S.E. Exam Review:
Steel Design

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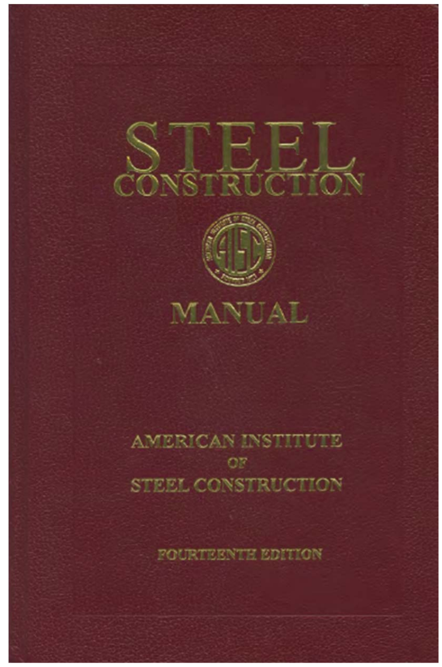
ASCE | KNOWLEDGE & LEARNING

Table of Contents

SCM – Steel Construction Manual

- Fourteenth Edition
- ASCE 360 – Specification for Structural Steel Buildings



NCEES Topics

- Steel Construction Manual
- LRFD
- Beams
- Columns
- Members subjected to combined forces
- Column base plates
- Beam bracing
- Connections

Design for Stability

Chapter C – Design for Stability

Chapter C presents requirements for design for stability

Primary method is the *Direct Analysis Method* of design

Advantage is use $K = 1$

Design → determination of required strength of components and proportioning the components to have adequate available strength.

Chapter C – Design for Stability

Appendix 7 presents alternate methods:

a) effective length method

b) first order analysis method

Appendix 8 presents methods of approximate second order analysis.

Unless noted otherwise, all loads in this presentation are required loads computed using one of the approved methods in Chapter C.

Chapter C – Design for Stability

C1 – General Stability Requirements

C2 – Calculation of Required Strengths

C3 – Calculation of Available Strengths

Direct Analysis Method

Section C1 – General Stability Requirements

Stability shall be provided for the structure as a whole and each of its elements

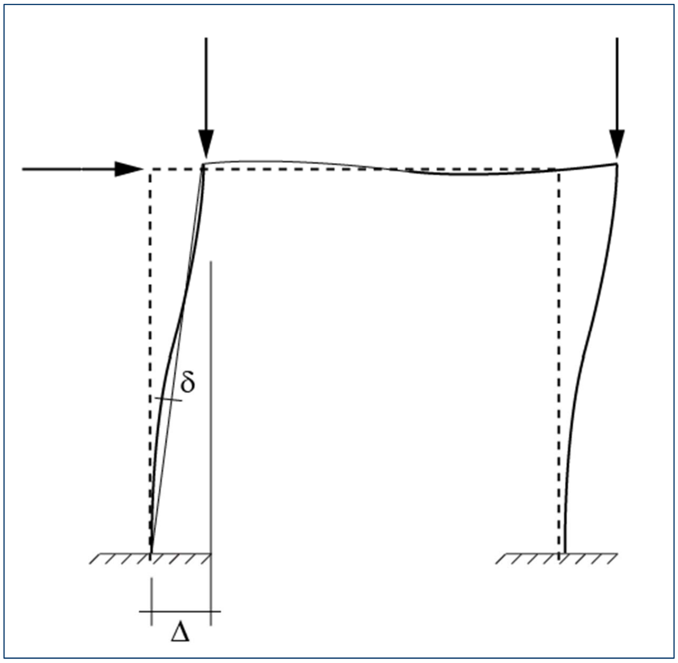
Flexural, shear and axial member deformations and all other deformations that contribute to the displacement of the structure must be *considered*

Second-order effects (both P-Δ and P-δ)

Geometric imperfections

Stiffness reduction due to inelasticity

Second-order Effects



Section C1 – General Stability Requirements

Any rational method that considers these effects is permitted

All load-dependent effects are calculated using LRFD load combinations or 1.6 times ASD load combinations

Uncertainty in strength and stiffness

For strength uncertainty, resistance factor (ϕ) or safety factor (Ω)

Section C2 – Calculation of Required Strengths

Consider flexural, shear and axial member deformations, and all other component and *connection* deformations that contribute to the displacement of the structure

Use any *second-order* analysis that considers *P-Δ* and *P-δ* effects

P-δ effects may be ignored under certain circumstances (See Section C2.1(2))

Section C2 – Calculation of Required Strengths

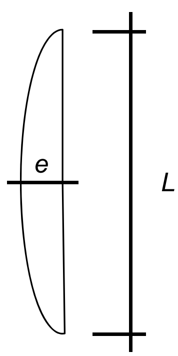
Include all gravity and other applied loads that influence the stability of the structure

Load-dependent effects are calculated using the LRFD load combinations or 1.6 times ASD load combinations

If using ASD, divide analysis results by 1.6 to obtain required strengths

Consideration of Initial Imperfections

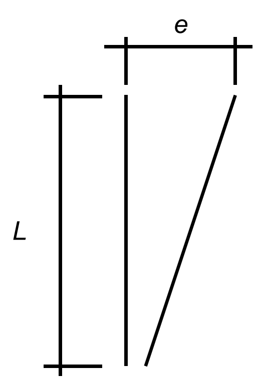
Initial Imperfections



Out-of-straightness

ASTM A6 tolerance L / 1000

Chapter E



Out-of-plumbness

Code of Standard Practice tolerance L / 500

Initial Imperfections

C2.2 Consideration of Initial Imperfections

"The effect of initial imperfections on the stability of the structure shall be taken into account either by direct modeling of imperfections in the analysis as specified in Section 2.2a or by application of notional loads as specified in Section 2.2b"

C2.2a - Direct modeling

C2.2b - Notional loads

Use of Notional Loads to Represent Imperfections

Permitted for structures that support gravity loads primarily through nominally-vertical columns, walls or frames

Use nominal geometry

Distributed in same manner as gravity load

Applied in the direction that provides the greatest destabilizing effect

Use of Notional Loads to Represent Imperfections

$N_i = 0.002\alpha Y_i$

$N_i \rightarrow$ notional load applied at level *i*

$Y_i \rightarrow$ gravity load applied at level *i* from LRFD load combination or ASD load combination

$\alpha = 1.0$ (LRFD); $\alpha = 1.6$ (ASD)

If $\frac{\Delta_{2nd-order}}{\Delta_{1st-order}}(B_2) \leq 1.7$ in all stories, notional loads may be applied only in gravity load combinations and not in combinations that include other lateral loads.

Notional Loads

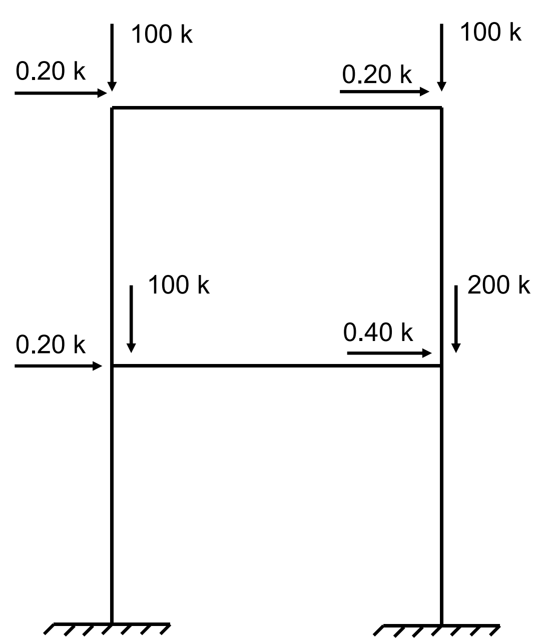
This is a gravity only LC

$N_1 = 0.002(1.0)(100\text{ k} + 200\text{ k})$

$= 0.6\text{ k}$

$N_2 = 0.002(1.0)(100\text{ k} + 100\text{ k})$

$= 0.4\text{ k}$



Adjustment to Stiffness

For stiffnesses that contribute to stability of the structure

Optionally applicable to all members

Direct adjustment – C2.3(1), (2)

Notional loads – C2.3(3)

Notional loads combine with notional loads from imperfections

Stiffness Reduction – C2.3(1), (2)

Direct adjustment

$EA^* = 0.8EA$

$EI^* = 0.8E\tau_b I$

$\alpha P_r/P_y \le 0.5 \quad \tau_b = 1.0$

$\alpha P_r/P_y > 0.5 \quad \tau_b = 4(\alpha P_r/P_y)[1 - (\alpha P_r/P_y)]$

$\alpha = 1.0$ (LRFD); $\alpha = 1.6$ (ASD)

$P_r \rightarrow$ required axial compressive strength using LRFD or ASD load combinations

$P_y \rightarrow$ axial yield strength (= $F_y A_g$)

Stiffness Reduction – C2.3(3)

Gravity loads supported primarily by nominally-vertical columns, walls or frames

Notional loads

For $\frac{\alpha P_r}{P_y} > 0.5$ may use $\tau_b = 1$

If notional load $= 0.001\alpha Y_i$ is applied at all levels for all load combinations

Y_i → gravity load applied at level *i* from LRFD load combination or ASD load combination

Section C3 – Calculation of Available Strengths

Follow provisions of Chapters *D* through *K* with no further consideration of overall structural stability.

K = 1 unless smaller value can be justified.

Bracing requirements of Appendix 6 are not applicable to bracing that is included as part of the overall force resisting system.

Chapters *D* through *K*

Tension Members

Tension Members – Chapter D

Yield limit state

Fracture limit state

- Net area
- Effective net area

Block shear rupture limit state

Block Shear Rupture Strength – Chapter J

ASCE 360 Section J4.3 Page 16.1-129

Available (design) strength $= \phi R_n$

$R_n = 0.6F_u A_{nt} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$

$\phi = 0.75$

U_{bs} = 1 when the tensile stress is uniform

U_{bs} = 0.5 when the tensile stress is non-uniform

A_{gv} = gross area subject to shear, in^2

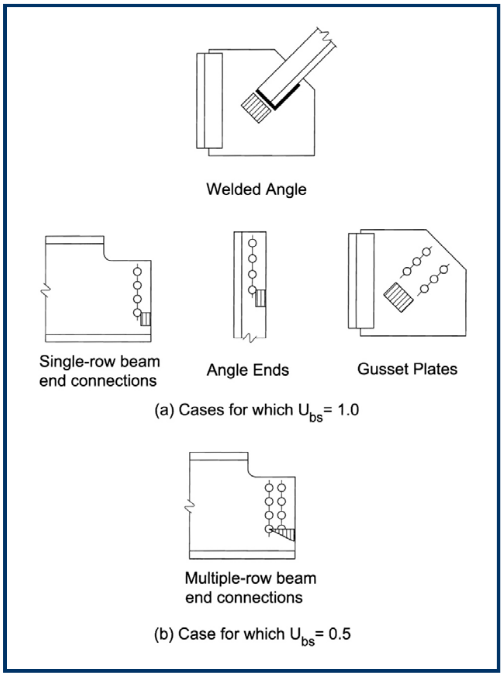
A_{gt} = gross area subject to tension, in^2

A_{nv} = net area subject to shear, in^2

A_{nt} = net area subject to tension, in^2

U_{bs}

ASCE 360 Commentary
Figure C-J4.2
Page 16.1-412

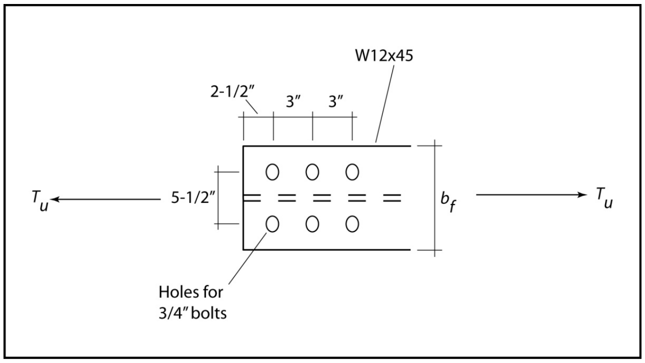


Block Shear Rupture Example

Determine if a W12x45 of A992 steel is adequate for the following loads: D = 90 *k*, L = 130 *k*, W = 145 *k.*

The member is connected to gusset plates as shown.

Use 3/4 inch bolts – 4 rows, 3 bolts each row.



