Shear Lag Effects in Steel Tension Members

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INTRODUCTION

The non-uniform stress distribution that occurs in a tension member adjacent to a connection, in which all elements of the cross section are not directly connected, is commonly referred to as the shear lag effect. This effect reduces the design strength of the member because the entire cross section is not fully effective at the critical section location. Shear lag effects in bolted tension members have been accounted for in the American Institute of Steel Construction (AISC) allowable stress design specification[1] (ASD) since 1978. The 1986 load and resistance factor design specification[2] (LRFD) and the 1989 ASD specification[3] stipulate that the shear lag effects are applicable to welded, as well as bolted, tension members.

Past research on the subject of shear lag has focused primarily on bolted tension members. Recently, more attention has been given to welded members, evident by their inclusion in the AISC specifications. Shear lag provisions for welded members were introduced into the specifications primarily because of a large welded hanger plate failure.[8] To maintain a uniform approach to both welded and bolted members, the same provisions for shear lag in bolted members were applied to welded members. Additional requirements for welded plates were added. However, the application of the shear lag requirements to welded members has raised several questions.

This paper examines shear lag in steel tension members in the following context. First, the background for the current AISC specification provisions is reviewed. Second, the results of an experimental research program in which 27 welded tension members were loaded to failure is presented. Third, based on the first two parts of the paper, recommended changes to the AISC specifications are presented.

BACKGROUND FOR CURRENT DESIGN PROVISIONS

Bolted Connections

The shear lag provisions in the current AISC specifications[2,3] are based on work reported by Chesson and Munse.[6,11] This work included experimental tests of riveted and bolted tension members conducted by Chesson and Munse and a review of experimental tests by other researchers. Chesson and Munse[6] defined test efficiency as the ratio, in percentage, of the ultimate test load to the product of the material tensile stress and the gross area of the specimen, and used this ratio to evaluate the test results. Several factors influence the test efficiency of connections failing through a net section: the net section area, a geometrical efficiency factor, a bearing factor, a shear lag factor, and a ductility factor.

The data base Chesson and Munse gathered included tests that failed in a variety of ways, including rupture of the net section, rivet or bolt shear, and gusset plate shear or tear-out. However, only tests exhibiting a net section rupture, approximately 200, were included in the validation of the tension member reduction coefficients. Munse and Chesson seldom observed efficiencies greater than 90 percent and therefore recommended, for design use, an upper limit efficiency of 85 percent.[11] Chesson[5] reported on two additional studies that recommended maximum efficiencies of 0.75 and 0.85.

Fourteen of the 30 tests conducted by Chesson and Munse[6] failed by net section rupture. Nine of the 14 tests failed at load levels exceeding the gross cross section yield load. Tests reported by Davis and Boomslitter[7] were used in the overall data base and also exhibited net section failures at load levels exceeding gross section yield. References to other tests are given by Chesson and Munse.

Research reported prior to 1963 indicated that shear lag was a function of the connection length[5] and the eccentricity of the connected parts.[7] Combining previous research results with their own investigation of structural joints, Munse and Chesson[11] developed empirical expressions to account for various factors influencing the section efficiency. The two most dominant parts of their formulation were the net section calculation, which accounts for stagger of the fasteners, and the shear lag effect. The shear lag expression is given by

$$U = 1 - \frac{\bar{x}}{l} \tag{1}$$

where

U = shear lag coefficient
$\bar{x}$ = connection eccentricity
l = connection length

An AISC Task Committee concluded from a review of Munse and Chesson's results that the recommended design

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procedure could be simplified.[10] The simplification is in the form of coefficients given in the AISC Specifications.[2,3] Although the work of Chesson and Munse included the effects of several factors on the net section efficiency, the AISC specifications only account for the two dominant factors, net area and shear lag. The commentaries of both specifications include Equation 1 as an alternate approach for determining the shear lag coefficients. The calculation of the effective net area, A_e, incorporates the shear lag coefficient and is given by

$$A_e = UA_n \tag{2}$$

where

A_n = net area

Welded Connections

In 1931 the American Bureau of Welding published the results of an extensive study in which safe working stresses for welds were determined. The American Bureau of Welding was an advisory board for welding research and standardization of the American Welding Society (AWS) and the National Research Council Division of Engineering.[4] The study was a collaborative effort between three steel mills, 39 fabricators, 61 welders, 18 inspectors, and 24 testing laboratories. Several specimen configurations were used in the test program and were assigned a series designation, e.g. 2400, 2500, etc., based on the configuration. Those directly applicable to this discussion consist of flat plate specimens, welded either longitudinally or both longitudinally and transversely. Both single and double plate tension specimens, as shown in Figure 1, were tested in the research program.

Most of the tests in the AWS program failed through the throat of the weld; but several of the specimens ruptured through the plate. The tests that ruptured are the ones applicable to the study described here. Key results from these tests have been taken from the report and are presented in Table 1. Figure 2 is a plot of the results in terms of plate thickness vs. experimental shear lag coefficient (efficiency), U_e.

Several trends are apparent in Figure 2. First, as the plate thickness increases, the scatter in the data tends to increase, with the average experimental shear lag coefficient increasing slightly. This trend appears to hold except for the ⅝-in. group, which shows the least scatter, although this is the group with the smallest number of tests.

Second, the amount of scatter in the ¾-in. group is unexpectedly high. There are groups of tests in which specimens have virtually identical details, yet the results vary by as much as 30 percent. For instance, consider the two ¾-in. specimens in series 2200. The specimen details are nearly the same, yet the experimental efficiency varies from 0.69 to 1.03. Likewise, the ¾-in. specimens of series 2400 had very similar details, but the experimental efficiencies varied from 0.65 to 0.94.

A number of factors may have caused the scatter, including variation in the quality of the welds. An interesting observation pertaining to the issue of weld quality was made while reviewing the AWS report. The last column of Table 1 is a code used in the report to indicate the welding process (arc or gas), fabricating shop, welder, and mill that supplied the steel. Nine specimens, all of which were arc-welded, failed at an efficiency less that 0.80. Most of these specimens had companion specimens, which had similar fabrication details, yet they exhibited test efficiencies well above 0.80. A hypothesis that welding techniques, which may have created gouges or notches in the base material, caused the scatter in the data was formed by the authors of this paper. This seems plausible because the nine tests with efficiencies below 0.80 were fabricated in two shops, by three welders, using steel from two mills (3 heats), and seven of those were welded in the same shop by two welders. Unfortunately, this hypothesis cannot be confirmed for tests conducted more than 60 years ago.

