SHEAR LAG IN DOUBLE ANGLE TRUSS CONNECTIONS

D.B. Bauer[1] and A. Benaddi[2]
[1, 2] Department of Construction Engineering, École de technologie supérieure,
Montréal, Canada H3C 1K3

ABSTRACT

Shear lag can be described as a phenomenon that creates a loss in resistance in a tension member connected through only part of its cross-section. It is a complex problem which has been under study for many years by researchers. Parameters that influence the shear lag phenomenon are many and difficult to assess: type and size of cross-section, type of connection, length of welds, length of member, joint eccentricities, etc. Connection between double-angle web members and chords in trusses or open web steel joists are considered herein. Resistances and modes of failures are described as determined experimentally. Yield and ultimate loads are compared with the values calculated using the design guidelines of the Canadian Standard, which are reviewed in the article. Finally, tentative conclusions are drawn regarding the influence of shear lag in double angle truss connections.

KEYWORDS

Shear lag, steel structure, truss, welded connection, double-angle, tension web member.

INTRODUCTION

Steel trusses fabricated with double-angle web members are considered herein, in which joints are made economically by welding one leg of the web angles directly to the chord member. See Figure 1. Tension web members can be designed using Clause 12.3.3. of Canadian Standard S16.1-94 Limit States Design of Steel Structures (CSA 1994) in which shear lag is taken into account. Shear lag is a phenomenon that affects tension members connected at one or both ends through only part of the cross-section. Tensile stresses are transferred from the member into the connected parts and the stress distribution along the connection is non linear, resulting in a loss of strength. Yet, according to certain fabricators, the strength of such connections could be higher than suggested by current Standards.

LITTERATURE REVIEW

The amount of shear lag present in a connection is calculated using the $1-\bar{x}/L$ function, where $\bar{x}$ and L are an eccentricity and a characteristic length of the connection, respectively. See Figure 2. This function was presented by Munse and Chesson (1963) in an article dealing mostly with riveted and bolted connections. In his book on steel structures, McGuire (1968) computes bending stresses due to joint eccentricities for angle web members connected through one leg. He explains that the connection is strong enough for the gross section to reach yield and that, because of stress redistribution, the ultimate strength of the connection is only slightly affected. According to McGuire, this justifies that no reduction in the net area was required at the time in the AISC Standard. He mentions the work of Munse and Chesson for built-up sections, but questions the relevance of using the $1-\bar{x}/L$ function for connections that are not solicited in fatigue. In a recent study, Kirkham and Miller (2000) comment the AISC Standard (1993) regarding the treatment of shear lag. The authors point out that the definition of the eccentricity, $\bar{x}$, is not clear and should be revised. They mention that parts of the design recommendations are in fact based on extrapolations of previous research findings or empirical rules that have not been verified experimentally. They observe that most tests have been done on samples that were too short to develop a uniform stress distribution away from the connection according to St-Venant's principle; the short length of samples affected the stress distributions and was a source of errors. Furthermore, tested samples did not cover the full range of cross-section sizes. The authors recommend that tests be done on full size specimens in order to verify whether tests on small samples are valid, as well as tests and parametric studies on specimens covering a wide range of cross-section sizes.

THE CURRENT CANADIAN CODE

The resistance of a member subjected to uni-axial tension is given in Canadian Standard S16.1-94 Limit States Design of Steel Structures (CSA 1994). For double-angle members with welded connections, the factored tensile resistance, T_r, is to be taken as the least of (Clause 12.2 a)):

$$T_r = \phi A_g F_y \tag{1}$$

$$T_r = 0.85\phi A'_{ne} F_u \tag{2}$$

where ϕ is a performance factor, A_g is the gross cross sectional area, A'_{ne} is the effective net area reduced for shear lag, F_y is the yield strength and F_u is the ultimate strength. Eqn. 1 represents full plasticity along the entire length of the member occurring when the gross cross-section reaches yield, which is a desirable mode of failure, and no reduction due to shear lag is included. Eqn. 2 represents rupture of the effective net area reduced in order to take into account shear lag occurring at the joint. The 0.85 factor takes into account the fact that there is no reserve in strength beyond fracture. For welded connections, the reduced effective net area is computed as:

$$A'_{ne} = A_{ne1} + A_{ne2} + A_{ne3} \tag{3}$$

where A_{ne1}, A_{ne2} and A_{ne3} are the effective net area of the connected parts computed in the following manner:

a) for a part connected by a transverse weld

$$A_{ne1} = wt \tag{4}$$

where w is the width and t is the thickness of the part.

b) for a part connected by longitudinal welds along two parallel edges

$$\begin{aligned} &\text{(i) when } L \geq 2w, && A_{ne2} = 1.00wt \\ &\text{(ii) when } 2w > L \geq 1.5w, && A_{ne2} = 0.87wt \\ &\text{(iii) when } 1.5w > L \geq w, && A_{ne2} = 0.75wt \end{aligned} \tag{5}$$

where L is the average length of welds on the two edges.

c) for a part connected by a single line of welds

$$A_{ne3} = \left(1 - \frac{\bar{x}}{L}\right) wt \tag{6}$$

where $\bar{x}$ is the eccentricity of the weld with respect to the centroid of the part and L is the length of the weld in the direction of the loading.