Block Shear Rupture Example

Determine the maximum load effect

$1.4D = 1.4(90) = 126\ \text{k}$

$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) =$

$1.2(90\ \text{k}) + 1.6(130\ \text{k}) = 316\ \text{k}$

$1.2D + 1.0W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$

$= 1.2(90\ \text{k}) + 1.0(145\ \text{k}) + 0.5(130\ \text{k}) =$

318 k ← Controls

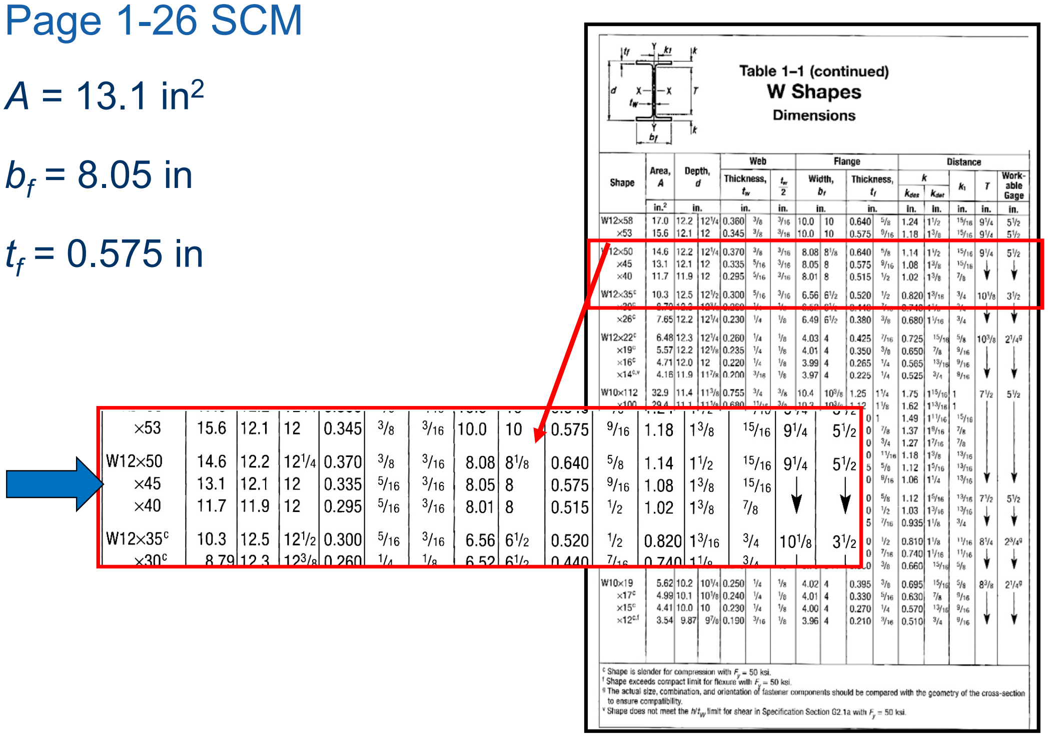
See footnote 1, page 2-10 of SCM for more information on the load factor to use with *L*.

Page 1-26 SCM

A = 13.1 in^2

b_f = 8.05 in

t_f = 0.575 in



Shape	Area, A (in.²)	Depth, d (in.)		Web Thickness, t_w (in.)		$\frac{t_w}{2}$ (in.)	Flange Width, b_f (in.)		Flange Thickness, t_f (in.)		k_{des} (in.)	k_{det} (in.)	k_1 (in.)	T (in.)	Workable Gage (in.)
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8	↓	↓
W12×35[c]	10.3	12.5	12 1/2	0.300	5/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3/4	10 1/8	3 1/2
×30[c]	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	3/4		

Block Shear Rupture Example

F_y = 50 ksi

F_u = 65 ksi

Yield limit state: $\phi_t T_n = \phi_t A_g F_y = (0.9)(13.1\ \text{in}^2)(50\ \text{ksi}) = 590 > 318\ \text{k Ok}$

$A_e = UA_n$

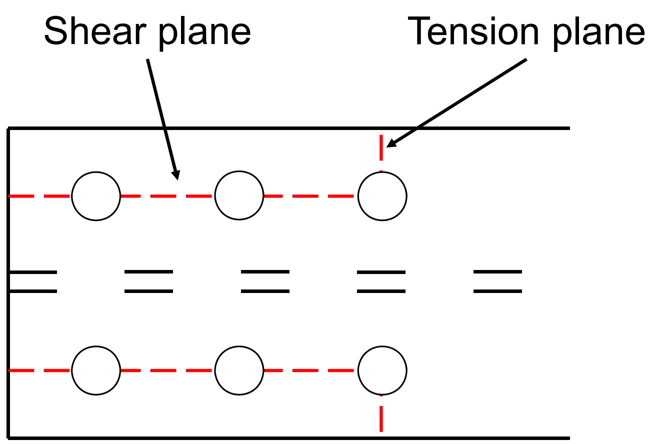
$A_n = A_g - 4d_{beff}t_f = 13.1\ \text{in}^2 - 4\left(\frac{3}{4}\ \text{in} + \frac{1}{8}\ \text{in}\right)(0.575\ \text{in}) = 11.09\ \text{in}^2$

U = 1

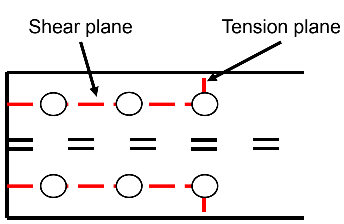
A_e = 11.09 in^2

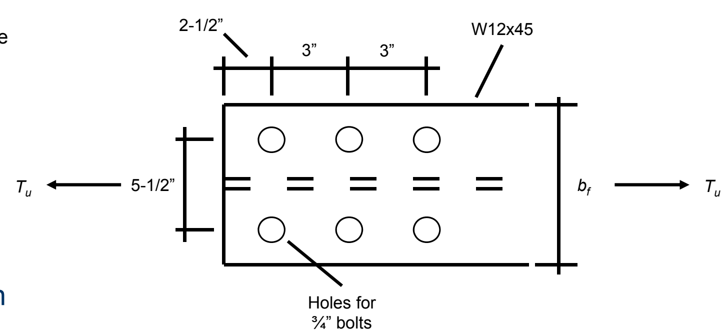
Fracture limit state: $\phi_t T_n = \phi_t A_e F_u = (0.75)(11.09\ \text{in}^2)(65\ \text{ksi}) = 541\ \text{k} > 318\ \text{k OK}$

Block Shear Rupture Example



Block Shear Rupture Example





For one shear rupture path

$$A_{gt} = \left(\frac{8.05\text{ in} - 5.5\text{ in}}{2}\right)(0.575\text{ in})$$

$$= 0.7331\text{ in}^2$$

$$A_{gv} = (2.5\text{ in} + 3\text{ in} + 3\text{ in})(0.575\text{ in}) = 4.887\text{ in}^2$$

$$A_{nt} = 0.7331\text{ in}^2 - (0.5)\left(0.75\text{ in} + \frac{1}{8}\text{ in}\right)(0.575\text{ in}) = 0.4815\text{ in}^2$$

$$A_{nv} = 4.887\text{ in}^2 - (2.5)\left(0.75\text{ in} + \frac{1}{8}\text{ in}\right)(0.575\text{ in}) = 3.629\text{ in}^2$$

Block Shear Rupture Example

U_{bs} = 1 (see Commentary Figure C-J4.2 Page 16.1-412)

There are four BSR planes; calculate for one plane and multiply by four:

$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

$0.6F_uA_{nv} + U_{bs}F_uA_{nt} = 0.6(65\text{ ksi})(3.629\text{ in}^2) + (1)(65\text{ ksi})(0.4815\text{ in}^2) = 172.8\text{ k controls}$

$0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(50\text{ ksi})(4.887\text{ in}^2) + (1)(65\text{ ksi})(0.4815\text{ in}^2) = 177.9\text{ k}$

$\phi_tR_n = 0.75(4)(172.8\text{ k}) = 518.4\text{ k} > 318\text{ k OK}$

ASD/LRFD Comparison

ASD/LRFD Comparison

Compare for the limit state of tension yield

Nominal strength (P_n) calculation is identical for both methods

LRFD

$\phi_t P_n = 0.9 P_n$

ASD

$\frac{P_n}{\Omega_t} = \frac{P_n}{1.67}$

$\frac{1}{\Omega_t} \rightarrow \phi_t$

Assume an average LRFD load factor of 1.5

$\frac{1.5}{1.67} = 0.898 \approx 0.9 = \phi_t$

Members Subjected to Combined Forces and Torsion – Chapter H

Combined Forces Notation

$P_r \rightarrow$ required axial strength

$P_n \rightarrow$ nominal axial strength

$P_c \rightarrow$ design axial strength ($= \phi_c P_n$)

$\phi_c \rightarrow$resistance factor for compression (= 0.90)

$M_r \rightarrow$ required flexural strength

$M_n \rightarrow$ nominal flexural strength

$M_c \rightarrow$ design flexural strength ($= \phi_b M_n$)

$\phi_b \rightarrow$ resistance factor for flexure (= 0.90)

$x \rightarrow$ subscript relating symbol for strong axis of bending

$y \rightarrow$ subscript relating symbol for weak axis of bending

LRFD Combined Forces Provisions

$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$ when $\frac{P_r}{P_c} \geq 0.2$ (ASCE 360 H1-1a)

$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$ when $\frac{P_r}{P_c} < 0.2$ (ASCE 360 H1-1b)

Equation H1-1b controls for members with relatively small axial load

LRFD Combined Forces Provisions

Inequalities H1-1a and H1-1b may be written, respectively:

$$pP_r + b_x M_{rx} + b_y M_{ry} \leq 1.0$$

$$\frac{1}{2}pP_r + \frac{9}{8}\left(b_x M_{rx} + b_y M_{ry}\right) \leq 1.0$$

where:

$$p = \frac{1}{\phi_c P_n}$$

$$b_x = \frac{8}{9(\phi_b M_{nx})}$$

$$b_y = \frac{8}{9(\phi_b M_{ny})}$$

Values of *p*, b_x and b_y are tabulated in SCM Table 6-1, Pages 6-5

Table 6-1
Caveats:
- F_y = 50 ksi
- C_b = 1
- Column buckling about the *y* axis

LRFD Combined Forces Provisions

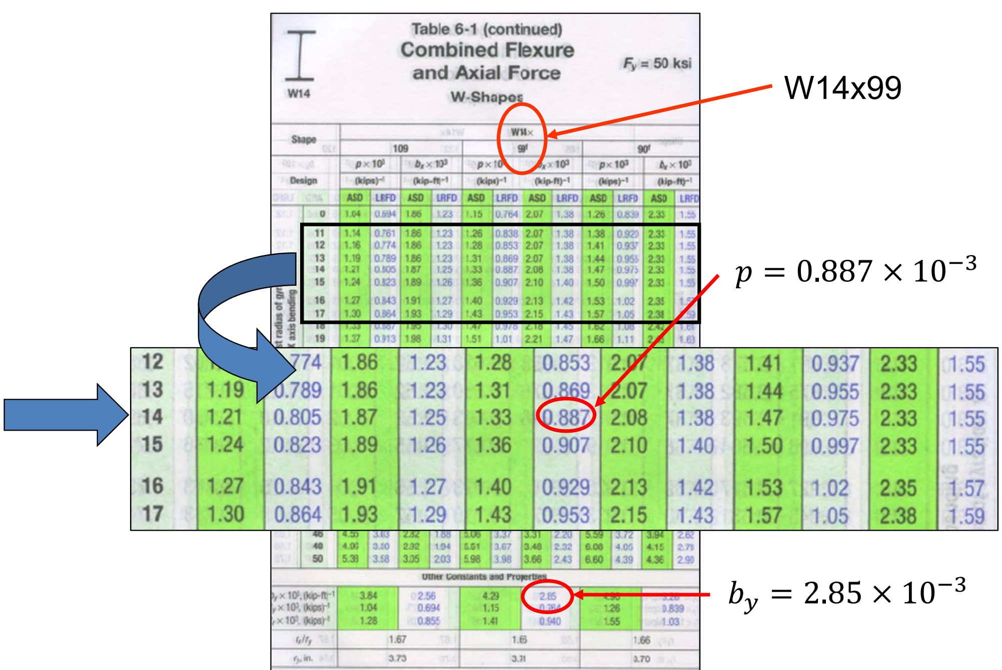
Confirm that y-axis buckling controls. If *x* axis buckling controls, enter Table 6-1 with $(KL)_{y-equiv} = (KL)_x/\left(r_x/r_y\right)$.

If C_b > 1, adjust the tabular value of b_x by dividing by C_b. The adjusted value of b_x may not be less than that corresponding to $\phi_b M_{nx} = \phi_b M_{px}$, or the minimum value of b_x found in Table 6-1. Using C_b = 1 is conservative.

