DESIGN EXAMPLES

Version 13.1

AMERICAN INSTITUTE
OF
STEEL CONSTRUCTION

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Printed in the United States of America
Revision: May 2010

PREFACE

The AISC *Design Examples*, V. 13.1, is an update of the original V. 13.0 and provides examples of the application of the 2005 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-05) and the AISC *Steel Construction Manual*, 13th Edition. The examples illustrate how the Specification and Manual can be used to determine solutions to common engineering problems efficiently, and outline the background to many of the tabulated values found in the Manual.

The design examples are not stand-alone documents. They are intended to be used in conjunction with the Specification, its Commentary, and the Manual.

Part I of these examples is organized to correspond with the organization of the Specification. The Chapters titles match the corresponding chapters in the Specification.

Part II is devoted primarily to connection examples that draw on the tables from the Manual, recommended design procedures, and the breadth of the Specification. The chapters of Part II are labeled II-A, II-B, II-C, etc.

Part III addresses aspects of design that are linked to the performance of a building as a whole. This includes coverage of lateral stability and second order analysis, illustrated through a four-story braced-frame and moment-frame building.

The Design Examples are arranged with LRFD and ASD designs presented side by side, for consistency with the *Manual*. Design with ASD and LRFD are based on the same nominal strength for each element so that the only differences between the approaches are which set of load combinations from ASCE 7 are used for design and whether the resistance factor for LRFD or the safety factor for ASD is used.

CONVENTIONS

The following conventions are used throughout these examples:

1. The 2005 AISC *Specification for Structural Steel Buildings* is referred to as the Specification and the AISC *Steel Construction Manual,* 13th Edition, is referred to as the Manual.

2. The source of equations or tabulated values taken from the Specification or Manual is noted along the right-hand edge of the page.

3. When the design process differs between LRFD and ASD, the designs equations are presented side-by-side. This rarely occurs, except when the resistance factor, ϕ, and the safety factor, Ω, are applied.

4. The results of design equations are presented to 3 significant figures throughout these calculations.

ACKNOWLEDGEMENTS

The AISC Committee on Manuals reviewed and provided input on the *Design Examples*, V. 13.1:

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The AISC Committee on Manuals gratefully acknowledges the contributions of the primary author, Charles R. Page, P.E., and the following individuals who assisted in the development of this document:

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The Committee also gratefully acknowledges the contributions of the following individuals who provided technical reviews of this document.

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CHAPTER IIA SIMPLE SHEAR CONNECTIONS

CHAPTER A
GENERAL PROVISIONS

GENERAL PROVISIONS

A1. SCOPE

All of the examples on this CD are intended to illustrate the application of the 2005 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-05) and the AISC *Steel Construction Manual*, 13th Edition in low-and moderate-seismic applications, (i.e. with R equal to or less than 3). For information on design applications involving R greater than 3, the AISC *Seismic Design Manual*, available at www.aisc.org, should be consulted.

A2. REFERENCED SPECIFCATIONS, CODES AND STANDARDS

Section A2 includes a detailed list of the specifications, codes and standards referenced throughout the Specification.

A3. MATERIAL

Section A3 includes a list of the steel materials that are approved for use with the Specification. The complete ASTM standards for the most commonly used steel materials can be found in *Selected ASTM Standards for Structural Steel Fabrication,* available at www.aisc.org.

CHAPTER B
DESIGN REQUIREMENTS

B1. GENERAL PROVISIONS

B2. LOADS AND LOAD COMBINATIONS

In the absence of an applicable building code, the default load combinations to be used with this Specification are those from ASCE/SEI 7-05.

B3. DESIGN BASIS

Chapter B of the Specification and Part 2 of the Manual describe the basis of design, for both LRFD and ASD.

This Section describes three basic types of connections: Simple Connections, Fully Restrained (FR) Moment Connections, and Partially Restrained (PR) Moment Connections. Several examples of the design of each of these types of connection are given in Part II of these design examples.

Information on the application of serviceability and ponding criteria may be found in Specification Chapter L, and its associated commentary. Design examples and other useful information on this topic are given in AISC Design Guide 3, *Serviceability Design Consideration for Steel Buildings, Second Edition.*

Information on the application of fire design criteria may be found in Specification Appendix 4, and its associated commentary. Design examples and other useful information on this topic are presented AISC Design Guide19, *Fire Resistance of Structural Steel Framing*.

Corrosion protection and fastener compatibility are discussed in Chapter 2 of the Manual.

B4. CLASSIFICATION OF SECTIONS FOR LOCAL BUCKLING

Specification Table B4.1 gives the complete list of limiting width-thickness ratios for all compression and flexural members defined by the Specification.

Except for one section, the W-shapes presented in the compression member selection tables as column sections meet the criteria as non-slender element sections. The W-shapes presented in the flexural member selection tables as beam sections meet the criteria for compact sections, except for 10 specific shapes. When non-compact or slender element members are tabulated in the design aids, local buckling criteria are accounted for in the tabulated design values.

The shapes listing and other member-design tables in the Manual also include footnoting to highlight sections that exceed local buckling limits in their most commonly available material grades. These footnotes include the following notations:

[c] Shape is slender in compression
[f] Shape exceeds compact limit for flexure
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1

CHAPTER C
STABILITY ANALYSIS AND DESIGN

C1. STABILITY DESIGN REQUIREMENTS

The Specification requires that the design account for both the stability of the structural system as a whole, and the stability of individual elements. Thus, the lateral analysis used to assess stability must include consideration of the combined effect of gravity and lateral loads, as well as member inelasticity, out-of-plumbness, out-of-straightness and the resulting second-order effects, P-Δ and P-δ. The effects of "leaning columns" must also be considered, as illustrated in the four-story building design example in Part III of AISC *Design Examples*.

P-Δ and P-δ effects are illustrated in Commentary Figure C-C1.1. Several methods for addressing stability, including P-Δ and P-δ effects are provided in Specification Section C2.

C2. CALCULATION OF REQUIRED STRENGTHS

The calculation of required strengths is illustrated in the four-story building design example in Part III of AISC *Design Examples*.

CHAPTER D
DESIGN OF MEMBERS FOR TENSION

INTRODUCTION

D1. SLENDERNESS LIMITATIONS

Section D1 does not establish a slenderness limit for tension members, but recommends limiting L/r to a maximum of 300. This is not an absolute requirement, and rods and hangers are specifically excluded from this recommendation.

D2. TENSILE STRENGTH

Both tensile yield strength and tensile rupture strengths must be considered for the design of tension members. It is not unusual for tensile rupture strength to govern the design of a tension member, particularly for small members with holes or heavier sections with multiple rows of holes.

For preliminary design, tables are provided in Part 5 of the Manual for W-shapes, L-shapes, WT shapes, Rectangular HSS, Square HSS, Round HSS, Pipe and 2L-shapes. The calculations in these tables for available tensile rupture strength assume an effective area, A_e, of $0.75A_g$. If the actual effective area is greater than $0.75A_g$, the tabulated values will be conservative and manual calculations can be performed to obtain higher available strengths. If the actual effective area is less than $0.75A_g$, the tabulated values will be unconservative and manual calculations are necessary to determine the available strength.

D3. AREA DETERMINATION

The gross area, A_g, is the total cross-sectional area of the member.

In computing net area, A_n, an extra $1/16$ in. is added to the bolt hole diameter and an allowance of $1/16$ in. is added to the width of slots in HSS gusset connections.

A computation of the effective area for a chain of holes is presented in **Example D.9**.

Unless all elements of the cross-section are connected, $A_e = A_n U$, where U is a reduction factor to account for shear lag. The appropriate values of U can be obtained from Table D3.1 of the Specification.

D4. BUILT-UP MEMBERS

The limitations for connections of built-up members are discussed in Section D4 of the Specification.

D5. PIN-CONNECTION MEMBERS

An example of a pin-connected member is given in **Example D.7**.

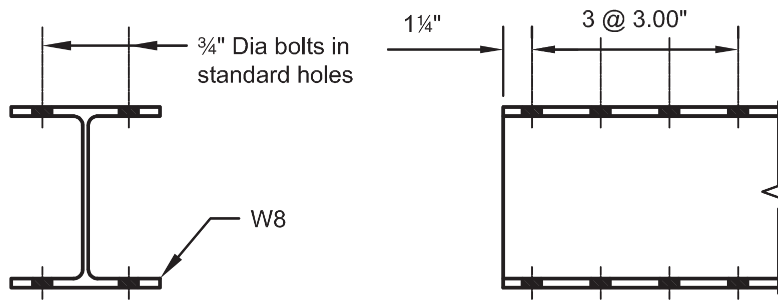
D6. EYEBARS

An example of an eyebar connection is given in **Example D.8.** The strength of an eyebar meeting the dimensional requirements of Section D6 is governed by tensile yielding of the body.

Example D.1 W-Shape Tension Member

Given:

Select an 8 in. W-shape, ASTM A992, to carry a dead load of 30 kips and a live load of 90 kips in tension. The member is 25 ft long. Verify the member strength by both LRFD and ASD with the bolted end connection shown. Verify that the member satisfies the recommended slenderness limit. Assume that connection limit states do not govern.



Solution:

Calculate the required tensile strength

LRFD	ASD
P_u = 1.2(30 kips) + 1.6(90 kips) = 180 kips	P_a = 30 kips + 90 kips = 120 kips

From Table 5-1, try a W8×21.

Material Properties:

W8×21 ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Geometric Properties:

W8×21 A_g = 6.16 in.^{2} b_f = 5.27 in. t_f = 0.400 in. d = 8.28 in. — Manual Table 1-1
r_y = 1.26 in.
$\bar{y}$ = 0.831 in. (for WT4×10.5) — Table 1-8

Check tensile yield limit state using tabulated values

LRFD	ASD
277 kips > 180 kips **o.k.**	184 kips > 120 kips **o.k.**

Manual Table 5-1

Check the available tensile rupture strength at the end connection

Verify the table assumption that $A_e / A_g \geq 0.75$ *for this connection*

Calculate U as the larger of the values from Table D3.1 case 2 or case 7

Case 2 – Check as 2 WT-shapes — Commentary Fig. C-D3.1

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.831 \text{ in.}}{9.00 \text{ in.}} = 0.908$$ — Table D3.1 Case 2

Case 7

$b_f = 5.27$ in. $d = 8.28$ in. $b_f < 2/3d$ — Table D3.1 Case 7

$U = 0.85$

Use $U = 0.908$

Calculate A_n — Section D3.2

$A_n = A_g - 4(d_h + \text{1/16 in.})t_f$
$= 6.16 \text{ in.}^2 - 4(\text{13/16 in.} + \text{1/16 in.})(0.400 \text{ in.}) = 4.76 \text{ in.}^2$

Calculate A_e — Section D3.3

$A_e = A_nU$ — Eqn. D3-1
$= 4.76 \text{ in.}^2(0.908) = 4.32 \text{ in.}^2$

$A_e/A_g = 4.32 \text{ in.}^2 / 6.16 \text{ in.}^2 = 0.701 < 0.75$; therefore, **table values for rupture are not valid.**

$P_n = F_uA_e = (65 \text{ ksi})(4.32 \text{ in.}^2) = 281$ kips — Eqn. D2-2

LRFD	ASD
$\phi_t = 0.75$ $\phi_tP_n = 0.75(281 \text{ kips}) = 211$ kips 211 kips > 180 kips **o.k.**	$\Omega_t = 2.00$ $P_n / \Omega_t = (281 \text{ kips})/2.00 = 141$ kips 141 kips > 120 kips **o.k.**

Section D2

Check the recommended slenderness limit — Section D1

$$L / r = \left(\frac{25.0 \text{ ft}}{1.26 \text{ in.}}\right)\left(\frac{12.0 \text{ in.}}{\text{ft}}\right) = 238 < 300 \quad \textbf{o.k.}$$

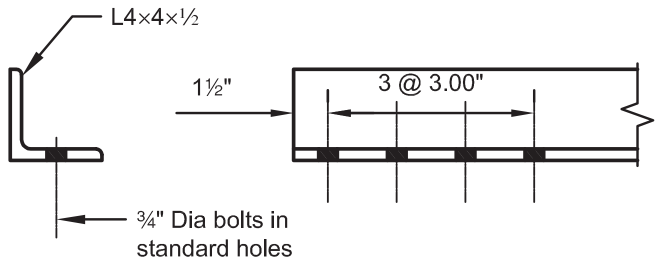
The W8×21 available tensile strength is governed by the tensile rupture limit state at the end connection.

See Chapter J for illustrations of connection limit state checks.

Example D.2 Single-Angle Tension Member

Given:

Verify, by both ASD and LRFD, the strength of an L4×4×½, ASTM A36 with one line of (4) ¾ in. diameter bolts in standard holes. The member carries a dead load of 20 kips and a live load of 60 kips in tension. Calculate at what length this tension member would cease to satisfy the recommended slenderness limit. Assume that connection limit states do not govern.



Solution:

Material Properties:

L4×4×½ ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi — Manual Table 2-3

Geometric Properties:

L4×4×½ $A_g = 3.75$ in.^{2} $r_z = 0.776$ in. $\bar{y} = 1.18$ in. $= \bar{x}$ — Manual Table 1-7

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(20 \text{ kips}) + 1.6(60 \text{ kips})$ $= 120 \text{ kips}$	$P_a = 20 \text{ kips} + 60 \text{ kips}$ $= 80.0 \text{ kips}$

Calculate the available tensile yield strength

$P_n = F_yA_g = (36\text{ksi})(3.75\text{in.}^2) = 135 \text{ kips}$ — Eqn. D2-1

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(135 \text{ kips}) = 122 \text{ kips}$	$\Omega_t = 1.67$ $P_n / \Omega_t = (135 \text{ kips})/1.67 = 80.8 \text{ kips}$

Section D2

Calculate the available tensile rupture strength

Calculate U as the larger of the values from Table D3.1 case 2 or case 8

Case 2

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.18\,\text{in.}}{9.00\,\text{in.}} = 0.869$$

Table D3.1 Case 2

Case 8 with 4 or more fasteners per line in the direction of loading

$$U = 0.80$$

Table D3.1 Case 8

Use $U = 0.869$

Calculate A_n — Section D3.2

$$A_n = A_g - (d_h + \tfrac{1}{16})t$$
$$= 3.75\text{ in.}^2 - (\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.}) = 3.31\text{ in.}^2$$

Calculate A_e — Section D3.3

$$A_e = A_n U = 3.31\text{ in.}^2(0.869) = 2.88\text{ in.}^2$$ Eqn. D3-1

$$P_n = F_u A_e = (58\text{ ksi})(2.88\text{ in.}^2) = 167\text{ kips}$$ Eqn. D2-2

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(167\text{ kips}) = 125\text{ kips}$	$\Omega_t = 2.00$ $P_n / \Omega_t = (167\text{ kips})/2.00 = 83.5\text{ kips}$

Section D2

The L4×4×½ available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$\phi_t P_n = 122\text{ kips}$ 122 kips > 120 kips **o.k.**	$P_n / \Omega_t = 80.8\text{ kips}$ 80.8 kips > 80.0 kips **o.k.**

Calculate recommended L_{max}

$$L_{max} = 300 r_z = (300)(0.776\text{ in.})\left(\frac{\text{ft}}{12.0\text{ in.}}\right) = 19.4\text{ ft}$$ Section D1

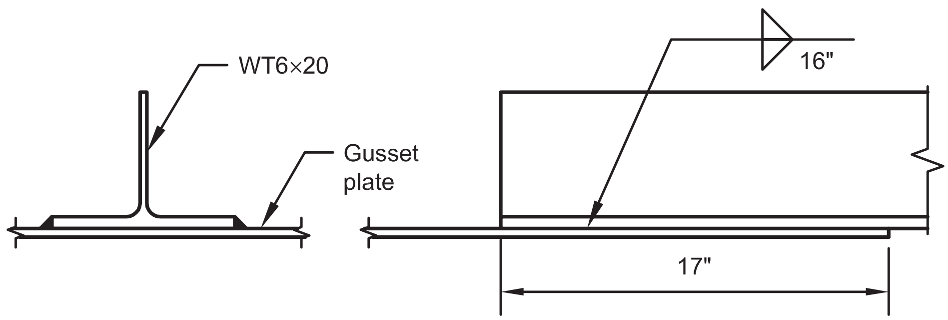
Note: The *L/r* limit is a recommendation, not a requirement.

See Chapter J for illustrations of connection limit state checks.

Example D.3 WT-Shape Tension Member

Given:

A WT6×20, ASTM A992 member has a length of 30 ft and carries a dead load of 40 kips and a live load of 120 kips in tension. Assume the end connection is fillet welded on each side for 16 in. Verify the member strength by both LRFD and ASD. Assume that the gusset plate and the weld are satisfactory.



Solution:

Material Properties:

WT6×20 ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi

Manual Table 2-3

Geometric Properties:

WT6×20 $A_g = 5.84$ in.^{2} $r_x = 1.57$ in. $\bar{y} = 1.09$ in. $= \bar{x}$ (in equation for U)

Manual Table 1-8

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(40\text{ kips}) + 1.6(120\text{ kips})$ $= 240$ kips	$P_a = 40\text{ kips} + 120\text{ kips}$ $= 160$ kips

Check tensile yielding limit state using tabulated values

LRFD	ASD
$\phi_t P_n = 263\text{ kips} > 240\text{ kips}$ **o.k.**	$P_n / \Omega_t = 175\text{ kips} > 160\text{ kips}$ **o.k.**

Manual Table 5-3

Check tensile rupture limit state using tabulated values

LRFD	ASD
$\phi_t P_n = 214\text{ kips} < 240\text{ kips}$ **n.g.**	$P_n / \Omega_t = 142\text{ kips} < 160\text{ kips}$ **n.g.**

Manual Table 5-3

The tabulated available rupture strengths may be conservative for this case, therefore calculate the exact solution.

Calculate U

Table D3.1 Case 2

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.09\text{ in.}}{16.0\text{ in.}} = 0.932$$

Section D3.2

Calculate A_n

$A_n = A_g = 5.84 \text{ in.}^2$ (because there are no reductions due to holes or notches)

Section D3.3

Calculate A_e

Eqn. D3-1

$$\begin{aligned} A_e &= A_n U \\ &= 5.84 \text{ in.}^2(0.932) = 5.44 \text{ in.}^2 \end{aligned}$$

Section D2

Calculate P_n

Eqn. D2-2

$$\begin{aligned} P_n &= F_u A_e \\ &= 65 \text{ ksi}(5.44 \text{ in}^2) = 354 \text{ kips} \end{aligned}$$

Section D2

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(354 \text{ kips}) = 266 \text{ kips}$ 266 kips > 240 kips **o.k.**	$\Omega_t = 2.00$ $P_n / \Omega_t = (354 \text{ kips})/2.00 = 177 \text{ kips}$ 177 kips > 160 kips **o.k.**

Alternately, the available tensile rupture strengths can be determined by modifying the tabulated values. The available tensile rupture strengths published in the tension member selection tables are based on the assumption that $A_e = 0.75A_g$. The actual available strengths can be determined by adjusting the table values as follows:

Manual Table 5-3

LRFD	ASD
$\phi_t P_n = 214 \text{ kips} \left(\frac{A_e}{0.75A_g}\right)$ $= 214 \text{ kips} \left(\frac{5.44 \text{ in.}^2}{0.75\left(5.84 \text{ in.}^2\right)}\right) = 266 \text{ kips}$	$P_n / \Omega_t = 142 \text{ kips} \left(\frac{A_e}{0.75A_g}\right)$ $= 142 \text{ kips} \left(\frac{5.44 \text{ in.}^2}{0.75\left(5.84 \text{ in.}^2\right)}\right) = 176 \text{ kips}$

The WT6×20 available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$\phi_t P_n = 263 \text{ kips}$ 263 kips > 240 kips **o.k.**	$P_n / \Omega_t = 175 \text{ kips}$ 175 kips > 160 kips **o.k.**

Check the non-mandatory slenderness limit

Section D1

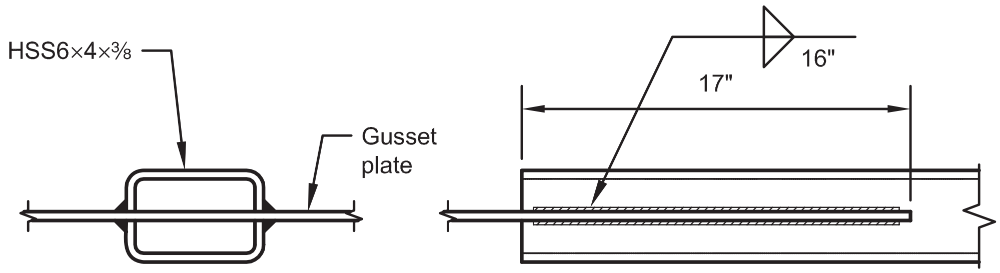
$$L / r = \left(\frac{30.0\text{ ft}}{1.57\text{ in.}}\right)\left(\frac{12.0\text{ in.}}{\text{ft}}\right) = 229 < 300 \quad \textbf{o.k.}$$

See Chapter J for illustrations of connection limit state checks.

Example D.4 Rectangular HSS Tension Member

Given:

Verify, by LRFD and ASD, the strength of an HSS6×4×⅜, ASTM A500 grade B with a length of 30 ft. The member is carrying a dead load of 35 kips and a live load of 105 kips in tension. Assume the end connection is a fillet welded ½ in. thick single concentric gusset plate with a weld length of 16 in. Assume that the gusset plate and weld are satisfactory.



Solution:

Material Properties:

HSS6×4×⅜ ASTM A500 grade B $F_y = 46$ ksi $F_u = 58$ ksi — Manual Table 2-3

Member Geometric Properties:

HSS6×4×⅜ $A_g = 6.18$ in.^{2} $r_y = 1.55$ in. $t = 0.349$ in. — Manual Table 1-11

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(35\text{ kips}) + 1.6(105\text{ kips})$ $= 210$ kips	$P_a = 35\text{ kips} + 105\text{ kips}$ $= 140$ kips

Check available tensile yield strength

LRFD	ASD
$\phi_t P_n = 256\text{ kips} > 210\text{ kips}$ **o.k.**	$P_n / \Omega_t = 170\text{ kips} > 140\text{ kips}$ **o.k.**

Manual Table 5-4

Check available tensile rupture strength

LRFD	ASD
$\phi_t P_n = 201\text{ kips} < 210\text{ kips}$ **n.g.**	$P_n / \Omega_t = 134\text{ kips} < 140\text{ kips}$ **n.g.**

Manual Table 5-4

The tabulated available rupture strengths may be conservative in this case, therefore calculate the exact solution.

Calculate U

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{(4.00\text{ in.})^2 + 2(4.00\text{ in.})(6.00\text{ in.})}{4(4.00\text{ in.} + 6.00\text{ in.})} = 1.60\text{ in.}$$

Table D3.1 Case 6

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.60\text{ in.}}{16.0\text{ in.}} = 0.900$$

Allowing for a 1⁄16 in. gap in fit-up between the HSS and the gusset plate, Section D3.2

$A_n = A_g - 2(t_p + \text{1⁄16 in.})t$

$= 6.18\text{ in.}^2 - 2(\text{½ in.} + \text{1⁄16 in.})(0.349\text{ in.}) = 5.79\text{ in.}^2$

Calculate A_e Section D3.3

$A_e = A_n U$ Eqn. D3-1

$= 5.79\text{ in.}^2(0.900) = 5.21\text{ in.}^2$

Calculate P_n Section D2

$P_n = F_u A_e$ Eqn. D2-2

$= 58\text{ ksi}(5.21\text{ in}^2) = 302\text{ kips}$

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(302\text{ kips}) = 227\text{ kips}$ 227 kips > 210 kips **o.k.**	$\Omega_t = 2.00$ $P_n / \Omega_t = (302\text{ kips})/2.00 = 151\text{ kips}$ 151 kips > 140 kips **o.k.**

Section D2

The HSS available tensile strength is governed by the tensile rupture limit state.

Check the non-mandatory slenderness limit Section D1

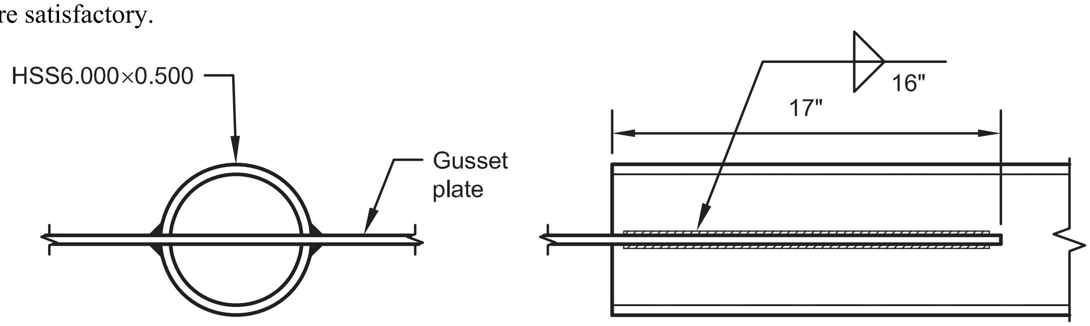
$$L / r = \left(\frac{30.0\text{ ft}}{1.55\text{ in.}}\right)\left(\frac{12.0\text{ in.}}{\text{ft}}\right) = 232 < 300 \quad \textbf{o.k.}$$

See Chapter J for illustrations of connection limit state checks.

Example D.5 Round HSS Tension Member

Given:

Verify, by LRFD and ASD, the strength of an HSS6.000×0.500, ASTM A500 grade B with a length of 30 ft. The member carries a dead load of 40 kips and a live load of 120 kips in tension. Assume the end connection is a fillet welded ½ in. thick single concentric gusset plate with a weld length of 16 in. Assume that the gusset plate and weld are satisfactory.



Solution:

Material Properties:

HSS6.000×0.500 ASTM A500 grade B F_y = 42 ksi F_u = 58 ksi

Manual Table 2-3

Member Geometric Properties:

HSS6.000×0.500 A_g = 8.09 in.^{2} r = 1.96 in. t = 0.465 in.

Manual Table 1-13

Calculate the required tensile strength

LRFD	ASD
P_u = 1.2(40 kips) + 1.6(120 kips) = 240 kips	P_a = 40 kips + 120 kips = 160 kips

Check available tensile yield strength

LRFD	ASD
$\phi_t P_n$ = 306 kips > 240 kips **o.k.**	P_n / Ω_t = 203 kips > 160 kips **o.k.**

Manual Table 5-6

Check available tensile rupture strength

LRFD	ASD
$\phi_t P_n$ = 264 kips > 240 kips **o.k.**	P_n / Ω_t = 176 kips > 160 kips **o.k.**

Manual Table 5-6

Check that $A_e \geq 0.75A_g$ *as assumed in table*

L = 16.0 in. D = 6.00 in. L/D = 16.0 in./(6.00 in.) = 2.67 > 1.3

U = 1.0

Manual Table D3.1 Case 5

Allowing for a $\frac{1}{16}$ *in. gap in fit-up between the HSS and the gusset plate,*

$A_n = A_g - 2(t_p + \frac{1}{16}\text{ in.})t$ — Section D3.3

$= 8.09\text{ in.}^2 - 2(0.500\text{ in.} + \frac{1}{16}\text{ in.})(0.465\text{ in.}) = 7.57\text{ in.}^2$

Calculate A_e

$A_e = A_nU$

$= 7.57\text{ in.}^2(1.0) = 7.57\text{ in.}^2$ — Eqn. D3-1

$A_e/A_g = 7.57\text{ in.}^2/(8.09\text{ in.}^2) = 0.936 > 0.75$ **o.k., but conservative**

Calculate available rupture strength using the Specification.

$P_n = F_uA_e$

$= (58\text{ ksi})(7.57\text{ in.}^2) = 439\text{ kips}$ — Eqn. D2-2

Section D2

LRFD	ASD
$\phi_t = 0.75$ $\phi_tP_n = 0.75(439\text{ kips}) = 329\text{ kips}$ 329 kips > 240 kips **o.k.**	$\Omega_t = 2.00$ $P_n/\Omega_t = (439\text{ kips})/2.00 = 220\text{ kips}$ 220 kips > 160 kips **o.k.**

Check the non-mandatory slenderness limit — Section D1

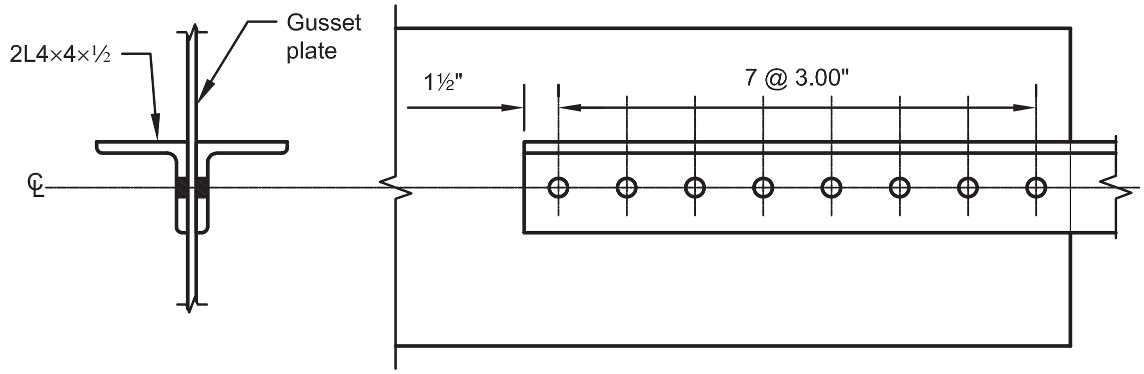
$$L/r = \left(\frac{30.0\text{ ft}}{1.96\text{ in.}}\right)\left(\frac{12.0\text{ in.}}{\text{ft}}\right) = 184 < 300 \quad \textbf{o.k.}$$

See Chapter J for illustrations of connection limit state checks.

Example D.6 Double-Angle Tension Member

Given:

A 2L4×4×½ (⅜-in. separation), ASTM A36, has one line of (8) ¾-in. diameter bolts in standard holes and is 25 ft in length. The double angle is carrying a dead load of 40 kips and a live load of 120 kips in tension. Verify the member strength by both LRFD and ASD. Assume that the gusset plate and bolts are satisfactory.



Solution:

Material Properties:

2L4×4×½ ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-3

Geometric Properties:

For a single L4×4×½ A_g = 3.75 in.^{2} $\bar{x}$ = 1.18 in.
For a 2L4×4×½ (s = ⅜ in.) r_y = 1.83 in. r_x = 1.21 in. — Manual Table 1-7, Table 1-15

Calculate the required tensile strength

LRFD	ASD
P_n = 1.2(40 kips) + 1.6(120 kips) = 240 kips	P_n = 40 kips + 120 kips = 160 kips

Calculate the available tensile yield strength

$$P_n = F_yA_g = (36 \text{ ksi})(2)(3.75\text{in.}^2) = 270 \text{ kips}$$ Eqn. D2-1

LRFD	ASD
$\phi_t = 0.90$ ϕ_tP_n = 0.90(270 kips) = 243 kips	$\Omega_t = 1.67$ P_n / Ω_t = (270 kips)/1.67 = 162 kips

Section D2

Calculate the available tensile rupture strength

Calculate U

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.18\text{ in.}}{21.0\text{ in.}} = 0.944$$ Table D3-1 Case 2

Calculate A_n Section D3.2

$$A_n = A_g - 2(d_h + \tfrac{1}{16}\text{ in.})t$$
$$= 2(3.75\text{ in.}^2) - 2(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.})(\tfrac{1}{2}\text{ in.}) = 6.63\text{ in.}^2$$

Calculate A_e Section D3.3

$$A_e = A_nU = 6.63\text{ in.}^2(0.944) = 6.26\text{ in.}^2$$ Eqn. D3-1

$$P_n = F_uA_e = (58\text{ ksi})(6.26\text{ in.}^2) = 363\text{ kips}$$ Eqn. D2-2

LRFD	ASD
$\phi_t = 0.75$ $\phi_tP_n = 0.75(363\text{ kips}) = 272\text{ kips}$	$\Omega_t = 2.00$ $P_n / \Omega_t = (363\text{ kips})/2.00 = 182\text{ kips}$

Section D2

The available strength is governed by the tensile yielding limit state.

LRFD	ASD
243 kips > 240 kips **o.k.**	162 kips > 160 kips **o.k.**

Check the non-mandatory slenderness limit Section D1

$$L/r = \left(\frac{25.0\text{ ft}}{1.21\text{ in.}}\right)\left(\frac{12.0\text{ in.}}{\text{ft}}\right) = 248 < 300 \quad \textbf{o.k.}$$

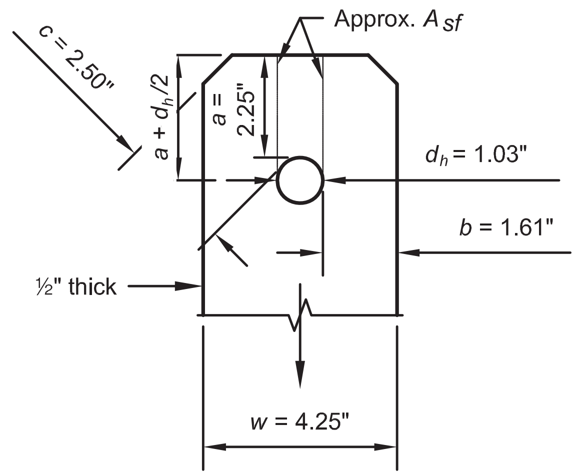
Note: The longitudinal spacing of connectors between components of built-up members should preferably limit the slenderness ratio in any component between the connectors to a maximum of 300. Section D4

See Chapter J for illustrations of connection limit state checks.

Example D.7 Pin-Connected Tension Member

Given:

An ASTM A36 pin connected tension member with the dimensions shown below carries a dead load of 12 kips and a live load of 4 kips in tension. The diameter of the pin is 1 inch, in a 1⁄32–in. oversized hole. Assume that the pin itself is adequate. Verify the strength by both LRFD and ASD.



Solution:

Material Properties:

Plate ASTM A36 F_y = 36 ksi F_u = 58 ksi Manual Table 2-4

Geometric Properties:

w = 4.25 in. t = 0.500 in. d = 1.00 in. a = 2.25 in. c = 2.50 in.
d_h = 1.03 in.

Check dimensional requirements: Section D5.2

1) $b_{eff} = 2t + 0.63$ in. = 2(0.500 in.) + 0.63 in. = 1.63 in. ≤ 1.61 in.
   b_{eff} = 1.61 in. **controls**

2) $a \geq 1.33b_{eff}$ 2.25 in. ≥ (1.33)(1.61 in.) = 2.14 in. **o.k.**

3) $w \geq 2b_{eff} + d$ 4.25 in. ≥ 2(1.61 in.) + 1.00 in. = 4.22 in. **o.k.**

4) $c \geq a$ 2.50 in. > 2.25 in. **o.k.**

Calculate the required tensile strength

LRFD	ASD
P_u = 1.2(12 kips) + 1.6(4 kips) = 20.8 kips	P_a = 12 kips + 4 kips = 16.0 kips

Calculate the available tensile rupture strength on the net effective area

$P_n = 2tb_{eff}F_u$ = (2)(0.500 in.)(1.61 in.)(58 ksi) = 93.4 kips   Eqn. D5-1

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n$ = 0.75(93.4 kips) = 70.1 kips	$\Omega_t = 2.00$ P_n / Ω_t = (93.4 kips) / 2.00 = 46.7 kips

Section D5.1

Calculate the available shear rupture strength

$A_{sf} = 2t(a + d/2)$ = 2(0.500 in.)[2.25 in. + (1.00 in. /2)] = 2.75 in.^{2} Section D5.1

$P_n = 0.6F_uA_{sf}$ = (0.6)(58 ksi)(2.75 in.^{2}) = 95.7 kips   Eqn. D5-2

LRFD	ASD
$\phi_{sf} = 0.75$ $\phi_{sf} P_n$ = 0.75(95.7 kips) = 71.8 kips	$\Omega_{sf} = 2.00$ P_n / Ω_{sf} = (95.7 kips) / 2.00 = 47.9 kips

Section D5.1

Calculate the available bearing strength

A_{pb} = 0.500 in.(1.00 in.) = 0.500 in.^{2}

$R_n = 1.8F_yA_{pb}$ = 1.8(36 ksi)(0.500 in.^{2}) = 32.4 kips   Eqn. J7-1

LRFD	ASD
$\phi = 0.75$ ϕP_n = 0.75(32.4 kips) = 24.3 kips	$\Omega = 2.00$ P_n / Ω = (32.4 kips) / 2.00 = 16.2 kips

Section J7

Calculate the available tensile yielding strength

A_g = 4.25 in. (0.500 in.) = 2.13 in.^{2} Section D2

$P_n = F_yA_g$ = 36 ksi (2.13 in.^{2}) = 76.7 kips   Eqn. D2-1

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n$ = 0.90(76.7 kips) = 69.0 kips	$\Omega_t = 1.67$ P_n / Ω_t = (76.7 kips) / 1.67 = 45.9 kips

Section D2

The available tensile strength is governed by the bearing strength limit state.

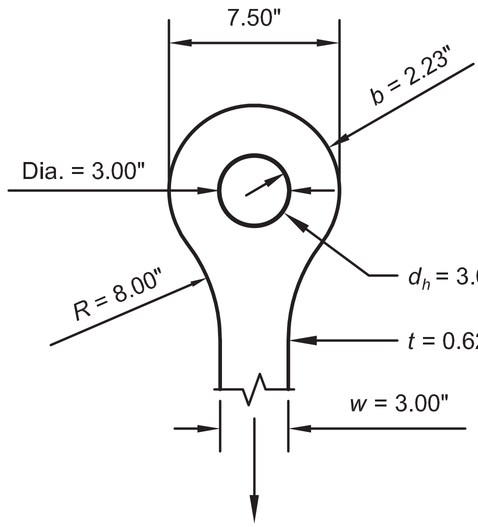
LRFD	ASD
ϕP_n = 24.3 kips 24.3 kips > 20.8 kips **o.k.**	P_n / Ω = 16.2 kips 16.2 kips > 16.0 kips **o.k.**

See Example J.6 for an illustration of the limit state calculations for a pin in a drilled hole.

Example D.8 Eyebar Tension Member

Given:

A ⅝ in. thick eyebar member as shown, ASTM A36, carries a dead load of 25 kips and a live load of 15 kips in tension. The pin diameter d is 3 in. Verify the strength by both LRFD and ASD.



