Behavior of Bearing Critical Double-Angle Beam Connections

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The design of bolted connections generally incorporates limiting or allowable loads or stresses on the fastener and on the connected parts in such a way that the strength of the connected part or member, not that of the fastener, governs the ultimate behavior of the structure. For simple bolted lap splices, the net section strength, the bearing strength, and the fastener shear strength are evaluated in terms of nominal stresses for a particular loading to determine the adequacy of the connection detail. In an effort to ensure that one of the basic strength mechanisms of the connected member will control the ultimate behavior, the fastener allowable stresses have traditionally been quite conservative.

While the philosophy of ensuring that member failure governs is reasonable, a critical evaluation of recent bolted connection research by Fisher and Struik[1] has led to recommendations of higher allowable shear stresses on bolts (potentially, 20 to 30 percent fewer bolts are required according to the shear stress requirements). However, a margin of safety is maintained for member control of behavior. The bearing strength criteria were also reevaluated and higher allowable bearing stresses based on a new bearing strength model were recommended.

The new design criteria for fastener shear and material bearing strength[1] have been incorporated in the Canadian Standards Association Standard S16.1-1974, *Steel Structures for Buildings—Limit States Design,*[2] and in the *Specification for Structural Joints Using ASTM A325 or A490 Bolts,*[3] approved by the Research Council on Riveted and Bolted Structural Joints, February, 1976.

In the past, the bearing and bolt shear strength criteria developed for simple tensile plate splices have been applied to other connection types without further validation. However, in consideration of the significantly higher bearing values, the Canadian Institute of Steel Construction arranged for tests of simple double-angle beam-column connections at the University of Toronto. These tests and their evaluation formed a portion of a fourth year undergraduate thesis[4] and are the focal point of this paper.

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The bearing critical detail which was tested showed a reasonable margin of safety when the beam was left uncoped. Coping of the top flange significantly reduced the connection strength and suggested that the strength model developed for tension splices does not apply, without modification, to the double-angle beam connection, and that further research is desirable on other connection details.

BEARING STRENGTH CRITERIA

In the 1969 AISC Specification,[5] and also in the Canadian allowable stress standard, CSA Standard S16, 1969,[6] the allowable bearing stress, F_p, on the projected area is taken as $1.35F_y$, where F_y is the yield stress of the connected part. The 1976 RCRBSJ Specification increased the allowable stress in bearing to the lesser of:

$$\frac{LF_u}{2d} \tag{1}$$

or

$$1.5F_u \tag{2}$$

For a steel with F_y = 44 ksi and F_u = 65 ksi, such as CSA G40.21 grade 44W,[7] the maximum increase in allowable stress is 1.64 times the previous values. The failure modes which are guarded against by these two relationships have been well described by Fisher and Struik and are, for Eq. (1), fastener tear-out through the end zone and, for Eq. (2), large hole deformations and material piling up in front of the fastener. The experimental results used to substantiate the bearing/tear-out failure mechanism were from tests of tension lap splices connected with from one to three fasteners.

For fastener tear-out, Eq. (1), the significant parameter is L, the distance from the center of the bolt hole to the end of the plate in the direction of the force, or more simply, the end distance. Although for a tension lap splice this model is straightforward, for standard bolted connections L has also been defined in the 1976 RCRBSJ Specification as the distance from the center of one bolt hole to the nearest edge of the adjacent bolt hole in the direction of the force. Thus, the tear-out strength of bolted plate material is a function of pitch and/or end distance.