NUMERICAL EXAMPLE USING THE CANADIAN CODE

Check the tensile load resistance of a 2-L76x51x4.8 double-angle member with short legs back to back, connected with longitudinal welds. CSA-G40.21 380W steel with F_y = 380 MPa and F_u = 480 MPa. In order to calculate the required length of weld, L, it is assumed that the member must carry a factored load, T_f, equal to the factored yield resistance, T_r, of the member:

$$T_r = \phi A_g F_y = 0.9 \times 2 \times 582 \text{ mm}^2 \times 380 \text{ MPa} = 398 \text{ kN} \tag{7}$$

As is often the case in trusses and due to fabrication constraints, the welds will be longitudinal and of equal length on either side of the connected leg, and no transverse weld will be used. With a weld size of 5 mm and E480XX electrodes, the required length of weld, L, is equal to:

$$L = \frac{398 \text{ kN}}{0.762 \text{ kN/mm} \times 2 \text{ angles} \times 2 \text{ sides}} = 131 \text{ mm} \tag{8}$$

Since the length of the welds, L = 131 mm, is more than twice the width of the connected leg, there is no reduction due to shear lag (see Eqn. 5 above). Therefore:

$$A_{ne2} = 1.00 \times (51 - 4.76) \text{ mm} \times 4.76 \text{ mm} = 220 \text{ mm}^2 \tag{9}$$

For the outstanding leg, the eccentricity, $\bar{x}$, is equal to $76/2 = 38$ mm and the effective net area of this leg is equal to (from Eqn. 6 above):

$$A_{ne3} = \left(1 - \frac{38}{131}\right) 76 \text{ mm} \times 4.76 \text{ mm} = 257 \text{ mm}^2 \tag{10}$$

Hence, the reduced effective net area of a complete angle is equal to:

$$A'_{ne} = 220 \text{ mm}^2 + 257 \text{ mm}^2 = 477 \text{ mm}^2 \tag{11}$$

The ultimate tensile resistance of the double-angle member is equal to:

$$T_r = 0.85\phi A'_{ne} F_u = 0.85 \times 0.90 \times 2 \times 477 \text{ mm}^2 \times 480 \text{ MPa} = 350 \text{ kN} \tag{12}$$

The latter value is lower than 398 kN found using Eqn. 7, hence it governs. The reduction in the ultimate resistance of the cross-section due to shear lag is equal to:

$$\% \text{ reduction} = 1 - \frac{A'_{ne}}{A_{ne}} = 1 - \frac{477 \text{ mm}^2}{582 \text{ mm}^2} = 18\% \tag{13}$$

The length of weld required to resist a load of 350 kN is somewhat smaller than 131 mm calculated with Eqn. 8 above. A shorter length of weld would lower the A_{ne3} value as well as the ultimate tensile resistance, T_r, calculated using Eqn. 12, and this again would require a smaller length of weld. After a few cycles of such calculations, the ultimate resistance is found to be equal to 336 kN, the required weld length is equal to 110 mm and the percent reduction due to shear lag is equal to 21 %. The effect of shear lag reduces the design member resistance only if Eqn. 2 governs over Eqn. 1. For 300W steel with $F_y = 300$ MPa and $F_u = 450$ MPa, this will be the case when the reduction in area due to shear lag is greater than 22 %. With higher strength steels, for example 380W with $F_y = 380$ MPa and $F_u = 480$ MPa, Eqn. 2 will govern if the reduction in area due to shear lag is greater than 7 %.