Solution:

Material Properties:

Plate ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi — Manual Table 2-4

Geometric Properties:

$w = 3.00$ in. $b = 2.23$ in. $t = 0.625$ in. $d_{head} = 7.50$ in.
$d = 3.00$ in. $d_h = 3.03$ in. $R = 8.00$ in.

Check dimensional requirements — Section D6.2

1) $t \geq$ ½ in. — 0.625 in. $\geq$ 0.500 in. **o.k.**

2) $w \leq 8t$ — 3.00 in. $\leq$ 8(0.625 in.) = 5.00 in. **o.k.**

3) $d \geq 7/8w$ — 3.00 in. $\geq$ 7/8 (3.00 in.) = 2.63 in. **o.k.**

4) $d_h \leq d + 1/32$ in. — 3.03 in. $\leq$ 3.00 in. + (1/32 in.) = 3.03 in. **o.k.**

5) $R \geq d_{head}$ — 8.00 in. $\geq$ 7.50 in. **o.k.**

6) $2/3w \leq b \leq 3/4w$ — 2/3(3.00 in.) $\leq$ 2.23 in. $\leq$ 3/4(3.00 in.)
2.00 in. $\leq$ 2.23 in. $\leq$ 2.25 in. **o.k.**

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(25.0 \text{ kips}) + 1.6(15.0 \text{ kips})$ $= 54.0 \text{ kips}$	$P_a = 25.0 \text{ kips} + 15.0 \text{ kips}$ $= 40.0 \text{ kips}$

Calculate the available tensile yield strength at the eyebar body (at w)

$A_g = 3.00 \text{ in.}(0.625 \text{ in.}) = 1.88 \text{ in.}^2$

$P_n = F_yA_g = (36 \text{ ksi})(1.88 \text{ in.}^2) = 67.7 \text{ kips}$ Eqn. D2-1

Section D2

LRFD	ASD
$\phi_t = 0.90$ $\phi_tP_n = 0.90(67.7 \text{ kips}) = 60.9 \text{ kips}$ 60.9 kips > 54.0 kips **o.k.**	$\Omega_t = 1.67$ $P_n / \Omega_t = (67.7 \text{ kips})/1.67 = 40.5 \text{ kips}$ 40.5 kips > 40.0 kips **o.k.**

The eyebar tension member available strength is governed by the tension yield limit state.

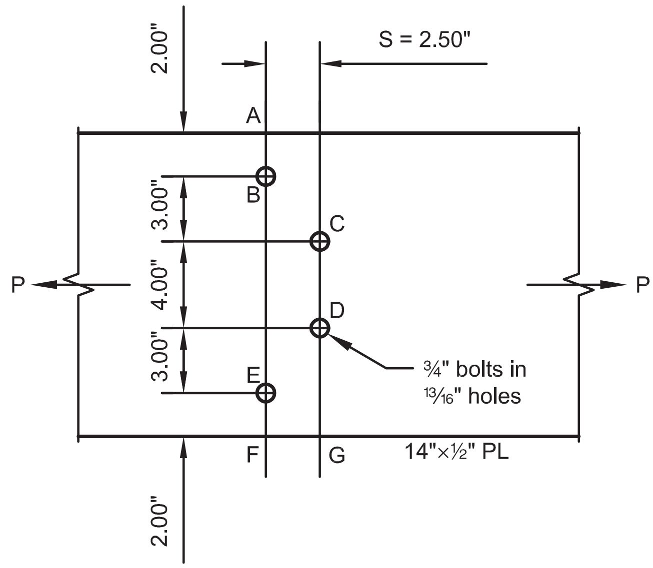
Note: The eyebar detailing limitations ensure that the tensile yielding limit state at the eyebar body will control the strength of the eyebar itself. The pin should also be checked for shear yielding, and if the material strength is less than that of the eyebar, bearing.

See Example J.6 for an illustration of the limit state calculations for a pin in a drilled hole.

Example D.9 Find A_e of a Plate with Staggered Bolts

Given:

Compute A_n and A_e for a 14 in. wide and ½ in. thick plate subject to tensile loading with staggered holes as shown.



Solution:

Calculate net hole diameter — Section D3.2

$d_{net} = d_h + 1/16 \text{ in.} = 13/16 + 1/16 \text{ in.} = 0.875 \text{ in.}$

Compute the net width for all possible paths across the plate
Because of symmetry, many of the net widths are identical and need not be calculated

$$w = 14.0 - \sum d_{net} + \sum \frac{s^2}{4g}$$

Section D3.2

Line A-B-E-F: $w = 14.0 \text{ in.} - 2(0.875 \text{ in.}) = 12.3 \text{ in.}$

Line A-B-C-D-E-F: $w = 14.0 \text{ in.} - 4(0.875 \text{ in.}) + \dfrac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})} + \dfrac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})} = 11.5 \text{ in.}$ (controls)

Line A-B-C-D-G: $w = 14.0 \text{ in.} - 3(0.875 \text{ in.}) + \dfrac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})} = 11.9 \text{ in.}$

Line A-B-D-E-F: $w = 14.0 \text{ in.} - 3(0.875 \text{ in.}) + \dfrac{(2.50 \text{ in.})^2}{4(7.00 \text{ in.})} + \dfrac{(2.50 \text{ in.})^2}{4(3.00 \text{ in.})} = 12.1 \text{ in.}$

Therefore, $A_n = (11.5 \text{ in.})(0.500 \text{ in.}) = 5.75 \text{ in.}^2$

Calculate U

Since tension load is transmitted to all elements by the fasteners

$U = 1.0$ — Table D3.1 Case 1

$A_e = A_n(1.0) = (5.75 \text{ in.}^2)(1.0) = 5.75 \text{ in.}^2$ — Eqn. D3-1

CHAPTER E
DESIGN OF MEMBERS FOR COMPRESSION

This chapter covers the design of compression members, the most common of which are columns. The Manual includes design tables for the following compression member types in their most commonly available grades:

- wide-flange column shapes
- HSS
- double angles
- single angles

LRFD and ASD information is presented side by side for quick selection, design or verification. All of the tables account for the reduced strength of sections with slender elements.

The design and selection method for both LRFD and ASD designs is similar to that of previous specifications, and will provide similar designs. In the new Specification, ASD and LRFD will provide identical designs when the live load is approximately three times the dead load.

The design of built-up shapes with slender elements can be tedious and time consuming, and it is recommended that standard rolled shapes be used, when possible.

E1. GENERAL PROVISIONS

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n/Ω_c, are determined as follows:

P_n = nominal compressive strength based on the controlling buckling mode

$\phi_c = 0.90$ (LRFD) $\qquad \Omega_c = 1.67$ (ASD)

Because F_{cr} is used extensively in calculations for compression members, it has been tabulated in Table 4-22 for all of the common steel yield strengths.

E2. SLENDERNESS LIMITATIONS AND EFFECTIVE LENGTH

In this edition of the Specification, there is no limit on slenderness, KL/r. Per the Commentary, it is recommended that KL/r not exceed 200, as a practical limit based on professional judgment and construction economics.

Although there is no restriction on the unbraced length of columns, the tables of the Manual are stopped at common or practical lengths for ordinary usage. For example, a double 3×3×¼ angle, with a ⅜-in. separation has an r_y of 1.25 in. At a KL/r of 200, this strut would be 20'-10" long. This is thought to be a reasonable limit based on fabrication and handling requirements.

Throughout the Manual, shapes that contain slender elements when supplied in their most common material grade are footnoted with the letter "c". For example, see a W14×22[c].

E3. COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Non-slender (compact and non-compact) sections, including non-slender built-up I-shaped columns and non-slender HSS columns, are governed by these provisions. The general design curve for critical stress versus KL/r is shown in Figure E-1.

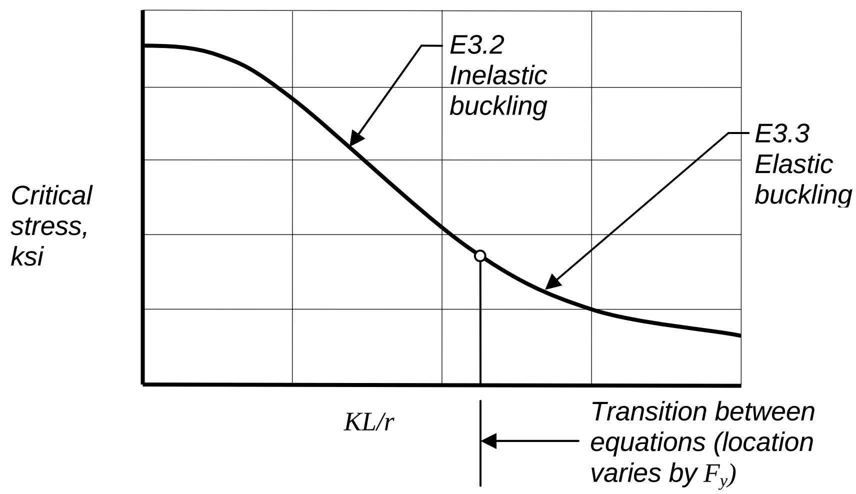


Figure E-1 Standard Column Curve

TRANSITION POINT LIMITING VALUES OF *KL*/*r*		
F_y **ksi (MPa)**	**Limiting *KL*/*r***	**0.44**F_y **ksi (MPa)**
36 (248)	134	15.8 (109)
50 (345)	113	22.0 (152)
60 (414)	104	26.4 (182)
70 (483)	96	30.8 (212)

BRACING AND BRACE POINTS

The term *L* is used throughout this chapter to describe the length between points that are braced against lateral and/or rotational displacement.

E4. COMPRESSIVE STRENGTH FOR TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section is most commonly applicable to double angles and WT sections, which are singly symmetric shapes subject to torsional and flexural-torsional buckling. The available strength in axial compression of these shapes is tabulated in Part 4 of the Manual and examples on the use of these tables have been included in this chapter, for the shapes with $KL_z = KL_y$.

E5. SINGLE ANGLE COMPRESSION MEMBERS

The available strength of single-angle compression members is tabulated in Part 4 of the Manual.

E6. BUILT-UP MEMBERS

There are no tables for built-up shapes in the Manual, due to the number of possible geometries. This section suggests the selection of built-up members without slender elements, thereby making the analysis relatively straightforward.

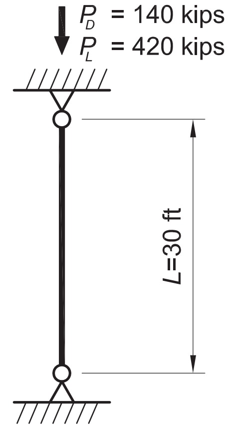
E7. MEMBERS WITH SLENDER ELEMENTS

The design of these members is similar to members without slender elements except that the formulas are modified by a reduction factor for slender elements, Q. Note the similarity of Equation E7-2 with Equation E3-2, and the similarity of Equation E7-3 with Equation E3-3.

The Tables of Part 4 incorporate the appropriate reductions in available strength to account for slender elements.

Design examples have been included in this Chapter for built-up I-shaped members with slender webs and slender flanges. Examples have also been included for a double angle, WT, and an HSS shape with slender elements.

Example E.1a W-Shape Column Design with Pinned Ends



Given:

Select an ASTM A992 (F_y = 50 ksi) W-shape column to carry an axial dead load of 140 kips and live load of 420 kips. The column is 30 feet long, and is pinned top and bottom in both axes. Limit the column size to a nominal 14 in. shape.

Solution:

Calculate the required strength

LRFD	ASD
P_u = 1.2(140 kips) + 1.6(420 kips) = 840 kips	P_a = 140 kips + 420 kips = 560 kips

Select a column using Manual Table 4-1

For a pinned-pinned condition, K = 1.0 Commentary Table C-C2.2

Because the unbraced length is the same in both the x-x and y-y directions and r_x exceeds r_y for all W-shapes, y-y axis bucking will govern.

Enter the table with an effective length, KL_y, of 30 ft, and proceed across the table until reaching the least weight shape with an available strength that equals or exceeds the required strength. Select a W14×132.

Table 4–1 (continued)
Available Strength in
Axial Compression, kips
W Shapes

F_y = 50 ksi

W14

Shape	W14×											
Wt/ft	145		132		120		109		99		90	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
ive length KL (ft) with respect to least radius of gyration r_y												
0	1280	1920	1160	1740	1060	1590	959	1440	872	1310	792	1190
6	1250	1870	1130	1700	1030	1550	934	1400	849	1280	771	1160
7	1240	1860	1120	1680	1020	1530	924	1390	840	1260	763	1150
8	1220	1840	1110	1660	1010	1510	914	1370	831	1250	754	1130
9	1210	1820	1090	1640	995	1500	902	1360	820	1230	745	1120
10	1200	1800	1080	1620	981	1470	889	1340	808	1210	734	1100
11	1180	1770	1060	1590	965	1450	875	1320	795	1200	722	1090
12	1160	1740	1040	1570	949	1430	860	1290	781	1170	709	1070
13	1140	1720	1020	1540	931	1400	844	1270	767	1150	696	1050
14	1120	1690	1000	1510	912	1370	827	1240	751	1130	682	1020
15	1100	1650	982	1480	893	1340	809	1220	734	1100	667	1000
16	1080	1620	959	1440	872	1310	790	1190	717	1080	651	978
17	1050	1580	936	1410	851	1280	771	1160	699	1050	635	954
18	1030	1550	912	1370	829	1250	751	1130	681	1020	618	928
19	1000	1510	887	1330	806	1210	730	1100	662	995	600	902
20	979	1470	862	1300	783	1180	709	1070	642	966	583	876
22	926	1390	809	1220	735	1100	665	1000	602	906	546	821
24	871	1310	756	1140	685	1030	620	932	562	844	509	765
26	815	1230	702	1050	636	956	575	864	520	782	471	708
28	759	1140	647	973	586	881	530	797	479	720	434	652
30	702	1060	594	892	537	807	485	730	438	659	397	596

The available strengths in axial compression for a y-y axis effective length of 30 ft are:

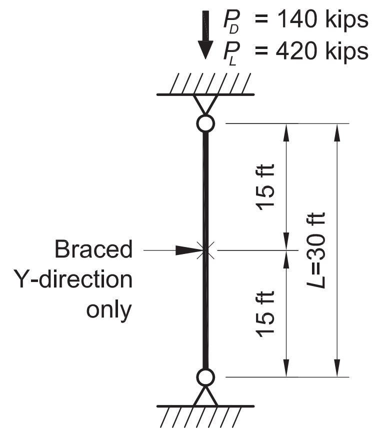
LRFD	ASD
$\phi_c P_n$ = 892 kips > 840 kips **o.k.**	P_n / Ω_c = 594 kips > 560 kips **o.k.**

Manual Table 4-1

Example E.1b W-Shape Column Design with Intermediate Bracing

Given:

Redesign the column from Example E.1a assuming the column is laterally braced about the y-y axis and torsionally braced at the midpoint.



Solution:

Calculate the required strength

LRFD	ASD
P_u = 1.2(140 kips) + 1.6(420 kips) = 840 kips	P_a = 140 kips + 420 kips = 560 kips

Select a column using Manual Table 4-1.

For a pinned-pinned condition, K = 1.0 Commentary Table C-C2.2

Since the unbraced lengths differ in the two axes, select the member using the y-y axis then verify the strength in the x-x axis.

Enter Table 4-1 with a y-y axis effective length, KL_y, of 15 ft and proceed across the table until reaching a shape with an available strength that equals or exceeds the required strength. Try a W14×90. A 15 ft long W14×90 provides an available strength in the y-y direction of

LRFD	ASD
$\phi_c P_n$ = 1000 kips	P_n / Ω_c = 667 kips

The r_x / r_y ratio for this column, shown at the bottom of Manual Table 4-1, is 1.66. The equivalent y-y axis effective length for strong axis buckling is computed as

$$KL = \frac{30.0\text{ ft}}{1.66} = 18.1\text{ ft}$$

DESIGN OF COMPRESSION MEMBERS 4–11

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

$F_y = 50$ ksi — W14

Shape	W14×											
Wt/ft	145		132		120		109		99		90	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
0	1280	1920	1160	1740	1060	1590	959	1440	872	1310	792	1190
6	1250	1870	1130	1700	1030	1550	934	1400	849	1280	771	1160
7	1240	1860	1120	1680	1020	1530	924	1390	840	1260	763	1150
8	1220	1840	1110	1660	1010	1510	914	1370	831	1250	754	1130
9	1210	1820	1090	1640	995	1500	902	1360	820	1230	745	1120
10	1200	1800	1080	1620	981	1470	889	1340	808	1210	734	1100
11	1180	1770	1060	1590	965	1450	875	1320	795	1200	722	1090
12	1160	1740	1040	1570	949	1430	860	1290	781	1170	709	1070
13	1140	1720	1020	1540	931	1400	844	1270	767	1150	696	1050
14	1120	1690	1000	1510	912	1370	827	1240	751	1130	682	1020
15	1100	1650	982	1480	893	1340	809	1220	734	1100	667	1000
16	1080	1620	959	1440	872	1310	790	1190	717	1080	651	978
17	1050	1580	936	1410	851	1280	771	1160	699	1050	635	954
18	1030	1550	912	1370	829	1250	751	1130	681	1020	618	928
19	1000	1510	887	1330	806	1210	730	1100	662	995	600	902
20	979	1470	862	1300	783	1180	709	1070	642	966	583	876

vith respect to least radius of gyration r_y

From the table, the available strength of a W14×90 with an effective length of 18 ft is

LRFD	ASD
$\phi_c P_n$ = 928 kips > 840 kips **o.k.**	P_n/Ω_c = 618 kips > 560 kips **o.k.**

Manual Table 4-1

The available compression strength is governed by the *x-x* axis flexural buckling limit state.

The available strengths of the columns described in Examples E.1a and E.1b are easily selected directly from the Manual Tables. The available strengths can also be verified by hand calculations, as shown in the following Examples E.1c and E.1d.

Example E.1c W-Shape Available Strength Calculation

Calculate the available strength of a W14×132 column with unbraced lengths of 30 ft in both axes. The material properties and loads are as given in Example E.1a.

Material properties:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi — Manual Table 2-3

Geometric Properties:

W14×132 $A_g = 38.8$ in.^{2} $r_x = 6.28$ in. $r_y = 3.76$ in. — Manual Table 1-1

Calculate the available strength — Commentary Table C-C2.2

For a pinned-pinned condition, $K = 1.0$

Since the unbraced length is the same for both axes, the *y-y* axis will govern.

$$\frac{K_y L_y}{r_y} = \frac{1.0(30.0\text{ ft})}{3.76\text{ in.}}\frac{12.0\text{ in}}{\text{ft}} = 95.7$$

For $F_y = 50$ ksi, the available critical stresses, $\phi_c F_{cr}$ and F_{cr}/Ω_c for $KL/r = 95.7$ are interpolated from Manual Table 4-22 as

LRFD	ASD
$\phi_c F_{cr} = 23.0$ ksi $\phi_c P_n = 38.8\text{ in.}^2(23.0\text{ ksi})$ $= 892\text{ kips} > 840\text{ kips}$ **o.k.**	$F_{cr}/\Omega_c = 15.3$ ksi $P_n/\Omega_c = 38.8\text{ in.}^2(15.3\text{ ksi})$ $= 594\text{ kips} > 560\text{ kips}$ **o.k.**

Manual Table 4-22

Note that the calculated values match the tabulated values.

Example E.1d W-Shape Available Strength Calculation

Calculate the available strength of a W14×90 with a strong axis unbraced length of 30.0 ft and weak axis and torsional unbraced lengths of 15.0 ft.

Material properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Geometric Properties:

W14x90 A_g = 26.5 in.^{2} r_x = 6.14 in. r_y = 3.70 in. — Manual Table 1-1

Check both slenderness ratios

$K = 1.0$ — Commentary Table C-C2.2

$$\frac{KL_x}{r_x} = \frac{1.0(30.0\text{ ft})}{6.14\text{ in.}}\frac{12\text{ in.}}{\text{ft}} = 58.6 \qquad \textbf{governs}$$

$$\frac{KL_y}{r_y} = \frac{1.0(15.0\text{ ft})}{3.70\text{ in.}}\frac{12\text{ in.}}{\text{ft}} = 48.6$$

The available critical stresses may be interpolated from Manual Table 4-22 or calculated directly as follows.

Calculate the elastic critical buckling stress, F_e

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 29{,}000\text{ ksi}}{(58.6)^2} = 83.3\text{ ksi}$$ Eqn. E3-4

Calculate flexural buckling stress, F_{cr}

Check limit

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 113$$

Because $\frac{KL}{r} = 58.6 < 4.71\sqrt{\frac{E}{F_y}} = 113$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{50.0\text{ ksi}}{83.3\text{ ksi}}}\right]50.0\text{ ksi} = 38.9\text{ ksi}$$ Eqn. E3-2

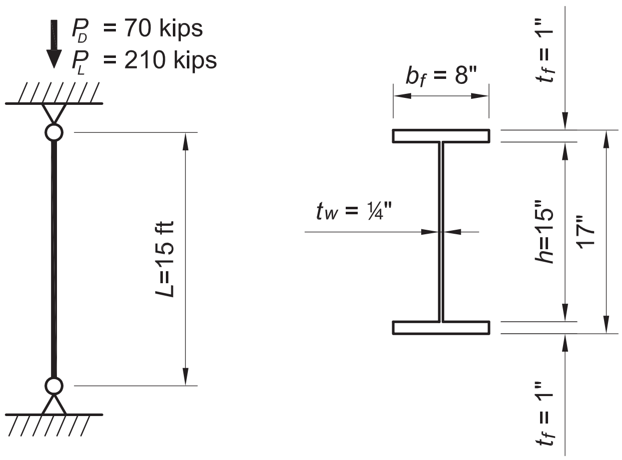
$$P_n = F_{cr}A_g = 38.9\text{ ksi}\left(26.5\text{ in}^2\right) = 1030\text{ kips}$$ Eqn. E3-1

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(1030\text{ kips})$ $= 927\text{ kips} > 840\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $P_n / \Omega_c = (1030\text{ kips}) / 1.67$ $= 617\text{ kips} > 560\text{ kips}$ **o.k.**

Example E.2 Built-up Column with a Slender Web

Given:

Verify that a built-up, ASTM A572 grade 50, column with PL1 in.×8 in. flanges and a PL¼ in.×15 in. web is sufficient to carry a dead load of 70 kips and live load of 210 kips in axial compression. The column length is 15 ft and the ends are pinned in both axes.



Solution:

Material Properties:

ASTM A572 Grade 50 F_y = 50 ksi F_u = 65 ksi

Manual Table 2-4

Geometric Properties:

Built-up Column d = 17.0 in. b_f = 8.00 in. t_f = 1.00 in. h = 15.0 in. t_w = 0.250 in.

Calculate the required strength

LRFD	ASD
P_u = 1.2(70 kips) + 1.6(210 kips) = 420 kips	P_a = 70 kips + 210 kips = 280 kips

Calculate built-up section properties (ignoring fillet welds)

$$A = 2(8.00\text{ in.})(1.00\text{ in.}) + (15.0\text{ in.})(0.250\text{ in.}) = 19.8\text{ in.}^2$$

$$I_y = \frac{2(1.00\text{ in.})(8.00\text{ in.})^3}{12} + \frac{(15.0\text{ in.})(0.250\text{ in.})^3}{12} = 85.4\text{ in.}^4$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{85.4\text{ in.}^4}{19.8\text{ in.}^2}} = 2.08\text{ in.}$$

$$I_x = \sum Ad^2 + \sum I_x$$

$$= 2\left(8.00\text{ in.}^2\right)\left(8.00\text{ in.}\right)^2 + \frac{(0.250\text{ in.})(15.00\text{ in.})^3}{12} + \frac{2(8.00\text{ in.})(1.00\text{ in.})^3}{12} = 1100\text{ in.}^4$$

Calculate the elastic flexural buckling stress

Section E7

For a pinned-pinned condition, $K = 1.0$

Commentary Table C-C2.2

Since the unbraced length is the same for both axes, the y-y axis will govern by inspection.

$$\frac{KL_y}{r_y} = \frac{1.0(15.0\text{ ft})}{2.08\text{ in.}}\frac{12.0\text{ in.}}{\text{ft}} = 86.5$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(86.5)^2} = 38.3\text{ ksi}$$

Eqn. E3-4

Calculate the elastic critical torsional buckling stress

Note: Torsional buckling generally will not govern if $KL_y \geq KL_z$, however, the check is included here to illustrate the calculation.

$$C_w = \frac{I_y h_o^2}{4} = \frac{85.4\text{ in.}^4(16.0\text{ in.})^2}{4} = 5470\text{ in.}^6$$

Design Guide No. 9 Eqn. 3-5

$$J = \sum\frac{bt^3}{3} = \frac{2(8.00\text{ in.})(1.00\text{ in.})^3 + (15.0\text{ in.})(0.250\text{ in.})^3}{3} = 5.41\text{ in}^4$$

Design Guide No. 9 Eqn. 3-4

$$F_e = \left[\frac{\pi^2 EC_w}{(K_zL)^2} + GJ\right]\frac{1}{I_x+I_y}$$

Eqn. E4-4

$$= \left[\frac{\pi^2(29{,}000\text{ ksi})(5470\text{ in.}^6)}{(180\text{ in.})^2} + (11{,}200\text{ ksi})(5.41\text{ in.}^4)\right]\frac{1}{1100\text{ in.}^4 + 85.4\text{ in.}^4}$$

$$= 91.9\text{ ksi} > 38.3\text{ ksi}$$

Therefore, the flexural buckling limit state controls.

Use $F_e = 38.3$ ksi

Check for slender elements using Table B4.1

Determine Q_s, the unstiffened element (flange) reduction factor

Section E7.1

Calculate k_c

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{15.0/0.250}} = 0.516 \text{ which is between 0.35 and 0.76}$$

Table B4.1 Note [a]

$$\frac{b}{t} = \frac{4.00\text{ in.}}{1.00\text{ in.}} = 4.0 < 0.64\sqrt{\frac{k_cE}{F_y}} = 0.64\sqrt{\frac{0.516(29{,}000\text{ ksi})}{50\text{ ksi}}} = 11.1$$

Table B4.1 Case 4

Therefore, the flange is not slender.

$Q_s = 1.0$ Eqn. E7-7

Determine Q_a, the stiffened element (web) reduction factor Section E7.2

$$\frac{h}{t} = \frac{15.0 \text{ in.}}{0.250 \text{ in.}} = 60.0 > 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 35.9$$, therefore the web is slender. Table B4.1 Case 10

$Q_a = \dfrac{A_{eff}}{A}$ where A_{eff} is the effective area based on the reduced effective width of the web, b_e. Eqn. E7-16

For equation E7-17, take f as F_{cr} with F_{cr} calculated based on $Q = 1.0$

$KL/r = 86.5$ from above

$$4.71\sqrt{\frac{E}{QF_y}} = 4.71\sqrt{\frac{29{,}000 \text{ ksi}}{1.0(50 \text{ ksi})}} = 113 > 86.5$$

When $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}}$

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y = 1.0\left[0.658^{\frac{1.0(50 \text{ ksi})}{38.3 \text{ ksi}}}\right](50 \text{ ksi}) = 29.0 \text{ ksi}$$ Eqn. E7-2

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b, \text{ where } b = h$$ Eqn. E7-17

$$= 1.92(0.250 \text{ in.})\sqrt{\frac{29{,}000 \text{ ksi}}{29.0 \text{ ksi}}}\left[1 - \frac{0.34}{(15.0 \text{ in.}/0.250 \text{ in.})}\sqrt{\frac{29{,}000 \text{ ksi}}{29.0 \text{ ksi}}}\right] \leq 15.0 \text{ in.}$$

= 12.5 in. < 15.0 in., therefore compute A_{eff} with reduced effective web width.

$$A_{eff} = b_e t_w + 2b_f t_f = (12.5 \text{ in.})(0.250 \text{ in.}) + 2(8.00 \text{ in.})(1.00 \text{ in.}) = 19.1 \text{ in.}^2$$

$$Q_a = \frac{A_{eff}}{A} = \frac{19.1 \text{ in.}^2}{19.8 \text{ in.}^2} = 0.965$$ Eqn. E7-16

$Q = Q_sQ_a = (1.00)(0.965) = 0.965$ Section E7

Determine whether Specification Equation E7-2 or E7-3 applies

$KL/r = 86.5$ from above

$$4.71\sqrt{\frac{E}{QF_y}} = 4.71\sqrt{\frac{29{,}000 \text{ ksi}}{0.965(50 \text{ ksi})}} = 115 > 86.5$$

Therefore, Specification Equation E7-2 applies.

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y = 0.965\left[0.658^{\frac{0.965(50\text{ ksi})}{38.3\text{ ksi}}}\right](50\text{ ksi}) = 28.5\text{ ksi}$$

Eqn. E7-2

Calculate the nominal compressive strength

$$P_n = F_{cr}A_g = 28.5\text{ ksi}\left(19.8\text{ in}^2\right) = 564\text{ kips}$$

Eqn. E7-1

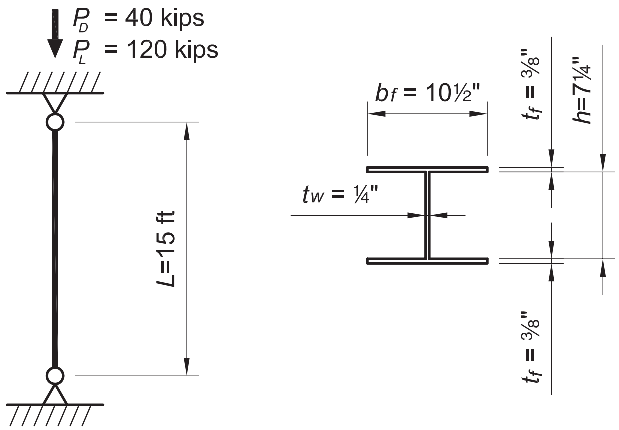
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(564\text{ kips})$ $= 508\text{ kips} > 420\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $P_n / \Omega_c = 564\text{ kips}/1.67$ $= 338\text{ kips} > 280\text{ kips}$ **o.k.**

Section E1

Example E.3 Built-up Column with Slender Flanges

Given:

Determine if a built-up, ASTM A572 grade 50 column with PL⅜ in.×10½ in. flanges and a PL¼ in.×7¼ in. web has sufficient available strength to carry a dead load of 40 kips and a live load of 120 kips in axial compression. The column unbraced length is 15.0 ft in both axes and the ends are pinned.



Solution:

Material Properties:

ASTM A572 Gr. 50 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-4

Geometric Properties:

Built-up Column d = 8.00 in. b_f = 10.5 in. t_f = 0.375 in. h = 7.25 in.
t_w = 0.250 in.

Calculate the required strength

LRFD	ASD
P_u = 1.2(40 kips) + 1.6(120 kips) = 240 kips	P_a = 40 kips + 120 kips = 160 kips

Calculate built-up section properties (ignoring fillet welds)

A = 2(10.5 in.)(0.375 in.) + (7.25 in.)(0.250 in.) = 9.69 in.^{2}

Since the unbraced length is the same for both axes, the weak axis will govern.

$$I_y = 2\left[\frac{(0.375\text{ in.})(10.5\text{ in.})^3}{12}\right] + \frac{(7.25\text{ in.})(0.250\text{ in.})^3}{12} = 72.4\text{ in.}^4$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{72.4\text{ in.}^4}{9.69\text{ in.}^2}} = 2.73\text{ in.}$$

$$I_x = 2(10.5\text{ in.})(0.375\text{ in.})(3.81\text{ in.})^2 + \frac{(0.25\text{ in.})(7.25\text{ in.})^3}{12} + \frac{2(10.5\text{ in.})(0.375\text{ in.})^3}{12}$$
$$= 122\text{ in.}^4$$

Check web slenderness

For a stiffened element (web) in a doubly symmetric I-shaped section, under uniform compression,

Table B4.1
Case 10

$$\lambda_r = 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 35.9$$

$$\frac{h}{t_w} = \frac{7.25\text{ in.}}{0.250\text{ in.}} = 29.0 < 35.9 \text{ Therefore, the web is not slender.}$$

Note that the fillet welds are ignored in the calculation of h for built up sections.

Check flange slenderness

Calculate k_c

Table B4.1
Note [a]

$$k_c = \frac{4}{\sqrt{\frac{h}{t_w}}} = \frac{4}{\sqrt{\frac{7.25\text{ in.}}{0.250\text{ in.}}}} = 0.743 \text{ where } 0.35 \le k_c \le 0.76 \textbf{ o.k.}$$

Use k_c = 0.743

For flanges of a built-up I-shaped section under uniform compression;

Table B4.1
Case 4

$$\lambda_r = 0.64\sqrt{\frac{k_cE}{F_y}} = 0.64\sqrt{\frac{0.743(29{,}000\text{ ksi})}{50\text{ ksi}}} = 13.3$$

$$\frac{b}{t} = \frac{5.25\text{ in.}}{0.375\text{ in.}} = 14.0 > 13.3 \text{ Therefore, the flanges are slender.}$$

Section E7

For compression members with slender elements, Section E7 of the Specification applies. The nominal compressive strength, P_n, shall be determined based on the limit states of flexural, torsional, and flexural-torsional buckling. Depending on the slenderness of the column, Specification Equation E7-2 or E7-3 applies. F_e is used in both equations and is calculated as the lesser of Specification Equations E3-4 and E4-4.

Commentary
Table
C-C2.2

For a pinned-pinned condition, K = 1.0.

Since the unbraced length is the same for both axes, the weak axis will govern.

$$\frac{K_yL_y}{r_y} = \frac{1.0(15.0\text{ ft})}{2.73\text{ in.}}\frac{12\text{ in.}}{\text{ft}} = 65.9$$

Calculate the elastic critical stress, F_e, for flexural buckling

Eqn. E3-4

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 29{,}000 \text{ ksi}}{(65.9)^2} = 65.9 \text{ ksi}$$

Calculate the elastic critical stress, F_e, for torsional buckling

Design Guide No. 9 Eqn 3-5

Note: This limit state is not likely to govern, but the check is included here for completeness.

Design Guide No. 9 Eqn 3-4

$$C_w = \frac{I_y h_o^2}{4} = \frac{72.4 \text{ in.}^4 (7.63 \text{ in.})^2}{4} = 1{,}050 \text{ in.}^6$$

$$J = \sum \frac{bt^3}{3} = \frac{2(10.5 \text{ in.})(0.375 \text{ in.})^3 + 7.25 \text{ in.}(0.250 \text{ in.})^3}{3} = 0.407 \text{ in.}^4$$

Eqn. E4-4

$$\begin{aligned} F_e &= \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right]\frac{1}{I_x + I_y} \\ &= \left[\frac{\pi^2 (29{,}000 \text{ ksi})(1{,}050 \text{ in.}^6)}{(180 \text{ in.})^2} + (11{,}200 \text{ ksi})(0.407 \text{ in.}^4)\right]\frac{1}{122 \text{ in.}^4 + 72.4 \text{ in.}^4} \\ &= 71.2 \text{ ksi} > 65.9 \text{ ksi} \end{aligned}$$

Therefore, use F_e = 65.9 ksi

Determine Q, the slenderness reduction factor

$Q = Q_s Q_a$, where Q_a = 1.0 because the web is not slender — Section E7.1

Calculate Q_s, the unstiffened element (flange) reduction factor

Determine the proper equation for Q_s by checking limits for Equations E7-7 to E7-9 — Section E7.1(b)

$$\frac{b}{t} = 14.0 \text{ from above}$$

$$0.64\sqrt{\frac{Ek_c}{F_y}} = 0.64\sqrt{\frac{29{,}000 \text{ ksi}(0.743)}{50 \text{ ksi}}} = 13.3$$

$$1.17\sqrt{\frac{Ek_c}{F_y}} = 1.17\sqrt{\frac{29{,}000 \text{ ksi}(0.743)}{50 \text{ ksi}}} = 24.3$$

$0.64\sqrt{\dfrac{Ek_c}{F_y}} < \dfrac{b}{t} \le 1.17\sqrt{\dfrac{Ek_c}{F_y}}$ therefore, Equation E7-8 applies.

Eqn. E7-8

$$\begin{aligned} Q_s &= 1.415 - 0.65\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{Ek_c}} \\ &= 1.415 - 0.65(14.0)\sqrt{\frac{50 \text{ ksi}}{(29{,}000 \text{ ksi})(0.743)}} = 0.977 \end{aligned}$$

$Q = Q_s Q_a$ = (0.977)(1.0) = 0.977

Calculate nominal compressive strength

$$4.71\sqrt{\frac{E}{QF_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{0.977\left(50\text{ ksi}\right)}} = 115 > 65.9$$ therefore, Specification Eqn. E7-2 applies.

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y = 0.977\left[0.658^{\frac{0.977(50\text{ ksi})}{65.9\text{ ksi}}}\right]\left(50\text{ ksi}\right) = 35.8\text{ ksi}$$

Eqn. E7-2

$$P_n = F_{cr}A_g = \left(35.8\text{ ksi}\right)\left(9.69\text{ in}^2\right) = 347\text{ kips}$$

Eqn. E7-1

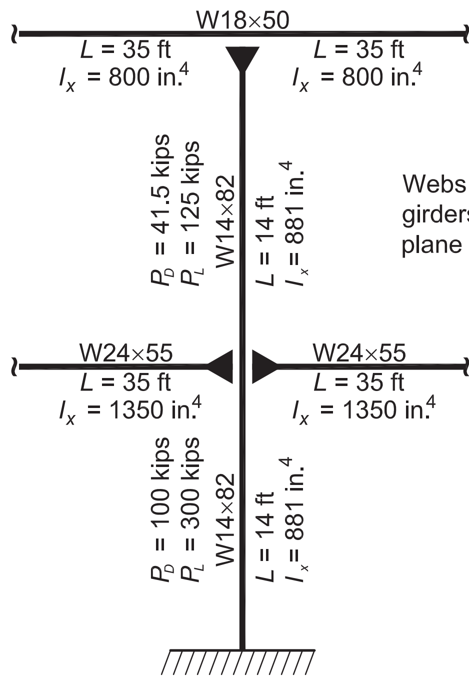
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90\left(347\text{ kips}\right)$ $= 312\text{ kips} > 240\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $P_n / \Omega_c = (347\text{ kips})/1.67$ $= 208\text{ kips} > 160\text{ kips}$ **o.k.**

Section E1

Note: Built-up sections are generally more expensive than standard rolled shapes; therefore, a standard compact shape, such as a W8×35 might be a better choice even if the weight is somewhat higher. This selection could be taken directly from Manual Table 4-1.

Example E.4a W-Shape Compression Member (Moment Frame)

This example is primarily intended to illustrate the use of the alignment chart for sidesway uninhibited columns.