The results of the AWS research were considered in the development of the AISC specification provisions accounting for shear lag in welded members. However, as will be pre-

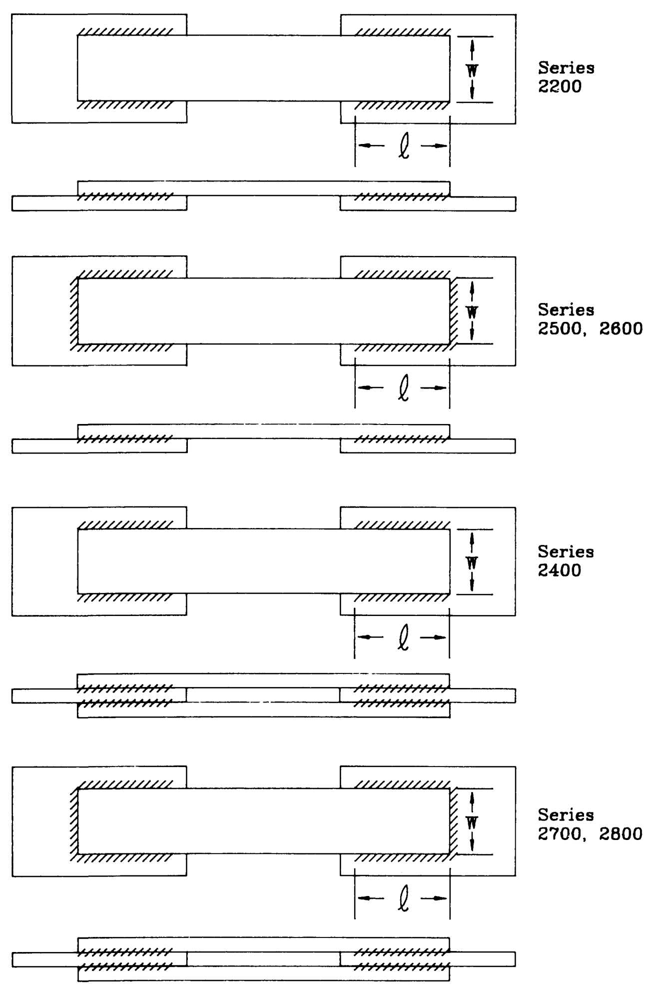


Fig. 1. AWS test specimen configuration.

Table 1. AWS Test Results[a]										
AWS Series	t_p (in.)	w (in.)	l (in.)	F_y (ksi)	F_u (ksi)	A_gF_y (kips)	A_gF_u (kips)	Test Load (kips)	U_e	Proc-Fab-Weld-Mill[b]
2200	0.75	7.5	12.0	36.3	57	204	321	221	0.69	A-Q-A-C
2200	0.75	7.5	12.0	33.2	56.9	187	320	329	1.03	G-AZ-B-I
2400	0.375	7.5	6.0	35.7	58	201	326	248	0.76	A-Q-B-C
2400	0.5	7.5	8.0	37.2	60.2	279	452	406	0.9	A-P-A-C
2400	0.5	7.5	8.0	37	59.2	278	445	303	0.68	A-Q-B-C
2400	0.5	7.5	8.0	39.2	62.2	294	467	382	0.82	G-AZ-B-I
2400	0.75	7.5	12.0	36.5	59.2	411	667	432	0.65	A-C-A-B
2400	0.75	7.5	12.0	36.4	59.6	410	671	484	0.72	A-Q-B-C
2400	0.75	7.5	12.0	33.5	57	377	641	600	0.94	G-AZ-B-I
2500	0.75	4.0	4.0	35.6	60.4	106.8	181.2	170.6	0.94	G-AZ-A-I
2600	0.5	7.5	4.0	37	59.3	139	222	149	0.67	A-Q-A-C
2600	0.75	7.5	8.0	36.5	59.2	205	333	186	0.56	A-C-A-B
2600	0.75	7.5	8.0	36.4	59.6	205	335	200	0.6	A-Q-B-C
2700	0.5	4.0	2.0	36.8	62.1	147.2	248.4	237.6	0.96	G-AZ-A-I
2700	0.75	4.0	4.0	35.6	60.4	213.6	362.4	350	0.97	G-AZ-B-I
2700	0.75	4.0	4.0	35.6	60.4	213.6	362.4	345	0.95	G-AZ-B-I
2800	0.375	7.5	2.0	35.7	58	201	326	282	0.87	A-N-A-C
2800	0.375	7.5	2.0	35.7	58	201	326	239	0.73	A-Q-A-C
2800	0.375	7.5	2.0	37.5	58.2	211	327	278	0.85	G-AZ-B-I
2800	0.375	7.5	2.0	37.5	58.2	211	327	275	0.84	A-CZ-A-I
2800	0.5	7.5	4.0	39.2	62.2	294	467	417	0.89	G-AZ-A-I
2800	0.625	7.5	6.0	36.6	61.6	343	578	500	0.87	A-C-A-B
2800	0.625	7.5	6.0	37.3	57	350	534	475	0.89	A-Q-A-C
2800	0.625	7.5	6.0	33.4	57	313	534	499	0.93	G-AZ-B-I
2800	0.625	7.5	6.0	33.4	57	313	534	520	0.97	A-CZ-A-I
2800	0.75	7.5	8.0	36.4	59.6	411	671	606	0.90	A-N-A-C
2800	0.75	7.5	8.0	36.4	59.6	411	671	590	0.88	A-P-A-C

a. All welds nominally $3/8$-in.; measured variation between $3/8$ and $1/2$-in.

b. Proc—welding process; A = arc welding G = gas welding
Fab—fabricator designation
Weld—welder designation (within particular fabricating shop)
Mill—mill designation for steel supply

sented later in this paper, questions have arisen regarding the application of the provisions to welded members. A research program was initiated to address the questions. The remainder of this paper presents the results of the research program.

RESEARCH PROGRAM FOR WELDED TENSION MEMBERS

This section of the paper summarizes a research project conducted at Virginia Tech focusing on the application of shear lag specification provisions to welded tension members, presenting both experimental and analytical results. The experimental program included tests of 27 welded tension members, along with the associated tensile coupon tests. Analytical studies included elastic finite element analyses of the experimental specimens, as well as a review of the AISC specification provisions pertaining to shear lag.

Description of Experimental Specimens

Each test specimen consisted of two members welded back-to-back to gusset plates, as shown in Figure 3. The gusset plates were then gripped in a universal testing machine and pulled until failure. Use of double members minimized the distortion due to the out-of-plane eccentricity, however, eccentric effects were ignored in the design of the test specimens.

Three types of member were tested: plates, angles, and channels. Fillet weld configurations used for each member type, except the plates, were longitudinal, transverse, and a combination of both longitudinal and transverse. For the plates, two different lengths of longitudinal weld and a combination of longitudinal and transverse welds were used.

For a given specimen configuration, three nominally identical tests were conducted; specimens with only transverse welds were the exception. Calculations indicate that tension members connected with only transverse fillet welds will always fail through the welds. For the purpose of confirming the calculations three specimens were fabricated with only transverse welds. Details of the specimens are given in Table 2. Test designations in Table 2 indicate the type of member (P = plate, L = angle, C = channel), weld configuration (L = longitudinal, T = transverse, B = longitudinal and

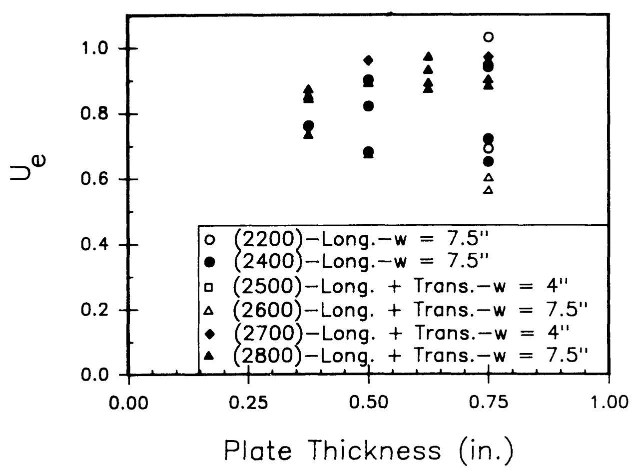


Fig. 2. Plate thickness vs. experimental shear lag coefficient for AWS tests.

