

Shear Lag Effects in Tension Members with Short Longitudinal Welds

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ABSTRACT

Shear lag can reduce the strength of tension members and must be considered when calculating tensile rupture strength—for example, by using the shear lag factor. The shear lag factor defined in Case 4 of AISC *Specification* Table D3.1 (2022) applies to tension members where the load is transmitted by longitudinal welds only. This shear lag factor was developed through analysis of physical experiments, but predominantly with experiments on specimens that failed by weld rupture, potentially leading to unnecessarily low values of the shear lag factor. As a result, the behavior of tension members with short longitudinal welds, for which the shear lag factor can be much less than unity, is uncertain. In this work, previously published experiments are reevaluated, and new experiments are reported. The results shed light on the behavior of tension members with short longitudinal welds. Despite current design provisions indicating that tensile rupture is an important limit state for tension members with short welds, no evidence that tensile rupture or shear lag influences strength of such members was identified. For plate tension members, use of a shear lag factor of $U = 1$ was found to be acceptable because other applicable limits states that better represent the physical behavior control. These results show the need for further research to better characterize the behavior of welded tension members and to simplify design provisions.

Keywords: tension members, welded connection, tensile rupture, shear lag factor.

INTRODUCTION

The nonuniform stress distribution that occurs in a tension member adjacent to a connection, especially when not all elements of the cross section are directly connected, is commonly referred to as the shear lag effect (Easterling and Gonzalez Giroux, 1993). Shear lag effects can decrease strength, and accurate accounting of them in design ensures safety and efficiency. For design of tension members according to the AISC *Specification* (2022), the available strength for the limit state of tensile rupture in the net section depends on the shear lag factor, U , which is determined according to AISC *Specification* Table D3.1. This table lists several cases. Case 4 applies to plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. The shear lag factor for Case 4 is defined as:

$$U = \frac{3l^2}{3l^2 + w^2} \left(1 - \frac{\bar{x}}{l} \right) \quad (1)$$

where l is the length of the connection, w is the spacing between welds, and $\bar{x}$ is the eccentricity of the connection. Figure 1 shows dimensions l and w . A footnote to AISC *Specification* Table D3.1 defines l as $(l_1 + l_2)/2$ and specifies that l_1 and l_2 shall not be less than 4 times the weld size.

Case 4 uses the shear lag factor from Case 2 (i.e., $U = 1 - \bar{x}/l$) as a base but modifies it to account for the length of the longitudinal welds, l , and the width between them, w . When the length of the welds is short relative to the width between the welds, the resulting shear lag factor can be very low. Note, however, that AWS D1.1 (2020), clause 4.9.2, states “If longitudinal fillet welds are used alone in lap joints of end connections of flat bar or plate members, the length of each fillet weld shall be no less than the perpendicular distance between them”—thus effectively limiting the ratio between l and w .

The current form of the shear lag factor for Case 4 (i.e., Equation 1) was introduced in the 2016 edition of the AISC *Specification* (2016) and is based on research by Fortney and Thornton (2012). Prior to this study, Case 4 applied only to plates, and the shear lag factor was defined as $U = 1.0$ when $l/w \geq 2$, $U = 0.87$ when $2 > l/w \geq 1.5$, and $U = 0.75$ when $1.5 > l/w \geq 1.0$. The shear lag factor was undefined when $l/w < 1$, implying that the weld length should not be less than the plate width. Fortney and Thornton proposed a shear lag factor applicable to all member types, cases with unequal weld lengths, and cases with weld lengths less than the weld spacing (i.e., $l/w < 1$). Fortney and Thornton developed their recommendations based on comparisons to experimental results. Three studies were included in their evaluation (Easterling and Gonzalez Giroux, 1993; Gibson

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and Wake, 1942; SSWC, 1931). Of the 175 specimens evaluated by Fortney and Thornton, 151 were reported by the Structural Steel Welding Committee of the American Bureau of Welding (SSWC, 1931), and 149 of these specimens failed by weld rupture and not tensile rupture. Fortney and Thornton asserted that data from these specimens are valid for evaluating shear lag effects.

To illustrate the pertinent strength calculations, consider the plate tension member with a welded connection shown in Figure 2. Five limit states must be considered to determine the available strength of the tension member: tensile yielding in the gross section, tensile rupture in the net section, shear yielding adjacent to the welds, shear rupture adjacent to the welds, and weld rupture. The shear yielding and shear rupture limit states can be considered as the base metal check for the weld. The strength for each of these limit states, except for gross section tensile yielding, depends on the length of the weld. Strength equations are shown in Table 1, where t is the thickness of the plate and t_w is the throat of the weld. The design strength (i.e., with LRFD resistance factors applied) of the tension member for

each limit state is shown as a function of weld length, l , in Figure 3.

Interestingly, the weld strength and tensile rupture strength are nearly equal for weld lengths less than 6 in. (i.e., the width of the plate). This may be a consequence of the shear lag factor being derived predominantly from experiments where weld failure occurred. It is questionable that tension members with these very small weld lengths would experience a net section tensile rupture failure, one where the strength is proportional to the cross-sectional area of the tension member, instead of a weld or base metal failure. If the weld and base metal checks are sufficient to capture the strength of members with short welds, including potential shear lag effects, then using the shear lag factor, U , to also capture the strength of members with short welds as part of the tensile rupture limit state is redundant and not representative of physical behavior.

The objective of this work is to shed new light on the strength of tension members connected with short longitudinal welds. Assumptions made in prior research are reevaluated by examining the results of previously published

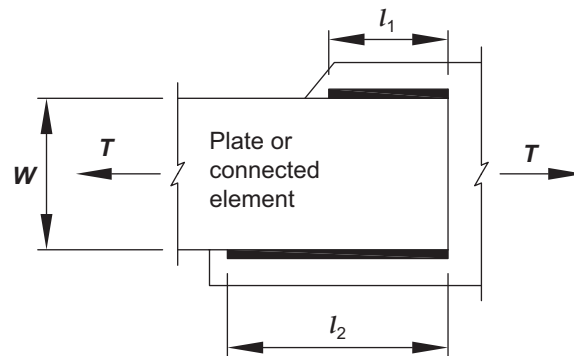


Fig. 1. Diagram for Case 4 in AISC Specification Table D3.1 (2022).

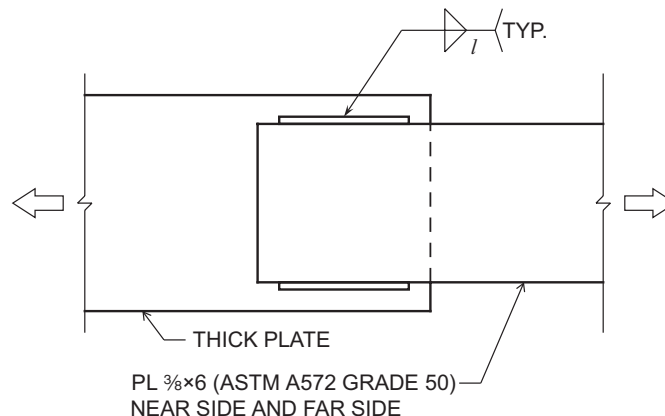


Fig. 2. Double-plate tension member for example calculations.

