The shear lag effects on welded steel single angle tension members

H.T. Zhu\textsuperscript{a}, Michael C.H. Yam\textsuperscript{b}\textsuperscript{*}, Angus C.C. Lam\textsuperscript{a}, V.P. Iu\textsuperscript{a}

\textsuperscript{a}Department of Civil and Environmental Engineering, University of Macau, Macao, China

\textsuperscript{b}Department of Building and Real Estate, The Hong Kong Polytechnic University, Hung Hom, Kowloon, Hong Kong, China

Abstract

This paper presents a study of the shear lag effects on the behaviour and strength of welded steel single angle tension members. A total of thirteen single angles with welded end connections were tested in tension. The test parameters included long and short leg connections, balanced and unbalanced weld arrangements and longitudinal fillet weld lengths. Out of the thirteen specimens, nine failed by fracture of the gross section and four failed in the welds. The efficiency of the specimens, which is defined as the ratio of the test ultimate load ($P_{\text{test}}$) to the tensile capacity (a product of the gross sectional area and the tensile strength of the material) of the specimens varied from 0.82 to 1.02. It can be observed from the test results that both the ultimate loads sustained by the short leg connected angles and the ductility of all the angle specimens were greater when the balanced weld arrangement was used in the connections than when the specimens were connected using the unbalanced welded arrangement. Finite element analyses of the specimens were conducted and the analysis results compared well with the test results. The capacities of the test specimens were also evaluated using various design approaches. In general the design specifications (AISC-LRFD, BS5950-1:2000, and CSA-S16-01) provided good predictions of the tensile capacity of the single angle specimens with a reasonable degree of conservatism. However, the
design specifications underestimated the tensile capacity of specimens which were
connected by the short leg and with a balanced weld arrangement.

*Corresponding author. Tel.: +852 2766 4380; fax: +852 2764 5131
Email address: bsmym@polyu.edu.hk (Michael C.H. Yam)

1. Introduction

In steel construction, hot-rolled structural shapes such as angles, tee-sections and
channels are often used as tension members. These members are either bolted or
welded to the connecting elements (e.g. a gusset plate) as shown in Fig. 1. It is
common practice with these shapes to connect only part of the cross-section at the
connections. Because of this, only part of the section is effective in carrying the loads,
and this leads to the phenomenon known as shear lag. Shear lag results in the non-
uniform distribution of stress in tension members (Fig. 1). In addition, since the line of
action of the load usually does not coincide with the centroidal axis of a tension
member section, loading eccentricity is also created and hence secondary bending of
the member is induced. The combined effects of shear lag, connection eccentricity and
stress concentrations at the connected region would initiate premature fracture of the
section and the tensile capacity of the member is significantly reduced.

The commonly used equation to account for the effects of shear lag in tension
members was developed by Munse and Chesson [1, 2]:

\[
K_x = 1 - \frac{x}{L}
\]  

(1)
where $K_4$ = shear lag coefficient, $\bar{x}$ = distance from the shear plane to the centre of gravity of the material connected to the shear plane and $L$ = connection length, taken as the distance between the extreme fasteners as illustrated in Fig. 2. This equation was derived from tests on bolted and riveted connections and the product of the net area of the member and the shear lag coefficient, $K_4$, is defined as the effective net area. This equation was further verified by a comparison with data from more than 1000 tests (Munse and Chesson [1]) and the equation has been adopted in a number of design specifications such as the AISC-LRFD [3] and the CSA/CAN-S16-01 [4] for tension members with either bolted or welded end connections.

The shear lag effects on the strength and behaviour of tension members with welded end connections have been studied by a number of researchers. Davis and Boomsliter [5] conducted research to evaluate the strength of tension members composed of angles with welded or riveted joints. The ratio of the ultimate load to the full tensile capacity was established at 0.87 for double angles failing in the net section. The test results from Gibson and Wake [6] showed little difference between the ultimate strengths of welded angle specimens with balanced welds and those with unbalanced welds. Regan and Salter [7] conducted seventeen single angle tests with unbalanced equal longitudinal welds and transverse welds. Based on the test results, the proposed capacity equation for the angle tension members was $P_t = p_y [A_g - 0.2a_2]$, where $P_t$ is the design tension capacity; $p_y$ is the design material strength of steel; $A_g$ is the gross cross-sectional area; and $a_2$ is the gross area of the unconnected leg. However, it should be noted that the majority of the specimens in the study were of relatively small size. Gonzalez and Easterling [8] conducted tension tests on double angles, plates and channels welded to a gusset plate. The test results indicated that the
shear lag coefficients for angles in tension were not affected by the presence of the transverse welds.

Uzoegbo [9] concluded that the shear lag effect should be evaluated individually for each leg of the angle specimen based on his test results for steel angles with welded connections. Petretta [10] tested 23 double angle specimens with welded end connections. The author recommended a modification to the Munse and Chesson shear lag equation to better fit the test results. Bauer and Benaddi [11] tested six specimens with welded double angles. Based on their limited test data, they concluded that the CSA/CAN-S16-01 [4] specification provides conservative predictions of the ultimate strength of welded double angles under conditions of shear lag effects. Abi-Saad and Bauer [12] proposed to calculate the reduced strength of steel tension members allowing for the shear lag effects based on an assumed distribution of forces in the member end.

Although the research work described above has investigated the effect of shear lag effect on the strength and behaviour of welded angles, channels and plates, more experimental data is required to properly examine the effects of shear lag on the strength and behaviour of single angle tension members connected by welds. The main objective of this study, therefore, was to provide more experimental data on the shear lag effects on larger size single angle tension members with welded end connections. The evaluation of the ultimate strength of the test specimens using current design specifications (AISC-LRFD [3], BS5950-1:2000 [13], CSA/CAN-S16.1 [4], BS EN 1993-1-8:2005 [14] and AS 4100-1998 [15]) is also presented below.

