Examination of AISC LRFD Shear Lag Design Provisions

WILLAM J. KIRKHAM and THOMAS H. MILLER

INTRODUCTION

Steel structures are often assembled from standard steel shapes connected by welds or other fasteners. Sometimes, a number of bolts or welds are used to connect all the elements (flanges, web, etc.) of a member to the elements of another member. In most structures, however, not all of the elements of a given cross-section are connected, for numerous reasons. When some elements of a member are not connected, shear lag may result.

Typically, in cross-bracing of trusses and frames (see Figure 1), an engineer determines the geometry, strength and economical design of the connections between major load-carrying members, joined by one or more gusset plates. When the braces enter the joint at an angle, often little clearance is available to effectively attach the web of the brace. The sizing of this bracing member is determined in part by the efficiency of directly connecting only the flanges to the gusset plates.

The transfer of forces and shear lag can be easily visualized in the case of a channel (Figure 2). Concentrated forces, simulating fillet weld connections, transfer tension into the channel. The channel is symmetric about its midspan, with only one end shown. A transfer of stresses occurs with distance from the point of application that results in a net tensile force at the midspan.

The term shear lag describes the process of transferring stresses from the concentrated forces to the cross-section of the channel. "Shear" refers to the combination of opposing stresses that transmit the edge forces from the welds to the interior elements. "Lag" indicates that there is a delay in effect, in this case, in approaching fully-developed normal stresses until reaching a sufficient distance from the application of the concentrated forces. When members are not fully connected (through all of their elements), stresses are especially concentrated in the area of the connection. In Figure 2, contours of equal longitudinal normal stress show the change in stresses along the length of the member, as well as areas of stress concentration near the points of application. As Saint Venant's Principle (Timoshenko, 1953) indicates, stresses become relatively uniform across the web at sufficiently large distances from the point of application of the forces. This is usually about two to five times the overall width of the section.

The channel flanges also show the effects of the stress redistribution. Stresses are concentrated at the points of load application. The flanges differ from the web, though,

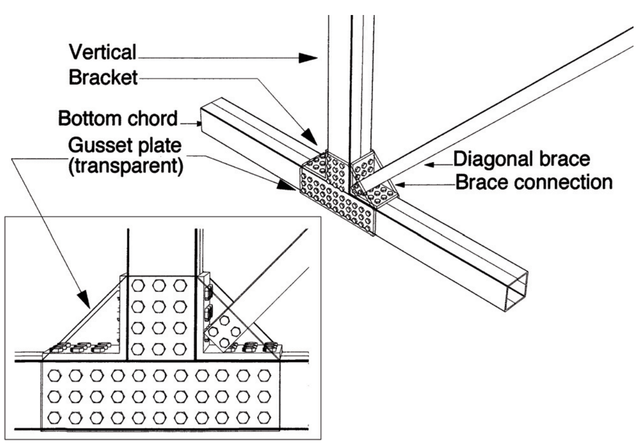

Fig. 1. Gusset plates attaching truss members.

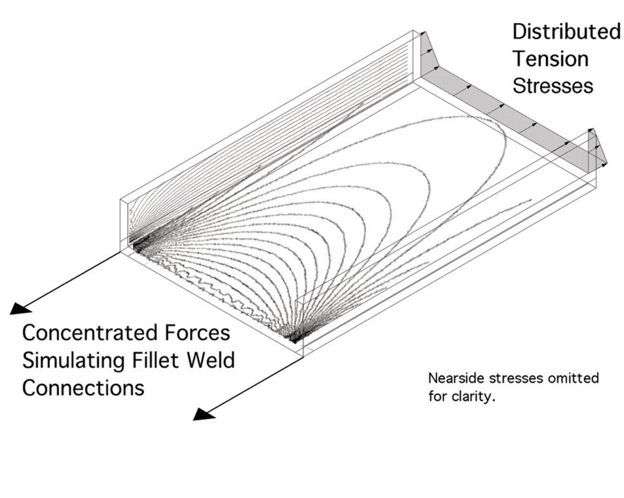

Fig. 2. Distribution of longitudinal normal stresses in a channel.

William J. Kirkham is design engineer, KPFF Consulting Engineers, Portland, OR.

Thomas H. Miller is associate professor, department of civil, construction and environmental engineering, Corvallis, OR.

because the stresses never fully equalize across the width of the flanges, due to the eccentric loading applied to the channel by the connection. Shear lag also involves a non-linear (inelastic) redistribution of stresses. If welded and bolted connections are properly and economically designed, this places a bound on the ratio between localized average stress and the yield stress. These effects of non-linear material behavior, different types of connections (bolts, rivets and welds) and eccentric application of longitudinally applied forces further complicate analysis of these situations. Closed-form solutions for real applications are not available, and finite element models for a single connection cannot generally be justified. Research on particular aspects of this problem led to development of empirical equations and "rules of thumb" for design. These are the basis of portions of the American Institute of Steel Construction (AISC) LRFD Specification and Commentary (AISC, 1993a,b) provisions for shear lag and tension member strength.

SPECIFICATION REQUIREMENTS

The 1993 LRFD Specification uses the concept of "effective net area" to account for the inefficiencies in shear lag-affected connections. Depending on the type of connection, various empirical formulae and "rules of thumb" are provided.

For riveted, bolted or welded shapes, other than plates, an efficiency, U, is calculated from the connection eccentricity $\bar{x}$ and the connection length l:

$$U = 1 - \frac{\bar{x}}{l} \leq 0.9 \qquad \text{(B3-2)}$$

The effective net area is then calculated as:

$$A_e = AU \qquad \text{(B3-1)}$$

For bolted or riveted shapes, A is the net area, A_n, and for welded shapes it is the gross cross-sectional area of the member, A_g.

Chesson and Munse (1963) proposed Equation B3-2 as a means of estimating the shear lag effect on tension member strength. Figure 3 shows the definition of l for calculating shear lag effects. Unfortunately, the definition of $\bar{x}$ is not quite as clear in the 1993 LRFD Commentary as illustrated by Figures 3 and 4. The value of $\bar{x}$ was the location of the centroid measured from the gusset plate or the plane of connection. The 1993 LRFD Commentary shows diagrams that the lead to calculations of $\bar{x}$ as the distance between one of the bolts and the centroid of the unconnected portion of the member. Refer to Figure 3 and Figures 4b and 4c. The distance to this non-centroidal $\bar{x}$ is measured parallel to the plane of the connection. The 1993 LRFD Commentary also states that the maximum $\bar{x}$ be used where two values are examined for a section.

The Commentary to the 1999 AISC LRFD Specification (AISC, forthcoming) deletes the non-centroidal $\bar{x}$ portions of the figures (see Figures 3 and 4) because the test data did not support this usage.

For longitudinally welded flat plates, U is either 0.75, 0.87 or 1.00, depending on the connection length.

In addition, other methods for evaluating shear lag effects are found in the 1993 LRFD Specification, but are not the subject of this paper.

COMMENTARY PROVISIONS

The 1993 LRFD Commentary indicates that "Previous issues of this Specification have presented values for U for bolted or riveted connections of W, M, and S shapes…These values are acceptable for use in lieu of calculated values from Equation B3-2 and are retained here for the convenience of designers."*

The alternative provisions for bolted or riveted connections presented on pp. 6-172 through 6-174 of the 1993 LRFD Commentary provide a method for determining U values (0.90, 0.85 or 0.75) based on classification of a connection by section type, section geometry, and the number of fasteners in a line. This historic approach was maintained in the 1993 LRFD Commentary for use in specific appropriate instances. It does not require a calculation for U, and it is not affected by the positions of the holes in the members. A designer using the 1993 LRFD Commentary would thus not need to employ a trial-and-error process to design a connection, as is often needed using Equatoin B3-2.