Given:

The member sizes shown for the moment frame illustrated here (sidesway uninhibited in the plane of the frame) have been determined to be adequate for lateral loads. The material for both the column and the girders is ASTM A992 grade 50. The loads shown at each level are the accumulated dead loads and live loads at that story. The column is fixed at the base about the *x-x* axis of the column.

Determine if the column is adequate to support the gravity loads shown. Assume the column is continuously supported in the transverse direction (the *y-y* axis of the column).

Material Properties:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi — Manual Table 2-3

Geometric Properties:

Manual Table 1-1

W18×50 $I_x = 800$ in.^{4}

W24×55 $I_x = 1350$ in.^{4}

W14×82 $A_g = 24.0$ in.^{2} $I_x = 881$ in.^{4}

Calculate the required strength for the column between the roof and floor

LRFD	ASD
$P_u = 1.2(41.5\text{ kips}) + 1.6(125\text{ kips}) = 250\text{ kips}$	$P_a = 41.5 + 125 = 167\text{ kips}$

Calculate the effective length factor, K

LRFD	ASD
$\dfrac{P_u}{A_g} = \dfrac{250\text{ kips}}{24.0\text{ in.}^2} = 10.4\text{ ksi} < 18\text{ ksi}$	$\dfrac{P_a}{A_g} = \dfrac{167\text{ kips}}{24.0\text{ in.}^2} = 6.96\text{ ksi} < 12\text{ ksi}$

Manual

$\tau = 1.00$	$\tau = 1.00$	Table 4-21

Therefore, no reduction in stiffness for inelastic buckling will be used.

Determine G_{top} and G_{bottom}

$$G_{top} = \tau \frac{\sum (I_c / L_c)}{\sum (I_g / L_g)} = (1.00)\frac{\left(\dfrac{881 \text{ in.}^4}{14.0 \text{ ft}}\right)}{2\left(\dfrac{800 \text{ in.}^4}{35.0 \text{ ft}}\right)} = 1.38$$

Commentary C2.2

$$G_{bottom} = \tau \frac{\sum (I_c / L_c)}{\sum (I_g / L_g)} = (1.00)\frac{2\left(\dfrac{881 \text{ in.}^4}{14.0 \text{ ft}}\right)}{2\left(\dfrac{1{,}350 \text{ in.}^4}{35.0 \text{ ft}}\right)} = 1.63$$

Commentary C2.2

From the alignment chart, K is slightly less than 1.5. Because the column available strength tables are based on the KL about the y-y axis, the equivalent effective column length of the upper segment for use in the table is:

Commentary Fig. C-C2.4

$$KL = \frac{(KL)_x}{\left(\dfrac{r_x}{r_y}\right)} = \frac{1.5(14.0 \text{ ft})}{2.44} = 8.61 \text{ ft}$$

Take the available strength of the W14x82 *from Manual Table 4.1*

At KL = 9 ft, the available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 942 \text{ kips} > 250 \text{ kips}$ **o.k.**	$P_n / \Omega_c = 627 \text{ kips} > 167 \text{ kips}$ **o.k.**

Manual Table 4-1

Calculate the required strength for the column segment between the floor and the foundation

LRFD	ASD
P_u = 1.2(100 kips) + 1.6 (300 kips) = 600 kips	P_a = 100 kips + 300 kips = 400 kips

Calculate the effective length factor, K

LRFD	ASD
$\dfrac{P_u}{A_g} = \dfrac{600 \text{ kips}}{24.0 \text{ in.}^2} = 25.0 \text{ ksi}$	$\dfrac{P_a}{A_g} = \dfrac{400 \text{ kips}}{24.0 \text{ in.}^2} = 16.7 \text{ ksi}$
$\tau = 0.890$	$\tau = 0.890$
$G_{top} = \tau \dfrac{\Sigma\left(\dfrac{I}{L}\right)_c}{\Sigma\left(\dfrac{I}{L}\right)_g} = (0.890)\dfrac{2\left(\dfrac{881 \text{ in.}^4}{14.0 \text{ ft}}\right)}{2\left(\dfrac{1350 \text{ in.}^4}{35.0 \text{ ft}}\right)} = 1.45$	$G_{top} = \tau \dfrac{\Sigma\left(\dfrac{I}{L}\right)_c}{\Sigma\left(\dfrac{I}{L}\right)_g} = (0.890)\dfrac{2\left(\dfrac{881 \text{ in.}^4}{14.0 \text{ ft}}\right)}{2\left(\dfrac{1350 \text{ in.}^4}{35.0 \text{ ft}}\right)} = 1.45$

Manual Table 4-21

Commentary C2.2

$G_{bottom} = 1 \text{ (fixed)}$

Commentary

Section C2.2b

From the alignment chart, K is approximately 1.38. Because the column available strengths are based on the KL about the y-y axis, the effective column length of the lower segment for use in the table is:

$$KL = \frac{(KL)_x}{\left(\frac{r_x}{r_y}\right)} = \frac{1.38(14.0\text{ ft})}{2.44} = 7.92\text{ ft}$$

Take the available strength of the W14x82 *from Manual Table 4-1*

At L = 9 ft, (conservative) the available strength in axial compression is:

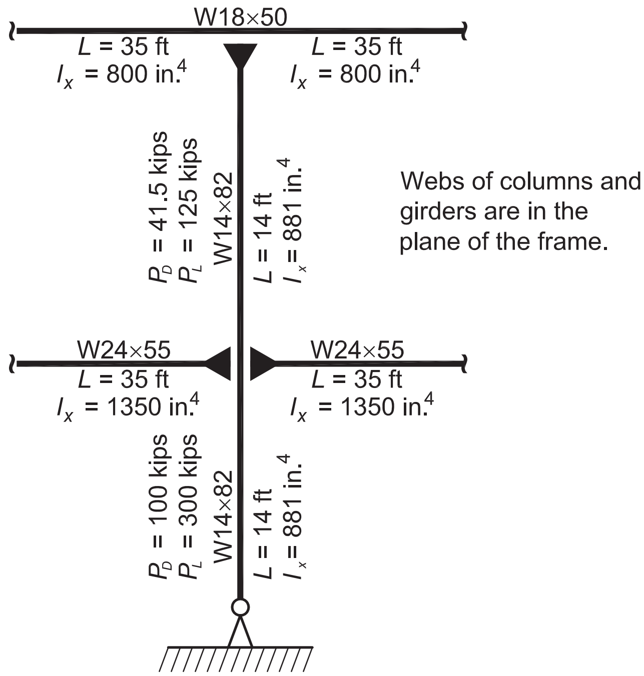
LRFD	ASD
$\phi_c P_n = 942\text{ kips} > 600\text{ kips}$ **o.k.**	$P_n / \Omega_c = 627\text{ kips} > 400\text{ kips}$ **o.k.**

Manual Table 4-1

A more accurate strength could be determined by interpolation from Manual Table 4-1.

Example E.4b W-Shape Compression Member (Moment Frame)

Determine the available strength of the column shown subject to the same gravity loads shown in Example E.4a with the column pinned at the base about the *x*-*x* axis. All other assumptions remain the same.



As determined in Example E.4a, for the column segment between the roof and the floor, the column strength is adequate.

As determined in Example E.4a, for the column segment between the floor and the foundation,
$G_{top} = 1.45$

At the base,
G_{bot} = 10 (pinned)

Commentary Section C2.2b

Note: this is the only change in the analysis.

From the alignment chart, K is approximately equal to 2.0. Because the column available strength Tables are based on the effective length, KL, about the y-y axis, the effective column length of the lower segment for use in the table is:

Commentary Figure C-C2.4

$$KL = \frac{(KL)_x}{\left(\frac{r_x}{r_y}\right)} = \frac{2.0(14.0 \text{ ft})}{2.44} = 11.5 \text{ ft}$$

Interpolate the available strength of the W14×82 *from Manual Table 4-1*

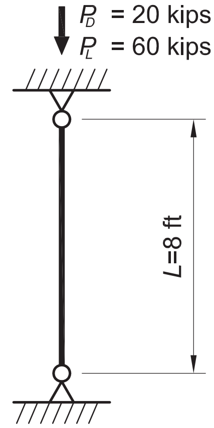
LRFD	ASD
$\phi_c P_n = 863$ kips > 600 kips **o.k.**	$P_n / \Omega_c = 574$ kips > 400 kips **o.k.**

Manual Table 4-1

Example E.5 Double Angle Compression Member without Slender Elements

Given:

Verify the strength of a 2L4×3½×⅜ LLBB (¾-in. separation) strut with a length of 8 ft and pinned ends carrying an axial dead load of 20 kips and live load of 60 kips. Also, calculate the required number of fully tightened or welded intermediate connectors required.



Material Properties:

ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-3

Geometric Properties:

2L4×3½×⅜ LLBB r_z = 0.719 in. (single angle) r_x = 1.25 in. — Manual Table 1-15

r_y = 1.55 in. for ⅜ inch separation

r_y = 1.69 in. for ¾ inch separation — Table 1-7

Calculate the required strength

LRFD	ASD
P_u = 1.2(20 kips) + 1.6(60 kips) = 120 kips	P_a = 20 kips + 60 kips = 80.0 kips

Select a column using Manual Table 4-9

K = 1.0 — Commentary Table C-C2.2

For $(KL)_x$ = 8 ft, the available strength in axial compression is taken from the upper (x-x) portion of the table as

LRFD	ASD
$\phi_c P_n$ = 127 kips > 120 kips **o.k.**	P_n / Ω_c = 84.2 kips > 80.0 kips **o.k.**

Manual Table 4-9

For buckling about the y-y axis, the values are tabulated for a separation of ⅜ in.

To adjust to a spacing of ¾ in., $(KL)_y$ is multiplied by the ratio of the r_y for a ⅜-in. separation to the r_y for a ¾-in. separation. Thus,

$$(KL)_y = 1.0(8.00 \text{ ft})\left(\frac{1.55 \text{ in.}}{1.69 \text{ in.}}\right) = 7.34 \text{ ft}$$

The calculation of the equivalent $(KL)_y$ above is a simplified approximation of Specification Section E6.1. To ensure a conservative adjustment for a ¾ in. separation, take $(KL)_y$ = 8 ft.

The available strength in axial compression is taken from the lower (*y-y*) portion of the table as:

LRFD	ASD
$\phi_c P_n = 130$ kips > 120 kips **o.k.**	$P_n / \Omega_c = 86.4$ kips > 80.0 kips **o.k.**

Manual Table 4-9

Therefore, *x-x* axis flexural buckling governs.

Determine the number of intermediate connectors required

Per Table 4-9, at least two welded or pretensioned bolted intermediate connectors are required. This can be verified as follows:

$$a = \text{distance between connectors} = \frac{(8.00 \text{ ft})\left(12 \text{ in.}/\text{ft}\right)}{3 \text{ spaces}} = 32.0 \text{ in.}$$

The effective slenderness ratio of the individual components of the built-up member based upon the distance between intermediate connectors, *a*, must not exceed three quarters of the controlling slenderness ratio of the overall built-up member.

Section E6.2

Therefore: $\dfrac{Ka}{r_i} \le \dfrac{3}{4}\left(\dfrac{Kl}{r}\right)_{\max}$

Solving for *a* gives, $a \le \dfrac{3r_i\left(\dfrac{KL}{r}\right)_{\max}}{4K}$

$$\frac{KL}{r_x} = \frac{1.0(8.00 \text{ ft})(12 \text{ in./ft})}{1.25 \text{ in.}} = 76.8 \qquad \textbf{controls}$$

$$\frac{KL}{r_y} = \frac{1.0(8.00 \text{ ft})(12 \text{ in./ft})}{1.69 \text{ in.}} = 56.8$$

Thus, $a \le \dfrac{3r_z\left(\dfrac{KL}{r}\right)_{\max}}{4K} = \dfrac{3(0.719 \text{ in.})(76.8)}{4(1.0)} = 41.4 \text{ in.} > 32.0 \text{ in.}$ **o.k.**

The available strength can be easily determined by using the tables of the Manual. Available strength values can be verified by hand calculations, as shown below.

Calculation Solution:

Geometric Properties:

2L4×3½×⅜ LLBB $A_g = 5.35 \text{ in.}^2$ $r_y = 1.69$ in. $\bar{r}_o = 2.33$ in. $H = 0.813$
$J = 0.132 \text{ in.}^4$ (single angle) $r_y = 1.05$ in. (single angle)
$\bar{x} = 0.947$ in. (single angle)

Manual Table 1-15
Table 1-7

Check for slender elements

$$\frac{b}{t} = \frac{4.0 \text{ in.}}{0.375 \text{ in.}} = 10.7$$

Table B4.1

$\lambda_r = 0.45\sqrt{E/F_y} = 0.45\sqrt{29{,}000\text{ ksi}/36\text{ ksi}} = 12.8 > 10.7$ Case 5

Therefore, there are no slender elements.

For compression members without slender elements, Specification Sections E3 and E4 apply.

The nominal compressive strength, P_n, shall be determined based on the limit states of flexural, torsional, and flexural-torsional buckling.

Section E3

Check flexural buckling about the x-x axis

$$\frac{KL}{r_x} = \frac{1.0(8.00\text{ ft})(12\text{ in./ft})}{1.25\text{ in.}} = 76.8$$

Eqn. E3-4

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(76.8)^2} = 48.5\text{ ksi}$$

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} = 134 > 76.8,\ \text{therefore}$$

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{36\text{ ksi}}{48.5\text{ ksi}}}\right](36\text{ ksi}) = 26.4\text{ ksi}\quad \textbf{controls}$$ Eqn. E3-2

Check torsional and flexural-torsional buckling Section E4

For non-slender double angle compression members, Specification Equation E4-2 applies.

F_{cry} is taken as F_{cr}, for flexural buckling about the *y-y* axis from Specification Equation E3-2 or E3-3 as applicable.

Compute the modified $\frac{KL}{r_y}$ *for built up members with fully tightened or welded connectors* Section E6

a = 96.0 in. / 3 = 32.0 in.

$r_{ib} = r_y$ (single angle) = 1.05 in.

$$\alpha = \frac{h}{2r_{ib}} = \frac{2\bar{x} + 0.750\text{in.}}{2r_y} = \frac{2(0.947\text{in.}) + 0.750\text{in.}}{2(1.05\text{ in.})} = 1.26$$

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2}$$ Eqn. E6-2

$$= \sqrt{(56.8)^2 + 0.82\frac{(1.26)^2}{(1+1.26^2)}\left(\frac{32.0\text{in.}}{1.05\text{in.}}\right)^2} = 60.8 < 134$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(60.8)^2} = 77.4\text{ ksi}$$ Eqn. E3-4

$$F_{cry} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{36\text{ ksi}}{77.4\text{ ksi}}}\right](36\text{ ksi}) = 29.6\text{ ksi}$$ Eqn. E3-2

$$F_{crz} = \frac{GJ}{A_g\bar{r}_o^2} = \frac{(11{,}200\text{ ksi})(2\text{ angles})(0.132\text{ in.}^4)}{(5.35\text{ in.})(2.33\text{ in.})^2} = 102\text{ ksi}$$ Eqn. E4-3

$$F_{cr} = \left(\frac{F_{cry}+F_{crz}}{2H}\right)\left[1-\sqrt{1-\frac{4F_{cry}F_{crz}H}{(F_{cry}+F_{crz})^2}}\right]$$ Eqn. E4-2

$$= \left(\frac{29.6\text{ ksi}+102\text{ ksi}}{2(0.813)}\right)\left[1-\sqrt{1-\frac{4(29.6\text{ ksi})(102\text{ ksi})(0.813)}{(29.6\text{ ksi}+102\text{ ksi})^2}}\right]$$

= 27.7 ksi **does not control**

$$P_n = F_{cr}A_g = (26.4\text{ ksi})5.35\text{ in.}^2 = 141\text{ kips}$$ Eqn. E4-1

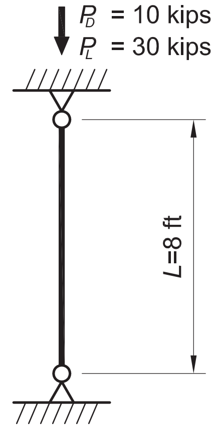
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(141\text{ kips})$ $= 127\text{ kips} > 120\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $P_n/\Omega_c = \frac{141\text{ kips}}{1.67}$ $= 84.4\text{ kips} > 80.0\text{ kips}$ **o.k.**

Section E1

Example E.6 Double Angle Compression Member with Slender Elements

Given:

Determine if a 2L5×3×¼ LLBB (¾-in. separation) strut with a length of 8 ft and pinned ends has sufficient available strength to support a dead load of 10 kips and live load of 30 kips in axial compression. Also, calculate the required number of fully tightened or welded intermediate connectors.



Material Properties:

ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-3

Geometric Properties:

2L5×3×¼ LLBB r_z = 0.652 in. (single angle) r_x = 1.62 in. — Manual Table 1-15, Table 1-7

r_y = 1.19 in. for ⅜ inch separation

r_y = 1.33 in. for ¾ inch separation

Calculate the required strength

LRFD	ASD
$P_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips}) = 60.0 \text{ kips}$	$P_a = 10 \text{ kips} + 30 \text{ kips} = 40.0 \text{ kips}$

Table Solution:

K = 1.0 — Commentary Table C-C2.2

From the upper portion of Manual Table 4-9, the available strength for buckling about the x-x axis, with $(KL)_x$ = 8 ft is — Manual Table 4-9

LRFD	ASD
$\phi_c P_{nx} = 87.2 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$P_{nx} / \Omega_c = 58.0 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

For buckling about the y-y axis, the tabulated values are based on a separation of ⅜ in. To adjust for a spacing of ¾ in., $(KL)_y$ is multiplied by the ratio of r_y for a ⅜-in. separation to r_y for a ¾-in. separation.

$$(KL)_y = 1.0(8.0 \text{ ft})\left(\frac{1.19 \text{ in.}}{1.33 \text{ in.}}\right) = 7.16 \text{ ft}$$

This calculation of the equivalent $(KL)_y$ does not completely take into account the effect of Section E6.1 and is slightly unconservative.

From the tabulated values in the lower portion of Manual Table 4-9, interpolate for a value at

$(KL)_y$ = 7.16 ft

The available strength in compression is:

LRFD	ASD
$\phi_c P_{ny}$ = 65.1 kips > 60.0 kips **o.k.**	P_{ny} / Ω_c = 43.3 kips > 40.0 kips **o.k.**

Manual Table 4-9

These strengths are approximate due to the linear interpolation from the Table and the approximate value of the equivalent $(KL)_y$ noted above. These can be compared to the more accurate values calculated in detail below.

Determine the number of intermediate connectors required.

From the tabulated values, it is determined that at least two welded or pretensioned bolted intermediate connectors are required. This can be confirmed by calculation, as follows:

Manual Table 4-9

$$a = \text{distance between connectors} = \frac{(8.00\text{ ft})(12\text{ in./ft})}{3\text{ spaces}} = 32.0\text{ in.}$$

The effective slenderness ratio of the individual components of the built-up member based upon the distance between intermediate connectors, a, must not exceed three quarters of the controlling slenderness ratio of the overall built-up member.

Section E6.2

Therefore, $\dfrac{Ka}{r_i} \le \dfrac{3}{4}\left(\dfrac{Kl}{r}\right)_{max}$

Solving for a gives, $a \le \dfrac{3r_i\left(\dfrac{KL}{r}\right)_{max}}{4K}$

$r_i = r_z = 0.652$ in.

$$\frac{KL_x}{r_x} = \frac{1.0(8.0\text{ ft})(12.0\text{ in./ft})}{1.62\text{ in.}} = 59.3$$

$$\frac{KL_y}{r_y} = \frac{1.0(8.0\text{ ft})(12.0\text{ in./ft})}{1.33\text{ in.}} = 72.2 \qquad \textbf{controls}$$

Thus, $a \le \dfrac{3r_z\left(\dfrac{KL}{r}\right)_{max}}{4K} = \dfrac{3(0.652\text{ in.})(72.2)}{4(1.0)} = 35.3\text{ in.} > 32.0\text{ in.}$ **o.k.**

The governing slenderness ratio used in the calculations of the Manual Table include the effects of the provisions of Section E6.1 and is slightly higher as a result. See below for these calculations. As a result, the maximum connector spacing calculated here is slightly conservative.

Available strength values can be verified by hand calculations, as shown below.

Calculation Solution:

Geometric Properties:

2L5×3×¼ LLBB A_g = 3.88 in.^{2} r_y = 1.33 in. $\bar{r}_o$ = 2.59 in. H = 0.657

J = 0.0438 in.^{4} (single angle) r_y = 0.853 in. (single angle)

$\bar{x}$ = 0.648 in. (single angle)

Manual Table 1-15
Table 1-7

Determine if the section is noncompact or slender

$$\frac{b}{t}=\frac{5.00\text{ in.}}{0.250\text{ in.}}=20.0$$

Table B4.1
Case 5

Calculate the limiting width-thickness ratios

$$\lambda_r=0.45\sqrt{\frac{E}{F_y}}=0.45\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}=12.8<20.0$$ Therefore the angle has a slender element.

For a double angle compression member with slender elements, Specification Section E7 applies. The nominal compressive strength, P_n, shall be determined based on the limit states of flexural, torsional, and flexural-torsional buckling. Depending on the elastic critical buckling stress, F_e, of the member, F_{cr} will be determined by Specification E7-2 or E7-3.

Determine Q, the slender element reduction factor

Section E7

$Q = Q_s(Q_a\text{=}1.0)$ for members composed of unstiffened slender elements.

Calculate Q_s for the angles individually using Specification Section E7.1c

Section
E7.1c

$$0.45\sqrt{\frac{E}{F_y}}=0.45\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}=12.8<20.0$$

$$0.91\sqrt{\frac{E}{F_y}}=0.91\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}}=25.8>20.0$$

Therefore, Specification Equation E7-11 applies.

$$Q_s=1.34-0.76\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}$$

Eqn. E7-11

$$=1.34-0.76\left(20.0\right)\sqrt{\frac{36\text{ ksi}}{29{,}000\text{ ksi}}}=0.805$$

$Q_a=1.0$ (no stiffened elements)

Therefore, $Q=Q_sQ_a=0.805\left(1.0\right)=0.805$

Determine the applicable equation for the critical stress, F_{cr}

From above, K = 1.0

Specification Equation E7-2 requires the computation of F_e. For singly symmetric members, Specification Equations E3-4 and E4-5 apply.

Check x-x axis flexural buckling

$$\frac{K_xL}{r_x} = \frac{1.0(8.0\text{ ft})(12.0\text{ in./ft})}{1.62\text{ in.}} = 59.3$$

$$F_e = \frac{\pi^2 E}{\left(\dfrac{K_xL}{r_x}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(59.3)^2} = 81.4\text{ ksi}$$ **does not govern** Eqn. E3-4

Check torsional and flexural-torsional buckling

$$\frac{K_yL}{r_y} = \frac{1.0(8.0\text{ ft})(12.0\text{ in./ft})}{1.33\text{ in.}} = 72.2$$

Section E6

Compute the modified $\dfrac{KL}{r_y}$ *for built up members with fully tightened or welded connectors*

a = 96.0 in. / 3 = 32.0 in.

$r_{ib} = r_y$ (single angle) = 0.853 in.

$$\alpha = \frac{h}{2r_{ib}} = \frac{2\bar{x} + 0.750\text{ in.}}{2r_y} = \frac{2(0.648\text{ in.}) + 0.750\text{ in.}}{2(0.853\text{ in.})} = 1.20$$

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2}$$ Eqn. E6-2

$$= \sqrt{(72.2)^2 + 0.82\frac{(1.20)^2}{(1+1.20^2)}\left(\frac{32.0\text{ in.}}{0.853\text{ in.}}\right)^2} = 76.8$$

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_yL}{r_y}\right)_m^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(76.8)^2} = 48.5\text{ ksi}$$ Eqn. E4-10

$$F_{ez} = \left(\frac{\pi^2 EC_w}{(K_zL)^2} + GJ\right)\frac{1}{A_g\bar{r}_o^2}$$ Eqn. E4-11

For double angles, omit term with C_w per User Note at end of Section E4.

$$F_{ez} = \frac{GJ}{A_g\bar{r}_o^2} = \frac{(11{,}200\text{ ksi})(2\text{ angles})(0.0438\text{ in.}^4)}{3.88\text{ in.}(2.59\text{ in.})^2} = 37.7\text{ ksi}$$

$$F_e = \left(\frac{F_{ey}+F_{ez}}{2H}\right)\left[1-\sqrt{1-\frac{4F_{ey}F_{ez}H}{(F_{ey}+F_{ez})^2}}\right]$$ Eqn. E4-5

$$= \left(\frac{48.5\text{ ksi} + 37.7\text{ ksi}}{2(0.657)}\right)\left[1-\sqrt{1-\frac{4(48.5\text{ ksi})(37.7\text{ ksi})(0.657)}{(48.5\text{ ksi}+37.7\text{ ksi})^2}}\right]$$

$= 26.6$ ksi **governs**

$0.44QF_y = 0.44(0.805)(36\text{ ksi}) = 12.8\text{ ksi} < 26.6\text{ ksi}$, therefore Equation E7-2 applies.

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y = 0.805\left[0.658^{\frac{(0.805)(36\text{ ksi})}{26.6\text{ ksi}}}\right](36\text{ ksi}) = 18.4\text{ ksi}$$

Eqn. E7-2

$$P_n = F_{cr}A_g = (18.4\text{ ksi})3.88\text{ in.}^2 = 71.4\text{ kips}$$

Eqn. E7-1

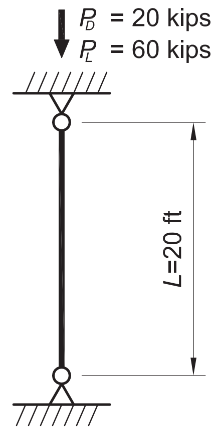
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(71.4\text{ kips})$ $= 64.3\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $P_n / \Omega_c = \dfrac{71.4\text{ kips}}{1.67}$ $= 42.8\text{ kips} > 40.0\text{ kips}$ **o.k.**

Section E1

Example E.7 Design of a WT Compression Member without Slender Elements

Given:

Select a WT-shape compression member with a length of 20 ft to support a dead load of 20 kips and live load of 60 kips in axial compression. The ends are pinned.



Because WT sections are cut from ASTM A992 W-shape beams, the material properties are:

Material Properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Calculate the required strength

LRFD	ASD
$P_u = 1.2(20\text{ kips}) + 1.6(60\text{ kips}) = 120\text{ kips}$	$P_a = 20\text{ kips} + 60\text{ kips} = 80.0\text{ kips}$

Table Solution:

K = 1.0 Therefore $(KL)_x = (KL)_y$ = 20.0 ft — Commentary Table C-C2.2

Select the lightest member from Table 4-7 with sufficient available strength about both the x-x (upper portion of the table) and the y-y axis (lower portion of the table) to support the required strength.

Try a WT7×34

The available strength in compression is: — Manual Table 4-7

LRFD	ASD
$\phi_c P_{nx} = 128\text{ kips} > 120\text{ kips}$ **Controls** **o.k.**	$P_{nx}/\Omega_c = 85.2\text{ kips} > 80.0\text{ kips}$ **Controls** **o.k.**
$\phi_c P_{ny} = 221\text{ kips} > 120\text{ kips}$ **o.k.**	$P_{ny}/\Omega_c = 147\text{ kips} > 80.0\text{ kips}$ **o.k.**

The available strength can be easily determined by using the tables of the Manual. Available strength values can be verified by hand calculations, as shown below.

Calculation Solution:

Geometric Properties: — Manual Table 1-8

WT7×34 $A_g = 9.99\text{ in.}^2$ $r_x = 1.81\text{ in.}$ $r_y = 2.46\text{ in.}$ $J = 1.50\text{ in.}^4$

$\bar{y} = 1.29\text{ in.}$ $I_x = 32.6\text{ in.}^4$ $I_y = 60.7\text{ in.}^4$

d = 7.02 in. t_w = 0.415 in. b_f = 10.0 in. t_f = 0.720 in.

Check for slender elements

$$\frac{d}{t_w}=\frac{7.02\text{ in.}}{0.415\text{ in.}}=16.9<0.75\sqrt{\frac{E}{F_y}}=0.75\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}=18.1$$ therefore, the web is not slender.

Table B4.1
Case 8

$$\frac{b_f}{2t_f}=\frac{10\text{ in.}}{2(0.720\text{ in.})}=6.94<0.56\sqrt{\frac{E}{F_y}}=0.56\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}=13.5$$ therefore, the flange is not slender.

Table B4.1
Case 3

There are no slender elements.

For compression members without slender elements, Specification Sections E3 and E4 apply. The nominal compressive strength, P_n, shall be determined based on the limit states of flexural, torsional, and flexural-torsional buckling.

Check flexural buckling about the x-x axis

Section E3

$$\frac{KL}{r_x}=\frac{1.0(20.0\text{ ft})(12\text{ in./ft})}{1.81\text{ in.}}=133$$

$$4.71\sqrt{\frac{E}{F_y}}=4.71\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}}=113<133$$ therefore, Specification Equation E3-3 applies.

$$F_e=\frac{\pi^2E}{\left(\frac{KL}{r}\right)^2}=\frac{\pi^2\left(29{,}000\text{ ksi}\right)}{\left(133\right)^2}=16.2\text{ ksi}$$

Eqn. E3-4

$F_{cr}=0.877F_e=0.877(16.2\text{ ksi})=14.2\text{ ksi}$ **controls**

Eqn. E3-3

Because the WT7×34 section does not have any slender elements, Specification Section E4 will be applicable for torsional and flexural-torsional buckling. F_{cr} will be calculated using Specification Equation E4-2.

Calculate F_{cry}

F_{cry} is taken as F_{cr} From Specification Section E3, where $\frac{KL}{r}=\frac{KL}{r_y}$

$$\frac{KL}{r_y}=\frac{1.0\left(20.0\text{ ft}\right)\left(12\text{ in./ft}\right)}{2.46\text{ in.}}=97.6<113$$ therefore, Eqn. E3-2 applies

$$F_e=\frac{\pi^2E}{\left(\frac{KL}{r}\right)^2}=\frac{\pi^2\left(29{,}000\text{ ksi}\right)}{\left(97.6\right)^2}=30.0\text{ ksi}$$

Eqn. E3-4

$$F_{cry}=F_{cr}=\left[0.658^{\frac{F_y}{F_e}}\right]F_y=\left[0.658^{\frac{50.0\text{ ksi}}{30.0\text{ ksi}}}\right]50.0\text{ ksi}=24.9\text{ ksi}$$

Eqn. E3-2

The shear center for a T-shaped section is located on the axis of symmetry at the mid-depth of the flange.

$$x_0 = 0.0 \text{ in.}$$

$$y_0 = \bar{y} - \frac{t_f}{2} = 1.29 \text{ in.} - \frac{0.720 \text{ in.}}{2} = 0.930 \text{ in.}$$

$$\bar{r}_0^2 = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} = (0.0 \text{ in.})^2 + (0.930 \text{ in.})^2 + \frac{32.6 \text{ in.}^4 + 60.7 \text{ in.}^4}{9.99 \text{ in.}^2} = 10.2 \text{ in.}^2$$ Eqn. E4-7

$$\bar{r}_0 = \sqrt{\bar{r}_0^2} = \sqrt{10.2 \text{ in.}^2} = 3.19 \text{ in.}$$

$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{r}_0^2} = 1 - \frac{(0.0 \text{ in.})^2 + (0.930 \text{ in.})^2}{10.2 \text{ in.}^2} = 0.915$$ Eqn. E4-8

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} = \frac{(11{,}200 \text{ ksi})(1.50 \text{ in.}^4)}{(9.99 \text{ in.}^2)(3.19 \text{ in.})^2} = 165 \text{ ksi}$$ Eqn. E4-3

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{(F_{cry} + F_{crz})^2}}\right]$$ Eqn. E4-2

$$= \left(\frac{24.9 \text{ ksi} + 165 \text{ ksi}}{2(0.915)}\right)\left[1 - \sqrt{1 - \frac{4(24.9 \text{ ksi})(165 \text{ ksi})(0.915)}{(24.9 \text{ ksi} + 165 \text{ ksi})^2}}\right]$$

$= 24.5$ ksi **does not control**

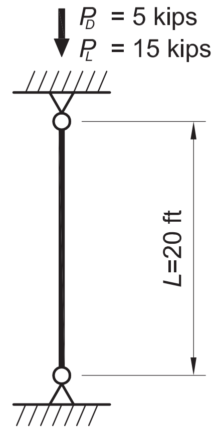
x-x axis flexural buckling governs, therefore

$$P_n = F_{cr} A_g = (14.2 \text{ ksi}) 9.99 \text{ in.}^2 = 142 \text{ kips}$$ Eqn. E3-1

LRFD	ASD
$\phi_c P_n = 0.90(142 \text{ kips})$ $= 128 \text{ kips} > 120 \text{ kips}$ **o.k.**	$P_n / \Omega_c = \frac{142 \text{ kips}}{1.67}$ $= 85.0 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

Section E1

Example E.8 Design of a WT Compression Member with Slender Elements



Given:
Select a WT-shape compression member with a length of 20 ft to support a dead load of 5 kips and live load of 15 kips in axial compression. The ends are pinned.

Because WT sections are cut from ASTM A992 W-shape beams, the material properties are:

Material Properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Calculate the required strength

LRFD	ASD
$P_u = 1.2(5\text{ kips}) + 1.6(15\text{ kips}) = 30.0\text{ kips}$	$P_a = 5\text{ kips} + 15\text{ kips} = 20.0\text{ kips}$

Table Solution:

K = 1.0, therefore $(KL)_x$ = $(KL)_y$ = 20 ft. — Commentary Table C-C2.2

Select the lightest member from Table 4-7 with sufficient available strength about the both the *x-x* (upper portion of the table) and the *y-y* axis (lower portion of the table) to support the required strength.

Try a WT7×15

Determine the available strength in axial compression from Manual Table 4-7 — Manual Table 4-7

LRFD	ASD
$\phi_c P_{nx} = 66.6\text{ kips} > 30.0\text{ kips}$ **o.k.**	$P_{nx}/\Omega_c = 44.3\text{ kips} > 20.0\text{ kips}$ **o.k.**
$\phi_c P_{nx} = 36.5\text{ kips} > 30.0\text{ kips}$ **controls o.k.**	$P_{ny}/\Omega_c = 24.3\text{ kips} > 20.0\text{ kips}$ **controls o.k.**

The available strength can be easily determined by using the tables of the Manual. Available strength values can be verified by hand calculations, as shown below.

Calculation Solution:

Geometric Properties:

WT7×15 $A_g = 4.42\text{ in.}^2$ $r_x = 2.07\text{ in.}$ $r_y = 1.49\text{ in.}$ $J = 0.190\text{ in.}^4$ $Q_s = 0.609$ — Manual Table 1-8

$\bar{y} = 1.58\text{ in.}$ $I_x = 19.0\text{ in.}^4$ $I_y = 9.79\text{ in.}^4$

d = 6.92 in. t_w = 0.270 in. b_f = 6.73 in. t_f = 0.385 in.

Check for slender elements

$$\frac{d}{t_w} = \frac{6.92 \text{ in.}}{0.270 \text{ in.}} = 25.6 > 0.75\sqrt{\frac{E}{F_y}} = 0.75\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 18.1$$ therefore, the web is slender. Table B4.1 Case 8

$$\frac{b_f}{2t_f} = \frac{6.73 \text{ in.}}{2(0.385 \text{ in.})}$$ Table B4.1 Case 3

$$= 8.74 < 0.56\sqrt{\frac{E}{F_y}} = 0.56\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 13.5$$ therefore, the flange is not slender.

Because this WT7×15 has a slender web, Specification Section E7 is applicable. The nominal compressive strength, P_n, shall be determined based on the limit states of flexural, torsional, and flexural-torsional buckling.

Calculate the x-x axis critical elastic flexural buckling stress

$$\frac{K_xL}{r_x} = \frac{1.0(20.0 \text{ ft})(12 \text{ in./ft})}{2.07 \text{ in.}} = 116$$

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000 \text{ ksi})}{(116)^2} = 21.3 \text{ ksi}$$ **does not control** Eqn. E3-4

Calculate the critical elastic torsional and flexural-torsional buckling stress

$$\frac{K_yL}{r_y} = \frac{1.0(20.0 \text{ ft})(12 \text{ in./ft})}{1.49 \text{ in.}} = 161$$

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_yL}{r_y}\right)^2} = \frac{\pi^2(29{,}000 \text{ ksi})}{(161)^2} = 11.0 \text{ ksi}$$ Eqn. E4-10

Calculate the torsional parameters

The shear center for a T-shaped section is located on the axis of symmetry at the mid-depth of the flange.

$$x_o = 0.0 \text{ in.}$$

$$y_o = \bar{y} - \frac{t_f}{2} = 1.58 \text{ in.} - \frac{0.385 \text{ in.}}{2} = 1.39 \text{ in.}$$

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} = (0.0 \text{ in.})^2 + (1.39 \text{ in.})^2 + \frac{19.0 \text{ in.}^4 + 9.79 \text{ in.}^4}{4.42 \text{ in.}^2} = 8.45 \text{ in.}^2$$ Eqn. E4-7

$$\bar{r}_o = \sqrt{\bar{r}_o^2} = \sqrt{8.45 \text{ in.}^2} = 2.91 \text{ in.}$$

$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{r}_0^2} = 1 - \frac{(0.0 \text{ in.})^2 + (1.39 \text{ in.})^2}{8.45 \text{ in.}^2} = 0.771$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ \right) \frac{1}{A_g \bar{r}_o^2}$$ Omit term with C_w per User Note at end of Section E4 Eqn. E4-8

Eqn. E4-11

$$F_{ez} = \frac{GJ}{A_g \bar{r}_o^2} = \frac{11{,}200\text{ ksi}(0.190\text{ in.}^4)}{4.42\text{ in.}^2(2.91\text{ in.})^2} = 56.9\text{ ksi}$$

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H} \right) \left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}} \right]$$

$$F_e = \left(\frac{11.0\text{ ksi} + 56.9\text{ ksi}}{2(0.771)} \right) \left[1 - \sqrt{1 - \frac{4(11.0\text{ ksi})(56.9\text{ ksi})(0.771)}{(11.0\text{ ksi} + 56.9\text{ ksi})^2}} \right] = 10.5\text{ ksi}$$ **controls** Eqn. E4-5

Check limit for the applicable equation

$0.44QF_y = 0.44(0.609)(50\text{ ksi}) = 13.4\text{ ksi} > 10.5\text{ ksi}$ therefore Eqn. E7-3 applies

$$F_{cr} = 0.877F_e = 0.877(10.5\text{ ksi}) = 9.21\text{ ksi}$$

$$P_n = F_{cr}A_g = (9.21\text{ ksi})4.42\text{ in.}^2 = 40.7\text{ kips}$$

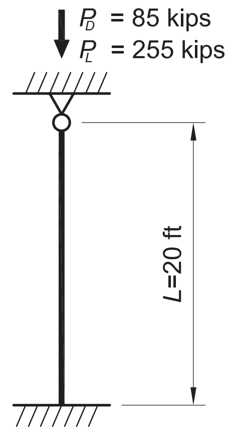
Eqn. E7-3

Eqn. E7-1

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(40.7\text{ kips})$ $= 36.6\text{ kips} > 30.0\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $P_n / \Omega_c = 40.7\text{ kips}/1.67$ $= 24.4\text{ kips} > 20.0\text{ kips}$ **o.k.**

Section E1

Example E.9 Design of a Rectangular HSS Compression Member without Slender Elements



Given:
Select a rectangular HSS compression member, with a length of 20 ft, to support a dead load of 85 kips and live load of 255 kips in axial compression. The base is fixed and the top is pinned.