Table 1. Limit States and Strength Equations for Longitudinally Welded Double Plate Tension Member		
Limit State and AISC Specification Reference	Nominal Strength Equations	Resistance Factor and Safety Factor
Tensile yielding Section D2	$P_{n,TY} = F_y A_g$ $A_g = 2wt$	$\phi_t = 0.90$ $\Omega_t = 1.67$
Tensile rupture Section D2	$P_{n,TR} = F_u A_e = F_u A_n U$ $A_n = 2wt$ See Equation 1 for U	$\phi_t = 0.75$ $\Omega_t = 2.00$
Shear yielding Section J4.2	$P_{n,SY} = 0.60 F_y A_{gv}$ $A_{gv} = 4lt$	$\phi = 1.00$ $\Omega = 1.50$
Shear rupture Section J4.2	$P_{n,SR} = 0.60 F_u A_{nv}$ $A_{nv} = 4lt$	$\phi = 0.75$ $\Omega = 2.00$
Weld rupture Section J2.4	$P_{n,WR} = F_{nw} A_{we} = 0.60 F_{EXX} A_{we}$ $A_{we} = 4lt_w$	$\phi = 0.75$ $\Omega = 2.00$

physical experiments as well as new physical experiments described in this work. Comparisons are made to current design provisions and recommendations are made for further work to improve the tensile rupture provisions in the AISC Specification.

PREVIOUS EXPERIMENTAL RESULTS

A database of previously published physical experiments on longitudinally welded tension members was compiled to form a benchmark against which design equations can

be compared. The data was taken from five prior studies (SSWC, 1931; Gibson and Wake, 1942; Easterling and Gonzalez Giroux, 1993; Mannem, 2002; Orloff et al., 2020). The members tested in these experimental studies include single and double plates, single and double angles, and double channels. Only specimens with two parallel longitudinal welds were included in the database. Specimens with a transverse weld or only one longitudinal weld were excluded. Specimens that did not achieve their full strength (e.g., machine capacity limits were reached) were also excluded from the database. Altogether, the database developed for

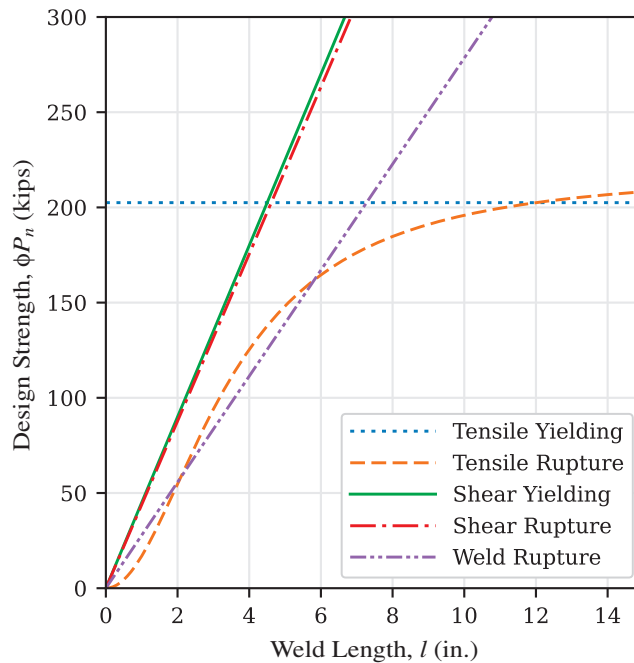


Fig. 3. Design strength of example double-plate tension member as a function of weld length.

Table 2. List of Source References for the Database and Count of Specimens in the Database by Experimental Failure Type			
	Experimental Failure Type		
Reference	Member	Weld	Total
SSWC (1931)	9	289	298
Gibson and Wake (1942)	3	7	10
Easterling and Gonzalez Giroux (1993)	12	0	12
Mannem (2002)	23	3	26
Orloff et al. (2020)	8	12	20
Total	55	311	366

this work includes 366 previously tested specimens as listed in Table 2 by reference and failure type. Most of the specimens failed by weld rupture, while 55 were reported to have failed in the member. The database is published in a publicly available data repository (Muncy and Denavit, 2025).

The Structural Steel Welding Committee (SSWC) of the American Bureau of Welding was organized to obtain “reliable information upon which to base safe unit working stresses in the designing of welded structures.” The report by the committee (SSWC, 1931) describes more than 2,000 physical experiments, 298 of which are included in the database (151 single-plate tension members and 147 double-plate tension members). The SSWC report was also the source of most of the data evaluated by Fortney and Thornton (2012); however, Fortney and Thornton only evaluated data from the 151 single-plate tension member experiments. Given the purpose of the testing, the vast majority of specimens reported by the SSWC failed by weld rupture. Only 9 of the 298 specimens reported by SSWC and included in the database developed for this work experienced a member failure.

Gibson and Wake (1942) investigated the strength of welded angle tension members with a primary goal of determining if unbalanced weld arrangements are as effective as balanced weld arrangements (i.e., those for which the center of resistance of the welds aligns with the center of gravity of the angle). Gibson and Wake tested 54 specimens in 15 different arrangements, most of which failed by weld rupture. Six single-angle arrangements and four double-angle specimens are included in the database. Gibson and Wake tested three specimens of each single-angle arrangement and reported only the average, which was taken as a single specimen in the database. Fortney and Thornton (2012) included only the four single-angle arrangements with balanced welds in their evaluation.

Easterling and Gonzalez Giroux (1993) investigated shear lag effects in welded tension members to inform updates to the AISC *Specification*. They tested 27 tension members, including double-plate, double-angle, and double-channel specimens, 12 of which were included in the database.

Fortney and Thornton (2012) included these specimens in their evaluation plus four additional double-tee specimens reported in Gonzalez Giroux’s master’s thesis (Gonzalez, 1989).

The remaining two studies were not evaluated by Fortney and Thornton (2012). Mannem (2002) and Orloff et al. (2020) both investigated shear lag effects in welded steel angles and plates. Mannem tested 49 specimens, 26 of which are included in the database, while Orloff et al. tested 20 specimens, all of which are included in the database.

The database includes recorded data such as weld dimensions and experimental strength as well as calculated values analyzing the specimen’s strength compared to AISC *Specification* equations. The following fields are included in the database:

- Reference (i.e., author and year of publication)
- Specimen name
- Specimen member type
- Weld length, l (in.)
- Weld spacing, w (in.)
- Net area, A_n (in.²)
- Connection eccentricity, $\bar{x}$ (in.)
- Measured yield stress, F_y , and tensile strength, F_u , of tension member (ksi)
- Nominal weld strength, F_{EXX} (ksi)
- Weld throat dimension, t_w (in.)
- Experimental failure type (i.e., member or weld)
- Experimental strength, P_{exp} (kips)
- Plate thickness, t (in.) for plate specimens only

The net area for double component specimens (i.e., double-plate, double-channel, or double-angle) was taken as the combined net area for both shapes. Likewise, the

experimental strength of double-component specimens was taken as the total force in both components at failure.

The data encompasses a range of ratios of weld length to weld spacing (i.e., l/w) and connection eccentricity to weld length (i.e., $\bar{x}/l$). These parameters are important because they define the shear lag factor in Equation 1. Figure 4 shows the range of these ratios for specimens in the database. The specimens are separated into two subfigures by experimental failure type. Each marker represents a single specimen. The shape and color of the marker indicates the member type. The connection eccentricity for all plates was assumed to be zero as is typical for design (Fortney and Thornton, 2012). Apart from the plate tension members, the eccentricity ratio, $\bar{x}/l$, ranges from 0.069 to 0.465. The length to width ratios ranges from 0.667 to almost 4.0. Specimens with lower values of $\bar{x}/l$ or higher values of l/w tended to fail in the member, while specimens with higher values of $\bar{x}/l$ or lower values of l/w tended to fail in the weld. Specimens with the highest values of l/w tend to have relatively low values of $\bar{x}/l$ and vice versa (i.e., few specimens were tested with high l/w and high $\bar{x}/l$).

Most of the specimens exhibited significant member yielding prior to failure. This can be seen in the load-deformation response of the specimens for which that data was reported and also by comparing the experimental strength to the tensile yielding strength, $A_g F_y$ (using measured values). The ratio of experimental strength to tensile yielding strength is shown in Figure 5. Only one specimen reported to have failed in the member, 2443-Q from SSWC (1931), failed at a load less than $A_g F_y$. Many specimens reported to have failed in the weld had a peak load less than $A_g F_y$, but most had a peak load above this limit. These results indicate that even when rupture and a sudden loss of load was observed in the experiment, it was often preceded by significant yielding.