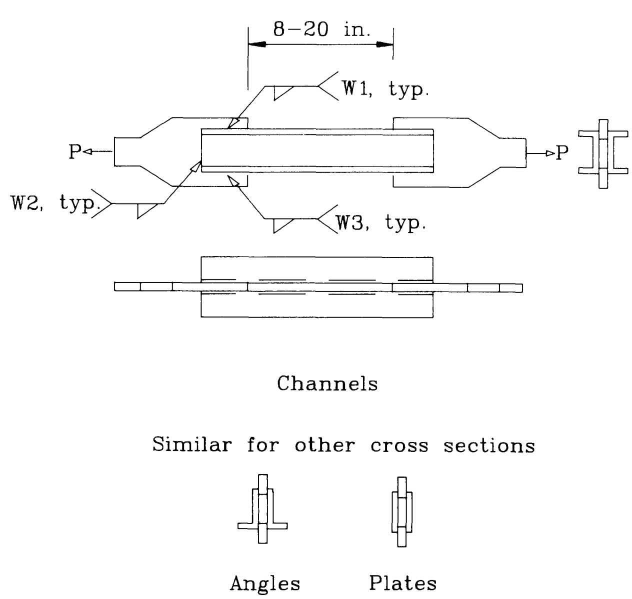


Fig. 3. Test specimen configuration.

transverse) and specimen number for a given member type and weld configuration. For instance, test designation P-B-2 is a plate specimen with both longitudinal and transverse welds and is the second test in that particular group. An additional number appears in the weld designation for some of the plate specimens (e.g. P-L2-3). This is because the longitudinal weld lengths were varied in some of the plate specimens that were fabricated with only longitudinal welds.

In an attempt to ensure net section failures in the members, all welds, except the transverse welds, were designed to have 10–15 percent greater strength than the gross section tensile strength of the member. The width and thickness of the connected member elements prevented oversizing of the transverse welds. Welds were balanced by size for all angle specimens, except L-B-1a, with the longitudinal weld lengths being equal on each specimen. Specimen L-B-1a was unbalanced with the two longitudinal welds being the same size and length.

Strain gages were used in one of the tests for each member type to study the stress distribution near the critical section of the member and the distribution of stress in the member along the length of the connected region. A displacement transducer was used to monitor the overall cross head movement. This measurement is only of qualitative value since it includes any slip between the specimen and the testing machine grips. Each specimen was whitewashed before testing to permit the observance of qualitative yield pattern formation. Complete specimen details are reported by Gonzalez and Easterling.[9]

Two aspects of the authors' research program should be kept in mind while reviewing the following results. The first is that the number of tests was limited, compared to the many tests available for consideration when the shear lag provisions were developed by Munse and Chesson. Second, the member sizes used to fabricate the test specimens were small. The capacity of the testing equipment available at the time the tests were conducted limited the member sizes. There are undoubtedly size effects that the results of this study do not reflect. However, the same can be said of the data base that forms the basis for the current shear lag specification provisions, and thus the results of this study can be considered similar to the bolted and riveted test results.

Description of Analytical Models

Linear elastic finite element analyses were performed for experimental test specimens using ANSYS, a commercial finite element analysis package.[12] None of the transverse welded members were analyzed.

A two-dimensional, four node, isoparametric plane stress element was used to model the plate specimens. The angles and channels were modeled using three-dimensional, four node, quadrilateral shell elements. Linear elastic spring elements simulated the welds. The spring force constant for the weld elements was determined using a calibration procedure. Only the members and the welds were modeled elastically;

Table 2. Experimental Specimen Details									
				Weld Configuration					
				W_1[a]		W_2[a]		W_3[a]	
Test No.	Test Designation	Member	A_g[b] (in.^{2})	Length (in.)	Size (in.)	Length (in.)	Size (in.)	Length (in.)	Size (in.)
1	P-L1-1a	PL4×3/8	1.47	5½	¼	—	—	5½	¼
2	P-L1-1b	PL3×¼	0.785	4¼	¼	—	—	4¼	¼
3	P-L1-2	PL3×¼	0.783	4¼	¼	—	—	4¼	¼
4	P-L1-3	PL3×¼	0.781	4¼	¼	—	—	4 ¼	¼
5	P-L2-1	PL3×¼	0.785	5	¼	—	—	5	¼
6	P-L2-2	PL3×¼	0.784	5	¼	—	—	5	¼
7	P-L2-3	PL3×¼	0.777	5	¼	—	—	5	¼
8	P-B-l	PL3×¼	0.780	3	¼	3	¼	3	¼
9	P-B-2	PL3×¼	0.777	3	¼	3	¼	3	¼
10	P-B-3	PL3×¼	0.783	3	¼	3	¼	3	¼
11	L-L-1	L2×2×3/16	0.760	4½	3/16	—	—	4½	3/8
12	L-L-2	L2×2×3/16	0.761	4½	3/16	—	—	4½	3/8
13	L-L-3	L2×2×3/16	0.756	4½	3/16	—	—	4 ½	3/8
14	L-B-1a	L4×3×¼	1.68	3½	¼	4	¼	3½	¼
15	L-B-1b	L2×2×3/16	0.756	3	3/16	2	3/16	3	7/16
16	L-B-1c	L2×2×3/16	0.771	3	3/16	2	3/16	3	7/16
17	L-B-2	L2×2×3/16	0.764	3	3/16	2	3/16	3	7/16
18	L-B-3	L2×2×3/16	0.750	3	3/16	2	3/16	3	7/16
19	L-T-1	L4×3×¼	1.67	—	—	4	¼	—	—
20	C-L-1	C3×4. 1	1.29	5	3/8	—	—	5	3/8
21	C-L-2	C3×4.1	1.28	5	3/8	—	—	S	3/8
22	C-L-3	C3×4.1	1.26	5	3/8	—	—	5	3/8
23	C-B-1	C3×4.1	1.24	5	3/16	3	3/16	5	3/16
24	C-B-2	C3×4.1	1.19	5	3/16	3	3/16	5	3/16
25	C-B-3	C3×4.1	1.22	5	3/16	3	3/16	5	3/16
26	C-T-1	C4×5.4	1.58	—	—	4	¼	—	—
27	C-T-2	C3×4.1	1.19	—	—	3	3/16	—	—

a. See Figure 3.
b. Gross area based on measured cross section dimensions.

the gusset plates were considered to be rigid. Interface, or gap, elements were used to prevent the member from displacing into the gusset plate. A typical finite element mesh and boundary conditions for a model of an angle specimen are illustrated in Figure 4. Plate and channel models were constructed in a similar manner. Only results from the plate models are presented in this paper. Results from other analyses are reported by Gonzalez and Easterling.[9]

The stiffness for the weld elements was determined by calibrating a model of a plate with 5-in. longitudinal welds to the corresponding experimental specimen. In the calibration process, the model was analyzed with eight different weld stiffness values, ranging from 100 to 5,000 k/in. The completely rigid case was also considered. The remaining analytical stresses and displacements were then compared to those observed experimentally. A spring constant of 350 k/in. provided the best correlation between the analytical and experimental stresses. The calibration weld size was 3/16-in. The spring constant for other weld sizes were determined assuming a linear relationship between the shear stiffness of the weld and the spring constant. All models contained the same number of spring elements per linear inch of weld.