Material Properties:

ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi Manual Table 2-3

Calculate the required strength

LRFD	ASD
$P_u = 1.2(85 \text{ kips}) + 1.6(255 \text{ kips}) = 510 \text{ kips}$	$P_a = 85 \text{ kips} + 255 \text{ kips} = 340 \text{ kips}$

Table Solution:

K = 0.8 Commentary Table C-C2.2

$(KL)_x = (KL)_y = 0.8(20.0 \text{ ft}) = 16.0 \text{ ft}$

Enter Manual Table 4-3 for rectangular sections or Table 4-4 for square sections.
Try an HSS12×10×⅜

Determine the available strength in axial compression

LRFD	ASD
$\phi_c P_n = 517 \text{ kips} > 510 \text{ kips}$ **o.k.**	$P_n / \Omega_c = 344 \text{ kips} > 340 \text{ kips}$ **o.k.**

Manual Table 4-3

The available strength can be easily determined by using the tables of the Manual. Available strength values can be verified by hand calculations, as shown below.

Calculation Solution:

Geometric Properties: Manual Table 1-11

HSS12×10×⅜ $A_g = 14.6 \text{ in.}^2$ $r_x = 4.61$ in. $r_y = 4.01$ in. $t_{des} = 0.349$ in.

Check for slender elements

Note: According to Specification Section B4.2, if the corner radius is not known, *b* and *h* shall be taken as the outside dimension less three times the design wall thickness. This is generally a conservative assumption.

Calculate b/t of the most slender wall

Table B4.1
Case 12

$$\frac{h}{t} = \frac{12.0\text{ in.} - 3(0.349\text{ in.})}{0.349\text{ in.}} = 31.4 < 1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 35.2$$

Therefore, the section does not contain slender elements.

Since $r_y < r_x$ and $(KL)_x = (KL)_y$, r_y will govern the available strength.

Determine the applicable equation

$$\frac{K_y L}{r_y} = \frac{0.8(20.0\text{ ft})(12\text{ in./ft})}{4.01\text{ in.}} = 47.9$$

Section E3

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 118 > 47.9\text{, therefore, use Specification Equation E3-2.}$$

Eqn. E3-4

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000)}{(47.9)^2} = 125\text{ ksi}$$

Eqn. E3-2

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y = \left(0.658^{\frac{46\text{ ksi}}{125\text{ ksi}}}\right)(46\text{ ksi}) = 39.4\text{ ksi}$$

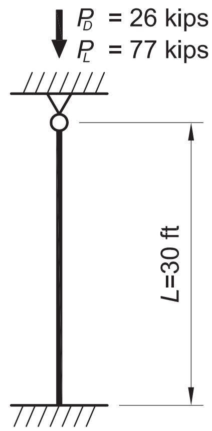
Eqn. E3-1

$$P_n = F_{cr}A_g = (39.4\text{ ksi})14.6\text{ in.}^2 = 575\text{ kips}$$

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(575\text{ kips})$ $= 518\text{ kips} > 510\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $P_n/\Omega_c = \dfrac{575\text{ kips}}{1.67}$ $= 344\text{ kips} > 340\text{ kips}$ **o.k.**

Specification
Section E1

Example E.10 Design of a Rectangular HSS Compression Member with Slender Elements



Given:

Select a rectangular HSS12×8 compression member with a length of 30 ft, to support an axial dead load of 26 kips and live load of 77 kips. The base is fixed, the top is pinned.

A column with slender elements has been selected to demonstrate the design of such a member.

Material Properties:

ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi — Manual Table 2-3

Calculate the required strength

LRFD	ASD
$P_u = 1.2(26 \text{ kips}) + 1.6(77 \text{ kips}) = 154 \text{ kips}$	$P_a = 26 \text{ kips} + 77 \text{ kips} = 103 \text{ kips}$

Table Solution:

$K = 0.8$ Therefore $(KL)_x = (KL)_y = 0.8(30.0 \text{ ft}) = 24.0 \text{ ft}$ — Commentary Table C-C2.2

Enter Manual Table 4-3, for the HSS12×8 section and proceed to the lightest section with an available strength that is equal to or greater than the required strength, in this case a HSS 12×8×3⁄16.

Determine the available strength in axial compression

LRFD	ASD
$\phi_c P_n = 155 \text{ kips} > 154 \text{ kips}$ **o.k.**	$P_n / \Omega_c = 103 \text{ kips} \geq 103 \text{ kips}$ **o.k.**

Manual Table 4-3

The available strength can be easily determined by using the tables of the Manual. Available strength values can be verified by hand calculations, as shown below, including adjustments for slender elements.

Calculation Solution:

Geometric Properties:

HSS12×8×3⁄16 $A_g = 6.76 \text{ in.}^2$ $r_x = 4.56 \text{ in.}$ $r_y = 3.35 \text{ in.}$ $\frac{b}{t} = 43.0$ — Manual Table 1-11

$\frac{h}{t} = 66.0$ $t_{des} = 0.174 \text{ in.}$

Check for slender elements

Calculate the limiting width-thickness ratio.

$$\lambda_r = 1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 35.2 < 43.0$$ therefore both the 8 in. and 12 in. walls are slender elements.

Table B4.1
Case 12

Note that for determining the width-thickness ratio, b is taken as the outside dimension minus three times the design wall thickness.

For the selected shape,

b = 8.0 in. – 3(0.174 in.) = 7.48 in.
h = 12.0 in. – 3(0.174 in.) = 11.5 in.

Specification
Section
B4.2d

For a HSS member with slender elements, the nominal compressive strength, P_n, is determined based upon the limit states of flexural buckling. Torsional buckling will not govern unless the torsional unbraced length greatly exceeds the controlling flexural unbraced length.

Compute effective area, A_{eff}

Section E7

$$Q_a = \frac{A_{eff}}{A}$$

Eqn. E7-16

where

A_{eff} = summation of the effective areas of the cross section based on the reduced effective widths, b_e.

For flanges of square and rectangular slender-element sections of uniform thickness,

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1-\frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b$$

Eqn. E7-18

where $f = P_n / A_{eff}$, but can conservatively be taken as F_y

For the 8 in. walls,

$$b_e = 1.92t\sqrt{\frac{E}{F_y}}\left[1-\frac{0.38}{(b/t)}\sqrt{\frac{E}{F_y}}\right]$$

Eqn. E7-18

$$= 1.92(0.174\text{ in.})\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}}\left[1-\frac{0.38}{(43.0)}\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}}\right] = 6.53\text{ in.} \le 7.48\text{ in.}$$

Length that is ineffective = $b - b_e$ = 7.48 in. – 6.53 in. = 0.950 in.

For the 12 in. walls,

$$b_e = 1.92t\sqrt{\frac{E}{F_y}}\left[1-\frac{0.38}{(b/t)}\sqrt{\frac{E}{F_y}}\right] \le b$$

Eqn. E7-18

$$= 1.92(0.174\text{in.})\sqrt{\frac{29{,}000\text{ksi}}{46\text{ksi}}}\left[1-\frac{0.38}{(66.1)}\sqrt{\frac{29{,}000\text{ksi}}{46\text{ksi}}}\right] = 7.18\text{ in.} \leq 11.5\text{ in.}$$

Length that can not be used = 11.5 in. – 7.18 in. = 4.32 in.

A_{eff} = 6.76 in.^{2} – 2(0.174 in.)(0.950 in.) – 2(0.174 in.)(4.32 in.) = 4.93 in.^{2}

$$Q = Q_a = \frac{A_{eff}}{A} = \frac{4.93\text{ in.}^2}{6.76\text{ in.}^2} = 0.729$$ Eqn. E7-16

Determine the appropriate equation for F_{cr}

$$\frac{K_y L}{r_y} = \frac{0.8(30.0\text{ ft})(12\text{ in./ ft})}{3.35\text{in.}} = 86.0$$

Section E7

$$4.71\sqrt{\frac{E}{QF_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{0.729(46\text{ ksi})}} = 139 > 86.0$$, therefore Specification Equation E7-2 applies.

Eqn. E3-4

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(86.0)^2} = 38.7\text{ ksi}$$

Eqn. E7-2

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y = 0.729\left[0.658^{\frac{0.729(46\text{ ksi})}{38.7\text{ ksi}}}\right]46\text{ ksi} = 23.3\text{ ksi}$$

Eqn. E7-1

$P_n = F_{cr}A_g$ = (23.3 ksi)6.76 in.^{2} = 158 kips

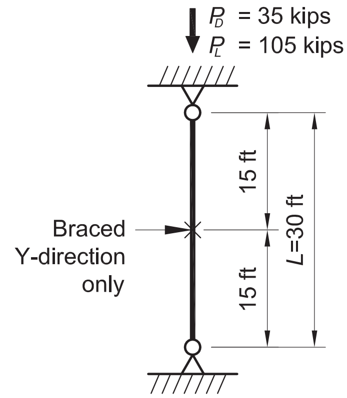
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(158\text{ kips}) = 142\text{ kips} < 154\text{ kips}$ **See note below**	$\Omega_c = 1.67$ $P_n / \Omega_c = \frac{158\text{ kips}}{1.67} = 94.6\text{ kips} < 103\text{ kips}$ **See note below**

Section E1

Note: A conservative initial assumption ($f = F_y$) was made in applying Specification Equation E7-18. A more exact solution is obtained by iterating from the *Compute effective area, A_{eff}* step using $f = P_n/A_{eff}$ until the value of f converges. The HSS column strength tables in the Manual were calculated using this iterative procedure.

Example E.11 Design of a Pipe Compression Member

Given:
Select a Pipe compression member with a length 30 ft to support a dead load of 35 kips and live load of 105 kips in axial compression. The column is pin-connected at the ends in both axes and braced at the midpoint in the *y-y* direction.



Material Properties:

ASTM A53 Gr. B F_y = 35 ksi F_u = 60 ksi — Manual Table 2-3

Calculate the required strength

LRFD	ASD
$P_u = 1.2(35\text{ kips}) + 1.6(105\text{ kips}) = 210\text{ kips}$	$P_a = 35\text{ kips} + 105\text{ kips} = 140\text{ kips}$

Table Solution:

K = 1.0 Therefore, $(KL)_x$ = 30.0 ft and $(KL)_y$ = 15.0 ft Buckling about the x-x axis controls. — Commentary Table C-C2.2

Enter Manual Table 4-6 with a *KL* of 30 ft and proceed across the table until reaching the lightest section with sufficient available strength to support the required strength.

Try a 10 inch Standard Pipe. The available strength in axial compression is:

LRFD	ASD
$\phi_c P_n = 215\text{ kips} > 210\text{ kips}$ **o.k.**	$P_n / \Omega_c = 143\text{ kips} > 140\text{ kips}$ **o.k.**

Manual Table 4-6

The available strength can be easily determined by using the tables of the Manual. Available strength values can be verified by hand calculations, as shown below.

Calculation Solution:

Geometric Properties:

Pipe 10 Std. $A_g = 11.1\text{ in.}^2$ $r = 3.68\text{ in.}$ $\dfrac{D}{t} = 31.6$ — Manual Table 1-14

No Steel Pipes shown in Manual Table 4-10 are slender at 35 ksi, so no local buckling check is required; however, some Round HSS are slender at higher steel strengths. The following calculations illustrate the required check.

Calculate the limiting width/thickness ratio

$\lambda_r = 0.11E/F_y = 0.11(29{,}000\text{ ksi}/35\text{ ksi}) = 91.1 > 31.6$; therefore, the pipe is not slender — Table B4.1 Case 15

$$\frac{KL}{r} = \frac{(30.0\text{ ft})(12\text{ in./ft})}{3.68\text{ in.}} = 97.8$$ Section E3

$$4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000\text{ ksi}}{35\text{ ksi}}} = 136 > 97.8\text{, therefore Specification Equation E3-2 applies.}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(97.8)^2} = 29.9\text{ ksi}$$ Eqn. E3-4

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right)F_y = \left(0.658^{\frac{35\text{ ksi}}{29.9\text{ ksi}}}\right)(35\text{ ksi}) = 21.4\text{ ksi}$$ Eqn. E3-2

$$P_n = F_{cr}A_g = (21.4\text{ ksi})11.1\text{ in.}^2 = 238\text{ kips}$$ Eqn. E3-1

LRFD	ASD
$\phi_c = 0.90$	$\Omega_c = 1.67$
$\phi_c P_n = 0.90(238\text{ kips})$	$P_n/\Omega_c = \dfrac{238\text{ kips}}{1.67}$
$= 214\text{ kips} > 210\text{ kips}$ **o.k.**	$= 143\text{ kips} > 140\text{ kips}$ **o.k.**

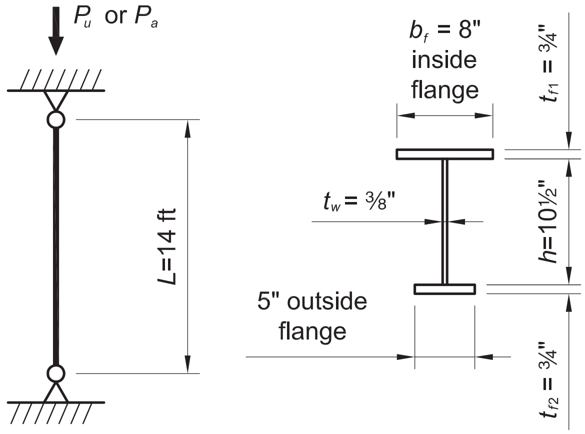
Section E1

Note that the design procedure would be similar for a Round HSS column.

Example E.12 Built-up I-Shaped Member with Different Flange Sizes

Given:

Compute the available strength of a built-up compression member with a length of 14 ft. The ends are pinned. The outside flange is PL¾×5, the inside flange is PL¾×8, and the web is PL⅜×10½. Material is ASTM A572 Grade 50.



Material Properties:

ASTM A572 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-4

Solution:

User note: There are no tables for special built-up shapes.

Determine if the shape has any slender elements

Check outside flange slenderness

Calculate k_c

$$k_c = \frac{4}{\sqrt{\frac{h}{t_w}}} = \frac{4}{\sqrt{\frac{10.5\text{ in.}}{0.375\text{ in.}}}} = 0.756 \qquad 0.35 \le k_c \le 0.76 \quad \text{o.k.}$$

Table B4.1 Note [a]

$$\frac{b}{t} = \frac{2.50\text{ in.}}{0.75\text{ in.}} = 3.33$$

$$\lambda_r = 0.64\sqrt{\frac{k_c E}{F_y}} = 0.64\sqrt{\frac{0.756(29{,}000\text{ ksi})}{50\text{ ksi}}} = 13.4$$

Table B4.1 Case 4

$\frac{b}{t} \le \lambda_r$ therefore, the outside flange is not slender.

Check inside flange slenderness

$$\frac{b}{t} = \frac{4.0\text{ in.}}{0.750\text{ in.}} = 5.33$$

Table B4.1 Case 4

$\frac{b}{t} \le \lambda_r$ therefore, the inside flange is not slender.

Check web slenderness

$$\frac{h}{t} = \frac{10.5 \text{ in.}}{0.375 \text{ in.}} = 28.0$$

$$\lambda_r = 1.49\sqrt{\frac{E}{F_y}} = 1.49\sqrt{\frac{29{,}000 \text{ ksi}}{50 \text{ ksi}}} = 35.9$$

Table B4.1
Case 14

$\frac{h}{t} < \lambda_r$ therefore, the web is not slender.

Calculate section properties (ignoring welds)

$$A_g = b_{f1}t_{f1} + ht_w + b_{f2}t_{f2} = (8.00 \text{ in.})(0.750 \text{ in.}) + (10.5 \text{ in.})(0.375 \text{ in.}) + (5.00 \text{ in.})(0.75 \text{ in.})$$
$$= 13.7 \text{ in.}^2$$

$$\bar{y} = \frac{\Sigma A_i y_i}{\Sigma A_i} = \frac{(6.00 \text{ in.}^2)(11.6 \text{ in.}) + (3.94 \text{ in.}^2)(6.00 \text{ in.}) + (3.75 \text{ in.}^2)(0.375 \text{ in.})}{(6.00 \text{ in.}^2) + (3.94 \text{ in.}^2) + (3.75 \text{ in.}^2)} = 6.91 \text{ in.}$$

Note that the neutral axis location is measured from the bottom of the outside flange.

$$I_x = \left[\frac{(8.0 \text{ in.})(0.75 \text{ in.})^3}{12} + (8.0 \text{ in.})(0.75 \text{ in.})(4.72 \text{ in.})^2\right] +$$
$$\left[\frac{(0.375 \text{ in.})(10.5 \text{ in.})^3}{12} + (0.375 \text{ in.})(10.5 \text{ in.})(0.910 \text{ in.})^2\right] +$$
$$\left[\frac{(5.0 \text{ in.})(0.750 \text{ in.})^3}{12} + (5.0 \text{ in.})(0.750 \text{ in.})(6.54 \text{ in.})^2\right] = 334 \text{ in.}^4$$

$$r_x = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{334 \text{ in.}^4}{13.7 \text{ in.}^2}} = 4.94 \text{ in.}$$

$$I_y = \left[\frac{(0.75 \text{ in.})(8.0 \text{ in.})^3}{12}\right] + \left[\frac{(10.5 \text{ in.})(0.375 \text{ in.})^3}{12}\right] + \left[\frac{(0.750 \text{ in.})(5.0 \text{ in.})^3}{12}\right] = 39.9 \text{ in.}^4$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{39.9 \text{ in.}^4}{13.7 \text{ in.}^2}} = 1.71 \text{ in.}$$

Calculate x-x axis flexural elastic critical buckling stress, F_e

$$\frac{K_x L}{r_x} = \frac{1.0(14.0 \text{ ft})(12 \text{ in./ft})}{4.94 \text{ in.}} = 34.0$$

Section E3

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(34.0)^2} = 246\text{ ksi}$$ **does not control** Eqn. E3-4

Calculate the flexural- torsional critical elastic buckling stress

Torsional Constant

$$J = \Sigma\left(\frac{bt^3}{3}\right) = \frac{(8.00\text{ in.})(0.750\text{ in.})^3}{3} + \frac{(10.5\text{ in.})(0.375\text{ in.})^3}{3} + \frac{(5.00\text{ in.})(0.750\text{ in.})^3}{3} = 2.01\text{ in.}^4$$ Design Guide #9

Distance between flange centroids

$$h_o = d - \frac{t_{f1}}{2} - \frac{t_{f2}}{2} = 12.0\text{ in.} - \frac{0.750\text{ in.}}{2} - \frac{0.750\text{ in.}}{2} = 11.3\text{ in.}$$ Design Guide #9

Warping constant

$$C_w = \frac{t_f h_o^2}{12}\left(\frac{b_1^3 b_2^3}{b_1^3 + b_2^3}\right) = \frac{(0.750\text{ in.})(11.3\text{ in.})^2}{12}\left(\frac{(8.00\text{ in.})^3(5.00\text{ in.})^3}{(8.00\text{ in.})^3 + (5.00\text{ in.})^3}\right) = 802\text{ in.}^6$$ Design Guide #9

Due to symmetry, both the centroid and the shear center lie on the y-axis. Therefore $x_o = 0$. The distance from the center of the outside flange to the shear center is,

$$e = h_o\left(\frac{b_1^3}{b_1^3 + b_2^3}\right) = 11.3\text{in.}\left(\frac{(8.00\text{ in.})^3}{(8.00\text{ in.})^3 + (5.00\text{ in.})^3}\right) = 9.08\text{ in.}$$

Add one-half the flange thickness to determine the shear center location measured from the bottom of the outside flange.

$$e + \frac{t_f}{2} = 9.08\text{ in.} + \frac{0.750\text{ in.}}{2} = 9.46\text{ in.}$$

$$y_o = \left(e + \frac{t_f}{2}\right) - \bar{y} = 9.46\text{ in.} - 6.91\text{ in.} = 2.55\text{ in.}$$

$$\bar{r}_0^2 = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g} = 0.0 + (2.55\text{ in.})^2 + \frac{334\text{ in.}^4 + 39.9\text{ in.}^4}{13.7\text{ in.}^2} = 33.8\text{ in.}^2$$ Eqn. E4-7

$$H = 1 - \frac{x_0^2 + y_0^2}{\bar{r}_0^2} = 1 - \frac{0.0 + (2.55\text{ in.})^2}{33.8\text{ in.}^2} = 0.808$$ Eqn. E4-8

Since the ends are pinned, $K = 1.0$ Commentary Table C.C-2.2

$$\frac{KL}{r_y} = \frac{1.0(14.0\text{ ft})(12.0\text{ in./ft})}{1.71\text{ in.}} = 98.2$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} = \frac{\pi^2(29{,}000\text{ ksi})}{(98.2)^2} = 29.7\text{ ksi}$$ Eqn. E4-10

$$F_{ez} = \left[\frac{\pi^2 EC_w}{(K_z L)^2} + GJ\right]\left(\frac{1}{A_g \bar{r}_o^2}\right)$$ Eqn. E4-11

$$= \left[\frac{\pi^2 (29{,}000 \text{ ksi})(802 \text{ in.}^6)}{\left[(1.0)(14.0 \text{ ft})(12 \text{ in./ft})\right]^2} + (11{,}200 \text{ ksi})(2.01 \text{ in.}^4)\right]\left(\frac{1}{(13.7 \text{ in.}^2)(33.8 \text{ in.}^2)}\right) = 66.2 \text{ ksi}$$

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{(F_{ey} + F_{ez})^2}}\right]$$ Eqn. E4-5

$$= \left(\frac{29.7 \text{ ksi} + 66.2 \text{ ksi}}{2(0.808)}\right)\left[1 - \sqrt{1 - \frac{4(29.7 \text{ ksi})(66.2 \text{ ksi})(0.808)}{(29.7 \text{ ksi} + 66.2 \text{ ksi})^2}}\right]$$

$= 26.4$ ksi **controls**

Torsional and flexural-torsional buckling governs

$0.44F_y = 0.44(50 \text{ ksi}) = 22.0 \text{ ksi} < 26.4 \text{ ksi}$, therefore Specification Equation E3-2 applies.

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{50 \text{ ksi}}{26.4 \text{ksi}}}\right](50 \text{ ksi}) = 22.6 \text{ ksi}$$ Eqn. E3-2

$$P_n = F_{cr}A_g = (22.6 \text{ ksi})13.7 \text{ in.}^2 = 310 \text{ kips}$$ Eqn. E3-1

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(310 \text{ kips}) = 279 \text{ kips}$	$\Omega_c = 1.67$ $P_n / \Omega_c = \dfrac{310 \text{ kips}}{1.67} = 186 \text{ kips}$

Section E1

CHAPTER F
DESIGN OF MEMBERS FOR FLEXURE

INTRODUCTION

This Specification chapter contains provisions for calculating the flexural strength of members subject to simple bending about one principal axis. Included are specific provisions for I-shaped members, channels, HSS, tees, double angles, single angles, rectangular bars, rounds and unsymmetrical shapes. Also included is a section with proportioning requirements for beams and girders.

There are selection tables in the Manual for standard beams in the commonly available yield strengths. The section property tables for most cross sections provide information that can be used to conveniently identify noncompact and slender element sections. LRFD and ASD information is presented side by side.

Most of the formulas from this chapter are illustrated by examples below. The design and selection techniques illustrated in the examples for both LRFD and ASD will result in similar designs.

F1. GENERAL PROVISIONS

Selection and evaluation of all members is based on deflection requirements and strength, which is determined as the design flexural strength, $\phi_b M_n$, or the allowable flexural strength, M_n/Ω_b, where:

M_n = the lowest nominal flexural strength based on the limit states of yielding, lateral torsional-buckling and local buckling, where applicable

$\phi_b = 0.90$ (LRFD) $\qquad \Omega_b = 1.67$ (ASD)

This design approach is followed in all examples.

The term L_b is used throughout this chapter to describe the length between points which are either braced against lateral displacement of the compression flange or braced against twist of the cross section. Requirements for bracing systems and the required strength and stiffness at brace points are given in Specification Appendix 6.

The use of C_b is illustrated in several examples below. Manual Table 3-1 provides tabulated C_b values for some common situations.

F2. DOUBLY-SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

Section F2 applies to the design of compact beams and channels. As indicated in the User Note in Section F2 of the Specification, the vast majority of rolled I-shaped beams and channels fall into this category. The curve presented as a solid line in Figure F-1 below is a generic plot of the nominal flexural strength, M_n, as a function of the unbraced length, L_b. The horizontal segment of the curve at the far left, between $L_b = 0$ ft and L_p, is the range where the strength is limited by flexural yielding. In this region, the nominal strength is taken as the full plastic moment strength of the section as given by Specification Equation F2-1. In the range of the curve at the far right, starting at L_r, the strength is limited by elastic buckling. The strength in this region is given by Specification Equation F2-3. Between these regions, within the linear region of the curve between $M_n = M_p$ at L_p on the left, and $M_n = 0.7M_y = 0.7F_yS_x$ at L_r on the right, the strength is limited by inelastic buckling. The strength in this region is provided in Specification Equation F2-2.

The curve plotted as a heavy solid line represents the case where $C_b = 1.0$, while the heavy dashed line represents the case where C_b exceeds 1.0. The nominal strengths calculated in both Equations F2-2 and F2-3 are linearly proportional to C_b, but are limited to M_p as shown in the figure.

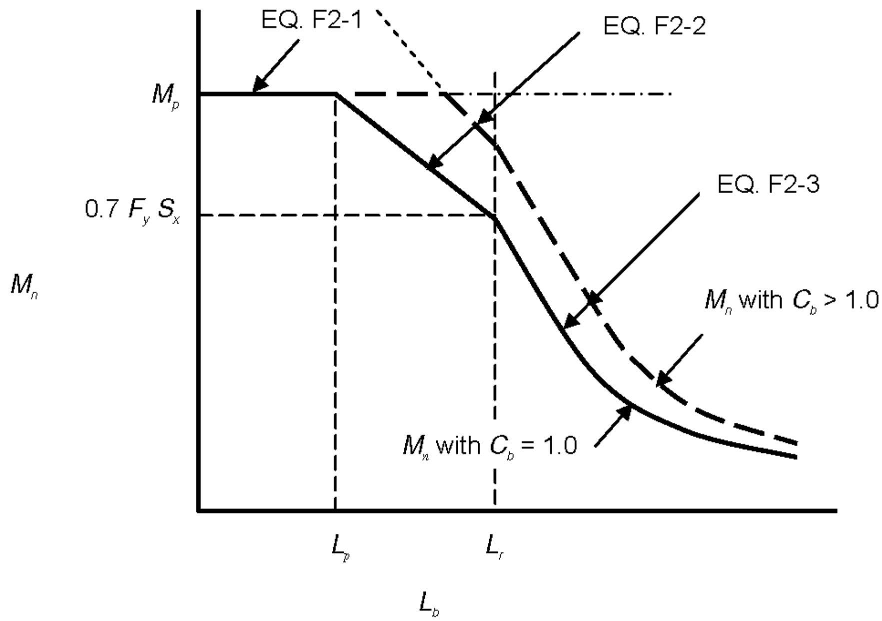


Figure F-1. Beam Strength versus Unbraced Length

$M_n = M_p = F_yZ_x$ Eqn. F2-1

$$M_n = C_b\left[M_p - (M_p - 0.70F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$ Eqn. F2-2

$M_n = F_{cr}S_x \le M_p$: where F_{cr} is evaluated as shown below Eqn. F2-3

$$F_{cr} = \frac{C_b\pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_xh_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$ Eqn. F2-4

The provisions of this section are illustrated in **Example F.1**(W-shape beam) and **Example F.2** (channel).

Plastic design provisions are given in Appendix 1. L_{pd}, the maximum unbraced length for plastic design is less than L_p.

F3. DOUBLY-SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES, BENT ABOUT THEIR MAJOR AXIS

The strength of shapes designed according to this section is limited by local buckling of the compression flange. Only a few standard wide flange shapes have noncompact flanges. For these sections, the strength reduction for F_y = 50 ksi steel varies. The approximate percentages of M_p about the strong axis that can be developed by noncompact members when braced such that $L_b \leq L_p$ are shown below:

W21×48 = 99%	W14×99 = 99%	W14×90 = 97%	W12×65 = 98%
W10×12 = 99%	W8×31 = 99%	W8×10 = 99%	W6×15 = 94%
W6×8.5 = 97%			

The strength curve for the flange local buckling limit state, shown in Figure F-2, is similar in nature to that of the lateral-torsional buckling curve. The horizontal axis parameter is $\lambda = \frac{b_f}{2t_f}$. The flat portion of the curve to the left of λ_{pf} is the plastic yielding strength, M_p. The curved portion to the right of λ_{rf} is the strength limited by elastic buckling of the flange. The linear transition between these two regions is the strength limited by inelastic flange buckling.

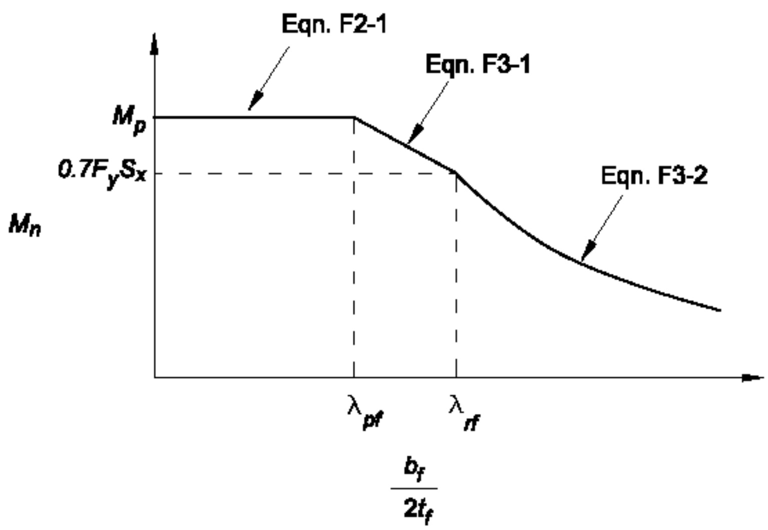


Figure F-2. Flange Local Buckling Strength

$$M_n = M_p = F_y Z_x \qquad \text{Eqn. F2-1}$$

$$M_n = \left[M_p - (M_p - 0.7F_y S_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right] \qquad \text{Eqn. F3-1}$$

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2}, \text{ where } k_c = \frac{4}{\sqrt{h/t_w}}$$

Eqn. F3-2
Table B4.1
Footnote

The strength reductions due to local flange buckling of the few standard rolled shapes with noncompact flanges are incorporated into the design tables in Chapter 3 of the Manual.

There are no standard I-shaped members with slender flanges. The noncompact flange provisions of this section are illustrated in **Example F.3**.

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section of the Specification applies to doubly symmetric I-shaped members with noncompact webs and singly symmetric I-shaped members (those having different flanges) with compact or noncompact webs.

F5. DOUBLY-SYMMETRIC AND SINGLY-SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to I-shaped members with slender webs, formerly designated as "plate girders".

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis are not subject to lateral-torsional buckling. Rolled or built up shapes with noncompact or slender flanges, as determined by Specification Table B4.1, must be checked for strength based on the limit state of local flange buckling using Equations F6-2 or F6-3 as applicable.

The vast majority of W-, M-, C-, and MC-shapes have compact flanges, and can therefore develop the full plastic moment, M_p about the minor axis. The provisions of this section are illustrated in **Example F.5.**

F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

Square and rectangular HSS need only be checked for the limit states of yielding and local buckling. Although lateral-torsional buckling is theoretically possible for very long rectangular HSS bent about the strong axis, deflection will control the design as a practical matter.

The design and section property tables in the Manual were calculated using a design wall thickness of 93% of the nominal wall thickness. Strength reductions due to local buckling have been accounted for in the Manual design tables. The selection of rectangular or square HSS with compact flanges is illustrated in **Example F.6**. The provisions for rectangular or square HSS with noncompact flanges are illustrated in **Example F.7**. The provisions for HSS with slender flanges are illustrated in **Example F.8**. Available strengths of square and rectangular HSS are listed in Tables 3-12 and 3-13.

F8. ROUND HSS AND PIPES

The definition of HSS encompasses both tube and pipe products. The lateral-torsional buckling limit state does not apply, but round HSS are subject to strength reductions from local buckling. Available strengths of round HSS and Pipes are listed in Manual Tables 3-14 and 3-15. The tabulated properties and strengths of these shapes in the Manual are calculated using a design wall thickness of 93% of the nominal wall thickness. The design of a round HSS is illustrated in **Example F.9**.

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

The specification provides a check for flange local buckling, which applies only when the flange is in compression due to flexure. This limit state will seldom govern. No explicit check for local buckling of the web is provided, but the lateral-torsional limit state equation converges to the local buckling limit state strength as the length approaches zero. Thus, this limit state must still be checked for members with very short or zero unbraced length when the tip of the stem is in flexural compression. As noted in the commentary, when the unbraced length is zero, the equation converges to $M_n = 0.424\dfrac{EJ}{d}$. When the tip of the tee is in flexural tension and the beam is continuously braced, this limit state need not be checked. Attention should be given to end conditions of tees to avoid inadvertent fixed end moments which induce compression in the web unless this limit state is checked. The design of a WT-shape in bending is illustrated in **Example F.10**.

F10. SINGLE ANGLES

Section F10 permits the flexural design of single angles using either the principal axes or geometric axes (*x-x* and *y-y* axes). When designing single angles without continuous bracing using the geometric axis design provisions, M_y must be multiplied by 0.80 for use in Equations F10-1, F10-2 and F10-3. The design of a single angle in bending is illustrated in **Example F.11**.

F11. RECTANGULAR BARS AND ROUNDS

There are no design tables in the Manual for these shapes. The local buckling limit state does not apply to any bars. With the exception of rectangular bars bent about the strong axis, solid square, rectangular and round bars are not subject to lateral-torsional buckling and are governed by the yielding limit state only. Rectangular bars bent about the strong axis are subject to lateral torsional buckling and are checked for this limit state with Equations F11-2 and F11-3 where applicable.

These provisions can be used to check plates and webs of tees in connections. A design example of a rectangular bar in bending is illustrated in **Example F.12**. A design example of a round bar in bending is illustrated in **Example F.13**.

F12. UNSYMMETRICAL SHAPES

Due to the wide range of possible unsymmetrical cross sections, specific lateral-torsional and local buckling provisions are not provided in this Specification section. A general template is provided, but appropriate literature investigation and engineering judgment are required for the application of this section. A Z-shaped section is designed in **Example F.14**.

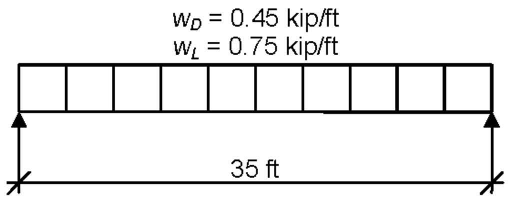
F13. PROPORTIONS FOR BEAMS AND GIRDERS

This section of the Specification includes a limit state check for tensile rupture due to holes in the tension flange of beams, proportioning limits for I-shaped members, detail requirements for cover plates and connection requirements for beams connected side to side

Example F.1-1a W-Shape Flexural Member Design in Strong-Axis Bending, Continuously Braced

Given:

Select an ASTM A992 W-shape beam with a simple span of 35 feet. Limit the member to a maximum nominal depth of 18 in. Limit the live load deflection to L/360. The nominal loads are a uniform dead load of 0.45 kip/ft and a uniform live load of 0.75 kip/ft. Assume the beam is continuously braced.



Beam Loading & Bracing Diagram
(full lateral support)

Solution:

Material Properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi

Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.45 \text{ kip/ft}) + 1.6\ (0.75 \text{ kip/ft})$ $= 1.74 \text{ kip/ft}$ $M_u = \dfrac{1.74 \text{ kip/ft}\left(35.0 \text{ ft}\right)^2}{8} = 266 \text{ kip-ft}$	$w_a = 0.45 \text{ kip/ft} + 0.75 \text{ kip/ft}$ $= 1.20 \text{ kip/ft}$ $M_a = \dfrac{1.20 \text{ kip/ft}\left(35.0 \text{ ft}\right)^2}{8} = 184 \text{ kip-ft}$

Calculate the required moment of inertia for live-load deflection criterion of L/360

$$\Delta_{max} = \frac{L}{360} = \frac{35.0\,\text{ft}(12\text{ in./ft})}{360} = 1.17\text{ in.}$$

$$I_{x(reqd)} = \frac{5wl^4}{384E\Delta_{max}} = \frac{5(0.750\text{ kip/ft})(35.0\text{ ft})^4(12\text{ in./ft})^3}{384\ (29{,}000\text{ ksi})(1.17\text{in.})} = 746\text{ in.}^4$$

Manual Table 3-23 Diagram 1

Select a W18×50 from Table 3-2

Per the User Note in Section F2, the section is compact. Since the beam is continuously braced and compact, only the yielding limit state applies.

LRFD	ASD
$\phi_b M_n = \phi_b M_{px} = 379\,\text{kip-ft} > 266\text{ kip-ft}$ **o.k.**	$M_n / \Omega_b = M_{px} / \Omega_b = 252\,\text{kip-ft} > 184\text{ kip-ft}$ **o.k.**

Manual Table 3-2

$I_x = 800\text{ in.}^4 > 746\text{ in.}^4$ **o.k.**

Manual Table 3-2

Example F.1-1b W-Shape Flexural Member Design in Strong-Axis Bending, Continuously Braced

Given:

Verify the flexural strength of the W18×50, A992 beam selected in Example F.1-1a by applying the requirements of the AISC Specification directly.

