Column Flange Strength at Moment End-Plate Connections

LARRY E. CURTIS and THOMAS M. MURRAY

INTRODUCTION

Current American Institute of Steel Construction (AISC) design recommendations for moment end-plate connections are basically limited to the end-plate, bolts, and the compression region of the column side of the connection (AISC[2,3,4]). Although specific design procedures for column flange strength at the tension regions of the connection have not been included in AISC design manuals, much research on this topic has been conducted in Europe (Zoetemeijer,[16] Packer and Morris,[15] Mann and Morris,[12] Kennedy, Vinnakota, and Sherbourne,[8]) and in the United States.[4,7] (See Murray, Ref. 13, for a more complete list.)

The purpose of this paper is to present design recommendations for required column flange strength at the tension region of the moment end-plate connection configurations shown in Figs. 1 and 2. The configuration shown in Fig. 1 will be referred to as the 4-bolt stiffened end-plate and that in Fig. 2 as the 8-bolt stiffened end-plate. A design procedure for the latter configuration has recently been published.[14]

BACKGROUND

Limit states associated with the column flange at moment end-plate connections include column flange flexural strength, connection stiffness, and the effect on tension bolt forces because of flange bending. Criteria to evaluate these limit states have typically been developed using a tee-stub analogy. In this analogy, a prescribed effective column flange length is used for the length of the tee-stub flange as shown in Fig. 3. Procedures utilizing yield-line theory and finite element analysis have been used to analyze this tee-stub model.

Yield-line based studies were performed by Zoetemeijer;[16] Packer and Morris;[15] Mann and Morris;[12] and Kennedy, Vinnakota, and Sherbourne,[8] among others. All these studies utilize the concept of an effective column flange length and an assumed yield-line pattern over this length. The first three studies develop design methods based on experimentally tested beam-to-column moment end-plate connections. The latter study utilizes two tee-stub tests to justify the results.

Finite element studies, have been performed by Krishnamurthy,[9,10] Ahuja,[2] and Ghassemich.[6] The first studies resulted in design procedure for 4-bolt, stiffened end-plates

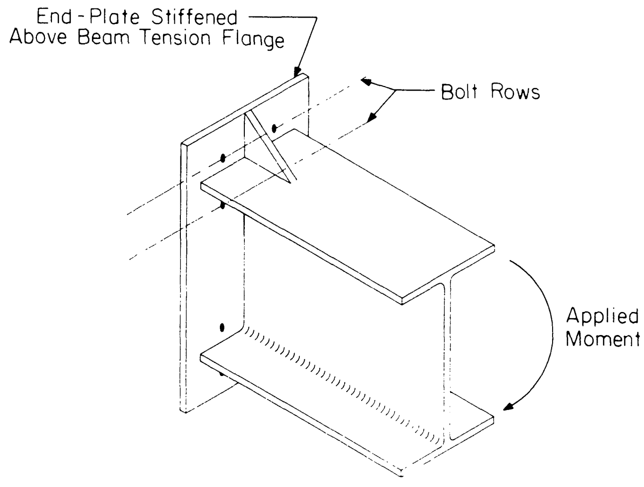

Fig. 1. 4-Bolt, Stiffened Moment End-Plate Connection

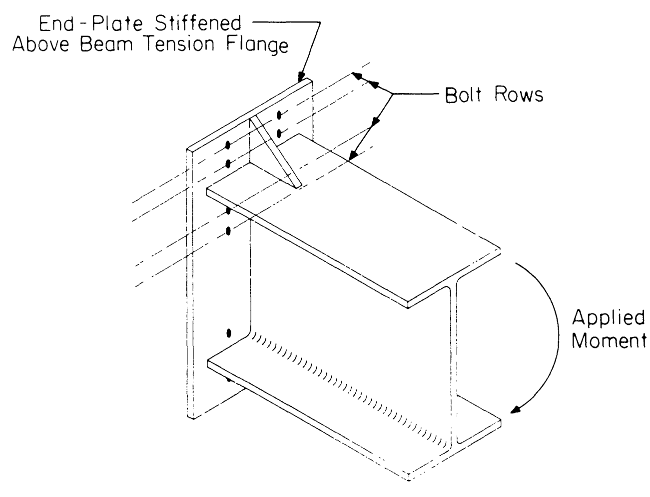

Fig. 2. 8-Bolt, Stiffened Moment End-Plate Connection

Larry E. Curtis is structural engineer, Frankfurt-Short-Bruza Associates P.C., Oklahoma City, Oklahoma.

Thomas M. Murray is Montague-Betts Professor of Structural Steel Design, The Charles E. Via, Jr., Department of Civil Engineering, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

(Fig. 1). The latter two studies used the tee-stub analogy to develop design procedures for the 8-bolt stiffened, end-plate configuration shown in Fig. 2. All of these studies resulted in design equations for end-plate strength that were developed using regression analyses techniques and finite element analysis results. The latter two studies also provide regression analysis based equations for predicting end-plate stiffness and bolt force including prying effects.