In the case of the standard double-angle beam connection, bolted to the beam web, and beam flanges uncoped, the distance L for the web would logically be interpreted

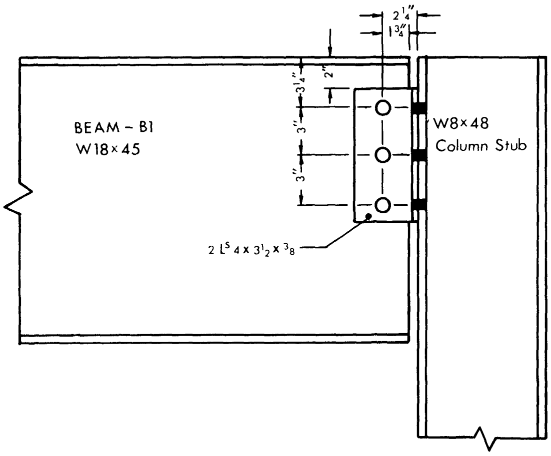

Fig. 1. Bolted double-angle beam connection

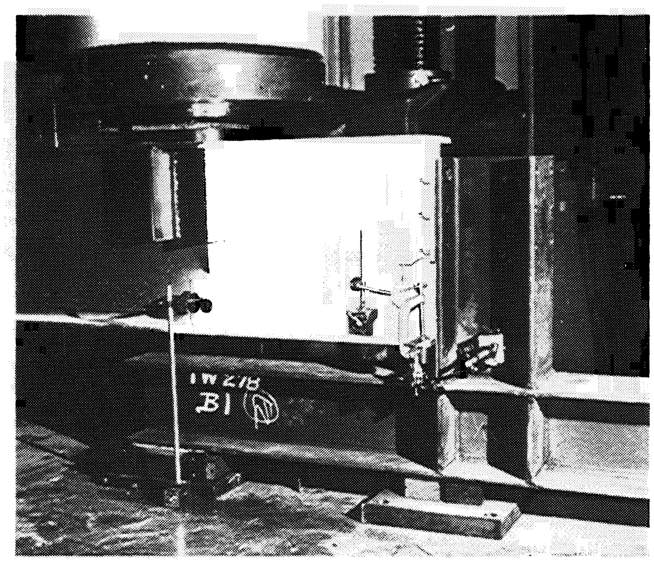

Fig. 2. Specimen for Test I-1

to be the bolt pitch minus one-half the hole diameter, as the flange provides obvious restraint to fastener tear-out.

Thus, in the design of a bolted connection, the member strength is a function of the geometry and the ultimate tensile strength of the member material.

DOUBLE-ANGLE BEAM CONNECTION

A bolted double-angle beam connection was selected for a W18X45 section such that bearing strength would control the allowable end reaction. The beam material met the Canadian Standards Association Standard G40.21, *Structural Quality Steels,* in the Grade 44W,[7] which has a specifieid minimum yield strength of 44 ksi; Table 1 summarizes the nominal and measured properties of the test beam. The basic connection detail is shown diagrammatically in Fig. 1.

Allowable stress or load computations which are well known for this detail are summarized in Table 2. Here, a direct comparison is made of the 1969 AISC Specification[5] and the new recommendations set forth in the 1976 Research Council Specifications.[3] In both cases, the bearing requirements control the allowable reaction and the connection is shown to develop only a portion of the total allowable beam shear.

The test set-up was very straightforward (see Fig. 2); the beam was connected with the double angles to a column stub and loaded near the reaction to preclude flexural failure of the beam. The applied load and the smaller of the two beam reactions were measured to determine the load on the test connection. The connection did behave flexibly (i.e., negligible end moment) throughout the test, as evidenced by a calculation of the moment at the test connection using the measured applied load and opposite end reaction.

A plot of the reaction (nominal shear load on connection) vs. the net deflection of the end of the beam with respect to the column is shown in Fig. 3.

