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Ultimate tensile deformation and strength capacities of bolted-angle connections

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ABSTRACT

This paper reports an experimental study of 31 bolted double-angle connections under a pure tension load. Failure modes such as angle rupture and bolt tension rupture were observed. The test results included load versus deformation curve, ultimate tensile deformation and strength. A new equation for predicting axial deformation capacity was proposed based on a regression analysis. The merits of a frame model and a truss model were investigated in predicting the ultimate tensile strength of the tested connections. A component-based model was then proposed to predict the load versus deformation curve of the connection. It was found that the current AISC equation on the prying action of bolts seriously underestimate the prying force of bolts when used with angles. A new prying equation was thus proposed for the design of bolts to ensure that the ductility of bolted-angle connections would be achieved.

1. Introduction

Bolted-angle connections are referred to a single-angle or a double-angle whose legs are connected to both supporting and supported members by high-strength bolts. This type of connections is commonly used in both simple (e.g. [1]) and semi-rigid constructions (e.g., top-and seated angle beam-to-column joints [2]).

Studies on the tensile behavior of bolted-angle connections were for the purpose either to establish the moment–rotation model of an angle connection, e.g., [2–5] or to quantify the ultimate tensile resistance of a connection for the design of connection robustness in order to preserve the integrity of a structure, e.g., [6–10].

The robustness of connections has attracted a great deal of attentions in recent years in the context of progressive collapse of structures [11]. A typical loading condition for evaluating connection robustness is the so-called column-removal scenario. As demonstrated by various researchers, such as Gong [12] and Yu et al. [13], the dominant action developed within a simple steel connection under the column-removal scenario is catenary tensile force. The failure of a connection under such a large deflection of beams or girders is typically characterized by the rupture of connection components. Gong [12] also demonstrated that connection ductility or deformation capacity plays an equally important role as its strength in the development of a catenary action.

An examination of the existing tension tests on bolted-angle connections [3,6,7,14] revealed that a large dispersion of test results existed. Therefore, a test program, which consisted of 31 double-angle connections, was conducted in this study to investigate further the behaviors of this type of connections subjected to a tensile load. As pointed out by Gong [12] that the supply of connection ductility is at the core of connection robustness design, the primary purpose of this study was to quantify the ultimate tensile deformation capacity of bolted-angle connections in addition to quantifying their ultimate strength. Comparisons with the existing test results found from literatures were also provided where available.

2. Angle specimens and test setup

Each connection specimen consisted of two identical angles (Fig. 1). The angle length was 152 mm with two bolts on each leg. Bolt pitch and end distance were 76 mm and 38 mm, respectively. The framing leg of the angles, whose width was L_2, was bolted to the upper loading arm (which would be a beam web or flange in an actual beam-to-column joint). The outstanding leg of the angles, whose width was L_1, was bolted to the lower loading arm (which would be the flange of a column in an actual beam-to-column joint). The upper arm was a 25 mm thick plate. The lower arm was T-shaped and was made to be rigid through two stiffeners. The deformation of the loading arms during test was negligible compared with that of angles. The set-back of the upper arm plate for the angles was 28 mm.

Table 1 gives the dimensions of the specimens in addition to the test results. Each specimen was named a unique ID as follows: the first letter represents its group name, followed by the thickness of angle (rounded to the closest integer), then the bolt gauge g_1 (Fig. 1), and the last, the

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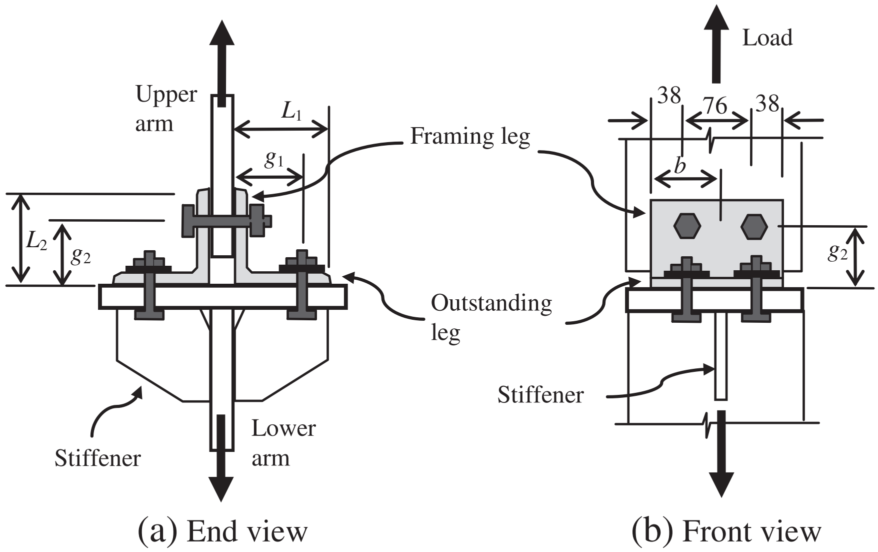