Combined Forces Example 1

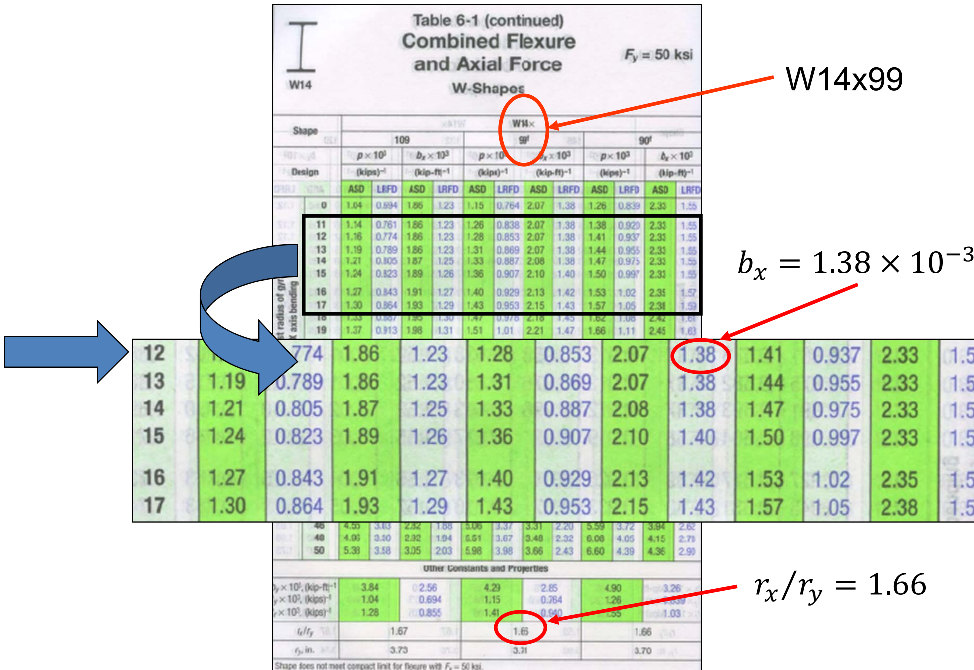
Determine if a W14x99 section of A992 steel is adequate to resist the following factored loads: P_r = 400 *k*, M_{rx} = 250 *k-ft*, M_{ry} = 80 *k-ft*. Use $(KL)_x$ = 20 *ft*, $(KL)_y$ = 14 *ft*, L_b = 12 *ft*, C_b = 1.0.

Table 6-1, Page 6-70



12		.774	1.86	1.23	1.28	0.853	2.07	1.38	1.41	0.937	2.33	1.55
13	1.19	0.789	1.86	1.23	1.31	0.869	2.07	1.38	1.44	0.955	2.33	1.55
14	1.21	0.805	1.87	1.25	1.33	0.887	2.08	1.38	1.47	0.975	2.33	1.55
15	1.24	0.823	1.89	1.26	1.36	0.907	2.10	1.40	1.50	0.997	2.33	1.55
16	1.27	0.843	1.91	1.27	1.40	0.929	2.13	1.42	1.53	1.02	2.35	1.57
17	1.30	0.864	1.93	1.29	1.43	0.953	2.15	1.43	1.57	1.05	2.38	1.59

Table 6-1, Page 6-70



Combined Forces Example 1

From Table 6-1, Pages 6-70, for a W14x99; $p = 0.887 \times 10^{-3}$; $b_x = 1.38 \times 10^{-3}$; $b_y = 2.85 \times 10^{-3}$

$\frac{r_x}{r_y} = 1.66 > \frac{(KL)_x}{(KL)_y} = \frac{20\ \text{ft}}{14\ \text{ft}} = 1.429 \therefore y$ axis buckling controls

Determine which equation applies:

$p = \frac{1}{\phi_c P_n} = \frac{1}{P_c} = 0.887 \times 10^{-3}; P_c = 1{,}130\ \text{k}$

$\frac{P_r}{P_c} = \frac{400\ \text{k}}{1{,}130\ \text{k}} = 0.3540 \therefore$ Use H1-1a

Evaluate the section:

$0.887 \times 10^{-3}(400\ \text{k}) + 1.38 \times 10^{-3}(250\ \text{k} \cdot \text{ft}) + 2.85 \times 10^{-3}(80\ \text{k} \cdot \text{ft}) = 0.928 < 1.0\ \text{OK}$

A W14x99 is adequate.

Combined Forces Example 2

Determine if a W14x99 section of A992 steel is adequate to resist the following factored loads: P_r = 400 *k*, M_{rx} = 250 *k-ft*, M_{ry} = 80 *k-ft*. Use $(KL)_x$ = 20 *ft*, $(KL)_y$ = 14 *ft*, L_b = 26 *ft*, C_b = 1.10.

Table 6-1, Page 6-70

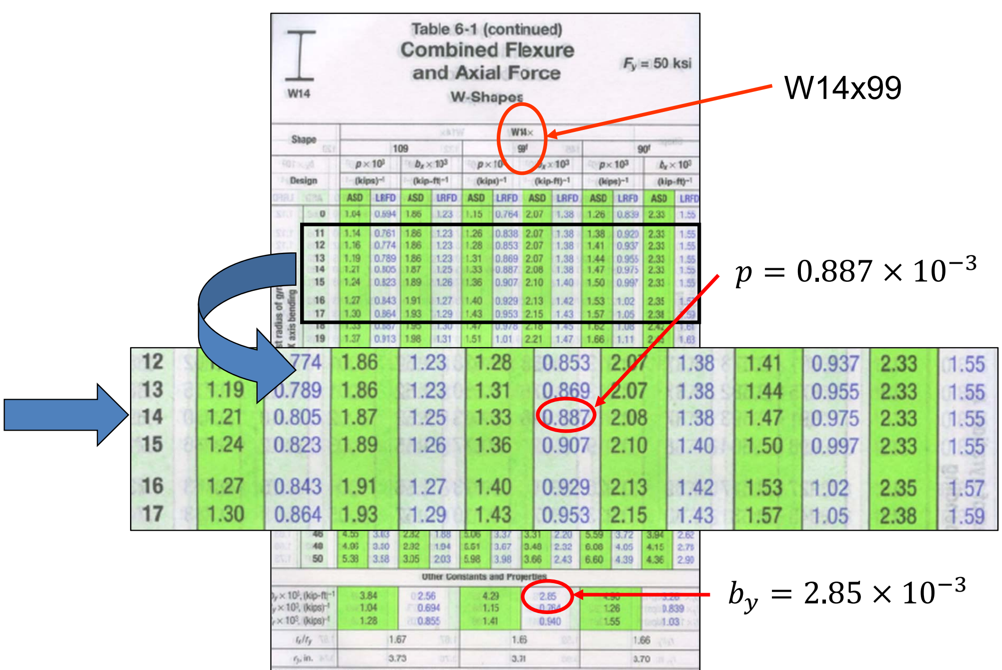
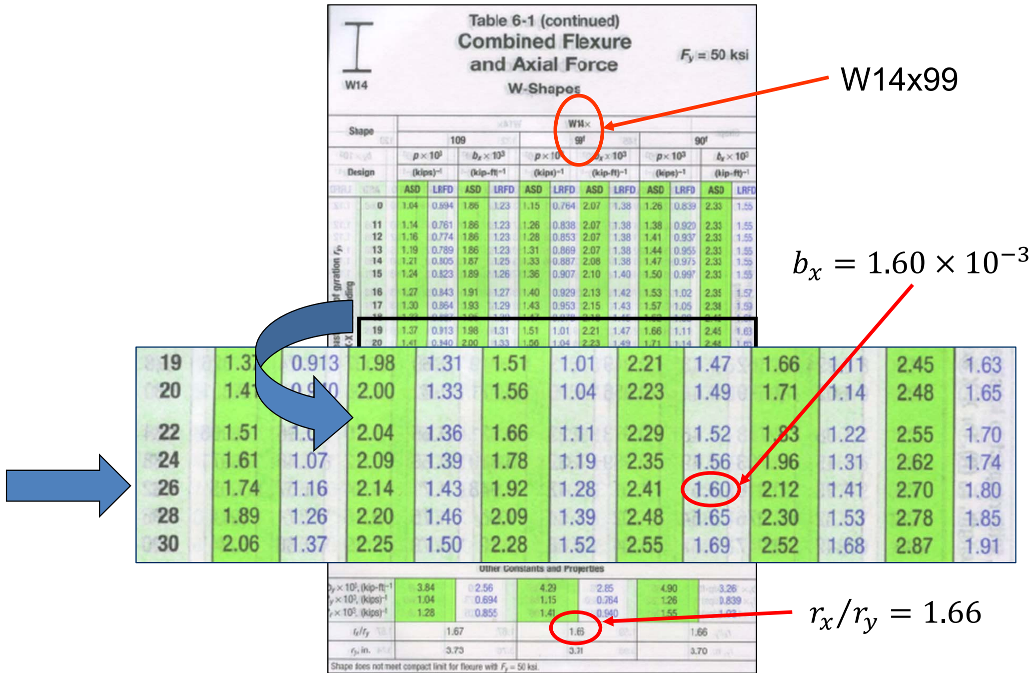


Table 6-1, Page 6-70



Combined Forces Example 2

From Table 6-1, Pages 6-70, try a W14x99; $p = 0.887 \times 10^{-3}$; $b_x = 1.60 \times 10^{-3}$; $b_y = 2.85 \times 10^{-3}$

For $C_b = 1.10$, $b_x = 1.60 \times 10^{-3}/1.10 = 1.455 \times 10^{-3} > 1.38 \times 10^{-3}$

∴ Use 1.455×10^{-3}

$\frac{r_x}{r_y} = 1.66 > \frac{(KL)_x}{(KL)_y} = \frac{20\text{ ft}}{14\text{ ft}} = 1.426$ ∴ y axis buckling controls

Determine which equation applies:

$p = \frac{1}{\phi_c P_n} = \frac{1}{P_c} = 0.887 \times 10^{-3}$; $P_c = 1{,}130\text{ k}$

$\frac{P_r}{P_c} = \frac{400\text{ k}}{1{,}130\text{ k}} = 0.3540$ ∴ Use H1-1a

Evaluate the section:

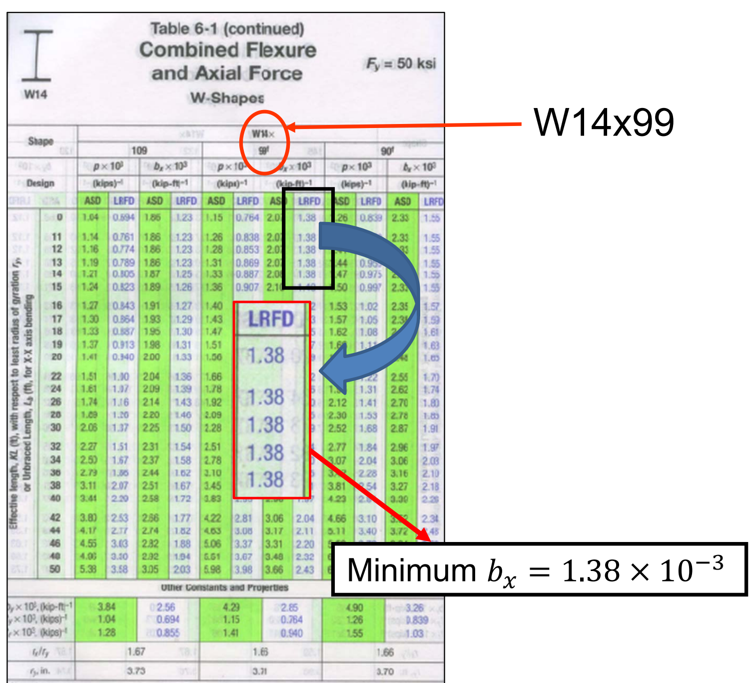
$0.887 \times 10^{-3}(400\text{ k}) + 1.455 \times 10^{-3}(250\text{ k} \cdot \text{ft}) + 2.85 \times 10^{-3}(80\text{ k} \cdot \text{ft}) = 0.947 < 1.0\text{ OK}$

A W14x99 is adequate.

Combined Forces Example 3

Determine if a W14x99 section of A992 steel is adequate to resist the following factored loads: P_r = 200 *k*, M_{rx} = 250 *k-ft*, M_{ry} = 80 *k-ft*. Use $(KL)_x$ = 30 *ft*, $(KL)_y$ = 14 *ft*, L_b = 26 *ft*, C_b = 1.80.