The strength of the specimens according to the 2022 AISC *Specification* (AISC, 2022) is compared to the experimental strength in Figure 6. Specimens reported to have failed in the member are compared to the tensile rupture strength, while specimens reported to have failed in the weld are compared to the weld rupture strength. Tensile rupture strength was computed using measured material properties, while weld rupture strength was calculated using nominal material properties (due to a lack of reporting on measured weld material properties). Both were computed without resistance or safety factors.

The average test-to-predicted ratio in Figure 6(a) is above unity. The four specimens shown in Figure 6(a) that have a test-to-predicted ratio less than 0.95 were reported by the SSWC (1931). For three of the specimens (i.e., 2263-Q, 2463-C, and 2463-Q), the failure occurred above the yield strength, $F_y A_g$, while the fourth specimen (i.e., 2443-Q) failed at 89% of the yield strength. Overall,

the test-to-predicted ratio tends to increase as the weld length-to-width ratio decreases, particularly when l/w is less than 2.0. Test-to-predicted ratios greater than unity mean that the specimen exceeded the expected strength calculated according to the AISC *Specification*. Significantly surpassing this expected strength could imply either the tensile rupture equation or the shear lag factor equation underestimates the strength of the specimen. Since the specimens with higher length-to-width ratios have relatively accurate predictions, it is likely that the shear lag factor is the cause of the underestimated strength for the specimens with a lower length-to-width ratio. The shear lag factor equation (i.e., Equation 1) for these specimens is strongly dependent on the weld length-to-width ratio. These results could indicate that the shear lag factor is overly sensitive to the weld length-to-width ratio.

The average test-to-predicted ratio in Figure 6(b) is also above unity, but many specimens have a test-to-predicted ratio less than unity. Notably, two single-angle specimens with $l/w = 2.5$ have a low test-to-predicted ratio of 0.36 [the markers for these two specimens overlap in Figure 6(b)]. These two specimens, UEA4 and UEA7, experienced “failures through the weld by pull out at the outer edge on the heel of the angle” (Mannem, 2002). Tearing at the heel of the angle at the net section was also observed. The weld failures occurred at a load 10% greater than $F_y A_g$ and the computed tensile rupture strength (both approximately 103 kips). Therefore, while weld rupture was the limit state causing loss of load-carrying capacity for these specimens, other damage occurred, and strength equations for limit states associated with the damage conservatively underpredict the experimental strength. The other specimens with test-to-predicted ratio less than unity in Figure 6(b) do not necessarily indicate that the current provisions are unsafe because the scatter is considered in reliability analyses performed to determine resistance and safety factors (e.g., Dowswell and Cox, 2024). However, the lack of a clear trend between l/w and the weld test-to-predicted ratio in Figure 6(b) indicates that shear lag is not a major factor in the strength of longitudinal welds in end connections.

The strength data shown versus the weld length-to-width ratio in Figure 6 is shown versus the eccentricity ratio in Figure 7. There is an upward trend in Figure 7(a) where the test-to-predicted ratio for double-angle specimens increases with increasing eccentricity ratio. However, the trend is not observed for single-angle specimens. The two single-angle specimens with low weld test-to-predicted ratios discussed previously are seen again in Figure 7(b), but no other obvious trends are identified from Figure 7(b).

Overall, the previous experimental results indicate that tensile rupture strength equations in the AISC *Specification* (2022) underestimate the true strength of longitudinally welded tension members, likely due to a lower

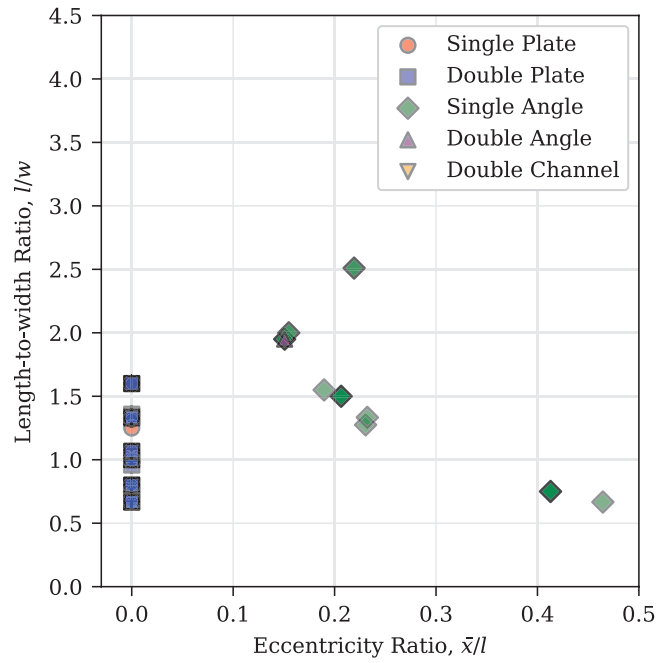
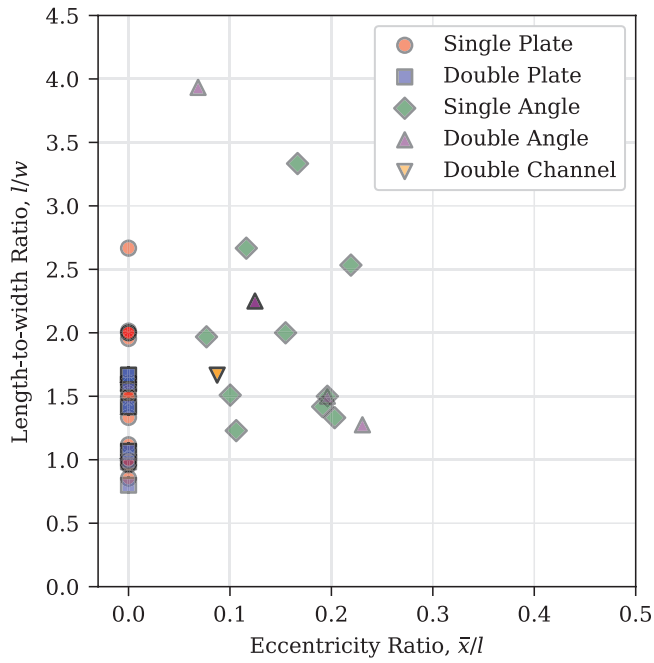


Fig. 4. Length-to-width ratio versus eccentricity ratio for specimens in the database.