Although these procedures are for end-plate design, they can be adapted for the design of the column flange in the tension region of a beam-to-column moment end-plate connection if an effective column flange length is defined. Hendrick and Murry[7] conducted a limited series of tests to evaluate several European design methods for use with North American rolled sections. They concluded that the method proposed by Man and Morris[12] is the most suitable for the evaluation of unstiffened column flanges in the tension region of 4-bolt, unstiffened end-plate connections. They also modified the Krishnamurthy[10] results by introducing an effective column flange length equal to 3.5 times the vertical bolt pitch at the beam tension flange to obtain the same results as found with the Mann and Morris[12] equations. Finally, they developed a "rule of thumb," found in the AISC *Engineering for Steel Construction,*[3] which states that, under certain limitations, the column flange is adequate if its thickness is greater than the required bolt diameter from the Krishnamurthy end-plate design procedure.[11]

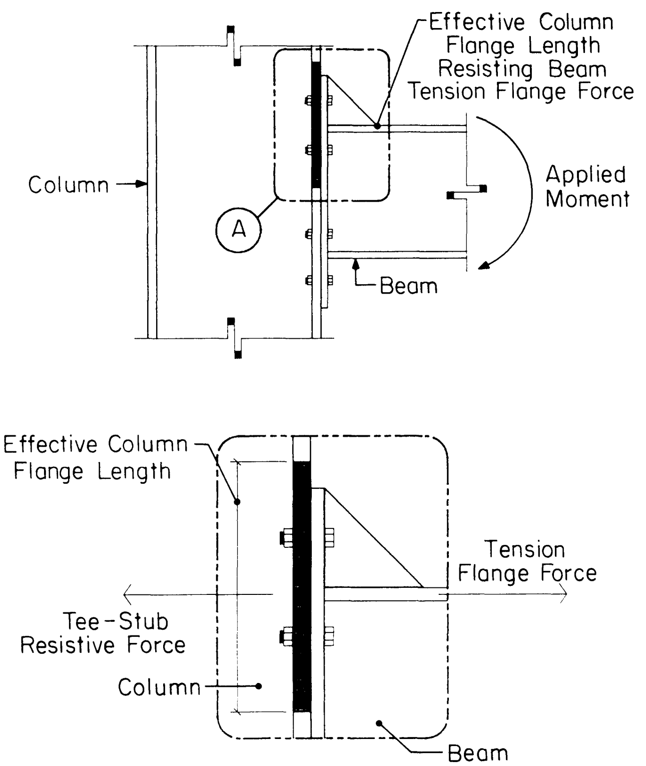

Fig. 3. Tee-Stub Analogy and Effective Length of Column Flange

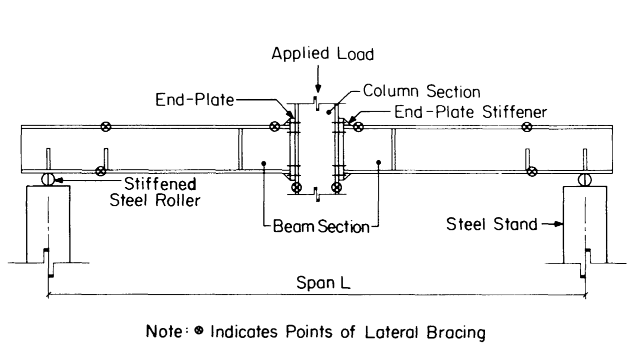

Fig. 4. Test Configuration

Curtis[5] conducted extensive analytical and experimental studies to determine column flange strength, connection stiffness, and bolt force predictions for the 4-bolt stiffened (Fig. 1) and the 8-bolt stiffened configuration (Fig. 2). Four 4-bolt stiffened and nine 8-bolt stiffened tests were conducted using the test setup shown in Fig. 4. Column, beam, and end-plate data are identified in Tables 1, 2, and 3. The specimens were instrumented and monitored for separation at the intersection of the planes of the beam tension flange and the beam/column webs, between the outside edges of the beam and column flanges and between column flanges. Bolt force measurements were also made. A typical result is shown in Fig. 5. The tests were stopped when either excessive flange deformation or high bolt forces occurred.

Ultimate load predictions were made for column flange strength and bolt strength. In addition, load predictions were made for connection stiffness at 0.015 in. plate separation.

Column flange strength predictions were calculated using the design procedure for unstiffened, extended end-plates found in the AISC manuals,[2,3] with several modifications. That is, the column flange was treated as an unstiffened end-plate having a width equal to the effective length of the

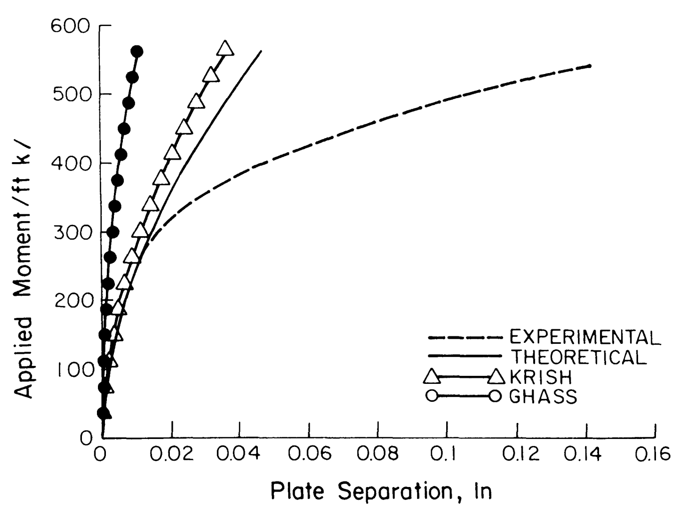

Fig. 5. Typical Test Results

column flange. The column flange flexural strength, M_{ef}, was determined from

$$M_{ef} = 1.67(0.75F_{yc})t_{fc}^2 b_s/6 \tag{1}$$

where t_{fc} = column flange thickness, b_s = effective column flange length, and F_{yc} = column flange material yield strength. The constant 1.67 represents the implied factor of safety in the AISC procedure. The column flange strength is related to the applied test moment, M_{beam}, as follows

$$F_f = \frac{4M_{ef}}{\alpha_m p_e} \tag{2}$$

where F_f = beam flange force, α_m = constant depending on connection geometry and material yield stresses, and p_e = effective bolt distance. And