Solution:

Material Properties:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi — Manual Table 2-3

Geometric Properties:

W18×50 $Z_x = 101$ in.^{3} — Manual Table 1-1

Required strength from Example F.1-1a

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Calculate the nominal flexural strength, M_n

Per the User Note in Section F2, the section is compact. Since the beam is continuously braced and compact, only the yielding limit state applies.

$$M_n = M_p = F_y Z_x = 50 \text{ ksi}\left(101 \text{ in.}^3\right) = 5050 \text{ kip-in. or } 421 \text{ kip-ft}$$ Eqn. F2-1

Calculate the available flexural strength

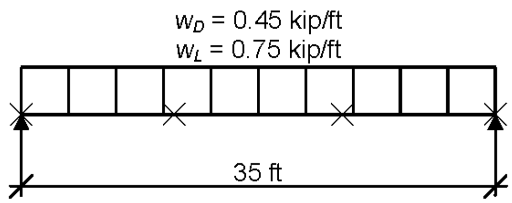
Section F1

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(421 \text{ kip-ft})$ $= 379 \text{ kip-ft} > 266 \text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $M_n / \Omega_b = (421 \text{ kip-ft})/1.67$ $= 252 \text{ kip-ft} > 184 \text{ kip-ft}$ **o.k.**

Example F.1-2a W-Shape Flexural Member Design in Strong-Axis Bending, Braced at Third Points

Given:

Verify the flexural strength of the W18×50, A992 beam selected in Example F.1-1a with the beam braced at the ends and third points. Use the Manual tables.



Beam Loading & Bracing Diagram
(bracing at ends and third points)

Solution:

Required flexural strength at midspan from Example F.1-1a

LRFD	ASD
M_u = 266 kip-ft	M_a = 184 kip-ft

$$L_b = \frac{35.0 \text{ ft}}{3} = 11.7 \text{ ft}$$

By inspection, the middle segment will govern. For a uniformly loaded beam braced at the ends and third points, C_b = 1.01 in the middle segment. Conservatively neglect this small adjustment in this case.

Manual
Table 3-1

Obtain the available strength from Table 3-10

Enter Table 3-10 and find the intersection of the curve for the W18×50 with an unbraced length of 11.7 ft. Obtain the available strength from the appropriate vertical scale to the left.

LRFD	ASD
$\phi_b M_n \approx$ 302 kip-ft > 266 kip-ft **o.k.**	$M_n / \Omega_b \approx$ 201 kip-ft > 184 kip-ft **o.k.**

Manual
Table 3-10

Example F.1-2b W-Shape Flexural Member Design in Strong-Axis Bending, Braced at Third Points

Given:

Verify the flexural strength of the W18×50, A992 beam selected in Example F.1-1a with the beam braced at the ends and third points. Apply the requirements of the AISC Specification directly.

Solution:

Material Properties:

W18×50 ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Geometric Properties:

W18×50 S_x = 88.9 in.^{3} — Manual Table 1-1

Required strength from Example F.1-1a

LRFD	ASD
M_u = 266 kip-ft	M_a = 184 kip-ft

Calculate the nominal flexural strength, M_n

Calculate C_b

For the lateral-torsional buckling limit state, the nonuniform moment modification factor can be calculated using Specification Equation F1-1.

$$C_b = \frac{12.5M_{max}}{2.5M_{max}+3M_A+4M_B+3M_C}R_m \leq 3.0$$ — Eqn. F1-1

For the center segment of the beam, the required moments for Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max} = 1.00, M_A = 0.972, M_B = 1.00, M_C = 0.972.

R_m = 1.0 for doubly-symmetric members — Section F1

$$C_b = \frac{12.5(1.00)}{2.5(1.00)+3(0.972)+4(1.00)+3(0.972)}(1.0) = 1.01$$

For the end-span beam segments, the required moments for Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max} = 0.889, M_A = 0.306, M_B = 0.556, and M_C = 0.750.

$$C_b = \frac{12.5(0.889)}{2.5(0.889)+3(0.306)+4(0.556)+3(0.750)}(1.0) = 1.46$$

Thus, the center span, with the higher required strength and lower C_b, will govern.

L_p = 5.83 ft   Manual
L_r = 17.0 ft   Table 3-2

Note: The more conservative formula for L_r given in the User Note in Specification Section F2 can yield very conservative results.

For a compact beam with an unbraced length of $L_p < L_b \le L_r$, the lesser of either the flexural yielding limit-state or the inelastic lateral-torsional buckling limit-state controls the nominal strength.

M_p = 5050 kip-in. (from Example F.1-1b)

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$ Eqn. F2-2

$$M_n = 1.01\left[5050\text{ kip-in.} - \left(5050\text{ kip-in.} - 0.7\left(50\text{ ksi}\right)\left(88.9\text{ in.}^3\right)\right)\left(\frac{11.7\text{ ft} - 5.83\text{ ft}}{17.0\text{ ft} - 5.83\text{ ft}}\right)\right]$$

$$\le 5050\text{kip-in.}$$

= 4070 kip-in. or 339 kip-ft

Calculate the available flexural strength

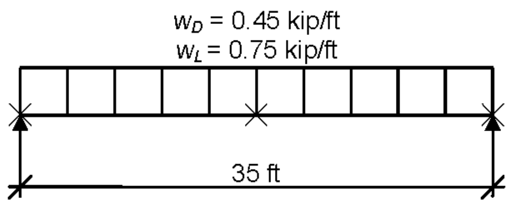
LRFD	ASD
ϕ_b = 0.90 $\phi_b M_n$ = 0.90(339 kip-ft) = 305 kip-ft > 266 kip-ft **o.k.**	Ω_b = 1.67 M_n/Ω_b = (339 kip-ft)/1.67 = 203 kip-ft > 184 kip-ft **o.k.**

Section F1

Example F.1-3a W-Shape Flexural Member Design in Strong-Axis Bending, Braced at Midspan

Given:

Verify the strength of the W18×50, A992 beam selected in Example F.1-1a with the beam braced at the ends and center point. Use the Manual tables.



Beam Loading & Bracing Diagram
(bracing at ends & midpoint)

Solution:

Required flexural strength at midspan from Example F.1-1a

LRFD	ASD
M_u = 266 kip-ft	M_a = 184 kip-ft

$$L_b = \frac{35.0\text{ ft}}{2} = 17.5\text{ ft}$$

For a uniformly loaded beam braced at the ends and at the center point, C_b = 1.30. There are several ways to make adjustments to Table 3-10 to account for C_b greater than 1.0.

Manual Table 3-1

Procedure A.

Available moments from the sloped and curved portions of the plots from Manual Table 3-10 may be multiplied by C_b, but may not exceed the value of the horizontal portion (ϕM_n for LRFD, M_n/Ω for ASD).

Obtain the available strength of a W18×50 with an unbraced length of 17.5 ft from Manual Table 3-10

Enter Table 3-10 and find the intersection of the curve for the W18×50 with an unbraced length of 17.5 ft. Obtain the available strength from the appropriate vertical scale to the left.

LRFD	ASD
$\phi_b M_n \approx 222$ kip-ft $\phi_b M_p = 379$ kip-ft (upper limit on $C_b M_n$) *Adjust for C_b* (1.30)(222 kip-ft) = 289 kip-ft	$M_n / \Omega_b \approx 147$ kip-ft $M_p / \Omega_b = 252$ kip-ft (upper limit on $C_b M_n$) *Adjust for C_b* (1.30)(147 kip-ft) = 191 kip-ft

Manual Table 3-10

LRFD	ASD
Check Limit 289 kip-ft ≤ $\phi_b M_p$ = 379 kip-ft **o.k.** *Check available versus required strength* 289 kip-ft > 266 kip-ft **o.k.**	*Check Limit* 191 kip-ft ≤ M_p / Ω_b = 252 kip-ft **o.k.** *Check available versus required strength* 191 kip-ft > 184 kip-ft **o.k.**

Procedure B.

For preliminary selection, the required strength can be divided by C_b and directly compared to the strengths in Table 3-10. Members selected in this way must be checked to ensure that the required strength does not exceed the available plastic moment strength of the section.

Calculate the adjusted required strength

LRFD	ASD
M_u' = 266 kip-ft / 1.3 = 205 kip-ft	M_a' = 184 kip-ft / 1.3 = 142 kip-ft

Obtain the available strength for a W18×50 with an unbraced length of 17.5 ft from Manual Table 3-10

Manual Table 3-10

LRFD	ASD
$\phi_b M_n$ ≈ 222 kip-ft > 205 kip-ft **o.k.** $\phi_b M_p$ = 379 kip-ft > 266 kip-ft **o.k.**	M_n / Ω_b ≈ 147 kip-ft > 142 kip-ft **o.k.** M_p / Ω_b = 252 kip-ft > 184 kip-ft **o.k.**

Example F.1-3b W-Shape Flexural Member Design in Strong-Axis Bending, Braced at Midspan

Given:

Verify the strength of the W18×50, A992 beam selected in Example F.1-1a with the beam braced at the ends and center point. Apply the requirements of the AISC Specification directly.

Solution:

Material Properties:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi — Manual Table 2-3

Geometric Properties:

W18×50 $r_{ts} = 1.98$ in. $S_x = 88.9$ in.^{3} $J = 1.24$ in.^{4} $h_o = 17.4$ in. — Manual Table 1-1

Required strength from Example F.1-1a

LRFD	ASD
$M_u = 266$ kip-ft	$M_a = 184$ kip-ft

Calculate the nominal flexural strength, M_n

Calculate C_b

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} R_m \leq 3.0$$ Eqn. F1-1

The required moments for Equation F1-1 can be calculated as a percentage of the maximum midspan moment as: M_{max}= 1.00, M_A = 0.438, M_B = 0.750, and M_C = 0.938.

$R_m = 1.0$ for doubly-symmetric members — Section F1

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.438) + 4(0.750) + 3(0.938)}(1.0) = 1.30$$

$L_p = 5.83$ ft
$L_r = 17.0$ ft — Manual Table 3-2

For a compact beam with an unbraced length $L_b > L_r$, the limit state of elastic lateral-torsional buckling applies.

Calculate F_{cr} with $L_b = 17.5$ ft

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$$ where $c = 1.0$ for doubly symmetric I-shapes — Eqn. F2-4

$$F_{cr} = \frac{1.30\pi^2(29{,}000 \text{ ksi})}{\left(\frac{17.5 \text{ ft}(12 \text{ in./ft})}{1.98 \text{ in.}}\right)^2}\sqrt{1+0.078\frac{\left(1.24 \text{ in.}^4\right)1.0}{\left(88.9 \text{ in.}^3\right)\left(17.4 \text{ in.}\right)}\left(\frac{17.5 \text{ ft}(12 \text{ in./ft})}{1.98 \text{ in.}}\right)^2} = 43.2 \text{ ksi}$$

$M_n = F_{cr}S_x \le M_p$ Eqn. F2-3

$M_n = 43.2 \text{ ksi}\left(88.9 \text{ in.}^3\right) = 3840 \text{ kip-in.} < 5050 \text{ kip-in.}$
$M_n = 3840$ kip-in or 320 kip-ft

Calculate the available flexural strength

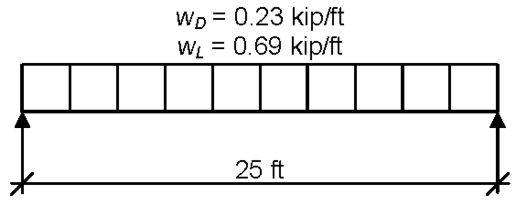
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(320 \text{ kip-ft}) = 288$ kip-ft 288 kip-ft > 266 kip-ft **o.k.**	$\Omega_b = 1.67$ $M_n / \Omega_b = 320 \text{ kip-ft} / 1.67 = 192$ kip-ft 192 kip-ft > 184 kip-ft **o.k.**

Section F1

Example F.2-1a Compact Channel Flexural Member, Continuously Braced

Given:

Select an ASTM A36 channel to serve as a roof edge beam with a simple span of 25 ft. Limit the live load deflection to L/360. The nominal loads are a uniform dead load of 0.23 kip/ft and a uniform live load of 0.69 kip/ft. The beam is continuously braced.



Beam Loading & Bracing Diagram
(Full lateral support)

Solution:

Material Properties:

ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi

Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.23\text{ kip/ft}) + 1.6(0.69\text{ kip/ft})$ $= 1.38\text{ kip/ft}$ $M_u = \dfrac{1.38\text{ kip/ft}\left(25.0\text{ ft}\right)^2}{8} = 108\text{ kip-ft}$	$w_a = 0.23\text{ kip/ft} + 0.69\text{ kip/ft}$ $= 0.920\text{ kip/ft}$ $M_a = \dfrac{0.920\text{ kip/ft}\left(25.0\text{ ft}\right)^2}{8} = 71.9\text{ kip-ft}$

Select a trial section

Per the User Note in Section F2, all ASTM A36 channels are compact. Because the beam is compact and continuously braced, the yielding limit state governs and $M_n = M_p$. Try C15×33.9 from Manual Table 3-8.

LRFD	ASD
$\phi_b M_n = \phi_b M_p = 137\text{ kip-ft} > 108\text{ kip-ft}$ **o.k.**	$M_n / \Omega_b = M_p / \Omega_b = 91.2\text{ kip-ft} > 71.9\text{ kip-ft}$ **o.k.**

Check live load deflection

$$\Delta_{max} = \frac{L}{360} = \frac{25.0\text{ ft}\left(12\text{ in./ft}\right)}{360} = 0.833\text{ in.}$$

For C15×33.9, $I_x = 315$ in.^{4}

Manual Table 1-5

$$\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5\left(0.690\text{ kip/ft}\right)\left(25.0\text{ ft}\right)^4\left(12\text{ in./ft}\right)^3}{384\left(29{,}000\text{ ksi}\right)\left(315\text{ in.}^4\right)} = 0.664\text{ in.} < 0.833\text{ in.} \quad \textbf{o.k.}$$

Manual Table 3-23 Diagram 1

Example F.2-1b Compact Channel Flexural Member, Continuously Braced

Given:

Example F.2-1a can be easily solved by utilizing the tables of the AISC *Steel Construction Manual*. Alternatively, this problem can be solved by applying the requirements of the AISC Specification directly.

Solution:

Material Properties:

C15×33.9 ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi — Manual Table 2-3

Geometric Properties:

C15×33.9 $Z_x = 50.8$ in.^{3} — Manual Table 1-5

Required strength from Example F.2-1a

LRFD	ASD
$M_u = 108$ kip-ft	$M_a = 71.9$ kip-ft

Calculate the nominal flexural strength, M_n

Per the User Note in Section F2, all ASTM A36 C- and MC-shapes are compact.

A channel that is continuously braced and compact is governed by the yielding limit state.

$M_n = M_p = F_y Z_x = 36\text{ ksi}(50.8\text{ in.}^3) = 1830$ kip-in. or 152 kip-ft — Eqn. F2-1

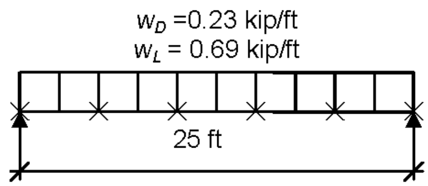
Section F1

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(152\text{ kip-ft})$ $= 137\text{ kip-ft} > 108\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $M_n / \Omega_b = 152\text{ kip-ft} / 1.67$ $= 91.0\text{ kip-ft} > 71.9\text{ kip-ft}$ **o.k.**

Example F.2-2a Compact Channel Flexural Member with Bracing at Ends and Fifth Points

Given:

Check the C15×33.9 beam selected in Example F.2-1a, assuming it is braced at the ends and the fifth points rather than continuously braced.



Beam Loading & Bracing Diagram
(Bracing at ends & ⅕ points)

Solution:

Material Properties:

ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-3

The center segment will govern by inspection.

Required strength at midspan from Example F.2-1a

LRFD	ASD
M_u = 108 kip-ft	M_a = 71.9 kip-ft

With an almost uniform moment across the center segment, C_b = 1.0, so no adjustment is required. — Manual Table 3-1

$$L_b = \frac{25.0\text{ ft}}{5} = 5.00\text{ ft}$$

Obtain the strength of the C15×33.9 *with an unbraced length of 5.00 ft from Manual Table 3-11*

Enter Table 3-11 and find the intersection of the curve for the C15×33.9 with an unbraced length of 5.00 ft. Obtain the available strength from the appropriate vertical scale to the left. — Manual Table 3-11

LRFD	ASD
$\phi_b M_n \approx$ 130 kip-ft > 108 kip-ft **o.k.**	$M_n / \Omega_b \approx$ 87.0 kip-ft > 71.9 kip-ft **o.k.**

Example F.2-2b Compact Channel Flexural Member with Bracing at End and Fifth Points

Given:

Verify the results from Example F.2-2a by calculation using the provisions of the Specification.

Solution:

Material Properties:

C15×33.9 ASTM A36 $F_y = 36$ ksi $F_u = 58$ ksi — Manual Table 2-3

Geometric Properties:

C15×33.9 $S_x = 42.0$ in.^{3} — Manual Table 1-5

Required strength from Example F.2-2a

LRFD	ASD
$M_u = 108$ kip-ft	$M_a = 71.9$ kip-ft

Calculate the nominal flexural strength, M_n

Per the User Note in Section F2, all ASTM A36 C- and MC-shapes are compact.

For the center segment of a uniformly loaded beam braced at the ends and the fifth points, $C_b = 1.0$ — Manual Table 3-1

$L_p = 3.75$ ft
$L_r = 14.5$ ft — Manual Table 3-8

For a compact channel with $L_p < L_b \le L_r$, the lesser of the flexural yielding limit state or the inelastic lateral-torsional buckling limit-state controls the available flexural strength.

Lateral-torsional buckling limit state

From Example F.2-1b, $M_p = 1830$ kip-in.

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \quad \text{Eqn. F2-2}$$

$$M_n = 1.0\left[1830\text{ kip-in.} - \left(1830\text{ kip-in.} - 0.7\left(36\text{ ksi}\right)\left(42.0\text{ in.}^3\right)\right)\left(\frac{5.00\text{ ft} - 3.75\text{ ft}}{14.5\text{ ft} - 3.75\text{ ft}}\right)\right] \le 1830\text{ kip-in.}$$

$= 1740$ kip-in. < 1830 kip-in. **o.k.**

$M_n = 1740$ kip-in. or 145 kip-ft

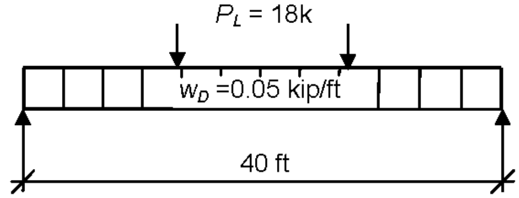
Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(145\text{ kip-ft}) = 131\text{ kip-ft}$ 131 kip-ft > 108 kip-ft **o.k.**	$\Omega_b = 1.67$ $M_n / \Omega_b = 145\text{ kip-ft} / 1.67 = 86.8\text{ kip-ft}$ 86.8 kip-ft > 71.9 kip-ft **o.k.**

Example F.3a W-Shape Flexural Member with Noncompact Flanges in Strong-Axis Bending

Given:

Select an ASTM A992 W-shape beam with a simple span of 40 feet. The nominal loads are a uniform dead load of 0.05 kip/ft and two equal 18 kip concentrated live loads acting at the third points of the beam. The beam is continuously braced. Also calculate the deflection.



Beam Loading & Bracing Diagram
(Continuous bracing)

Note: A beam with noncompact flanges will be selected to demonstrate that the tabulated values of the *Steel Construction Manual* account for flange compactness.

Solution:

Material Properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Calculate the required flexural strength at midspan

LRFD	ASD
w_u = 1.2(0.05 kip/ft) = 0.0600 kip/ft P_u = 1.6(18.0 kips) = 28.8 kips $M_u = \dfrac{(0.06\text{ kip/ft})(40.0\text{ ft})^2}{8} + (28.8\text{ kips})\dfrac{40.0\text{ ft}}{3}$ = 396 kip-ft	w_a = 0.05 kip/ft P_a = 18.0 kips $M_a = \dfrac{(0.05\text{ kip/ft})(40.0\text{ ft})^2}{8} + (18.0\text{ kips})\dfrac{40.0\text{ ft}}{3}$ = 250 kip-ft

Select the lightest section with the required strength from the bold entries in Manual Table 3-2

Try W21×48. — Manual Table 3-2

This beam has a noncompact compression flange at F_y = 50 ksi as indicated by footnote "f" in Manual Table 3-2. This is also footnoted in Manual Table 1-1. — Manual Table 1-1

Check the available strength

LRFD	ASD
$\phi_b M_n = \phi_b M_{px}$ = 398 kip-ft > 396 kip-ft **o.k.**	$M_n / \Omega_b = M_{px} / \Omega_b$ = 265 kip-ft > 250 kip-ft **o.k.**

Manual Table 3-2

Note: the value M_{px} in Table 3-2 includes the strength reductions due to the noncompact nature of the shape

Calculate deflection

I_x = 959 in.^{4}

Manual Table 1-1

$$\Delta_{max} = \frac{5wl^4}{384EI} + \frac{Pl^3}{28EI}$$

$$= \frac{5(0.0500\text{ kip/ft})(40.0\text{ ft})^4(12\text{ in./ft})^3}{384\ (29{,}000\text{ ksi})(959\text{ in.}^4)} + \frac{18.0\text{ kips}(40.0\text{ ft})^3(12\text{ in./ft})^3}{28(29{,}000\text{ ksi})(959\text{ in}^4)}$$

$$= 2.66\text{ in.}$$

Manual Table 3-23 Diagrams 1 and 9

This deflection can be compared with the appropriate deflection limit for the application. Deflection will often be more critical than strength in beam design.

Example F.3b W-Shape Flexural Member with Noncompact Flanges in Strong-Axis Bending

Given:

Verify the results from Example F.3a by calculation using the provisions of the Specification.

Solution:

Material Properties:

W21×48 ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Geometric Properties:

S_x = 93.0 in.^{3} Z_x = 107 in.^{3} $\dfrac{b_f}{2t_f} = 9.47$ — Manual Table 1-1

Required Strength from Example F.3a

LRFD	ASD
M_u = 396 kip-ft	M_a = 250 kip-ft

Check flange slenderness

$$\lambda = \frac{b_f}{2t_f} = 9.47$$

The limiting width-thickness ratios for the compression flange are: — Table B4.1 Case 1

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 9.15$$

$$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}} = 1.0\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 24.1$$

$\lambda_{rf} > \lambda > \lambda_{pf}$; therefore, the compression flange is noncompact. This could also be determined from the footnote "f" in Manual Table 1-1.

Calculate the nominal flexural strength, M_n

Since the beam is continuously braced, and therefore not subject to lateral-torsional buckling, the available strength is governed by Section F3

$M_p = F_yZ_x$ = 50 ksi(107 in.^{3}) = 5350 kip-in. or 446 kip-ft.

$$M_n = \left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right]$$ — Eqn. F3-1

$$M_n = \left[5350\text{ kip-in.} - \left(5350\text{ kip-in} - 0.7\left(50\text{ ksi}\right)\left(93.0\text{ in.}^3\right)\right)\left(\frac{9.47 - 9.15}{24.1 - 9.15}\right)\right]$$

$= 5310$ kip-in. or 442 kip-ft

Calculate the available flexural strength

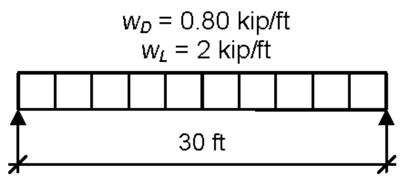
LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(442\text{ kip-ft})$ $= 398\text{ kip-ft} > 396\text{ kip-ft}$ **o.k.**	$\Omega = 1.67$ $M_n/\Omega = 442\text{ kip-ft}/1.67$ $= 265\text{ kip-ft} > 250\text{ kip-ft}$ **o.k.**

Note that these available strengths are identical to the tabulated values in Manual Table 3-2, which account for the noncompact flange.

Example F.4 W-shape Flexural Member, Selection by Moment of Inertia for Strong-Axis Bending

Given:

Select an ASTM A992 W-shape flexural member by the moment of inertia, to limit the live load deflection to 1 in. The span length is 30 ft. The nominal loads are a uniform dead load of 0.80 kip/ft and a uniform live load of 2 kip/ft. Assume the beam is continuously braced.



Beam Loading & Bracing Diagram
(Full lateral support)

Solution:

Material Properties:

Manual Table 2-3

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.80 \text{ kip/ft}) + 1.6(2 \text{ kip/ft})$ $= 4.16 \text{ kip/ft}$ $M_u = \dfrac{4.16 \text{ kip/ft}(30.0 \text{ ft})^2}{8} = 468 \text{ kip-ft}$	$w_a = 0.80 \text{ kip/ft} + 2 \text{ kip/ft}$ $= 2.80 \text{ kip/ft}$ $M_a = \dfrac{2.80 \text{ kip/ft}(30.0 \text{ ft})^2}{8} = 315 \text{ kip-ft}$

Calculate the minimum required moment of inertia

The maximum deflection, Δ_{max}, occurs at mid-span and is calculated as

$$\Delta_{max} = \frac{5wl^4}{384EI}$$

Manual Table 3-23 Diagram 1

Rearranging and substituting $\Delta_{max} = 1.00$ in.

$$I_{min} = \frac{5(2.00 \text{ kips/ft})(30.0 \text{ ft})^4 (12\text{in./ft})^3}{384(29{,}000 \text{ ksi})(1.00 \text{ in.})} = 1{,}260 \text{ in.}^4$$

Select the lightest section with the required moment of inertia from the bold entries in Manual Table 3-3

Try a W24×55

Manual Table 1-1

$I_x = 1{,}350 \text{ in.}^4 > 1{,}260. \text{ in.}^4$ **o.k.**

Because the W24×55 is continuously braced and compact, its strength is governed by the yielding limit state and Section F2.1

Obtain the available strength from Manual Table 3-2

LRFD	ASD
$\phi_b M_n = \phi_b M_{px} = 503$ kip-ft 503 kip-ft > 468 kip-ft **o.k.**	$M_n / \Omega_b = M_{px} / \Omega_b = 334$ kip-ft 334 kip-ft > 315 kip-ft **o.k.**

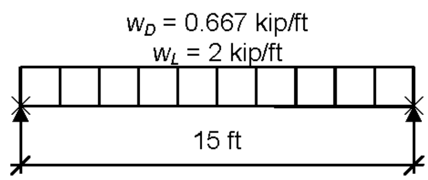
Manual Table 3-2

Example F.5 I-shaped Flexural Member in Minor-Axis Bending

Given:

Select an ASTM A992 W-shape beam loaded in its minor axis with a simple span of 15 ft. The nominal loads are a total uniform dead load of 0.667 kip/ft and a uniform live load of 2 kip/ft. Limit the live load deflection to L/240. Assume the beam is braced at the ends only.

Note: Although not a common design case, this example is being used to illustrate Specification Section F6 (I-shaped members and channels bent about their minor axis).



Beam Loading & Bracing Diagram
(Braced at ends only)

Solution:

Material Properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi

Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
w_u = 1.2(0.667 kip/ft) + 1.6(2 kip/ft) = 4.00 kip/ft $M_u = \dfrac{4.00\text{kip/ft}\left(15.0\text{ ft}\right)^2}{8}$ = 113 kip-ft	w_a = 0.667 kip/ft + 2 kip/ft = 2.67 kip/ft $M_a = \dfrac{2.67\text{kip/ft}\left(15.0\text{ ft}\right)^2}{8}$ = 75.1 kip-ft

Calculate the minimum required moment of inertia

$$\Delta_{max} = \frac{L}{240} = \frac{15.0\text{ ft(12 in./ft)}}{240} = 0.750\text{ in.}$$

$$I_{req} = \frac{5wl^4}{384E\Delta_{max}} = \frac{5\left(2.00\text{ kip/ft}\right)\left(15.0\text{ ft}\right)^4\left(12\text{ in./ft}\right)^3}{384\left(29{,}000\text{ ksi}\right)\left(0.750\text{ in.}\right)} = 105\text{ in.}^4$$

Manual Table 3-23 Diagram 1

Select the lightest section from the bold entries in Manual Table 3-5, due to the likelihood that deflection will govern this design.

Try a W12×58

Geometric Properties:

W12×58 S_y = 21.4 in.^{3} Z_y = 32.5 in.^{3} I_y = 107 in.^{4} — Manual Table 1-1

I_y = 107 in.^{4} > 105 in.^{4} **o.k.**

Specification Section F6 applies. Since the W12×58 has compact flanges per the User Note in this Section, the yielding limit state governs the design.

$M_n = M_p = F_yZ_y \le 1.6F_yS_y$ — Eqn. F6-1

$= 50 \text{ ksi}(32.5 \text{ in}^3) \le 1.6(50 \text{ ksi})(21.4 \text{ in}^3)$

= 1630 kip-in. ≤ 1710 kip-in. **o.k.**

= 1630 kip-in. or 136 kip-ft

Calculate the available flexural strength

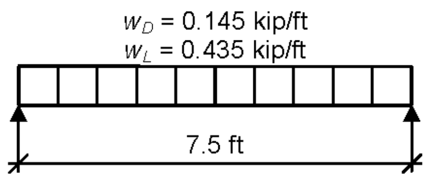
LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
ϕ_bM_n = 0.90(136 kip-ft) = 122 kip-ft	$M_n / \Omega_b = \dfrac{136 \text{ kip-ft}}{1.67}$ = 81.4 kip-ft
122 kip-ft > 113 kip-ft **o.k.**	81.4 kip-ft > 75.1 kip-ft **o.k.**

Section F1

Example F.6 HSS Flexural Member with Compact Flange

Given:

Select a square ASTM A500 Gr. B HSS beam to span 7.5 feet. The nominal loads are a uniform dead load of 0.145 kip/ft and a uniform live load of 0.435 kip/ft. Limit the live load deflection to $L/240$. Assume the beam is continuously braced.



Beam Loading & Bracing Diagram
(Full lateral support)

Solution:

Material Properties:

ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi — Manual Table 2-3

Calculate the required strength

LRFD	ASD
w_u = 1.2(0.145 kip/ft) + 1.6(0.435 kip/ft) = 0.870 kip/ft $M_u = \dfrac{(0.870\text{ kip/ft})(7.50\text{ ft})^2}{8} = 6.12$ kip-ft	w_a = 0.145 kip/ft + 0.435 kip/ft = 0.580 kip/ft $M_a = \dfrac{(0.580\text{ kip/ft})(7.50\text{ ft})^2}{8} = 4.08$ kip-ft

Calculate the minimum required moment of inertia

Manual Table 3-23 Diagram 1

$$\Delta_{max} = \frac{L}{240} = \frac{7.50\text{ ft}(12\text{ in./ft})}{240} = 0.375\text{ in.}$$

$$I_{req} = \frac{5wl^4}{384E\Delta_{max}}$$

$$= \frac{5(0.435\text{ kip/ft})(7.50\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(0.375\text{ in.})} = 2.85\text{ in.}^4$$

Select an HSS with a minimum I_x of 2.85 in.^{4}, using Manual Table 1-12, having adequate available strength, using Manual Table 3-13.

Try HSS3×3×¼

I_x = 3.02 in.^{4} > 2.85 in.^{4} **o.k.** — Manual Table 1-12

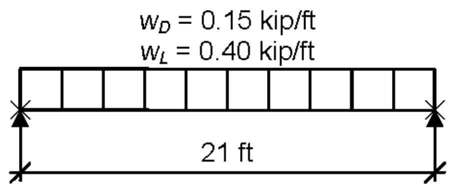
Obtain the available strength from Table 3-13 — Manual Table 3-13

LRFD	ASD
$\phi_b M_n$ = 8.55 kip-ft > 6.12 kip-ft **o.k.**	M_n / Ω_b = 5.69 kip-ft > 4.08 kip-ft **o.k.**

Example F.7a HSS Flexural Member with Noncompact Flange

Given:

Select a rectangular ASTM A500 Gr. B HSS beam with a span of 21 ft. The nominal loads include a uniform dead load of 0.15 kip/ft and a uniform live load of 0.40 kip/ft. Limit the live load deflection to L/240. Assume the beam is braced at the end points only. A noncompact member was selected here to illustrate the relative ease of selecting noncompact shapes from the Manual, as compared to designing a similar shape by applying the Specification requirements directly, as shown in Example F.7b.



Beam Loading & Bracing Diagram
(Braced at end points only)

Solution:

Material Properties:

ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi — Manual Table 2-3

Calculate the required strength

LRFD	ASD
$w_u = 1.2(0.15 \text{ kip/ft}) + 1.6(0.40 \text{ kip/ft})$ $= 0.820$ kip/ft $M_u = \dfrac{0.820 \text{ kip/ft}(21.0 \text{ ft})^2}{8} = 45.2$ kip-ft	$w_a = 0.15 \text{ kip/ft} + 0.40 \text{ kip/ft}$ $= 0.550$ kip/ft $M_a = \dfrac{0.550 \text{ kip/ft}(21.0 \text{ ft})^2}{8} = 30.3$ kip-ft

Calculate the minimum moment of inertia

$$\Delta_{max} = \frac{L}{240} = \frac{21.0 \text{ ft}(12 \text{ in./ft})}{240} = 1.05 \text{ in.}$$

$$\Delta_{max} = \frac{5wl^4}{384EI}$$

Manual Table 3-23 Diagram 1

Rearranging and substituting Δ_{max} = 1.05 in.

$$I_{min} = \frac{5(0.400 \text{ kip/ft})(21.0 \text{ ft})^4(12 \text{ in./ft})^3}{384(29{,}000 \text{ ksi})(1.05 \text{ in.})} = 57.5 \text{ in.}^4$$

Select a rectangular HSS with a minimum I_x of 57.5 in.^{4}, using Manual Table 1-11, having adequate available strength, using Manual Table 3-12.

Try a HSS10×6×3⁄16 oriented in the strong direction. This rectangular HSS section was purposely selected for illustration purposes because it has a noncompact flange.

I_x = 74.6 in.^{4} > 57.5 in.^{4} **o.k.**

Manual Table 1-11

Obtain the available strength from Table 3-12

LRFD	ASD
$\phi_b M_n$ = 57.0 kip-ft > 45.2 kip-ft **o.k.**	M_n / Ω_b = 37.9 kip-ft > 30.3 kip-ft **o.k.**

Manual Table 3-12

Example F.7b HSS Flexural Member with Noncompact Flanges

Given:

Notice that in Example F.7a the required information was easily determined by consulting the tables of the *Steel Construction Manual*. The purpose of the following calculation is to demonstrate the use of the Specification equations to calculate the flexural strength of a HSS member with a noncompact compression flange.

Solution:

Material Properties:

HSS10×6×3⁄16 ASTM A500 Gr. B $F_y = 46$ ksi $F_u = 58$ ksi

Manual Table 2-3

Geometric Properties:

HSS10×6×3⁄16 $Z_x = 18.0$ in.^{3} $S_x = 14.9$ in.^{3}

Manual Table 1-11

Check for flange compactness

$$\lambda = \frac{b}{t} = 31.5$$

Manual Table 1-11

The limiting ratio for a compact HSS flange in flexure is

$$\lambda_p = 1.12\sqrt{\frac{E}{F_y}} = 1.12\sqrt{\frac{29{,}000 \text{ ksi}}{46 \text{ ksi}}} = 28.1$$

Table B4.1, Case 12

Check flange slenderness

The limiting ratio for a slender HSS flange in flexure is

$$\lambda_r = 1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29{,}000 \text{ ksi}}{46 \text{ ksi}}} = 35.2$$

Table B4.1, Case 12

$\lambda_p < \lambda < \lambda_r$ therefore the flange is noncompact. For this situation, Specification Eqn. F7-2 applies

Section F7

Check web slenderness

$$\lambda = \frac{h}{t} = 54.5$$

Manual Table 1-11

The limiting ratio for a compact HSS web in bending is

$$\lambda_p = 2.42\sqrt{\frac{E}{F_y}} = 2.42\sqrt{\frac{29{,}000 \text{ ksi}}{46 \text{ ksi}}} = 60.8 > 54.5$$, therefore the web is compact.

Table B4.1 Case 13

For HSS with noncompact flanges and compact webs, Specification Section F7.2(b) applies.

$M_p = F_yZ = 46\text{ ksi}\left(18.0\text{ in.}^3\right) = 828\text{ kip-in.}$

$$M_n = M_p - \left(M_p - F_yS\right)\left(3.57\frac{b}{t}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \qquad \text{Eqn. F7-2}$$

$$= \left(828\text{ kip-in.}\right) - \left[\left(828\text{ kip-in.}\right) - \left(46\text{ ksi}\right)\left(14.9\text{ in.}^3\right)\right]\left(3.57\left(31.5\right)\sqrt{\frac{46\text{ ksi}}{29{,}000\text{ ksi}}} - 4.0\right)$$

$= 760$ kip-in. or 63.3 kip-ft

Calculate the available flexural strength

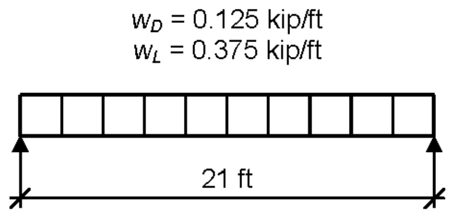
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90\left(63.3\text{ kip-ft}\right) = 57.0\text{ kip-ft}$	$\Omega_b = 1.67$ $M_n / \Omega_b = \dfrac{63.3\text{ kip-ft}}{1.67} = 37.9\text{ kip-ft}$

Section F1

Example F.8a HSS Flexural Member with Slender Flanges

Given:

Verify the strength of an ASTM A500 Gr. B HSS8×8×3⁄16 with a span of 21 ft. The nominal loads are a dead load of 0.125 kip/ft and a live load of 0.375 kip/ft. Limit the live load deflection to $L/240$.



Beam Loading & Bracing Diagram
(Full lateral support)

Solution:

Material Properties:

HSS8×8×3⁄16 ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi — Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
w_u = 1.2(0.125 kip/ft) + 1.6(0.375 kip/ft) = 0.750 kip/ft $M_u = \dfrac{(0.750\text{ kip/ft})(21.0\text{ ft})^2}{8} = 41.3\text{ kip-ft}$	w_a = 0.125 kip/ft + 0.375 kip/ft = 0.500 kip/ft $M_a = \dfrac{(0.500\text{ kip/ft})(21.0\text{ ft})^2}{8} = 27.6\text{ kip-ft}$

Obtain the available flexural strength of the HSS8×8×3⁄16 from Manual Table 3-13

LRFD	ASD
$\phi_b M_n = 43.3\text{ kip-ft} > 41.3\text{ kip-ft}$ **o.k.**	$M_n/\Omega_b = 28.8\text{ kip-ft} > 27.6\text{ kip-ft}$ **o.k.**

Manual Table 3-13

Note that the strengths given in Table 3-13 incorporate the effects of nonslender and slender elements.