2. Experimental program

2.1. Test specimens
A total of thirteen single angles with welded end connections were tested in tension. The details of the specimens are shown in Tables 1 and 2. These tables should be read together with Figs. 3 and 4. The test parameters were as follows: (1) lengths of the longitudinal weld; (2) balanced and unbalanced weld arrangements (for balanced welds the longitudinal welds along the heel and the toe of the angle are designed to produce no eccentric moment about the centroidal line of the angle; for unbalanced welds equal lengths of longitudinal welds are used along the heel and toe of the angle as shown in Fig. 4); and (3) long and short leg connections. It should be noted that transverse welds were used for all the specimens. The specimen designation includes the type of section, length of weld, weld arrangement and the element which is connected. For example, A1-200BL represents type A1 (125 x 75 x 10) angle with a longitudinal weld length of 200 mm, balanced weld arrangement (B) and connected by the long leg (L).

Two angle sections (Type A1 and A2) were used to fabricate the test specimens (SCI Guide [16]) as illustrated in Table 1. Grade S275 steel conforming to BS EN 10025-2:2004 [17] was originally requested for the angle specimens. However, it can be seen below from the material test results that the steel materials were believed to be S235 instead of S275. All the gusset plates were fabricated with 400 mm wide and 16 mm thick steel plates conforming to BS EN 10025-2:2004 [17] Grade 355 steel with different lengths to meet the required connection dimensions.

A clear space between the two inner edges of gusset plates was maintained at 800 mm for all the specimens in order to fit the tension testing machine as shown in Fig. 5. The total length of the specimens varied from 1200 mm to 1460 mm to allow for different longitudinal weld lengths. The distance from the start of the angle to the free edge of the gusset plate was 170 mm for all specimens. Specimens were designed to ensure that failure would occur at the section under investigation. A weld size of 8 mm
was used for the angle specimens. Tension coupon tests were conducted according to
the ASTM A370 [18] to determine the actual material properties of the specimens.

2.2. Test setup, instrumentation and test procedure

A photo of the test arrangement is shown in Fig. 6. A SATEC universal testing
machine with a 2000 kN tensile capacity was employed. The two end fixtures, which
were fixed into the two cross-heads of the machine, were properly designed to respond
elastically during the entire loading process. The specimens were installed vertically
into the end fixtures using seven 24 mm diameter Grade 8.8 bolts (BS5950-1:2000
[13]) at each end of the gusset plates. The specimens were then loaded in tension under
quasi-static conditions.

The applied load was measured by the built-in load cell of the testing machine. The
total extension of the specimens which included the elongation of both the specimens
and the gusset plates, the elastic deformation of the end fixtures and the elastic
shortening of the alloy columns of the testing machine, were obtained from the cable
transducers integrated with the testing machine. In order to record the longitudinal
strain distribution over the section, six longitudinal strain gauges were evenly located at
the critical section of the specimens near the outer ends of the longitudinal welds as
shown in Fig. 7. Another set of strain gauges was also mounted on the specimens at the
middle section of the clear space. Before testing, white wash coatings were applied
over the whole specimens to help detect yielding.

In general, the test procedures were similar for all tests. After aligning the
specimens in the testing machine, a small preload was applied to seat the specimen so
that the weight of the upper cross-heads was unloaded from the specimens and the bolts
properly bore against the gusset plate. Hence, major slippages of the connections were
eliminated. Once the installation of the specimens was completed, all readings were initialised before testing started. The loading process was divided into two stages: a load control at 50 kN for each step before yielding and a stroke control of 2.0 mm per min when yielding of the specimens had commenced. In addition, the stroke was held constant at regular intervals to allow the specimen to stabilize. The static load reading was obtained at the end of each such interval. The test was terminated when the gross section fractured or the welds completely failed.

3. Test results

3.1. Material test results

The tension coupon test results are shown in Table 3. It should be noted that the strengths of the materials shown in the table are slightly lower than that of a typical grade S275 steel. Based on the material test results, it is believed that grade S235 steel was supplied by the fabricator instead of the grade S275 expected. Nevertheless, the material exhibited the typical stress-strain behaviour of structural steel as shown in Fig. 8 and hence, the validity of the study would not be affected. As shown in Table 3, the average static yield strengths (static ultimate strength) were 268 MPa (418 MPa) and 262 MPa (432 MPa) for type A1 and A2 angle specimens, respectively. The final elongations of all the tension coupons were over 20%.

3.2. General

The test results including the ultimate loads of the specimens, $P_{\text{test}}$, and the final elongation of the specimens are illustrated in Table 2. At the beginning of the testing programme, two specimens failed in the welds. It was later discovered that the welding quality of some of the test specimens was not satisfactory. Therefore, in order to
ensure a gross section failure mode for the remaining specimens, the original 8 mm welds of the remaining specimens were reinforced to 10 mm. Consequently, altogether nine specimens failed by fracture of the gross sections while the other four specimens ultimately failed in the welds. It is believed that the specimens which failed in the welds were due to the original weld defects such as slag inclusions that could not be compensated by the weld reinforcements. However, all the specimens responded similarly when loaded in tension irrespective of the final failure modes.

Gross section fractures of the specimens mostly occurred at the section near the inner edge of the gusset plate, defined as the critical section as shown in Fig. 9a. Fractures usually started at the toe of the critical section and spread from the connected leg to the unconnected leg. In the case of the long leg connected specimens with balanced welds, yielding started at the heel of the critical section and then spread to the two legs, extending into the middle section at the same time. At ultimate, those specimens with balanced welds and long leg connections exhibited a necking process in the section before it ruptured a little further away from the inner edge of the gusset plate, as shown in Fig. 9b. The overall deformed shape of a typical angle specimen near ultimate is shown in Fig. 9c. Weld failures were observed in four of the test specimens. The weld failure always started in the heel weld where it was over-stressed. Once a heel weld failed, the rest of the weld ruptured rapidly due to the significant decrease of weld capacity.