OBJECTIVES

The 1993 AISC LRFD Specification and Commentary provide two different methods for evaluating the shear lag effects on connections in a tension member. Depending on the particular design, differing strengths can be predicted by the two methods. In some cases, calculation of an extremely low strength is possible using Equation B3-2. Some sections of the 1993 LRFD Specification and Commentary appear to be logical extensions of existing research and practice that have not been verified by experimental results.

This paper will examine the state of knowledge in this area and discuss sections of the 1993 AISC LRFD Specification and Commentary to aid in refining them and to recommend areas where future research is needed. It will also demonstrate areas of concern and define areas in need of further clarification.

*Note that a typographical error in an early printing of the Commentary incorrectly referenced Equation B3-3.

PREVIOUS RESEARCH

During the 1930s and 1940s, design of high performance military aircraft motivated research efforts on shear lag. Hildebrand (1943) derived closed-form solutions for a number of basic loading conditions in thin-skinned panels by applying a stress analysis approach to stringer sheet theory. Levy, Singer and Baruch (1975) applied energy methods to stringer sheet theory to develop a different closed-form solution, and correlated it to experimental research. Chesson and Munse (1963, 1959, 1958) and Munse and Chesson (1963) performed numerous studies of riveted and bolted connections, usually to determine connection strengths. They developed a design approach which is included in the 1993 LRFD Specification as Equations B3-1 and B3-2. Fuller, Leahey and Munse (1955) tested a large S shape connected with rivets and bolts using flat and beveled washers. Foreman and Rumpf (1961) tested compact bolted joints in large plates. Compact joints were made using ASTM A325 bolts, with gage-to-hole diameter and pitch-to-hole diameter ratios of approximately four, and connection lengths not more than five times the pitch. Davis, Woodruff and Davis (1940) performed tension tests of 40 large riveted plate joints. The tests were designed to determine the effects of net width, end pitch, plate thickness, joint length, combinations of rivet and plate steel types, and pitch in various lap, butt and shingle splices. Gibson and Wake (1942) performed tension tests of 54 welded angles, all 2.5×2.5×5/16, showing shear lag effects. Butler and Kulak (1971) studied fillet weld strength as a function of load direction, and Butler, Pal and Kulak (1972) researched eccentrically loaded welded connections, where the load is applied and acts in the plane of the weld. Davis and Boomslitter (1934) researched balanced and unbalanced welds in numerous configurations, all with 3×3×5/16 angles.

Easterling and Giroux (1993) and Gonzalez and Easterling (1989) applied modern methods to the problem of welded plates, tees, channels and angles. Each specimen was designed according to the 1986 edition of the LRFD Specification (AISC, 1986), so that the weld strength would be 10 to 15 percent greater than the gross section tensile strength of the member, to ensure a shear lag-affected tensile failure. Their study is outstanding in the clarity of presentation, methods of analysis and in the detail of recorded data. Strain gauges were applied to record stress distributions in the members. Longitudinal displacement was measured and recorded. Out of plane bending was meas-

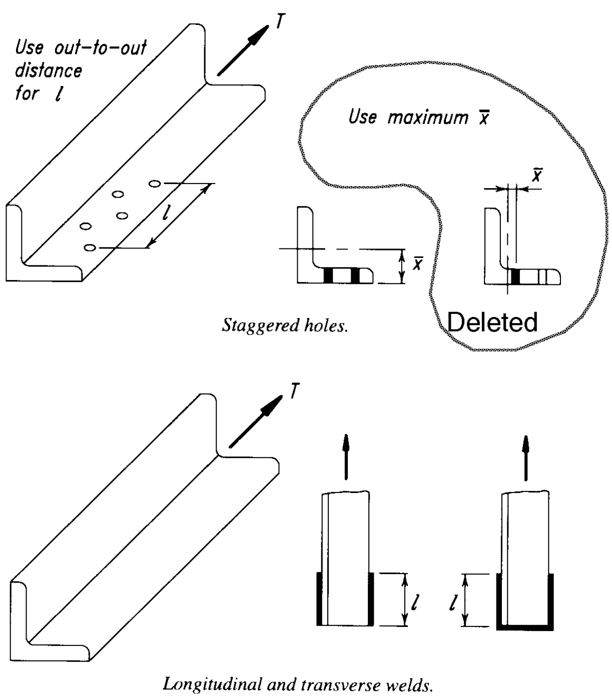

Fig. 3. Definition of l for bolted and welded angle connections. (AISC, 1993b)

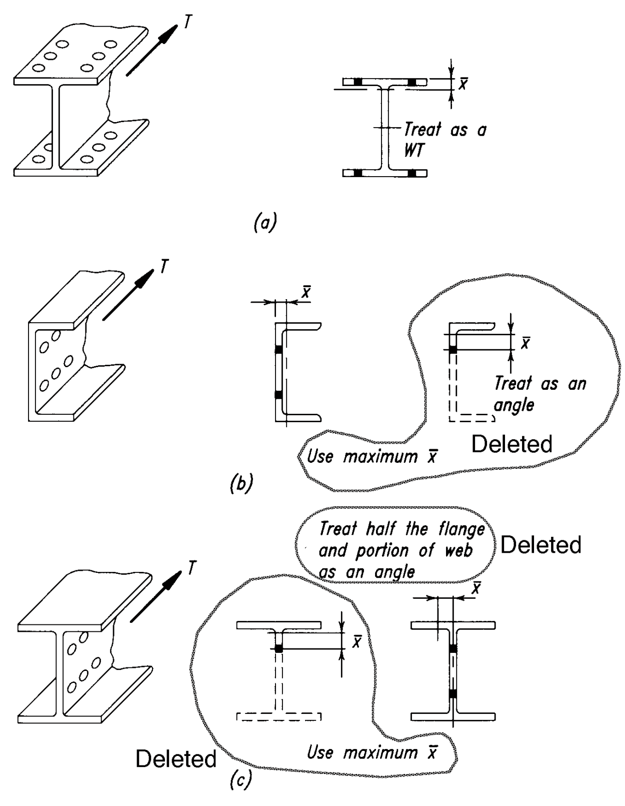

Fig. 4. Definition of $\bar{x}$. (AISC, 1993b)

ured at the midpoint of the member. Loading was applied incrementally, and data were recorded for each increment. A linear-elastic finite element model, using ANSYS, was developed and compared to the results of the experiments.

Overall, their studies demonstrated that the shear lag provisions for bolted connections were also appropriate for the welded plate and angle configurations considered. Of 11 plate configurations, nine failed due to shear lag as expected and two exceeded the hydraulic test machine capacity. Nine angle configurations were tested with two weld failures and seven shear lag failures. However, of 11 channel configurations tested, five failed in the welds and five suffered gross cross-section failure away from the connection. Of 10 tee configurations, seven failed in the welds.

Kulak and Wu (1997) tested 24 bolted single and double angle specimens. Gusset plates of varying stiffness were used in the single angle tests. Stress-strain behavior of the angles was observed under incremental loading. The formation of gaps between the angle and gusset plate, as well as the process of crack formation and spreading are clearly explained. A non-linear finite element model was developed using ANSYS, to compare with experimental behavior. Design recommendations are provided.

Finally, Cheng, Kulak and Khoo (1998) tested nine round hollow structural section (HSS) specimens, slotted and welded to a gusset plate. Tubes were instrumented with linearly variable displacement transducers (LVDT) and strain gages. A finite element model was developed using ABAQUS, with an isotropic hardening plasticity model, and its behavior compared to the experimental data.

LIMITATIONS OF PREVIOUS STUDIES

Table 1 shows the shapes tested in the major studies. Multiple samples are frequently cut from one long standard piece, so the total number of shapes tested in a study is often limited. The researchers often performed many tests for each shape, varying the connection geometry (lines and rows of bolts, balanced and unbalanced welds), method of connection (rivets, bolts, welds), and method of fabrication (drilled, punched, welded). In some studies, members were tested singly (allowing gusset plates to flex), where in other studies the members are connected symmetrically to opposite sides of the gusset plate, effectively producing fixed end conditions.