$$M_{beam} = F_f(d - t_{fb}) \tag{3}$$

where d = beam depth and t_{fb} = beam flange thickness. In the AISC procedure

$$\alpha_m = C_a C_b (A_f/A_w)^{1/3}(p_e/d_b)^{1/4} \tag{4}$$

where C_a = constant depending on the yield stress of the beam and end-plate material and type of bolt, $C_b = (b_f/b_p)^{1/2}$, b_f = beam flange width, b_p = end-plate width, A_f = area of beam tension flange, A_w = area of beam web, d_b = the bolt diameter, and the effective bolt distance is given by

$$p_e = p_f - d_b/4 - w_t \tag{5}$$

where p_f = distance from centerline of the tension bolts to the nearer surface of the beam tension flange and w_t = fillet weld throat size or reinforcement of groove weld. Based on the recommendations of Hendrick and Murray,[7] the following modifications were made in the basic AISC procedures: $C_b = 1.0$, $A_f/A_w = 1.0$ and

$$p_e = p_f - d_b/4 - r_c \tag{6}$$

where, as shown in Figure 6, $p_f = (g - t_{wc})/2$, g = vertical bolt row gage, t_{wc} = column web thickness, and

$$r_c = k_1 - t_{wc}/2 - wt \tag{7}$$

where k_1 = the tabulated column section "k_1" distance and w_t = 1/16 in. The effective column flange thickness was taken as

$$b_s = 3.5c \tag{8}$$

for the four-bolt connection[7] and

$$b_s = 6\, p_b + 2c \tag{9}$$

for the 8-bolt connection where c = vertical distance between the bolt rows nearer the beam tension flange = $p_f + t_{fb} + p_f$, and p_b = vertical distance beam bolt rows away from the beam tension flange. The resulting predicted column flange strengths for the four 4-bolt and nine 8-bolt stiffened end-plate tests are found in Tables 4 and 5, respectively.

Ultimate loads for the bolt strength limit state were predicted by Curtis[5] based on the works of Krishnamurthy[10,11] and Ghassemieh.[16] Kirshnamurthy concluded that prying forces are negligible in a 4-bolt, stiffened end-plate configuration, thus the predicted failure moment is

$$M_{ub} = 4T_u(d - t_{fb}) \tag{10}$$

where T_u = the tensile strength of one bolt. Ghassemieh's finite element study of the 8-bolt stiffened end-plate config-

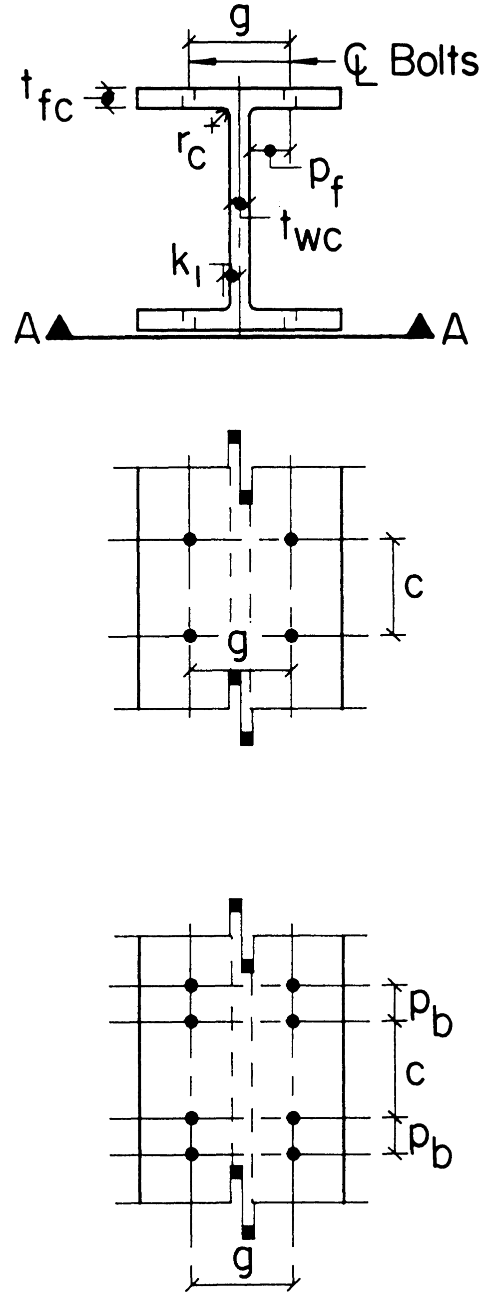

Fig. 6. Nomenclature at Column Tension Flange

Table 1. Four-Bolt, Stiffened Configuration Test Data						
				Column Properties		
Test Designation	Column Section	Beam Section	End-Plate	Flange Thickness t_{fc} (in.)	Web Thickness t_{wc} (in.)	Yield Stress F_{yc} (ksi)
FP5-14x74-4	W14x74	W27x114	EP5	0.777	0.461	44.87
FP7-10x49-4	W10x49	W27x114	EP5	0.531	0.330	40.79
FP9-14x61-4	W14x61	W27x114	EP5	0.613	0.401	35.96
FP11-14x99-4	W14x99	W24x100	EP4	0.803	0.555	35.58

uration resulted in a regression analyses equation relating beam flange force, F_f, to bolt force, T,

$$T = \frac{0.2305 \times 10^{-4}\, p_f^{0.591}\, F_f^{2.583}}{t_p^{0.885} d_b^{1.909} t_s^{0.327} b_p^{0.965}} + P_t \quad (11)$$

where t_p = end-plate thickness, t_s = stiffener thickness and P_t = bolt pretension force. The predicted connection capacity for the 8-bolt configuration is determined by calculating the ultimate beam flange, F_u, using Equation (11) with T equal to the tensile capacity of the bolts being used and substituting F_u in place of $4T_u$ in Equation (10). Results for both end-plate configurations are shown in Tables 4 and 5.