Table 1. Nominal and Measured Properties of Specimen

Property or Dimension	Nominal	Measured
W18x45 Beam		
Beam depth, d, in.	17.86	18.0
Flange width, b_f, in.	7.48	7.5
Flange thickness, t_f, in.	0.499	0.500
Web thickness, t_w, in.	0.335	0.305
CSA G40.21 grade 44W Steel		
F_y, ksi	44	52.5
F_u, ksi	65	79.0
ASTM A325-¾-in. ϕ Bolts		
F_u, ksi	120	150.8

Table 2. Allowable Beam Reactions

Design Criterion	Allowable Beam Reaction (kips)[a] Based on Nominal Properties	Allowable Beam Reaction (kips)[a] Based on Measured Properties
(1) AISC Specification, 1969:		
Beam Shear	105	115
Bolt Shear	58	73
Bearing	44.8	48.6
(2) RCRBSJ Specification, 1976:		
Bolt Shear	79	100
Bearing	73.5	81.3

[a] For W18x45 beam in G40.21 grade 44W steel and A325-¾-in. ϕ bolts.

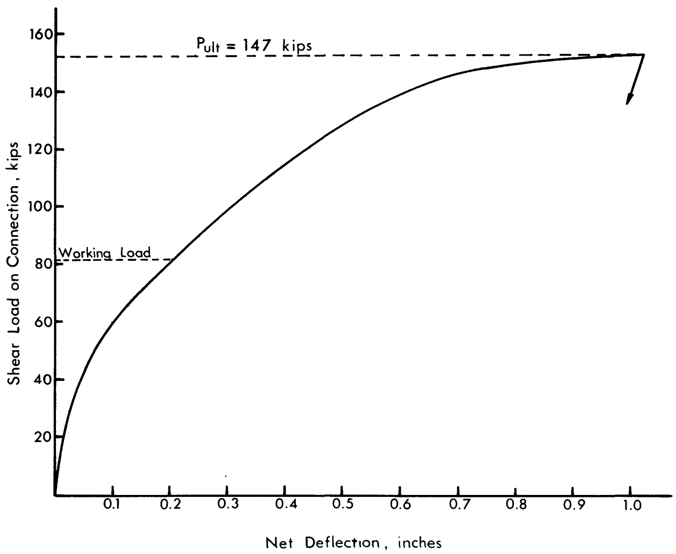

Fig. 3. Nominal shear load on connection vs. net deflection of beam end for Test I-1, top flange uncoped

Note that the load deflection gradient leveled off at 147 kips. A sharp report and sudden unloading was cause for terminating the test. Disassembly of the angles from the beam web showed the cause of the sound was the tensile splitting failure of the end of the beam at the lower bolt hole, Fig. 4. General yielding combined with the high bearing force on the lower bolt caused the fracture. It should be noted that the combination of eccentric loading on the bolts and flexibility of the angles caused a significant component of force towards the end of the beam at the lower bolt. Also evident in Fig. 4 is the participation of the upper flange in containing the web yielding; the flange itself developed a plastic hinge. The failure load divided by the allowable load (using measured properties) was 1.8. A higher load may have been achieved if the lower fastener had not pulled out of the end of the beam.

Concerning deflection, refer to Fig. 3 and note that if one assumes elastic loading from a maximum service load of 81.3 kips (Table 1), then a residual deflection of approximately 0.1–0.2 in. would result.

After some consideration of the failure mechanism, the question of the effect of coping the top flange was raised. Obviously coping of the top flange would remove the containing effect evidenced in the first test and might lead to a more general rupture of the beam web. To this end, the original beam was tested in several additional configurations to assess the effect of a cope.

COPED BEAM

For the connection test with a coped beam, all details of the connection were left unchanged with the exception of the cope. The coped connection detail is shown diagrammatically in Fig. 5. The details of the top flange cope were

Fig. 4. Beam end at failure, Test I-1

chosen to leave an end distance of $3d$ (2.25 in.) from the center of the upper fastener hole to the edge of the cope in the direction of the nominal applied shear force. By so doing, the design criteria would permit the same loads (see Table 2) as the uncoped beam.

