Design Aid for Triangular Bracket Plates Using AISC Specifications

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A typical triangular bracket plate rigidly supported on the shorter sides and free on the longer side is shown in Figures 1a and 1b. Such triangular gusset plates are found in base plate to column connections (Figure 1c), stiffened seated connections (Figure 1d), and bracket to column connections. The behavior of such bracket plates under an applied load has been provided by Salmon (1962) and Tall (1964).

Bracket plates are normally designed using either the elastic strength method (Salmon, 1962) or the plastic strength method (Tall, 1964). The elastic method of design is based on the assumption that the centroid of applied load is at approximately 0.6 times the length of the loaded side of the bracket plate from the 90° corner (i.e., $s \approx 0.6b$ in Figure 1a). The plastic strength method assumes that the plastic strength develops on a section normal to the free edge and passing through the 90° corner at point *o* (Figure 1a). The relations in the elastic method do not include the variable distance, *s* (position of the load), while the plastic strength method does. The laboratory tests (Salmon, Beuttner and O'Sheridan, 1964) showed that yielding of the free edge occurs prior to buckling.

The method presented in this paper is an adoption of the method presented by Martin and Robinson (1982), developed in the United Kingdom, with the incorporation of relations for compressive stresses from the AISC *Specification for Structural Steel Buildings,* hereafter referred to as the AISC *Specification* (AISC, 2005a). This method assumes that failure of the plate occurs by flexural buckling of the plate. Therefore, both elastic and inelastic compressive strength relations for buckling given in the AISC *Specification* are included to arrive at the strength of the bracket plate.

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The task of the designers of a bracket plate is to find the thickness, *t*, of the plate (Figure 1b) based on the known values of plate aspect ratio, *a*/*b*, position of load, *s*, design load (required strength, P_u or P_a), and steel properties (e.g., modulus of elasticity, *E*, and yield stress, F_y). The derivation of design relations is given in the next section.

DERIVATION OF DESIGN RELATIONS

The bracket plate, with an elemental strut of width *dz* parallel to the free edge and length l_z, is shown in Figure 2a. The end conditions for this strut are assumed to be fixed-fixed. The stress distribution across *B* is shown in Figure 2b. The effective slenderness ratio (Kl_z/r) for this strut in terms of plate geometry parameters can be written as

$$\frac{Kl_z}{r} = \sqrt{12}K\left(\frac{a}{b} + \frac{1}{a/b}\right)\frac{z}{t} \tag{1}$$

where

K = effective length factor
a = height of the supporting edge
b = length of the loaded edge
dz = width of the elemental strut
t = thickness of the elemental strut/bracket plate
l_z = unsupported length of the elemental strut at distance z from o

$$= \left(\frac{a}{b} + \frac{1}{a/b}\right)z \tag{2}$$

r = radius of gyration of the elemental strut

$$= \sqrt{\frac{I}{A}} = \sqrt{\frac{dz\,t^3}{12\,dz\,t}} = \frac{t}{\sqrt{12}} \tag{3}$$

The normal distance *B* from the inside corner to the free edge of the plate can be expressed in terms of plate geometry parameter as