Table 2 also shows the efficiency of the specimens, which is defined as the ratio of the test ultimate load \( P_{\text{test}} \) to the tensile capacity \( (A_g \times F_u) \) of the specimens. The efficiency of the angle specimens varied from 0.82 to 1.02. The short leg connected specimens combined with an unbalanced weld arrangement producing the smallest efficiencies.
3.3. Load deformation behaviour

Typical applied load versus elongation curves for the angle specimens are shown in Fig. 10. The horizontal axis elongation values are the total extension of the specimens obtained from the universal testing machine. The vertical axis values give the corresponding applied load. Figure 10 shows that the load elongation curves generally exhibit ductile behaviour. In particular, the ultimate elongations for long leg connected angle specimens are generally larger than those of short leg connected specimens with the same weld arrangement (i.e. balanced versus unbalanced). Ductile behaviour allows stress redistribution to occur and therefore, increases the ultimate loads the specimens can carry.

In general, linear load deformation behaviour was observed for all specimens in the early loading stage. As the applied load increased, bending of the gusset plates became visible causing the centroidal axis of the section to gradually approach the line of the applied load as illustrated by the deformation mode shown in Fig. 9c. Yielding usually occurred first at the edge of the connected leg around the critical section and then developed into the unconnected leg in the adjacent region as the applied load increased. Subsequently, the yielding region extended into the clear space of the specimen. As the yielding developed further, non-linear load deformation behaviour of the specimens was observed. The slopes of the load deformation curves then decreased as the applied load increased. The load deformation curves terminated when either gross section fracture or weld failure was observed.

3.4. Strain distributions
The strain readings obtained from the tests demonstrate the combined effects of shear lag and loading eccentricity. For simplicity, only the strain distributions based on the strain gauge (SG) readings at the critical section (SG 1 ~ SG 6) and the middle section (SG 7 ~ SG 12) of specimens A1-250US and A1-250UL are discussed and the strain distributions are shown in Fig. 11.

Figure 11 shows that the strain readings in SG4 and SG10 are generally smaller than the strains recorded on the connected legs. Since the locations of SG4 and SG10 are close to the centroid of the section, it is believed that the smaller tensile strains recorded are mainly due to the effects of shear lag. In addition, the figure also illustrates that at the beginning of loading, the strains in the section at SG6 and SG12 were in compression due to the combined effects of shear lag and loading eccentricity and the strains in other locations were all in tension. As the applied load increased, the strains at SG6 and SG12 further developed in compression and subsequently changed to tension in the latter stage of loading. It can also be seen from the figures that the strain distributions across the connected elements of the specimens were generally quite uniform (except SG1 of specimen A1-250UL due to a malfunction of the gauge in the early loading stage) indicating a minimal influence of the effects of shear lag. It is believed that this behaviour was due to the presence of the transverse weld along the edge of the connected leg which was able to almost fully mobilize the connected leg (Petretta [10]). In fact, both the AISC-LRFD [3], and the CSA/CAN-S16.1 [4] specifications stipulate that the full strength of the connected element can be used if it is connected by a transverse weld.

4. Effects of test parameters

4.1. Effect of connection length on angles
As shown in Table 2, the A1 group of specimens illustrate the effects of the connection length on the ultimate strength of the angle specimens. However, it is unfortunate that two of the specimens (A1-300UL and A1-250US) failed in the welds. Nevertheless, a good indication of the connection length effect could still be observed from the test results. When comparing specimen A1-200UL and A1-250UL, it can be seen that an increase in the connection length of 25% could only increase the ultimate strength of the specimens by about 2.9%. This is due to the fact that for the long leg connected angle specimens the major portion of the strength of the angle was contributed by the connected long leg (through the transverse weld) and only a moderate length of weld would be required to mobilize the unconnected short leg. Therefore, additional increase in the length of weld has only minor influence on the strength of the specimens. On the other hand, for the specimens connected by the short leg (A1-200US and A1-300US), an appreciable increase in the ultimate strength (13.7%) was observed when the connection length was increased by 50%. For these short leg connected specimens, a substantial length of weld would be required to mobilize the unconnected long leg. Therefore, the strength of these specimens was affected considerably by increase in the weld length. Figures 10a and b illustrate that in general, the final elongation of the specimens with unbalanced weld arrangement increases with increasing length of the connection (ignoring those specimens which failed in the weld).

4.2. Effect of long or short leg connections

Table 2 shows the effects on ultimate loads of connection leg lengths and balanced or unbalanced weld arrangement. For example, specimen A1-200UL achieved an ultimate load of 760 kN whereas the corresponding short leg connected specimen (A1-
200US) only obtained 665 kN, a 12.5% decrease in the load carrying capacity. Although some of the specimens failed in the weld, it can be seen from Table 2 that the long leg connected specimens were still able to sustain a higher ultimate load than the corresponding short leg connected specimens with an unbalanced weld arrangement. These observations can be explained firstly by the increase in area of the connected leg in the long leg connected specimens when compared to that of the short leg connected specimens as a result of the transverse weld. Secondly, for the short leg connected specimens the eccentricity about the plane of the gusset plate would be larger than for the long leg connected specimens and hence introducing a larger eccentric moment. Finally, it is self evident that if the unconnected leg is large, it would be more difficult for the connection to fully mobilize the unconnected leg area. The above reasons explain why lower ultimate loads occurred in short leg connected specimens. The situation for the specimens with balanced welds however, is different. Table 2 shows that specimens A1-200BL and A1-200BS reached similar ultimate loads of 786 kN and 782 kN, respectively. For specimens A2-200BL and A2-200BS-d, the ultimate loads were 990 kN and 936 kN, respectively. Hence, it can be seen that the effects of the connected leg on the ultimate load of the specimens when a balanced weld arrangement applied was not significant. This issue is further examined in the following section.