Table 6-1, Page 6-70



Combined Forces Example 3

From Table 6-1, Pages 6-70, try a W14x99; $p = 0.887 \times 10^{-3}$; $b_x = 1.60 \times 10^{-3}$; $b_y = 2.85 \times 10^{-3}$

For $C_b = 1.80$, $b_x = 1.60 \times 10^{-3}/1.80 = 0.889 \times 10^{-3} < 1.38 \times 10^{-3}$

∴ Use 1.38×10^{-3}

$\frac{r_x}{r_y} = 1.66 > \frac{(KL)_x}{(KL)_y} = \frac{30 \text{ ft}}{14 \text{ ft}} = 2.143$ ∴ x axis buckling controls

Re-enter table with $(KL)_{y,equiv} = \frac{30 \text{ ft}}{1.66} = 18.07 \text{ ft} \approx 18 \text{ ft}$

$p = 0.978 \times 10^{-3}$

Determine which equation applies:

$p = \frac{1}{\phi_c P_n} = \frac{1}{P_c} = 0.978 \times 10^{-3}$; $P_c = 1{,}020 \text{ k}$

$\frac{P_r}{P_c} = \frac{200 \text{ k}}{10{,}200 \text{ k}} = 0.1961$ ∴ Use H1-1b

Combined Forces Example 3

$$\frac{1}{2} p P_y + \frac{9}{8}\left(b_x M_{rx} + b_y M_{ry}\right) \leq 1.0$$

Evaluate the section:

$$\left[\frac{0.978 \times 10^{-3}}{2}\right](200 \text{ k})$$

$$+\frac{9}{8}\left[1.38 \times 10^{-3}(250 \text{ k} \cdot \text{ft}) + 2.85 \times 10^{-3}(80 \text{ k} \cdot \text{ft})\right]$$

$$= 0.742 < 1.0 \text{ OK}$$

A W14x99 is adequate.

Column Base Plates

Column Base Plates

Reference –

Fisher, J. M. and Kloiber, L. A., 2006, AISC Design Guide No. 1, *Base Plate and Anchor Rod Design*, Second Edition, AISC, Chicago, IL.

See also pages 14-4 to 14-7 and Section J8, page 16.1-132 SCM

Axially Loaded Base Plates

Concentric compressive axial loads

Tensile axial loads

Design of column base plates with small moments

Design of column base plates with large moments

Design for shear

Axially Loaded Base Plates

$$P_p = 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}} \leq 1.7 f_c' A_1 \text{ (J8-1)}$$

$$\phi_c = 0.65$$

$$\sqrt{\frac{A_2}{A_1}} \leq 2$$

Case I – $A_2 = A_1$

$$m = \frac{N - 0.95d}{2}$$

Case II – $A_2 \geq 4A_1$

$$n = \frac{B - 0.80 b_f}{2}$$

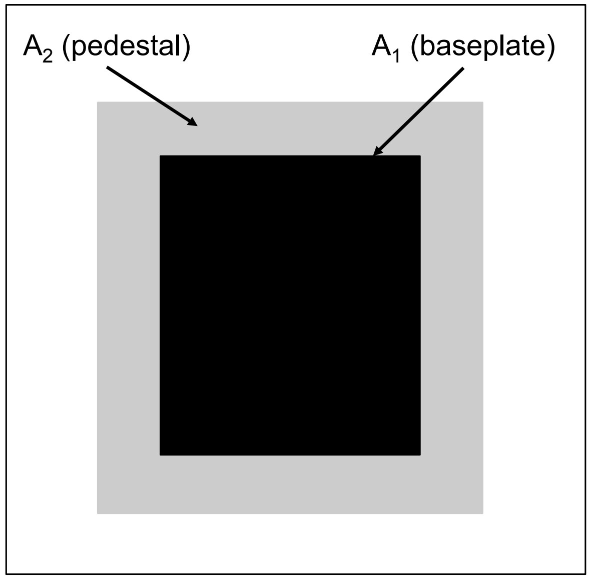
Case III – $A_1 < A_2 < 4A_1$

$$X = \left[\frac{4 d b_f}{(d + b_f)^2}\right] \frac{P_u}{\phi_c P_p}$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$$

$$\lambda n' = \lambda \frac{\sqrt{d b_f}}{4}$$

Axially Loaded Base Plates

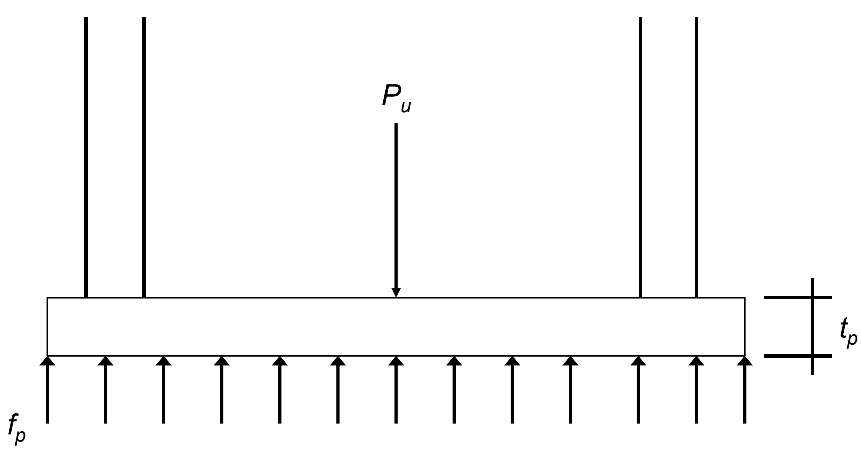


Footing Plan

ASCE | KNOWLEDGE & LEARNING

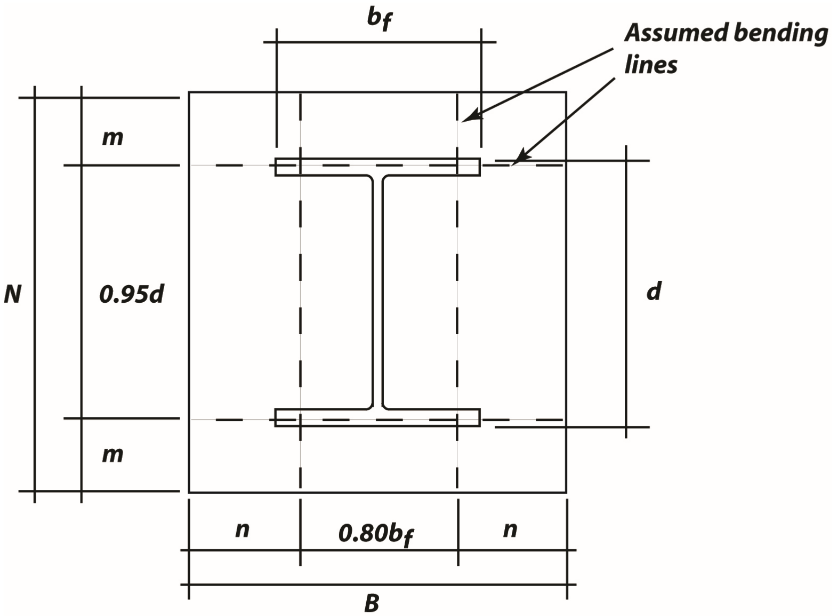
Axially Loaded Base Plates

Assumed bearing stress distribution



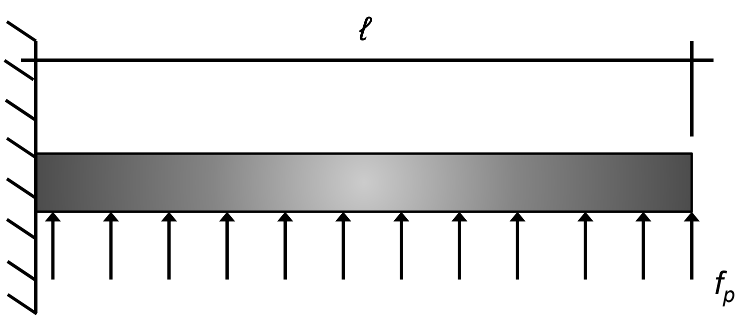
ASCE | KNOWLEDGE & LEARNING

Axially Loaded Base Plates



Axially Loaded Base Plates

$$\ell = \max\,(m, n, \lambda n')$$



Axially Loaded Base Plate Example

Design a base plate for a W12x106 (d = 12.9 in; b_f = 12.2 in.) column supporting a dead load of 300 k and a live load of 400 k. The plate bears on a 30 x 30 in. concrete pedestal. The specified concrete strength is 3 ksi and the column is made of A992 steel. The base plate is made of A36 steel.

Axially Loaded Base Plate Example

$P_u = 1.2(300 \text{ k}) + 1.6(400 \text{ k}) = 1{,}000 \text{ k}$

Assume full concrete confinement (Case II – $A_2 \geq 4A_1$)

$A_2 = (30 \text{ in})^2 = 900 \text{ in}^2$

$A_{1(reqd)} = \frac{P_u}{2\phi_c \times 0.85 \times f'_c} = \frac{1{,}000 \text{ k}}{2(0.65)(0.85)(3 \text{ ksi})} = 301.7 \text{ in}^2$

Optimize base plate dimension

$\Delta = 0.5\left[0.95d - 0.8b_f\right] = 0.5[0.95(12.9 \text{ in}) - 0.8(12.2 \text{ in})] = 1.248 \text{ in}$

$N \approx \sqrt{A_1} + \Delta = \sqrt{301.7 \text{ in}^2} + 1.248 \text{ in} = 18.62 \text{ in}$

Try N = 19 in

$B = \frac{301.7 \text{ in}^2}{19 \text{ in}} = 15.9 \text{ in}$

Try B = 16 in

Axially Loaded Base Plate Example

$A_1 = (19\text{ in})(16\text{ in}) = 304\text{ in}^2 > 301.7\text{ in}^2$

Calculate the area A_2 geometrically similar to A_1

Based on the 30 inch pedestal

$N_2 = 30\text{ in}$

$B/N = 16\text{ in}/19\text{ in} = 0.8421$

$B_2 = 0.8421(30\text{ in}) = 25.26\text{ in}$

$A_2 = (30\text{ in})(25.26\text{ in}) = 757.9\text{ in}^2$

$757.9\text{ in}^2 < 4(301.7\text{ in}^2) = 1{,}207\text{ in}^2$ ∴ Case III applies

Axially Loaded Base Plate Example

Use trial and error

Try N = 23 in; B = 19 in

$A_1 = (23\text{ in})(19\text{ in}) = 437\text{ in}^2$

$N_2 = 30\text{ in}$

$B/N = 19\text{ in}/23\text{ in} = 0.8261$

$B_2 = 0.8261(30\text{ in}) = 24.78\text{ in}$

$A_2 = (30\text{ in})(24.78\text{ in}) = 743.4\text{ in}^2$

$$\phi_c P_p = \phi_c \times 0.85 f_c' A_1 \sqrt{\frac{A_2}{A_1}} = 0.65(0.85)(3\text{ ksi})(437\text{ in}^2)\sqrt{\frac{743.4\text{ in}^2}{437\text{ in}^2}}$$

$$= 944.8\text{ k} < 1{,}000\text{ k}$$

Axially Loaded Base Plate Example

Try 24 in x 20 in

$A_1 = (24\text{ in})(20\text{ in}) = 480\text{ in}^2$

$N_2 = 30\text{ in}$

$B/N = 20\text{ in}/24\text{ in} = 0.8333$

$B_2 = 0.8333(30\text{ in}) = 25.0\text{ in}$

$A_2 = (30\text{ in})(25.0\text{ in}) = 750.0\text{ in}^2$

$$\phi_c P_p = \phi_c \times 0.85 A_1 \sqrt{\frac{A_2}{A_1}} = 0.65(0.85)(3\text{ ksi})(480\text{ in}^2)\sqrt{\frac{750.0\text{ in}^2}{480\text{ in}^2}}$$

$$= 994.5\text{ k} < 1{,}000\text{ k}$$

Axially Loaded Base Plate Example

Try 25 in x 22 in

$A_1 = (25\text{ in})(22\text{ in}) = 550\text{ in}^2$

$N_2 = 30\text{ in}$

$B/N = 22\text{ in}/25\text{ in} = 0.8800$

$B_2 = 0.8800(30\text{ in}) = 26.4\text{ in}$

$A_2 = (30\text{ in})(26.4\text{ in}) = 792.0\text{ in}^2$

$$\phi_c P_p = \phi_c \times 0.85 A_1 \sqrt{\frac{A_2}{A_1}} = 0.65(0.85)(3\text{ ksi})(550\text{ in}^2)\sqrt{\frac{792.0\text{ in}^2}{550\text{ in}^2}}$$

$$= 1{,}094\text{ k} > 1{,}000\text{ k}$$

Axially Loaded Base Plate Example

$$m = \frac{N - 0.95d}{2} = \frac{25\text{ in} - 0.95(12.9\text{ in})}{2} = 6.373$$

$$n = \frac{B - 0.80b_f}{2} = \frac{22\text{ in} - 0.80(12.2\text{ in})}{2} = 6.120$$

$$X = \left[\frac{4db_f}{\left(d + b_f\right)^2}\right]\frac{P_u}{\phi_c P_p} = \left[\frac{4(12.9\text{ in})(12.2\text{ in})}{(12.9\text{ in} + 12.2\text{ in})^2}\right]\frac{1{,}000\text{ k}}{1{,}094\text{ k}} = 0.9134$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} = \frac{2\sqrt{0.9134}}{1 + \sqrt{1 - 0.9134}} = 1.477 \rightarrow 1$$

$$\lambda n' = \lambda\frac{\sqrt{db_f}}{4} = (1)\frac{\sqrt{(12.9\text{ in})(12.2\text{ in})}}{4} = 3.136$$

Axially Loaded Base Plate Example

l = max (6.373 in, 6.120 in, 3.136 in) = 6.373 in

$$t_p = m\sqrt{\frac{2P_u}{\phi F_y BN}} = (6.373\text{ in})\sqrt{\frac{2(1{,}000\text{ k})}{0.9(36\text{ ksi})(22\text{ in})(25\text{ in})}} = 2.135\text{ in}$$

Use 25” x 2-1/4” x 22”

Beam Bracing – Appendix 6

Beam Bracing

ASCE 360 Appendix 6 – Stability Bracing for Beams and Columns

Applies to design of bracing that is not part of the lateral force resisting system – requirements for bracing that is part of the lateral force resisting system (that is, included in the analysis of the structure) are addressed in Chapter C.