$$B = \frac{a}{\sqrt{1 + \left(\dfrac{a}{b}\right)^2}} \tag{4}$$

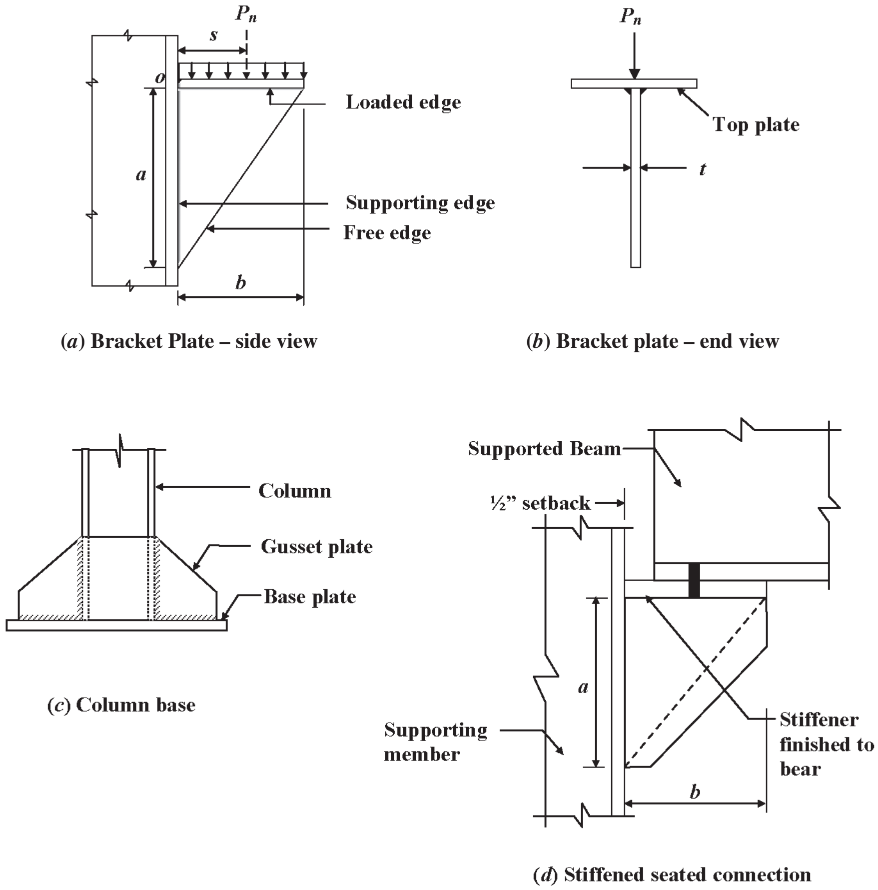

Fig. 1. Triangular bracket plates.

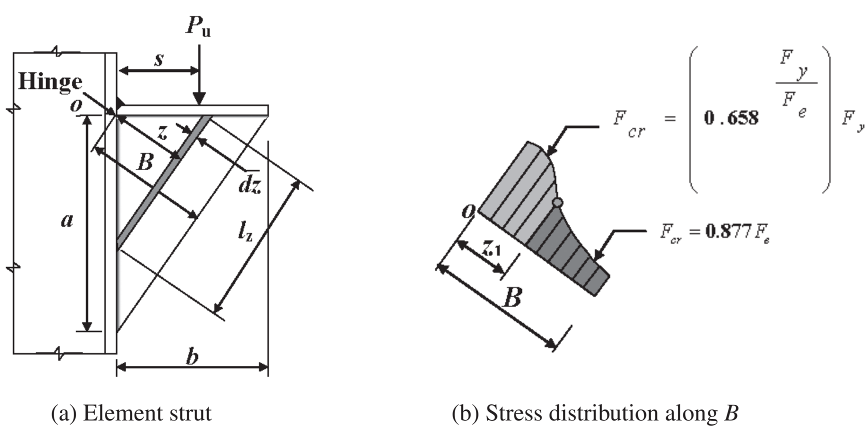

Fig. 2. Triangular bracket plate with an elemental strut.

Table 1. Limiting Values of *t*/b*							
F_y (ksi)	**0.5**	**0.75**	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**
36	0.0188	0.0210	0.0238	0.0303	0.0376	0.0453	0.0532
50	0.0222	0.0248	0.0281	0.0358	0.0444	0.0534	0.0627

The expressions for the critical stress, F_{cr}, given in the AISC *Specification* (Equations E3-2 and E3-3) are

For $\frac{Kl_z}{r} \le 4.71\sqrt{\frac{E}{F_y}}$

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y \tag{5}$$

For $\frac{Kl_z}{r} > 4.71\sqrt{\frac{E}{F_y}}$

$$F_{cr} = 0.877F_e \tag{6}$$

where

F_e = elastic critical buckling stress

$$= \frac{\pi^2 E}{\left(\frac{Kl_z}{r}\right)^2} \tag{7}$$

Substituting Equation 7 into Equations 5 and 6 and simplifying, we can rewrite Equations 5 and 6