General Results

In all tests with cross section ruptures, the failure occurred after the cross section yielded. The yielding was qualitatively observed by flaking of whitewash and quantitatively observed in the instrumented specimens from strain readings and in all specimens from load cell readings, which exceeded the yield load. Ideally, specimens used to determine shear lag coefficients would rupture at the critical section prior to yielding on the gross cross section. Shear lag coefficients determined from tests in which yielding occurs on the gross cross section prior to rupture on the net cross section may differ from those determined from tests that do not yield prior to rupture. This hypothesis has not been verified in the study reported here, nor in past studies. As indicated in the review of past research, this limitation was also present in most of the tests conducted as part of the research reported by Chesson and Munse.[6]

Yield lines, indicated by flaking of the whitewash, generally were not observed within the directly connected portion of the members. In some instances, the portion of the cross section that was not connected, e.g. outstanding angle leg, showed indications of yielding. Yielding mostly occurred in the portion of the member between the welded ends.

Experimental results are given in Table 3. The experimental shear lag coefficients, U_e, were calculated as the ratio of the failure load to the rupture strength (gross area × tensile stress). The shear lag coefficients for the specimens that did not exhibit rupture at the critical cross section can be taken at least equal to those shown in Table 3. The values for these tests do not explicitly represent shear lag coefficients because rupture was not the controlling limit state. Calculated shear lag coefficients, U_t, were determined using Equation 1, except for the plate specimens and the transversely welded specimens. Coefficients for the plate specimens were determined

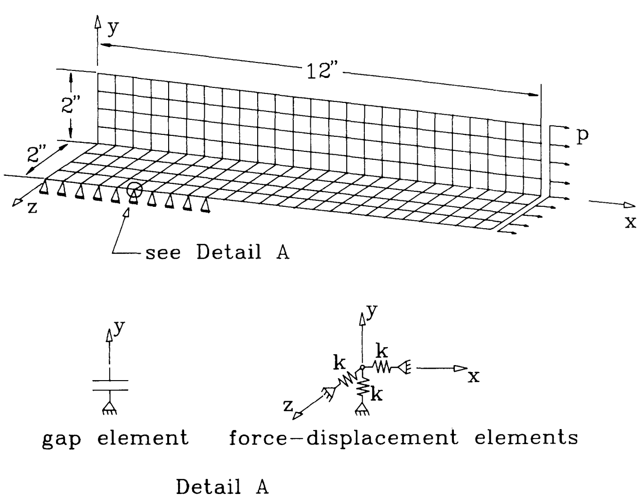


Fig. 4. Finite element model of typical angle.

from the current AISC specifications,[2,3] which give the coefficients according to:

a. If $l \geq 2w$ . . . . $U = 1.0$
b. If $2w > l \geq 1.5w$ . . . . $U = 0.87$
c. If $1.5w > l \geq w$ . . . . $U = 0.75$

where

w = plate width (distance between welds)

The shear lag coefficient for longitudinally welded plates can also be calculated using Equation 1 with each half of the plate treated independently. Therefore, $\bar{x}$ would be one-fourth of the plate width. These values are not shown in Table 3. Coefficients for the transversely welded members were calculated as the ratio of the area of the directly connected elements to the gross area. This is also an AISC specification provision. The calculation procedure for the shear lag coefficients is deemed acceptable if the ratios of experimental to calculated shear lag coefficients, given in Table 3, fall within a 10 percent scatter band, i.e. 0.9 to 1.1. A similar evaluation was made for bolted and riveted tests reported by Munse and Chesson.[11]

Plate Specimens

Results are summarized in Table 3 and as indicated, the plate tests can be divided into three groups according to the specification shear lag coefficients of 0.75, 0.87, and 1.0. (Values computed using Equation 1, as described in the previous paragraph, are 0.82, 0.85, and 1.0.) Two of the groups have only longitudinal welds and one has both transverse and longitudinal welds.

The plate specimens exhibited tearing across the member at the critical section, which was at the end of the welds. Yielding in the plates was first observed at the critical cross section at the end of the welds. None of the plate specimens displayed significant out-of-plane effects. For all of the plate specimens the ratio of U_e / U_t was greater than 0.9. Six of the nine tests have values of U_e / U_t greater than or equal to 1.1. Note that a failure load was not obtained for Test 1 because the testing machine capacity was exceeded. However a shear lag coefficient of 0.92 is reported. This represents the maximum load applied, and the true coefficient would have been greater.

Longitudinal strains were recorded across the width of the instrumented plate specimens near the end of the welds. The strain gage locations for Test 3 are shown in Figure 5. Strains were converted to stresses and distributions plotted along the critical section. The stress distribution at various load levels within the elastic range of material behavior for Test 3 is illustrated in Figure 6.

Note the unsymmetrical stress distributions shown in Figure 6(a), most likely caused by imbalance in the longitudinal welds or eccentrically applied load. The welds were detailed for a balanced configuration, but given the stress distribu-

Table 3. Experimental Results

Test No.	Test Designation	F_y (ksi)	F_u (ksi)	Failure Load Per Member, $P_u/2$ (k)	Calculated Shear Lag Coefficient, U_e	Theoretical Shear Lag Coefficient, U_t	U_e/U_t
1	P-L1-1a	48.4	73.2	99.0[a]	0.92	0.75	1.23
2	P-L1-1b	51.9	73.0	53.7	0.94	0.75	1.25
3	P-L1-2	51.9	73.0	56.0	0.98	0.75	1.31
4	P-L1-3	51.9	73.0	57.5	1.00	0.75	1.33
5	P-L2-1	51.9	73.0	55.9	0.98	0.87	1.13
6	P-L2-2	51.9	73.0	55.8	0.98	0.87	1.13
7	P-L2-3	51.9	73.0	54.4	0.96	0.87	1.10
8	P-B-1	51.9	73.0	51.2	0.90	1.00	0.9
9	P-B-2	51.9	73.0	56.1	0.99	1.00	0.99
10	P-B-3	51.9	73.0	55.7	0.97	1.00	0.97
11	L-L-1	54.1	81.1	50.0	0.81	0.87	0.93
12	L-L-2	54.1	81.1	50.5	0.82	0.87	0.94
13	L-L-3	54.1	81.1	50.4	0.82	0.87	0.94
14	L-B-1a	47.8	71.3	98.7	0.82	0.80	1.03
15	L-B-1b	54.1	81.1	49.5[b]	—	0.81	—
16	L-B-1c	54.1	81.1	50.0	0.80	0.81	0.99
17	L-B-2	54.1	81.1	46.2	0.75	0.81	0.93
18	L-B-3	54.1	81.1	48.8	0.80	0.81	0.99
19	L T-1	47.8	71.3	55.8[b]	—	0.59	—
20	C-L-1	57.0	75.5	87.0[c]	0.89	0.91	0.98
21	C-L-2	57.0	75.5	86.7[c]	0.90	0.91	0.99
22	C-L-3	57.0	75.5	86.9[c]	0.91	0.91	1.00
23	C-B-1	55.7	76.6	85.1[c]	0.92	0.91	1.01
24	C-B-2	55.7	76.6	84.0[c]	0.92	0.91	1.01
25	C-B-3	56.0	77.1	83.1	0.88	0.91	0.97
26	C-T-1	58.5	77.6	60.0[b]	—	0.44	—
27	C-T-2	51.1	73.8	32.3[b]	—	0.49	—

a. Testing machine capacity exceeded.
b. Weld failure.
c. Gross cross section failure away from welds.

