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Investigation of Angle Welded Connection under Tension by Full-scale Tests

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ABSTRACT

American Institute of Steel Construction Specification (AISC 2010) recommends that design of welded end connection of single angle, double angle, and similar members need not consider the effect of eccentricity. However, the eccentricity in the angle may affect strength of the connections, especially welded connection at the end of single angles which out-of-plane bending naturally occurs. This research was aimed to study the effects of eccentricity on the strength of the welded connection in the angle tension member by test. Specimens included single and double angles with balanced and unbalanced weld arrangements, with various thicknesses of gusset plates, lengths of connection, and sizes of angle. Every specimen was designed so that weld rupture would be the governing limit state. The results showed that unbalanced weld arrangements and out-of-plane bending could altogether reduce weld rupture strength from nominal value up to 20 %.

1) INTRODUCTION

It is well known to every engineer who uses steel that the centroid of the angle does not lie in the middle of the section, both in-plane and out-of-plane. General design practice for an angle welded connection is that one would try to balance the connection by placing the centroid of the welded connection at the same location as that of the angle for in-plane consideration. As a result, for the balanced welded end connections for angles, the weld on the toe of the angle is shorter than that on the heel as shown in Figure 1.

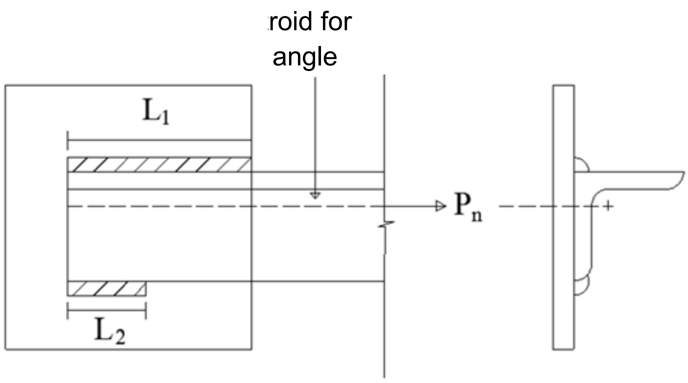


Figure 1 Balanced welded end connection for angle

American Institute of Steel Construction Specification (AISC 2010) states that design of a welded end connection of single angle, double angle, and similar members under static loading conditions need not consider the effect of eccentricity. That is, design of a weld group does not necessarily have to be balanced, in other words, the centroid of the welded end connection may be in a different location from that of the angle itself. This would create eccentricity between the angle and the weld group, which may affect strength of the connections, especially welded connections at the end of single angles which out-of-plane bending naturally occurs.

2) PAST RESEARCH

There have been plenty of research carried out in the past regarding the strength of the welded end connections of the angles, with the majority focusing on the tensile strength of the member itself under the load transfer phenomenon called "shear lag." As a result, review of past research is categorized into two sections: weld-related and angle-related.

2.1 Weld-related Research

As early as 1930, Weiskopf and Male studied strain in longitudinal welds by measuring the strain on the steel plate on various locations. It was found that strain at the two ends of the welds were significantly higher than that in the middle region. Hollister and Gelman (1932) carried out the test to investigate elastic stresses on welds. Double plates were used to eliminate the effects of eccentricity with various welding configurations. It was concluded that use of only longitudinal welds would result in stress distribution being less uniform than use of transverse ones. An increase of weld length or use of both longitudinal and transverse welds could help improve the stress distribution.

Gibson and Wake (1942) investigated strength of welded end connections of single and double angles by testing 28 specimens, 15 configurations in total, consisting of both balanced and unbalanced welds. One third of specimens failed at the angles due to unexpected uneven stress distribution in angles. It was concluded that eccentricity between the centroid of the angle and that of the welds affected strength of welds and that the difference of strength between balanced and unbalanced welds was insignificant. AISC commentary (2010) cited this to support their recommendation on design of welded end connection of angles. However, it should be noted that, out of specimens with unbalanced welds, only two configurations were welds with only longitudinal components, while the rest had both longitudinal and transverse welds.