A total of 11 L shapes were tested, varying in leg size from 2 in. to 5 in. Thicknesses varied from 3/16 in. to 7/16 in. Two C shapes were tested. Two WT shapes were tested. One built-up H shape was tested, and one S shape was tested. Three HSS shapes were tested. Almost 100 plate tests were examined for this study, though other less relevant reports are available. The exact total may exceed 200 when riveted plate tests are included.

To categorize the geometry of tested members from these previous studies, the authors of this paper define the "aspect ratio" as the ratio of the height of the unconnected element to the width of the connected element (see Figure 5):

$$\text{Aspect ratio} = \frac{H}{B}$$

For vertically symmetric shapes, the height is measured to the line of symmetry.

Table 2 shows a summary of the shapes, tested aspect ratios and the range from minimum to maximum aspect ratio for the standard structural shapes (AISC, 1993c) (see Figure 6 for descriptions of connection geometry). Although there is some distribution of tested aspect ratios, most are near the mid-points of the available standard section ranges. It is also interesting to note that no wide flange W shapes have been tested.

Lengths of members were often much shorter than those used in most structures (Table 3). The longest tested specimen was only 4.75 ft in length. These are probably due to testing machine capacity and size limitations. Some test-

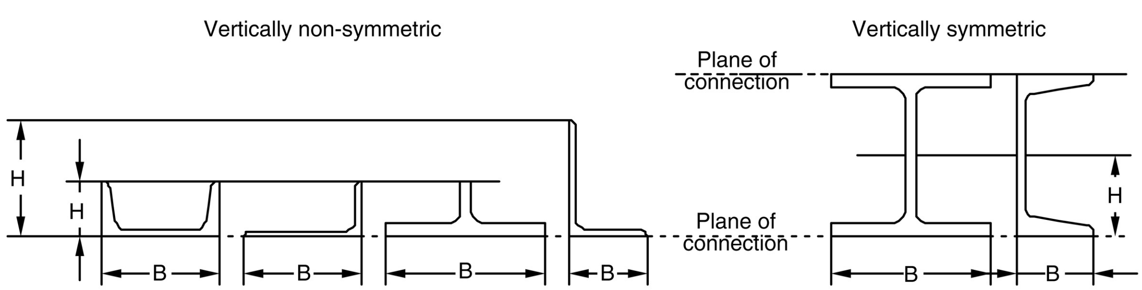

Fig. 5. Definition of aspect ratio.

ed members may in fact be so short that the stress distributions are affected, and errors introduced in the experimental results.

Because Saint Venant's Principle (Timoshenko, 1953) applies to the connection, rather than the member length, Table 3 compares the unattached member length from the connection to mid-span, *l*/2, with the width, *w*. Saint Venant's Principle (Timoshenko, 1953) would suggest that the length be several times the width, to provide fully developed stresses.

More than 50 percent of the tested shapes fall below a ratio of 2, and almost 90 percent are less than 3. Only 10 percent of the tested specimens are long enough to fully develop stresses at the midpoint of the member.

Several observations can be made concerning the previous research efforts:

1. Most of the 1993 AISC LRFD Specification and Commentary provisions for determining shear lag effects are derived from tests on only a few basic sec-

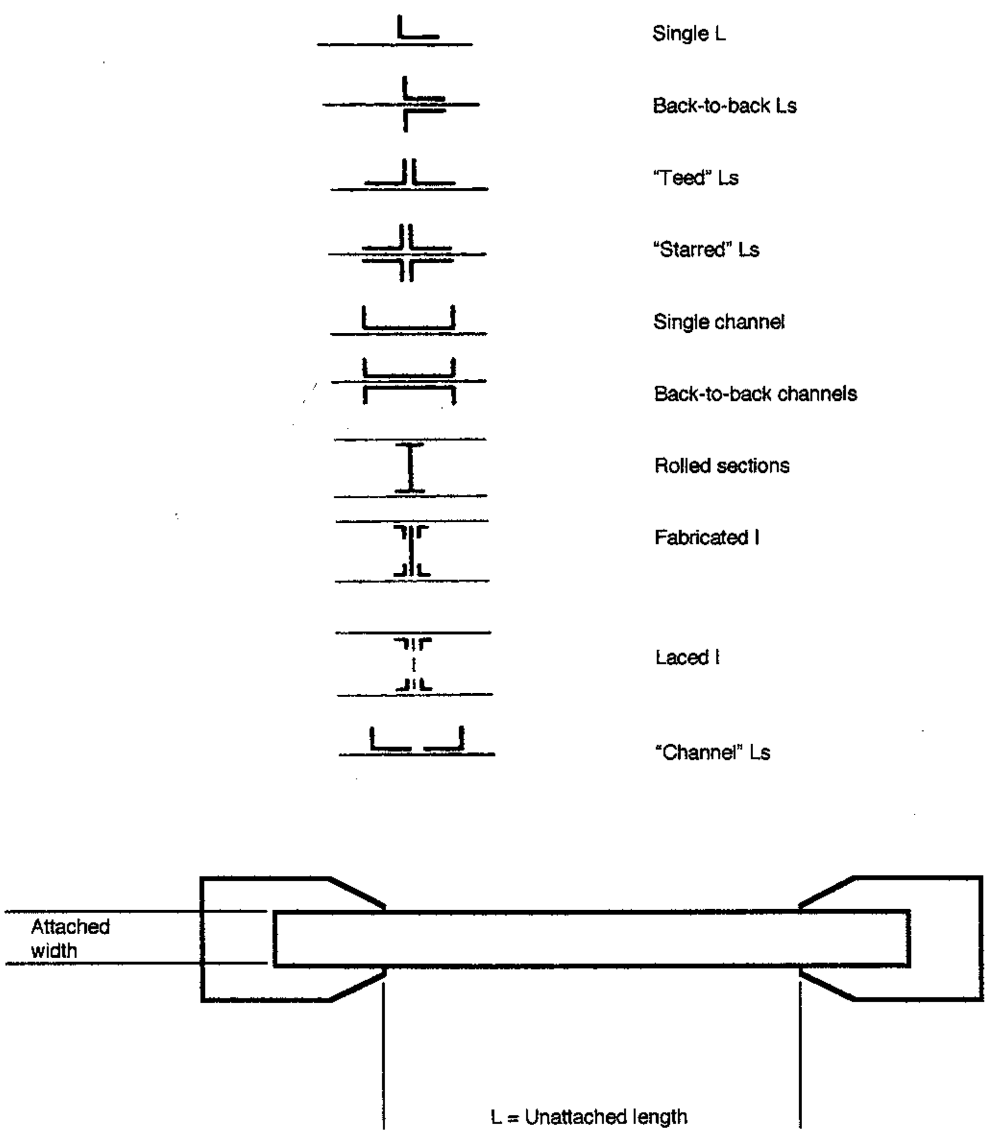

Fig. 6. Configuration types adapted from Chesson and Munse (1963), Table 5.