For $\lambda_c \le 1.5$

$$F_{cr} = \left(0.658^{\lambda_c^2}\right)F_y \tag{8}$$

For $\lambda_c > 1.5$

$$F_{cr} = \left(\frac{0.877}{\lambda_c^2}\right)F_y \tag{9}$$

where

$$\lambda_c = \frac{Kl_z}{r\pi}\sqrt{\frac{F_y}{E}} \tag{10}$$

Using Equation 1, Equation 10 for the elemental strut at distance z from point o (see Figure 2a) can be expressed as

$$\lambda_{cz} = \alpha z \tag{11}$$

where

$$\alpha = \frac{2\sqrt{3}K}{\pi}\sqrt{\frac{F_y}{E}}\left(\frac{a}{b} + \frac{1}{a/b}\right)\frac{1}{t} \tag{12}$$

The experimental work (Martin, 1979; Martin and Robinson, 1982) showed that the stress in the strip at the 90° corner is at yield and the stress in the strip near the free edge depends on the slenderness ratio of the strip at the free edge. Therefore, it is necessary to determine the boundary of the two regions where inelastic buckling and elastic buckling control the critical compressive stress of the strip. The location of this boundary denoted by z_1 (measured from the inside 90° corner) can be determined from Equation 8 with the help of Equations 9 and 10 and is given here:

$$z_1 = \frac{3}{2\alpha} = \frac{\sqrt{3}}{4}\frac{\pi}{K}\sqrt{\frac{E}{F_y}}\left[\frac{a/b}{1+\left(a/b\right)^2}\right]t \tag{13}$$

This relation may be written in a dimensionless form by scaling with Equation 4 and simplifying. Thus, we have

$$\frac{z_1}{B} = \frac{\sqrt{3}}{4}\frac{\pi}{K}\sqrt{\frac{E}{F_y}}\left[\frac{t/b}{\sqrt{1+\left(a/b\right)^2}}\right] \tag{14}$$

Note that a value of $z_1/B > 1$ from Equation 14 indicates that all the strips fail by inelastic buckling. Substituting this condition into Equation 14 results

$$\frac{t^*}{b} = \frac{4}{\sqrt{3}}\frac{K}{p}\sqrt{\frac{F_y}{E}}\sqrt{1+\left(\frac{a}{b}\right)^2} \tag{15}$$

The limiting values of t^*/b for different values of a/b and F_y/E are presented in Table 1.

Note that if the actual value of t/b is less than the limiting value given in Table 1 for given a/b and F_y, the strength relation (see Equation 19) will have contributions of both inelastic and elastic buckling stresses.

The nominal compressive strength based on the limit state of flexural buckling for a compression member is given by

$$P_n = A_g F_{cr} \tag{16}$$

where

A_g = gross area of member

The relationships of the required strengths (P_u or P_a) to the nominal load, P_n, from the AISC *Specification* (AISC, 2005a) may be written as

$$P_u = \phi_c P_n \tag{17a}$$

$$P_a = \frac{P_n}{\Omega_c} \tag{17b}$$

where

ϕ_c = 0.90 for load and resistance factor design (LRFD)

Ω_c = 1.67 for allowable stress design (ASD)

The 90° corner of the plate is assumed to act as a hinge. The strut near the 90° corner could attain yield stress while the stresses in the farther struts depend on the slenderness ratio of the strut. See Figure 2b for the distribution of critical compressive stress. Equating sum of the moments of all elemental struts due to compressive forces about the hinge at point *o* to the external moment due to the externally applied load, we have

$$P_n\, s = \int_0^B F_{cr}\,(t\,dz)\; z \tag{18}$$

Restoring Equations 8 and 9 for F_{cr} and with the limits as shown in Figures 2b, Equation 18 can be expressed as

$$P_n\, s = tF_y \left[\int_0^{z_1} 0.658^{\lambda_{cz}^2} z\,dz + 0.877 \int_{z_1}^{B} \frac{z dz}{\lambda_{cz}^2} \right] \tag{19}$$