It was almost 30 years afterward that Butler and Kulak (1971) carried out another notable research on fillet weld strength. Behavior and strength of welds as a function of directions of forces acting on them were studied. Tests were carried out on 23 specimens of 6.35 mm welds with various angles (0°, 30°, 60°, and 90°). Results showed that weld strength was a function of the direction of the load. Based on the load-deformation curves obtained from the research, the method for calculating strength of welds was developed by using the concept of instantaneous center of rotation. The method has been adopted by AISC, and design tables available in the Manual (2010) have been constructed on this foundation.

Kanvinde et al (2008) studied efficiency of two fillet weld strength prediction methods, i.e., traditional fracture mechanics and micromechanics-based fracture models or Stress Modified Critical Strain (SMCS), by testing 24 specimens with various parameters. It was found that SMCS could predict the strength of the welds closer to values from the tests than the traditional method. Kanvinde et al (2009) also studied the effects of root notch on strength and ductility of transverse welds by test and finite element analysis. Results showed that root notch did not have any significant contribution on neither strength nor ductility of the welds.

Picón and Cañas (2009) compared the strength of welds calculated based on two design approaches: ISO and Eurocode 3. Test results based on these methods demonstrated that Eurocode 3 approach could predict the strength of longitudinal welds better than ISO. On the other hand, ISO could predict the strength of transverse welds better.

2.2 Angle-related Research

Early in 1934, Davis and Boomsliter tested tensile strength of single and double angles, five configurations in total, connected to the support using either rivets or welds. It was found that tensile strength from tests was lower than that from the calculation based on the assumption that stress in the member was uniform. Drawing conclusions from test results, it was proposed that the effective area for calculating tensile strength of the angle be the leg attached to the support and half of the outstanding leg.

Chesson and Munse (1963) also carried out tests on tensile strength of angles, but the specimens were connected to the support only by rivets and bolts. For the first time, the phenomenon that reduced the tensile strength of the angle was called “Shear Lag.” Chesson and Munse also stated that the length of the connection affected tensile strength and proposed the following formula for calculating shear lag factor to be applied to the net area of the section:

$$\mathrm{U} = 1 - \bar{x}/L \tag{1}$$

Where U shear lag factor

$\bar{\mathrm{x}}$ connection eccentricity (mm)

L length of connection (mm)

The formula in Eq. (1) was adopted by AISC and has been in the specification ever since.

Regan and Salter (1984) tested angles connected to a gusset plate by welds to study the impact of plate thickness and angle span of tensile strength of angles. Angles used were both equal- and unequal-leg, while only unbalanced welds were employed, i.e., lengths of welds on the heel and the toe of the angle were equal. Results showed that out-of-plane bending occurred at the connection. Strain at the connection on the outstanding leg was initially compressive and would later become tensile after the load increased. It was also concluded that an increase of plate thickness helped improve stress distribution in the angle and reduce out-of-plane bending.

Easterling and Gonzalez (1993) later investigated calculation of shear lag proposed by Chesson and Munse by using full-scale tests and finite element analyses. Welds were used to connect three types of sections: plate, angle, and channel. Results from the research led to justification of the formula and conclusions that transverse welds at the end of angles did not affect shear lag. It was also proposed that the shear lag factor used not exceed 0.9, the limitation which does not appear in AISC. However, the arrangement of the connection for single and double angles should be such that U is greater than 0.6.

Recent research on strength and behavior of angles include full-scale tests on 23 specimens of welded angle connection to study effects of shear lag by Petretta (1999), tests on equal- and unequal-leg double angles with unbalanced welds to observe shear lag by Bauer and Benaddi (2002), tests on unequal-leg welded single angles to examine shear lag by Zhu et al (2009), and by Fang et al (2013).