4.3. Effect of balanced or unbalanced weld arrangement on angles

As stated above, the weld arrangement had a minor effect on the ultimate loads of the long leg connected specimens as shown in Table 2 and Figs. 10a and 10c. As can be seen from the figures, the load deflection behaviour for these specimens was quite similar. Although specimen A2-200UL eventually failed as a result of weld fractures, its load deflection behaviour was still very similar to that of A2-200BL. On the other
hand, the weld arrangement had a substantial effect on the ultimate load and the load
deflection behaviour of short leg connected specimens. Figures 10b and 10c show that
the load deflection behaviour for the short leg connected specimens with either
balanced or unbalanced weld was similar in the initial loading stage. However,
deviation of the load deflection curves was observed as the loading continued to
increase and it can be seen from the figures that the short leg connected specimens with
unbalanced weld failed at a substantially lower ultimate load than the specimens with
balanced weld. Table 2 shows that there was about 17.6% increase in the ultimate
loads when a balanced weld arrangement was used to connect the short leg specimens
for both the A1 and A2 type specimens instead of an unbalanced weld arrangement.

The efficiency (defined as $P_{\text{test}}/A_gF_u$) of the member also increased substantially from
0.83 (A1-200US) to 0.97 (A1-200BS) and 0.82 (A2-200US) to 0.96 (A2-200BS-d) for
the A1 and A2 type specimens respectively when using a balanced weld arrangement.

In addition, it can be observed from Fig. 10 that the ductility of the angle specimens
increased when the balanced weld arrangement was used for both the long leg and short
leg connections.

The above discussion indicates that the member efficiency increased when the
angle tension members were connected by the short leg using a balanced weld
arrangement. This increase in efficiency may be explained by the fact that the balanced
weld arrangement eliminates the in-plane eccentricity that is caused by the unbalanced
weld arrangement. Furthermore, a longer weld length used along the heel of the angle
(a balanced weld condition) better mobilizes the participation of the unconnected leg in
carrying the load, hence reducing the effect of shear lag. However, for the long leg
connected specimens, the influence of the balanced weld arrangement is not that
obvious since most of the angle area has been connected through the long leg (via the
transverse weld) and the out-of-plane eccentricity is generally much smaller than is the case with the short leg connected angles.

5. Finite element analysis of test specimens

5.1. Finite element model

Three-dimensional nonlinear finite element (FE) models for the test specimens were developed using the finite-element program, ABAQUS [19]. Both material and geometric non-linearities were represented in the modelling. The finite element models were established using the measured dimensions of the specimens. The S4R 4-node, three dimensions, general-purpose shell elements were used to model the angles and the gusset plates. Figure 12 shows a typical finite element model for the entire specimen. The end of the gusset plate was fixed at 90 mm from the start of the angle to the centre of the bolt group connecting to the end fixtures. Because the cross-heads of the machine and the end fixtures are relatively rigid compared to the gusset plate, the plate was restrained at its leading edge except for displacement in the longitudinal direction. Uniform displacements were applied to the leading edge of the gusset plate to simulate the applied load (a stroke control). At the other gusset plate, linear spring elements (SPRING1) were applied to the end of the plate to simulate the sum of the axial rigidities of the gusset plates, the end fixtures and the supporting columns of the testing machine. All other displacements of the gusset plate were restrained. The stiffness of these spring elements was determined to be 55 kN/mm calibrated by the experimental load-elongation curves in the early load stage (linear load deflection behaviour) of the test specimens. To simplify the modelling of the weld, the multi-point constraints (MPC) in ABAQUS were used. The MPC provides a rigid link between
two nodes to constrain the displacement and rotation at the first node to be identical to
the displacement and rotation at the second node.

During loading, the specimens deformed so that the centroidal axis of the section
moved towards the line of action of the force. Hence, the gusset plate had to bend
because of the deformation of the specimens. To accurately simulate this behaviour, the
interaction between the angles and the gusset plates was modelled as contact surfaces.
A small gap of 0.1 mm was applied between the contact surfaces in order to avoid
overlapping of the surfaces. The action at the surfaces was assumed to be a small
degree of sliding without friction, and the symmetric master-slave contact pairs method
in ABAQUS was also used to prevent the surfaces of the angles and the gusset plates
from penetrating into each other. If the gap was closed, the SURFACE BEHAVIOR,
NO SEPARATION parameter of ABAQUS would be applied to model the
corresponding surfaces contact. These parameters prevent the contacting surfaces from
separating subsequently. These assumptions are reasonable considering the fact that
most welds should be in good condition during the whole loading process and because
the contact surfaces of the gusset plates and the angles deformed together in unison
without obvious slides and gaps in the physical tests.

Material and geometric non-linearities were included in the models. The material
properties were based on the tension coupon test results. The isotropic elastic-plastic
multi-linear properties combined with the von Mises yield criterion was used to
represent the material nonlinear effects. In order to simulate the material behaviour,
the values of the true stress and true strain were input to ABAQUS. These true stress
and strain values were obtained by Eqs. (2) and (3) based on the nominal stresses and
nominal strains from the tension coupon test results.

$$\sigma_{\text{true}} = \sigma_{\text{nom}} \left( 1 + \varepsilon_{\text{nom}} \right)$$  \hspace{1cm} (2)
The nominal stress ($\sigma_{\text{nom}}$) and nominal strain ($\epsilon_{\text{nom}}$) in the static average curve of the coupon tests were then converted into the multi-linear curve of true stress ($\sigma_{\text{true}}$) and true plastic strain ($\epsilon_{\text{true}}^p$). Furthermore, the material was assumed to be perfectly plastic beyond a true plastic strain of 0.22. Table 4 summarises of the material models.