After integrating, simplifying and making Equation 19 dimensionless by scaling both sides with (b^3E), we obtain

$$\frac{P_n s}{b^3 E} = \frac{\pi^2}{12K^2} \frac{\left(a/b\right)^2}{\left[1+\left(a/b\right)^2\right]^2} \left(\frac{t}{b}\right)^3 \times \left[0.72877 + 0.877 \log_e \left(\frac{B}{z_1}\right)\right] \tag{20}$$

The details of integration are shown in Appendix A. Note that $\frac{B}{z_1} > 1$ in the preceding equation.

Making use of Equation 14 in Equation 20 yields

$$\frac{P_n s}{b^3 E} = \frac{\pi^2}{12K^2} \frac{\left(a/b\right)^2}{\left[1+\left(a/b\right)^2\right]^2} \left(\frac{t}{b}\right)^3 \times \left\{0.72877 + 0.877 \log_e \left[\frac{4}{\sqrt{3}} \frac{K}{\pi} \sqrt{\frac{F_y}{E}} \sqrt{1+\left(\frac{a}{b}\right)^2} \frac{1}{t/b}\right]\right\} \tag{21}$$

Knowing the applied load, the plate geometry, and the material properties, Equation 21 can be solved for *t*/*b* by trial and error or a simple computer program may be used to solve numerically.

When $\frac{B}{z_1} > 1$, the compressive stress is controlled by inelastic buckling in all strips and elastic buckling does not occur. For this case, Equation 18 after integration with the upper limit as *B* and simplification results in

$$\frac{P_n s}{b^3 E} = \frac{\pi^2}{12K^2} \frac{\left(a/b\right)^2}{\left[1+\left(a/b\right)^2\right]^2} \left(\frac{t}{b}\right)^3 \times \left[\frac{0.658^{\left(\frac{12K^2 F_y [1+(a/b)^2]}{\pi^2 E (t/b)^2}\right)} - 1}{2 \log_e 0.658}\right] \tag{22}$$

COMPARISON TO EXPERIMENTAL AND THEORETICAL RESULTS

Equations 21 and 22 are used to compute the nominal strengths, P_n, and are compared with the experimental and theoretical results, P_{expt}, (Salmon, Buettner and O'Sheridan, 1964; Martin, 1979; Martin and Robinson, 1982). This comparison is shown in Tables 2a, 2b and 2c in terms of $P_{expt.}/P_n$. Note that the value of effective length factor, *K*, is taken equal to 0.5 in these calculations for comparison with other theoretical results. Also, the values of yield stresses in Tables 2a, 2b, and 2c are the measured values. The comparison shows that the authors' developed relations predict nominal load closer to the experimental results than the other theoretical methods.

Table 2a. Comparison with Experimental Results of Salmon et al. (1964) (E = 29,000 ksi)									
Specimen No.	b in.	a in.	t in.	s in.	F_y ksi	$P_{expt.}$ kips	$P_{expt.}/P_n$		
							Salmon et al. (1964)	Martin (1979)	Authors
1	9.0	12.0	0.386	5.4	43.2	97.8	1.87	1.56	1.41
2	22.5	30.0	0.277	13.5	41.2	63.3	1.58	1.56	1.43
3	22.5	30.0	0.384	13.5	43.2	125.8	1.18	1.54	1.34
4	9.0	9.0	0.268	5.4	41.2	40.0	1.91	1.37	1.18
5	9.0	9.0	0.378	5.4	43.2	69.5	2.25	1.40	1.26
6	30.0	30.0	0.268	18.0	41.2	49.5	2.19	1.54	1.37
7	30.0	30.0	0.385	18.0	43.2	101.7	1.52	1.41	1.20
8	13.5	9.0	0.271	8.1	41.2	31.3	2.00	1.26	1.10
9	13.5	9.0	0.374	8.1	43.2	64.5	2.85	1.52	1.37
10	30.0	20.0	0.276	18.0	41.2	35.8	1.60	1.27	1.21
11	30.0	20.0	0.384	18.0	43.2	80.1	1.55	1.42	1.28
12	18.0	9.0	0.274	10.8	41.2	29.8	2.10	1.42	1.38
13	18.0	9.0	0.387	10.8	43.2	46.6	2.21	1.25	1.19
14	30.0	15.0	0.373	18.0	43.2	57.6	1.70	1.37	1.39
15	30.0	15.0	0.373	18.0	43.2	58.5	1.73	1.39	1.41