3) TEST SETUP AND SPECIMENS

In this research, 14 specimens of single and double angles, welded with both balanced and unbalanced configurations to a gusset plate with multiple thicknesses, were constructed and tested (Mekpramual 2014). Every specimen was designed in accordance with AISC (2010) to fail by weld rupture. Balanced welds were calculated based on equilibrium so that the centroid of the member and that of the weld group coincided. Details of specimens are summarized in Table 1. Lengths L_1 and L_2 are shown in Figure 2, while test setup can ben seen in Figures 3 and 4. Weld size used in every specimen was 6 mm.

All material properties were obtained in accordance with ASTM d638-02a (2013) and ASME (2010) and are summarized in Table 2. Specimens were also equipped with strain gauges at the welds and LVDT at the support to monitor a displacement. A static load was applied via an actuator mounted on one side of the specimen until failure occurred. Weld rupture was the only failure observed as intended, as can be seen in Figure 5.

Table 1 Details of the specimens

Specimen	Angle (mm)	Gusset Plate (mm)	Weld Lengths[1] (mm)		Weld Configuration
			L_1	L_2	
S1-B-G10	65x65x8	10	120	50	Balanced
S1-U-G10	65x65x8	10	85	85	Unbalanced
D1-B-G10	2-65x65x8	10	120	50	Balanced
D1-U-G10	2-65x65x8	10	85	85	Unbalanced
S1-B-G16	65x65x8	16	120	50	Balanced
S1-U-G16	65x65x8	16	85	85	Unbalanced
S1-B-G22	65x65x8	22	120	50	Balanced
S1-U-G22*	65x65x8	22	85	85	Unbalanced
S2-B-G10*	120x120x8	10	120	50	Balanced
S2-U-G10*	120x120x8	10	85	85	Unbalanced
D2-B-G10*	2-120x120x8	10	120	50	Balanced
D2-U-G10*	2-120x120x8	10	85	85	Unbalanced
S2-B-G10-a*	120x120x8	10	240	90	Balanced
S2-U-G10-a*	120x120x8	10	165	165	Unbalanced

Note * denotes that the specimen was strengthened at the support to reduce lateral deformation.

a denotes that longer welds were used.

B denotes that welds were balanced whereas U denotes that welds were unbalanced

S denotes that the specimen consisted of a single angle whereas D denotes that the specimen had two angles