Table 1.
Tested Shapes in Previous Research

	Purpose of Study	Angles	Plates	I Shapes	Channels	Tees	HSS
Butler & Kulak (1971); Butler, Pal & Kulak (1972)	Fillet weld strength as a function of load		Various				
Cheng, Kulak & Khoo (1998)	Shear lag in round HSS members						HSS 4×1/4 HSS 4×3/16 HSS 8 5/8×5/16
Chesson & Munse (1958; 1959)	Test "...full-sized angle, and built-up truss-type members...", to "...provide a means of accurately computing their strengths, based on geometries."	L3.5×3.5×7/16 L5×5×3/8 L5×3×3/8 [a,b]		Built-up I section with (4) L5×3×3/8 for flanges and a PL1/2×16 web [a]			
Davis & Boomslitter (1934)	"...determine the strength of tension members composed of angles with both welded and riveted joints."	L3×3×5/16					
Davis, Woodruff & Davis (1940)	"…tests on large riveted joints…[to describe] the behavior of the various parts…"		Plates 3/8-5/8"× 18-20"				
Fuller, Leahey & Munse (1955)	Test large I-Sections with truss-type connections.			S18×54.7			
Gibson & Wake (1942)	Test balanced and unbalanced weld configurations.	L2.5×2.5×5/16					
Gonzalez & Easterling (1989); Easterling & Giroux (1993)	Verify "...application of the shear lag requirements to welded members..."	L2×2×3/16 L4×3×1/4	PL3/8×4 PL1/4×3		C3×4.1 C4×5.4	WT5×6 WT3×4.5	
Kulak & Wu (1997)	Study shear lag in angles	L4×4×1/4 L3×3×3/16 L3×2×3/8 L3×2×3/16					

[a] Long legs attached to gusset plate
[b] Bolts attach angles back-to-back

Table 2. Aspect Ratios of Tested Shapes

	H	B	Aspect ratio Tested	Aspect ratio Range [a]	H	B	Aspect ratio Tested	Aspect ratio Range [a]
Angles	Long-leg attached			0.5-1.0	Short-leg attached			1.0-2.0
L2×2×3/16	2.00	2.00	1.00		2.00	2.00	1.00	
L2.5×2.5×5/16	2.50	2.50	1.00		2.50	2.50	1.00	
L3×2×3/8 & 3/16	2.00	3.00	0.67		–	–	–	
L3×3×5/16 & 3/16	3.00	3.00	1.00		3.00	3.00	1.00	
L3.5×3.5×7/16	3.50	3.50	1.00		3.50	3.50	1.00	
L4×3×1/4	3.00	4.00	0.75		4.00	3.00	1.33	
L4×4×1/4	4.00	4.00	1.00		4.00	4.00	1.00	
L5×3×3/8	3.00	5.00	0.60		5.00	3.00	1.67	
L5×5×3/8	5.00	5.00	1.00		5.00	5.00	1.00	
Plates and Bars				[b]				
PL 3/8×4	3/8	4	0.09					
PL ¼×3	1/4	3	0.08					
I-Beams				0.47-1.96				
S18×54.7	18	6.001	1.50					
Built-up I section with (4) L5×3×3/8 for flanges and a PL1/2×16 web	16	10.5	0.76					
Channels	Web-connected			0.13-0.58	Flange-connected			0.77-5.79
C3×4.1	1.41	3	0.47					
C4×5.4	1.584	4	0.40					
Tees	Flange-connected			0.47-1.96				
WT5× 6	4.94	3.96	1.25					
WT3×4.5	2.95	3.94	0.75					
HSS				0.50 [c]				
HSS 4×1/4, 3/16	2.00	4.00	0.50					
HSS 8 5/8×5/16	4.31	8.62	0.50					

[a] Range of possible values for standard shapes

[b] Plates and bars vary greatly in dimensions, no range of values is provided

[c] Installing a gusset plate symmetrically through a non-rectangular tube always results in an aspect ratio of 0.5

Sources: Data from Butler and Kulak (1971), Butler, Pal and Kulak (1972), Chang, Kulak and Khoo (1998), Chesson and Munse (1958, 1959), Davis and Boomslitter (1934), Easterling and Giroux (1993), Fuller, Leahey and Munse (1955), Gibson and Wake (1942), Gonzalez and Easterling (1989), and Kulak and Wu (1997).

Table 3. Tested Specimen Dimensions					
Study	**Specimen**	**Type**	**Attached Width (in.)**	**Unattached Length (in.)**	**$l/(2w)$**
Chesson & Munse (1963)	SA	Starred Ls	7.0	10.0	0.71
	SB	Starred Ls	10.0	10.0	0.50
	SD	Starred Ls	10.0	10.0	0.50
	SE	Starred Ls	10.0	10.0	0.50
	A & AM	Fab. I	16.0	28.0	0.88
	C & CM	Fab. I	10.0	28.0	1.40
Chesson & Munse (1958, 1959)	AD1, AD2	Fab. I	7.0	30.5	2.18
	BD1-2, BP1-2	Laced I	10.0	28.0	1.40
	CD1, CD2	Fab. I	10.0	28.0	1.40
	DD1-2, DP1-2	Laced I	10.0	28.0	1.40
	ED1-2, EP1-2	Laced I	10.0	28.0	1.40
Davis & Boomslitter (1934)	R1	1 L	3.0	16.0	2.67
	R2	2 Ls (Teed)	6.0	16.0	1.33
	R3	2 Ls (Back)	3.0	11.5	1.92
	R4	2 Ls (Chan)	3.0	11.5	1.92
	R5	4 Ls	3.0	11.5	1.92
	W1	1 L	3.0	19.5	3.25
	W2	2 Ls (Teed)	6.0	18.0	1.50
	W3	2 Ls (Back)	3.0	14.5	2.42
	W4	2 Ls (Chan)	3.0	14.5	2.42
	W5	4 Ls	3.0	14.5	2.42
Fuller, et al. (1955)	S18x54.7	S	6.0	24.0	2.00
Foreman & Rumpf (1961)	B1	Plates	18.0	36.0	1.00
	B2	Plates	18.0	43.0	1.19
	B3	Plates	18.0	50.0	1.39
	B4	Plates	18.0	43.0	1.19
	B5	Plates	18.0	43.0	1.19
	B6	Plates	18.0	57.0	1.58
	BR2	Plates	18.0	43.0	1.19
	A3	Plates	18.0	45.0	1.25
	G1	Plates	18.0	53.0	1.47
Gibson & Wake (1942)		1 L	2.5	30.0	6.00
Gonzalez & Easterling (1989); Easterling & Giroux (1993) (same data)	PL 4x3/8	Plate	4.0	13.0	1.63
	PL 3x1/4	Plate	3.0	15.5	2.58
	PL 3x1/4	Plate	3.0	14.0	2.33
	PL 3x1/4	Plate	3.0	18.0	3.00
	L2x2x3/16	2 Ls (Back)	2.0	15.0	3.75
	L4x3x1/4	2 Ls (Back)	4.0	17.0	2.13
	L2x2x3/16	2 Ls (Back)	2.0	18.0	4.50
	L4x3x1/4	2 Ls (Back)	4.0	24.0	3.00
	C3x4.1	2 Cs (Back)	3.0	11.0	1.83
	C3x4.1	2 Cs (Back)	3.0	14.0	2.33
	C4x5.4	2 Cs (Back)	4.0	24.0	3.00
	C3x4.1	2 Cs (Back)	3.0	24.0	4.00
	WT5x6	2 Ts (Back)	5.0	13.0	1.30
	WT3x4.5	2 Ts (Back)	3.0	11.0	1.83
	WT5x6	2 Ts (Back)	5.0	17.0	1.70
	WT3x4.5	2 Ts (Back)	3.0	15.0	2.50
	WT5x6	2 Ts (Back)	5.0	24.0	2.40
	WT3x4.5	2 Ts (Back)	3.0	24.0	4.00
		Average	7.57	23.29	2.03
		Maximum	18.00	57.00	6.00
		Minimum	2.00	10.00	0.50
		Std. Deviation	5.75	12.52	1.09

tion geometries, generally near aspect ratios (height-to-width ratios) of 1.