LABORATORY TESTS

Test samples

Laboratory tests on simplified specimens were carried out recently by the authors. Six specimens were tested with double-angle members ranging in size from 2-L38x38x4.8 up to 2-L76x76x4.8. The main test parameters are shown in Table 1. All specimens had an overall length of 2500 mm, the longest possible that would fit conveniently in the testing machine, which left a clear length for the double-angle member of over one meter between connections. Specimens were built symmetrical, except for the welds that were sized such that one end only would fail during the tests. Welds at the joints under study were longitudinal and of equal length, and no transverse weld was used, as it is the preferred practice for truss joints. The centroid of the double angles was aligned with the centerline of the end gusset plates in order to eliminate eccentricity measured parallel to the gusset plates. The other eccentricity, measured perpendicular to the gusset plates, is unavoidable for that type of connection. Two coupons per specimen were prepared and tested according to Canadian Standard CAN/CSA-G40.20-98 General Requirements for Rolled or Welded Structural Quality Steel (CSA 1998). Average measured yield and ultimate strengths are given in Table 1. Measured values match those of CSA G40.21 Grade 300W steel with a minimum specified yield value $F_y = 300$ MPa and an ultimate strength $F_u = 450$ to 620 MPa, rather than Grade 380W which was initially assumed in the design of the test specimens. Cross sectional areas calculated using measured widths and thicknesses were 1.5 % on average higher than the nominal areas, equal to $(b_1 + b_2 - t) \times t$ where b_1 and b_2 are the width of the legs and t is the thickness. Because the actual cross sectional area is difficult to measure yet close to the nominal value, the nominal cross sectional areas were used in calculating the expected yield and ultimate loads of the specimens. Table 2 shows calculations of the effective net area for both legs of the angles. For all specimens, the width of the connected leg is smaller than twice the weld length and hence there is no reduction in area due to shear lag for that leg. For the outstanding leg, all specimens have an area reduced to a value between 72 % and 78 %. The predicted reduction in area for the complete angles varies between 12 % and 17 %. The weld length was calculated in order to resist the double-angle member yield load assuming $F_y = 380$ MPa and this lead to somewhat long welds. Further tests with smaller assumed loads and shorter welds could be needed.

Test set-up and Instrumentation

Tests were done in a MTS universal testing machine with a capacity of 11 000 kN, located in the Structures Laboratory of the Department of Civil Engineering and Applied Mechanics at McGill University. See Figure 3. The specimens were loaded in tension under quasi-static conditions. Loading was displacement controlled at a fixed rate of 0,01 mm/sec up to the beginning of strain hardening and then was increased to 0,1 mm/sec up to failure. The applied load was measured with the load cell integral with the testing machine. The displacement of the loading end was measured with the LVDT integral with the machine. Eight strain gauges were placed on each specimen at the connection under study, two on the connected leg and six on the outstanding leg. Strains were measured up to 1.5 %, that is, about ten times the yield strain. The measured strains will be compared elsewhere with strain distributions obtained from finite element models of the joints. Two LVDT's were placed perpendicular to the double-angle member at mid-height in order to measure transverse deformations. On each specimen, one angle was covered with whitewash that revealed zones of high strains by spalling during the tests. A Vishay 6000 data acquisition system was used to record all the loads, displacements and strains.

Test Results and Discussion

The load versus overall deformation curves for the six specimens are shown in Figure 4. All specimens underwent yielding over their entire length, strain hardened and failed with large plastic deformations. Specimen No. 2 reached 18 % overall strain and the other specimens reached about 10 % strain. Specimen No. 2 failed in a tensile mode with the final break located towards the center of the double-angle member. The other specimens failed also in tension, either very close or right at the connection under study. The failure mode of specimen No. 4 is shown in Figure 5. Predicted as well as experimental yield and ultimate loads are given in Table 3. Loads were predicted using measured values of F_y and F_u and the reduced effective net areas from Eqns. 5 and 6. Comparisons between these loads are given in the same Table. Regarding the yield loads, only specimen No. 2 tested 10 % lower than the predicted value. All other specimens yielded at about the predicted load with specimen No. 3 being 10 % stronger than predicted. Concerning the behaviour at ultimate load, all specimens were between 3 % and 20 % stronger than the $A'_{ne}F_u$ values, with an average of 12 %. Test results were on average only 3 % lower than A_gF_u values, specimen No. 2 being 9 % lower and specimen No. 3 being 2 % higher. These latter values, i.e. T_u/A_gF_u ratios given in Table 3, are experimental reduction factors which can be compared directly with the predicted reduction factors given in Table 2 for the complete angles.

CONCLUSIONS

This section summarizes the conclusions of this paper. These are briefly given below:

1. Six double-angle members, with sizes ranging from 2-L38x38x4.8 up to 2-L76x76x4.8 and connected with equal length longitudinal welds, were tested in tension. All specimens yielded over their entire length, strain hardened and broke with a final overall strain of about 10 % (18 % for specimen No. 2).

2. For all specimens, there was no reduction in the yield load from that predicted using measured values of the yield strength, except specimen No. 2 for which there was a 10 % reduction. This agrees with the recommendation found in current Standards that no reduction due to shear lag need be considered for calculating the yield resistance of tension members.