Since moment end-plate connections are generally used in Type I or FR construction, connection stiffness must also be investigated. To do this, Curtis[5] considered both the column and end-plate sides of the connection.

Figure 7 shows both sides of a column end-plate connection with elemental plate separations δ_c (column) and δ_{ep} (end-plate). Both of these deflections are maximum at the column web/beam flange intersection and

$$\delta_{ps} = \delta_c + \delta_{ep} \quad (12)$$

where δ_{ps} is the total connection plate separation.

To predict these elemental plate separations, regression equations formulated from finite element results were used. Krishnamurthy[11] presented the following relationship to predict 4-bolt stiffened and unstiffened end-plate deflections.

$$\delta_{ep} = \frac{2.667\delta_o (F/T_u)^{1.832} t_p^{0.894} b_e^{0.398}}{p_f^{0.666} d_b^{0.626} e^{[6.227(t_s/b_e)]}} \quad (13)$$

where e = 2.718 and

$$\delta_o = NT_u p_f^3/(2Eb_e t_p^3) \quad (14)$$

where N = number of tension bolts, b_e = end-plate width or effective column flange length and E = modulus of elasticity. (Note that the last term in the denominator is unity for the unstiffened case, e.g., t_s = 0.0) Ghassemieh[6] developed the following expresssion for the end-plate side of the 8-bolt configuration

$$\delta_{ep} = \frac{3.833 \times 10^{-7} p_f^{1.821} g^{1.204} F_f^{1.903}}{t_p^{2.087} d_b^{1.928} t_s^{0.233} b_p^{1.424}} \quad (15)$$

Curtis[5] evaluated his test results using Equations (13) or (15) for the end-plate side and Equation (13) for the column side utilizing

$$b_s = b_e = 1.625\, c \quad (16)$$

for 4-bolt configurations, and

$$b_s = b_e = c + 2.25\, p_b \quad (17)$$

for 8-bolt configurations and found reasonable correlation for separations as large as approximately 0.02 in. But he

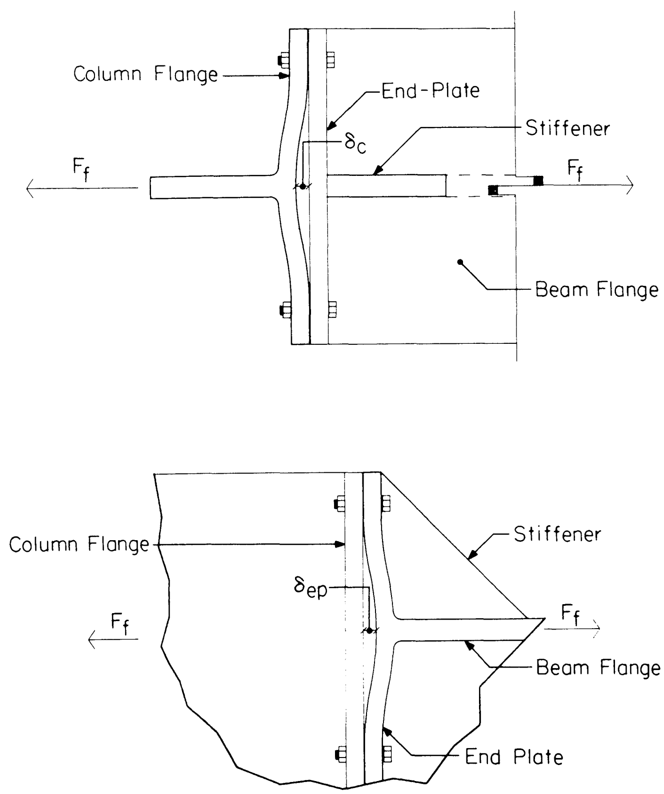

Fig. 7. Separation in Tension Region

Table 2. Eight-Bolt, Stiffened Configuration Test Data						
				Measured Column Properties		
Test Designation	Column Section	Beam Section	End-Plate	Flange Thickness t_{fc} (in.)	Web Thickness t_{wc} (in.)	Yield Stress F_{yc} (ksi)
FP4-14×61-8	W14×61	W27×114	EP5	0.636	0.404	44.57
FP8-10×49-8	W10×49	W27×114	EP5	0.530	0.321	40.91
CF4-U12×87	W12×87	W24×100	EP4	0.815	0.515	46.77
CF4-U12×106	W12×106	W24×100	EP4	0.993	0.615	38.50
CF4-U12×120	W12×120	W24×100	EP4	1.181	0.740	52.50
CF5-U10×68	W10×68	W27×114	EP5	0.781	0.494	40.61
CF6-U12×96	W12×96	W27×114	EP6	0.872	0.560	36.65
CF6-U14×158	W14×158	W27×114	EP6	1.180	0.725	37.73
CF8-U14×159	W14×159	W27×114	EP8	1.215	0.743	35.02

recommended that total separation, δ_{ps}, be limited to 0.015 in. if the connection is used in Type I or FR construction.

Because the use of Equations 12–15 to evaluate connection stiffness is rather complicated, a further evaluation was undertaken. It was found that, if the effective column flange length is taken as

$$b_s = 2.5c \tag{18}$$

for 4-bolt connections and

$$b_s = 3.5p_b + c \tag{19}$$

for 8-bolt connections, and the AISC manual[2,3] procedure is used to determine the required column flange thickness, adequate stiffness is provided. Comparisons of the limiting moments from these criteria and measured moments at 0.015-in. separation are shown in Tables 4 and 5.