Table 2b. Comparison with Experimental Results of Martin (1979) (E = 29,877 ksi)									
Specimen No.	b in.	a in.	t in.	s in.	F_y ksi	$P_{expt.}$ kips	$P_{expt.}/P_n$		
							Salmon et al. (1964)	Martin (1979)	Authors
1	5.71	5.71	0.240	3.15	43.9	28.7	2.27	1.31	1.17
2	5.83	11.61	0.257	4.02	37.4	42.6	1.32	1.69	1.59
3	5.94	11.77	0.255	4.06	37.4	43.7	1.32	1.52	1.62
4	4.92	4.92	0.254	2.52	37.0	30.2	3.11	1.52	1.42
5	4.92	4.92	0.254	2.52	37.0	35.8	3.68	1.80	1.68
6	4.80	14.84	0.253	2.56	37.0	39.6	1.10	1.28	1.38
7	4.72	14.65	0.254	2.46	37.0	47.9	1.34	1.52	1.64

DESIGN AID TABLES

In design problems, the values of E, F_y, a, b, $\phi_c P_n$ (= required strength, P_u in LRFD) or P_n/Ω_c (= required strength, P_a in ASD), and s are known and the thickness, t, of the plate is to be determined. Therefore, design tables would become very helpful in the design process. Two design aids in tabular form, namely, Tables 3 and 4 are presented for steel with F_y = 36 ksi and 50 ksi, respectively. The plate aspect ratio, a/b, ranges from 0.50 to 3.00 and the dimensionless moment, $P_n\, s/b^3E$, varies from 0.25×10^{-6} to 50×10^{-6} in these tables. Knowing the design load, the design procedure is straightforward and the various steps are enumerated as follows:

1. Compute a/b, the nominal load, and $P_n\, s/b^3E$ from the known design information.

2. Determine t/b from Table 3 or 4, depending on the steel grade.

Table 2c. Comparison with Experimental Results of Martin and Robinson (1984) (E = 29,877 ksi)							
Specimen No.	b in.	a in.	t in.	s in.	F_y ksi	$P_{expt.}$ kips	$P_{expt.}/P_n$ Authors
1	3.94	3.94	0.157	1.97	54.8	14.7	1.00
2	7.87	7.87	0.157	3.94	54.8	19.8	0.99
3	7.87	7.87	0.157	3.94	54.8	21.3	1.07
4	11.81	11.81	0.157	5.91	54.8	17.7	0.93
5	15.75	15.75	0.157	7.88	54.8	15.0	0.87
6	15.75	15.75	0.157	7.88	54.8	15.6	0.90
7	19.69	19.69	0.157	9.85	54.8	17.5	1.11
8	19.69	19.69	0.157	9.85	54.8	18.0	1.14
9	7.87	1.97	0.157	3.94	54.8	4.9	1.66
10	7.87	3.94	0.157	3.94	54.8	12.3	1.28
11	7.87	3.94	0.157	3.94	54.8	13.0	1.36
12	7.87	5.91	0.157	3.94	54.8	19.0	1.19
13	7.87	7.87	0.157	3.94	54.8	19.5	0.98
14	7.87	7.87	0.157	3.94	54.8	20.7	1.04
15	7.87	9.84	0.157	3.94	54.8	24.1	1.12
16	7.87	9.84	0.157	3.94	54.8	27.0	1.26
17	7.87	11.81	0.157	3.94	54.8	24.6	1.15
18	7.87	11.81	0.157	3.94	54.8	27.3	1.28
19	7.87	15.75	0.157	3.94	54.8	24.6	1.30
20	7.87	15.75	0.157	3.94	54.8	25.0	1.32
21	7.87	19.69	0.157	3.94	54.8	26.4	1.65
22	7.87	19.69	0.157	3.94	54.8	27.0	1.69
23	7.87	23.62	0.157	3.94	54.8	23.4	1.76
24	7.87	23.62	0.157	3.94	54.8	26.4	1.98