3. The ratio of experimental to predicted ultimate load, $T_u/A'_{ne}F_u$, was 1.19 and 1.20 for specimens Nos. 3 and 5 with double angles connected with the short legs back to back. For equal leg double

angles, the ratio was between 1.09 and 1.11 for specimens Nos. 1, 4 and 6, and 1.03 for specimen No. 2.

4. The experimental ultimate load was 3 % on average less than the predicted load, $A_g F_u$, based on the gross cross sectional area of the member. In other words, a 3 % reduction in area due to shear lag was found experimentally, compared to a value of 13 % predicted using current design recommendations.

It can be concluded that, with the limited test data presented here, design recommendations found in the Canadian Standard are adequate regarding the yield resistance and somewhat conservative regarding the effect of shear lag and the ultimate resistance. In order to get a better understanding of the joint behaviour, further analyses of the test results will include comparison between strain gauge readings and stresses calculated using finite element models of the joints, as well as calculations of the bending stresses due to joint eccentricities. Also, a second series of tests on complete truss panels is being prepared in order to determine the influence of more realistic loading conditions on the behaviour of double-angle tension web member joints.

ACKNOWLEDGMENTS

The present research was made possible by grants from the Steel Structures Education Foundation of the Canadian Institute of Steel Construction for the years 2001 and 2002, from the National Sciences and Engineering Research Council of Canada (N° 238967-01), and from the École de technologie supérieure (PSIRE-Research 2001 and 2002). The authors would like to thank Canam Steel for giving the test samples, and the Department of Civil Engineering and Applied Mechanics of McGill University for the complimentary use of their universal testing machine.

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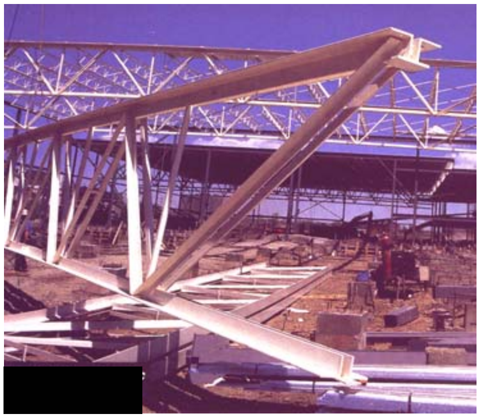

Figure 1: Typical truss with double-angle web members (Canam Steel)