RECOMMENDED DESIGN PROCEDURE

Based on the above studies, the recommended design procedure for determining required unstiffened column flange thickness at 4-bolt, and 8-bolt, stiffened, moment end-plate connections is as follows. Figs. 8 and 9 identify the various geometric parameters.

To prevent column flange yielding in the tension region of the connection and to provide adequate connection stiffness, the following must be satisfied assuming A36 material even if the column material yield stress is higher:

$$t_{fc} \geq t_{fr} \tag{20}$$

where t_{fc} = column flange thickness, in. and t_{fr} = required column flange thickness, in. The required column flange thickness is determined from:

Table 3. End-Plate Data						
End-Plate Designation	t_p (in.)	P_f (in.)	d_b (in.)	t_s (in.)	b_p (in.)	g (in.)
EP4	1	1.375	0.875	0.625	12	5.5
EP5	1	1.5	1	0.625	10	5.5
EP6	1.25	2	1	0.625	10	6.5
EP8	1.75	2.125	1.125	0.75	13	6.5

Allowable Stress Design

$$t_{fr} = \sqrt{(6M_e)/(F_{bc}\,b_s)} \tag{21}$$

with

F_{bc} = 0.75 × 36 = 27 ksi

b_s = 2.5c for 4-bolt connections (Fig. 8)

= $3.5p_b + c$ for 8-bolt connections (Fig. 9)

$M_e = \alpha_m F_f p_e/4$

$\alpha_m = 1.13(p_e/d_b)^{1/4}$ (A325 bolts)

$F_f = M/(d - t_{fb})$

And, from Equations 6 and 7, neglecting w_t,

$P_e = g/2 - d_b/4 - k_1$

Load and Resistance Factor Design

$$t_{fr} = \sqrt{(4M_{eu})/(0.9F_{yc}\,b_s)} \tag{22}$$

with

F_{yc} = 36 ksi

b_s = 2.5c for 4-bolt connections (Fig. 8)

= $3.5p_b + c$ for 8-bolt connections (Fig. 9)

$M_{eu} = \alpha_{mu} F_{fu} p_e/4$

$\alpha_{mu} = 1.35\ (p_e/d_b)^{1/4}$

$F_{fu} = M_u/(d - t_{fb})$

And, from Equations 6 and 7, neglecting w_t,

$p_e = g/2 - d_b/4 - k_1$

ASD EXAMPLES

ASD Example 1

For the 4-bolt stiffened end-plate connection shown in Fig. 10, determine if the column flange is adequate. M = 200

Table 4. Experimental Versus Theoretical Results for Four-Bolt Configurations						
	Predicted Ultimate Moments (ft-kips)					
Designation	Maximum Applied Moment (ft-kips)	Column Flange Strength (M_{uc})	Bolt Strength 2×Allow (M_{ub})	Measured Moment @0.015″	Predicted Moment @0.015″	Failure Mode
FP5-14×74-4	405	406	608	240	290	High bolt forces and excessive deformation
FP7-10×49-4	225	147	608	120	105	Excessive column flange deformation
FP9-14×61-4	293	197	608	127	141	Excessive column flange deformation
FP11-14×99-4	375	246	411	175	176	Excessive column flange deformation

ft-kips. A36 steel. The end-plate and bolts have been previously designed.

Beam W24×55:

$d = 23.57$ in.

$t_{fb} = 0.505$ in.

Column W14×159:

$t_{fc} = 1.190$ in.

$k_1 = 1$ in.

End-Plate:

$p_f = 1\frac{3}{8}$ in.

$g = 5\frac{1}{2}$ in.

A325 Bolts:

$\frac{7}{8}$ in. diameter

Beam Flange Force:

$$F_f = M/(d - t_{fb}) = (200 \times 12)/(23.57 - 0.505) = 104.1 \text{ kips}$$

Effective Column Flange Length:

$$b_s = 2.5c = 2.5(p_f + t_{fb} + p_f) = 2.5(1.375 + 0.505 + 1.375) = 8.14 \text{ in.}$$

Column Flange Moment:

$$p_e = g/2 - d_b/4 - k_1 = 5.5/2 - 0.875/4 - 1.0 = 1.53 \text{ in.}$$

$$\alpha_m = 1.13(p_e/d_b)^{1/4} = 1.13(1.53/0.875)^{1/4} = 1.30$$

$$M_e = \alpha_m F_f p_e/4 = 1.30(104.1)(1.53)/4 = 51.76 \text{ in.-kips}$$

Required Column Flange Thickness:

$$t_{fr} = \sqrt{6M_e/27b_s} = \sqrt{(6\times51.76)/(27\times8.14)} = 1.19 \text{ in.} \leq 1.190 \text{ in.}$$

The column flange is adequate.

ASD Example 2

For the 8-bolt stiffened end-plate connection shown in Fig. 11, determine if the column flange is adequate M = 700 ft-kips. A36 steel. The end-plate and bolts have been previously designed using the procedure found in Murray and Kukreti.[14]

Beam W33×118:

$d = 32.86$ in.

$t_{fb} = 0.740$ in.

Column W14×311:

$t_{fc} = 2.260$ in.

$k_1 = 1\frac{5}{16}$ in.

End-Plate:

$p_f = 1\frac{5}{8}$ in.

$p_b = 3\frac{3}{8}$ in.

$g = 6$ in.