Check deflection

$$\Delta_{max} = \frac{l}{240} = \frac{(21.0\text{ ft})(12\text{ in./ft})}{240} = 1.05\text{ in.}$$

I_x = 54.4 in.^{4} — Manual Table 1-12

$$\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5(0.375\text{ kip/ft})(21.0\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(54.4\text{ in.}^4)} = 1.04\text{ in.} < 1.05\text{ in.} \quad \textbf{o.k.}$$

Manual Table 3-23 Case 1

Example F.8b HSS Flexural Member with Slender Flanges

Given:

In Example F.8a the available strengths were easily determined from the tables of the *Steel Construction Manual*. The purpose of the following calculation is to demonstrate the use of the Specification equations to calculate the flexural strength of a HSS member with slender flanges.

Solution:

Material Properties:

HSS8×8×3⁄16 ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi — Manual Table 2-3

Geometric Properties:

HSS8×8×3⁄16 I_x = 54.4 in.^{4} Z_x = 15.7 in.^{3} S_x = 13.6 in.^{3}
b_f = 8.0 in. t_f = 0.174 in. b/t = 43.0 h/t = 43.0 — Manual Table 1-12

Required strength from Example F8.a

LRFD	ASD
M_u = 41.3 kip-ft	M_a = 27.6 kip-ft

Check flange slenderness

The assumed outside radius of the corners of HSS shapes is 1.5*t*. The design thickness is used to check compactness. The limiting ratio for HSS flanges in bending is as follows:

The limiting ratio for a slender HSS flange in flexure is:

$$\lambda_r = 1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 35.2$$

Table B4.1 Case 12

$\lambda = \dfrac{b}{t}$ = 43.0 > λ_r, therefore flange is slender.

Check for web compactness

The limiting ratio for a compact web in bending is:

$$\lambda_p = 2.42\sqrt{\frac{E}{F_y}} = 2.42\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 60.8$$

Table B4.1 Case 13

$\lambda = \dfrac{h}{t}$ = 43.0 < λ_p, therefore the web is compact.

For HSS sections with slender flanges and compact webs, Specification Section F7.2(c) applies.

Eqn. F7-3

$M_n = F_y S_{eff}$

Where S_{eff} is the effective section modulus determined with the effective width of the compression flange taken as:

Eqn. F7-4

$$b_e = 1.92t\sqrt{\frac{E}{F_y}}\left[1-\frac{0.38}{b/t}\sqrt{\frac{E}{F_y}}\right] \leq b$$

$$b_e = 1.92\left(0.174\text{ in.}\right)\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}}\left[1-\frac{0.38}{43.0}\sqrt{\frac{29{,}000\text{ ksi}}{46\text{ ksi}}}\right] = 6.53\text{ in.}$$

Section B4.2d

$b = 8.00\text{ in.} - 3\left(0.174\text{ in.}\right) = 7.48\text{ in.} > 6.53\text{ in.}$ **o.k.**

The ineffective width of the compression flange is:

$b - b_e = 7.48\text{ in.} – 6.53\text{ in.} = 0.950\text{ in.}$

An exact calculation of the effective moment of inertia and section modulus could be performed taking into account the ineffective width of the compression flange and the resulting neutral axis shift. Alternatively, a simpler but slightly conservative calculation can be performed by removing the ineffective width symmetrically from both the top and bottom flanges.

$$I_{eff} \approx 54.4\text{ in.}^4 - 2\left[\left(0.950\text{ in.}\right)\left(0.174\text{ in.}\right)\left(3.91\right)^2 + \frac{\left(0.950\text{ in.}\right)\left(0.174\text{ in.}\right)^3}{12}\right] = 49.3\text{ in.}^4$$

The effective section modulus can now be calculated as follows:

$$S_{eff} = \frac{I_{eff}}{d/2} = \frac{49.3\text{ in.}^4}{\left(8.00\text{ in.}\right)/2} = 12.3\text{ in.}^3$$

Eqn. F7-3

Calculate the nominal flexural strength, M_n

$M_n = F_y S_{eff} = 46\text{ ksi}(12.3\text{ in.}^3) = 566\text{ kip-in. or }47.2\text{ kip-ft}$

Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90\left(47.2\text{ kip-ft}\right)$ $= 42.5\text{ kip-ft} > 41.3\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $M_n / \Omega_b = \dfrac{47.2\text{ kip-ft}}{1.67}$ $= 28.3\text{ kip-ft} > 27.6\text{ kip-ft}$ **o.k.**

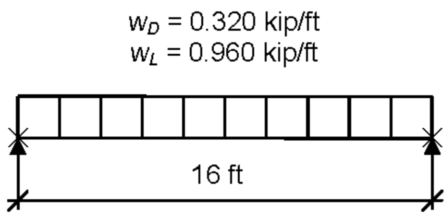
Section F1

Note that the calculated available strengths are somewhat lower than those in Manual Table 3-13 due to the use of the conservative calculation of the approximate effective section modulus above.

Example F.9a Pipe Flexural Member

Given:

Select an ASTM A53 Gr. B Pipe shape with a simple span of 16 ft. The nominal loads are a total uniform dead load of 0.32 kip/ft and a uniform live load of 0.96 kip/ft. Assume there is no deflection limit for this beam. The beam is braced only at the ends.



Beam Loading & Bracing Diagram
(Braced at end points only)

Solution:

Material Properties:

ASTM A53 Gr. B $F_y = 35$ ksi $F_u = 60$ ksi Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.32 \text{ kip/ft}) + 1.6(0.96 \text{ kip/ft})$ $= 1.92 \text{ kip/ft}$ $M_u = \dfrac{1.92 \text{ kip/ft}\left(16.0 \text{ ft}\right)^2}{8} = 61.4 \text{ kip-ft}$	$w_a = 0.32 \text{ kip/ft} + 0.96 \text{ kip/ft}$ $= 1.28 \text{ kip/ft}$ $M_a = \dfrac{1.28 \text{ kip/ft}\left(16.0 \text{ ft}\right)^2}{8} = 41.0 \text{ kip-ft}$

Select a member from Manual Table 3-15 having the required strength

Select Pipe 8 X-Strong.

LRFD	ASD
$\phi_b M_n = 81.4 \text{ kip-ft} > 61.4 \text{ kip-ft}$ **o.k.**	$M_n / \Omega_b = 54.1 \text{ kip-ft} > 41.0 \text{ kip-ft}$ **o.k.**

Manual Table 3-15

Example F.9b Pipe Flexural Member

Given:

The available strength in Example F.9a was easily determined using Manual Table 3-15. The following calculation demonstrates the calculation of the available strength by directly applying the requirements of the Specification.

Solution:

Material Properties:

ASTM A53 Gr. B $F_y = 35$ ksi $F_u = 60$ ksi — Manual Table 2-3

Geometric Properties:

Pipe 8 X-Strong $Z = 31.0$ in.^{3} $D = 8.63$ in. $t = 0.465$ in. $D/t = 18.5$ — Manual Table 1-14

Required flexural strength from Example F.9a

LRFD	ASD
$M_u = 61.4$ kip-ft	$M_a = 41.0$ kip-ft

Check compactness

For circular HSS in flexure, the limiting diameter-to-thickness ratio for a compact section is:

$$\lambda_p = \frac{0.07E}{F_y} = \frac{0.07(29{,}000 \text{ ksi})}{35 \text{ ksi}} = 58.0$$

Table B4.1 Case 15

$\lambda = \dfrac{D}{t} = 18.5 < \lambda_p$, therefore the section is compact and the limit state of flange local buckling does not apply.

$\dfrac{D}{t} < \dfrac{0.45E}{F_y} = 373$, therefore Specification Section F8 applies. — Section F8

Calculate the nominal flexural strength based on the flexural yielding limit state

$$M_n = M_p = F_y Z = (35 \text{ ksi})(31.0 \text{ in.}^3) = 1{,}090 \text{ kip-in. or } 90.4 \text{ kip-ft}$$

Eqn. F8-1

Calculate the available flexural strength

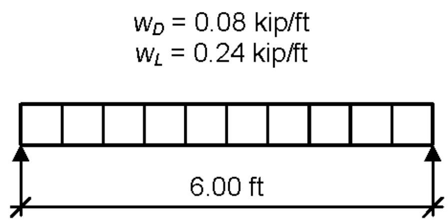
LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(90.4 \text{ kip-ft})$	$M_n / \Omega_b = \dfrac{90.4 \text{ kip-ft}}{1.67}$
$= 81.4$ kip-ft > 61.4 kip-ft **o.k.**	$= 54.1$ kip-ft > 41.0 kip-ft **o.k.**

Section F1

Example F.10 WT Shape Flexural Member

Given:

Select an ASTM A992 WT beam with a simple span of 6 ft. The toe of the stem of the WT is in tension. The nominal loads are a uniform dead load of 0.08 kip/ft and a uniform live load of 0.24 kip/ft. There is no deflection limit for this member. Assume full lateral support.



Beam Loading & Bracing Diagram
(Full lateral support)

Solution:

Material properties:

ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi — Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.08 \text{ kip/ft}) + 1.6(0.24 \text{ kip/ft})$ $= 0.480 \text{ kip/ft}$ $M_u = \dfrac{0.480 \text{ kip/ft}\left(6.00 \text{ ft}\right)^2}{8} = 2.16 \text{ kip-ft}$	$w_a = 0.08 \text{ kip/ft} + 0.24 \text{ kip/ft}$ $= 0.320 \text{ kip/ft}$ $M_a = \dfrac{0.320 \text{ kip/ft}\left(6.00 \text{ ft}\right)^2}{8} = 1.44 \text{ kip-ft}$

Try WT 5×6

Geometric Properties:

WT 5×6 $I_x = 4.35 \text{ in.}^4$ $Z_x = 2.20 \text{ in.}^3$ $S_x = 1.22 \text{ in.}^3$ $b_f = 3.96$ in. — Manual Table 1-8

$t_f = 0.210$ in. $\bar{y} = 1.36$ in. $S_{xc} = \dfrac{I_x}{y_c} = \dfrac{4.35 \text{ in.}^4}{1.36 \text{ in.}} = 3.20 \text{ in.}^3$

Calculate the nominal flexural strength, M_n

Flexural yielding limit state

$M_p = F_y Z_x \leq 1.6 M_y$ for stems in tension — Eqn. F9-2

$$1.6M_y = 1.6F_yS_x = 1.6\left(50 \text{ ksi}\right)\left(1.22 \text{ in.}^3\right) = 97.6 \text{ kip-in.}$$

$M_p = F_y Z_x = (50\text{ ksi})(2.20\text{ in.}^3) = 110\text{ kip-in.} > 97.6\text{ kip-in.}$, therefore use

M_p = 97.6 kip-in. or 8.13 kip-ft

Lateral-torsional buckling limit state — Section F9.2

Because the WT is fully braced and the stem is in tension, no check of the lateral-torsional buckling limit state is required. Note that if the stem is in compression, Equation F9-4 must be checked even for fully braced members, since the equation converges to the web local buckling limit state check at an unbraced length of zero. See Commentary Section F9.

Flange local buckling limit state

Check flange compactness

$$\lambda = \frac{b_f}{2t_f} = \frac{3.96\text{ in.}}{2(0.210\text{ in.})} = 9.43$$, or look up in Manual Table 1-1

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 9.15 < 9.43$$; therefore the flange is not compact. — Table B4.1, Case 7

Check flange slenderness

$$\lambda_r = 1.0\sqrt{\frac{E}{F_y}} = 1.0\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 24.1 > 9.43$$, therefore the flange in not slender. — Table B4.1, Case 7

Calculate critical flange local buckling stress

For a Tee with a noncompact flange, the critical stress is:

$$F_{cr} = F_y\left(1.19\text{-}0.50\left(\frac{b_f}{2t_f}\right)\sqrt{\frac{F_y}{E}}\right) = (50\text{ ksi})\left(1.19\text{-}(0.50)(9.43)\sqrt{\frac{50\text{ ksi}}{29{,}000\text{ ksi}}}\right) = 49.7\text{ ksi}$$ — Eqn. F9-7

Calculate the nominal flexural strength

$M_n = F_{cr}S_{xc}$ = 49.7 ksi(3.20 in.^{3}) = 159 kip-in. or 13.3 kip-ft **does not control** — Eqn. F9-6

Calculate the available flexural strength

$M_n = M_p$ = 8.13 kip-ft **yielding limit state controls** — Eqn. F9-1

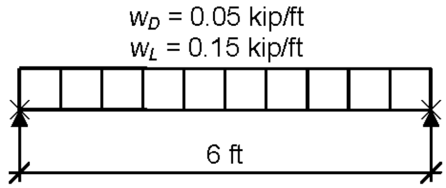
LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(8.13\text{ kip-ft})$	$M_n / \Omega_b = \dfrac{8.13\text{ kip-ft}}{1.67}$
$= 7.32\text{ kip-ft} > 2.16\text{ kip-ft}$ **o.k.**	$= 4.87\text{ kip-ft} > 1.44\text{ kip-ft}$ **o.k.**

Section F1

Example F.11 Single Angle Flexural Member

Given:

Select an ASTM A36 single angle with a simple span of 6 ft. The vertical leg of the single angle is down and the toe is in tension. The nominal loads are a uniform dead load of 0.05 kip/ft and a uniform live load of 0.15 kip/ft. There is no deflection limit for this angle. Conservatively assume C_b = 1.0. Assume bending about the geometric *x*-*x* axis and that there is no lateral-torsional restraint.



Beam Loading & Bracing Diagram
(Braced at end points only)

Solution:

Material Properties:

ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-3

Calculate the required flexural strength

LRFD	ASD
w_u = 1.2(0.05 kip/ft) + 1.6(0.15 kip/ft) = 0.300 kip/ft $M_u = \dfrac{(0.300\text{ kip/ft})(6\text{ ft})^2}{8} = 1.35$ kip-ft	w_a = 0.05 kip/ft + 0.15 kip/ft = 0.200 kip/ft $M_a = \dfrac{(0.200\text{ kip/ft})(6\text{ ft})^2}{8} = 0.900$ kip-ft

Try L4×4×¼

Geometric Properties:

L4×4×¼ S_x = 1.03 in.^{3} I_x = 3.00 in.^{4} $\bar{y} = 1.08$ in. — Manual Table 1-7

Calculate the nominal flexural strength, M_n

For all calculations, M_y is taken as 0.80 times the yield moment calculated using the geometric section modulus.

$$M_y = 0.80S_xF_y = 0.80\left(1.03\text{ in.}^3\right)\left(36\text{ ksi}\right) = 29.7\text{ kip-in.}$$ Section F10.2

Flexural yielding limit state

$$M_n = 1.5M_y = 1.5\left(29.7\text{ kip-in.}\right)$$ Eqn. F10-1
$$= 44.6\text{ kip-in. or } 3.71\text{ kip-ft}$$

Lateral-torsional buckling limit state

Determine M_e

For bending about one of the geometric axes of an equal-leg angle without continuous lateral-torsional restraint and with maximum tension at the toe, use Equation F10-4b.

Eqn. F10-4b

$$M_e = \frac{0.66Eb^4tC_b}{L^2}\left(\sqrt{1+0.78\left(\frac{Lt}{b^2}\right)^2}+1\right)$$

$$= \frac{0.66(29{,}000\text{ ksi})(4.00\text{ in.})^4(0.250\text{ in.})(1.0)}{(72.0\text{ in.})^2}\left(\sqrt{1+0.78\left(\frac{(72.0\text{ in.})(0.250\text{ in.})}{(4.00\text{ in.})^2}\right)^2}+1\right)$$

= 569 kip-in. > 29.7 kip-in. therefore, Equation F10-3 is applicable.

Eqn. F10-3

$$M_n = \left(1.92-1.17\sqrt{\frac{M_y}{M_e}}\right)M_y \le 1.5M_y$$

$$= \left(1.92-1.17\sqrt{\frac{29.7\text{ kip-in.}}{569\text{ kip-in.}}}\right)29.7\text{ kip-in.} \le 1.5(29.7\text{ kip-in.})$$

= 49.1 kip-in. > 44.6 kip-in.; therefore M_n = 44.6 kip-in.

Leg local buckling limit state

Check slenderness of outstanding leg in compression.

$$\lambda = \frac{b}{t} = \frac{4.00\text{ in.}}{0.250\text{ in.}} = 16.0$$

The limiting width-thickness ratios are:

Table B4.1
Case 6

$$\lambda_p = 0.54\sqrt{\frac{E}{F_y}} = 0.54\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} = 15.3$$

$$\lambda_r = 0.91\sqrt{\frac{E}{F_y}} = 0.91\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} = 25.8$$

$\lambda_p < \lambda < \lambda_r$ Therefore the outstanding leg is noncompact in flexure.

Eqn. F10-7

$$M_n = F_yS_c\left(2.43-1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right)$$

$$S_c = 0.80\left(\frac{3.00\text{ in.}^4}{1.08\text{ in.}}\right) = 2.22\text{ in.}^3$$

$$M_n = (36\text{ ksi})(2.22\text{ in.}^3)\left(2.43-1.72(16.0)\sqrt{\frac{36\text{ ksi}}{29{,}000\text{ ksi}}}\right) = 117\text{ kip-in. or } 9.73\text{ kip-ft}$$

The flexural yielding limit state controls.

M_n = 44.6 kip-in. or 3.72 kip-ft.

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(3.72\text{ kip-ft})$	$M_n / \Omega_b = \dfrac{3.72\text{ kip-ft}}{1.67}$
$= 3.35\text{ kip-ft} > 1.35\text{ kip-ft}$ **o.k.**	$= 2.23\text{ kip-ft} > 0.900\text{ kip-ft}$ **o.k.**

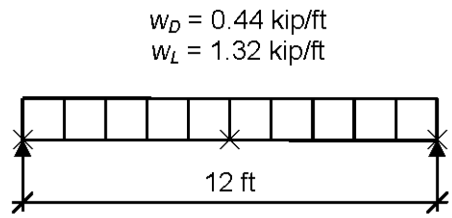
Section F1

Note: In this example, the toe of the vertical leg of the single angle is in tension. The designer should also consider the possibility that restrained end conditions of a single angle member could unintentionally cause the toe of the vertical leg to be in compression.

Example F.12 Rectangular Bar in Strong-Axis Bending

Given:

Select an ASTM A36 rectangular bar with a span of 12 ft. The bar is braced at the ends and at the midpoint. Conservatively use C_b = 1.0. Limit the depth of the member to 5 in. The nominal loads are a total uniform dead load of 0.44 kip/ft and a uniform live load of 1.32 kip/ft.



Beam Loading & Bracing Diagram
(Bracing at ends & midpoint)

Solution:

Material Properties:

ASTM A36 F_y = 36 ksi F_u = 58 ksi

Manual Table 2-4

LRFD	ASD
w_u = 1.2(0.44 kip/ft) + 1.6(1.32 kip/ft) = 2.64 kip/ft $M_u = \dfrac{2.64 \text{ kip/ft}(12.0 \text{ ft})^2}{8}$ = 47.5 kip-ft	w_a = 0.44 kip/ft + 1.32 kip/ft = 1.76 kip/ft $M_a = \dfrac{1.76\text{kip/ft}(12.0 \text{ ft})^2}{8}$ = 31.7 kip-ft

Try a 5 in.×3 in. bar.

Geometric Properties:

Rectangular bar 5×3

$$S_x = \frac{bd^2}{6} = \frac{(3.00 \text{ in.})(5.00 \text{ in.})^2}{6} = 12.5 \text{ in.}^3$$

$$Z_x = \frac{bd^2}{4} = \frac{(3.00 \text{ in.})(5.00 \text{ in.})^2}{4} = 18.8 \text{ in.}^3$$

Manual Table 17-27

Solution:

Calculate nominal flexural strength, M_n

Section F11.1

Flexural yielding limit state

Check limit

$$\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$$

$$\frac{(72.0\text{ in.})(5.00\text{ in.})}{(3.00\text{ in.})^2} \le \frac{0.08(29{,}000\text{ ksi})}{(36\text{ ksi})}$$

40.0 < 64.4, therefore the yielding limit state applies.

$$M_n = M_p = F_y Z \le 1.6M_y$$ Eqn. F11-1

$$1.6M_y = 1.6F_y S_x = 1.6(36\text{ ksi})(12.5\text{ in.}^3) = 720\text{ kip-in.}$$
$$M_p = F_y Z_x = (36\text{ ksi})(18.8\text{ in.}^3) = 677\text{ kip-in.} < 720\text{ kip-in.}$$

Use $M_n = M_p = 677$ kip-in. or 56.4 kip-ft

Lateral-torsional buckling limit state Section F11.1

As calculated above, $\frac{L_b d}{t^2} < \frac{0.08E}{F_y}$, therefore the lateral-torsional buckling limit state does not apply.

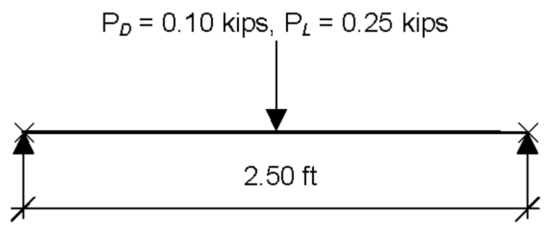
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(56.4\text{ kip-ft})$ $= 50.8\text{ kip-ft} > 47.5\text{ kip-ft}$ **o.k.**	$\Omega_b = 1.67$ $M_n / \Omega_b = \frac{56.4\text{ kip-ft}}{1.67}$ $= 33.8\text{ kip-ft} > 31.7\text{ kip-ft}$ **o.k.**

Section F1

Example F.13 Round Bar in Bending

Given:

Select an ASTM A36 round bar with a span of 2.50 feet. The bar is unbraced. The material is ASTM A36. Assume C_b = 1.0. Limit the diameter to 2 in. The nominal loads are a concentrated dead load of 0.10 kips and a concentrated live load of 0.25 kips at the center. The weight of the bar is negligible.



Beam Loading & Bracing Diagram
(Braced at end points only)

Solution:

Material Properties:

ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-4

Calculate the required flexural strength

LRFD	ASD
$P_u = 1.2(0.10 \text{ kip}) + 1.6(0.25 \text{ kip})$ $= 0.520 \text{ kip}$ $M_u = \dfrac{(0.520 \text{ kip})(2.50 \text{ ft})}{4} = 0.325 \text{ kip-ft}$	$P_a = 0.10 \text{ kip} + 0.25 \text{ kip}$ $= 0.350 \text{ kip}$ $M_a = \dfrac{(0.350 \text{ kip})(2.50 \text{ ft})}{4} = 0.219 \text{ kip-ft}$

Manual Table 3-23 Diagram 7

Try 1 in. diameter rod.

Geometric Properties:

Round bar

$$S_x = \frac{\pi d^3}{32} = \frac{\pi(1.00 \text{ in.})^3}{32} = 0.0982 \text{ in.}^3$$

$$Z_x = \frac{d^3}{6} = \frac{(1.00 \text{ in.})^3}{6} = 0.167 \text{ in.}^3$$

Manual Table 17-27

Calculate the nominal flexural strength, M_n

Flexural yielding limit state

$M_n = M_p = F_y Z_x \leq 1.6\ M_y$ — Eqn. F11-1

$$1.6M_y = 1.6F_y S_x = 1.6(36 \text{ ksi})(0.0982 \text{ in}^3) = 5.66 \text{ kip-in}$$

$$F_y Z_x = 36 \text{ ksi}(0.167 \text{ in}^3) = 6.01 \text{ kip-in} > 5.66 \text{ kip-in.}$$

Therefore, M_n = 5.66 kip-in. or 0.472 kip-ft

Lateral-torsional buckling limit state — Section F11.2

This limit state need not be considered for rounds.

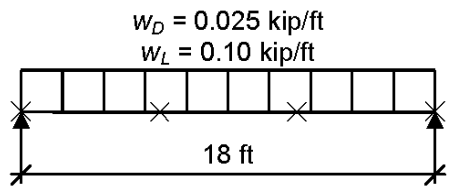
Calculate the available flexural strength — Section F1

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(0.472 \text{ kip-ft})$	$M_n / \Omega_b = \dfrac{0.472 \text{ kip-ft}}{1.67}$
$= 0.425 \text{ kip-ft} > 0.325 \text{ kip-ft}$ **o.k.**	$= 0.283 \text{ kip-ft} > 0.219 \text{ kip-ft}$ **o.k.**

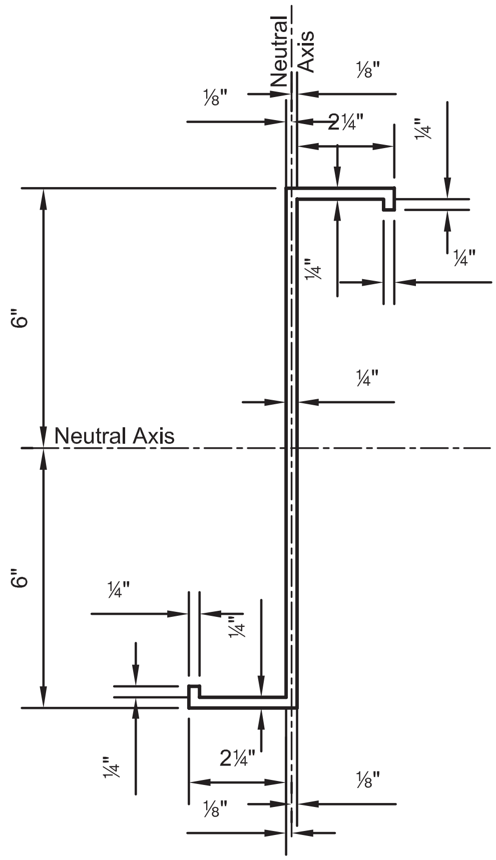
Example F.14 Point-Symmetrical Z-shape in Strong-Axis Bending

Given:

Determine the available strength of the ASTM A36 Z-shape shown for a simple span of 18 ft. The Z-shape is braced at 6 ft on center. Assume a C_b = 1.0. The nominal loads are a uniform dead load of 0.025 kip/ft and a uniform live load of 0.10 kip/ft. The profile of the purlin is shown below.



Beam Loading & Bracing Diagram
(Braced at ends and third points)



Solution:

Material properties:

Z Purlin F_y = 36 ksi F_u = 58 ksi Manual Table 2-4

Geometric Properties:

$t_w = t_f = 0.250$ in.

$$A = (2.50\text{ in.})(0.25\text{ in.})(2) + (0.25\text{ in.})(0.25\text{ in.})(2) + (11.5\text{ in.})(0.25\text{ in.}) = 4.25\text{ in.}^2$$

$$\begin{aligned}I_x &= \left[\frac{(0.25\text{ in.})(0.25\text{ in.})^3}{12} + (0.25\text{ in.})^2(5.63\text{ in.})^2\right](2) \\ &\quad + \left[\frac{(2.50\text{ in.})(0.25\text{ in.})^3}{12} + (2.50\text{ in.})(0.25\text{ in.})(5.88\text{ in.})^2\right](2) \\ &\quad + \frac{(0.25\text{ in.})(11.5\text{ in.})^3}{12} \\ &= 78.9\text{ in.}^4\end{aligned}$$

$$\bar{y} = 6.00\text{ in.}$$

$$S_x = \frac{I_x}{\bar{y}} = \frac{78.9\text{ in.}^4}{6.00\text{ in.}} = 13.2\text{ in.}^3$$

$$\begin{aligned}I_y &= \left[\frac{(0.25\text{ in.})(0.25\text{ in.})^3}{12} + (0.25\text{ in.})^2(2.25\text{ in.})^2\right](2) \\ &\quad + \left[\frac{(0.25\text{ in.})(2.50\text{ in.})^3}{12} + (2.50\text{ in.})(0.25\text{ in.})(1.13\text{ in.})^2\right](2) \\ &\quad + \frac{(11.5\text{ in.})(0.25\text{ in.})^3}{12} \\ &= 2.90\text{ in.}^4\end{aligned}$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2.90\text{ in.}^4}{4.25\text{ in.}^2}} = 0.826\text{ in.}$$

$$r_{ts} \approx \frac{b_f}{\sqrt{12\left(1 + \dfrac{ht_w}{6b_f t_f}\right)}} = \frac{2.50\text{ in.}}{\sqrt{12\left(1 + \dfrac{(11.5\text{ in.})(0.250\text{ in.})}{6(2.50\text{ in.})(0.250\text{ in.})}\right)}} = 0.543\text{ in.}$$

User Note Section F2.2

Calculate the required flexural strength

LRFD	ASD
w_u = 1.2(0.025 kip/ft) + 1.6(0.10 kip/ft) = 0.190 kip/ft	w_a = 0.025 kip/ft + 0.10 kip/ft = 0.125 kip/ft
$M_u = \dfrac{(0.190\text{ kip/ft})(18.0\text{ ft})^2}{8} = 7.70$ kip-ft	$M_a = \dfrac{(0.125\text{ kip/ft})(18.0\text{ ft})^2}{8} = 5.06$ kip-ft

Flexural yielding limit state

$F_n = F_y = 36$ ksi   Eqn. F12-2

$M_n = F_nS = 36\text{ ksi}\left(13.2\text{ in.}^3\right) = 475$ kip-in. or 39.6 kip-ft   Eqn. F12-1

Local buckling limit state

There are no specific local buckling provisions for Z-shapes in the Specification. Use provisions for rolled channels from Specification Table B4.1.

Check for flange slenderness

Conservatively neglecting the end return:

$$\lambda = \frac{b}{t_f} = \frac{2.50\text{ in.}}{0.250\text{ in.}} = 10.0$$

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} = 10.8 > 10.0\text{, therefore the flange is compact}$$

Table B4.1 Case 1

Check for web slenderness

$$\lambda = \frac{h}{t_w} = \frac{11.5\text{ in.}}{0.250\text{ in.}} = 46.0$$

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} = 107 > 46.0\text{, therefore the web is compact.}$$

Table B4.1 Case 9

Therefore, no limit state for local buckling applies.

Lateral-torsional buckling limit state

Per the User Note in Section F12, take the critical lateral-torsional buckling stress as half that of the equivalent channel.

Calculate limiting unbraced lengths

For bracing at 6 ft on center, $L_b = \left(6.00\text{ ft}\right)\left(12\text{ in./ft}\right) = 72.0$ in.

$$L_p = 1.76r_y\sqrt{\frac{E}{F_y}} = 1.76\left(0.826\text{ in.}\right)\sqrt{\frac{29{,}000\text{ ksi}}{36\text{ ksi}}} = 41.3\text{ in.} < 72.0\text{ in.}$$

Eqn. F2-5

$$L_r = 1.95r_{ts}\left(\frac{E}{0.7F_y}\right)\sqrt{\frac{Jc}{S_xh_0}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_yS_xh_0}{EJc}\right)^2}}$$

Eqn. F2-6

Per the user note in Specification Section F2, the square root term in Specification Equation F2-4 can conservatively be taken equal to one and Equation F2-6 becomes,

$$L_r = \pi r_{ts}\sqrt{\frac{E}{0.7F_y}} = \pi(0.543\text{ in.})\sqrt{\frac{29{,}000\text{ ksi}}{0.7(36\text{ ksi})}} = 57.9\text{ in.} < 72.0\text{ in.}$$

Calculate one half of the critical lateral-torsional buckling stress of the equivalent channel

$L_b > L_r$, therefore,

$$F_{cr} = (0.5)\frac{C_b\pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1+0.078\left(\frac{Jc}{S_x h_0}\right)\left(\frac{L_b}{r_{ts}}\right)^2}$$ Eqn. F2-4

Conservatively taking the square root term as 1.0,

$$F_{cr} = (0.5)\frac{C_b\pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} = (0.5)\frac{1.0\pi^2(29{,}000\text{ ksi})}{\left(\frac{72.0\text{ in.}}{0.543\text{ in.}}\right)^2} = 8.14\text{ ksi}$$

$F_n = F_{cr} \le F_y$ Eqn. F12-3

$= 8.14$ ksi < 36 ksi **o.k.**

Eqn. F12-1

$$M_n = F_n S$$
$$= (8.14\text{ ksi})(13.2\text{ in.}^3) = 107\text{ kip-in. or } 8.95\text{ kip-ft}$$ **controls**

Calculate the available strength

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(8.95\text{ kip-ft})$	$M_n/\Omega_b = \dfrac{8.95\text{ kip-ft}}{1.67}$
$= 8.06$ kip-ft > 7.70 kip-ft **o.k.**	$= 5.36$ kip-ft > 5.06 kip-ft **o.k.**

Section F1

CHAPTER G
DESIGN OF MEMBERS FOR SHEAR

INTRODUCTION

This chapter covers webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles, HSS sections, and shear in the weak direction of singly or doubly symmetric shapes.

Most of the formulas from this chapter are illustrated by example. Tables for all standard ASTM A992 W-shapes and ASTM A36 channels are included in the Manual. In the tables, where applicable, LRFD and ASD shear information is presented side-by-side for quick selection, design and verification.

LRFD shear strengths have been increased slightly over those in the previous LRFD Specification for members not subject to shear buckling. ASD strengths are essentially identical to those in the previous ASD Specification. LRFD and ASD will produce identical designs for the case where the live load effect is approximately three times the dead load effect.

G1. GENERAL PROVISIONS

The design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v are determined as follows:

V_n = nominal shear strength based on shear yielding or shear buckling

$$V_n = 0.6F_y A_w C_v \qquad \text{Eqn. G2-1}$$

ϕ_v = 0.90 (LRFD) $\quad\quad$ Ω_v = 1.67 (ASD).

Exception: For all current ASTM A6, W, S, and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26, and W12×14 for F_y = 50 ksi:

ϕ_v = 1.00 (LRFD) $\quad\quad$ Ω_v = 1.50 (ASD).

Section G2 does not utilize tension field action. Section G3 specifically addresses the use of tension field action.

Strong axis shear values are tabulated for W-shapes in Manual Tables 3-2 and 3-6, for S-shapes in Manual Table 3-7, for C-shapes in Manual Table 3-8 and for MC-shapes in Manual Table 3-9. Weak axis shear values for W-shapes, S-shapes, C-shapes and MC-shapes and shear values for angles, rectangular HSS and box members, and round HSS are not tabulated.

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

As indicated in the User Note of this section, virtually all W, S and HP shapes are not subject to shear buckling and are also eligible for the more liberal safety and resistance factors, ϕ_v = 1.00 (LRFD) and Ω_v = 1.50 (ASD). This is presented in **Example G.1** for a W-shape. A channel shear strength design is presented in **Example G.2**.

G3. TENSION FIELD ACTION

A built-up girder with a thin web and vertical stiffeners is presented in **Example G.8**.

G4. SINGLE ANGLES

Rolled angles are typically made from ASTM A36 steel. All single angles listed in the Manual have a C_v =1.0. A single angle example is illustrated in **Example G.3**.

G5. RECTANGULAR HSS AND BOX MEMBERS

The shear height, h, is taken as the clear distance between the radii. If the corner radii are unknown, the outside radius is taken as 1.5 times the design thickness. An HSS example is provide in **Example G.4**.

G6. ROUND HSS

For all Round HSS and Pipes of ordinary length listed in the Manual, F_{cr} can be taken as $0.6F_y$ in Specification Equation G6-1. A round HSS example is illustrated in **Example G.5**.

G7. WEAK AXIS SHEAR IN SINGLE AND DOUBLY SYMMETRIC SHAPES

For a weak axis shear example see **Example G.6** and **Example G.7**.

G8. BEAMS AND GIRDERS WITH WEB OPENINGS

For a beam and girder with web openings example see AISC Design Guide 2.

Example G.1a W-Shape in Strong-Axis Shear

Given:

Determine the available shear strength of a W24×62 ASTM A992 beam using the AISC Manual with end shears of 48 kips from dead load and 145 kips from live load.

Solution:

Material Properties:

W24×62 ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Geometric Properties:

W24×62 d = 23.7 in. t_w = 0.430 in. — Manual Table 1-1

Calculate the required shear strength

LRFD	ASD
V_u = 1.2(48.0 kips) + 1.6(145 kips) = 290 kips	V_a = 48.0 kips + 145 kips = 193 kips

Take the available shear strength from Manual Table 3-2

LRFD	ASD
$\phi_v V_n$ = 306 kips 306 kips > 290 kips **o.k.**	V_n / Ω_v = 204 kips 204 kips > 193 kips **o.k.**

Manual Table 3-2

Example G.1b W-Shape in Strong-Axis Shear

Given:

The available shear strength, which can be easily determined by the tabulated values of the Manual, can be verified by directly applying the provisions of the Specification. Determine the available shear strength for the W-shape and loading given in Example G.1a by applying the provisions of the Specification.

Solution:

Except for very few sections, which are listed in the User Note, Specification Section G2.1(a) is applicable to the I-shaped beams published in the Manual when $F_y \leq 50$ ksi.

$C_v = 1.0$ Eqn. G2-2

Calculate A_w

Section G2.1b

$A_w = dt_w = 23.7\text{ in.}\,(0.430\text{ in.}) = 10.2\text{ in.}^2$

Calculate V_n

$V_n = 0.6F_yA_wC_v = 0.6(50\text{ ksi})(10.2\text{ in.}^2)(1.0) = 306\text{ kips}$ Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
$\phi_v = 1.00$ $\phi_vV_n = 1.00(306\text{ kips}) = 306\text{ kips}$	$\Omega_v = 1.50$ $V_n / \Omega_v = 306\text{ kips} / 1.50 = 204\text{ kips}$

Section G2.1a

Example G.2a C-Shape in Strong-Axis Shear

Given:

Verify the shear strength of a C15×33.9 channel with end shears of 17.5 kips from dead load and 52.5 kips from live load.

Solution:

Material Properties:

C15×33.9 ASTM A36 F_y = 36 ksi F_u = 58 ksi

Manual Table 2-3

Geometric Properties:

C15×33.9 d = 15.0 in. t_w = 0.400 in.

Manual Table 1-5

Calculate the required strength

LRFD	ASD
V_u = 1.2(17.5 kips) + 1.6(52.5 kips) = 105 kips	V_a = 17.5 kips + 52.5 kips = 70.0 kips

Take the available shear strength from Manual Table 3-8

LRFD	ASD
$\phi_v V_n$ =117 kips 117 kips > 105 kips **o.k.**	V_n / Ω_v = 77.6 kips 77.6 kips > 70.0 kips **o.k.**

Manual Table 3-8

Example G.2b C-Shape in Strong-Axis Shear

Given:

The available shear strength, which can be easily determined by the tabulated values of the Manual, can be verified by directly applying the provisions of the Specification. Determine the available shear strength for the channel with the loading in Example G.2a.