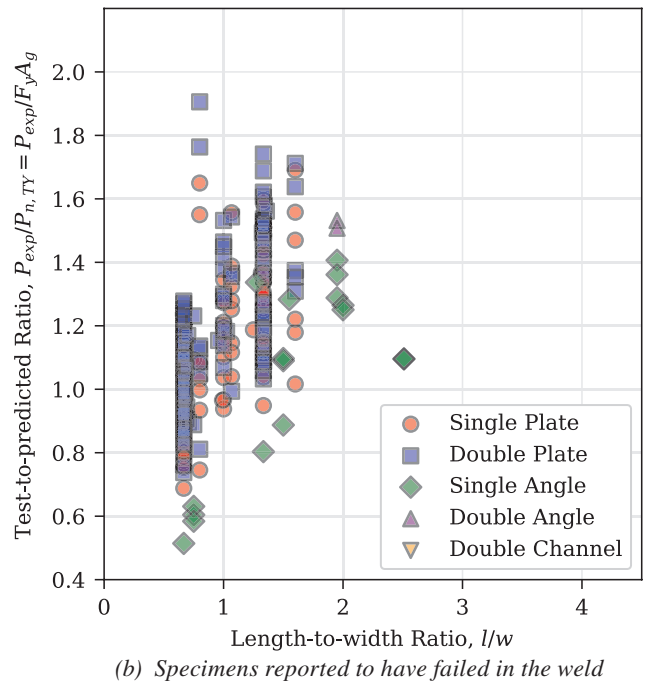
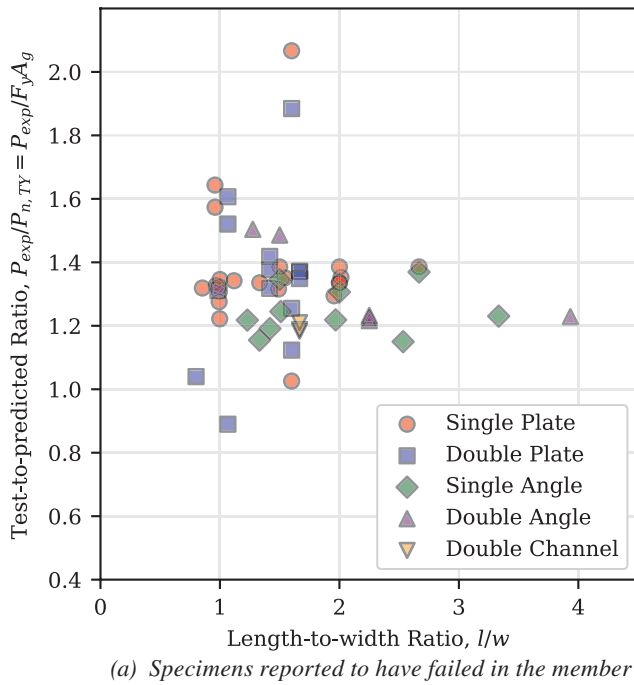
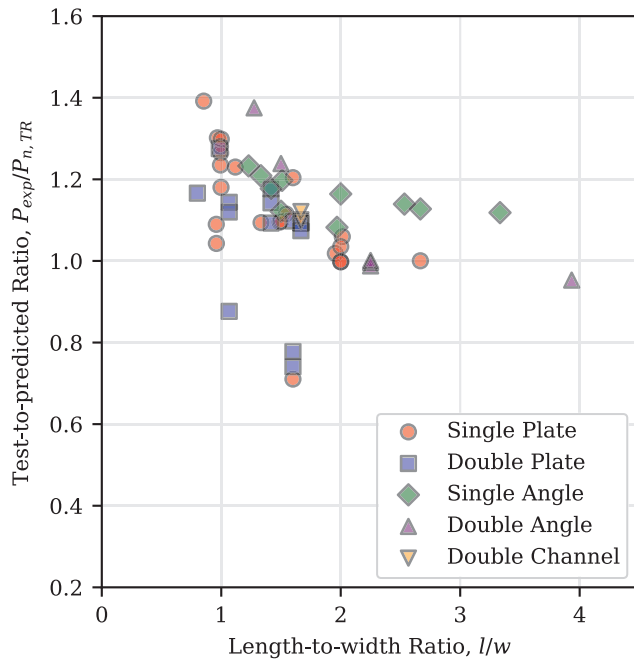
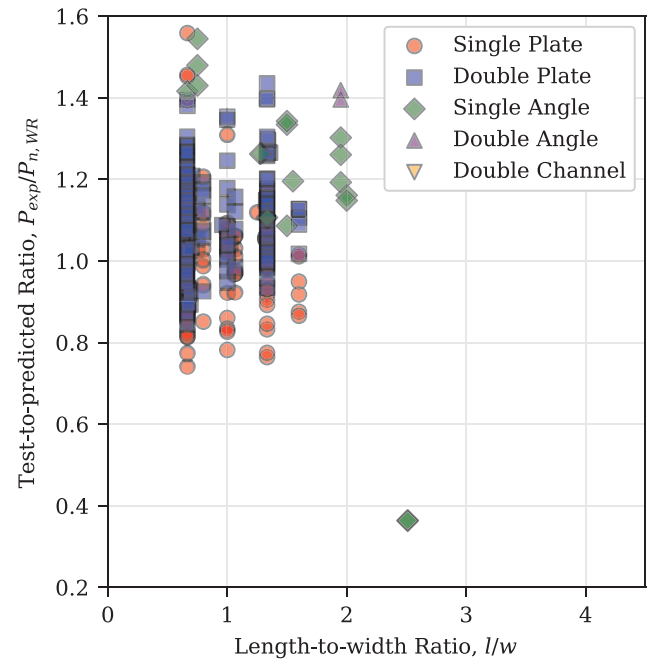


Fig. 5. Comparison of experimental strength to calculated yield strength for specimens in the database.

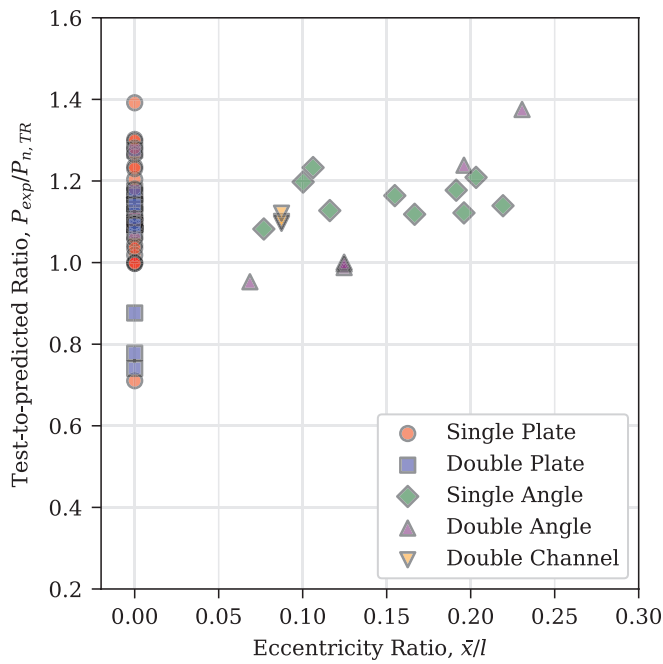


(a) Specimens reported to have failed in the member

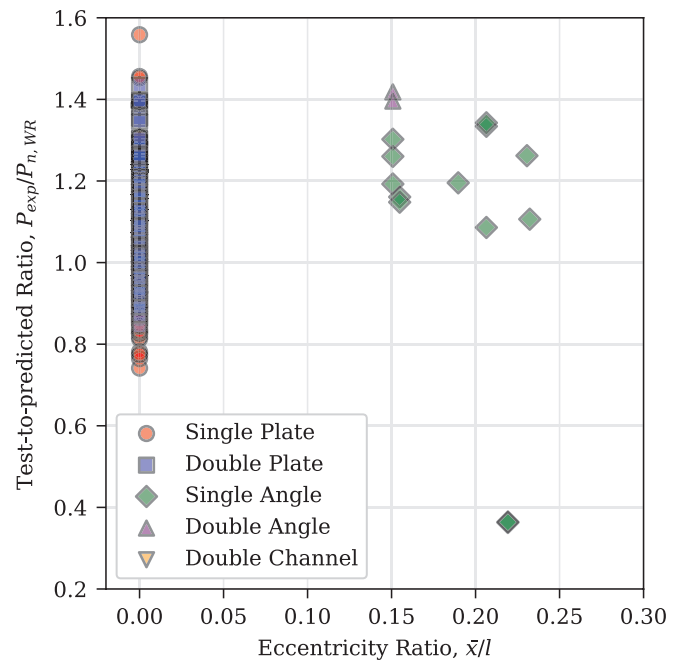


(b) Specimens reported to have failed in the weld

Fig. 6. Comparison of experimental strength to calculated strength shown versus length-to-width ratio for specimens in the database.