tions, they apparently were not fabricated symmetrically. One would expect the distributions to be symmetric if the welds were balanced and the load applied concentrically.

Figure 7 is a plot of the stress distributions at approximately the critical section for Tests 3, 6, and 8. The strain gages were within 0.5 in. of the critical section. Note that both specimens welded only longitudinally exhibited unsymmetric stress distributions, while the specimen that was welded with both longitudinal and transverse welds exhibited essentially a symmetric distribution. Assuming that the stress distribution would be symmetric if the welds were balanced, then the experimental stresses may be modified to permit an evaluation of the influence of the longitudinal weld length. Figure 8 is a plot of the distributions in which the symmetric strain readings, e.g. gages 1 and 5 and gages 2 and 4, were averaged prior to converting the values to stresses.

In Figure 8, the three tests show similar distribution patterns, but with varying magnitudes of stress. The highest stresses occurred for Test 8, which had 3-in. longitudinal welds along with a transverse weld. Test 3, which had 4¼-in. longitudinal welds, exhibited lower stresses than Test 6, which had 5-in. longitudinal welds. The variations in the stresses for Tests 3 and 6 ranged between 3 and 7 percent, while the stresses for Test 8 were 5–7 percent higher than those for Test 6.

The analytical, based on finite element analyses, and ex-

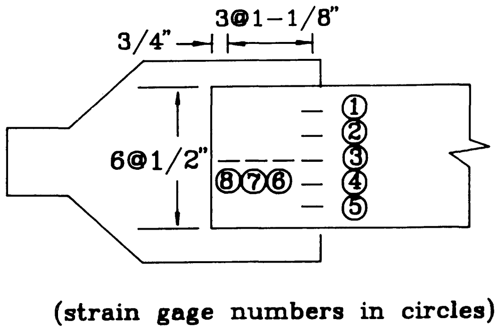


Fig. 5. Strain gage locations for Test 3.

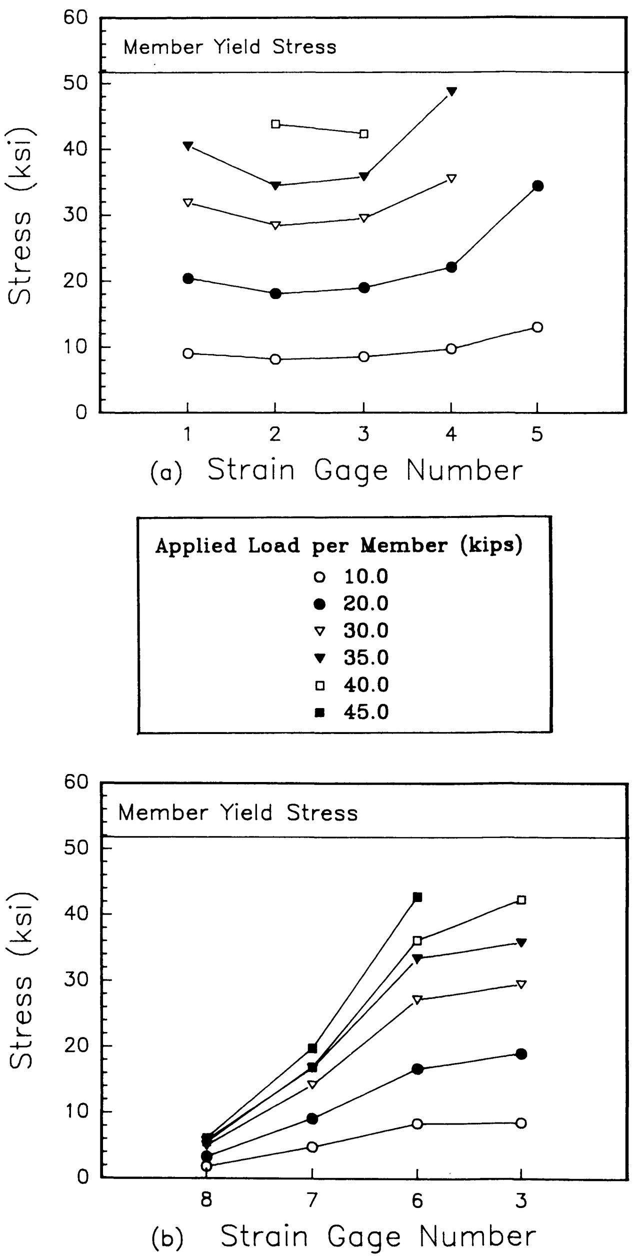


Fig. 6. Experimental stresses for Test 3.

perimental stresses for Tests 3, 6, and 8 can be compared with Figures 9–11. The trend for each of the three cases is similar. The experimental stresses near the center of the plate are approximately the same, or somewhat less, than the calculated stresses. Experimental stresses nearer the edge of the plate are greater than the calculated stresses. This trend may have been caused by the stopping or starting of the welding process, causing imperfections at the critical section in the form of gouges or notches due to blow out. These imperfections would result in stress concentrations adjacent to the edge of the plate, which would in turn cause yielding earlier in the loading process. Stress concentrations caused by the imperfections at the critical cross section were not considered in the finite element model.

The experimental results for the plate specimens indicated that the longitudinal weld length appears to not influence the rupture strength based on shear lag effects. This observation was reinforced by the elastic finite element results which showed virtually the same stress distribution at the critical section for models in which the weld length is 3, 4¼ and 5 in. Neither the stress distribution at the critical section nor the experimental shear lag coefficient were significantly affected by the addition of the transverse weld, as compared to the specimens with only longitudinal welds. However, note that the differences in longitudinal weld length were relatively small.