The test set up was similar to the previous test, except that a brace consisting of a plate from the column stub to the top flange of the beam was provided to improve lateral

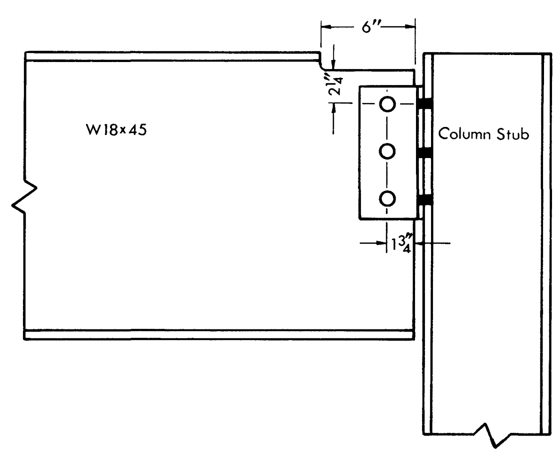

Fig. 5. Detail of Test I-2 with top flange coped

stability. This plate acted to improve the lateral shear and weak axis flexural stiffness in the vicinity of the cope without contributing significantly to the strong axis shear resistance under study.

A plot of the reaction vs. the net deflection of the end of the beam with respect to the column is displayed in Fig. 6. When the load in the connection had attained 90 kips, the deflection rate increased until the load deflection gradient leveled off at 112 kips. At this load the test was terminated and the specimen dismantled.

Massive hole elongation can be observed in the coped beam specimen (see Fig. 7); the splitting of the beam web at the lower bolt hole had progressed to the point that the web material had necked down and surface cracking had commenced. The nature of the more localized yielding of the web compared to that of Test I-1, Fig. 4, and the lower ultimate load, confirmed the earlier observation about the significance of the presence of the flange for distributing the shear in the beam web and for contributing to the strength of the connection.

Some local buckling of the web was also observed at the coped edge adjacent to the top fastener. The absence of the stiffening effect of the flange, the force component at the upper fastener caused by the eccentric loading, and the state of general yielding in that region contributed to this buckling.

Because of the important ramifications of the reduced beam strength for the coped case, two supplemental tests were devised to study the web failure mechanism more closely.

SUPPLEMENTAL TESTS

A special loading fixture and specimen were designed to allow testing of the coped region in shear with a universal testing machine. Two 18-in. lengths of beam were cut from the unyielded portion of the test beam and fabricated as shown in Fig. 8. The top flange was removed and the two models of coped connections were fabricated by cutting a slot in the web at the center of the specimen and drilling bolt holes. The hole diameters, pitch, and end and edge distances remained the same as those for the coped beam test. Two long plates, $7\frac{1}{2}$-in. by $\frac{1}{2}$-in., were welded to the beam section ends to serve as loading plates; the other ends of these plates were bolted to a special loading fixture held by the lower head of the testing machine.

Finally, four angles were bolted to the web plate, as shown in Fig. 8(b), and to a plate perpendicular to the beam web at their upper end, which served as a tongue for gripping and application of the load by the upper head of the test machine. With this symmetrical arrangement, a shear load could be applied to each of two connections using a universal test frame.

For the first test, II-1, the two pairs of angles were not bolted together, whereas they were in the second test, II-2. The angle pairs were bolted together with spacers for additional restraint after observing a tendency for the model beam ends to rotate about the lower point of attachment.

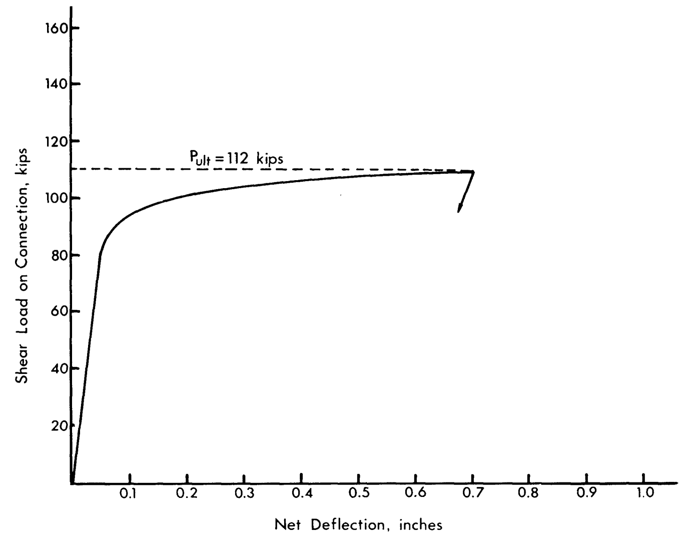

Fig. 6. Nominal shear load on connection vs. net deflection of beam end for Test I-2, top flange coped

