F_{cr} A_g = 42 \times (7.25 \times 0.625)$$

$$= 190.3 > 137.8 \text{ kips} \quad \textbf{o.k.}$$

Use PL⅝×7¼×18½-in. and two rows of three ⅞-in. A325-X bolts at 3-in. spacing and 4¼-in. gage on each side of the splice centerline.

Web Splice Design:

Try 2 PL3⁄16×6½×9-in. and one row of three ⅞-in. A325-X bolts at 3-in. spacing on each side of the splice.

Determine the controlling design strength per one bolt:

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

$$= \begin{Bmatrix} \phi r_n = 54.1 \text{ kips} \\ \phi r_n = 102 \times 0.355 = 36.2 \text{ kips} \end{Bmatrix}$$

Therefore, material bearing on the beam web controls the design of the web bolts and ϕr_n = 36.2 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:

$$r_1 = \frac{V_u}{n} = \frac{65}{3} = 21.67 \text{ kips}$$

Additional shear force on critical bolt due to eccentricity:

$$r_2 = \frac{V_u e C_y}{I_p} = \frac{65 \times 1.75 \times 3}{2 \times 3^2} = \frac{113.8 \times 3}{18} = 18.96 \text{ kips}$$

Resultant shear force on critical bolt:

$$r_u = \sqrt{r_1^2 + r_2^2} = \sqrt{21.67^2 + 18.96^2}$$

$$= 28.79 \text{ kips} < \phi r_n = 36.2 \text{ kips} \quad \textbf{o.k.}$$

Check flexural yielding of web plates:

$$\phi M_n = \phi F_y S_x$$

$$\phi M_n = 0.9 \times 50 \times \frac{3/8 \times 9^2}{6}$$

$$= 227.8 \text{ in.-kips} > V_u e = 113.8 \text{ in.-kips} \quad \textbf{o.k.}$$

Check flexural rupture of web plates:

$$\phi M_n = \phi F_u S_{net}$$

where S_{net} = 3.56 in.³ From Table 12-1.[1]

$$\phi M_n = 0.75 \times 65 \times 3.56$$

$$= 173.6 \text{ in.-kips} > 113.8 \text{ in-kips} \quad \textbf{o.k.}$$

Check shear yielding of web plates:

$$\phi R_n = \phi(0.6 F_y A_g)$$

$$\phi R_n = 0.9 \times 0.6 \times 50 \times (3/8 \times 9)$$

$$= 91.1 \text{ kips} > V_u = 65 \text{ kips} \quad \textbf{o.k.}$$

Check shear rupture of web plates:

$$\phi R_n = \phi(0.6 F_u A_n)$$

$$\phi R_n = 0.75 \times 0.6 \times 65 \times 3/8 \times (9 - 3 \times 1)$$

$$= 65.8 \text{ kips} > V_u = 65 \text{ kips} \quad \textbf{o.k.}$$

Check block shear rupture of web plates:

From Tables 8-47 and 8-48,[1] with three ⅞-in. diameter bolts and L_{ev} = L_{eh} = 1½-in., $0.6F_uA_{nv}$ = 146 × ⅜ = 54.8 kips > F_uA_{nt} = 48.8 × ⅜ = 18.3 kips.

Therefore,

$$\phi R_n = \phi[0.6 F_u A_{nv} + F_y A_{gt}]$$

$$\phi R_n = [146 + 56.3] \times 3/8 = 75.9 \text{ kips} > 65 \text{ kips} \quad \textbf{o.k.}$$

Use 2 PL3⁄16×6½×9-in. and one row of three ⅞-in. A325-X bolts at 3-in. spacing on each side of the web splice.

Design Example 2: (Fully Developed Flanges)

Design a bolted splice for an A572 Gr. 50 W18×50 beam. 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 = 65 kips and M_u = 2,975 in.-kips.

SOLUTION:

Flange Splice Design:

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

$$P_{uf} = \frac{M_u}{d - t_f} = \frac{2{,}975}{17.99 - 0.57} = 170.8 \text{ kips}$$

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

$$\phi R_n = 181.5 \text{ kips} > 170.8 \text{ kips} \quad \textbf{o.k.}$$

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

Based on tension yielding of flange:

$$\phi R_n = 192.2 \text{ kips} > 170.8 \quad \textbf{o.k.}$$

Based on tension rupture of flange:

ϕR_n = 152.6 kips < 170.8 kips **n.g.**

Therefore, the flanges are not capable of resisting the total factored design moment. The web must be designed to resist the moment not resisted by the flanges.

Determine the flange design force as the axial design strength of the flanges:

P_{uf} = 152.6 kips

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

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{152.6}{27.1} = 5.6 \approx 6 \text{ bolts}$$

Determine the number of ⅞-in. flange bolts required for material bearing on beam flange, using PL⅝×7¼×18½-in., L_e = 1½-in. > 1.5d, and s = 3 in.:

$$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{152.6}{102 \times 0.57} = 2.6 \approx 4 \text{ bolts}$$

Therefore, bolt shear is more critical than material bearing. Try two rows of three bolts at 3-in. spacing and 4¼-in. gage.

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

ϕR_n = 203.9 kips > 152.6 kips **o.k.**

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

ϕR_n = 160.0 kips > 152.6 kips **o.k.**

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

ϕR_n = 236.1 kips > 152.6 kips **o.k.**

Check the design compressive strength of flange plate (determined in Example 1):

ϕR_n = 190.3 > 152.6 kips **o.k.**

Use PL⅝×7¼×18½-in. and two rows of three ⅞-in. A325-X bolts at 3-in. spacing and 4¼-in. gage on each side of the splice centerline.