Beam Bracing

Lateral bracing – columns and beams

- Relative
- Nodal

Torsional bracing

- Nodal
- Continuous

Reference –

- Yura, J. A., *Fundamentals of Beam Bracing*, AISC Engineering Journal, First Quarter 2001

Beam Bracing

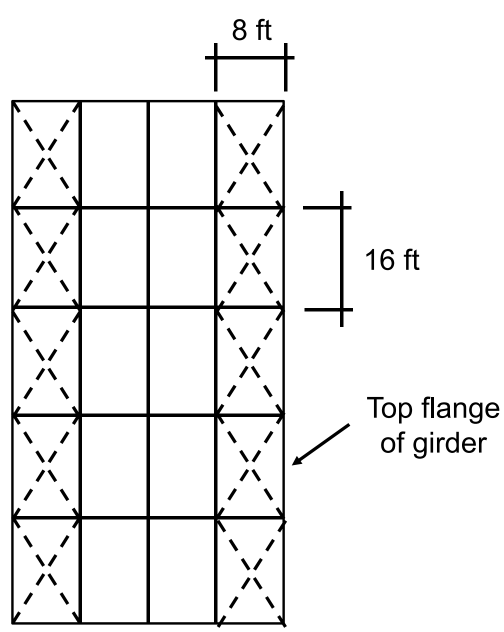
A relative brace controls the movement of the brace point with respect to adjacent brace points.

A discrete or nodal brace controls the movement at the brace point without interaction with adjacent points.

Beam Bracing Example

Determine the bracing requirements for the beam system shown. The simply supported beams are A992 W21x62. Each bracing truss stabilizes 2-1/2 beams. The factored moment in the beams is 350 k-ft. Assume that the x-bracing (relative bracing) is to be designed as a tension only system so that in each panel only one diagonal is effective. Bracing consists of A36 rods.

Beam Bracing Example



Plan View

Beam Bracing Example

$M_r = 350 \text{ k} \cdot \text{ft}$

Both strength and stiffness must be considered!

$L_b = 16 \text{ ft}$

$d = 21.0 \text{ in}$

$t_f = 0.615 \text{ in}$

$h_o = d - t_f = 21.0 \text{ in} - 0.615 \text{ in} = 20.39 \text{ in}$

$C_d = 1$ (singular curvature)

Φ = 0.75

Relative bracing
Section A6.3.1a
Eq. A-6-5

Required brace strength

Perpendicular to longitudinal axis of beam

$$P_{br} = \frac{0.008 M_r C_d}{h_o} = \frac{0.008(350 \text{ k·ft})(12 \text{ in/ft})(1)}{20.39 \text{ in}} = 1.648 \text{ k}$$

$(2.5)(1.648 \text{ k}) = 4.120 \text{ k}$

Beam Bracing Example

Relative bracing
Section A6.3.1a
Eq. A-6-6

Perpendicular to longitudinal axis of beam

Required brace stiffness

$$\beta_{br} = \frac{1}{\phi}\left(\frac{4 M_r C_d}{L_b h_o}\right) = \frac{1}{0.75}\left[\frac{4(350 \text{ k·ft})(12 \text{ in/ft})(1)}{(16 \text{ ft} \times 12 \text{ in/ft})(20.39 \text{ in})}\right] = 5.722 \text{ k/in}$$

$(2.5)(5.722 \text{ k/in}) = 14.31 \text{ k/in}$

Brace area to satisfy strength requirement

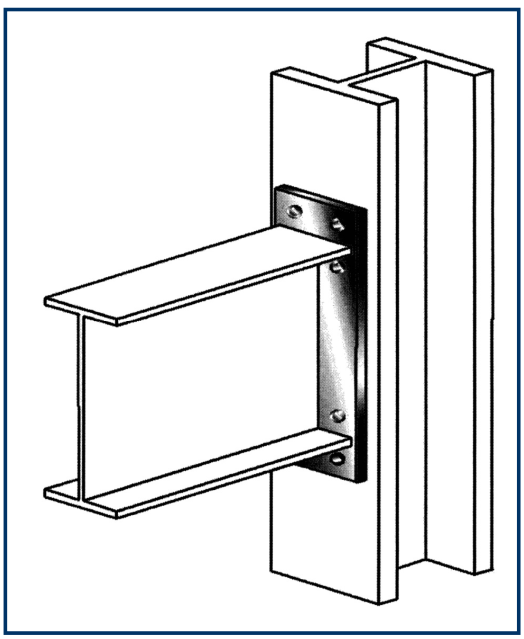
$$\phi A_b F_y = A_b(0.9)(36 \text{ ksi}) = (4.120 \text{ k})\sqrt{5};\ A_b = 0.284 \text{ in}^2$$

Brace area to satisfy stiffness requirement

$$\cos^2\theta\left(\frac{AE}{L}\right) = \left(\frac{1}{\sqrt{5}}\right)^2 \frac{A_b(29{,}000 \text{ ksi})}{8 \text{ ft} \times 12 \text{ in/ft} \times \sqrt{5}} = 14.31 \text{ k/in};\ A_b = 0.5296 \text{ in}^2 \leftarrow$$

Extended End Plate Connection

End-Plate Moment Connections



Moment End-Plate Connections

1. Carter, C. J., 2003, Steel Design Guide No. 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, AISC, Chicago, IL.
2. AISC, 2002, Steel Design Guide No. 16*, Flush and Extended Multiple-Row Moment End-Plate Connections*, AISC, Chicago, IL.
3. Murray, T. M. and Summer, E. A., 2003, Steel Design Guide No. 4, *Extended End-Plate Moment Connections, Seismic and Wind Applications*, Second Edition, AISC, Chicago, IL.

Moment End-Plate Connections

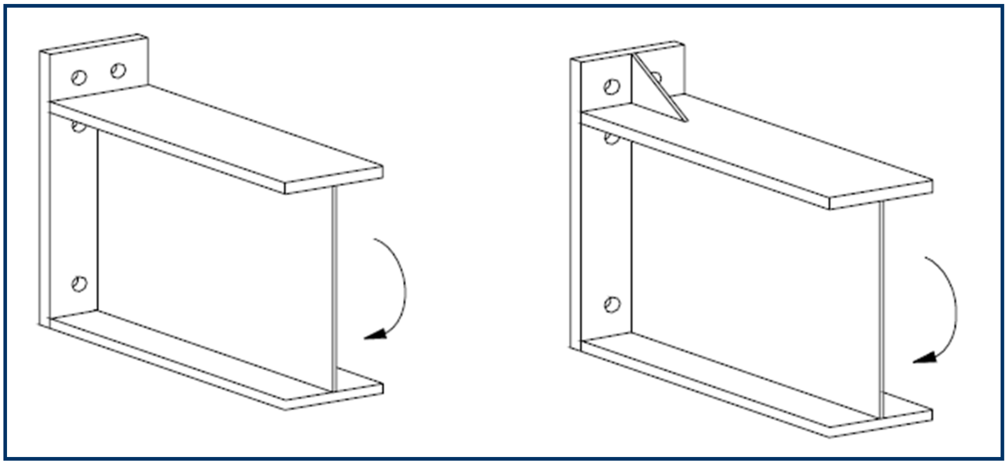
Flush

- Two-bolt unstiffened
- Four-bolt unstiffened
- Four-bolt stiffened with stiffener between tension bolts
- Four-bolt stiffened with stiffener inside tension bolts

Extended

- Four-bolt unstiffened
- Four-bolt stiffened
- Multiple-row 1/2 unstiffened
- Multiple-row 1/3 unstiffened
- Multiple-row 1/3 stiffened

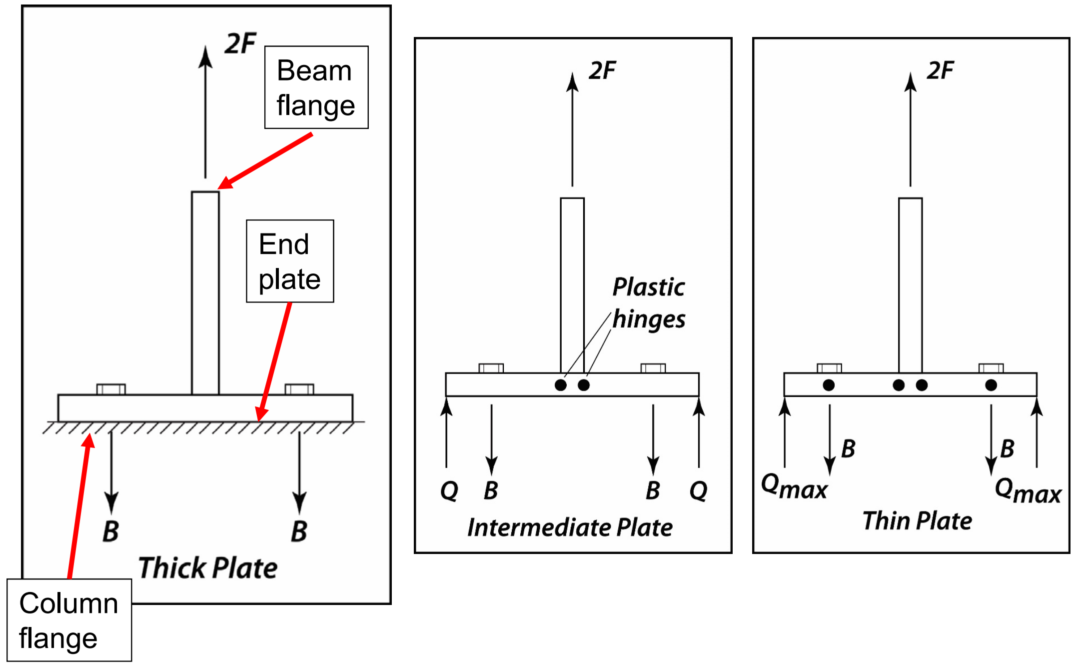
Extended Moment End-Plate Connections



Moment End-Plate Connection Limit States

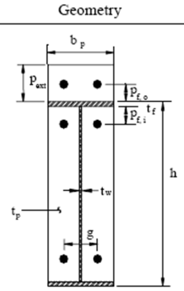
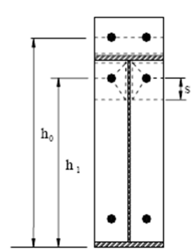
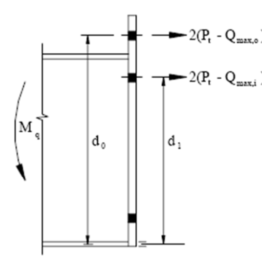
1. Flexural yielding of the beam section
2. Flexural yielding of the endplate
3. Yielding of the column panel zone
4. Tension failure of the endplate bolts
5. Shear failure of the endplate bolts
6. Failure of welds

Split-T Model



Connection Yield Parameter – *Y*

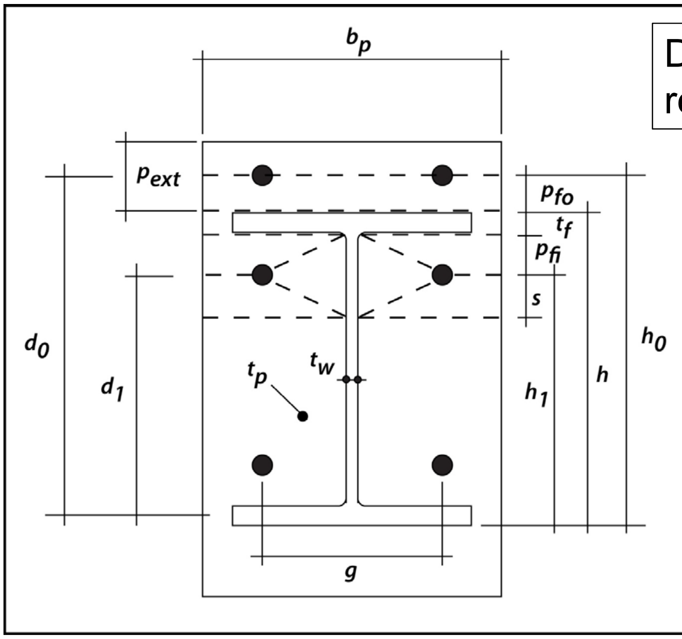
Four-bolt extended unstiffened

Geometry	Yield-Line Mechanism	Bolt Force Model
		

End-Plate Yield	$\phi M_n = \phi_b M_{pl} = \phi_b F_{py} t_p^2 Y$ $Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}} + \frac{1}{s}\right) + h_0\left(\frac{1}{p_{f,o}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1\left(p_{f,i} + s\right)\right]$ Note: Use $p_{f,i} = s$, if $p_{f,i} > s$ $s = \frac{1}{2}\sqrt{b_p g}$ $\phi_b = 0.90$