In order to estimate the onset of section fracture failure in the numerical study, the strain based failure criterion adopted by Cheng and Kulak [20] was used. Failure was assumed to have occurred when the maximum equivalent plastic strain (PEEQ) reached a critical value. The models predicted the critical PEEQ at the heel of the specimens in the clear space to be around 0.21 for the specimens with the balanced weld arrangement. For other cases, a critical PEEQ of approximately 0.15 was predicted at the toe of the critical section. These values of critical PEEQ were calibrated using the test results of those specimens which experienced a gross section fracture failure mode. Subsequently, the same critical PEEQs were then applied to evaluate the ultimate loads of the specimens that failed in the welds.

5.2. Analytical results

5.2.1. General

The analytical results of the specimens and the comparisons of the results with those of the tests are shown in Table 2. The ratio of the test ultimate loads ($P_{\text{test}}$) to the predicted ultimate loads ($P_{\text{FEM}}$) for the specimens which failed by gross section fracture varied from 0.97 to 1.04 with a mean of 1.00 and a standard deviation of 0.025. The predictions for the specimens that failed in the welds were also obtained based on the critical PEEQs criterion as stated above, assuming that these specimens failed in the
gross section. The predicted loads for these specimens were close to the test ultimate loads which indicated that these specimens had almost been able to achieve the gross section fracture failure mode. Table 2 also shows the final elongations of the test specimens obtained from the finite element analysis at the corresponding critical PEEQs. It can be seen that the final elongations of the specimens from the tests are in reasonable agreement with those obtained from the finite element analysis except for Specimen A1-200BS. A typical deformed shape of the specimens is shown in Fig. 13. The figure shows that necking occurs close to the critical section of the angles (near the edge of the gusset plate) and that the angle bent due to the loading eccentricity.

Typical applied load versus elongation curves from both the test and the finite element analysis results of the specimens (A1-200BS, A1-200US, A1-200BL and A1-200BL) are shown in Fig. 14. It can be seen that the load-deflection responses of the FE models compared well with the static load curve of the tests. All the specimens that failed in gross section fracture exhibited considerable ductility. However, in general, the specimens with a balanced weld arrangement were able to deform to a greater extent than those corresponding ones with an unbalanced welded arrangement.

5.2.2. Strain distribution

To further examine the results from the FE analyses, the strain distributions at the critical section and near the middle section were compared with those of the test data. Figures 15 and 16 illustrate the strain distributions in the early loading stage of specimen A1-200BS. All the strains were measured in the longitudinal direction (2-direction). In general, reasonable agreements were obtained between the test and the analytical strain values.
Figure 17 shows the PEEQ contours of specimen A1-250UL at the ultimate load level. As can be seen from the figure, the maximum PEEQ appeared at the toe region of the angle in the vicinity of the critical section where fracture of the angle was assumed to occur. The predicted location of the fracture compared well with that of the test results as shown in Fig. 9a. The fractures had to be initiated at the critical section because of the combined effects of the shear lag, the bending of the angle section due to the alignment of the line of applied load and the centroidal axis of the section and the stress concentration that existed at this location. For the specimens connected by a balanced weld arrangement, the maximum PEEQ usually occurred at the heel region with fractures initiated at a section slightly away from the critical section.

5.2.3. Stress distribution

Figure 18 and 19 compare the stress distributions at ultimate load levels for the A1 specimens that have an average length of fillet welds of 200 mm. It can be seen from Fig. 18 that the stress at the outer edge of the unconnected leg (SG 6) for specimen A1-200BS was significantly higher than for A1-200US. However, for the long leg connected angles (A1-200BL and A1-200UL), the increase in stress with a balanced weld arrangement at SG6 was not as significant. This observation substantiates the fact that a balanced weld arrangement can eliminate the in-plane eccentricity and reduce the effects of shear lag on the tensile strength of short leg connected angles. Figure 19 shows that approximately uniform stress distributions occurred across the middle section of the specimens. This uniform stress distribution indicates that both the shear lag effects and bending effects had diminished. The weld connection configurations had nearly no influence on the stress distribution in the middle section of the specimens. However, it can be seen that the stress levels for the specimens with the
balanced welds were generally higher than that of the specimens with unbalanced welds.

6. Comparison of test results with current design methods

6.1. General

Generally, the design of a tension member is based on the failure mode of the gross cross section in yielding and net section fracture at any reduced cross section. The ratio of the tensile capacity of a member to its ultimate tensile strength is termed net section efficiency, $U$, as shown in Eq. (4).

$$ U = \frac{P_u}{F_u A_n} \quad (4) $$

where $P_u$ is the tensile capacity of the member; $A_n$ is the net cross-sectional area; and $F_u$ is the ultimate tensile strength of the material. For a tension member with a welded end connection, $A_n$ is replaced by the gross section area, $A_g$.