EXAMPLES

For comparative purposes, the following two examples (Salmon and Johnson, 1990; Tall, 1974) are reworked.

Example 1

Determine the thickness required for a triangular bracket plate shown in Figure 3 to carry a factored load of 60 kips. Assume the load is located 15 in. from the face of support. Use the LRFD method with F_y = 36 ksi.

Solution

$a/b = 20/25 = 0.8$

$P_n = P_u/\phi_c = 60/0.9 = 66.7$ kips

$P_n\, s/b^3E = (66.7)(15)/[(25^3)(29,000)] = 2.208 \times 10^{-6}$

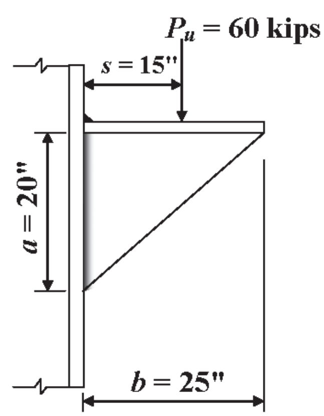

Fig. 3. Triangular bracket plate for Example 1.

Table 3. Nominal Strength of Triangular Bracket Plates Using AISC Column Strength Equations (F_y = 36 ksi, K = 0.65)

$P_n s/b^3E$ ($\times 10^{-6}$)	Values of t/b ($\times 10^{-3}$) for a/b equal to						
	0.50	**0.75**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**
0.25	8.06	6.78	6.41	6.54	7.06	7.69	8.35
0.50	10.93	9.07	8.52	8.65	9.31	10.12	10.98
0.75	13.16	10.79	10.09	10.20	10.95	11.89	12.89
1.00	15.11	12.24	11.40	11.47	12.30	13.35	14.46
2.00	22.12	16.89	15.46	15.36	16.36	17.69	19.12
4.00	36.37	24.67	21.60	20.89	22.03	23.68	25.50
6.00	51.37	32.47	27.14	25.36	26.47	28.30	30.37
8.00	66.81	40.50	32.71	29.44	30.35	32.28	34.52
10.00	82.47	48.75	38.38	33.43	33.96	35.90	38.26
12.00	98.27	57.16	44.17	37.43	37.44	39.29	41.72
14.00	114.10	65.69	50.06	41.46	40.90	42.56	45.01
16.00	130.10	74.30	56.03	45.54	44.36	45.78	48.18
18.00	146.10	82.98	62.08	49.66	47.83	48.99	51.28
20.00	162.10	91.70	68.19	53.83	51.32	52.19	54.36
22.00	178.10	100.50	74.34	58.05	54.84	55.41	57.43
24.00	194.10	109.20	80.53	62.30	58.39	58.64	60.50
26.00	210.20	118.10	86.76	66.60	61.97	61.88	63.57
28.00	226.20	126.90	93.01	70.92	65.57	65.15	66.66
30.00	242.30	135.70	99.28	75.28	69.21	68.43	69.76
32.00	258.30	144.60	105.60	79.66	72.87	71.74	72.87
34.00	274.40	153.50	111.90	84.06	76.55	75.06	75.99
36.00	290.50	162.30	118.20	88.49	80.25	78.41	79.14
38.00	306.60	171.20	124.50	92.93	83.98	81.77	82.29
40.00	322.70	180.10	130.90	97.39	87.73	85.16	85.47
42.00	338.70	189.00	137.20	101.90	91.49	88.56	88.66
44.00	354.80	197.90	143.60	106.30	95.27	91.98	91.86
46.00	370.90	206.80	150.00	110.80	99.06	95.41	95.08
48.00	387.00	215.70	156.30	115.40	102.90	98.86	98.32
50.00	403.10	224.60	162.70	119.90	106.70	102.30	101.60