The maximum loads attained for Tests II-1 and II-2 were 113 and 108 kips, respectively. In each case the failure mode consisted of tensile type rupture horizontally from

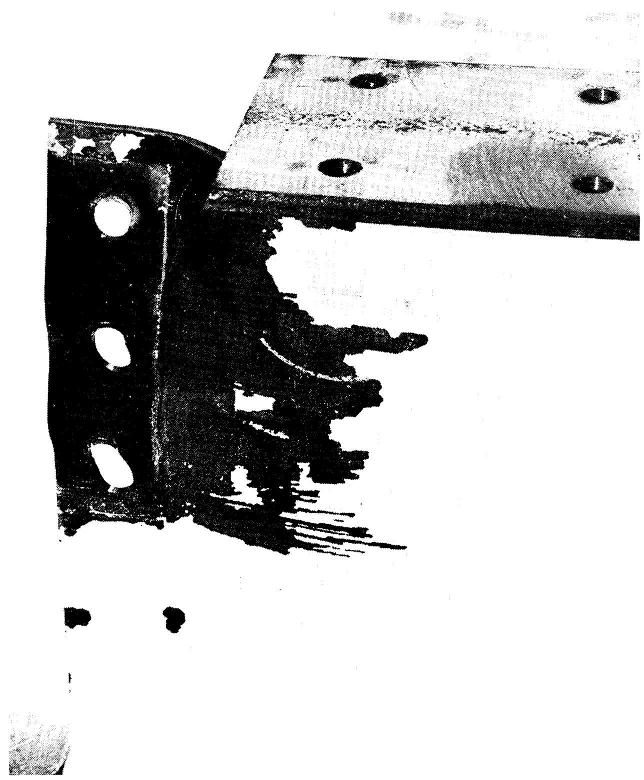

Fig. 7. Beam end at failure, Test I-2

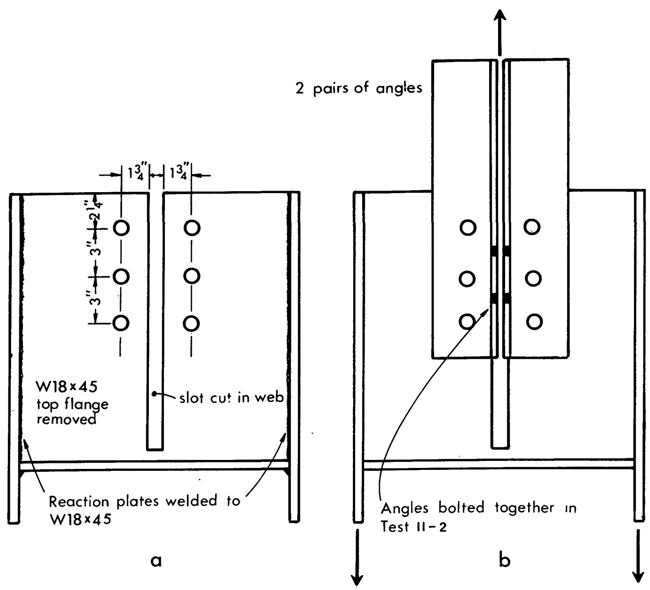

Fig. 8. Supplemental test arrangement

the lowest bolt hole to the edge of the web and a shearing rupture vertically from the lowest bolt hole through the web to the uppermost hole or to the edge of the cope, removing a block of the web material, Fig. 9. It should be noted that this model did not perfectly duplicate the beam-to-angle-to-column interaction, but the ultimate failure mechanism was identical to that which was incipient at the termination of the coped-beam test, I-2.