Shaded area represents use of larger angle size

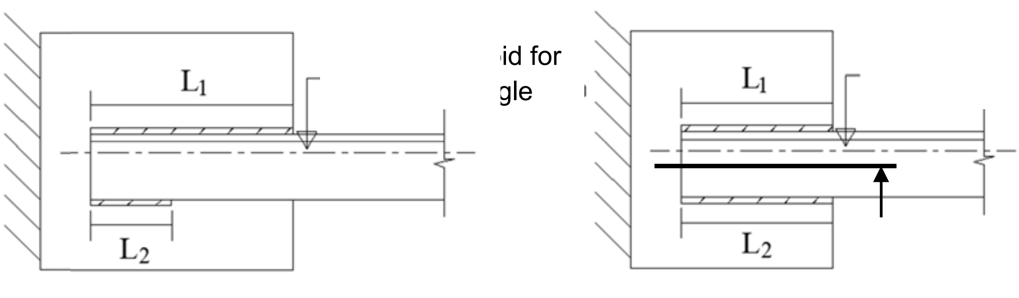


Figure 2 Lengths of welds in balanced and unbalanced connections

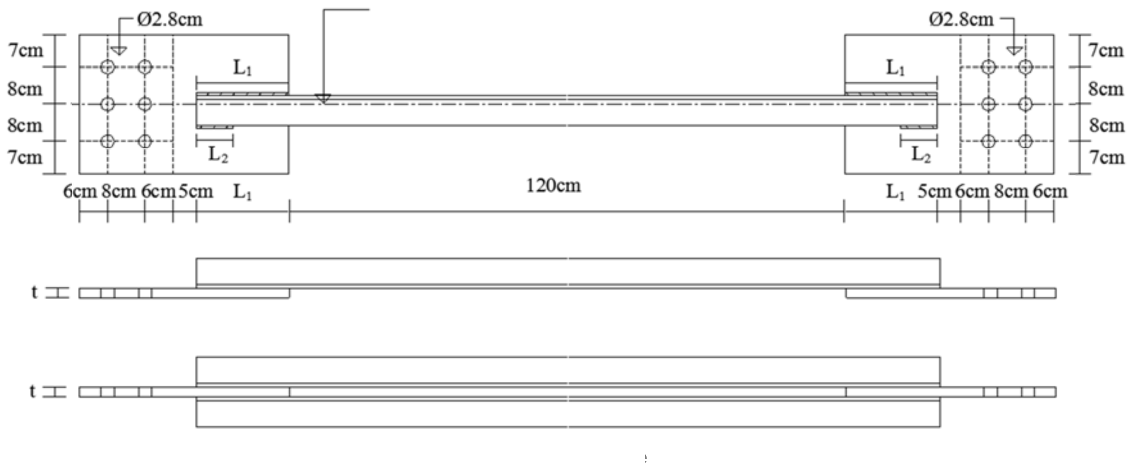


Figure 3 Typical details of specimen

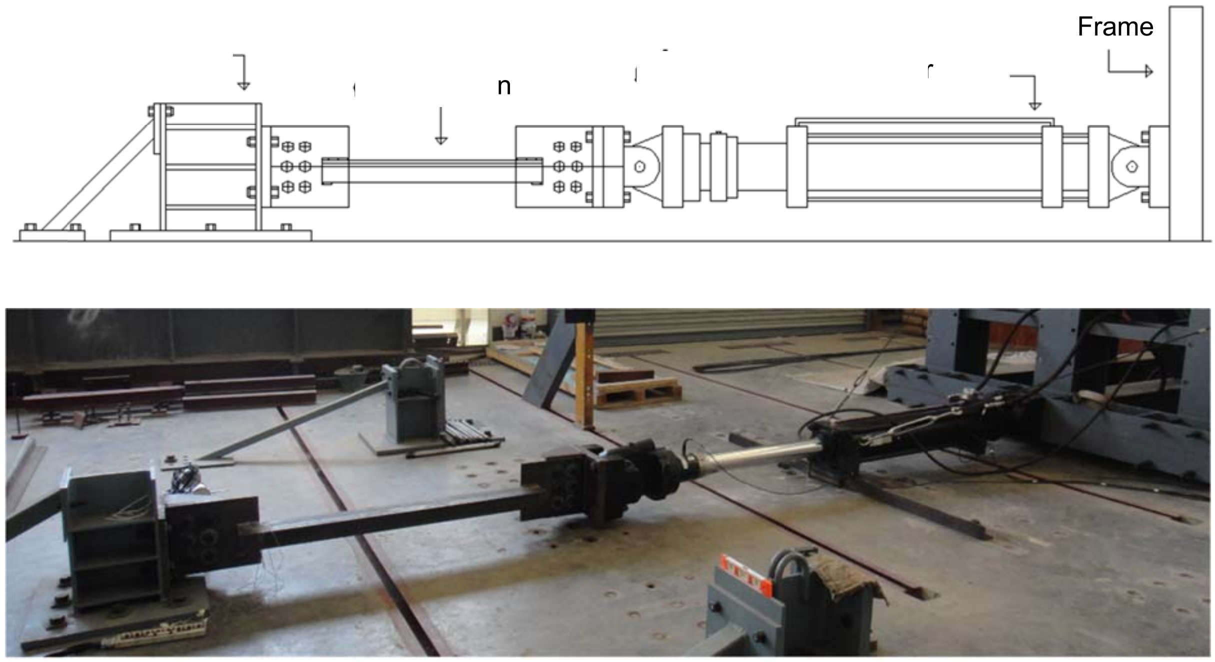


Figure 4 Test setup

Table 2 Properties of materials

Materials	Minimum Tensile Yielding (MPa)	Ultimate Tensile Strength (MPa)
120x120x8 mm Angle	326	453
65x65x8 mm Angle	295	428
Steel Plate	271	407
Welds (E60)	448	531