Angle Specimens

All but two of the angle specimens exhibited a tearing failure, with the tearing initiating at the welded toe. The welds sheared in Tests 15 and 19. The outstanding legs of the specimens generally exhibited more signs of yielding at the critical section, evident by whitewash flaking, than the area of the angles directly connected to the gusset

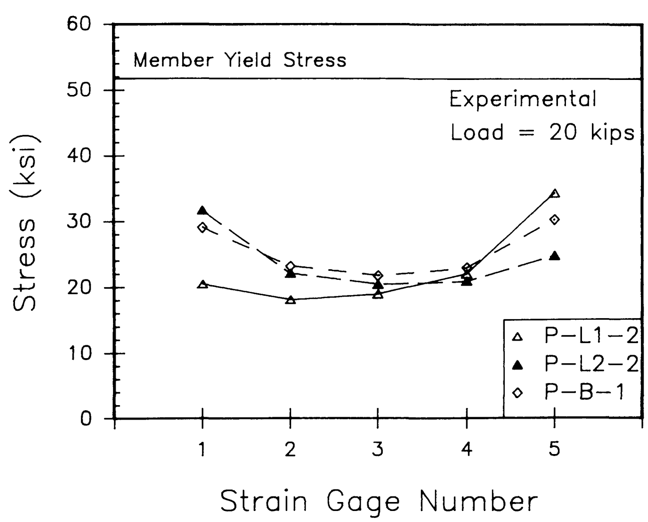


Fig. 7. Experimental stresses at the critical section for Tests 3, 6, and 8.

plates. This behavior was observed in both longitudinally welded specimens and specimens with a combination of longitudinal and transverse welds, and was attributed to combined stress caused by out-of-plane eccentricity. Yielding generally was first visible at the heel of the angle in the connected leg.

Significant out-of-plane bending occurred in the specimens fabricated with 2L4×3×¼. Bending in the plane of the connected leg also occurred in Test 14. The welds for this specimen were not balanced, nor was the centerline of the angle coincident with the centerline of the end plates, i.e. the line of load application. Negligible out-of-plane bending was observed in the specimens fabricated with 2L2×2×3⁄16.

The ratios of U_e / U_t, given in Table 3, for the angle specimens vary from 0.93 to 1.03, the majority of the values being less than or equal to 0.99. These results indicate that the calculated shear lag coefficients compare well to the experimental results for this group of tests. However, it is interesting to note that the experimental values for all but one test were between 0.80 and 0.82, while the calculated values ranged between 0.80 and 0.87. The increased length of the welds for Tests 11–13 did not affect the shear lag coefficient as expected. As with the plate tests, the addition of the transverse welds did not affect the maximum loads, or shear lag coefficients, for the angle tests.

Channel Specimens

The predominant limit state observed in the channel tests was rupture in the cross section away from the welded region. Each specimen in the series of longitudinally welded chan-

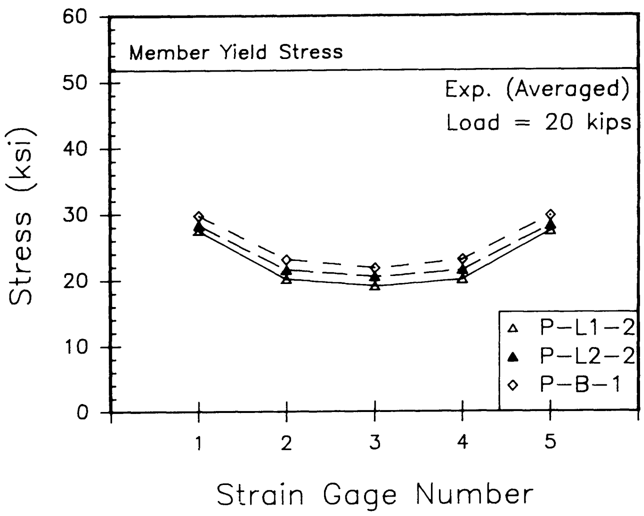


Fig. 8. Modified experimental stresses at the critical section for Tests 3, 6, and 8.

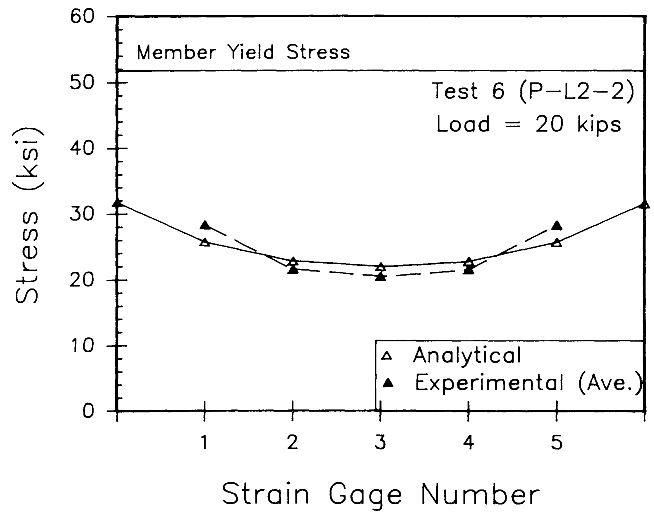


Fig. 10. Analytical vs. experimental stresses for Test 6.

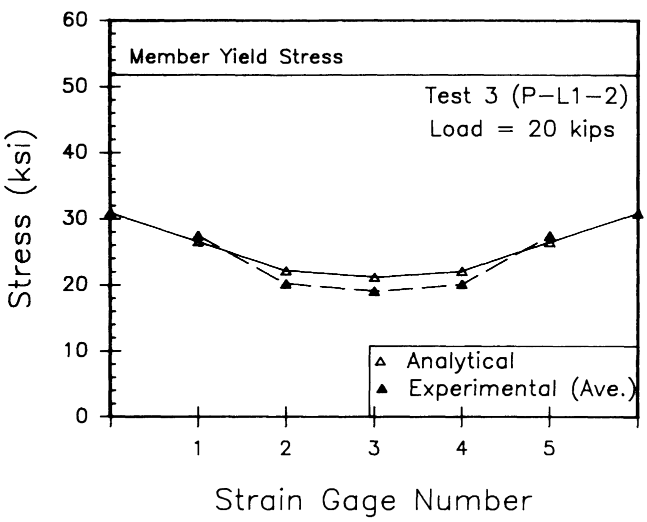


Fig. 9. Analytical vs. experimental stresses for Test 3.

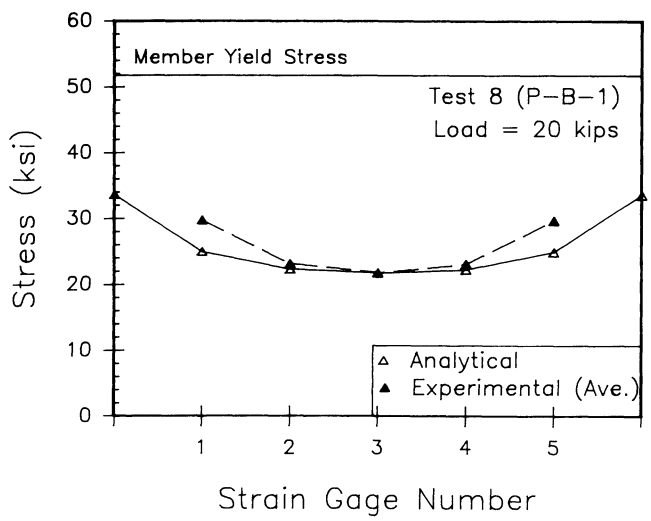


Fig. 11. Analytical vs. experimental stresses for Test 8.

nels, and all but one of the longitudinally and transversely welded channels, failed in the center of the specimen. This was attributed to the combined state of stress induced in the members by the out-of-plane load eccentricity.