Since the failure mechanism which developed in the coped-connection models was distinctly different from the one assumed in the development of the Eq. (1), two small tensile lap splices were fabricated and tested to provide a direct experimental comparison of the failure mechanisms.

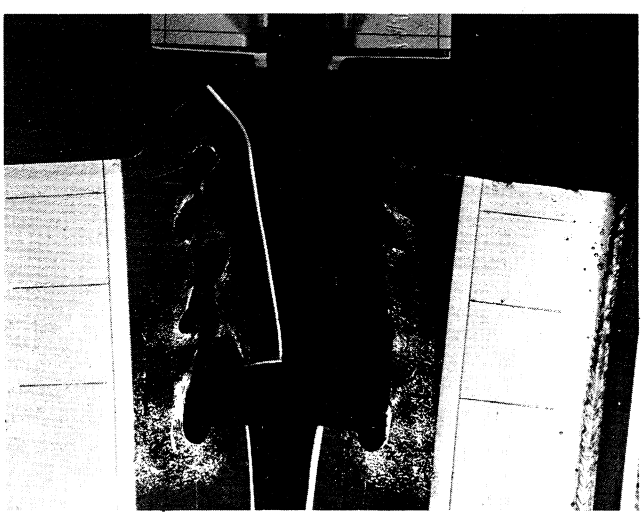

Fig. 9. Failure of supplemental test

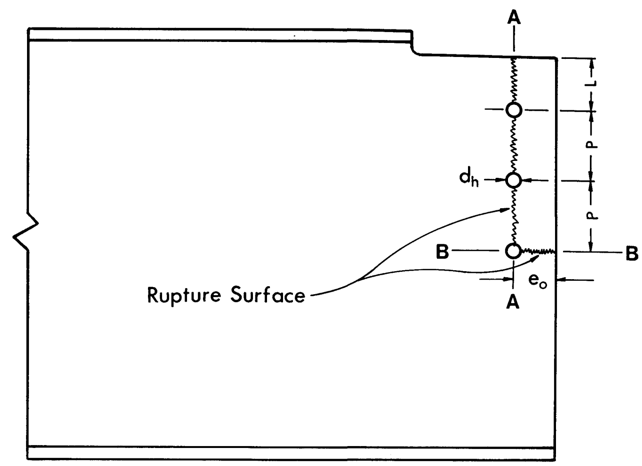

Fig. 10. Block shear model of failure

LAP SPLICE TESTS

A 9-in. wide piece of the beam web was removed and two lap splices fabricated, Tests III-1 and III-2. In the first, a $^3/_4$-in. diameter A325 single-bolt lap splice with an end distance of 1.75 in. failed by tearing out at 47 kips, or 2.23 times the working load predicted by the 1976 RCRBSJ Specification. The second, with three bolts with a pitch of $4d$ and an end distance of $3d$ failed by excessive hole deformation and eventual rupture at 152 kips, or 152/81.3 = 1.87 times the permitted working load; these results were in good agreement with the similar work reported by Fisher and Struik.[1] Table 3 lists the results of these and the preceding tests.

FAILURE MODE

As observed in Fig. 9, the predominant mode of failure in the coped tests is a shearing out of a block of the web, a "block shear" failure, before the bearing strength of the web is reached. Figure 10 illustrates a proposed model failure surface, where the resistance to "block shear" is provided by the tensile resistance of the web across plane BB and the shear resistance of the web along plane AA.

Table 3. Failure Load Summary

Test[a]	L/d	Failure Load (kips)
I-1	—	147
I-2	3.0	112
II-1	3.0	113
II-2	3.0	108
III-1	2.3	47
III-2	3.0	152

[a] Series I—denotes full size beam tests; II—denotes supplemental tests on coped region; III—denotes tests of simple tensile lap splices.