(a) Specimens reported to have failed in the member



(b) Specimens reported to have failed in the weld

Fig. 7. Comparison of experimental strength to calculated strength shown versus eccentricity ratio for specimens in the database.

than necessary shear lag factor. They also indicate that if shear lag effects reduce weld strength, then the reduction is minor. However, the data is limited, especially for tension members with very low weld length-to-width ratios for which shear lag effects are expected to be the most extreme. Therefore, additional testing, especially on specimens with weld length-to-width ratios less than 0.667, would be beneficial for better understanding the behavior of longitudinally welded tension members.

MATERIALS AND METHODS

Eight double-plate tension member specimens were designed, fabricated, and tested to failure to quantify their strength. A drawing of one of the specimens is shown in Figure 8. For each specimen, two $\frac{3}{8}$ -in.-thick steel test plates were welded to two $\frac{3}{4}$ -in.-thick steel support plates. Each support plate was bolted with two $\frac{7}{8}$ -in.-diameter

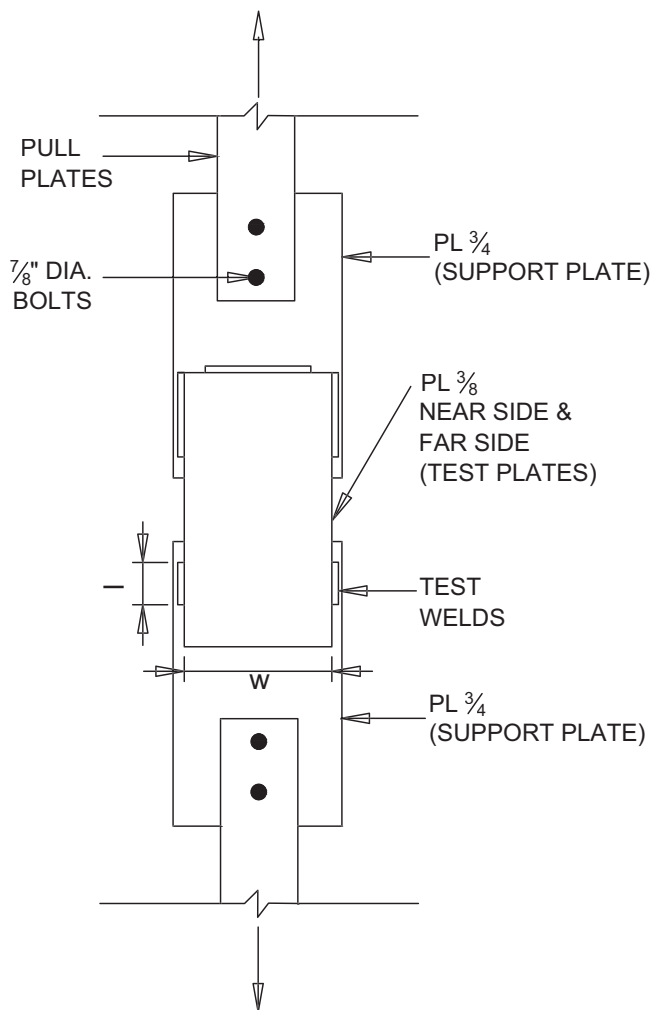


Fig. 8. Schematic of test specimen.

bolts to a pair of pull plates that were connected to a uni-axial test machine and applied tension to the specimen. A photograph of one of the specimens in the test machine is shown in Figure 9.

The test plates were all 13 in. long and $\frac{3}{8}$ in. thick but varied in width. Specimen names and dimensions are listed in Table 3. The first number in each specimen name indicates the width of the plate. The second number in each specimen name indicates the nominal length of the weld. All test plates were cut from a single piece of $\frac{3}{8}$ -in.-thick by 8-in.-wide by 20-ft-long steel, using a waterjet cutter. Three tensile coupons were also cut from the piece of steel and tested according to ASTM E8 (2022). The average yield stress was measured to be $F_y = 54.4$ ksi, the average ultimate strength was measured to be $F_u = 73.0$ ksi, and the average elongation was measured to be 26% (for an 8 in. gauge length). The thickness of the test plates was measured to be 0.372 in.

The welds were specified as $\frac{1}{4}$ in. and made in a single pass using a MIG process with ER70S-6 wire ($F_{EXX} = 70$ ksi). The weld lengths (tip-to-tip) and weld legs were

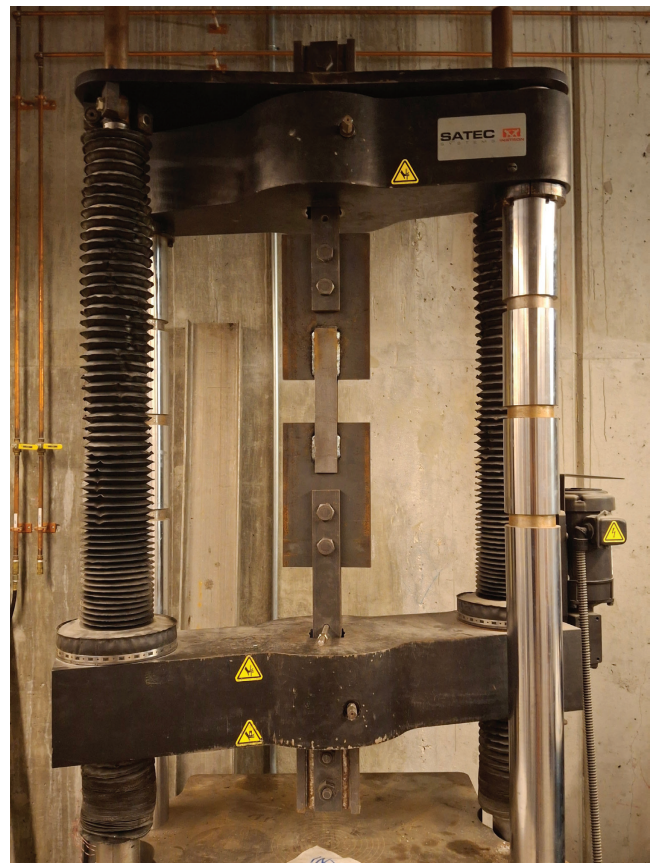


Fig. 9. Photograph of Specimen S22 in the testing machine prior to loading.

Table 3. Test Matrix								
Specimen —	t in.	w in.	l in.	t_w in.	l/w —	U —	$A_g = A_n$ in. ²	$A_{gv} = A_{nv}$ in. ²
S22	0.372	2	2.26	0.202	1.130	0.793	1.49	3.36
S23	0.372	2	3.22	0.188	1.610	0.886	1.49	4.79
S24	0.372	2	4.08	0.191	2.040	0.926	1.49	6.07
S32	0.372	3	2.20	0.191	0.733	0.617	2.23	3.27
S42	0.372	4	2.15	0.202	0.538	0.464	2.98	3.20
S52	0.372	5	2.24	0.205	0.448	0.376	3.72	3.33
S62	0.372	6	2.26	0.202	0.377	0.299	4.46	3.36
S72	0.372	7	2.35	0.207	0.336	0.253	5.21	3.50

measured using a caliper. The throat, t_w , was computed from the weld leg measurements as shown in Figure 10. Each specimen had four lines of test welds. The average of the four weld lengths and average of the four throats are reported in Table 3. Using measured dimensions, the specimens had weld length-to-width ratios ranging from 0.336 to 2.040.

Deformations of the specimens were measured using LED markers attached to the plates. Two markers were attached to the top support plate, and two markers were attached to the bottom support plate. An Optotrak position measurement machine recorded the three-dimensional position of the markers throughout the test. Connection deformation (i.e., elongation) was calculated from the position data after the test. Specimens were loaded in displacement control at a rate of the crosshead of 0.05 in./min or 0.10 in./min for specimen S24. Testing was stopped after weld rupture or tensile rupture occurred and the load-carrying capacity of the specimen dropped. Specimens S23 and S24 were stopped after necking initiated and the peak load had been achieved but prior to rupture.