Solution:

C_v is 1.0 for all rolled channels when $F_y \leq 36$ ksi, and Specification Equation G2-1 is applicable.

$C_v = 1.0$ Eqn. G2-2

Calculate A_w

Section G2.1b

$A_w = dt_w = 15.0 \text{ in.}(0.400 \text{ in.}) = 6.00 \text{ in.}^2$

Calculate V_n

$V_n = 0.6F_yA_wC_v = 0.6(36 \text{ ksi})(6.00 \text{ in.}^2)(1.0) = 130 \text{ kips}$ Eqn. G2-1

Calculate the available shear strength

The values of $\phi_v = 1.00$ (LRFD) and $\Omega_v = 1.50$ (ASD) do not apply to channels. The general values $\phi_v = 0.90$ (LRFD) and $\Omega_v = 1.67$ (ASD) must be used.

LRFD	ASD
$\phi_v V_n = 0.90(130 \text{ kips}) = 117 \text{ kips}$	$V_n / \Omega_v = 130 \text{ kips} / 1.67 = 77.8 \text{ kips}$

Example G.3 Angle in Shear

Given:

Determine the shear strength of a 5×3×¼ (LLV) ASTM A36 angle with end shears of 3.5 kips from dead load and 10.5 kips from live load.

Solution:

Material Properties:

L5×3×¼ ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-3

Geometric Properties:

L5×3×¼ b = 5.00 in. t = 0.250 in.

Calculate the required shear strength

LRFD	ASD
V_u = 1.2(3.50 kips) + 1.6(10.5 kips) = 21.0 kips	V_a = 3.50 kips + 10.5 kips = 14.0 kips

Note: There are no tables for angles in shear, but the available shear strength can be calculated as follows:

Specification Section G4 stipulates C_v = 1.0 and k_v = 1.2. — Sect G4

Calculate A_w

$A_w = bt$ = (5.00 in.)(0.250 in.) = 1.25 in.^{2} — Section G4

Calculate V_n

$V_n = 0.6F_yA_wC_v$ = 0.6(36 ksi)(1.25 in.^{2})(1.0) = 27.0 kips — Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
ϕ_v = 0.90 $\phi_v V_n$ = 0.90(27.0 kips) = 24.3 kips 24.3 kips > 21.0 kips **o.k.**	Ω_v = 1.67 V_n / Ω_v = 27.0 kips / 1.67 = 16.2 kips 16.2 kips > 14.0 kips **o.k.**

Section G1

Example G.4 Rectangular HSS in Shear

Given:

Determine the shear strength of a HSS6×4×⅜ ASTM A500 Grade B member with end shears of 11 kips from dead load and 33 kips from live load. The beam is oriented with the shear parallel to the 6 in. dimension.

Solution:

Material Properties:

HSS6×4×⅜ ASTM A500 Grade B F_y = 46 ksi F_u = 58 ksi

Manual Table 2-3

Geometric Properties:

HSS6×4×⅜ H = 6.00 in. B = 4.00 in. t = 0.349 in.

Manual Table 1-11

Calculate the required shear strength

LRFD	ASD
V_u = 1.2(11.0 kips) + 1.6(33.0 kips) = 66.0 kips	V_a = 11.0 kips + 33.0 kips = 44.0 kips

Note: There are no Manual Tables for shear in HSS shapes, but the available shear strength can be calculated as follows:

Calculate the nominal strength

For rectangular HSS in shear, use Section G2.1 with $A_w = 2ht$ and $k_v = 5$.

Section G5Section G5Eqn. G2-3

If the exact radius is unknown, h shall be taken as the corresponding outside dimension minus three times the thickness.

$h = H - (3t_w)$ = 6.00 in. – (3)(0.349 in.) = 4.95 in.

Eqn. G2-1

h/t = 4.95 in. / 0.349 in. = 14.2

$1.10\sqrt{k_v\,E/F_y} = 1.10\sqrt{5(29{,}000\text{ ksi}/46\text{ ksi})} = 61.8$

14.2 ≤ 61.8 Therefore C_v = 1.0

Note: Most standard HSS sections listed in the manual have C_v = 1.0 at $F_y \le$ 46 ksi.

Calculate A_w

$A_w = 2ht$ = 2(4.95 in.)(0.349 in.) = 3.46 in.^{2}

Calculate V_n

$V_n = 0.6F_yA_wC_v$ = 0.6(46 ksi)(3.46 in.^{2})(1.0) = 95.5 kips

Calculate the available shear strength

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(95.5 \text{ kips}) = 86.0 \text{ kips}$ 86.0 kips > 66.0 kips **o.k.**	$\Omega_v = 1.67$ $V_n / \Omega_v = 95.5 \text{ kips} / 1.67 = 57.2 \text{ kips}$ 57.2 kips > 44.0 kips **o.k.**

Section G1

Example G.5 Round HSS in Shear

Given:

Verify the shear strength of a round HSS16.000×0.375 ASTM A500 grade B member spanning 32 feet with end shears of 30 kips from dead load and 90 kips from live load.

Solution:

Material Properties:

HSS16.000×0.375 ASTM A500 Gr.B $F_y = 42$ ksi $F_u = 58$ ksi Manual Table 2-3

Geometric Properties:

HSS16.000×0.375 $D = 16.0$ in. $t = 0.349$ in. $A_g = 17.2$ in.^{2} Manual Table 1-13

Calculate the required shear strength

LRFD	ASD
$V_u = 1.2(30.0 \text{ kips}) + 1.6(90.0 \text{ kips})$ $= 180$ kips	$V_a = 30.0 \text{ kips} + 90.0 \text{ kips}$ $= 120$ kips

There are no Manual tables for round HSS in shear, but the available strength can be calculated as follows:

Calculate F_{cr} as the larger of: Section G6

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \qquad L_v = \text{half the span} = 192 \text{ in.}$$

Eqn. G6-2a

$$= \frac{1.60(29{,}000\text{ksi})}{\sqrt{\frac{192 \text{ in.}}{16.0 \text{ in.}}}\left(\frac{16.0 \text{ in.}}{0.349 \text{ in.}}\right)^{\frac{5}{4}}} = 112 \text{ ksi}$$

or

$$F_{cr} = \frac{0.78E}{\left(D/t\right)^{\frac{3}{2}}} = \frac{0.78(29{,}000\text{ksi})}{\left(\frac{16.0 \text{ in.}}{0.349 \text{ in.}}\right)^{\frac{3}{2}}} = 72.9 \text{ ksi}$$

Eqn. G6-2b

but, not to exceed

$F_{cr} = 0.6F_y = 0.6(42\text{ksi}) = 25.2$ ksi **controls**

Note: Equations G6-2a and G6-2b will not normally control for the sections published in the Manual except when high strength steel is used or the span is unusually long.

Calculate V_n Section G6

$$V_n = \frac{F_{cr}A_g}{2} = \frac{(25.2\text{ ksi})(17.2\text{ in.}^2)}{2} = 217\text{ kips}$$

Eqn. G6-1

Calculate the available shear strength

Section G1

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(217\text{ kips}) = 195\text{ kips}$ 195 kips > 180 kips **o.k.**	$\Omega_v = 1.67$ $V_n / \Omega_v = 217\text{ kips} / 1.67 = 130\text{ kips}$ 130 kips > 120 kips **o.k.**

Example G.6 Doubly-Symmetric Shape in Weak-Axis Shear

Given:

Verify the shear strength of a W21×48 ASTM A992 beam with end shears of 20 kips from dead load and 60 kips from live load in the weak direction.

Solution:

Material Properties:

W21×48 ASTM A992 F_y = 50 ksi F_u = 65 ksi

Manual Table 2-3

Geometric Properties:

W21×48 b_f = 8.14 in. t_f = 0.430 in.

Manual Table 1-1

Calculate the required shear strength

LRFD	ASD
V_u = 1.2(20.0 kips) + 1.6(60.0 kips) = 120 kips	V_a = 20.0 kips + 60.0 kips = 80.0 kips

For weak axis shear, use Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ for each flange and k_v = 1.2.

Section G7

Calculate A_w (multiply by 2 for both shear resisting elements)

$A_w = 2b_f t_f$ = 2(8.14 in.)(0.430 in.) = 7.00 in.^{2}

Calculate C_v

Section G2.1b

b_f/t_f = 8.14 in. / 0.430 in. = 18.9

$1.10\sqrt{k_v E/F_y} = 1.10\sqrt{1.2(29{,}000\text{ ksi}/50\text{ ksi})}$ = 29.0 > 18.9 therefore, C_v = 1.0

Eqn. G2-3

Note: For all ASTM A6 W, S, M, and HP shapes, when $F_y \leq$ 50 ksi, C_v = 1.0.

Calculate V_n

$V_n = 0.6F_yA_wC_v$ = 0.6(50 ksi)(7.00 in.^{2})(1.0) = 210 kips

Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
ϕ_v = 0.90 $\phi_v V_n$ = 0.90(210 kips) = 189 kips 189 kips > 120 kips **o.k.**	Ω_v = 1.67 V_n / Ω_v = 210 kips / 1.67 = 126 kips 126 kips > 80.0 kips **o.k.**

Section G1

Example G.7 Singly-Symmetric Shape in Weak-Axis Shear

Given:

Verify the shear strength of a C9×20 ASTM A36 channel with end shears of 5 kips from dead load and 15 kips from live load in the weak direction.

Solution:

Material Properties:

C9×20 ASTM A36 F_y = 36 ksi F_u = 58 ksi Manual Table 2-3

Geometric Properties:

C9×20 b_f = 2.65 in. t_f = 0.413 in. Manual Table 1-5

Calculate the required shear strength

LRFD	ASD
V_u = 1.2(5.00 kips) + 1.6(15.0 kips) = 30.0 kips	V_a = 5.00 kips + 15.0 kips = 20.0 kips

Note: There are no Manual tables for weak-axis shear in channel sections, but the available strength can be calculated as follows:

For weak axis shear, use Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ for each flange and k_v = 1.2. Section G7

Calculate A_w (multiply by 2 for both shear resisting elements)

$A_w = 2b_f t_f$ = 2(2.65 in.)(0.413in.) = 2.19 in.^{2} Section G7

Calculate C_v Section G2.1b

b_f / t_f = 2.65 in. / 0.413 in. = 6.42

$1.10\sqrt{k_v E/F_y} = 1.10\sqrt{1.2(29{,}000\text{ ksi}/36\text{ ksi})}$ = 34.2 > 6.42 Therefore, C_v = 1.0 Eqn. G2-3

Calculate V_n

$V_n = 0.6F_yA_wC_v$ = 0.6(36 ksi)(2.19 in.^{2})(1.0) = 47.3 kips Eqn. G2-1

Calculate the available shear strength Section G1

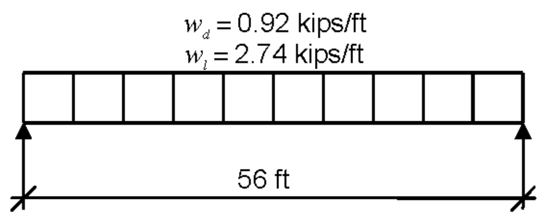
LRFD	ASD
ϕ_v = 0.90 $\phi_v V_n$ = 0.90(47.3 kips) = 42.6 kips 42.6 kips > 30.0 kips **o.k.**	Ω_v = 1.67 V_n / Ω_v = 47.3 kips / 1.67 = 28.3 kips 28.3 kips > 20.0 kips **o.k.**

Example G.8a Built-up Plate Girder with Transverse Stiffeners

Given:

A built up ASTM A36 I-shaped girder spanning 56 ft has a uniformly distributed dead load of 0.92 klf and a live load of 2.74 klf in the strong direction. The girder is 36 in. deep with 12 in. × 1½ in. flanges and a ⁵⁄₁₆ in. web. Determine if the member has sufficient available shear strength to support the end shear, without and with tension field action. Use transverse stiffeners, as required.

Note: This built-up girder was purposely selected with a thin web in order to illustrate the design of transverse stiffeners. A more conventionally proportioned plate girder would have at least a ½ in. web and slightly smaller flanges.



Beam Loading & Bracing Diagram
(Continuously braced)

Solution:

Material Properties:

Built-up girder ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-3

Geometric Properties:

Built-up girder t_w= 0.313 in. d = 36.0 in. $b_{ft} = b_{fc}$ = 12.0 in.
t_f= 1.50 in. h = 33.0 in.

Calculate the required shear strength at the support

LRFD	ASD
R_u =[1.2(0.92 klf) + 1.6(2.74 klf)](28.0 ft) = 154 kips	R_a = (0.92 klf + 2.74 klf)(28.0 ft) = 102 kips

Determine if stiffeners are required

$A_w = dt_w$ = (36.0 in.)(0.313 in.) = 11.3 in.^{2} — Section G2.1b

h/t_w = 33.0 in. / 0.313 in. = 105

105 < 260 Therefore, k_v = 5 — Section G2.1b

$$1.37\sqrt{k_v E / F_y} = 1.37\sqrt{5(29{,}000\text{ ksi})/(36\text{ ksi})} = 86.9$$

105 > 86.9 therefore, use Specification Eqn. G2-5 to calculate C_v

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} = \frac{1.51(29{,}000 \text{ ksi})(5)}{(105)^2(36 \text{ ksi})} = 0.552$$ Eqn. G2-5

Calculate V_n

$V_n = 0.6F_yA_wC_v = 0.6(36 \text{ ksi})(11.3 \text{ in.}^2)(0.552) = 135$ kips Eqn. G2-1

Check the available shear strength without stiffeners

LRFD	ASD
$\phi_v = 0.90$ $\phi_vV_n = 0.90(135 \text{ kips}) = 122$ kips 122 kips < 154 kips **not o.k.** **Therefore, stiffeners are required.**	$\Omega_v = 1.67$ $V_n / \Omega_v = 135 \text{ kips} / 1.67 = 80.8$ kips 80.8 kips < 102 kips **not o.k.** **Therefore, stiffeners are required.**

Section G1

Manual Tables 3-16a and 3-16b can be used to select stiffener spacings needed to develop the required stress in the web.

Limits on the Use of Tension Field Action: Section G3.1

Consideration of tension field action is not permitted for any of the following conditions:

a) end panels in all members with transverse stiffeners
b) members when a/h exceeds 3.0 or $[260/(h/t_w)]^2$
c) $2A_w/(A_{fc}+A_{ft}) > 2.5$
d) h/b_{fc} or $h/b_{ft} > 6.0$

Select stiffener spacing for end panel

Tension field action is not permitted for end panels, therefore use Table 3-16a.

LRFD	ASD
Use $V_u = \phi_vV_n$ to determine the required stress in the web by dividing by the web area $\frac{\phi_vV_n}{A_w} = \frac{V_u}{A_w} = \frac{154 \text{ kips}}{11.3 \text{ in.}^2} = 13.6$ ksi	*Use $V_a = V_n/\Omega_v$ to determine the required stress in the web by dividing by the web area* $\frac{V_n}{\Omega_vA_w} = \frac{V_a}{A_w} = \frac{102 \text{ kips}}{11.3 \text{ in.}^2} = 9.03$ ksi

Use Table 3-16a from the Manual to select the required stiffener ratio a/h based on the h/t ratio of the girder and the required stress. Interpolate and follow an available stress curve, $\frac{\phi_vV_n}{A_w}$ = 13.6 ksi for LRFD, $\frac{V_n}{\Omega_vA_w}$ = 9.03 ksi for ASD, until it intersects the horizontal line for a h/t_w value of 105. Project down from this intersection and take the maximum a/h value of 2.00 from the axis across the bottom. Since h = 33.0 in., stiffeners are required at (2.00)(33.0 in.) = 66.0 in. maximum. Therefore, use 60.0 in. spacing to be conservative.

Manual Table 3-16a

Select stiffener spacing for the second panel

Tension field action is allowed, but not required, since the second panel is not an end panel. Section G3.1

Calculate the required shear strength at the start of the second panel, 60 in. from end

LRFD	ASD
$V_u = 154 \text{ kips} - \left[1.2(0.920 \text{ klf}) + 1.6(2.74 \text{ klf})\right]\left(\dfrac{60.0 \text{ in.}}{12 \text{ in./ft}}\right)$ $= 127 \text{ kips}$	$V_a = 102 \text{ kips} - (0.920 \text{ klf} + 2.74 \text{ klf})\left(\dfrac{60.0 \text{ in.}}{12 \text{ in./ft}}\right)$ $= 83.7 \text{ kips}$

Check the available shear strength without stiffeners

Section G1

LRFD	ASD
$\phi_v = 0.90$ *From previous calculation* $\phi_v V_n = 0.90(135 \text{ kips}) = 122 \text{ kips}$ 122 kips < 127 kips **not o.k.** **Therefore additional stiffeners are required.** *Use* $V_u = \phi_v V_n$ *to determine the required stress in the web by dividing by the web area* $\dfrac{\phi_v V_n}{A_w} = \dfrac{V_u}{A_w} = \dfrac{127 \text{ kips}}{11.3 \text{ in.}^2} = 11.0 \text{ ksi}$	$\Omega_v = 1.67$ *From previous calculation* $V_n / \Omega_v = 135 \text{ kips} / 1.67 = 80.8 \text{ kips}$ 80.8 kips < 83.7 kips **not o.k.** **Therefore additional stiffeners are required.** *Use* $V_a = V_n/\Omega_v$ *to determine the required stress in the web by dividing by the web area* $\dfrac{V_n}{\Omega_v A_w} = \dfrac{V_a}{A_w} = \dfrac{83.7 \text{ kips}}{11.3 \text{ in.}^2} = 7.41 \text{ ksi}$

Use Table 3-16b from the Manual to select the required stiffener a/h ratio based on the h/t ratio of the girder and the required stress. Interpolate and follow an available stress curve, $\dfrac{\phi_v V_n}{A_w} = 11.2$ ksi for LRFD, $\dfrac{V_n}{\Omega_v A_w} = 7.41$ ksi for ASD, until it intersects the horizontal line for a h/t_w value of 105. Because the available stress does not intersect the h/t_w value of 105, the maximum value of 3.0 for a/h may be used. Since $h = 33.0$ in., an additional stiffener is required at (3.0)(33.0 in.) = 99.0 in. maximum from the previous one. Manual Table 3-16b

Select stiffener spacing for the third panel

Tension field action is allowed, but not required, since the next panel is not an end panel. Section G3.1

Calculate the required shear strength at the start of the third panel, 159 in. from end

LRFD	ASD
$V_u = 154 \text{ kips}$ $- [1.2(0.920 \text{ klf}) + 1.6(2.74 \text{ klf})]\left(\dfrac{159 \text{ in.}}{12 \text{ in./ft}}\right)$ $= 81.3 \text{ kips}$	$V_a = 102 \text{ kips}$ $- (0.920 \text{ klf} + 2.74 \text{ klf})\left(\dfrac{159 \text{ in.}}{12 \text{ in./ft}}\right)$ $= 53.5 \text{ kips}$

Check the available shear strength without stiffeners

LRFD	ASD
$\phi_v = 0.90$ *From previous calculation* $\phi_v V_n = 0.90(135 \text{ kips}) = 122 \text{ kips}$ 122 kips > 81.3 kips **o.k.** **Therefore additional stiffeners are not required.**	$\Omega_v = 1.67$ *From previous calculation* $V_n/\Omega_v = 135 \text{ kips} / 1.67 = 80.8 \text{ kips}$ 80.8 kips > 53.5 kips **o.k.** **Therefore additional stiffeners are not required.**

Section G1

The four Available Shear Stress tables, Manual Tables 3-16a, 3-16b, 3-17a and 3-17b, are useful because they permit a direct solution for the required stiffener spacing. Alternatively, you can select a stiffener spacing and check the resulting strength, although this process is likely to be iterative. In Example G.8b below, the stiffener spacings that were selected from the charts in the example above are used.

Example G.8b Built-up Plate Girder with Transverse Stiffeners

The stiffener spacings from Example G.8a, which were easily determined from the tabulated values of the *Steel Construction Manual*, are verified below by directly applying the provisions of the Specification.

Verify the shear strength of end panel — Section G2.1b

a/h = 60 in./33 in. = 1.82

$$k_v = 5 + \frac{5}{(a/h)^2} = 5 + \frac{5}{(1.82)^2} = 6.51$$

h/t_w = 33.0 in. / 0.313 in. = 105

Check a/h limits

a/h = 1.82 ≤ 3.0

$$a/h = 1.82 < \left[\frac{260}{(h/t_w)}\right]^2 = \left[\frac{260}{105}\right]^2 = 6.13$$

Therefore, use k_v = 6.51.

Tension field action is not allowed since the panel is an end panel.

Since $h/t_w > 1.37\sqrt{k_v E/F_y} = 1.37\sqrt{(6.51)(29{,}000\,\text{ksi})/(36\,\text{ksi})} = 99.2$, — Eqn. G2-5

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} = \frac{1.51(29{,}000\,\text{ksi})(6.51)}{(105)^2 36\,\text{ksi}} = 0.718$$ — Eqn. G2-1

$V_n = 0.6F_yA_wC_v = 0.6(36\text{ ksi})(11.3\text{ in.}^2)(0.718) = 175\text{ kips}$

Calculate the available shear strength — Section G1

LRFD	ASD
ϕ_v = 0.90 $\phi_v V_n$ = 0.90(175 kips) =158 kips 158 kips > 154 kips **o.k.**	Ω_v = 1.67 V_n/Ω_v = (175 kips) / 1.67 = 105 kips 105 kips > 102 kips **o.k.**

Verify the shear strength of the second panel

a/h for the second panel was 3.0

$$k_v = 5 + \frac{5}{(a/h)^2} = 5 + \frac{5}{(3.0)^2} = 5.56$$

Check a/h limits — Section G2.1b

$$a/h = \frac{99\text{ in.}}{33\text{ in.}} = 3.00 \le 3.0$$

$$a/h = 3.00 < \left[\frac{260}{(h/t_w)}\right]^2 = \left[\frac{260}{105}\right]^2 = 6.13$$

Therefore, use k_v = 5.56

Since $h/t_w > 1.37\sqrt{k_v E/F_y} = 1.37\sqrt{(5.56)(29{,}000\text{ ksi})/(36\text{ ksi})} = 91.7$,

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} = \frac{1.51(29{,}000\text{ ksi})(5.56)}{(105)^2(36\text{ ksi})} = 0.613$$ Eqn. G2-5

Check the additional limits for the use of tension field action: Section G3.1

Note the limits of $a/h \le 3.0$ and $a/h < [260/(h/t_w)]^2$ have already been calculated.

$$2A_w/(A_{fc} + A_{ft}) = 2(11.3\text{ in.}^2)/[2(12.0\text{ in.})(1.50\text{ in.})] = 0.628 < 2.5$$

$$h/b_{fc} = h/b_{ft} = 33\text{ in.}/12\text{ in.} = 2.75 < 6.0$$

Tension field action is permitted because the panel under consideration is not an end panel and the other limits indicate in Section G3.1 have been met.

Section G3.2

Since $h/t_w > 1.10\sqrt{k_v E/F_y} = 1.10\sqrt{5.56(29{,}000\text{ksi})/(36\text{ksi})}$ = 73.6, use Eqn. G3-2

$$V_n = 0.6F_y A_w\left[C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}\right]$$ Eqn. G3-2

$$= 0.6(36\text{ ksi})(11.3\text{ in.}^2)\left[0.613 + \frac{1-0.613}{1.15\sqrt{1+(3.00)^2}}\right]$$

V_n = 176 kips

Calculate the available shear strength

LRFD	ASD
ϕ_v= 0.90 $\phi_v V_n$ = 0.90(176 kips) =158 kips 158 kips > 127 kips **o.k.**	Ω_v = 1.67 V_n/Ω_v = (176 kips) / 1.67 = 105 kips 105 kips > 83.7 kips **o.k.**

Section G1

CHAPTER H
DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

For all interaction equations in Specification Section H, the required forces and moments must include second-order effects, as required by Chapter C of the Specification. ASD users are accustomed to using an interaction equation that includes a partial second-order amplification. This is now included in the analysis and need not be included in these interaction equations.

Example H.1a W-shape Subject to Combined Compression and Bending About Both Axes (braced frame)

Given:

Using Manual Table 6-1, determine if an ASTM A992 W14×99 has sufficient available strength to support the axial forces and moments listed below, obtained from a second order analysis that includes P-δ effects. The unbraced length is 14 ft and the member has pinned ends. $KL_x = KL_y = L_b = 14.0$ ft.

LRFD	ASD
$P_u = 400$ kips $M_{ux} = 250$ kip-ft $M_{uy} = 80.0$ kip-ft	$P_a = 267$ kips $M_{ax} = 167$ kip-ft $M_{ay} = 53.3$ kip-ft

Solution:

Material Properties:

W14×99 ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi Manual Table 2-3

Try a W14×99

Take combined strength parameters from Manual Table 6-1 Manual Table 6-1

LRFD	ASD
$p = \dfrac{0.886}{10^3 \text{ kips}}$ at 14.0 ft $b_x = \dfrac{1.38}{10^3 \text{ kip-ft}}$ at 14.0 ft $b_y = \dfrac{2.85}{10^3 \text{ kip-ft}}$ *Check limit for Equation H1-1a* $\dfrac{P_u}{\phi_c P_n} = pP_u = \left(\dfrac{0.886}{10^3 \text{ kips}}\right)(400 \text{ kips}) = 0.354$ Since $\dfrac{P_u}{\phi_c P_n} > 0.2$, $pP_u + b_xM_{ux} + b_yM_{uy} \le 1.0$ $0.354 + \left(\dfrac{1.38}{10^3 \text{ kip-ft}}\right)(250 \text{ kip-ft}) + \left(\dfrac{2.85}{10^3 \text{ kip-ft}}\right)(80.0 \text{ kip-ft}) \le 1.0$ $= 0.354 + 0.345 + 0.228 = 0.927 \le 1.0$ **o.k.**	$p = \dfrac{1.33}{10^3 \text{ kips}}$ at 14.0 ft $b_x = \dfrac{2.08}{10^3 \text{ kip-ft}}$ at 14.0 ft $b_y = \dfrac{4.29}{10^3 \text{ kip-ft}}$ *Check limit for Equation H1-1a* $\dfrac{P_a}{P_n / \Omega_c} = pP_a = \left(\dfrac{1.33}{10^3 \text{ kips}}\right)(267 \text{ kips}) = 0.355$ Since $\dfrac{P_a}{P_n / \Omega_c} > 0.2$, $pP_a + b_xM_{ax} + b_yM_{ay} \le 1.0$ $0.355 + \left(\dfrac{2.08}{10^3 \text{ kip-ft}}\right)(167 \text{ kip-ft}) + \left(\dfrac{4.29}{10^3 \text{ kip-ft}}\right)(53.3 \text{ kip-ft}) \le 1.0$ $= 0.355 + 0.347 + 0.229 = 0.931 \le 1.0$ **o.k.**

Manual Table 6-1 simplifies the calculation of Specification Equations H1-1a and H1-1b. A direct application of these equations is shown in **Example H.2**.

Example H.1b W-shape Column Subject to Combined Compression and Bending Moment About Both Axes (braced frame)

Given:

Using Manual tables to determine the available compressive and flexural strength, determine if an ASTM A992 W14×99 has sufficient available strength to support the axial forces and moments listed below, obtained from a second order analysis that includes P-δ effects. The unbraced length is 14 ft and the member has pinned ends. $KL_x = KL_y = L_b = 14.0$ ft.

LRFD	ASD
$P_u = 400$ kips $M_{ux} = 250$ kip-ft $M_{uy} = 80.0$ kip-ft	$P_a = 267$ kips $M_{ax} = 167$ kip-ft $M_{ay} = 53.3$ kip-ft

Solution:

Material Properties:

W14×99 ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi

Manual Table 2-3

Take the available axial and flexural strengths from the Manual Tables

LRFD	ASD
at $KL_y = 14.0$ ft, $P_c = \phi_c P_n = 1130$ kips	at $KL_y = 14.0$ ft, $P_c = P_n/\Omega_c = 751$ kips
at $L_b = 14.0$ ft, $M_{cx} = \phi M_{nx} = 642$ kip-ft	at $L_b = 14.0$ ft, $M_{cx} = M_{nx}/\Omega = 428$ kip-ft
$M_{cy} = \phi M_{ny} = 311$ kip-ft	$M_{cy} = M_{ny}/\Omega = 207$ kip-ft
$\dfrac{P_u}{\phi_c P_n} = \dfrac{400 \text{ kips}}{1{,}130 \text{ kips}} = 0.354$	$\dfrac{P_a}{P_n/\Omega_c} = \dfrac{267 \text{ kips}}{751 \text{ kips}} = 0.356$
Since $\dfrac{P_u}{\phi_c P_n} > 0.2$, use Eqn. H1-1a	Since $\dfrac{P_a}{P_n/\Omega_c} > 0.2$, use Eqn. H1-1a
$\dfrac{P_r}{P_c} + \dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \le 1.0$	$\dfrac{P_r}{P_c} + \dfrac{8}{9}\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \le 1.0$
$\dfrac{400 \text{ kips}}{1130 \text{ kips}} + \dfrac{8}{9}\left(\dfrac{250 \text{ kip-ft}}{642 \text{ kip-ft}} + \dfrac{80.0 \text{ kip-ft}}{311 \text{ kip-ft}}\right)$ $= 0.354 + \dfrac{8}{9}(0.389 + 0.257) = 0.928 < 1.0$ **o.k.**	$\dfrac{267 \text{ kips}}{751 \text{ kips}} + \dfrac{8}{9}\left(\dfrac{167 \text{ kip-ft}}{428 \text{ kip-ft}} + \dfrac{53.3 \text{ kip-ft}}{207 \text{ kip-ft}}\right)$ $= 0.356 + \dfrac{8}{9}(0.390 + 0.258) = 0.932 < 1.0$ **o.k.**

Manual Table 4-1

Manual Table 3-10

Manual Table 3-4

Eq. H1-1a

Example H.2 W-Shape Column Subject to Combined Compression and Bending Moment About Both Axes (by Specification Section H2)

Given:

Determine if an ASTM A992 W14×99 shown in **Example H.1** has sufficient available strength, using Specification Section H2. This example is included primarily to illustrate the use of Specification Section H2. $KL_x = KL_y = L_b = 14.0$ ft

LRFD	ASD
$P_u = 360$ kips $M_{ux} = 250$ kip-ft $M_{uy} = 80.0$ kip-ft	$P_a = 240$ kips $M_{ax} = 167$ kip-ft $M_{ay} = 53.3$ kip-ft

Solution:

Material Properties:

W14×99 ASTM A992 $F_y = 50$ ksi $F_u = 65$ ksi

Manual Table 2-3

Geometric Properties:

W14×99 $A = 29.1$ in.^{2} $S_x = 157$ in.^{3} $S_y = 55.2$ in.^{3}

Manual Table 1-1

Calculate the required flexural and axial stresses

LRFD	ASD
$f_a = \dfrac{P_u}{A} = \dfrac{360\text{ kips}}{29.1\text{ in.}^2} = 12.4\text{ ksi}$	$f_a = \dfrac{P_a}{A} = \dfrac{240\text{ kips}}{29.1\text{in.}^2} = 8.25\text{ ksi}$
$f_{bx} = \dfrac{M_{ux}}{S_x} = \dfrac{250\text{ kip-ft}}{157\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 19.1\text{ ksi}$	$f_{bx} = \dfrac{M_{ax}}{S_x} = \dfrac{167\text{ kip-ft}}{157\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 12.8\text{ ksi}$
$f_{by} = \dfrac{M_{uy}}{S_y} = \dfrac{80.0\text{ kip-ft}}{55.2\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 17.4\text{ ksi}$	$f_{by} = \dfrac{M_{ay}}{S_y} = \dfrac{53.3\text{ kip-ft}}{55.2\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 11.6\text{ ksi}$

Calculate the available flexural and axial stresses from the available strengths in ***Example H.1b***

LRFD	ASD
$F_a = \phi_c F_{cr} = \dfrac{\phi_c P_n}{A} = \dfrac{1{,}130\text{ kips}}{29.1\text{ in.}^2} = 38.8\text{ ksi}$	$F_a = \dfrac{F_{cr}}{\Omega_c} = \dfrac{P_n}{\Omega_c A} = \dfrac{751\text{ kips}}{29.1\text{ in.}^2} = 25.8\text{ ksi}$
$F_{bx} = \dfrac{\phi_b M_{nx}}{S_x} = \dfrac{642\text{ kip-ft}}{157\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 49.1\text{ ksi}$	$F_{bx} = \dfrac{M_{nx}}{\Omega_b S_x} = \dfrac{428\text{ kip-ft}}{157\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 32.7\text{ ksi}$
$F_{by} = \dfrac{\phi_b M_{ny}}{S_y} = \dfrac{311\text{ kip-ft}}{55.2\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 67.6\text{ ksi}$	$F_{by} = \dfrac{M_{ny}}{\Omega_b S_y} = \dfrac{207\text{ kip-ft}}{55.2\text{ in.}^3}\left(\dfrac{12\text{ in.}}{\text{ft}}\right) = 45.0\text{ ksi}$

As shown in the LRFD calculation of F_{by} above, the available flexural stresses can exceed the yield stress in cases where the available strength is governed by yielding and the yielding strength is calculated using the plastic section modulus.

Calculate the combined stress ratio

LRFD	ASD
$\left\lvert \dfrac{f_a}{F_a} + \dfrac{f_{bx}}{F_{bx}} + \dfrac{f_{by}}{F_{by}} \right\rvert \le 1.0$	$\left\lvert \dfrac{f_a}{F_a} + \dfrac{f_{bx}}{F_{bx}} + \dfrac{f_{by}}{F_{by}} \right\rvert \le 1.0$
$\left\lvert \dfrac{12.4 \text{ ksi}}{38.8 \text{ ksi}} + \dfrac{19.1 \text{ ksi}}{49.1 \text{ ksi}} + \dfrac{17.4 \text{ ksi}}{67.6 \text{ ksi}} \right\rvert = 0.966 \le 1.0$ **o.k.**	$\left\lvert \dfrac{8.25 \text{ ksi}}{25.8 \text{ ksi}} + \dfrac{12.8 \text{ ksi}}{32.7 \text{ ksi}} + \dfrac{11.6 \text{ ksi}}{45.0 \text{ ksi}} \right\rvert = 0.969 \le 1.0$ **o.k.**

Eqn. H2-1

A comparison of these results with those from Example H.1 shows that Equation H1-1a will produce less conservative results than Equation H2-1 when its use is permitted.

Note: this check is made at a point. The designer must therefore select which point along the length is critical, or check multiple points if the critical point can not be readily determined.

Example H.3 W-Shape Subject to Combined Axial Tension and Flexure

Given:

Select an ASTM A992 W-shape with a 14-in. nominal depth to carry nominal forces of 29 kips from dead load and 87 kips from live load in axial tension, as well as the following nominal moments due to uniformly distributed loads:

M_{xD} = 32.0 kip-ft M_{xL} = 96.0 kip-ft
M_{yD} = 11.3 kip-ft M_{yL} = 33.8 kip-ft

The unbraced length is 30 ft and the ends are pinned. Assume the connections are made with no holes.

Solution:

Material Properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Calculate the required strength

LRFD	ASD
P_u = 1.2(29.0 kips) + 1.6(87.0 kips) = 174 kips M_{ux} = 1.2(32.0 kip-ft) + 1.6(96.0 kip-ft) = 192 kip-ft M_{uy} = 1.2(11.3 kip-ft) + 1.6(33.8 kip-ft) = 67.6 kip-ft	P_a = 29.0 kips + 87.0 kips = 116 kips M_{ay} = 32.0 kip-ft + 96 kip-ft = 128 kip-ft M_{ay} = 11.3 kip-ft + 33.8 kip-ft = 45.1 kip-ft

Try a W14×82

Geometric Properties:

W14×82 A = 24.0 in.^{2} S_x = 123 in.^{3} Z_x = 139 in.^{3} S_y = 29.3 in.^{3} — Manual Table 1-1
Z_y = 44.8 in.^{3} I_y = 148 in.^{4}
L_p = 8.76 ft L_r = 33.1 ft — Table 3-2

Calculate the nominal gross tensile strength

$P_n = F_yA_g$ = (50 ksi)(24.0 in.^{2}) = 1,200 kips — Eqn. D2-1

Note that for a member with holes, the rupture strength of the member would also have to be computed using Specification Equation D2-2.

Calculate the nominal flexural strength for bending about the x-x axis

Yielding limit state

$M_{nx} = M_p = F_yZ_x$ = 50 ksi(139 in.^{3}) = 6,950 kip-in = 579 kip-ft — Eqn. F2-1

Lateral-torsional buckling limit state

L_b = 30.0 ft

Since $L_p < L_b \le L_r$, Equation F2-2 applies.

Calculate the lateral-torsional buckling modification factor, C_b

From Manual Table 3-1, C_b = 1.14, without considering the beneficial effects of the tension force. However, C_b may be increased because the column is in axial tension. Section H1.2

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2} = \frac{\pi^2 (29{,}000 \text{ ksi})(148 \text{ in.}^4)}{((30.0 \text{ ft})(12.0 \text{ in./ft}))^2} = 327 \text{ kips}$$

LRFD	ASD
$\sqrt{1+\frac{P_u}{P_{ey}}} = \sqrt{1+\frac{174\text{ kips}}{327\text{ kips}}} = 1.24$	$\sqrt{1+\frac{1.5P_a}{P_{ey}}} = \sqrt{1+\frac{1.5(116\text{ kips})}{327\text{ kips}}} = 1.24$

$$C_b = 1.24(1.14) = 1.41$$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$ Eqn. F2-2

$$M_n = 1.41\left[6{,}950 \text{ kip-in.} - (6{,}950 \text{ kip-in.} - 0.7(50 \text{ ksi})(123 \text{ in.}^3))\left(\frac{30.0\text{ ft} - 8.76\text{ ft}}{33.1\text{ ft} - 8.76\text{ ft}}\right)\right]$$

= 6,550 kip-in. < M_p therefore use:

M_n = 6,550 kip-in. or 546 kip-ft **controls**

Local buckling limit state

Per Manual Table 1-1, the cross section is compact at F_y = 50 ksi; therefore, the local buckling limit state does not apply.