6.2. Current design standards

6.2.1. AISC LRFD (2005)

The AISC-LRFD [3] specification stipulates that two failure modes must be considered when designing a tension member; namely, gross section yielding and net section fracture. The design equation for gross-section yielding is:

$$ \phi \sigma S_n = 0.9 \sigma_y A_g \quad (5) $$

and for fracture of the net section

$$ \phi \sigma S_n = 0.75 \sigma_u A_n \quad (6) $$
where $P_n$ is the nominal axial strength and $\phi_t$ is the resistance factor for tension. The effective area, $A_e$, for a tension member connected by welds is

$$A_e = UA_g$$

(7)

where $U$ is the shear lag factor. The specification adopted the shear lag coefficient empirically developed by Munse and Chesson [1, 2] to account for the shear lag effect for angles, as has been discussed above:

$$U = 1 - \frac{x}{L}$$

(8)

The specification indicates that $L$ is the length of the weld parallel to the line of force for welded connections. For a balanced weld condition, the length of the longitudinal welds on the heel and the toe of an angle would be different. Although not explicitly illustrated in the specification, for a balanced weld arrangement the longest segment of weld (i.e. the heel weld) should be used (Segui [21]).

**6.2.2. CAN/CSA-S16-01(2001)**

The Canadian Standard CAN/CSA-S16-01 [4] also considers the failure modes of yielding of the gross cross-section and the fracture of the net section using the two equations below.

$$T_r = \phi A_g F_y$$

(9)

$$T_r = 0.85\phi A_{ne} F_u$$

(10)

where $T_r$ is the factored tensile resistance; $\phi$ is the resistance factor; $A_g$ is the gross cross sectional area and $A_{ne}$ is the effective net area reduced for shear lag. A rather comprehensive approach is adopted by CAN/CSA-S16-01 [4] to evaluate the effective net sectional area in order to account for the shear lag effect. The approach considers
the effects of the weld configuration and also the contribution to the tensile strength of the angle based on the strength of the individual leg. The total effective net sectional area, \( A_{ne} \), is defined as:

\[
A_{ne} = A_{n1} + A_{n2} + A_{n3} \tag{11}
\]

where \( A_{n1}, A_{n2}, \) and \( A_{n3} \) are the effective net areas of the connected parts evaluated in the following manners:

a) For a part connected by a transverse weld:

\[
A_{n1} = wt \tag{12}
\]

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

(i) when \( L \geq 2w \)  

\[
A_{n2} = 1.00wt \tag{13}
\]

(ii) when \( 2w > L \geq w \)  

\[
A_{n2} = 0.50wt + 0.25Lt \tag{13}
\]

(iii) when \( w > L \)  

\[
A_{n2} = 0.75Lt \tag{13}
\]

where \( L = \) average length of welds on the two edges; \( w = \) plate width (distance between welds).

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

(i) when \( L \geq w \)

\[
A_{n3} = \left( 1 - \frac{x}{L} \right) wt \tag{14}
\]

(ii) when \( w > L \)

\[
A_{n3} = 0.50Lt \tag{15}
\]

where \( x = \) eccentricity of the weld with respect to the centroid of the connected element and \( L = \) length of weld in the direction of the loading. The unconnected leg of an angle is assumed to be connected by the (single) line of weld along the heel. It should be noted that when angles are considered, the effective section of the connected
part is based on Eq. (12) and (13), while that of the unconnected leg is obtained using Eq. (14) and (15). The term $\bar{x}$ is usually taken as a half of the length of the unconnected leg.

6.2.3. BS 5950-1:2000(2001)

As for BS 5950-1:2000 [13], the effective area is equal to the gross sectional area ($A_g$) subtracting a portion of the unconnected leg area for welded connections. Equation (16) presents the design of a single angle member with welded end connections, which makes use of the effective area multiplied by the design yield strength ($p_y$) of the material.

$$P_t = p_y (A_g - 0.3a_2)$$  \hspace{1cm} (16)

where $P_t$ = tension capacity and $a_2$ = difference between $A_g$ and $a_1$ (gross area of the connected leg for members with welded connections). If the expression for $A_g$ (equal to $a_1 + a_2$) is substituted into Eq. (16), the tension capacity of the member would be equal to the product of the material design strength and the sum of the area of the connected leg ($a_1$) and a portion of the area of the unconnected leg ($a_2$).

$$P_t = p_y (a_1 + 0.7a_2)$$  \hspace{1cm} (17)

In fact, the above equation is similar to that of the work conducted by Regan and Salter [7] which included a testing program consisting of 17 single angle specimens with welded connections. Based on the test results, they proposed the following equation to evaluate the effective sectional area ($a_e$) of single angles accounting for the effect of shear lag:

$$a_e = (a_1 + 0.8a_2)$$  \hspace{1cm} (18)
Hence, it can be seen that BS 5950-1:2000 [13] adopted a similar approach to that of Regan and Salter [7] for the design shear lag of tension members but introduced more conservatism into the codified equation.


The European code BS EN 1993-1-8:2005 [14] introduced a simpler way to deal with the shear lag effects. For an angle connected by one leg, the shear lag and the loading eccentricity effects are accounted for by using an effective cross-sectional area. For cases of equal leg angles or unequal leg angles connected through the long leg, the effective cross-sectional area is defined as the gross area of the angle section. However, for an unequal leg angle connected by its short leg, the effective cross-sectional area is taken as the area of an equal leg angle assuming that the leg length is equal to that of the short leg of the unequal leg angle.

6.2.5. **AS 4100 (1998)**

The Australian code AS 4100-1998 [15] adopted a relatively straightforward approach to account for shear lag of tension members. The design axial tension force \(N^*\) of a member is defined as:

\[ N^* = \phi N_t \] (19)

where \(\phi\) is the capacity factor and \(N_t\) is the nominal section capacity of a tension member which is taken as the lesser of

\[ N_t = A_g f_y ; \text{ and} \] (20)

\[ N_t = 0.85 k_t A_a f_u \] (21)
where $f_y$ is the yield stress used in design, $f_u$ is the tensile strength used in design and $k_t$ is a correction factor for distribution of forces. The correction factor, $k_n$, is equal to 0.75 for unequal angles connected by the short leg and 0.85 for other cases.