Fig. 1. Design of bolted-angle specimens.

specimen number. For example, B10-65-2 represents the second specimen in group B which has a nominal angle thickness of 9.5 mm and gauge g_1 of 65 mm. The test variables included bolt gauge g_1 (which was 45, 65 and 85 mm respectively for groups A, B and C), angle thickness t (ranging from 6.4 mm to 12.7 mm), and leg width L_1 (ranging from 76 mm to 127 mm). The bolt gauge g_2 was 65 mm for all the specimens.

Angles were made of CSA/G40.21 300 W steel [15]. Material coupons were cut for measuring strength. The measured angle thickness t, material yielding strength F_y and ultimate tensile strength F_u are given in Table 1. Since the specimens were fabricated in two batches, some angles having the same designation had different material strengths.

All the specimens used diameter $d_b = 22.2$ mm (7/8 in.) ASTM A325 high-strength bolts, which were snug-tightened during test. The average strength of a single-bolt was 284 kN with a coefficient of variation of 0.02 based on a direct tension test of 9 single-bolts. Punched standard holes of diameter 23.8 mm were used. As shown in Fig. 1, the washer of the tension bolts were placed against the outstanding leg in order to prevent the bolt from pulling through the leg, a failure mode observed in [6,14].

Table 1
Connection specimens and test results.

ID	Specified angle $L_1 \times L_2 \times t$ (mm)	Actual t (mm)	F_y (MPa)	F_u (MPa)	Δ_t (mm)	Bolt slip (mm)	T_t (kN)	Failure mode[a]	D.P.[b]
A6-45-1	76 × 102 × 6.4	6.55	370	518	22.4	2.0	319	R.A.	1
A6-45-2		6.74	370	518	22.3	0	350	R.A.	1
A10-45-1	76 × 102 × 9.5	9.62	387	534	25.6	1.8	565	R.A.	1
A10-45-2		9.69	387	534	26.5	1.9	576	R.B.	3
A10-45-3		9.92	402	571	23.1	0.8	573	R.B.	3
A10-45-4		9.80	402	571	24.9	1.9	591	R.B.	3
A13-45-1	76 × 102 × 13	12.71	377	555	16.2	0.7	696	R.B.	4
A13-45-2		12.87	377	555	15.2	0.8	691	R.B.	4
A13-45-3		12.86	377	555	19.6	3.2	708	R.B.	4
B6-65-1	102 × 102 × 6.4	6.62	362	512	41.3	0	361	R.A.	2
B6-65-2		6.53	362	512	54.3	2.2	449	R.A.	2
B8-65-1	102 × 102 × 7.9	7.93	366	505	30.3	0	323	R.A.	1
B8-65-2		7.96	366	505	26.4	0.6	290	R.A.	1
B10-65-1	102 × 102 × 9.5	9.68	362	527	35.3	0.3	500	R.A.	1
B10-65-2		9.63	362	527	31.2	2.0	423	R.A.	1
B10-65-3		9.79	387	553	29.1	0.8	388	R.A.	1
B10-65-4		9.77	387	553	31.7	0.5	405	R.A.	1
B13-65-1	102 × 102 × 13	12.44	383	579	27.0	0.2	565	R.A.	1
B13-65-2		12.39	383	579	25.7	0.6	560	R.B.	3
B13-65-3		12.77	351	591	26.2	0.9	559	R.B.	3
B13-65-4		12.82	351	591	29.4	1.6	576	R.B.	3
C8-85-1	127 × 127 × 7.9	7.79	394	512	27.8	3.2	199	R.A.	1
C8-85-2		7.68	394	512	33.1	0.1	252	R.A.	1
C10-85-1	127 × 127 × 9.5	9.36	395	533	29.8	0.3	311	R.A.	1
C10-85-2		9.38	395	533	31.5	0.1	355	R.A.	1
C10-85-3		9.78	405	556	38.5	1.9	371	R.A.	1
C10-85-4		9.69	405	556	37.8	0.6	374	R.A.	1
C13-85-1	127 × 127 × 13	12.36	377	500	27.5	3.1	409	R.A.	1
C13-85-2		12.29	377	500	33.1	2.1	471	R.A.	1
C13-85-3		12.93	371	516	36.8	0.6	503	R.A.	1
C13-85-4		12.61	371	516	42.1	1.5	556	R.A.	1

[a] R.A. = Rupture of angle; R.B. = Rupture of tension bolt.
[b] D.P. = deformation pattern, as defined in Fig. 4.