RESULTS

A summary of the test results is presented in Table 4. Two failure modes were observed: tensile rupture of the test plates for specimens S22, S23, and S24 and weld rupture for all other specimens. Load-deformation results of all eight specimens are shown in Figure 11.

The specimens that failed by tensile rupture (i.e., S22, S23, and S24) exhibited an initially stiff response, followed by yielding with a plateau, then strain hardening. Necking occurred away from the weld at approximately mid-distance between the two support plates as shown in Figure 12. Fracture of the two test plates occurred simultaneously for specimen S22. Testing of specimens S23 and S24 was halted after necking initiated but before fracture of the test plates. No other damage to the specimen or welds was identified.

The remaining five specimens (i.e., S32, S42, S52, S62, and S72) failed by weld rupture. A detailed view of the load-deformation response of these specimens is shown in Figure 11(b). The peak deformation was much less than the specimens that failed by tensile rupture. The specimens exhibited an initially stiff response, followed by

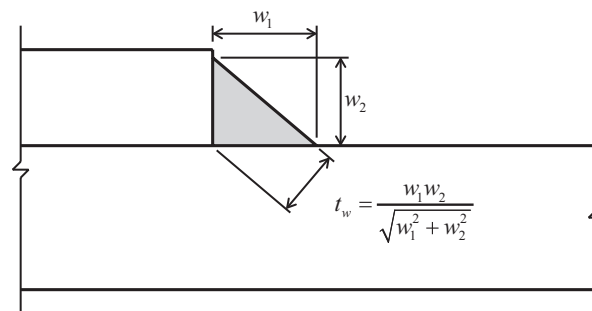


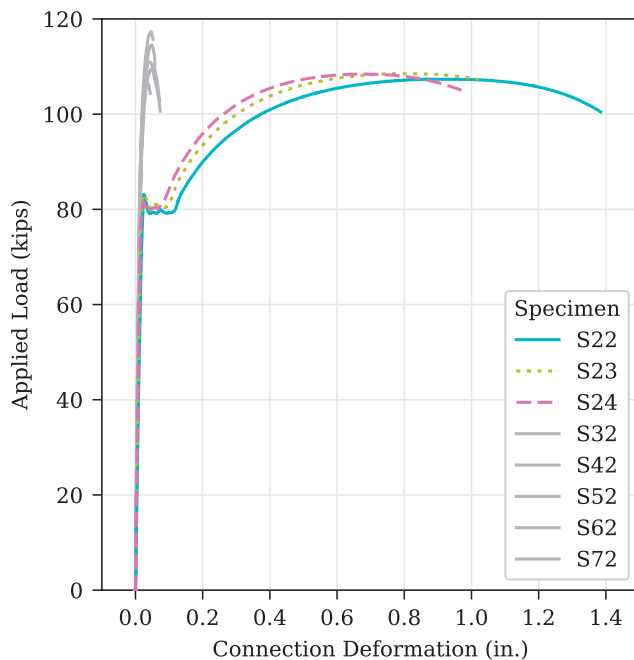
Fig. 10. Details of weld measurements.

Table 4. Experimental Results			
Specimen —	Peak Load, P_{exp} kips	Failure Mode —	$U_{exp} = P_{exp}/F_u A_n$ —
S22	107.3	Plate rupture	0.988
S23	108.2	Plate rupture	0.996
S24	108.2	Plate rupture	0.996
S32	114.3	Weld rupture	0.702
S42	110.6	Weld rupture	0.509
S52	109.1	Weld rupture	0.402
S62	116.9	Weld rupture	0.359
S72	106.3	Weld rupture	0.279

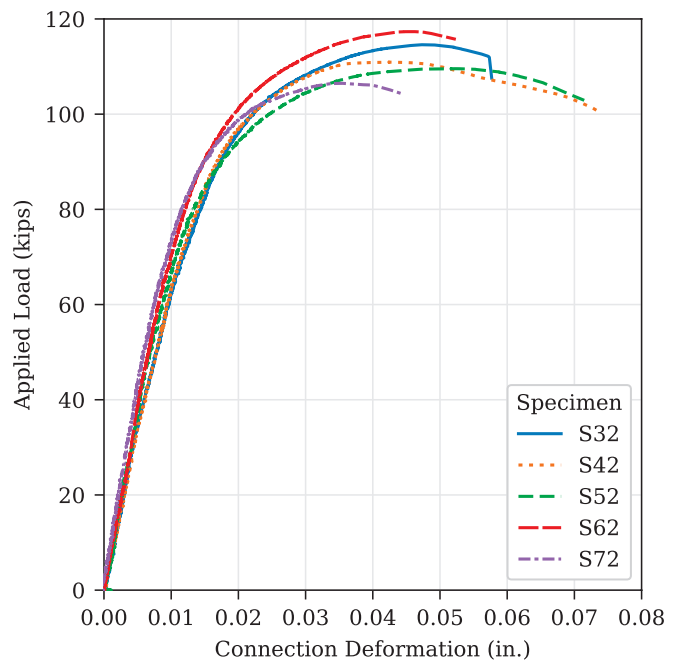
gradual softening, eventually reaching a peak load. After the peak load, the load dropped gradually until weld rupture occurred (as shown in Figures 13 and 14) and the connection lost load-carrying capacity. Failure of the welds occurred simultaneously and was generally through the throat at around 45°. Signs of minor shear yielding of the test plates in some locations adjacent to the weld were observed as shown in Figure 15, but no other damage to the test plates was observed.

DISCUSSION

Table 5 lists strengths for the specimens calculated according to the AISC *Specification* (2022) for each of the five limit states applicable to the test plates and welds: tensile yield (TY), tensile rupture (TR), shear yield (SY), shear rupture (SR), and weld rupture (WR). Equations for these limit states are shown in Table 1. For the strengths listed in Table 5, no resistance or safety factors were applied, measured values of F_y and F_u of the plate were used, measured



(a) All specimens



(b) Specimens that failed by weld rupture

Fig. 11. Measured load-deformation response.



Fig. 12. Photograph of specimens that failed in the plate.



Fig. 13. Photograph of specimens that failed in the weld.



Fig. 14. Photograph of weld failure for specimen S42.

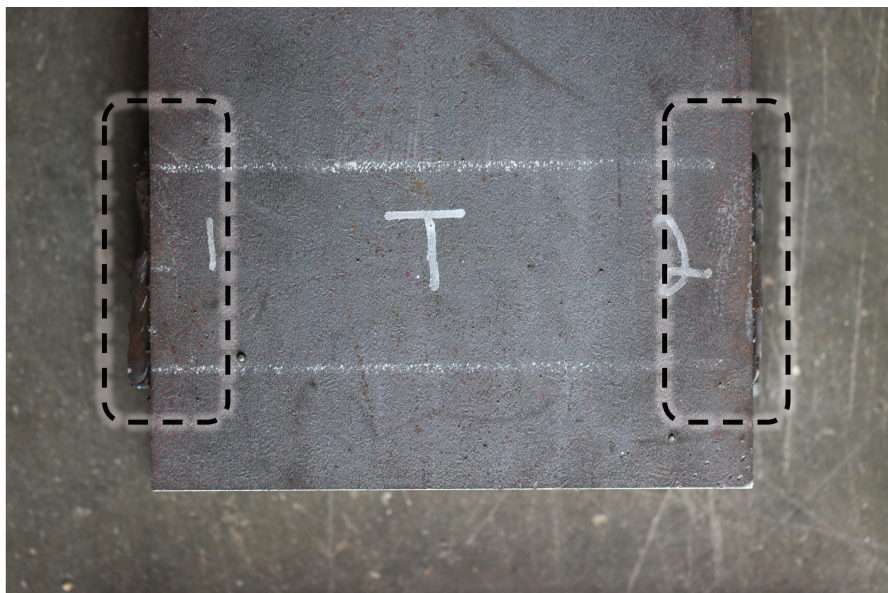


Fig. 15. Photograph of specimen S62 after testing with indications of shear yielding near the welds.