Figure 5 Weld rupture of specimen

4) TEST RESULTS

Results from the tests, which carried out until weld rupture occurred, were summarized in Table 3. In order to demonstrate the effects of eccentricity from both unbalanced condition and out-of-plane bending on weld strength, values of nominal weld shear ruptured calculated by using AISC Specification (without eccentricity) are incorporated into the table along with the ratios of test results and these calculated nominal strength. Locations of strain gauges on welds are illustrated in Figure 6 and selected plots of force vs. values read from strain gauges are shown in Figure 7.

Table 3 Test results and nominal strength of angle welded connections

Specimen	Calculated Nominal Weld Rupture (kN)	Test Results (kN)	Ratios of Test Results/Nominal Strength	Displacement of Gusset Plate (mm)
S1-B-G10	230	232	1.01	6.72
S1-U-G10	230	202	0.88	4.25
D1-B-G10	459	415	0.90	-
D1-U-G10	459	394	0.86	-
S1-B-G16	230	223	0.97	9.82
S1-U-G16	230	225	0.98	7.71
S1-B-G22	230	223	0.97	12.49
S1-U-G22	230	223	0.97	4.64*
S2-B-G10	230	200	0.87	0.60*
S2-U-G10	230	181	0.79	0.44*
D2-B-G10	459	417	0.91	-
D2-U-G10	459	373	0.81	-
S2-B-G10-a	446	363	0.81	6.09*
S2-U-G10-a	446	366	0.82	3.44*

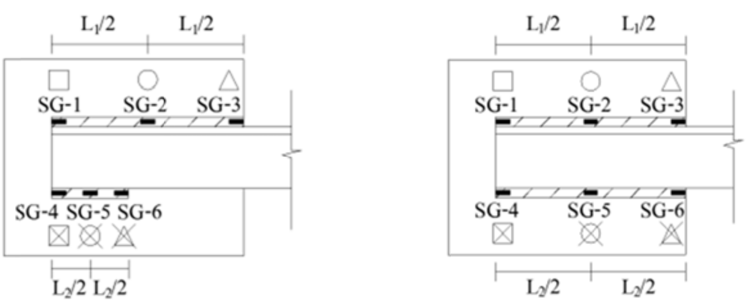
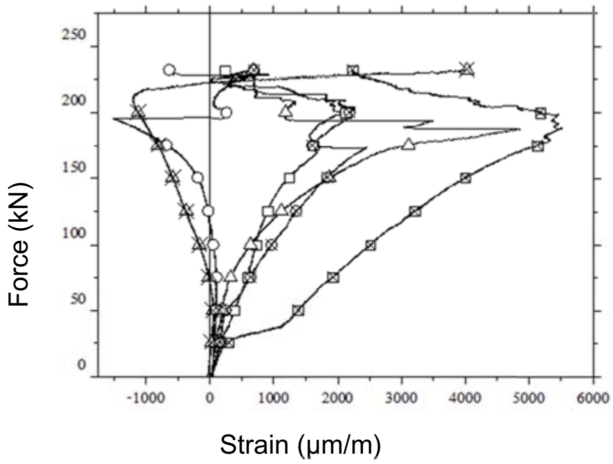
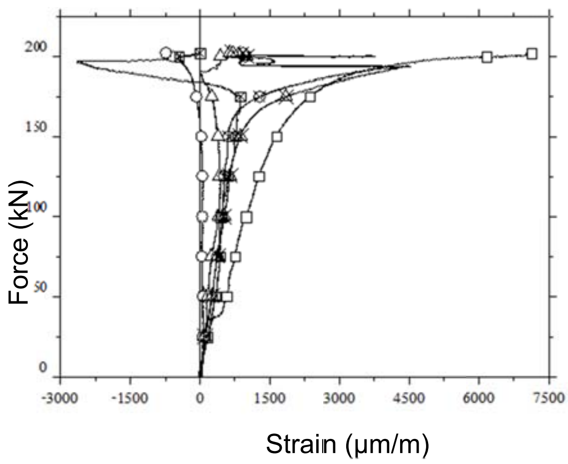