A325 Bolts:

$1\frac{1}{8}$ in. diameter

Beam Flange Force:

$$F_f = M/(d - t_{fb}) = (700 \times 12)/(32.86 - 0.740) = 261.5 \text{ kips}$$

Effective Column Flange Length:

$$b_s = 3.5p_b + c = 3.5p_b + (p_f + t_{fb} + p_f) = 3.5(3.375) + (1.625+0.740+1.625) = 15.80 \text{ in.}$$

Column Flange Moment:

$$p_e = g/2 - d_b/4 - k_1 = 6.0/2 - 1.125/4 - 1.3125 = 1.41 \text{ in.}$$

$$\alpha_m = 1.13(p_e/d_b)^{1/4} = 1.13(1.41/1.125)^{1/4} = 1.20$$

$$M_e = \alpha_m F_f p_e/4 = 1.20(261.5)(1.41)/4 = 110.6 \text{ kips}$$

Required Column Flange Thickness:

$$t_{fr} = \sqrt{6M_e/27b_s} = \sqrt{(6 \times 110.6)/(15.80 \times 27)} = 1.247 \text{ in.} < t_{fc} = 2.260 \text{ in.}$$

The column flange is adequate.

LRFD EXAMPLES

LRFD Example 1

For the 4-bolt unstiffened end-plate connection shown in Figure 10, determine if the column flange is adequate, M_u = 260 ft-kips. A36 steel. The end-plate and bolt have been previously designed.

Table 5.
Experimental Versus Theoretical Results for Eight-Bolt Configurations

	Predicted Ultimate Moments (ft-kips)					
Designation	**Maximum Applied Moment (ft-kips)**	**Column Flange Strength (M_{uc})**	**Bolt Strength 2×Allow (M_{ub})**	**Measured Moment @0.015″**	**Predicted Moment @0.015″**	**Failure Mode**
FP4-14×61-8	450	468	856	260	261	Excessive column flange deformation
FP8-10×49-8	315	246	856	170	137	Excessive column flange deformation
CF4-U12×87	585	622	675	360	348	Excessive column flange deformation
CF4-U12×106	850	736	675	388	409	Excessive column flange deformation
CF4-U12×120	830	1705	675	N/A	953	Bolt yield
CF5-U10×68	630	556	856	294	309	Excessive column flange deformation
CF6-U12×96	630	569	864	350	315	Excessive column flange deformation
CF6-U14×158	990	1044	864	520	575	Excessive column flange deformation
CF8-U14×159	1320	1203	1315	550	665	Excessive column flange deformation

Beam W24×55:

d = 23.57 in.

t_{fb} = 0.505 in.

End-Plate:

p_f = 1⅜ in.

g = 5½ in.

Column W14×159:

t_{fc} = 1.190 in.

k_1 = 1 in.

A325 Bolts:

⅞ in. diameter

Factored Beam Flange Force:

$$F_{fu} = M_u/(d - t_{fb})$$
$$= (260 \times 12)/(23.57 - 0.505) = 135.3 \text{ kips}$$

Effective Column Flange Length:

$$b_s = 2.5c = 2.5(p_f + t_{fb} + p_f)$$
$$= 2.5(1.375 + 0.505 + 1.375) = 8.14 \text{ in.}$$

Column Flange Moment:

$$p_e = g/2 - d_b/4 - k_1$$
$$= 5.5/2 - 0.875/4 - 1.0 = 1.53 \text{ in.}$$

$$\alpha_{mu} = 1.36(p_e/d_b)^{1/4}$$
$$= 1.35(1.53/0.875)^{1/4} = 1.55$$

$$M_{eu} = \alpha_{mu} F_{fu} p_e / 4$$
$$= 1.55(135.3)(1.53)/4 = 80.22 \text{ in.-kips}$$

Required Column Flange Thickness:

$$t_{fr} = \sqrt{4M_{eu}/\phi F_{yc} b_s} = \sqrt{(4\times 80.22)/(0.9\times 36\times 8.14)}$$
$$= 1.103 \text{ in.} \leq 1.190 \text{ in.}$$

The column flange is adequate.

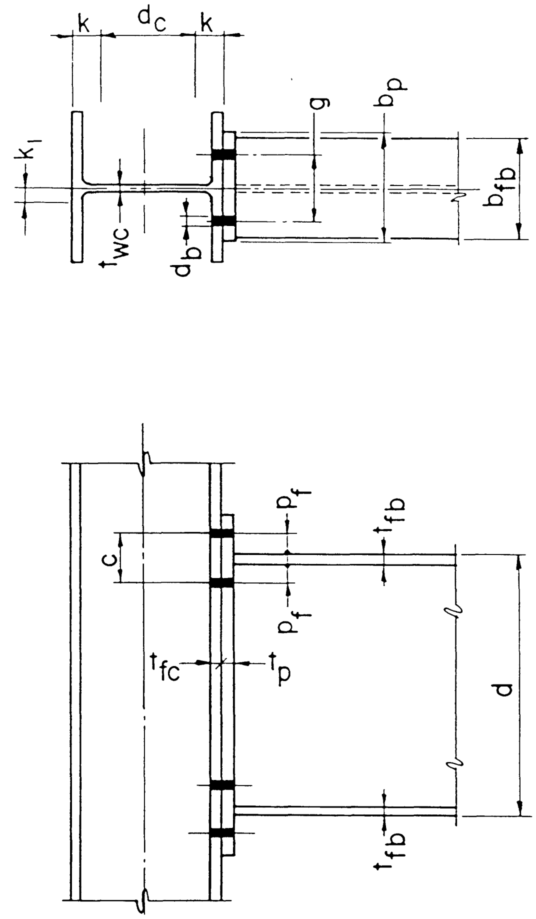

Fig. 8. 4-Bolt, Stiffened Moment End-Plate Connection Geometry

LRFD Example 2

For the 8-bolt stiffened end-plate connection shown in Fig. 11, determine if the column flange is adequate. M_u = 1050 ft-kips. A36 steel. The end-plate and bolts have been previously designed using the procedure found in Murray and Kukreti.[14]

Beam W33×118:

d = 32.86 in.
t_{fb} = 0.740 in.