Table 5. Calculated Strengths for the Test Specimens

Specimen —	$P_{n,TY}$ kips	$P_{n,TR}$ kips	$P_{n,TR(U=1)}$ kips	$P_{n,SY}$ kips	$P_{n,SR}$ kips	$P_{n,WR}$ kips	P_n kips	$P_{exp}/P_{n,TR}$ —	$P_{exp}/P_{n,WR}$ —
S22	80.9	86.1	109	110	147	76.6	76.6	1.25	1.40
S23	80.9	96.2	109	156	210	102	80.9	1.12	1.06
S24	80.9	101	109	198	266	132	80.9	1.08	0.83
S32	121	101	163	107	143	70.7	70.7	1.14	1.62
S42	162	101	217	104	140	72.8	72.8	1.10	1.52
S52	202	102	272	109	146	77.1	77.1	1.07	1.42
S62	243	97.3	326	110	147	76.6	76.6	1.20	1.53
S72	283	96.1	380	114	153	81.8	81.8	1.11	1.30

plate and weld dimensions (as shown in Table 3) were used, and the nominal weld strength was used. The minimum calculated strength, P_n , is shown along with select test-to-predicted ratios.

According to the AISC *Specification* equations, the specimens were controlled by either weld rupture or tensile yielding with controlling strengths ranging from 70.7 to 81.8 kips. All eight specimens had an experimental strength higher than their respective controlling strength according to AISC *Specification* equations. The calculated tensile rupture strength ranged from 86.1 kips for specimen S22 to 102.1 kips for specimen S52. All eight specimens also surpassed their calculated tensile rupture strength. For five of the eight specimens the experimental strength exceeded or was within 3% of the calculated shear yielding strength.

For the three specimens that failed by tensile rupture (i.e., S22, S23, and S24), the experimental strengths were nearly identical. They were also close to the tensile rupture strength calculated using $U = 1$. The only difference in these three specimens are their respective weld lengths represented by the second number in their name. The ratio of experimental strength to calculated tensile rupture strength (i.e., $P_{exp}/P_{n,TR}$) decreases toward unity with increasing weld length. This is due to the shear lag factor computed using Equation 1 increasing toward unity. These results, and the fact that the rupture occurred away from the welds, indicate that there were minimal shear lag effects in these three failed specimens despite $U < 1$ according to the AISC *Specification*.

For the five specimens that failed by weld rupture, the true tensile strength of the plates is not known. However, since the plates did not fail, it can be concluded that the tensile rupture strength of these specimens is greater than the experimental strength. Furthermore, the experimental strength for these specimens is greater than the calculated tensile rupture strength. Therefore, the shear lag factor is conservative, but to an unknown extent.

It is common in previous research, including Fortney and Thornton (2012), to compute an experimental shear lag factor as the experimental strength divided by the product of the tensile strength and net area:

$$U_{exp} = \frac{P_{exp}}{F_u A_n} \quad (2)$$

The experimental shear lag factor can be computed for any specimen, including those not failing by tensile rupture. Fortney and Thornton (2012) argued that experimental shear lag factors computed for specimens failing by weld rupture were relevant to the shear lag factor for tensile rupture. However, they are relevant only as a lower bound of the true shear lag factor depending on how close the specimen was to tensile rupture failure when the weld ruptured.

Figure 16 shows that the experimental shear lag factor for the specimens tested in this study decreases significantly as the width between the welds increases (i.e., l/w decreases) and the trend is well captured by Equation 1. However, for the specimens that failed by weld rupture, there was no indication of distress in the net section that would indicate that tensile rupture was imminent. Furthermore, other limit states (e.g., weld rupture and shear yielding) limit the calculated strength such that the overall calculated strength of the specimen is conservative, even if $U = 1$.

The five specimens that failed by weld rupture failed at loads greater than calculated using measured weld dimensions and nominal weld material properties. The ratio of test strength to calculated weld strength is 1.46, which is substantial but not out of the ordinary for similarly calculated test-to-predicted ratios for longitudinally loaded welds (e.g., Dowswell and Cox, 2024). Interestingly, the specimens achieved roughly the same strength despite the distance between the welds being different. These five specimens all had weld lengths of 2 in. and had different

widths between welds, represented by the first number in their name. The lack of variation in strength indicates that a shear lag factor is not needed for weld failures. The lack of variation also indicates that violating the requirement in AWS D1.1, clause 4.9.2, that the width between welds must be less than the length of the welds will not necessarily result in lower-than-expected strength. While these five specimens are not permitted according to AWS D1.1, they exhibited no obvious weaknesses in this study.

Given that all specimens tested in this work were stronger than expected and could be safely designed with $U = 1$, it is worth evaluating the other experimental data on plate tension members using $U = 1$ as well. This comparison includes 336 experiments: the plate specimens in the database of previous experimental results plus those tested in this study. For each of the specimens, strengths were computed for the five limit states listed in Table 1, except that U was taken as 1 for these calculations. Measured material and geometric properties as reported by the experimentalists were used. Mannem (2002) did not report the size of the welds; therefore, weld strength was not calculated for their 16 plate specimens. The lowest strength from the five limit states was taken as the controlling strength (denoted as P_n). The ratio of experimental strength to the controlling calculated strength (i.e., the test-to-predicted ratio P_{exp}/P_n) is plotted against the length-to-width ratio in Figure 17. The shape of the markers indicates the failure mode that controls the calculated strength, and the color of the markers indicates the reference describing the experiments.

Across the 336 plate tension member specimens, only 18 specimens have a controlling test-to-predicted ratio less than unity, 17 of which failed in the weld and one that failed in the member. These relatively few test-to-predicted ratios less than unity are not necessarily indicative of unsafe provisions because resistance and safety factors are calibrated to account for the scatter. No other specimens had a test-to-predicted ratio less than unity, which is especially notable since this evaluation used $U = 1$ and not Equation 1 for tensile rupture. Furthermore, there are no obvious trends between the test-to-predicted ratio and l/w , indicating that use of $U = 1$ (and thus no explicit use of l/w) in the strength equations is appropriate.

The calculated strength for all specimens is either controlled by the tensile yielding, weld rupture, or shear yielding limit states. With $U = 1$, tensile rupture did not control for any of the specimens. Tensile rupture may still control in a design situation where resistance factors or safety factors are applied.

CONCLUSIONS

The shear lag factor defined in Case 4 of AISC *Specification* Table D3.1 (2022) implies that net section tensile rupture is a prominent failure mode for tension members connected by short longitudinal welds. However, the shear lag factor in Case 4 was developed using data from experimental specimens that failed by weld rupture. With short welds, it is logical that weld rupture or base metal strength controls

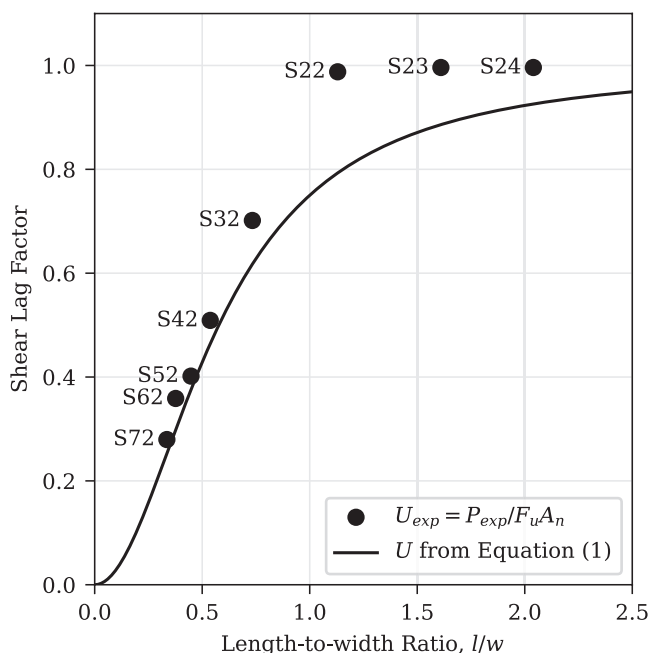


Fig. 16. Comparison of experimental shear lag factor to AISC shear lag factor ($\bar{x} = 0$).