Reference 2

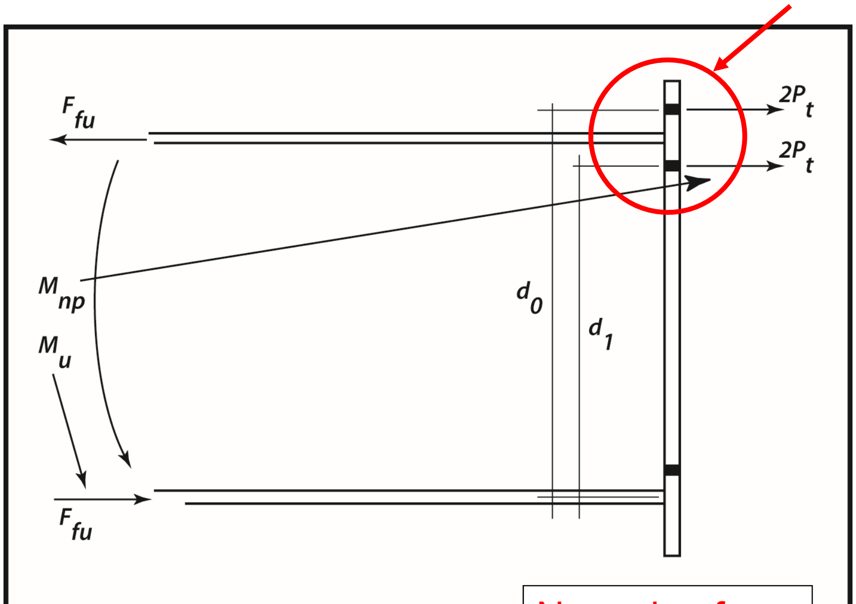
Plate Geometry and Yield Pattern



Dashed lines represent yield lines

Bolt Force Model

Split-T



No prying force

Extended Moment End-Plate Example

A W21x68 beam is to be connected to a W14x99 exterior column using a four-bolt unstiffened extended end plate connection. The moment that must be developed by the connection is 350 *k-ft* (M_r) and the required shear resistance is 45 *k*. The required axial strength of the column is P_r = 600 *k*. The connection will be used in a low-seismic application (*R* <= 3). The beam and column are made of A992 steel and the connection plate is made of A36 steel. ASTM A325-N snug-tight bolts are to be used and welds will be made with E70 electrodes.

Section and Material Properties

Beam data

$A_b = 20.0 \text{ in}^2$

$d_b = 21.1 \text{ in}$

$t_{wb} = 0.430 \text{ in}$

$b_{fb} = 8.27 \text{ in}$

$t_{fb} = 0.685 \text{ in}$

$k_b = 1.19 \text{ in}$

$k_1 = \frac{7}{8} \text{ in}$

Workable gage $= 5\frac{1}{2}$ in

$Z_{xb} = 160 \text{ in}^3$

Column Data

$A_c = 29.1 \text{ in}^2$

$d_c = 14.2 \text{ in}$

$t_{wc} = 0.485 \text{ in}$

$b_{fc} = 14.6 \text{ in}$

$t_{fc} = 0.780 \text{ in}$

$k_c = 1.38 \text{ in}$

$k_c = 1.38 \text{ in}$

$(h/t_w)_c = 23.5 \text{ in}$

Workable gage $= 5\frac{1}{2}$ in

$Z_{xc} = 173 \text{ in}^3$

Material Data

$F_{y,b,c} = 50 \text{ ksi}$

$F_{u,b,c} = 65 \text{ ksi}$

$F_{y,pl} = 36 \text{ ksi}$

$F_{u,pl} = 58 \text{ ksi}$

Bolt data

$F_t = 90 \text{ ksi}$

Beam Side

Geometric Design Data

$\gamma_r = 1$ (extended end plate connection)

$b_p = b_{fb} + 1\text{ in} = 8.27\text{ in} + 1\text{ in} = 9.27\text{ in}$; Use $b_p = 9.25$

$g = 5.5\text{ in}$

$p_{fi} = 2\text{ in}$

$p_{fo} = 2\text{ in}$

$d_e = 1\frac{5}{8}\text{ in}$

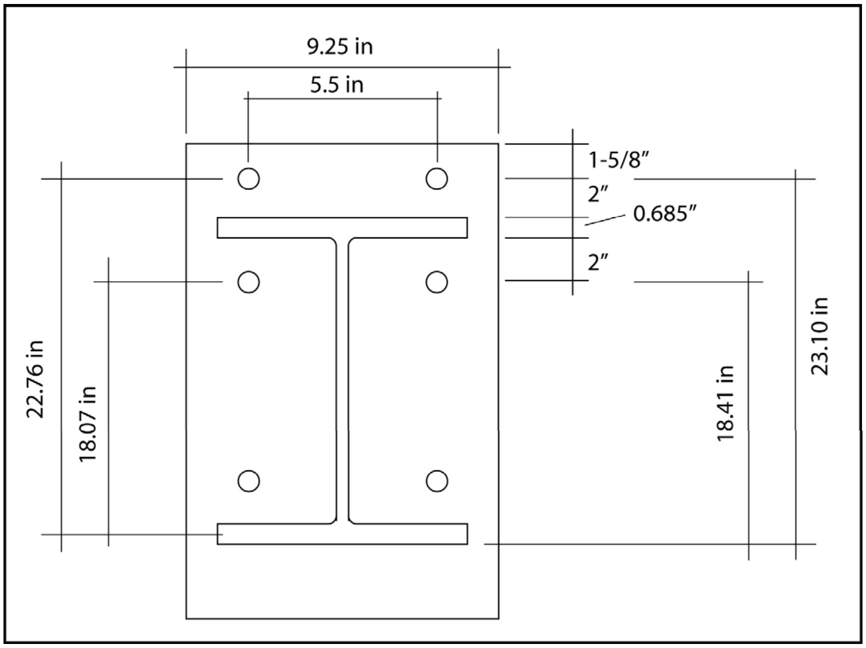
$d_0 = d_b + p_{fo} - \frac{t_{fb}}{2} = 21.1\text{ in} + 2\text{ in} - \frac{0.685\text{ in}}{2} = 22.76\text{ in}$

$h_0 = d_0 + \frac{t_{fb}}{2} = 22.76\text{ in} + \frac{0.685\text{ in}}{2} = 23.10\text{ in}$

$d_1 = d_b - t_{fb} - p_{fi} - \frac{t_{fb}}{2} = 21.1\text{ in} - 0.685\text{ in} - 2\text{ in} - \frac{0.685\text{ in}}{2} = 18.07\text{ in}$

$h_1 = d_1 + \frac{t_{fb}}{2} = 18.07\text{ in} + \frac{0.685\text{ in}}{2} = 18.41\text{ in}$

Beam/Plate Configuration



Select Bolt Diameter

Note: the symbol d_b is used to represent both beam depth and bolt diameter. The meaning of the symbol applies should be clear from the context.

Required bolt diameter

$$M_{rc} = (350\ \mathrm{k \cdot ft})(12\ \mathrm{in/ft}) = 4{,}200\ \mathrm{k \cdot in}$$

$$d_{b,reqd} = \sqrt{\frac{2M_{rc}}{\pi\phi F_t(d_0+d_1)}} = \sqrt{\frac{2(4{,}200\ \mathrm{k{\cdot}in})}{\pi(0.75)(90\ \mathrm{ksi})(22.76\ \mathrm{in}+18.07\ \mathrm{in})}}$$

$$= 0.9852\ \mathrm{in}$$

Try $d_b = 1\ \mathrm{in}$

Minimum $p_f = 1 + \frac{1}{2} = 1.5\ \mathrm{in} < 2\ \mathrm{in\ OK}$

Calculate No-Prying Moment

Thick plate – no prying force

$M_{np} \rightarrow$ no prying force moment

Bolt tensile strength

$$P_t = F_t A_b = (90\text{ ksi})\left[\frac{\pi(1\text{ in})^2}{4}\right] = 70.65\text{ k}$$

$$M_{np} = 2P_t(h_0 + h_1) = 2(70.65\text{ k})(22.76\text{ in} + 18.07\text{ in})$$

$$5769\text{ k}\cdot\text{in}$$

$$\phi M_{np} = 0.75(5{,}769\text{ k}\cdot\text{in}) = 4{,}326\text{ k}\cdot\text{in} > 4{,}200\text{ k}\cdot\text{in OK}$$

Select End Plate Thickness

End plate yield line mechanism parameter

$$s = \frac{1}{2}\sqrt{b_p g} = \frac{1}{2}\sqrt{(9.25\text{ in})(5.5\text{ in})} = 3.57\text{ in} > p_{fi} = 2.0$$

If $p_{fi} > s$, use $p_{fi} = s$

∴ Use $p_{fi} = 2.0$

$$Y_p = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{fi}} + \frac{1}{s}\right) + h_0\left(\frac{1}{p_{fo}}\right) - \frac{1}{2}\right] + \frac{2}{g}\left[h_1\left(p_{fi} + s\right)\right] =$$

$$\frac{9.25\text{ in}}{2}\left[(18.41\text{ in})\left(\frac{1}{2\text{ in}} + \frac{1}{3.57\text{ in}}\right) + (23.10\text{ in})\left(\frac{1}{2\text{ in}}\right) - \frac{1}{2}\right] +$$

$$\frac{2}{5.5\text{ in}}[(18.41\text{ in})(2\text{ in} + 3.57\text{ in})] = 154.8$$

Select End Plate Thickness – Flexure

Required end plate thickness

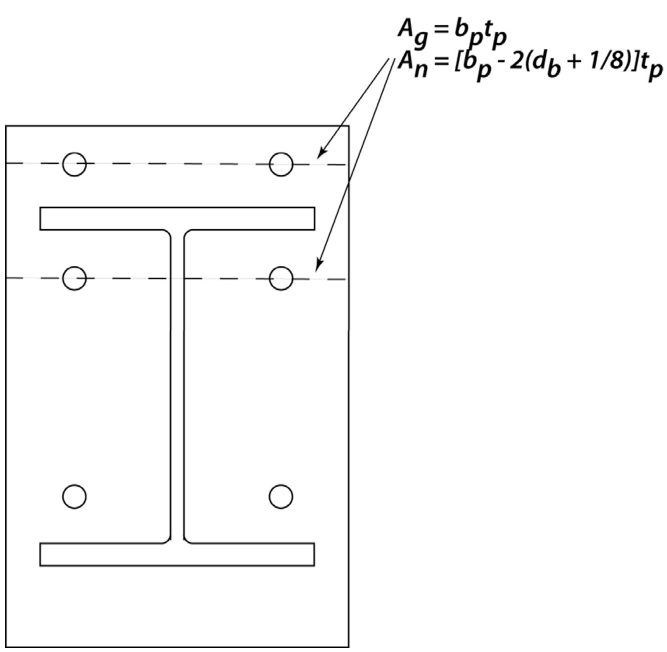
1.11 insures thick plate behavior

$$t_{p,reqd} = \sqrt{\frac{1.11\gamma_r(\phi M_{np})}{\phi_b F_{y,pl} Y_p}}$$

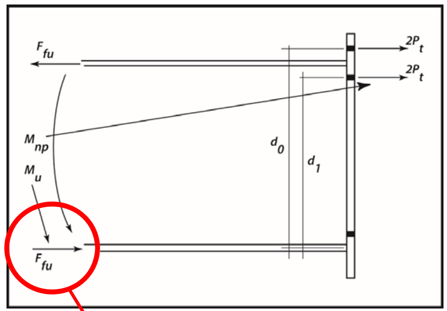
$$= \sqrt{\frac{1.11(1.0)(4{,}326\ \text{k}\cdot\text{in})}{0.9(36\ \text{ksi})(154.8)}} = 0.9785\ \text{in}$$