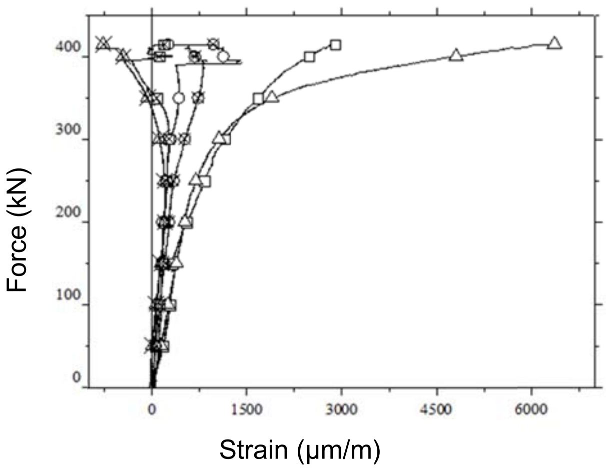
Figure 6 Locations of Strain Gauge on Welds



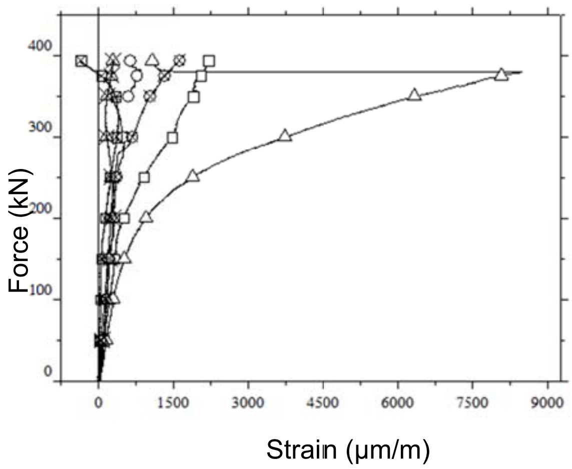
a) S1-B-G10



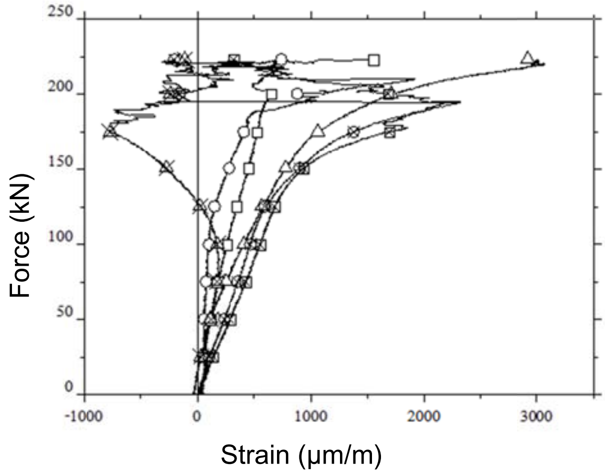
b) S1-U-G10



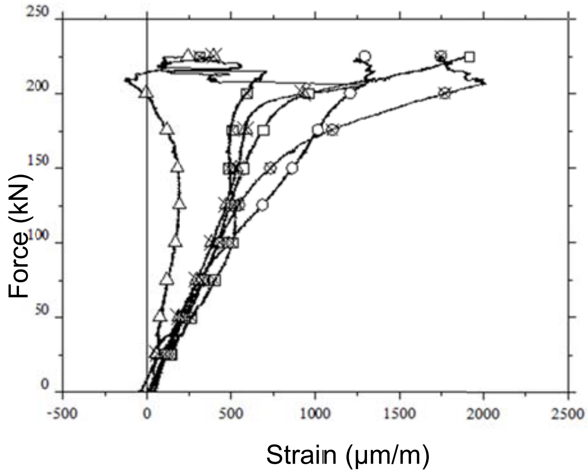
c) D1-B-G10



d) D1-U-G10



e) S1-B-G16



f) S1-U-G16

Figure 7 Selected Load vs. Strain plots of Some Specimens

5) DISCUSSIONS

Table 3 demonstrates that some specimens had test results as much as 20% lower than nominal strength calculated in accordance with AISC Specification. Results in Table 3 are rearranged into smaller groups based on parameters for further discussions.

An attempt to convert data from strain gauge into any meaningful stress did not materialize as the strain gauge itself was only suitable for straightforward normal stress. However, from Figure 7, it can be noticed that the maximum strain value always occurred on one of the corners, mostly at the top right corner (at the heel of the angle) which is located farthest away from the centroid of the weld group.

5.1 Effects of Unbalanced Arrangement on Angle Welded Connections

To appreciate the effects of unbalanced welding arrangement alone, only specimens with double angles are compared to exclude any involvement from out-of-plane bending in Table 4.

Table 4 Effects of unbalanced welds on shear rupture of angle welded connection

Specimen	Angle (mm)	Calculated Nominal Weld Rupture (kN)	Test Results (kN)	Ratios of Test Results/Nominal Strength
D1-B-G10	2-65x65x8	459	415	0.90
D1-U-G10	2-65x65x8	459	394	0.86
D2-B-G10	2-120x120x8	459	417	0.91
D2-U-G10	2-120x120x8	459	373	0.81

From Table 4, it can be clearly seen that strength of the welded end connection is 14% below calculated nominal strength in 65x65x8 mm-angle specimen and is 19% below in 120x120x8 mm-angle specimen. Use of larger angle means that the distance between the two welds, one at the heel and another at the toe, increases. This may worsen the shear rupture strength of the weld group. In addition, it should be notice that, even though the effects of unbalanced welds are conspicuous, rupture strength of balanced welds were 10% lower than the calculated nominal values.