Initial yielding was generally concentrated in the channel flanges and near the web-flange intersection. In the welded area, as with the angles, yielding was visible in the outstanding flanges while none was present in the directly connected web. The propagation of yielding into the channel flanges was attributed to the combined axial stress and bending stress due to out-of-plane eccentricity.

Note in Table 3 that all the experimental shear lag coefficients for the channels ranged between 0.88 and 0.92. However, the predominant limit state was rupture of the gross cross section approximately halfway between the two ends of the specimen, and not rupture of the net section. These results agree with the observed practical upper limit of 0.9 that Chesson and Munse[11] identified from their studies. Due to eccentricities and fabrication imperfections in welded specimens, an upper limit of 0.9 for the shear lag coefficient appears prudent. As with the plate and angle specimens, the maximum loads and experimental shear lag coefficients were not affected by the addition of a transverse weld.

SPECIFICATION REVIEW AND RECOMMENDED REVISIONS

The AISC design specifications for shear lag pertained only to bolted or riveted connections prior to the inclusion of welded members in the 1986 LRFD Specification,[2] and subsequently in the 1989 ASD Specification.[3] Welded members are treated similar to bolted members to maintain continuity in the specifications. However, the provisions for welded members are not clear in all instances and have therefore raised questions regarding their application. A review of the questions and related issues, along with recommended changes to the specification, are presented in this section of the paper.

Although the specification indicates that welded members are subject to shear lag reductions, there is no minimum weld length criterion to distinguish between different coefficient values in Chapter B3. The first set of subparagraphs a, b, and c in section B3 identify a minimum fastener length indirectly, by specifying a minimum number of fasteners, in the direction of stress for bolted or riveted connections but not for welded connections. In fact, because welding is not mentioned in subparagraphs a, b, or c, while bolting and riveting are, it is unclear that the definitions apply to welded members. Nevertheless, these sections are intended to be applicable to welded specimens.

Another unclear portion of the specification pertains to the use of members with both transverse and longitudinal welds. A specification provision is given for members connected by only transverse welds. If the first group of subparagraphs a, b, and c are assumed to apply only to members with longitudinal welds, then no provisions exist for cases in which a combination of longitudinal and transverse welds are used. Results of this study indicate that the addition of a transverse weld does not significantly affect the rupture strength compared to a specimen with only longitudinal welds.

The shear lag provision for members welded only with transverse welds specifies that the effective area shall be the area of the connected element. Reviewing the limit states of weld shear and shear lag, summarized in the Appendix of this paper, indicates that weld shear will always control the strength if fillet welds are used. If partial- or full-penetration welds are used then the present specification provision is appropriate.

According to the specification commentary, previous research[4] determined that plates welded only longitudinally can fail prematurely due to shear lag if the distance between the welds is too great. Thus, a minimum weld length equal to the plate width or distance between the welds, w, is required. Currently, the specification does not consider shear lag a limiting factor as long as the weld length is greater than twice the plate width. Two shear lag coefficients are specified for intermediate ranges of longitudinal weld length between w and $2w$. Results from this study indicate that the weld lengths greater than the distance between the welds have little influence on the shear lag coefficient. However, due to the limited number of tests conducted and to the small size of the members, no modifications are recommended to the shear lag coefficients for longitudinally welded plates.

Reviewing the AWS[4] results, along with the statement by Munse and Chesson[11] that efficiencies greater than 90 percent are seldom observed, an upper limit for U of 0.9 is deemed appropriate. This is also consistent with the upper limit that appears in the current specifications[2,3] in section B3 subparagraph a. The strength of welded tension members is reduced due to the coupled effects of shear lag, stress concentrations, and eccentricities. The stress concentrations are due to the sudden change in stiffness caused by the presence of the weld, or to notches or gouges created at the critical section by the welding process. Although all play a role in reducing the strength, it is difficult to determine the relative participation of each component. Using an empirical approach, such as the shear lag coefficient, is an approximate way to account for all the effects.

Recommended Revisions to the Specification and Commentary

Recommended revisions to the specification were developed jointly by the authors of this paper and the AISC Task Committee 108—Connections and Force Introduction. The recommended changes address all of the issues identified in the previous section. All of the recommended changes apply to section B3 of the AISC Specifications.[2,3]

Subparagraphs a, b, and c that follow the line "Unless a larger coefficient can be justified by tests or other rational criteria...." should be replaced with a single equation for U, given by:

$$U = 1 - \frac{\bar{x}}{l} \leq 0.9 \qquad (3)$$

The specific values of U, given for certain groups of sections in subparagraphs a, b, and c, are acceptable for use in lieu of the values calculated from Equation 3 and may be retained in the commentary for continued use by designers.

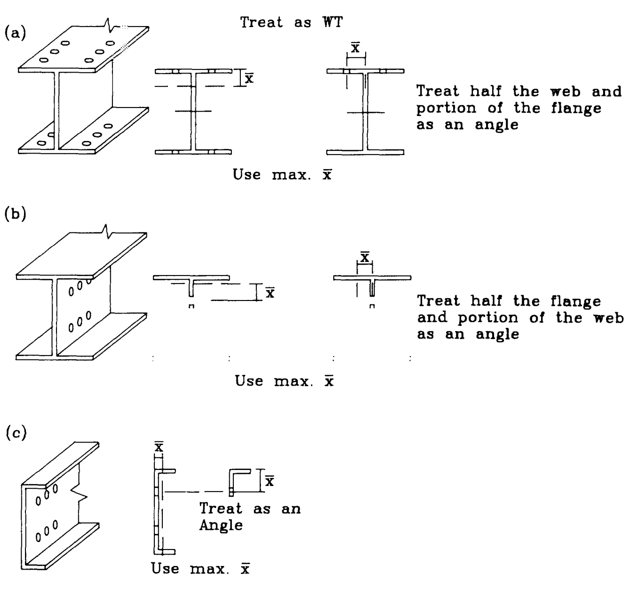
The section that addresses sections only connected with transverse welds should be modified to include all shapes, not just W, M, or S shapes and structural tees cut from these shapes. A provision should be added to indicate that this section is only applicable if partial- or full-penetration welds are used, and is not applicable if fillet welds are used as the transverse weld type.

In addition to changing the commentary to incorporate the information of subparagraphs a, b, and c, several other changes would help to clarify specification revisions. Several of these are indicated in the following paragraph. Although the primary focus of the research project reported in this paper was welded tension members, literature and specification provisions for bolted members were reviewed. The comments made in the following paragraphs pertaining to bolted members are the authors' judgment based on that review.

For any given profile and connected elements, $\bar{x}$ is a geometric property. It is defined as the distance from the connection plane, or face of the member, to the member centroid, as indicated in Figure 12. Note that the "member" may be a portion of the cross section for particular cases. Connection length, l, is dependent upon the number of fasteners or length of weld required to develop the given tensile force, and this in turn is dependent upon the mechanical properties of the member and the strength of the fasteners or weld used. The length l is defined as the distance, parallel to the line of force, between the first and last fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of determining l, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, use the out-to-out dimension for l (See Figure 13). For welded connections, l is the length of the member parallel to the line of force that is welded. For combinations of longitudinal and transverse welds (see Figure 14), l is the length of longitudinal weld because the transverse weld does not significantly affect the rupture strength based on shear lag. The presence of the transverse weld does little to get the load into the unattached portions of the member.