6.2.6. Predictions by the current design standards

In order to compare the test results with the predicted capacities of the specimens based on the various design specifications, the effective area ($A_e$) of the member sections was used. This is because the BS 5950-1:2000 [13] standard calibrated its design equation for tension members based on the design yield strength ($p_y$) of the material whereas the North American design specifications adopted the design approach of checking separately on both the gross section yielding and net section fracture. Nevertheless, the effective area ($A_e$) of the member sections is also an appropriate means to present the comparison between the test capacities and the predicted capacities by the various design specifications. Therefore, the test-to-predicted ratios for all the specimens are presented as $A_{test}/A_{code}$ as shown Table 5 where $A_{test} = P_{test}/F_{u}$ and $A_{code} =$ effective area used by the specifications (e.g. $A_{eAISC} =$ effective area based on AISC-LRFD [3]). In the table, the test results of the authors’ study and the test results of Regan and Salter [7] are compared with the predictions made by the various design specifications as discussed above. In fact, the test results from Regan and Salter [7] are comparable to those of the authors for similar specimens (A1-250UL, A1-300US, F1 and F2) as illustrated in Table 5. All the resistance factors for the design equations were taken as 1.0.

To facilitate the discussion, all the test-to-predicted ratios are also presented in Fig. 20. The figure also shows the means and the standard deviations of the ratios.
test-to-predicted ratios calculated based on AISC-LRFD [3] vary from 0.97 to 1.14 for the test data with a mean of 1.04 and a standard deviation of 0.044 as shown in Table 4. The test-to-predicted ratios using the CAN/CSA-S16-01 [4] specification range from 0.96 to 1.13 with a mean of 1.03 and a standard deviation of 0.045. Although the design approaches adopted by the AISC-LRFD [3] and the CAN/CSA-S16-01 [4] specifications for evaluating the tensile capacities of angle members are different, it can be seen that both design specifications produced similar predictions of the tensile capacity of single angle members. For the AISC-LRFD [3] design approach, the shear lag coefficient \( U = 1 - \frac{x}{L} \) is applied to the entire cross-section of the member, whereas the CAN/CSA-S16-01 [4] only applies the coefficient to the unconnected leg of the angle. However, it should be noted that the distance \( x \) used by CAN/CSA-S16-01 [4] is assumed to be the eccentricity of the weld with respect to the centroid of the unconnected element instead of the entire angle section as in AISC-LRFD [3]. Therefore, a smaller \( U \) is produced by CAN/CSA-S16-01 [4] than by AISC-LRFD [3].

For the predictions evaluated based on BS 5950-1:2000 [13], the test-to-predicted ratios are all above 1.0 which indicates a more conservative estimate of specimen capacity by this design specification. The mean and the standard deviation of the ratios are 1.10 and 0.054, respectively. The conservative estimates produced by BS 5950-1:2000 [13] are due to the fact that only 70% of the unconnected leg area (0.7a_2 as shown in Eq. (17)) contributed to the total tensile capacity of the angles irrespective of the amount of out-of-plane loading eccentricity \( (x) \). Similar to BS 5950-1:2000 [13], AS 4100-1998 [15] generally produces conservative predictions of the tensile capacity of the angle specimens with a mean test-to-predicted ratio of 1.16 and a standard deviation of 0.069. This conservatism is due to the simplified approach adopted by the
code of using two factors in determining the effective area of the angle sections (0.75 for unequal angles connected by the short leg and 0.85 for other cases) in order to account for the effects of shear lag. The test-to-predicted ratios evaluated using BS EN 1993-1-8:2005 [14] exhibit a large variation with a mean ratio of 1.11 and a standard deviation of 0.197. In fact, the ratio ranges from 0.94 to 1.47. This large variation is due to the oversimplified approach adopted in the specification to account for the shear lag effects as discussed in the previous section.

Overall, the current design specifications are able to predict conservatively the tensile capacity of the specimens allowing for the effects of shear lag. Both AISC-LRFD [3] and CAN/CSA-S16-01 [4] produced good estimates of the tensile capacity of the angle specimens. However, the predictions by BS 5950-1:2000 [13] and AS 4100-1998 [15] were slightly more conservative and the predictions by BS EN 1993-1-8:2005 [14] exhibited a large variation. The AISC-LRFD [3] approach is relatively straightforward, whereas a more elaborate approach is adopted by CAN/CSA-S16-01 [4]. Nevertheless, it should be noted that none of these design approaches make recognition of the beneficial effects, which can be quite substantial, as illustrated by the test results of using a balanced welded arrangement for single angle tension members with a short leg connection.

7. Summary and conclusions

A total of thirteen full-scale tension tests were conducted to investigate the shear lag effects on the strength and behaviour of single angle tension members with welded end connections. The test parameters included long and short leg connections, balanced and unbalanced weld arrangements and the length of longitudinal fillet welds. Out of the thirteen specimens, nine specimens failed by fracture of the gross section while the
other four specimens failed in the welds. The efficiency of the specimens varied from 0.82 to 1.02. The test results show that increasing the length of the connection does not improve the connection capacity for long leg connected specimens, but improvement in the tensile capacity was observed for the short leg connected cases. As expected, in general the efficiency of the specimens with long leg connections was higher than that of the specimens with short leg connections. It can also be observed from the test results that both the ultimate loads for the short leg connected angles and the ductility of all angle specimens increased when a balanced weld arrangement was used in the connections.

Finite element analyses of the specimens were conducted using ABAQUS. The finite element models were calibrated (based on the critical equivalent plastic strain criterion, PEEQ) using the test results of those specimens which experienced a gross section fracture failure mode. Subsequently, the same critical PEEQ was used to estimate the potential ultimate loads of the specimens that failed in the weld. In general, the load deflection behaviour of the specimens was predicted well by the finite element analyses. Good comparison between the test ultimate loads and those predicted by finite element analysis was observed. The test-to-predicted ratio varied between 0.96 and 1.04.