5.2 Effects of Out-of-plane Bending on Angle Welded Connection

To demonstrate the effects of what out-of-plane bending due to the fact that the centroid of the angle does not lie in its own physical body has on strength of welds, Table 5 is created to consist of balanced welds so that the impact of unbalancing the connection does not include in the discussion. Because of the difference of the number of angles used in the specimens, a comparison on the effects of out-of-plane bending is made on the ratios of test results and nominal strength which normalizes the number of angles.

Table 5 Effects of out-of-plane bending on shear rupture of angle welded connection

Specimen	Angle (mm)	Calculated Nominal Weld Rupture (kN)	Test Results (kN)	Ratios of Test Results/Nominal Strength
D1-B-G10	2-65x65x8	459	415	0.90
S1-B-G10	65x65x8	230	232	1.01
D2-B-G10	2-120x120x8	459	417	0.91
S2-B-G10	120x120x8	230	200	0.87

From Table 5, the comparison of specimens with 120x120x8 mm angles shows that reduction of weld rupture strength due to out of plane bending is less than 5%, whereas there is no reduction at all in case of specimens with 65x65x8 mm angles when the calculated nominal strength and the test result of the welded connection were almost identical. Even though the member itself will experience out-of-plane bending under tensile load, the impact is not transferred onto its welded end connections.

5.3 Effects of Gusset Plate Thickness on Angle Welded Connections

To study the impact of gusset plate thickness on strength and behavior of welded end connections of angles, three thicknesses were used in the experiments. Results of specimens are summarized in Table 6.