Web Splice Design:

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

$$M_{uw} = M_u - P_{uf}(d - t_f) = 2{,}975 - 152.6 \times (17.99 - 0.57)$$

$$= 316.7 \text{ in.-kips}$$

Try 2 PL³⁄₁₆×6½×15-in. and one row of four ⅞-in. A325-X bolts at 4-in. spacing on each side of the splice.

Determine the controlling design strength per one bolt:

As was determined in Example 1, material bearing on the beam web controls the design of the web bolts and ϕr_n = 36.2 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:

$$r_1 = \frac{V_u}{n} = \frac{65}{4} = 16.25 \text{ kips}$$

Additional shear force on critical bolt due to eccentricity of shear and design moment:

$$r_2 = \frac{(M_{uw} + V_u e)C_y}{I_p} = \frac{[316.7 + (65 \times 1.75)] \times 6}{2 \times (2^2 + 6^2)}$$

$$= \frac{430.45 \times 6}{80} = 32.28 \text{ kips}$$

Resultant shear force on critical bolt:

$$r_u = \sqrt{r_1^2 + r_2^2} = \sqrt{16.25^2 + 32.28^2}$$

$$= 36.1 \text{ kips} < \phi r_n = 36.2 \text{ kips} \quad \textbf{o.k.}$$

Check flexural yielding of web plates:

$$\phi M_n = \phi F_y S_x$$

$$\phi M_n = 0.9 \times 50 \times \frac{3/8 \times 15^2}{6}$$

$$= 632.8 \text{ in-kips} > (M_{uw} + V_u e) = 430.45 \text{ in.-kips} \quad \textbf{o.k.}$$

Check flexural rupture of web plates:

$$\phi M_n = \phi F_u S_{net}$$

where

$$S_{net} = \frac{t}{6}\left[d^2 - \frac{S^2 n(n^2 - 1)(d_b + 0.125)}{d}\right]$$

as specified in Table 12-1.[1]

$$S_{net} = \frac{3/8}{6}\left[15^2 - \frac{4^2 \times 4(4^2 - 1)(7/8 + 0.125)}{15}\right] = 10.06 \text{ in.}^3$$

$$\phi M_n = 0.75 \times 65 \times 10.06$$

$$= 490.4 \text{ in.-kips} > 430.45 \text{ in.-kips} \quad \textbf{o.k.}$$

Check shear yielding of web plates:

$$\phi R_n = \phi(0.6 F_y A_g)$$

$$\phi R_n = 0.9 \times 0.6 \times 50 \times (3/8 \times 15)$$

$$= 151.9 \text{ kips} > V_u = 65 \text{ kips} \quad \textbf{o.k.}$$

Check shear rupture of web plates:

$$\phi R_n = \phi(0.6 F_u A_n)$$

$$\phi R_n = 0.75 \times 0.6 \times 65 \times 3/8 \times [15 - (4 \times 1)]$$

$$= 120.7 \text{ kips} > V_u = 65 \text{ kips} \quad \textbf{o.k.}$$

Check block shear rupture of web plates:

$$0.6 F_u A_{nv} = 0.6 \times 65 \times (1.5 + 3 \times 4 - 3.5 \times (7/8 + 1/8)) \times 3/8$$

$$= 146.3 \text{ kips}$$

$$F_u A_{nt} = 65 \times (1.5 - 0.5 \times (7/8 + 1/8)) \times 3/8 = 24.4 \text{ kips}$$

Since $0.6 F_u A_{nv} > F_u A_{nt}$, use:

$$\phi R_n = \phi\,[0.6 F_u A_{nv} + F_y A_{gt}]$$

$$\phi R_n = 0.75\,[146.3 + 50 \times (1.5 \times 3/8)]$$

$$= 130.8 \text{ kips} > 65 \text{ kips} \quad \textbf{o.k.}$$

Use 2 PL$3/16 \times 6\frac{1}{2} \times 15$-in. and one row of four $7/8$-in. A325-X bolts at 4-in. spacing on each side of the web splice.

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NOMENCLATURE

α = ratio of axial load in the flange to the total applied shear

b = vertical spacing of web bolts

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

c = the shear strength of an eccentrically loaded group of bolts normalized with respect to the shear strength of one bolt

C = the compressive axial load in the top flange

d_w, t_w = depth and thickness of each web splice plate, respectively

e_g = geometric eccentricity; distance from centerline of splice to centroid of web bolts on one side of the joint

F_{yf} = the yield strength of flange splice plates

h = the depth between the centroid of the top flange splice and that of the bottom flange splice

L_F, L_f = Shear gap for top and bottom flange splice plate, respectively. The shear gap equals the distance from the first flange bolt on one side of the splice to the beam edge on the other side of the splice

m = number of web bolts in a horizontal row

M = the moment applied at the centerline of the splice

M_f = the moment resistance of the flange splices

$\overline{M}_f$ = the plastic moment resistance of the cross section of each flange splice plate in the absence of axial load

M_{Pc} = the plastic moment resistance of the cross section of each flange splice plate modified for the presence of axial load

M_{test} = the failure moment at the centerline of the splice

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

n = number of web bolts in a vertical row

P_{PM} = the estimated plastic model failure load

P_{test} = the static midspan failure load

R = the strength of an individual web bolt

T = the axial tensile load in the bottom flange

θ = the rotation of top flange splice plate at ultimate load

V = the applied shear at the centerline of the splice

V_f = the theoretical shear resisted by the flanges at the centerline of the splice

V_{test} = the failure shear at the centerline of the splice

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

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