2. Most tests involve the smaller sizes of each structural section type.
3. In each study, the researchers varied the method of attachment, but tested only a few different structural shapes. They were primarily interested in studying the strengths of different connection types (i.e. bolts, rivets or welds), rather than the effect of the member geometry on the eccentricity of the connection. No rolled W, M, HP, MC, MT or ST sections were tested.
4. Tests of longer members would be desirable to verify the general applicability of the previous research. Ideally, full length members should be tested to simulate the actual field situations. Shorter lengths may cause increased stresses that would result in overconservative design methods. These observations, however, are not surprising, considering the goals of the various research projects—usually to study connection behavior rather than the effects of various member geometries. In many cases, the equipment to conduct full scale tests was unavailable, and handling large-scale specimens was difficult or infeasible. However, studies of at least some full-scale members should now be performed to verify that the existing research database on small, short members is truly applicable to the design of full-scale structures.

COMPARISON OF SPECIFICATION AND COMMENTARY PROVISIONS

The 1993 LRFD Specification and Commentary provisions are based on the best available research and sound engineering judgment, and therefore, provisions from different sections should be comparable and provide consistent results.

In Figure 7, an angle and a plate are shown. The width and thickness of the plate are identical to those of the attached leg of the angle. How do the efficiencies of the angle and plate compare using the 1993 LRFD Specification provisions?

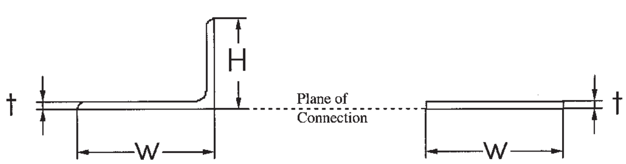

Fig. 7. Angle versus a plate with similar dimensions.

As the height, H, of the unattached leg is reduced, the angle should behave more like a plate. In fact, U values for longitudinally welded plates are larger than those for the angle when the longitudinal welds are long. For example, a plate with longitudinal welds twice as long as the width of the plate has a value of $U = 1.00$, while an angle can never have $U > 0.9$. Thus, the 1993 LRFD Specification provisions can predict in some cases that the additional area of a small unattached leg actually reduces the effective area (A_e) of the entire angle below that of a plate with similar dimensions. Although the small unattached leg introduces a minor eccentricity, this does not seem consistent. In fact, the 1993 LRFD Commentary indicates that "…it is probably conservative to use A_e equal to the net area of the connected element…" for connections with only one bolt per line, suggesting a lower limit for reasonable U values.

Flange-connected channels also result in some cases where entire members have less effective area (A_e) than the flanges alone treated as plates. See Figure 8 and Example 1 in the Appendix. The "no web participation" line divides these cases. The effective area (A_e) of the channel is calculated using Equation B3-2 and compared to the effective area (A_e) of two plates identical in dimensions to the flanges alone. In this study, welds were designed to provide the same tensile resistance as that of the member, through an iterative process. In each iteration, the weld length is reduced to provide a weld strength (ϕR_n) equal to the tensile strength (ϕT_n) of the member, reduced for the shear lag effect. Then, the member tensile strength (ϕT_n) is recomputed based on the previously reduced weld length. Because the weld is matched to the member strength, the yield strength of the steel (F_y) can affect the length of the weld. Figure 8 shows the comparison of the effective areas. Points above the diagonal line indicate shapes where the effective area (A_e) of the entire channel is greater than the effective area (A_e) of the flanges alone, treated as plates. This is what one would expect. Certain shapes however, fall below the diagonal, indicating that the flanges alone would have a greater effective area (A_e) according to the 1993 Specification. This does not seem reasonable, assuming concentric loading, and is reason for concern. Thus, according to the 1993 LRFD Specification, the additional area of the web reduces the overall effective area (A_e) of some channels, compared to the flanges alone, treated as plates.

Low and Negative *U* Values

For the net effective area of the channel to be less than that of the flanges treated as plates alone, the U value for the channel must be very low. Figure 9 demonstrates that negative U values in flange-connected channels (C shapes) can occur with aspect ratios above 2.8. These are standard available steel shapes with fully designed connections equal to

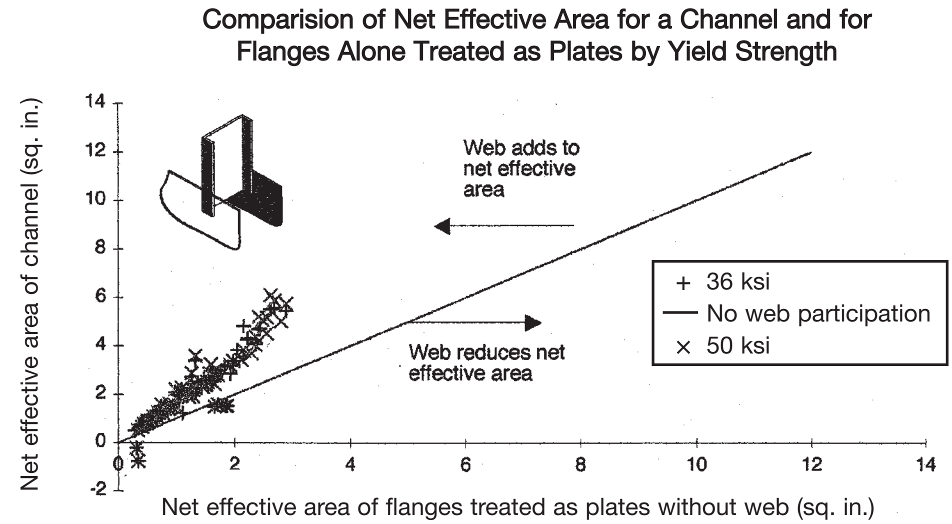

Fig. 8. Comparing net effective areas of a channel to its flanges alone treated as plates.

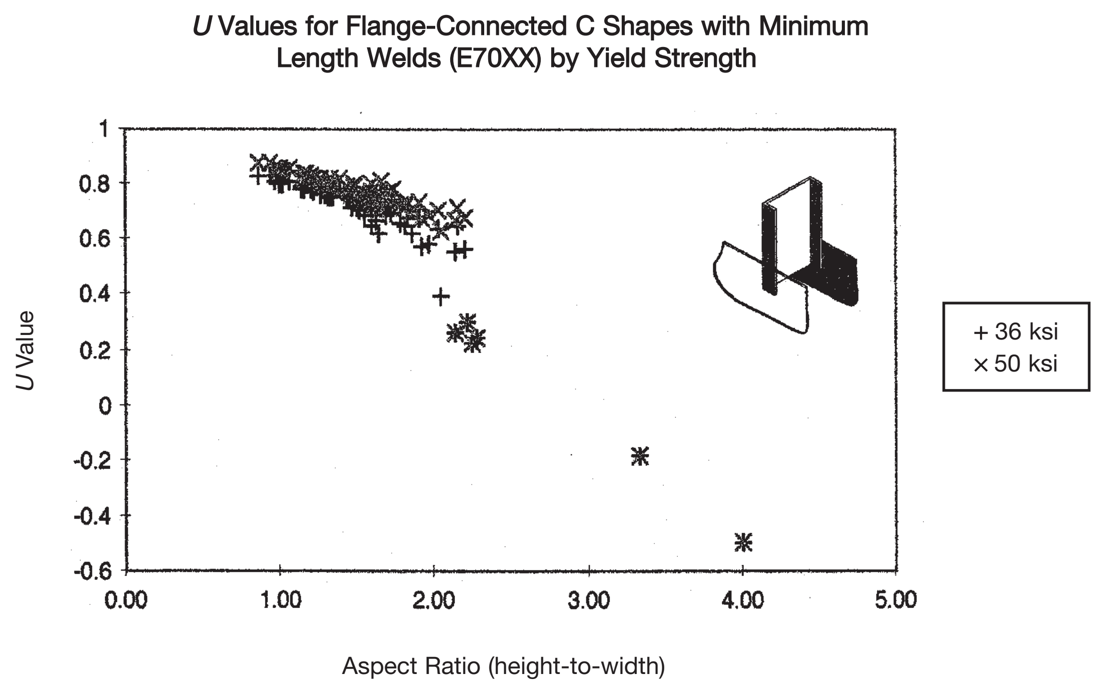

Fig. 9. Range of U values for flange-connected channels.

the design tensile strength of the member, the smaller of $\phi F_y A_g$ or $\phi F_u A_e$. Furthermore, calculated U values drop below 0.60 at aspect ratios of 2.0, even though researchers (Chesson and Munse, 1963; Kulak and Wu, 1997; Easterling and Giroux, 1993; Gonzalez and Easterling, 1989) have not observed U values below about 0.7 for members with shear lag failures. More than 50 percent of the shapes examined had calculated U values below 0.7. This effect is due to the value of $\bar{x}$ being large in relation to the weld length. The U value can lose its physical meaning when this happens (see Example 2 in the Appendix).