End-Plate:

$p_f = 1\frac{5}{8}$ in.

Column W14×311:

t_{fc} = 2.260 in.
$k_1 = 1\frac{5}{16}$ in.

A325 Bolts:

$1\frac{1}{8}$ in. diameter

$p_b = 3\frac{3}{8}$ in.
g = 6 in.

Factored Beam Flange Force:

$$F_{fu} = M_u/(d - t_{fb})$$
$$= (1050 \times 12)/(32.86 - 0.740) = 392.3 \text{ kips}$$

Effective Column Flange Length:

$$b_s = 3.5p_b + c = 3.5p_b + (p_f + t_{fb} + p_f)$$
$$= 3.5(3.375) + (1.625+0.740+1.625) = 15.80 \text{ in.}$$

Column Flange Moment:

$$p_e = g/2 - d_b/4 - k_1$$
$$= 6.0/2 - 1.125/4 - 1.3125 = 1.41 \text{ in.}$$

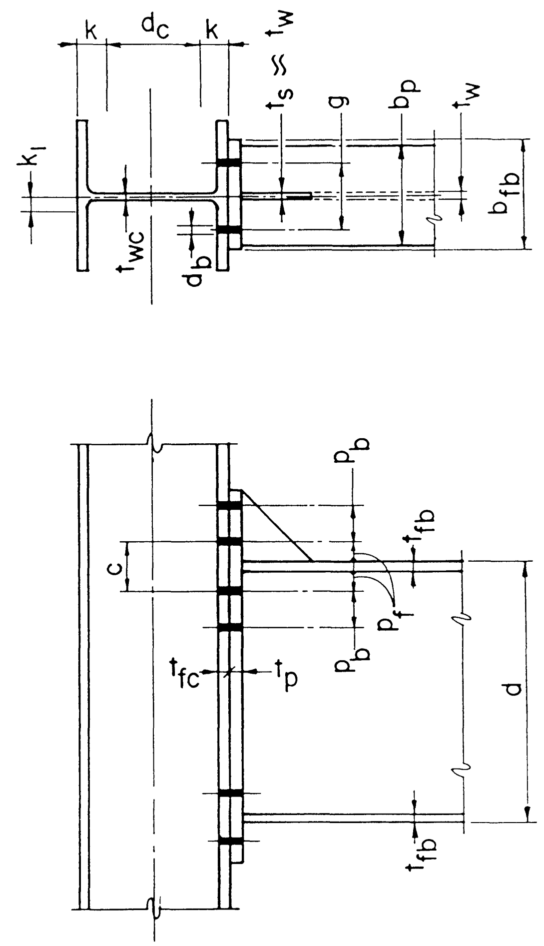

Fig. 9. 8-Bolt, Stiffened Moment End-Plate Connection Geometry

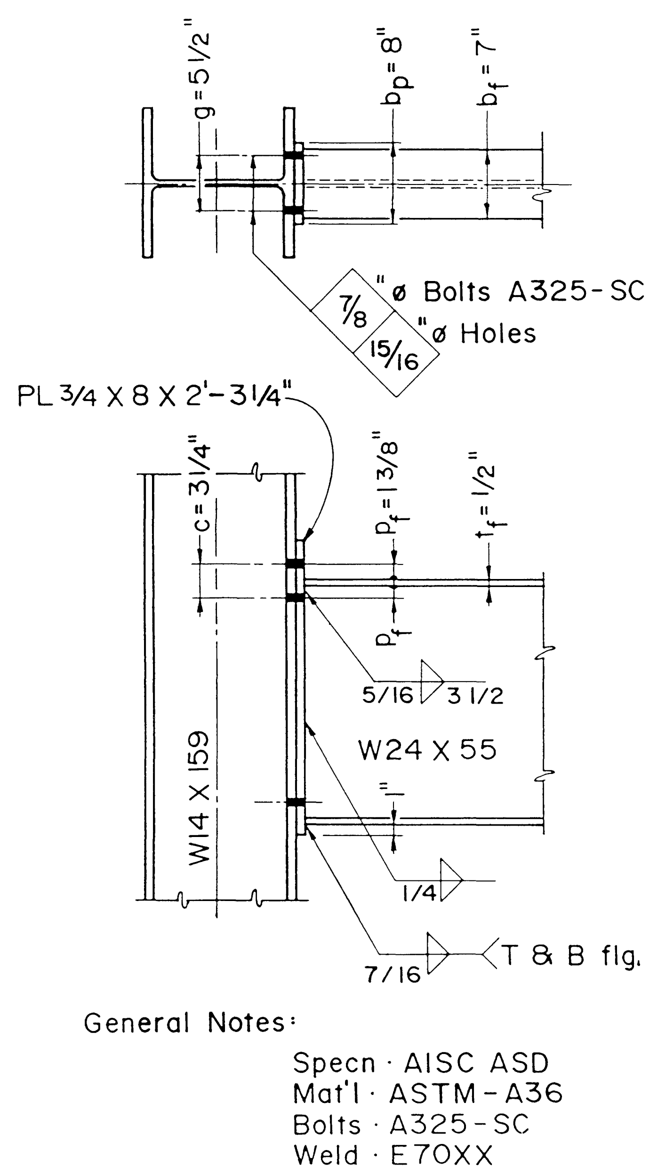

Fig. 10. Design Details, ASD Example 1 and LRFD Example 1

$$\begin{aligned} \alpha_m &= 1.35(p_e/d_b)^{1/4} \\ &= 1.35(1.41/1.125)^{1/4} = 1.43 \end{aligned}$$

$$\begin{aligned} M_{eu} &= \alpha_m F_{fu} p_e/4 \\ &= 1.43(392.3)(1.41)/4 = 197.7 \text{ in.-kips} \end{aligned}$$

Required Column Flange Thickness:

$$\begin{aligned} t_{fr} &= \sqrt{4M_{eu}/(0.9F_y b_s)} = \sqrt{(4\times197.7)/(0.9\times36\times15.80)} \\ &= 1.243 \text{ in.} < t_{fc} = 2.260 \text{ in.} \end{aligned}$$

The column flange is adequate.