In general, the current design specifications are able to conservatively predict the tensile capacity of the specimens allowing for the effects of shear lag. However, the predictions by BS 5950-1:2000 [13] and AS 4100-1998 [15] were slightly more conservative and the predictions by BS EN 1993-1-8:2005 [14] exhibited a large variation. Although conservative predictions of the tensile capacity of single angle tension members can be obtained using most of the current design specifications, none of these design approaches recognise the beneficial effects, which can be quite
substantial as illustrated by the test results (about 17.6% increase in ultimate loads), of
using a balanced welded arrangement for single angle tension members with a short leg
connection.

8. Acknowledgments

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University of Macau is also acknowledged.

9. References


design specification for structural steel buildings, Chicago, IL, USA, 2005.


[5] Davis RP, Boomsliter GP. Tensile tests of welded and riveted structural


<table>
<thead>
<tr>
<th>Designation</th>
<th>Section size</th>
<th>A (mm)</th>
<th>B (mm)</th>
<th>t (mm)</th>
<th>Note</th>
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Table 2 Specimen description and test results

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<tr>
<th>Specimen</th>
<th>Toe weld length ($L_1$, mm)</th>
<th>Heel weld length ($L_2$, mm)</th>
<th>Connected leg (mm)</th>
<th>Total length (mm)</th>
<th>$F_uA_g$ (kN)</th>
<th>Ultimate Load, $P_{test}$ (kN)</th>
<th>Final Elongation (Test) (mm)</th>
<th>Efficiency, $P_{test}/F_uA_g$ (Failure mode)</th>
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<th>Final Elongation (FEM) (mm)</th>
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Note: A: angles; B: balanced; U: unbalanced; d: duplicate; L: long leg connection; S: short leg connection; F: fracture of gross section; W: weld failure

#: final elongation from FE analysis at the critical PEEQ

Mean = 0.94
Standard deviation = 0.068
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<tr>
<th>Specimen type</th>
<th>Elastic modulus, $E$ (MPa)</th>
<th>Static yield strength, $F_y$ (MPa)</th>
<th>Static ultimate strength, $F_u$ (MPa)</th>
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Table 3 Material properties
Table 4 Summary of material model parameters

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<th>A2</th>
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### Table 5  Summary of the predicted capacity and the test capacity of the specimens

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<th>Weld details (mm)</th>
<th>Yield strength</th>
<th>Tensile strength</th>
<th>Failure mode</th>
<th>Ultimate load (kN)</th>
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<th>$A_{test}$</th>
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<tr>
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<td>(MPa)</td>
<td>(kN)</td>
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- Mean = 0.95 1.04 1.03 1.10 1.16 1.11
- Standard deviation = 0.046 0.044 0.045 0.053 0.069 0.197

FC = Failure at Critical section  WF = Weld Failure  FM = Failure along Member  Underline = Connected leg
Fig. 1  Shear lag of an angle
Fig. 2 Definition of $\bar{x}$ and $L$ from Munse and Chesson [2]
Fig. 3 Definition of symbols for the test specimens
Balanced weld arrangements

Unbalanced weld arrangements

Fig. 4 Weld arrangements
Fig. 5 Typical details of the specimens

A1-300US Details
Fig. 6 Test setup
Fig. 7 Typical strain gauge locations
Fig. 8  Typical coupon test results
Figure 9 Typical failed specimens

(a) Gross section fracture of specimen A1-250UL

(b) Necking and gross section fracture of specimen A1-200BL

(c) Deformation of specimen A1-300US near failure

Alignment of line of applied load to the angle centroidal axis

Bending of gusset plate
(a) Load vs. elongation curves of A1 specimens with long leg connection

(b) Load vs. elongation curves of A1 specimens with short leg connection
(c) Load vs. elongation curves of A2 specimens

Fig. 10   Load vs. elongation curves of all specimens
Fig. 11 Strain distribution for specimens A1-250US and A1-250US

(a) Strain distribution at the critical section of specimen A1-250US

(b) Strain distribution at the middle section of specimen A1-250US

(c) Strain distribution at the critical section of specimen A1-250UL

(d) Strain distribution at the middle section of specimen A1-250UL

Fig. 11 Strain distribution for specimens A1-250US and A1-250US
Fig. 12 Typical finite element model of a single angle
Fig. 13 Typical deformation mode
Figure 14

(a) Specimen A1-200BS

(b) Specimen A1-200US
Fig. 14 Comparison of load vs. elongation curves of specimens A1-200BS, A1-200US, A1-200BL and A1-200UL
Fig. 15 Comparison of strains at the critical section of specimen A1-200BS-d
Strain gauges at the middle section

\[ \text{Strain (micro strain)} \]

Fig. 16 Comparison of strains at the middle section of specimen A1-200BS-d
Fig. 17  PEEQ contour of specimen A1-250UL
Fig. 18  Comparison of stresses at the critical section at ultimate states of A1 specimens
Fig. 19 Comparison of stresses at the middle section at ultimate states of A1 specimens
Fig. 20 Comparison of the test-to-predicted ratios

Specimens

- Regan and Salter [8]
- Current study (angle)

- AISC-LRFD 2005: mean: 1.04, standard deviation: 0.044
- CAN S16-1 2001: mean: 1.03, standard deviation: 0.045
- BS5950 1:2000: mean: 1.10, standard deviation: 0.053
- Eurocode 2 2005: mean: 1.11, standard deviation: 0.197
- AS 4100-1998: mean: 1.16, standard deviation: 0.069
- U = $\frac{A_{e\text{est}}}{A_{e\text{predicted}}}$

Mean and standard deviation for each specimen type are provided.