Negative U values may be an indication that connections are unreasonably short. It is the concern of the authors that there is no quantitative limit that defines the shortest reasonable connection for shear lag effects. Several shapes in Figure 9 can be designed with calculated U values of 0.2 to 0.4, well outside the range of observed values from previous research.

Commentary Provisions

The 1993 LRFD Commentary Section B3 contains the following concluding line: "When a tension load is transmitted by fillet welds to some but not all elements of a cross section, the weld strength will control."

The authors do not understand the reason for this conclusion nor how it should be applied to design. A weld can be made which will not fail until the tension member fractures. Gonzalez and Easterling (1989) tested 41 welded specimens designed to fail due to shear lag. Of these, three would not fail, 14 experienced weld failures and, in the remaining 24, the section fractured. If the 1993 LRFD Commentary statement were literally correct, their research would have shown only weld failures, though numerous fractures of the net section occurred. Clearly, the strength of the fillet weld is not always the controlling factor in the strength of tension members.

The conclusion could also be interpreted as advising the designer to consider the weld strength as the controlling limit state in the design process. Fillet welds are designed with a ϕ of 0.75, while tension member design strength is determined with a ϕ of 0.75 or 0.90, depending on the controlling limit state. Nevertheless, the strength of all elements in the connection must be considered during design. This provision may have been adapted, out of context, from the conclusions of Gonzalez and Easterling (1989) on transverse welds.

Comparison of Specification and Commentary Sections

Designers are permitted to use the historic values in the 1993 LRFD Commentary, rather than 1993 Specification Equation B3-2. In fact, the *LRFD Manual of Steel Construction* (AISC, 1993c) states that, "In lieu of calculating U, the Commentary on the LRFD Specification (Section B3) permits the use of more conservative values of U listed therein." By analyzing all of the WT and MT sections with bolted connections designed using both the 1993 LRFD Specification and Commentary methods, the two approaches can be compared (see Figure 10). For each section, 1993

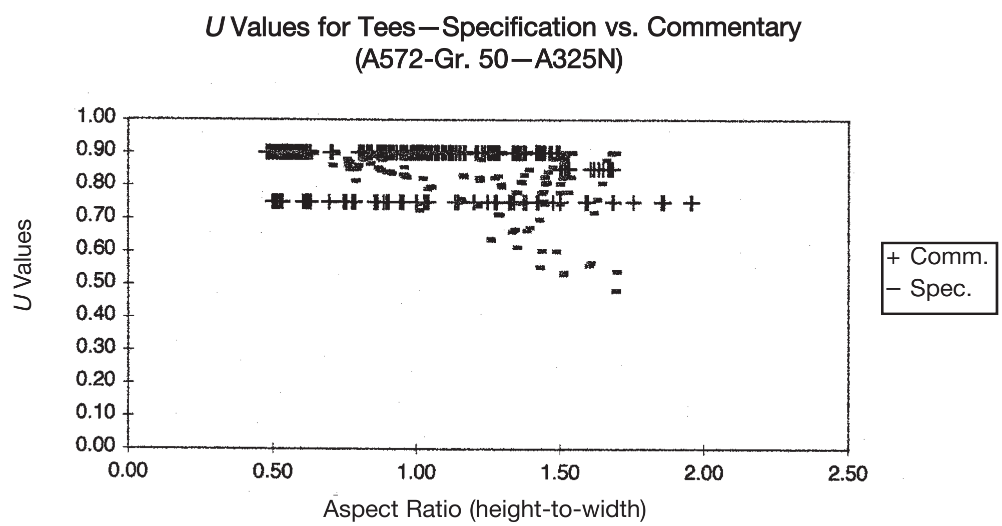

Fig. 10. Comparison of specification and commentary provisions for WT *and* MT *sections.*

LRFD Specification Sections B2, B3, J3 and J4 were used to design connections with the shortest possible length, *l*, for one line of at least two A325 bolts through the flange on each side of the stem. The maximum bolt diameter was limited to 1.5 in. Use of more lines of bolts, larger diameters of bolts or higher strength bolts would shorten the connection length, *l*, and further reduce the 1993 LRFD Specification *U*. As the figure shows, the 1993 LRFD Commentary provisions produce *U* values equal to or higher than the 1993 LRFD Specification provisions in most (but not all) cases and, thus, are not generally a conservative simplification. However, the historic *U* values in the 1993 LRFD Commentary have been used for many years without evidence of problems.

CONCLUSIONS

1. Existing research data are plentiful for angles (L) with aspect ratios (height-to-width of cross-section, as in Figure 5) between 0.60 and 1.67, as well as for plates and bars.
2. Data for web-connected channels (C shapes), flange-connected tees (WT sections), flange-connected I beams (S shapes) are limited to two shapes each, with aspect ratio ranges of 0.40-0.47, 0.75-1.25 and 0.76-1.50, respectively. Actual members can vary well outside of these ranges. The cited studies probably do not provide an adequate basis for demonstrating the applicability of existing provisions outside of these tested ranges.
3. Experimental research on HSS sections has been performed on a limited number of round shapes by a small group of researchers. The cited studies probably do not yet provide an adequate basis for demonstrating the applicability of existing provisions to hollow structural sections in general.
4. Because studies on flange-connected channels (C shapes), stem-connected tees (WT and MT sections), web-connected I beams (S shapes), and all wide flange beams, are nonexistent, there is currently no way to verify the applicability of the 1993 Specification or Commentary provisions to these sections.
5. The 1993 LRFD Specification provisions for shear lag in Section B3 using Equation B3-2 may underestimate the strength of T-section tension members with aspect ratios greater than 1, and flange-connected channels with aspect ratios greater than 1.5, since calculated *U* values can be substantially below 0.75. No researcher has observed *U* values less than 0.75 for the limited range of shapes tested. Very low and negative *U* values can also occur in calculations for these sections (see Figures 9 and 10).
6. The 1993 LRFD Specification and Commentary shear lag provisions are not in agreement concerning calculated values of net effective area, and neither is consistently more conservative. The methods of the 1993 LRFD Specification, which attempt to relate connection geometry factors, appear to have better justification, and are preferred to the 1993 LRFD Commentary methods, which appear to be "rules-of-thumb" that may not be valid for as wide a range of shapes.
7. Research has not yet shown either the 1993 LRFD Specification or Commentary provisions for shear lag to be more accurate in general (Easterling and Giroux, 1993; Gonzalez and Easterling, 1989). A thorough comparison of predicted and tested strengths is needed over a broad range of section types. Most research has examined sections with aspect ratios below 1.25, where both sets of provisions predict efficiencies (*U* values) above 75 percent, and where both are expected to be similar. Significant divergence in the provisions occurs as the 1993 LRFD Specification Equation B3-2 begins to predict efficiencies below 75 percent, the minimum value given by applying the 1993 LRFD Commentary.