Table 6 Effects of gusset plate thickness on strength and behavior of angle welded connection

Specimen	Gusset Plate Thickness (mm)	Calculated Nominal Weld Rupture (kN)	Test Results (kN)	Ratios of Test Results/Nominal Strength	Displacement of Gusset (mm)
S1-B-G10	10	230	232	1.01	6.72
S1-B-G16	16	230	223	0.97	9.82
S1-B-G22	22	230	223	0.97	12.49
S1-U-G10	10	230	202	0.88	4.25
S1-U-G16	16	230	225	0.98	7.71
S1-U-G22	22	230	223	0.97	4.64*

* denotes that the specimen was strengthened at the support to reduce lateral deformation.

In case of balanced welds, since the welded connection did not experience any strength reduction, an increase in a gusset plate thickness bears no discernable benefits. In case of unbalanced welds, thicker plates helped increase strength of the weld group significantly. However, it should be noted that an increase of plate thickness induced a huge amount of displacement of the plate itself. The displacement had become so large that the gusset plate had to strengthened by adding a stiffener at the back of it (Mekpramual 2014). It is also noticeable that displacements in specimens with balanced welds were slightly greater than those in unbalanced welds in every comparable case. Even though an increase of gusset plate thickness may help maintain the full strength of the welded connection, the size required may not be practical as the use of a 65x65x8 mm should not require a gusset plate as thick as 16 mm.

5.4 Effects of Dimensions of Welded End Connections

The effects of dimensions of the connection, i.e., the width of the connection or the distance between the two welds; and the length of the welds, are investigated by making a comparison of weld rupture in specimens with different single angle sizes as shown in Table 7, and specimens with different weld lengths as shown in Table 8, respectively.

From Table 7, it can be seen that the width of the connection, which, invariably, is equal to the leg of the angle, has significant effects on shear rupture strength of the welds. In the case of balanced welds, the strength of the weld group is lower than the nominal value by almost 15%, whereas that of the unbalanced weld group is more than 20%.

Table 7 Effects of Width of Connection on Welded End Connection

Specimen	Angle (mm)	Calculated Nominal Weld Rupture (kN)	Test Results (kN)	Ratios of Test Results/Nominal Strength
S1-B-G10	65x65x8	230	232	1.01
S2-B-G10	120x120x8	230	200	0.87
S1-U-G10	65x65x8	230	202	0.88
S2-U-G10	120x120x8	230	181	0.79

Table 8 Effects of Length of Connection on Welded End Connection

Specimen	Lengths of Welds (mm)		Calculated Nominal Weld Rupture (kN)	Test Results (kN)	Ratios of Test Results/Nominal Strength
	L_1	L_2			
S2-B-G10	120	50	230	200	0.87
S2-B-G10-a	240	90	446	363	0.81
S2-U-G10	85	85	230	181	0.79
S2-U-G10-a	165	165	446	366	0.82

Since longer welds naturally possess greater strength than the shorter ones, use of ratios of test results and nominal strength again has to be employed to normalize the results. From results shown in Table 8, it is seen that use of long welds may have further reduced its strength up to almost 20% in the balanced weld case. However, in the unbalanced case where the weld shear rupture strength was already deducted by 20%, no further reduction was evident. It is also probable that strength reduction in any case may not be beyond approximately 20%. In addition, it should be noted that the length of the welds used in the research is substantially less than 100 times its own size, which is the limit that AISC (2010) suggests that the effective length be reduced.

6. CONCLUSIONS AND SUGGESTIONS

From results and discussions in the previous sections, the following conclusions can be drawn from the research:

1. Use of unbalanced welds may reduce the strength of welded end connection as much as 20%.

2. Out-of-plane bending due to the eccentric nature of the angle itself does not penalize the strength of the welded end connection.

3. An increase of a gusset plate thickness helps strengthen the welds, but it may not be a practical solution.

4. A large welding configuration, i.e., with a large distance between the welds or with long lengths, may result in strength reduction up to 20%. The length of the welds use in the research is substantially lower than the 100 times the weld size, which is the limitation that AISC suggests that the effective length be used.

Design suggestions are as follows:

1. If possible, always use the balanced welds for end connections of angles to avoid strength reduction.

2. Exercise use of large size welded end connections with at least 20% reserve for precaution.

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