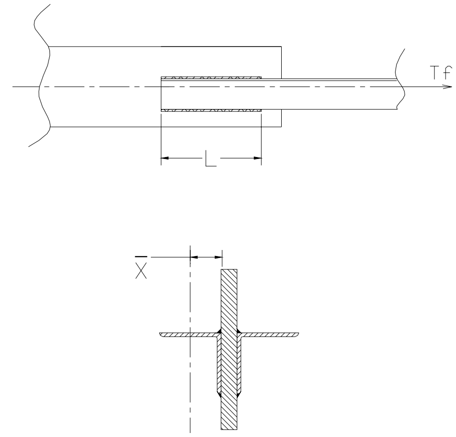

Figure 2: Connection with equal length longitudinal welds

Figure 3: Test set-up

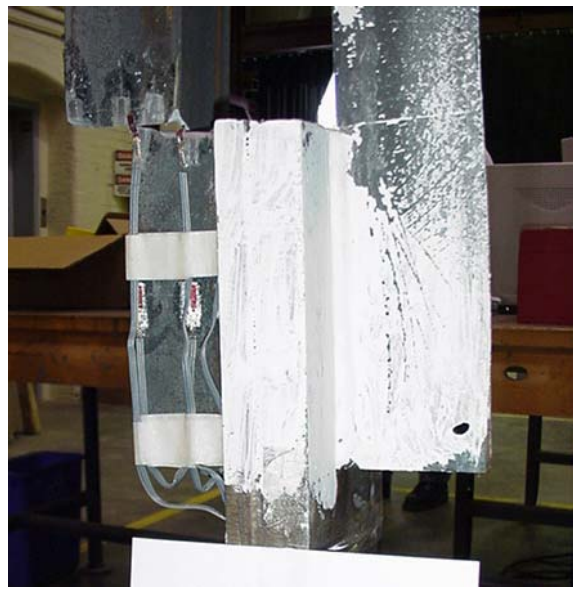

Figure 5: Failure of specimen No. 4

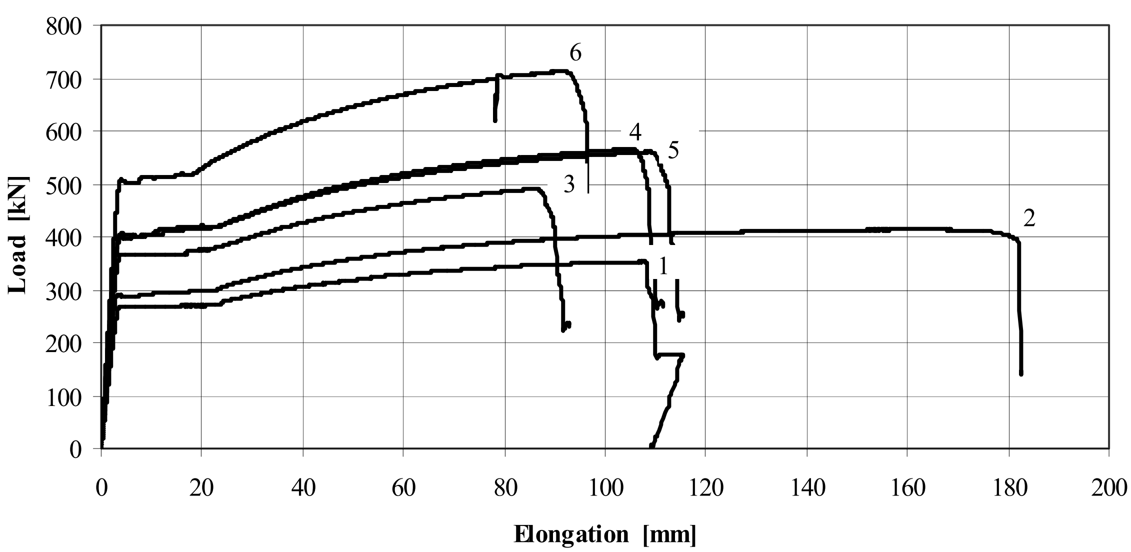

Figure 4: Load versus overall displacement for specimens Nos. 1 to 6

TABLE 1
TEST PARAMETERS AND AVERAGE MEASURED COUPON STRENGTH

No.	Size [mm x mm x mm]	Arrangement	Gross Area of One Angle [mm^2]	Weld Size [mm]	Measured Weld Length [mm]	Yield Strength F_y [MPa]	Ultimate Strength F_u [MPa]
1	2L-38 x 38 x 4.8	Equal Legs	340	5	87	393	531
2	2L-51 x 51 x 4.8	Equal Legs	461	5	112	349	492
3	2L-64 x 51 x 4.8	Short Legs Back to Back	521	5	122	318	461
4	2L-64 x 64 x 4.8	Equal Legs	582	5	136	345	499
5	2L-76 x 51 x 4.8	Short Legs Back to Back	582	5	138	339	487
6	2L-76 x 76 x 4.8	Equal Legs	703	5	169	348	527

TABLE 2
CALCULATION OF THE EFFECTIVE NET AREA REDUCED DUE TO SHEAR LAG

No.	Connected Leg			Outstanding Leg			Complete Angle	
	w_2 [mm]	Reduction Factor	A_{ne2} [mm^2]	w_3 [mm]	Reduction Factor	A_{ne3} [mm^2]	A'_{ne} [mm^2]	Reduction Factor [%]
1	38	1.00	158	38	0.78	141	300	0.88
2	51	1.00	220	51	0.77	187	408	0.88
3	51	1.00	220	64	0.74	225	445	0.85
4	64	1.00	282	64	0.76	233	515	0.88
5	51	1.00	220	76	0.72	262	482	0.83
6	76	1.00	339	76	0.78	280	620	0.88
							Average	0.87

TABLE 3
PREDICTED AND EXPERIMENTAL STRENGTHS OF SPECIMENS

No.	Predicted Strengths				Experimental Strengths		Ratios of Experimental / Predicted Strengths		
	$A_g F_y$ [kN]	$0.85 A'_{ne} F_u$ [kN]	$A'_{ne} F_u$ [kN]	$A_g F_u$ [kN]	T_y [kN]	T_u [kN]	$\frac{T_y}{A_g F_y}$	$\frac{T_u}{A'_{ne} F_u}$	$\frac{T_u}{A_g F_u}$
1	267	271	318	361	268	353	1.00	1.11	0.98
2	322	341	401	453	288	414	0.90	1.03	0.91
3	331	348	410	480	365	490	1.10	1.20	1.02
4	402	437	514	581	398	566	0.99	1.10	0.97
5	394	399	470	567	401	561	1.02	1.19	0.99
6	489	555	653	741	502	713	1.03	1.09	0.96
						Average	1.01	1.12	0.97