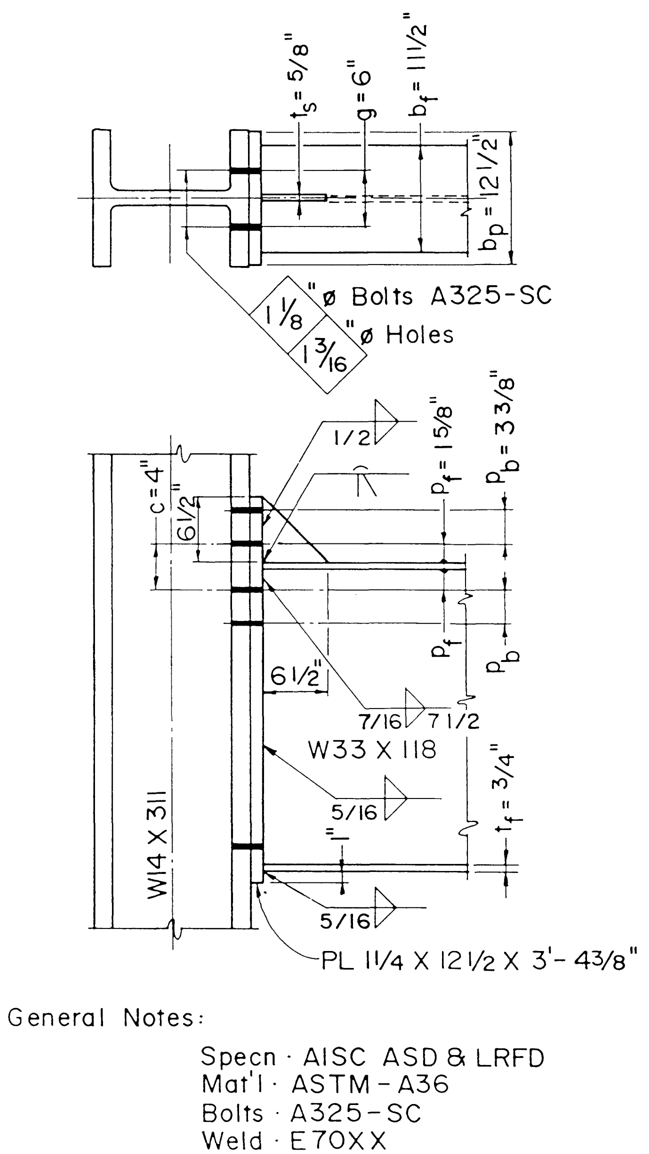

Fig. 11. Design Details, ASD Example 2 and LRFD Example 2

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NOMENCLATURE

A_f = area of beam in tension flange, in.^{2}
A_w = area of beam web, in.^{2}
b_f = beam flange width, in.
b_p = end-plate width, in.
b_e = end-plate width, in.; effective column flange length, in.
b_s = effective column flange length, in.
c = vertical distance between the bolt rows nearer the beam tension flange, in.
C_a = constant depending on the yield stress of the beam and end-plate materials and type of bolt.
C_b = $(b_f/b_p)^{1/2}$
d = beam depth, in.
d_b = bolt diameter, in.
e = 2.718
E = modulus of elasticity, ksi
F = flange force per bolt, kips
F_{bc} = allowable bending stress in column flange, 27 ksi
F_f = beam flange force, kips
F_{fu} = factored beam flange force, kips
F_u = ultimate beam flange force, kips
F_{yc} = column flange material yield strength, ksi
g = vertical bolt row gage, in.
k_1 = tabulated column section "k_1" distance
M = moment at connection, ft-kips
M_{beam} = applied test moment, in.-kips
M_e = ASD moment in column flange, in.-kips
M_{eu} = LRFD moment in column flange, in.-kips
M_{ef} = column flange flexural strength, in.-kips
M_u = factored moment at connection, ft-kips
M_{ub} = predicted failure moment for bolt strength limit state, in.-kips
M_{uc} = predicted failure moment for column flange bending strength limit state, in.-kips
N = number of tension bolts
p_b = vertical distance between beam bolt rows away from the beam tension flange, in.
p_e = effective bolt distance, in.
p_f = distance from centerline of the tension bolts to the nearer surface of the beam tension flange, in.
P_t = bolt pretension force, kips
r_c = from Equation 7
t_{fb} = beam flange thickness, in.
t_{fc} = column flange thickness. in.
t_{fr} = required column flange thickness, in.
t_p = end-plate thickness, in.
t_s = stiffener thickness, in.
t_{wc} = column web thickness, in.
T = bolt force including prying action effects, kips
T_u = tensile strength of one bolt, kips
w_t = fillet weld throat size or reinforcement of groove weld, in.
α_m = ASD constant depending on connection geometry and material yield stresses
α_{mu} = LRFD constant depending on connection geometry and material yield stresses
δ_c = separation due to column flange bending, in.
δ_{ep} = separation due to end-plate bending, in.
δ_{ps} = total separation, in.
δ_o = from Equation 13