Use $t_p = 1\ \text{in}$

Shear Areas



Check End Plate Thickness – Shear



Factored beam flange force

$$F_{fu} = \frac{M_{rc}}{d_b - t_{fb}} = \frac{4{,}200 \text{ k·in}}{21.1 \text{ in} - 0.685 \text{ in}} = 205.7 \text{ k}$$

Shear yielding in the extended part of the end plate – ASCE 360 Eq. J4-3

$$\phi R_n = 2[\phi(0.6F_{y,pl})b_p t_p]$$

$$= 2[(1.00)(0.6)(36 \text{ ksi})(9.25 \text{ in})(1 \text{ in})] = 400 \text{ k}$$

$205.7 \text{ k} < 400 \text{ k OK}$

Select End Plate Thickness – Shear

Shear rupture in the two row of bolts at top of connection

$$A_n = \left[b_p - 2\left(d_b + \frac{1}{8}\right)\right] t_p = [9.25 - 2(1 \text{ in} + 0.125 \text{ in})](1 \text{ in})$$

$$= 7.0 \text{ in}^2$$

$$\phi R_n = 0.75(0.6F_{u,pl})A_n = 0.75(0.6)(58 \text{ ksi})(7.0 \text{ in}^2) = 132.7 \text{ k}$$

$2(132.7) > 205.7 \text{ k OK}$

Compression Bolt Shear Rupture Capacity

$V_u = 45 \text{ k}$

ASCE 360 Section J3 and Table J3.2

$$\phi R_n = \phi n_b F_v A_b = 0.75(2)(54 \text{ ksi})\left[\frac{\pi(1 \text{ in})^2}{4}\right] = 63.6 \text{ k}$$

$45 \text{ k} < 63.6 \text{ k OK}$

Bolt Bearing/Tearout Capacity in Endplate

$R_n = 1.2 l_c t_p F_{u,pl}$

$1.2 l_c t_p F_{u,pl} \rightarrow$ tearout strength

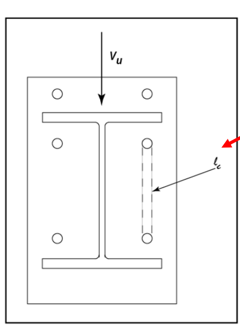
$2.4 d_b t_p F_{u,pl} \rightarrow$ bearing strength

Nominal bolt bearing strength – one bolt

$2.4 d t_p F_{u,pl} = 2.4(1 \text{ in})(1 \text{ in})(58 \text{ ksi})$

$= 139.2 \text{ k/bolt}$

Section J3.10 – ASCE 360



Tearout – one bottom bolt

$l_c \approx d_b - 2p_{fi} = 21.1 \text{ in} - 2(2 \text{ in}) = 17.1 \text{ in}$

$R_n = 1.2 l_c t_p F_{u,pl} = 1.2(17.1 \text{ in})(1 \text{ in})(58 \text{ ksi})$

$= 1{,}190 \text{ k}$

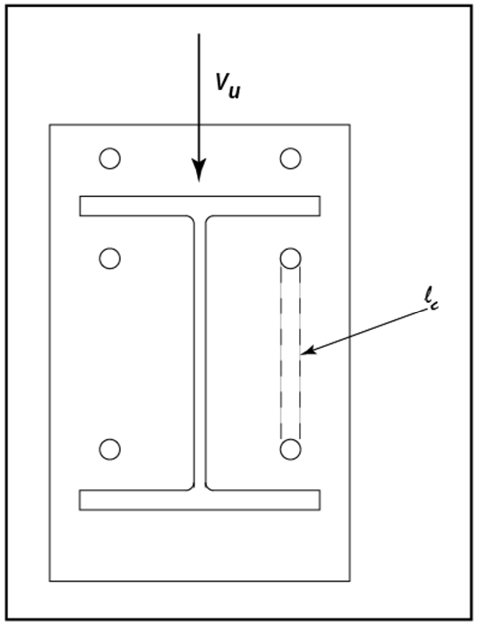
$1{,}190 \text{ k} > 139.2 \text{ k} \therefore$ bearing controls

$R_n = 139.2 \text{ k}$

Capacity for two bolts

$\phi R_n = 2(0.75)(139.2 \text{ k}) = 208.8 \text{ k} > 45 \text{ k OK}$

Bolt Bearing/Tearout End Plate



Bolt Bearing/Tearout Column

Two bolts bearing on column flange

$t_{fc} = 0.780\ in$

$2.4 d_b t_p F_{u,pl} = 2.4(1\text{ in})(0.780\text{ in})(65\text{ ksi})$

$= 121.7\text{ k/bolt}$

$\phi R_n = 2(0.75)(121.7\text{ k}) = 182.6\text{ k} > 45\text{ k OK}$

Beam Flange to End Plate Weld Design

Minimum fillet weld size → 5/16 in

Weld design force should be equal to the calculated flange force, but not less than $0.6F_yA_{fb}$

$R_u = F_{uf} \geq 0.6F_yA_{fb}$

$F_{uf} = 205.7\ \text{k}$

$0.6F_yA_{fb} = 0.6(50\ \text{ksi})(8.27\ \text{in})(0.685\ \text{in}) = 170.0\ \text{k} < 205.7\ \text{k}$

$R_u = 205.7\ \text{k}$

ASCE | KNOWLEDGE & LEARNING

Beam Flange to End Plate Weld Design

Beam flanges to end plate

$L_w = b_{fb} + (b_{fb} + t_{wb}) = 8.27\ \text{in} + (8.27\ \text{in} - 0.430\ \text{in}) = 16.11\ \text{in}$

$D = \frac{205.7\ \text{k}}{1.5(1.392)(16.11\ \text{in})} = 6.115\ \text{sixteenths} \rightarrow 7$

Use 7/16 in fillet weld

Eq. 8-2a
Page 8-8 SCM

ASCE | KNOWLEDGE & LEARNING

Beam Web to End Plate Weld Design Flexure

Minimum weld size → 5/16 in

$$D = \frac{\phi F_{yb} t_{wb}}{2(1.5)(1.392)} = \frac{0.9(50\text{ ksi})(0.430\text{ in})}{2(1.5)(1.392)} = 4.633\ sixteenths \rightarrow 5$$

Use 5/16 in fillet weld

Beam Web to End Plate Weld Design Flexure

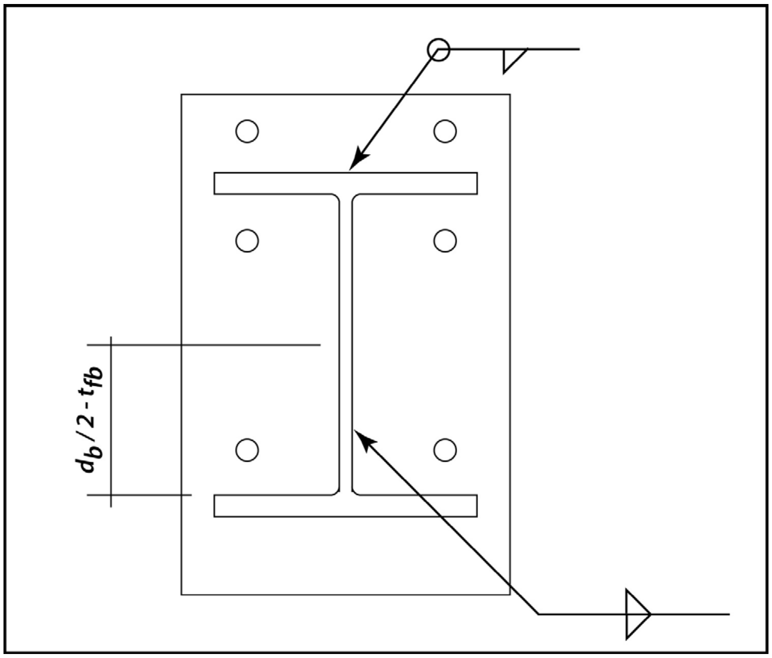
Minimum weld size → 5/16 in

$$L_w = 2\left(\frac{d_b}{2} - t_{fb}\right) = 2\left(\frac{21.1\text{ in}}{2} - 0.685\right)\text{in} = 19.73\text{ in}$$

$$D = \frac{V_u}{2(1.392)L_w} = \frac{45\text{ k}}{(1.392)(19.73\text{ in})} = 1.639\text{ sixteenths}$$

Minimum weld size controls

Beam to End Plate Welds



Column Side

Local Tensile Bending in Column Flange

$R_n = \left(\frac{b_s}{\alpha_m p_e}\right) t_{fc}^2 F_y C_y$ (Equation 2.2-9 Reference 1)

$R_n = 6.25 t_f^2 F_y$ (Eq. J10-1 – ASCE 360)

$\phi = 0.9;\ C_t = 1$

$b_s = 2.5(2p_f + t_{fb}) = 2.5(2[2\text{ in}] + 0.685\ in) = 11.71\text{ in}$

$p_e = \frac{g}{2} - \frac{d_b}{4} - k_1 = \frac{5.5\text{ in}}{2} - \frac{1\text{ in}}{4} - \frac{7}{8}\text{in} = 1.625\text{ in}$

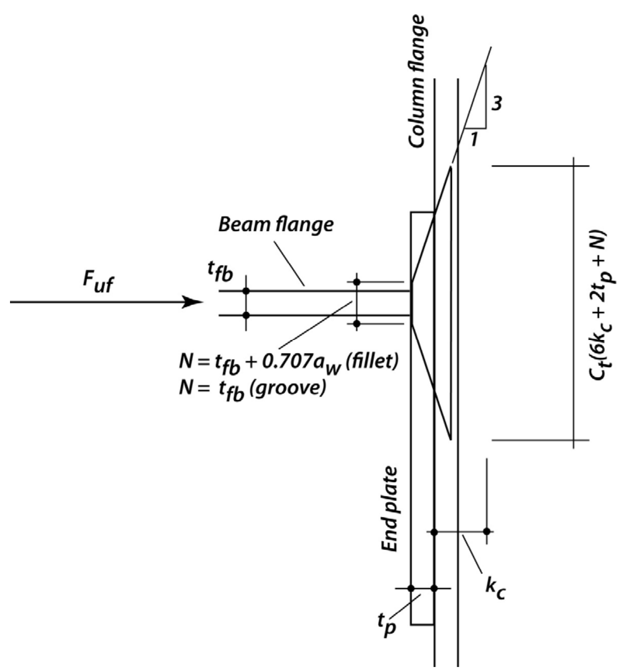
$\alpha_m = 1.36\left(\frac{p_e}{d_b}\right)^{1/4} = 1.36\left(\frac{1.625\text{ in}}{1\text{ in}}\right)^{1/4} = 1.536$

$R_n = \left(\frac{11.71\text{ in}}{1.536[1.625\text{ in}]}\right)(0.780\text{ in})^2(50\text{ ksi})(1) = 142.7\text{ k}$

$\phi R_n = 0.9(142.7\text{ k}) = 128.4\text{ k} < 205.7\text{ k}$ ∴ Column flange needs stiffening

$205.7\text{ k} - 128.4\text{ k} = 77.3\text{ k}$

Concentrated Force on Column Flange



Column Local Web Yielding

Assume connection not at top of column ($C_t = 1$)

$R_n = [C_t(6k_c + 2t_p + N)]F_{yc}t_{wc}$ (Equation 2.2-11 Reference 1)

$R_n = [(5k_c + l_b)]F_{yc}t_{wc}$ (Eq. J10-2 ASCE 360)

$\phi = 1;\ C_t = 1$

$N = t_{fb} + 0.707a_w = 0.685\text{ in} + 0.707(0.7375\text{ in}) = 0.9943\text{ in}$

$\phi R_n = (1)[(1)\{6(1.38\text{ in}) + 2(1\text{ in}) + 0.9943\text{ in}\}](50\text{ ksi})(0.485\text{ in})$

$= 273.4\text{ k} > F_{fu} = 205.7\text{ k}$

Stiffeners not required

Column Web Buckling

Because this is an exterior column, web buckling of the column does not need to be checker. If the connection was to an interior column (beams on each side of the column), the following check would be performed:

$$h = \left(\frac{h}{t_w}\right)_c t_{wc} = (23.5)(0.485\text{ in}) = 11.40\text{ in}$$

ASCE 360 Eq. J10-8

$$\phi R_n = \frac{\phi 24 t_{wc}^3 \sqrt{E F_{yc}}}{h} = \frac{0.9(24)(0.485\text{ in})^3\sqrt{(29{,}000\text{ ksi})(50\text{ ksi})}}{11.40\text{ in}}$$

$= 260.3\text{ k} > F_{uf} = 205.7\text{ k}$

Stiffeners not required

Column Web Crippling

ASCE 360 Equation J10-4

Force applied at distance greater than d/2 from end of member

$$\phi R_n = \phi 0.80 t_{wc}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^3\right]\sqrt{\frac{EF_{yc}t_{fc}}{t_{wc}}}$$