SUMMARY AND CONCLUSIONS

The purpose of this investigation was to review the shear lag provisions for welded tension members relative to those for bolted members, and to make recommendations for pertinent specification changes. Experimentally, three different member types and three different weld configurations were considered. Results of 27 tests were reported. Longitudinal stresses were determined analytically in a finite element study. Experimental strains were determined directly from



(use similar concepts for welded members)

Fig. 12. Definition of $\bar{x}$ for various members.

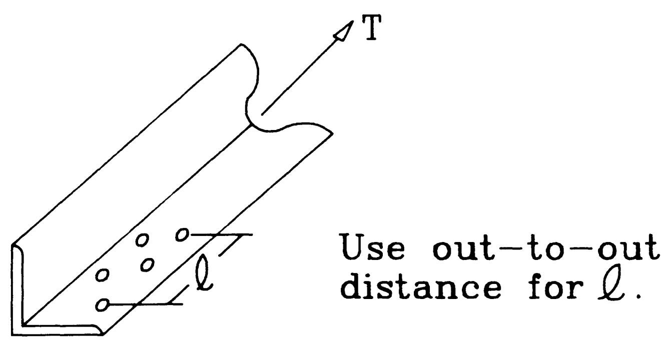


Fig. 13. Definition of l for bolted members with staggered holes.

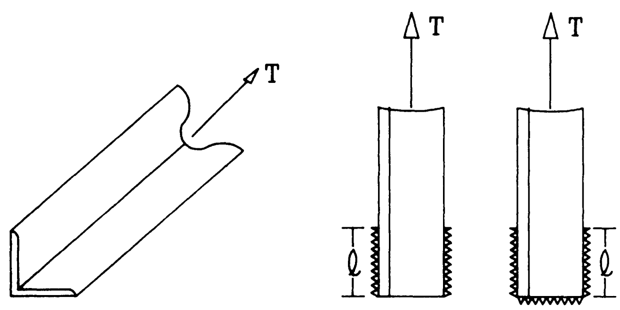


Fig. 14. Definition of l for welded members.

tensile tests. The analytical and experimental stress patterns in the elastic region were compared. Shear lag criteria are recommended based on the experimental results. The current AISC provisions have been reviewed and revisions recommended.

The recommended revisions to the specification are based on the results of the experimental and analytical studies reported here, a review of the specification and judgment of the authors. In particular the definitions of $\bar{x}$ in Figure 14 and l in Figures 15 and 16 are based on the authors' judgment. Further, the hypothesis that the net section failure is due to a combination of the shear lag effect and stress concentration caused by welding induced imperfections is also based on the authors' judgment, given the insight gained from the research program. Each of these topics would require further study to be proven explicitly.

The following conclusions have been drawn from the experimental and analytical investigations:

1. Shear lag controlled the strength of the angle and plate specimens.
2. For plates connected only by longitudinal welds, connection length had little influence on the experimental shear lag coefficient.
3. The transverse weld in the angle members welded both longitudinally and transversely did not increase the shear lag coefficient as expected. The experimental shear lag coefficients of the longitudinally welded angles and the angles with both longitudinal and transverse welds were equivalent.
4. Shear lag will not control the strength of tension members connected only by transverse fillet welds. Weld shear will be the controlling limit state, regardless of electrode strength or fillet weld size. This conclusion does not apply to partial- or full-penetration welds.
5. Due to the small size of the experimental specimens in this study as well as past studies, caution should be exercised when applying the design provisions to much larger tension members. There is a need for some limited confirmatory testing on large tension members designed so that shear lag effects control the strength.
6. The recommended upper limit for the shear lag coefficient is 0.9.
7. The implementation of the recommended changes to AISC specifications and commentaries would result in a simpler, more uniform approach to the application of shear lag provisions to bolted and welded tension members. The changes should result in fewer questions regarding the application of the provisions.

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APPENDIX

The strength of members welded only with transverse fillet welds will not be controlled by the rupture based on shear lag effects, but rather will be controlled by weld shear. This is true regardless of the steel or electrode strength. This can be shown by considering the following parameters:

$A_e = bt$ = area of connected element
E70XX electrodes (fillet welds)
A36 steel

The strength of the tension member based on rupture is given by:

$$\phi P_n = \phi F_u A_e = 0.75(58 \text{ ksi})bt \tag{A1}$$

where ϕ for tension rupture is 0.75.

The weld strength is given by:

$$\phi R_w = \phi 0.6 F_{EXX} A_w \tag{A2}$$

where ϕ for weld shear is 0.75 and R_w = nominal weld resistance.

If the weld area, A_w, is taken as $(0.707t)b$ (the maximum possible dimension for a fillet weld made along the edge of plate element), then ϕR_w becomes

$$\phi R_w = 0.75(0.6)(70 \text{ ksi})(0.707t)b = 0.75(29.7 \text{ ksi})bt \tag{A3}$$

Comparing Equations A1 and A3, one observes that the weld strength, Equation A3, is less and will therefore control the strength. The same conclusion will be reached for any practical combination of weld electrode and steel. If the submerged arc process were used, the weld strength result, Equation A3, would increase, but it would still remain less than Equation A1.

REFERENCES

1. American Institute of Steel Construction, *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings with Commentary,* Chicago: AISC, 1978.
2. American Institute of Steel Construction, *Load and Re-*

sistance Factor Design Specification for Structural Steel Buildings, Chicago: AISC, 1986.

3. American Institute of Steel Construction, *Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design,* Chicago: AISC, 1989.
4. American Bureau of Welding, "Report of Structural Steel Welding Committee," American Welding Society, New York, 1931.
5. Chesson, E., Jr. "Behavior of Large Riveted and Bolted Structural Connections." Thesis presented to the University of Illinois in partial fulfillment of the requirements for the degree of Doctor of Philosophy. Urbana, 1959.
6. Chesson, E., Jr. and Munse, W. H. "Riveted and Bolted Joints: Truss-Type Tensile Connections," *Journal of the Structural Division,* ASCE, Vol. 89(ST1), 1963, pp. 67–106.
7. Davis, R. P. and Boomslitter, G. P. "Tensile Tests of Welded and Riveted Structural Members," *Journal of the American Welding Society,* 13(4), 1934, 21–27.
8. Fisher, J. W., personal communication, ATLSS, Lehigh University, Bethlehem, PA, 1990.
9. Gonzalez, L. and Easterling, W. S. "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, December, 1989.
10. Kulak, G. L., Fisher, J. W. and Struik, J. H. A., *Guide to Design Criteria for Bolted and Riveted Joints,* 2nd Edition, John Wiley and Sons, New York, 1987.
11. Munse, W. H. and Chesson, E., Jr. "Riveted and Bolted Joints: Net Section Design," *Journal of the Structural Division,* ASCE, Vol. 89(ST1), 1963, pp. 107–126.
12. Swanson Analysis Systems, Inc., *ANSYS Engineering Analysis System User's Manual, Vol. I and II.* Houston, PA, 1989.