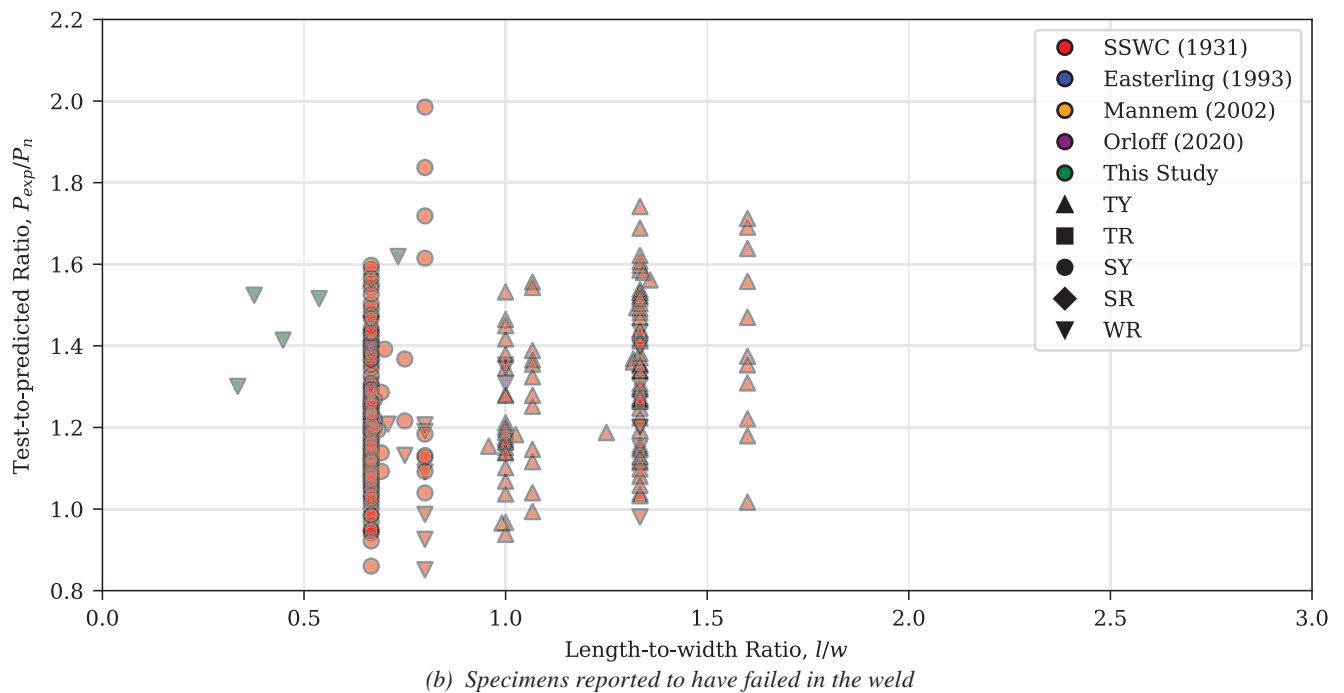
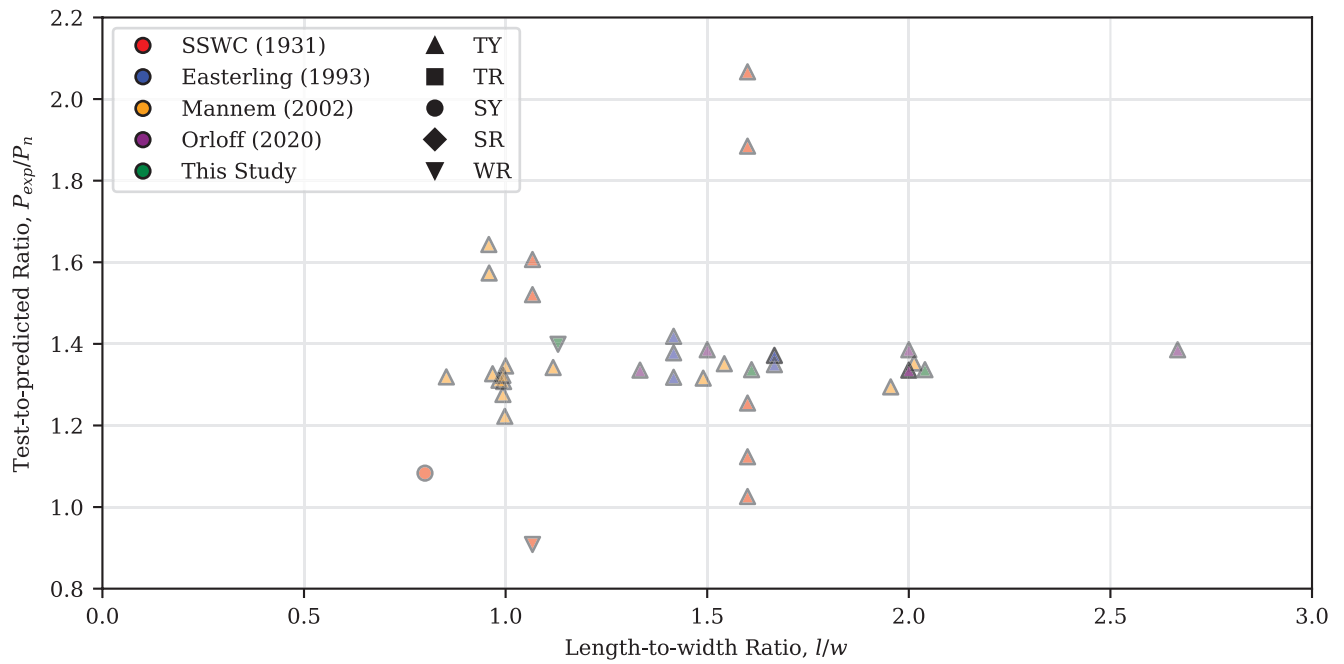


Fig. 17. Ratio of experimental strength to controlling calculated strength versus weld length-to-width ratio.

over member failures such as tensile rupture. To help clarify behavior and start building justification for simplifications to the AISC *Specification*, past experimental work was reviewed, and new experiments were performed in this study. The new experiments were of double-plate tension members and were unique in their use of very short welds with length-to-width ratios as low as 0.336. The primary conclusions of this study are as follows:

- Regardless of the weld length-to-width ratio or failure mode, all eight specimens failed at approximately 110 kips, indicating that shear lag had minimal impact on the tensile rupture strength and weld rupture strength of these specimens despite design provisions indicating otherwise.
- Using a shear lag factor of $U = 1$ for the eight plate specimens tested in this work results in calculated strengths that are safe and accurate in comparison with their experimental strengths.
- Using a shear lag factor of $U = 1$ for the hundreds of plate specimens tested in previous research results in calculated strengths that are safe and accurate in comparison with their experimental strengths.
- Use of $U = 1$ for single or double plates with short longitudinal welds is safe because other limit states, including weld rupture and shear yielding, control over tensile rupture.

These results indicate that simplifications to the tensile rupture provisions in the AISC *Specification* are possible and that the simplifications could enable design calculations that better represent the physical behavior of tension members. However, additional research is needed to fully justify such changes. Specifically, more data is needed for tension members other than plates, including angles and channels, especially those connected with short longitudinal welds.

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