RECOMMENDATIONS

Further Research

Well-planned research is needed for a variety of members not previously tested or where only limited research exists. The tests should include:

1. Various aspect ratios (height-to-width of cross-section, as in Figure 5):
   - Wide flange beams (W shapes), both flange and web-connected, with a variety of aspect ratios over the available range (0.4 to 2.0);
   - Tees (WT sections) with a range of aspect ratios, especially those between 0.4 to 0.8 and 1.2 to 2.0;
   - Channels (C shapes) in flange-connected orientations spread over the aspect ratio range of 0.7 to 5.7; and
   - Hollow structural sections (HSS) in round, square and rectangular shaped sections, with aspect ratios between 0.5 and 2.0.
2. Member lengths well over 10 times the width. To provide fully developed stresses at the midspan between the connections, the member length between connections should be much greater than the width of the connection.

3. A study of the effects of various distances between end connections using several different cross-sections. The distance between connections, L, in Figure 6, should vary from one to ten times the width of the member. This will determine the effect that the member length-to-width ratio has on strength and provide a basis for re-evaluating previous research.

The Specification (AISC, 1993a) uses geometric parameters ($\bar{x}$ = distance from the plane of the connection to the centroid of the resisting element and l = connection length) to compute a member efficiency, U, in Equation B3-2. The Commentary (AISC, 1993b) uses the section shape (W, M, S, WT, MT, ST), the ratio of flange breadth to member depth and the number of fasteners in a line as the parameters for determining the member efficiency. Each method uses some common parameters, and each also disregards some important geometric factors. Performing a detailed parameter study will help develop a better unified model for design.

For Consideration by Designers

1. Some engineers may believe it is conservative to design tension members by disregarding the areas of the unattached elements for tension strength calculations. Section B3.2(c) permits this where transverse welds are used to transmit load, and in Section B3.2(d) for plates where $l \geq 2w$. For all other conditions, it is probably reasonable, but there is no research to justify it. At present, however, the designer should check shear lag effects where 1993 LRFD Specification or Commentary provisions apply. The Specification provisions may indicate a shear lag reduced tensile strength for the overall member that is less than the strength for the attached element alone.

2. Designers using sections in tension, with aspect ratios greater than 1, should be aware that the 1993 LRFD Specification and Commentary provisions frequently disagree. For structures with many identical and critical connections, it may be prudent to conduct physical tests of the proposed connections to ensure adequate strength. Finite element methods can also be used, but the designer should be aware of the difficulties others have had in modeling these connections (Gonzalez and Easterling, 1993).

3. Because neither the 1993 LRFD Specification nor the Commentary provisions for shear lag are more conservative in all cases, a prudent designer should consider both before determining the appropriate U value to use.

4. In applying the 1993 LRFD Specification Equation B3-2, a designer should avoid increasing the connection length beyond "typical lengths" as a means of decreasing the shear lag effect. Abnormally long welds may be less efficient than the Specification indicates. Equation B3-2, which is an empirical correlation, indicates that efficiency increases as an inverse linear effect of weld length, whereas Hildebrand (1943) and Levy (Levy, Singer and Baruch, 1975) produced theoretical solutions that indicate an inverse exponential effect.

Possible Changes to the LRFD Specification, Commentary and Manual

Any changes to the 1993 LRFD Specification, Commentary and Manual need to be well documented and carefully thought through. The authors of this paper suggest that AISC consider the following as a basis for discussion of possible changes or clarifications to the Specification or as added explanation in the Commentary or Manual:

1. In Specification Section B3 (d) indicate that, "Fillet weld length shall not be less than the distance between welds." This is typically required for the weld strength to exceed the member strength, and is also recommended by Gonzalez and Easterling (1989).

2. In Specification Section B3, indicate that equation B3-2, "…applies to members with unattached element heights no more than 125 percent and no less than 75 percent of the attached element width." This would restrict the region of application of the provisions to that of the existing research database. Outside of this range, the engineer must apply good judgment in using the equation.

3. Change Example D-3 in the Manual from:

   > In lieu of calculating U, the Commentary on the LRFD Specification (Section B3) permits the use of more conservative values of U listed therein.

   to:

   > In lieu of calculating U, the Commentary on the LRFD Specification (Section B3) permits the use of the values of U listed therein.

   This eliminates the indication that Commentary U values are generally conservative.

4. Better definition of the term "connecting elements" in Specification Section J5 is required. In Section J5.2(b), the net area for tension rupture, A_n must be less than 0.85 A_g, the gross cross-sectional area. This provision is used in Example 11-8 of Vol. 2 of the Manual, where angles connect to tees in a truss. If it is properly applied in this example, the U value of Specification Section B3 can never be more than 0.85. While section

J5.2 indicates application to "…splice and gusset plates…," it does not clearly exclude use for the attached parts of tension members. Example 11-8 explicitly applies this provision to an angle which had a U value of 0.879, reducing it effectively to 0.85.

APPENDIX

Example 1:

Channel can have less tensile strength than two plates equivalent to flanges alone. Connect a C15×40 channel to two plates by welding the flanges. Assume the plates have sufficient strength to transfer any applied loads (see Figure 11). Use the largest permitted fillet welds (E70XX electrodes) with the shortest length permitted by the 1993 LRFD Specification. Use A36 steel.

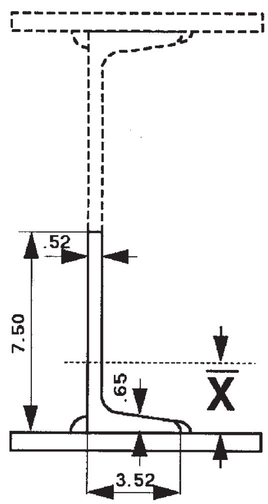

Fig. 11. Example 1.

Solution: Calculations are based on one half of the symmetric channel. A 1/2 inch fillet is used at the flange heel, and at the flange toe. The minimum weld length is equal to the 3.5 inch spacing between the welds, and there will be welds at each flange as shown in Figure 11.

$A_e = UA = 0.25(5.90 \text{ in.}^2) = 1.48 \text{ in.}^2$ (B3-1)

$\phi T_n = \phi F_y A_g = 0.9(36 \text{ ksi})(5.90 \text{ in.}^2)$

$$\phi R_n = 0.707 sl(\phi F_w)$$
$$= 0.707(1/2 \text{ in.}+1/2 \text{ in.})(3.5 \text{ in.})(0.75)(0.60)(70 \text{ ksi})$$
$$= 77.9 \text{ kips (weld strength)}$$

$$\bar{x} = \frac{(0.52 \text{ in.})\dfrac{(7.5 \text{ in.})^2}{2} + (3.52 \text{ in.} - 0.52 \text{ in.})\dfrac{(0.65 \text{ in.})^2}{2}}{(0.52 \text{ in.})(7.5 \text{ in.}) + (3.52 \text{ in.} - 0.52 \text{ in.})(0.65 \text{ in.})}$$
$$= 2.61 \text{ in.}$$

$$U = 1 - \frac{\bar{x}}{l} \le 0.90 = 1 - \frac{2.61 \text{ in.}}{3.5 \text{ in.}} = 0.25 \le 0.90 \qquad \text{(B3-2)}$$

$$A_g = \left(\frac{11.80 \text{ in.}^2}{2}\right) = 5.90 \text{ in.}^2$$

$= 191$ kips (yield) (D1-1)

$\phi T_n = \phi F_u A_e = 0.75(58 \text{ ksi})(1.48 \text{ in.}^2)$

$= 64.4$ kips (fracture) ⇐ controls strength (D1-2)

A plate equal in area to the channel flange is 3.52 inches by 0.65 inch. Weld strength is 77.9 kips as above. The design strength of that plate is calculated below:

$A_g = (3.52 \text{ in.})(0.65 \text{ in.}) = 2.29 \text{ in.}^2$

$U = 0.75$

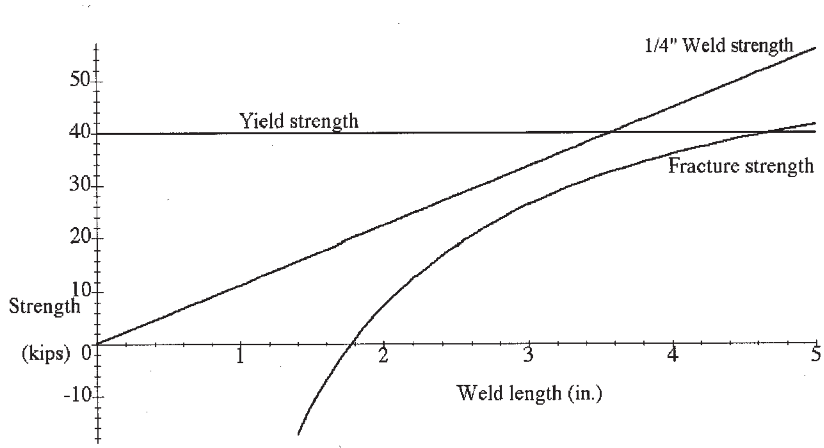

Fig. 12. Example 1.

$A_e = UA = 0.75(2.29\text{ in.}^2) = 1.72\text{ in.}^2$
$\phi T_n = \phi F_u A_e = 0.75(58\text{ ksi})(1.72\text{ in.}^2)$

$$\phi T_n = \phi F_y A_g = 0.9(36\text{ ksi})(2.29\text{ in.}^2)$$
$$= 74.2\ \text{ kips (yield)} \Leftarrow \text{ controls strength}$$

$$= 74.8\text{ kips (fracture)} \tag{D1-2}$$

Thus, applying the provisions of the Specification to a flange-attached channel results in a design tensile strength of 64.4 kips for the symmetric half channel. This is less than the 74.2 kips design tensile strength for a plate with a gross area equal to the channel flange alone.

Example 2:

Negative *U* values for channels. Connect an MC10×8.4 to two plates by the flanges.

Solution: Calculations are based on one half of the symmetric channel. A 1/4 inch fillet weld is used. The minimum permitted weld length is equal to the 1.5 inch spacing between the welds, and there will be welds at each flange as shown in the previous figure. The welding electrode is E70XX. A36 steel is used.

$$\bar{x} = \frac{(0.17\text{ in.})\dfrac{(5\text{ in.})^2}{2} + (1.50\text{ in.} - 0.17\text{ in.})\dfrac{(0.28\text{ in.})^2}{2}}{(0.17\text{ in.})(5\text{ in.}) + (1.50\text{ in.} - 0.17\text{ in.})(0.28\text{ in.})}$$
$$= 1.78\text{ in.}$$

$$U = 1 - \frac{\bar{x}}{l} \leq 0.90$$
$$U = 1 - \frac{1.78\text{ in.}}{1.5\text{ in.}} = -0.19 \leq 0.90 \tag{B3-2}$$

Negative *U* values, such as seen here, are an indication that weld lengths are too short and the connection is improperly detailed.

REFERENCES

AISC (1986), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, 1st ed., American Institute of Steel Construction, Chicago, IL.

AISC (1993a), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, 2nd ed., American Institute of Steel Construction, Chicago, IL.

AISC (1993b), *Commentary on the Load and Resistance Factor Design Specification for Structural Steel Buildings*, 2nd ed., American Institute of Steel Construction, Chicago, IL.

AISC (1993c), "Structural Members, Specifications, and Codes," *Manual of Steel Construction, Load and Resistance Factor Design*, 2nd ed., Vol. I, American Institute of Steel Construction, Chicago, IL.

AISC (1993d), "Connections," *Manual of Steel Construction, Load and Resistance Factor Design*, 2nd ed., Vol. 2, American Institute of Steel Construction, Chicago, IL.

AISC (forthcoming), *Commentary on the Load and Resistance Factor Design Specification for Structural Steel Buildings*, 3rd ed., American Institute of Steel Construction, Chicago, IL.

Butler, L. J. and Kulak, G. L. (1971), "Strength of Fillet Welds as a Function of Direction of Load," *Welding Journal,* Vol. 50, No. 5, pp. 231-234.

Butler, L. J., Pal, S., and Kulak, G. L. (1972), "Eccentrically Loaded Welded Connections," *Journal of Structural Engineering*, ASCE, Vol. 98, No. 5, pp. 989-1005.

Cheng, J. J. R., Kulak, G. L., and Khoo, H. (1998), "Strength of Slotted Tubular Tension Members," *Canadian Journal of Civil Engineering*, Vol. 25, pp. 982-991.

Chesson, E., Jr. and Munse, W. H. (1958), "Behavior of Riveted Truss-Type Connections," *Transactions*, ASCE, Vol. 123, pp. 1087-1128.

Chesson, E., Jr. and Munse, W. H. (1959), "Behavior of Large Riveted and Bolted Structural Connections," *Structural Research Series,* No. 174, University of Illinois, Urbana, IL.

Chesson, E., Jr. and Munse, W. H. (1963), "Riveted and Bolted Joints: Truss-Type Tensile Connections," *Journal of the Structural Division*, ASCE, Vol. 89 No. 1, pp. 67-106.

Davis, R. P. and Boomslitter, G. P. (1934), "Tensile Tests of Welded and Riveted Structural Members," *Journal of the American Welding Society*, Vol. 13, No. 4, pp. 21-27.

Davis, R. E., Woodruff, G. B., and Davis, H. E. (1940), "Tension Tests of Large Riveted Joints," *Transactions*, ASCE, Vol. 105, pp. 1193-1299.

Easterling, W. S. and Giroux, L. G. (1993), "Shear Lag Effects in Steel Tension Members," *Engineering Journal*, AISC, 3rd Quarter, pp. 77-89.

Foreman, R. T. and Rumpf, J. L. (1961), "Static Tension Tests of Compact Bolted Joints," *Transactions*, ASCE, Vol. 126, No. 2, pp. 228-254.

Fuller, J. R., Leahey, T. F., and Munse, W. H. (1955), "A Study of the Behavior of Large I-Section Connections," ASCE Proc. Separate No. 659, 81, pp. 1-26.

Gibson, G. J. and Wake, B. T. (1942), "An investigation of Welded Connections for Angle Tension Members," *Welding Journal*, Vol. 21, No. 1, pp. 44-49.

Gonzalez, L. and Easterling, W. S. (1989), "Investigation of the Shear Lag Coefficient for Welded Tension Members," Report No. CE/VPI-ST 89/13. Virginia Polytechnic Institute and State University, Blacksburg, VA.

Hildebrand, F. B. (1943), "The Exact Solution of Shear-Lag Problems in Flat Panels and Box Beams Assumed Rigid in the Transverse Directions," Technical Note 894, National Advisory Comm. for Aeronautics, Washington, D. C.

Kulak, G. L. and E. Y. Wu (1997), "Shear Lag in Bolted Angle Tension Members," *Journal of Structural Engineering*, ASCE, Vol. 123, No. 9, pp. 1144-1152.

Levy, A., Singer, J., Baruch, M. (1975), "Experimental Study of Shear Lag in Axially Loaded Panels," *Israel Journal of Technology*, Vol. 13, pp. 89-100.

Munse, W. H., and Chesson, E., Jr. (1963), "Riveted and Bolted Joints: Net Section Design," *Journal of the Structural Division*, ASCE, Vol. 89 (ST1), pp. 107-126.

Timoshenko, S. P. (1953), *History of Strength of Material*s, McGraw-Hill, New York, NY.