Table 4. Nominal Strength of Triangular Bracket Plates Using AISC Column Strength Equations (F_y = 50 ksi, K = 0.65)

$P_n s/b^3E$ ($\times 10^{-6}$)	Values of t/b ($\times 10^{-3}$) for a/b equal to						
	0.50	**0.75**	**1.00**	**1.50**	**2.00**	**2.50**	**3.00**
0.25	7.76	6.57	6.22	6.37	6.89	7.51	8.17
0.50	10.42	8.73	8.24	8.40	9.05	9.86	10.71
0.75	12.44	10.33	9.72	9.87	10.63	11.56	12.55
1.00	14.15	11.67	10.94	11.08	11.91	12.95	14.05
2.00	19.74	15.81	14.66	14.71	15.75	17.07	18.48
4.00	29.74	22.03	20.00	19.75	21.00	22.67	24.47
6.00	40.02	27.58	24.35	23.66	25.00	26.90	28.97
8.00	50.66	33.14	28.37	27.05	28.40	30.46	32.75
10.00	61.56	38.79	32.35	30.16	31.46	33.64	36.09
12.00	72.65	44.55	36.35	33.12	34.30	36.54	39.13
14.00	83.85	50.42	40.39	36.01	36.98	39.26	41.95
16.00	95.14	56.37	44.49	38.88	39.56	41.85	44.62
18.00	106.50	62.39	48.63	41.76	42.09	44.33	47.16
20.00	117.90	68.48	52.82	44.64	44.58	46.74	49.61
22.00	129.30	74.61	57.05	47.54	47.07	49.10	51.98
24.00	140.80	80.79	61.33	50.46	49.56	51.43	54.30
26.00	152.30	87.00	65.64	53.40	52.05	53.74	56.57
28.00	163.70	93.23	69.98	56.36	54.54	56.05	58.81
30.00	175.30	99.49	74.35	59.34	57.05	58.36	61.04
32.00	186.80	105.80	78.75	62.34	59.56	60.67	63.25
34.00	198.30	112.10	83.16	65.37	62.09	62.98	65.46
36.00	209.80	118.40	87.60	68.41	64.63	65.29	67.67
38.00	221.40	124.70	92.05	71.46	67.18	67.62	69.88
40.00	232.90	131.00	96.52	74.54	69.74	69.95	72.10
42.00	244.50	137.40	101.00	77.63	72.32	72.28	74.31
44.00	256.00	143.70	105.50	80.74	74.91	74.63	76.53
46.00	267.60	150.10	110.00	83.85	77.51	76.98	78.75
48.00	279.20	156.50	114.50	86.99	80.12	79.34	80.98
50.00	290.70	162.80	119.00	90.13	82.75	81.72	83.22

From Table 3, with F_y = 36 ksi and using interpolation:

$$t/b = 17.26 \times 10^{-3} \qquad \Rightarrow t = 0.432 \text{ in.}$$

The thickness for the example problem by Salmon and Johnson (1990) is 0.58 in. Note the difference in the result could be due to not considering the effect of variable s in the design.

Example 2

Determine the thickness of the triangular plate in the stiffened beam seat as shown in Figure 4. Use the ASD method with F_y = 36 ksi.

Solution

Neglect the plate material below the dashed line.

$a/b = 10/6 = 1.667$
$P_n = \Omega_c P_a = (1.67)(34) = 56.8$ kips
$P_n s/b^3E = (56.8)(3.8)/[(6^3)(29{,}000)] = 34.46 \times 10^{-6}$

From Table 3 with $F_y = 36$ ksi and using interpolation:

$t/b = 81.38 \times 10^{-3} \quad \Rightarrow t = 0.49$ in.