The shear stress at failure was determined from the tests and expressed as a ratio of the ultimate tensile stress. This determination is sensitive to the shear area assumed effective in resisting load; thus, from the single-bolt lap splice, specimen III-1, if the shear area is taken as the end distance, L, minus one-half the hole diameter,

$$\frac{\tau_u}{F_u} = \frac{47}{2 \times \left(1.75 - \frac{13/16}{2}\right) \times 0.305 \times 79} = 0.73$$

If, however, the shear area is taken as only the end distance, L,

$$\frac{\tau_u}{F_u} = \frac{47}{2 \times 1.75 \times 0.305 \times 79} = 0.56$$

Thus, for strength predictions, a conservative value of τ_u / F_u could be taken as 0.60.

Computing the resistance to "block shear" for the coped-beam web configuration in Fig. 10 for:

L = 2.25 in.
p = 3.0 in.
d_h = 13/16-in.
e_o = 1.75 in.

The shear area, A_s, is computed as:

$$\left[(2.25 + 3 + 3) - 2.5 \times \frac{13}{16}\right] \times 0.305 = 1.90 \text{ in.}^2$$

The net tensile area is:

$$\left[1.75 - \left(\frac{1}{2} \times \frac{13}{16}\right)\right] \times 0.305 = 0.41 \text{ in.}^2$$

Thus, the ultimate load is:

$$\begin{aligned} P_{ult} &= \text{shear resistance} + \text{tensile resistance} \\ &= [1.90 \times 0.6\ (79)] + (0.41 \times 79) \\ &= 122 \text{ kips} \end{aligned}$$

The average of the coped-beam tests was 112 kips.

If, for design, all the area is considered as shear area, then:

$$\begin{aligned} P_{ult} &= (1.90 + 0.41)[0.6\ (79)] \\ &= 110 \text{ kips} < 112 \text{ kips from tests.} \end{aligned}$$

Alternately, computing the shear area over the entire length of rupture minus the 3 holes, taken as the bolt diameter + ⅛-in., then:

$$\begin{aligned} P_{ult} &= [8.25 + 1.75 - 3(7/8)] \times 0.305 \times 0.6\ (79) \\ &= 107 \text{ kips} < 112 \text{ kips from tests.} \end{aligned}$$

SUMMARY

An evaluation of the application of the newly recommended higher bolt shear and bearing stresses was made with several tests of bolted double-angle beam connections, with top flanges both coped and uncoped. The findings are summarized as follows:

1. The presence of the top flange of a beam is important for the development of the web bearing strength derived from tests of tensile lap splices.
2. The effect of eccentricity on short, single line, bolted web connections may cause a high component of force towards the end of the beam and thus the critical dimension related to potential tear out would be the horizontal distance from the bolt line to the end of the beam.
3. For beams in which the top flange is coped, a potential failure mode of tearing out may occur in the web before the nominal ultimate bearing stress is attained; an intuitive failure model which was also observed in the tests shows reasonable agreement with the tests results reported herein.
4. Although the study presented here focuses primarily on ultimate load behavior, permanent or residual deformation at service loads attributable to bearing deformation may be unsuitable. For the uncoped beam test, the residual displacement at a working load is estimated to be on the order of 0.1–0.2 in.

The test results, which are presented here, focus on the behavior of only one connection with two detailing alternatives (coped and uncoped). For many framing applications, the loading and detailing practice may be such that proportions which develop the failure mechanism demonstrated here will not occur. It is, however, prudent to exercise caution in the application of the new bearing strength criteria to the wide variety of connection detailing for which the criteria have not been tested. It is further recommended that research be continued to develop design guidelines for the broader application of ultimate strength concepts in connection design.

ACKNOWLEDGMENTS

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SYMBOLS OR NOMENCLATURE

d = nominal bolt diameter
d_h = bolt hole diameter
e_o = edge distance
F_u = specified minimum tensile strength
F_y = specified minimum yield point
L = end distance
p = pitch
P_{ult} = ultimate reaction
τ_u = ultimate shear stress

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