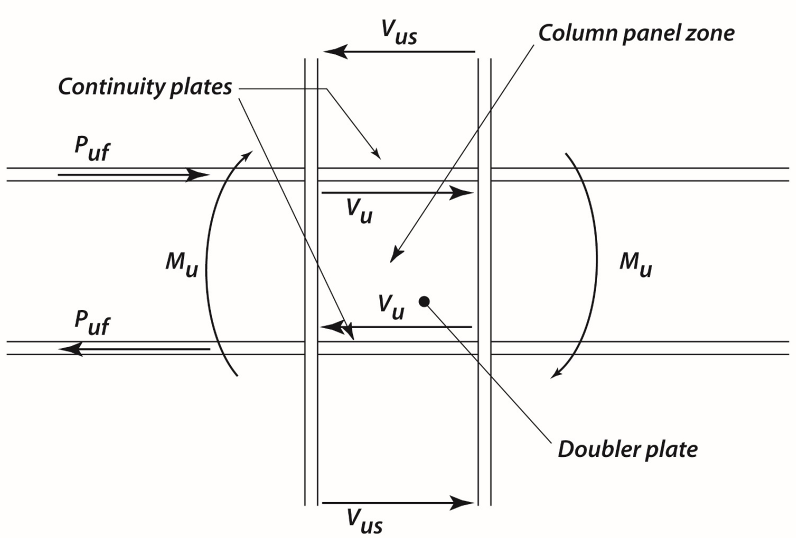
$$= 0.75(0.80)(0.485 \text{ in})^2 \times$$

$$\left[1 + 3\left(\frac{0.9943 \text{ in}}{14.2 \text{ in}}\right)\left(\frac{0.485 \text{ in}}{0.780 \text{ in}}\right)^3\right]\sqrt{\frac{(29{,}000 \text{ ksi})(50 \text{ ksi})(0.780 \text{ in})}{0.485 \text{ in}}}$$

$$= 226.4 \text{ k} > F_{uf} = 205.7 \text{ k}$$

Stiffeners not required

Column Panel Zone



Column Panel Zone

Column Panel Zone

This check also applies to interior connection.

ASCE 360 – Section J10.6(a)

$P_c = P_y = F_y A_c = (50\text{ ksi})(29.1\text{ in}^2) = 1{,}455\text{ k}$

$V_r = V_u = P_{uf} - V_{us} = \frac{M_u}{d_c} = \frac{4{,}200\text{ k}\cdot\text{in}}{14.7\text{ in}} - 0 = 285.7\text{ k}$ (neglect story shear V_{us})

$0.4P_c = 582.0\text{ k}; < P_r = 600\text{ k} > 0.4P_c$

$\phi R_n = \phi 0.6 F_y d_c t_{wc}\left(1.4 - \frac{P_r}{P_c}\right)$

$= 0.9(0.6)(50\text{ ksi})(14.7\text{ in})(0.645\text{ in})\left(1.4 - \frac{600\text{ k}}{1{,}455\text{ k}}\right)$

$= 252.8\text{ k} < 285.7\text{ k}$

Panel zone shear strength is not adequate – doubler plates are required

Column Stiffening

Stiffening requirements for this column are minimal. It may be more economical to increase the column size to W14x109 rather than stiffen the W14x99.

Column Stiffening

Column Stiffening Example

In the previous example, change the column to a W14x90. Design stiffeners for this column.

Section and Material Properties

Column data

$A_c = 26.5 \text{ in}^2$

$d_c = 14.0 \text{ in}$

$t_{wc} = 0.440 \text{ in}$

$b_{fc} = 14.5 \text{ in}$

$t_{fc} = 0.710 \text{ in}$

$k_c = 1.31 \text{ in}$

$(h/t_w)_c = 25.9 \text{ in}$

Workable gage = $5\frac{1}{2}$ in

$Z_{xc} = 157 \text{ in}^3$

Material Data

$F_{y,b,c} = 50 \text{ ksi}$

$F_{u,b,c} = 65 \text{ ksi}$

$F_{y,pl} = 36 \text{ ksi}$

$F_{u,pl} = 58 \text{ ksi}$

Bolt data

$F_t = 90 \text{ ksi}$

Local Tensile Bending in Column Flange

$R_n = \left(\frac{b_s}{\alpha_m p_e}\right) t_{fc}^2 F_y C_t$ (Equation 2.2-9 Reference 1)

$\phi = 0.9$

$b_s = 2.5(2p_f + t_{fb}) = 2.5(2[2 \text{ in}] + 0.685 \text{ in}) = 11.71 \text{ in}$

$p_e = \frac{g}{2} - \frac{d_b}{4} - k_1 = \frac{5.5 \text{ in}}{2} - \frac{1 \text{ in}}{4} - \frac{7}{8} \text{ in} = 1.625$

$\alpha_m = 1.36\left(\frac{p_e}{d_b}\right)^{1/4} = 1.36\left(\frac{1.625 \text{ in}}{1 \text{ in}}\right)^{1/4} = 1.536$

$R_n = \left(\frac{11.71 \text{ in}}{1.536[1.625 \text{ in}]}\right)(0.710 \text{ in})^2(50 \text{ ksi})(1) = 118.2 \text{ k}$

$\phi R_n = 0.9(118.2\ k) = 106.4\ k < 205.7 \therefore$ Column flange needs stiffening

$205.7\ k - 106.4\ k = 99.3\ k$ (tensile force)

Column Web Yielding

Unstiffened column capacity

Assume not at top of column ($C_t = 1$)

$$\phi R_n = \phi\left[C_t\left(6k_c + 2t_p + N\right)\right]F_{yc}t_{wc}$$

$$N = t_{bf} + 0.707a_w = 0.685 \text{ in} + 0.707(0.4375 \text{ in}) = 0.9943 \text{ in}$$

$$\phi R_n = (1)[(1)\{6(1.31 \text{ in}) + 2(1 \text{ in}) + 0.9943 \text{ in}\}](50 \text{ ksi})(0.440 \text{ in})$$

$$= 238.8 \text{ k} > F_{fu} = 205.7 \text{ k}$$

Stiffeners not required

Column Web Compression Buckling

This check must be made only if beams frame into column on two sides.

Unstiffened column capacity

$$h = \left(\frac{h}{t_w}\right)_c t_{wc} = (25.9)(0.440 \text{ in}) = 11.40 \text{ in}$$

$$\phi R_n = \frac{\phi 24 t_{wc}^3 \sqrt{EF_{yc}}}{h} = \frac{0.9(24)(0.440 \text{ in})^3\sqrt{(29{,}000 \text{ ksi})(50 \text{ ksi})}}{11.40 \text{ in}}$$

$$= 194.4 \text{ k} < F_{uf} = 205.7 \text{ k}$$

Stiffeners are required

$207.7 \text{ k} - 194.4 \text{ k} = 11.3 \text{ k}$ (compressive force)

Column Web Crippling

Unstiffened column capacity

$$\phi R_n = \phi 0.80 t_{wc}^2 \left[1 + 3\left(\frac{N}{d_c}\right)\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right]\sqrt{\frac{EF_{yc}t_{fc}}{t_{wc}}}$$

$$= 0.75(0.80)(0.440\text{ in})^2 \times$$

$$\left[1 + 3\left(\frac{0.9943\text{ in}}{14.0\text{ in}}\right)\left(\frac{0.440\text{ in}}{0.710\text{ in}}\right)^{1.5}\right]\sqrt{\frac{(29{,}000\text{ ksi})(50\text{ ksi})(0.710\text{ in})}{0.440\text{ in}}}$$

$$= 196.2\text{ k} < F_{uf} = 205.7\text{ k}$$

Stiffeners are required

$205.7\text{ k} - 196.2\text{ k} = 9.5\text{ k}$ (compressive force)

Column Panel Zone

AISC 360 – Section J10.6

$$P_c = P_y = F_y A_c = (50\text{ ksi})(26.5\text{ in}^2) = 1{,}325\text{ k}$$

$V_r = P_{uf} - V_{us} = \frac{M_u}{d_c} = \frac{4{,}200\text{ k}\cdot\text{in}}{14.0\text{ in}} - 0 = 300.0\text{ k}$ (neglect story shear)

$$0.4P_c = 530.0\text{ k} < P_r = 600\text{ k}$$

$$\phi R_n = \phi 0.6 F_y d_c t_{wc}\left(1.4 - \frac{P_r}{P_c}\right)$$

$$= 0.9(0.6)(50\text{ ksi})(14.0\text{ in})(0.440\text{ in})\left(1.4 - \frac{600\text{ k}}{1{,}325\text{ k}}\right)$$

$$= 157.0\text{ k} < 285.7\text{ k}$$

Panel zone shear strength is not adequate

$$V_{u,dp} = 300.0\text{ k} - 157.0\text{ k} = 143.0\text{ k}$$

Stiffener Design Requirements

Stiffeners must be designed to resist the difference between the required strength and the available strength.

In this example 99.3 k tension and 11.3 k compression. Panel zone must be reinforced additional force of 143 k.

Design requirements for tension are found in J4.1 and J10.8.

Design requirements for tension are found in J4.4 and J10.8.

Stiffener Design Requirements

Stiffeners in tension must be welded to the loaded flange and the web.

Stiffeners in compression must either bear on or be welded to the loaded flange; they must also be welded to the web.

Stiffener Design Requirements

Prescriptive requirements:

$$b_s + \frac{t_{wc}}{2} \geq \frac{b_{fc}}{3}$$

$$t_s \geq \frac{t_{wc}}{2} \geq \frac{b_s}{16}$$

$l_s \geq \frac{d_c}{2}$ except as required by section J10.5 and J10.7

$b_s \rightarrow$ stiffer width

$t_s \rightarrow$ stiffer thickness

$t_s \rightarrow$ stiffer length

Stiffener Design – Tension

$$R_{u,dp} = 99.3 \text{ k}$$

Tensile yield

$$A_{st,min} = \frac{R_{u,st}}{\phi F_{y,st}} = \frac{99.3 \text{ k}}{0.9(36 \text{ ksi})} = 3.06 \text{ in}^2$$

Tensile rupture

$$A_{st,min} = \frac{R_{u,st}}{\phi F_{y,st}} = \frac{99.3 \text{ k}}{0.75(58 \text{ ksi})} = 2.28 \text{ in}^2$$

Try two plates $3\frac{1}{4}" \times \frac{1}{2}" \times 7$“ – A36 – partial depth

$$A_{st} = 2(3.25 \text{ in})(0.5 \text{ in}) = 3.25 \text{ in}^2 > 3.06 \text{ in}^2$$

Stiffener Design – Compression

$R_{u,dp} = 11.3\text{ k}$

$\frac{KL}{r} = \frac{(1)(7)}{0.5/\sqrt{12}} = 48.5 > 25$ ∴ Use Chapter E

$\frac{b}{t} = \frac{3.25}{0.5} = 7.5$

$\lambda_r = 0.45\sqrt{\frac{E}{F_y}} = 0.45\sqrt{\frac{29{,}500}{36}} = 12.9 > 7.5$ ∴ Not slender

$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}500}{36}} = 135.0 > 48.5$

Stiffener Design – Compression

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}500)}{(48.5)^2} = 123.7\text{ ksi}$

$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{36}{123.7}}\right](36\text{ ksi}) = 31.9\text{ ksi}$

$P_n = F_{cr}A_g = (31.9\text{ ksi})(0.5)(3.25) = 51.83\text{ k}$

$\phi P_n = 0.9(51.83\text{ k}) = 46.64\text{ k} > 11.3\text{ k} \therefore \text{OK}$

Stiffener Design

$b_s + \frac{t_{wc}}{2} \geq \frac{b_{fc}}{3}$; $3.25 + \frac{0.440}{2} = 3.47 \text{ in} < \frac{14.5}{3} = 4.833 \text{ in}$

No good; Use $b_s = 4.75 \text{ in}$

$t_s \geq \frac{t_{wc}}{2} \geq \frac{b_s}{16}$; $0.5 \text{ in} > \frac{0.440}{2} = 0.220 \text{ in}$; $0.5 \text{ in} > \frac{4.75}{16} = 0.297 \text{ in}$

$l_s \geq \frac{d_c}{2}$; $7 \text{ in} = \frac{14.0}{2}$

Try two plates $4\frac{3}{4}$" × $\frac{1}{2}$" – A36 – full depth of column by Section J10.5

Recheck design using new dimension

Double Plate Design

Double plate design requirements for shear are covered in Section J10.9 and Chapter G

Required Capacity = 143.0 k

$$t_p \geq \frac{V_{u,dp}}{0.9 \times 0.6 F_{y,dp} d_c} = \frac{143.0 \text{ k}}{0.9(0.6)(36 \text{ ksi})(14 \text{ in})}$$

$$= 0.5254 \text{ in}$$

Use 5/8” plate

Questions

Thank you!