The thickness for the example problem by Tall (1974) is 0.57 in. Note the difference in the result is due to not considering the effect of variable *s* in the design.

Note that the design work also requires checking the bracket plate system for all the limit states for bracket plates as per AISC *Specification* Equations J4-1 to J4-4. These equations are also reproduced here.

For tensile yielding:

$$R_n = F_y A_g \tag{23}$$

where
$\phi_c = 0.90$ (LFRD)
$\Omega_c = 1.67$ (ASD)

For tensile rupture:

$$R_n = F_u A_e \tag{24}$$

where
F_u = minimum specified tensile strength
ϕ_c = 0.75 (LFRD)
Ω_c = 2.00 (ASD)

For shear yielding:

$$R_n = 0.6\, F_y A_g \tag{25}$$

where
ϕ_c = 1.00 (LFRD)
Ω_c = 1.50 (ASD)

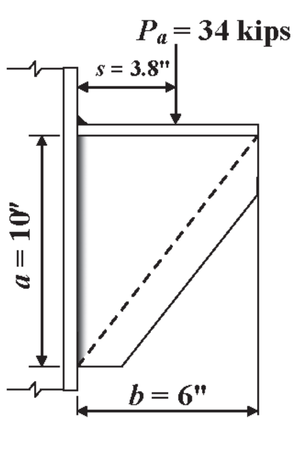

Fig. 4. Triangular bracket plate for Example 2.

For shear rupture:

$$R_n = 0.6\, F_u A_{nv} \tag{26}$$

where
A_{nv} = net area subject to shear
ϕ_c = 0.75 (LFRD)
Ω_c = 2.00 (ASD)

CONCLUSIONS

Based on the comparison of results based on the authors' developed relations with the results of design methods by (Salmon et al., 1964; Martin, 1979; Martin and Robinson, 1982) and the experimental results, the following conclusions can be made:

- The relations developed based on column strength equations from the AISC *Specification* are accurate and conservative.
- The developed relations can also be used to size the stiffener plate in a stiffened seated connection as shown in Figure 1d with $s = 0.8b$ [Refer to Figure 10-10b (AISC, 2005b)].
- The limiting value for plate thickness, t^*, is established (see Equation 15 and Table 1) to avoid elastic buckling failure.
- The equations developed are applicable to both ASD and LRFD design approaches.
- The authors developed relations comparable to other theoretical relations in that they include most of the design parameters.

NOMENCLATURE

A_e = net effective area of the plate member

A_g = gross area of the plate member

A_{nv} = net area subject to shear

a = height of bracket

b = length of the loaded side of plate

B = normal distance from hinge (corner) to free edge of plate

dz = width of the elemental strut

E = modulus of elasticity

F_{cr} = critical compressive stress

F_e = elastic critical buckling stress

F_u = minimum tensile stress of steel

F_y = minimum yield stress of steel

I = second moment of area of steel section

K = effective length factor

l_z = length of strut at z distance from o

P_a, P_u = service and factored load

P_n, R_n = nominal design strength

r = radius of gyration

s = position of load

t = thickness of bracket plate

ϕ_c = resistance factor (load and resistance factor design)

Ω_c = safety factor (allowable stress design)

APPENDIX A

Substituting Equation 11 for λ_{cz} into Equation 19, we get

$$P_n\, s = tF_y \left[\int_0^{z_1} 0.658^{(\alpha z)^2} z\, dz + 0.877 \int_{z_1}^{B} \frac{z dz}{(\alpha z)^2} \right] \quad \text{(A-1)}$$

Integrating Equation A-1, we have

$$P_n\, s = tF_y \left[\frac{\left[0.658^{\alpha^2 z^2} \right]_0^{z_1}}{2\alpha^2 \log_e 0.658} + \frac{0.877 \left[\log_e z \right]_{z_1}^{B}}{\alpha^2} \right] \quad \text{(A-2)}$$

Substituting for α and z_1 using Equations 12 and 13, scaling both sides by b^3E, and simplifying, Equation A-2 will reduce to Equation 20.

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