Calculate the nominal flexural strength for bending about the y-y axis and the intersection of flexure and tension

Yielding limit state

Since W14×82 has compact flanges, only the limit state of yielding applies. Eqn. F6-1

$M_{ny} = M_p = F_yZ_y \le 1.6F_yS_y$

= 50 ksi(44.8 in.^{3}) ≤ 1.6(50 ksi)(29.3 in.^{3})

= 2,240 kip-in. < 2,340 kip-in., therefore use:

M_{ny} = 2,240 kip-in. or 187 kip-ft

LRFD	ASD
$\phi_b = \phi_t = 0.90$ $P_c = \phi_t P_n = 0.90(1200\text{ kips}) = 1{,}080\text{ kips}$ $M_{cx} = \phi_b M_{nx} = 0.90(546\text{ kip-ft}) = 491\text{ kip-ft}$ $M_{cy} = \phi_b M_{ny} = 0.90(187\text{ kip-ft}) = 168\text{ kip-ft}$	$\Omega_b = \Omega_t = 1.67$ $P_c = P_n/\Omega_t = \dfrac{1{,}200\text{ kips}}{1.67} = 719\text{ kips}$ $M_{cx} = M_{nx}/\Omega_b = \dfrac{546\text{ kip-ft}}{1.67} = 327\text{ kip-ft}$ $M_{cy} = M_{ny}/\Omega_b = \dfrac{187\text{ kip-ft}}{1.67} = 112\text{ kip-ft}$

Sections D2 and F1

Check limit for Equation H1-1a

LRFD	ASD
$\dfrac{P_r}{\phi_t P_n} = \dfrac{P_u}{\phi_t P_n} = \dfrac{174\text{ kips}}{1{,}080\text{ kips}} = 0.161 < 0.2$	$\dfrac{P_r}{P_n/\Omega_t} = \dfrac{P_a}{P_n/\Omega_t} = \dfrac{116\text{ kips}}{719\text{ kips}} = 0.161 < 0.2$

Therefore, Equation H1-1b applies

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$$

Eqn. H1-1b

LRFD	ASD
$\dfrac{174\text{ kips}}{2(1{,}080\text{ kips})} + \left(\dfrac{192\text{ kip-ft}}{491\text{ kip-ft}} + \dfrac{67.6\text{ kip-ft}}{168\text{ kip-ft}}\right) \leq 1.0$ $0.874 < 1.0$ **o.k.**	$\dfrac{116\text{ kips}}{2(719\text{ kips})} + \left(\dfrac{128\text{ kip-ft}}{326\text{ kip-ft}} + \dfrac{45.1\text{ kip-ft}}{112\text{ kip-ft}}\right) \leq 1.0$ $0.876 < 1.0$ **o.k.**

Example H.4 W-Shape Subject to Combined Axial Compression and Flexure

Given:

Select an ASTM A992 W-shape with a 10 in. nominal depth to carry nominal axial compression forces of 5 kips from dead load and 15 kips from live load. The unbraced length is 14 ft and the ends are pinned. The member also has the following nominal required moment strengths due to uniformly distributed loads, not including second-order effects:

M_{xD} = 15 kip-ft M_{xL} = 45 kip-ft
M_{yD} = 2 kip-ft M_{yL} = 6 kip-ft

The member is not subject to sidesway.

Solution:

Material Properties:

ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Calculate the required strength, not considering second-order effects

LRFD	ASD
P_u = 1.2(5.00 kips) + 1.6(15.0 kips) = 30.0 kips M_{ux} = 1.2(15.0 kip-ft) + 1.6(45.0 kip-ft) = 90.0 kip-ft M_{uy} = 1.2(2.00 kip-ft) + 1.6(6.00 kip-ft) = 12.0 kip-ft	P_a = 5.00 kips + 15.0 kips = 20.0 kips M_{ax} = 15.0 kip-ft + 45.0 kip-ft = 60.0 kip-ft M_{ay} = 2.00 kip-ft + 6.00 kip-ft = 8.00 kip-ft

Try a W10×33

Geometric Properties:

W10×33 A = 9.71 in.^{2} S_x= 35.0 in.^{3} Z_x = 38.8 in.^{3} I_x = 171 in.^{4} r_x = 4.19 in.
S_y= 9.20 in.^{3} Z_y = 14.0 in.^{3} I_y = 36.6 in.^{4} r_y = 1.94 in.
L_p = 6.85 ft L_r = 21.8 ft

Manual Table 1-1
Table 3-2

Calculate the available axial strength

For a pinned-pinned condition, K = 1.0. — Commentary Table C-C2.2

Since $KL_x = KL_y$ = 14.0 ft and $r_x > r_y$, the *y-y* axis will govern.

LRFD	ASD
$P_c = \phi_c P_n$ = 253 kips	$P_c = P_n / \Omega_c$ = 168 kips

Manual Table 4-1

Calculate the required flexural strengths including second order amplification

Use the "Amplified First-Order Elastic Analysis" procedure from Section C2.1b. Since the member is not subject to sidesway, only P-δ amplifiers need to be added.

$$B_1 = \frac{C_m}{1-\alpha P_r / P_{e1}}$$ Eqn. C2-2

$C_m = 1.0$

X-X axis flexural magnifier

$$P_{e1} = \frac{\pi^2 EI_x}{(K_1 L_x)^2} = \frac{\pi^2 (29{,}000\text{ ksi})(171\text{ in.}^4)}{((1.0)(14.0\text{ ft})(12\text{ in./ft}))^2} = 1730\text{ kips}$$ Eqn. C2-5

LRFD	ASD
$\alpha = 1.0$ $B_1 = \dfrac{1.0}{1-1.0(30.0\text{ kips}/1{,}730\text{ kips})} = 1.02$ $M_{ux} = 1.02(90.0\text{ kip-ft}) = 91.8\text{ kip-ft}$	$\alpha = 1.6$ $B_1 = \dfrac{1.0}{1-1.6(20.0\text{ kips}/1{,}730\text{ kips})} = 1.02$ $M_{ax} = 1.02(60.0\text{ kip-ft}) = 61.2\text{ kip-ft}$

Eqn. C2-2

Y-Y axis flexural magnifier

$$P_{e1} = \frac{\pi^2 EI_y}{(K_1 L_y)^2} = \frac{\pi^2 (29{,}000\text{ ksi})(36.6\text{ in.}^4)}{((1.0)(14.0\text{ ft})(12\text{ in./ft}))^2} = 371\text{ kips}$$ Eqn. C2-5

LRFD	ASD
$\alpha = 1.0$ $B_1 = \dfrac{1.0}{1-1.0(30.0\text{ kips}/371\text{ kips})} = 1.09$ $M_{uy} = 1.09(12.0\text{ kip-ft}) = 13.1\text{ kip-ft}$	$\alpha = 1.6$ $B_1 = \dfrac{1.0}{1-1.6(20.0\text{ kips}/371\text{ kips})} = 1.09$ $M_{ay} = 1.09\ (8.00\text{ kip-ft}) = 8.72\text{ kip-ft}$

Eqn. C2-2

Calculate the nominal bending strength about the x-x axis

Yielding limit state

$M_{nx} = M_p = F_y Z_x = 50\text{ ksi}(38.8\text{ in.}^3) = 1940\text{ kip-in or }162\text{ kip-ft}$ Eqn. F2-1

Lateral-torsional buckling limit state

Since $L_p < L_b < L_r$, Equation F2-2 applies

From Manual Table 3-1, $C_b = 1.14$ Manual Table 3-1

$$M_{nx} = C_b\left[M_p - \left(M_p - 0.7F_yS_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$$ Eqn. F2-2

$$M_{nx} = 1.14\left[1{,}940\text{ kip-in.} - \left(1{,}940\text{ kip-in.} - 0.7\left(50\text{ ksi}\right)\left(35.0\text{ in.}^3\right)\right)\left(\frac{14.0\text{ ft} - 6.85\text{ ft}}{21.8\text{ ft} - 6.85\text{ ft}}\right)\right]$$

$= 1{,}820$ kip-in. $\le 1{,}940$ kip-in., therefore use:

$M_{nx} = 1{,}820$ kip-in. or 152 kip-ft **controls**

Local buckling limit state — Manual Table 1-1

Per Manual Table 1-1, the member is compact for $F_y = 50$ ksi, so the local buckling limit state does not apply

Calculate the nominal bending strength about the y-y axis — Section F6.2

Since a W10×33 has compact flanges, only the yielding limit state applies.

$M_{ny} = M_p = F_yZ_y \le 1.6F_yS_y$ — Eqn. F6-1

$= 50\text{ ksi}(14.0\text{ in.}^3) \le 1.6(50\text{ ksi})(9.20\text{ in.}^3)$

$= 700$ kip-in < 736 kip-in., therefore

Use $M_{ny} = 700$ kip-in. or 58.3 kip-ft

LRFD	ASD
$\phi_b = 0.90$ $M_{cx} = \phi_b M_{nx} = 0.90(152\text{ kip-ft}) = 137\text{ kip-ft}$ $M_{cy} = \phi_b M_{ny} = 0.90(58.3\text{ kip-ft}) = 52.5\text{ kip-ft}$	$\Omega_b = 1.67$ $M_{cx} = M_{nx}/\Omega_b = 152\text{ kip-ft}/1.67 = 91.0\text{ kip-ft}$ $M_{cy} = M_{ny}/\Omega_b = 58.3\text{ kip-ft}/1.67 = 34.9\text{ kip-ft}$

Section F1

Check limit for Equation H1-1a

LRFD	ASD
$\frac{P_r}{P_c} = \frac{P_u}{\phi_c P_n} = \frac{30.0\text{ kips}}{253\text{ kips}} = 0.119$, therefore, use Specification Equation H1-1b $\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0$ $\frac{30.0\text{ kips}}{2(253\text{ kips})} + \left(\frac{91.8\text{ kip-ft}}{137\text{ kip-ft}} + \frac{13.1\text{ kip-ft}}{52.5\text{ kip-ft}}\right)$ $0.0593 + 0.920 = 0.979 \le 1.0$ **o.k.**	$\frac{P_r}{P_c} = \frac{P_a}{P_n/\Omega_c} = \frac{20.0\text{ kips}}{168\text{ kips}} = 0.119$, therefore, use Specification Equation H1-1b $\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0$ $\frac{20.0\text{ kips}}{2(168\text{ kips})} + \left(\frac{61.2\text{ kip-ft}}{91.0\text{ kip-ft}} + \frac{8.72\text{ kip-ft}}{34.9\text{ kip-ft}}\right)$ $0.0595 + 0.922 = 0.982 \le 1.0$ **o.k.**

Section H1.1

Eqn. H1-1b

Example H.5a Rectangular HSS Torsional Strength

Given:

Determine the available torsional strength of an ASTM A500 Gr B HSS6×4×¼.

Solution:

Material Properties:

HSS6×4×¼	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi	Manual Table 2-3

Geometric properties:

HSS6×4×¼	$\dfrac{h}{t} = 22.8$	$\dfrac{b}{t} = 14.2$	$t = 0.233$ in.	Manual Table 1-11

Evaluate wall slenderness to determine the appropriate critical stress equation

$\dfrac{h}{t} > \dfrac{b}{t}$, therefore, $\dfrac{h}{t}$ governs.

$$\frac{h}{t} \le 2.45\sqrt{\frac{E}{F_y}}$$

$$22.8 \le 2.45\sqrt{\frac{29{,}000 \text{ ksi}}{46 \text{ ksi}}} = 61.5 \text{ therefore, use Eqn. H3-3}$$

$F_{cr} = 0.6\,F_y = 0.6(46 \text{ ksi}) = 27.6$ ksi

Eqn. H3-3

Calculate the nominal torsional strength

$C = 10.1$ in.^{3}

Manual Table 1-11

$T_n = F_{cr}C = 27.6 \text{ ksi}\,(10.1 \text{ in.}^3) = 279$ kip-in.

Eqn. H3-1

Calculate the available torsional strength

LRFD	ASD
$\phi_T = 0.90$ $\phi_T T_n = 0.90(279 \text{ kip-in.}) = 251$ kip-in.	$\Omega_T = 1.67$ $T_n / \Omega_T = 279 \text{ kip-in.}/1.67 = 167$ kip-in.

Section H3.1

Note: For more complete guidance on designing for torsion, see AISC Design Guide 9, *Torsional Analysis of Structural Steel Members*.

Example H.5b Round HSS Torsional Strength

Given:

Determine the available torsional strength of an ASTM A500 Gr. B HSS5.000×0.250 that is 14 ft long.

Solution:

Material Properties:

HSS5.000×0.250 ASTM A500 Gr. B F_y = 42 ksi F_u = 58 ksi — Manual Table 2-3

Geometric properties:

HSS5.000×0.250 $\frac{D}{t} = 21.5$ t = 0.233 in. D = 5.00 in. — Manual Table 1-13

Calculate the critical stress as the larger of

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} = \frac{1.23(29{,}000 \text{ ksi})}{\sqrt{\frac{14.0 \text{ ft (12 in./ft)}}{5.00 \text{ in.}}}(21.5)^{\frac{5}{4}}} = 133 \text{ ksi}$$

Eqn. H3-2a

and

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} = \frac{0.60(29{,}000 \text{ ksi})}{(21.5)^{\frac{3}{2}}} = 175 \text{ ksi}$$

Eqn. H3-2b

However, F_{cr} shall not exceed 0.60 F_y = 0.60(42 ksi) = 25.2 ksi, therefore, — Section H3.1

F_{cr} = 25.2 ksi

Manual Table 1-13

Eqn. H3-1

Calculate the nominal torsional strength

C = 7.95 in.^{3}

$T_n = F_{cr}C$ = 25.2 ksi (7.95 in.^{3}) = 200 kip-in.

Calculate the available torsional strength

LRFD	ASD
ϕ_T = 0.90 $\phi_T T_n$ = 0.90(200 kip-in.) = 180 kip-in.	Ω_T = 1.67 T_n / Ω_T = 200 kip-in./1.67 = 120 kip-in.

Section H3.1

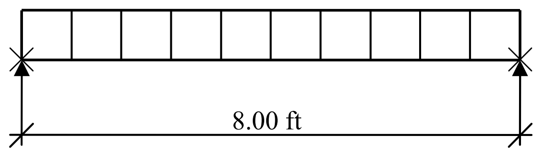
Note: For more complete guidance on designing for torsion, see AISC Design Guide 9, *Torsional Analysis of Structural Steel Members*.

Example H.5c HSS Combined Torsional and Flexural Strength

Given:

Verify the strength of an ASTM A500 Gr. B HSS6×4×¼ loaded as shown. The beam is simply supported with torsionally fixed ends. Bending is about the strong axis.

w_D = 0.460 kip/ft applied 6 in. off centerline
w_L = 1.38 kip/ft applied 6 in. off centerline



Solution:

Material Properties:

HSS6×4×¼ ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi — Manual Table 2-3

Geometric properties:

HSS6×4×¼ $\dfrac{h}{t} = 22.8$ $\dfrac{b}{t} = 14.2$ t = 0.233 in. Z_x = 8.53 in.^{3} — Manual Table 1-11

Calculate the required strengths

LRFD	ASD
$w_u = 1.2(0.460\text{ kip/ft}) + 1.6(1.38\text{ kip/ft})$ $= 2.76\text{ kip/ft}$	$w_a = 0.460\text{ kip/ft} + 1.38\text{ kip/ft}$ $= 1.84\text{ kip/ft}$

Calculate the maximum shear (at the supports)

LRFD	ASD
$V_r = V_u = \dfrac{w_u l}{2}$ $= \dfrac{2.76\text{ kip/ft}\left(8.00\text{ ft}\right)}{2} = 11.0\text{ kips}$	$V_r = V_a = \dfrac{w_a l}{2}$ $= \dfrac{1.84\text{ kip/ft}\left(8.00\text{ ft}\right)}{2} = 7.36\text{ kips}$

Calculate the maximum torsion (at the supports)

LRFD	ASD
$T_r = T_u = \dfrac{w_u le}{2}$ $= \dfrac{2.76\text{ kip/ft}\left(8.00\text{ ft}\right)\left(6.00\text{ in.}\right)}{2}$ $= 66.2\text{ kip-in.}$	$T_r = T_a = \dfrac{w_a le}{2}$ $= \dfrac{1.84\text{ kip/ft}\left(8.00\text{ ft}\right)(6.00\text{ in.})}{2}$ $= 44.2\text{ kip-in.}$

Calculate the nominal shear strength

h = 6.00 in. – 3(0.233 in.) = 5.30 in.

$A_w = 2ht$ = 2(5.30 in.)(0.233 in.) = 2.47 in.^{2} — Section G5

$k_v = 5$

Calculate the web shear coefficient

$$\frac{h}{t_w} = 22.8 \leq 1.10\sqrt{\frac{k_v E}{F_y}} = 1.10\sqrt{\frac{5(29{,}000\text{ksi})}{46\text{ ksi}}} = 61.8 \quad \text{therefore,} \quad C_v = 1.0$$ Eqn. G2-3

$$V_n = 0.6F_yA_wC_v = 0.6(46\text{ ksi})(2.47\text{ in.}^2)(1.0) = 68.2\text{ kips}$$ Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
$\phi_v = 0.90$ $V_c = \phi_v V_n$ = 0.90(68.2 kips) = 61.4 kips	$\Omega_v = 1.67$ $V_c = V_n / \Omega_v$ = 68.2 kips/1.67 = 40.8 kips

Section G1

Calculate the nominal flexural strength — Section F7

Flexural yielding limit state

$$M_n = M_p = F_yZ_x = 46\text{ ksi}(8.53\text{ in.}^3) = 392\text{ kip-in.}$$ Eqn. F7-1

Flange local buckling limit state

$\dfrac{b}{t} = 14.2 < 1.12\sqrt{\dfrac{E}{F_y}} = 1.12\sqrt{\dfrac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 28.1$; therefore the flange is compact and the flange local buckling limit state does not apply. — Table B4.1 Case 12

Web local buckling limit state

$\dfrac{h}{t} = 22.8 < 2.42\sqrt{\dfrac{E}{F_y}} = 2.42\sqrt{\dfrac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 60.8$; therefore the web is compact and the web local buckling limit state does not apply. — Table B4.1 Case 13

Therefore M_n = 392 kip-in., controlled by the flexural yielding limit state.

Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$ $M_c = \phi_b M_n$ = 0.90(392 kip-in.) = 353 kip-in.	$\Omega_b = 1.67$ $M_c = M_n / \Omega_b$ = 392 kip-in./1.67 =235 kip-in.

Section F1

Take the available torsional strength from ***Example H.5a***

LRFD	ASD
$T_c = \phi_T T_n$ = 0.90(279 kip-in.) = 251 kip-in.	$T_c = T_n / \Omega_T$ = 279 kip-in./1.67 = 167 kip-in.

Ex. H.5a

Check combined strength at several locations where $T_r > 0.2T_c$ — Section H3.2

Check at the supports, the point of maximum shear and torsion.

LRFD	ASD
$\dfrac{T_r}{T_c} = \dfrac{66.2\text{ kip-in.}}{251\text{ kip-in.}} = 0.264 > 0.20$ therefore, use Equation H3-6 $\left(\dfrac{P_r}{P_c} + \dfrac{M_r}{M_c}\right) + \left(\dfrac{V_r}{V_c} + \dfrac{T_r}{T_c}\right)^2 \le 1.0$ $(0+0)+\left(\dfrac{11.0\text{ kips}}{61.4\text{ kips}} + \dfrac{66.2\text{ kip-in.}}{251\text{ kip-in.}}\right)^2$ $= 0.196 < 1.0$ **o.k.**	$\dfrac{T_r}{T_c} = \dfrac{44.2\text{ kip-in.}}{167\text{ kip-in.}} = 0.265 > 0.20$ therefore, use Equation H3-6 $\left(\dfrac{P_r}{P_c} + \dfrac{M_r}{M_c}\right) + \left(\dfrac{V_r}{V_c} + \dfrac{T_r}{T_c}\right)^2 \le 1.0$ $(0+0)+\left(\dfrac{7.36\text{ kips}}{40.8\text{ kips}} + \dfrac{44.2\text{ kip-in.}}{167\text{ kip-in.}}\right)^2$ $= 0.198 < 1.0$ **o.k.**

Section H3.2

Eqn. H3-6

Check near the location where $T_r = 0.2T_c$. *This is the location with the largest bending moment required to be considered in the interaction.*

Calculate the shear and moment at this location, x.

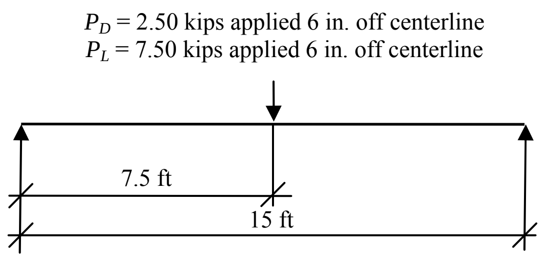
LRFD	ASD
$x = \dfrac{66.2\text{ kip-in.} - (0.20)(251\text{ kip-in.})}{(2.76\text{ kip/ft})(6.00\text{ in.})} = 0.966\text{ ft}$ $\dfrac{T_r}{T_c} = 0.20$ $V_r = 11.0\text{ kips} - (0.966\text{ ft})(2.76\text{ kips/ft})$ $= 8.33\text{ kips}$ $M_r = \dfrac{(2.76\text{ kip/ft})(0.966\text{ ft})^2}{2}$ $\times (8.33\text{ kips})(0.966\text{ ft})$ $= 9.33\text{ kip-ft} = 112\text{ kip-in.}$ $\left(0+\dfrac{112\text{ kip-in.}}{353\text{ kip-in.}}\right)+\left(\dfrac{8.33\text{ kips}}{61.4\text{ kips}} + 0.20\right)^2$ $= 0.430 \le 1.0$ **o.k.**	$x = \dfrac{44.2\text{ kip-in.} - (0.20)(167\text{ kip-in.})}{(1.84\text{ kip/ft})(6.00\text{ in.})} = 0.978\text{ ft}$ $\dfrac{T_r}{T_c} = 0.20$ $V_r = 7.36\text{ kips} - (0.978\text{ ft})(1.84\text{ kips/ft})$ $= 5.56\text{ kips}$ $M_r = \dfrac{(1.84\text{ kip/ft})(0.966\text{ ft})^2}{2}$ $\times (5.56\text{ kips})(0.966\text{ ft})$ $= 6.23\text{ kip-ft} = 74.8\text{ kip-in.}$ $\left(0+\dfrac{74.8\text{ kip-in.}}{235\text{ kip-in.}}\right)+\left(\dfrac{5.56\text{ kips}}{40.8\text{ kips}} + 0.20\right)^2$ $= 0.431 \le 1.0$ **o.k.**

Eqn. H3-6

Example H.6 W-Shape Torsional Strength

Given:

This design example is taken from AISC *Design Guide 9 – Torsional Analysis of Structural Steel Members*. As shown below, a W10×49 spans 15 ft (180 in.) and supports concentrated loads at midspan that act at a 6 in. eccentricity with respect to the shear center. Determine the stresses on the cross section and the adequacy of the section to support the loads.



Beam Loading Diagram

The end conditions are assumed to be flexurally pinned and unrestrained for warping torsion. The eccentric load can be resolved into a torsional moment and a load applied through the shear center.

Solution:

Material Properties:

W10×49 ASTM A992 F_y = 50 ksi F_u = 65 ksi

Manual Table 2-3

Geometric Properties:

W10×49 I_x = 272 in.^{4} S_x= 54.6 in.^{3} t_f = 0.560 in. t_w = 0.340 in.
J = 1.39 in.^{4} C_w = 2,070 in.^{6} Z_x = 60.4 in.^{3}

Manual Table 1-1

Additional Torsional Properties:

W10×49 S_{w1}= 33.0 in.^{4} a = 62.1 in. W_{no} = 23.6 in.^{2}
Q_f = 13.0 in.^{3} Q_w = 30.2 in.^{3}

Design Guide 9 Appendix A

Calculate the required strength

LRFD	ASD
P_u = 1.2(2.50 kips) + 1.6(7.50 kips) = 15.0 kips	P_a = 2.50 kips + 7.50 kips = 10.0 kips
$V_u = \dfrac{P_u}{2} = \dfrac{15.0 \text{ kips}}{2} = 7.50 \text{ kips}$	$V_a = \dfrac{P_a}{2} = \dfrac{10.0 \text{ kips}}{2} = 5.00 \text{ kips}$
$M_u = \dfrac{P_u l}{4} = \dfrac{15.0 \text{ kips}(15.0 \text{ ft})(12 \text{ in./ft})}{4}$ $= 675$ kip-in.	$M_a = \dfrac{P_a l}{4} = \dfrac{10.0 \text{ kips}(15.0 \text{ ft})(12 \text{ in./ft})}{4}$ $= 450$ kip-in.
$T_u = P_u e$ = 15.0 kips(6.00 in.) = 90.0 kip-in.	$T_a = P_a e$ = 10.0 kips(6.00 in.) = 60.0 kip-in.

Calculate the normal and shear stresses from flexure

LRFD	ASD
$\sigma_{ub} = \dfrac{M_u}{S_x} = \dfrac{675 \text{ kip-in.}}{54.6 \text{ in.}^3} = 12.4 \text{ ksi}$ (compression at top, tension at bottom) $\tau_{ub\ web} = \dfrac{V_u Q_w}{I_x t_w} = \dfrac{7.5 \text{ kips}\left(30.2 \text{ in.}^3\right)}{272 \text{ in.}^4\left(0.340 \text{ in.}\right)} = 2.45 \text{ ksi}$ $\tau_{ub\ flange} = \dfrac{V_u Q_f}{I_x t_f} = \dfrac{7.5 \text{ kips}\left(13.0 \text{ in.}^3\right)}{272 \text{ in.}^4\left(0.560 \text{ in.}\right)} = 0.640 \text{ ksi}$	$\sigma_{ab} = \dfrac{M_a}{S_x} = \dfrac{450 \text{ kip-in.}}{54.6 \text{ in.}^3} = 8.24 \text{ ksi}$ (compression at top, tension at bottom) $\tau_{ab\,web} = \dfrac{V_a Q_w}{I_x t_w} = \dfrac{5.00 \text{ kips}\left(30.2 \text{ in.}^3\right)}{272 \text{ in.}^4\left(0.340 \text{ in.}\right)} = 1.63 \text{ ksi}$ $\tau_{ab\,web} = \dfrac{V_a Q_f}{I_x t_f} = \dfrac{5.00 \text{ kips}\left(13.0 \text{ in.}^3\right)}{272 \text{ in.}^4\left(0.560 \text{ in.}\right)} = 0.427 \text{ ksi}$

Design Guide 9 Eqn.4.5

Design Guide 9 Eqn. 4.6

Design Guide 9 Eqn. 4.6

Calculate torsional stresses

The following functions are taken from AISC’s Design Guide 9 – *Torsional Analysis of Structural Steel Members* Appendix B, Case 3, with α = 0.5.

Design Guide 9 Appendix B Case 3, $\alpha = 0.5$

$$\frac{l}{a} = \frac{180 \text{ in.}}{62.1 \text{ in.}} = 2.90$$

At midspan ($z/l = 0.5$)

Using the graphs for θ, θ", θ' and θ"', select values

Design Guide 9 Appendix B Case 3, $\alpha = 0.5$

For θ; $\theta \times \left(\dfrac{GJ}{T_r}\right)\left(\dfrac{1}{l}\right) = +0.09$ Solve for $\theta = +0.09\dfrac{T_r l}{GJ}$

For θ"; $\theta'' \times \left(\dfrac{GJ}{T_r}\right) a = -0.44$ Solve for $\theta'' = -0.44\dfrac{T_r}{GJa}$

For θ' ; $\theta' \times \left(\dfrac{GJ}{T_r}\right) = 0$ Therefore $\theta' = 0$

For θ"'; $\theta''' \times \left(\dfrac{GJ}{T_r}\right) a^2 = -0.50$ Solve for $\theta''' = -0.50\dfrac{T_r}{GJa^2}$

At the support ($z/l = 0$)

Design Guide 9 Appendix B Case 3, $\alpha = 0.5$

For θ; $\theta \times \left(\dfrac{GJ}{T_r}\right)\left(\dfrac{1}{l}\right) = 0$ Therefore $\theta = 0$

For θ"; $\theta'' \times \left(\dfrac{GJ}{T_r}\right) a = 0$ Therefore $\theta'' = 0$

For θ'; $\theta' \times \left(\dfrac{GJ}{T_r}\right) = +0.28$ Solve for $\theta' = +0.28\dfrac{T_r}{GJ}$

For θ"'; $\theta''' \times \left(\dfrac{GJ}{T_r}\right) a^2 = -0.22$ Solve for $\theta''' = -0.22\dfrac{T_r}{GJa^2}$

In the above calculations note that the applied torque is negative with the sign convention used.

Calculate $\dfrac{T_r}{GJ}$ *for use below*

LRFD	ASD
$\dfrac{T_u}{GJ} = \dfrac{\text{-90.0 kip-in.}}{(11{,}200\text{ ksi})(1.39\text{ in.}^4)}$ $= -5.78 \times 10^{-3}$ rad/in.	$\dfrac{T_a}{GJ} = \dfrac{\text{-60.0 kip-in.}}{(11{,}200\text{ ksi})(1.39\text{ in.}^4)}$ $= -3.85 \times 10^{-3}$ rad/in.

Calculate the shear stresses due to pure torsion

$$\tau_t = Gt\theta'$$

Design Guide 9 Eqn. 4.1

LRFD	ASD
At midspan $\theta' = 0;\ \tau_{ut} = 0$ At the support, for the web; $\tau_{ut} = 11{,}200\text{ ksi}(0.340\text{ in.})(0.28)\left(\dfrac{-5.78\text{ rad}}{10^3\text{ in.}}\right)$ $= -6.16$ ksi At the support, for the flange; $\tau_{ut} = 11{,}200\text{ ksi}(0.560\text{ in.})(0.28)\left(\dfrac{-5.78\text{ rad}}{10^3\text{ in.}}\right)$ $= -10.2$ ksi	At midspan $\theta' = 0;\ \tau_{at} = 0$ At the support, for the web; $\tau_{at} = 11{,}200\text{ ksi}(0.340\text{ in.})(0.28)\left(\dfrac{-3.85\text{ rad}}{10^3\text{ in.}}\right)$ $= -4.11$ ksi At the support, for the flange; $\tau_{at} = 11{,}200\text{ ksi}(0.560\text{ in.})(0.28)\left(\dfrac{-3.85\text{rad}}{10^3\text{ in.}}\right)$ $= -6.76$ ksi

Design Guide 9 Eqn. 4.1

Design Guide 9 Eqn. 4.1

Calculate the shear stresses due to warping

$$\tau_w = \frac{-ES_{w1}\theta'''}{t_f}$$

Design Guide 9 Eqn. 4.2a

LRFD	ASD
At midspan $\tau_{uw} = \dfrac{-29{,}000\text{ksi}(33.0\text{ in.}^4)}{0.560\text{ in.}}\left[\dfrac{-0.50(-5.78\text{ rad})}{(62.1\text{in.})^2(10^3\text{ in.})}\right]$ $= -1.28$ ksi At the support $\tau_{uw} = \dfrac{-29{,}000\text{ksi}(33.0\text{in.}^4)}{0.560\text{in.}}\left[\dfrac{-0.22(-5.78\text{rad})}{(62.1\text{in.})^2(10^3\text{ in.})}\right]$ $= -0.563$ksi	At midspan $\tau_{aw} = \dfrac{-29{,}000\text{ksi}(33.0\text{in.}^4)}{0.560\text{in.}}\left[\dfrac{-0.50(-3.85\text{rad})}{(62.1\text{ in.})^2(10^3\text{ in.})}\right]$ $= -0.853$ ksi At the support $\tau_{aw} = \dfrac{-29{,}000\text{ksi}(33.0\text{in.}^4)}{0.560\text{in.}}\left[\dfrac{-0.22(-3.85\text{rad})}{(62.1\text{in.})^2(10^3\text{ in.})}\right]$ $= -0.375$ ksi

Design Guide 9 Eqn. 4.2a

Design Guide 9 Eqn. 4.2a

Calculate the normal stresses due to warping

$\sigma_w = EW_{no}\theta''$

Design Guide 9 Eqn. 4.3a

LRFD	ASD
At midspan $\sigma_{uw} = 29{,}000 \text{ ksi}\left(23.6 \text{ in.}^2\right)\left[\dfrac{-0.44\left(-5.78 \text{rad}\right)}{\left(62.1 \text{ in.}\right)\left(10^3 \text{ in.}\right)}\right]$ $= 28.0 \text{ ksi}$ At the support Since $\theta'' = 0, \sigma_{uw} = 0$	At midspan $\sigma_{aw} = 29{,}000 \text{ ksi}\left(23.6 \text{in.}^2\right)\left[\dfrac{-0.44\left(-3.85 \text{rad}\right)}{\left(62.1 \text{ in.}\right)\left(10^3 \text{ in.}\right)}\right]$ $= 18.7 \text{ ksi}$ At the support Since $\theta'' = 0, \sigma_{aw} = 0$

Design Guide 9 Eqn. 4.3a

Calculate the combined stresses

The summarized stresses are as follows:

Summary of Stresses Due to Flexure and Torsion (ksi)														
	LFRD							ASD						
Location	Normal Stresses			Shear Stresses				Normal Stresses			Shear Stresses			
	σ_{uw}	σ_{ub}	f_{un}	τ_{ut}	τ_{uw}	τ_{ub}	f_{uv}	σ_{aw}	σ_{ab}	f_{an}	τ_{at}	τ_{aw}	τ_{ab}	f_{av}
Midspan														
Flange	±28.0	±12.4	±40.4	0	-1.28	±0.640	-1.92	±18.7	±8.24	±26.9	0	-0.853	±0.427	-1.28
Web	----	----	----	0	----	±2.45	±2.45	----	----	----	0	----	±1.63	±1.63
Support														
Flange	0	0	0	-10.2	-0.563	±0.640	-11.4	0	0	0	-6.76	-0.375	±0.427	-7.56
Web	----	----	----	-6.16	----	±2.45	-8.61	----	----	----	-4.11	----	±1.63	-5.74
Maximum			±40.4				-11.4			±26.9				-7.56

LRFD	ASD
The maximum normal stress due to flexure and torsion occurs at the edge of the flange at midspan and is equal to 40.4 ksi. The maximum shear stress due to flexure and torsion occurs in the middle of the flange at the support and is equal to 11.4 ksi.	The maximum normal stress due to flexure and torsion occurs at the edge of the flange at midspan and is equal to 26.9 ksi. The maximum shear stress due to flexure and torsion occurs in the middle of the flange at the support and is equal to 7.56 ksi.

Calculate the available strength

LRFD	ASD
$\phi_T = 0.90$ *Normal Stresses* *Yielding limit state* $F_n = F_y = 50$ ksi **controls** *Lateral-torsional buckling limit state* $C_b = 1.32$	$\Omega_T = 1.67$ *Normal Stresses* *Yielding limit state* $F_n = F_y = 50$ ksi **controls** *Lateral-torsional buckling limit state* $C_b = 1.32$

Section H3.3

Eqn. H3-7

Manual Table 3-1 Case 1

Compute F_n using values from Table 3-10. with L_b = 15.0 ft and C_b = 1.0	Compute F_n using values from Table 3-10. with L_b = 15.0 ft and C_b = 1.0	Manual Table 3-10
$\phi_b M_n$ = 204 kip-ft	M_n / Ω_b = 136 kip-ft	
$F_n = F_{cr}$ $= C_b \dfrac{\phi_b M_n}{\phi_b S_x}$ $= 1.32 \dfrac{204\text{ kip-ft}}{0.90\left(54.6\text{ in.}^3\right)}\left(\dfrac{12\text{ in.}}{\text{ft}}\right)$	$F_n = F_{cr}$ $= C_b \Omega_b \dfrac{M_n / \Omega_b}{S_x}$ $= 1.32\left(1.67\right)\dfrac{136\text{ kip-ft}}{\left(54.6\text{ in.}^3\right)}\left(\dfrac{12\text{ in.}}{\text{ft}}\right)$	Eqn. H3-9
= 65.8 ksi **does not control**	= 65.9 ksi **does not control**	
Local buckling limit state	*Local buckling limit state*	
W10x49 is compact in flexure per user note in Specification Section F2	W10x49 is compact in flexure per user note in Specification Section F2	
$\phi_T F_n$ = 0.9(50 ksi) = 45.0 ksi > 40.4 ksi **o.k.**	F_n / Ω_T = 50 ksi/1.67 = 29.9 ksi > 26.9 ksi **o.k.**	
Shear yielding limit state	*Shear yielding limit state*	
$F_n = 0.6F_y$	$F_n = 0.6F_y$	
Design shear strength	*Allowable shear strength*	
$\phi_T F_n$ = 0.90(0.6)(50 ksi) = 27.0 ksi > 11.4 ksi **o.k.**	F_n / Ω_T = (0.6)(50 ksi)/1.67 = 18.0 ksi > 7.56 ksi **o.k.**	Eqn. H3-8

Calculate the maximum rotation at service load

The maximum rotation occurs at midspan. The service-load torque is:

$$T = Pe = -\left(2.50\text{ kips}+7.50\text{ kips}\right)\left(6.00\text{ in.}\right) = \text{-60.0 kip-in.}$$

Design Guide 9 Appendix B Case 3 $\alpha = 0.5$

The maximum rotation is:

$$\theta = +0.09\frac{Tl}{GJ} = \frac{0.09\left(-60.0\text{ kip-in.}\right)\left(180\text{ in.}\right)}{11{,}200\text{ ksi}\left(1.39\text{ in.}^4\right)} = -0.0624\text{ rads} = -3.58\text{ degrees}$$

See AISC *Design Guide 9: Torsional Analysis of Structural Steel Members* for additional guidance.

CHAPTER I
DESIGN OF COMPOSITE MEMBERS

I1. GENERAL PROVISIONS

The available strength of composite sections may be calculated by one of two methods - the plastic stress distribution method or the strain-compatibility method. The composite design tables in the *Steel Construction Manual* are based on the plastic stress distribution method.

I2. AXIAL MEMBERS

Generally, the available compressive strength of a composite member is based on a summation of the strengths of all of the components of the column. The Specification contains several requirements to ensure that the steel and concrete components work together.

For tension members, the concrete tensile strength is ignored and only the strength of the steel member and properly connected reinforcing is permitted to be used in the calculation of available tensile strength.

Because of concerns about the deformation compatibility of steel and concrete in resisting shear, either the steel or the reinforced concrete, but not both, are permitted to be used in the calculation of available shear strength. Whether the composite column is an encased column or a filled column, it is important to consider the load path within the composite member, and to provide shear transfer mechanisms and appropriate top and bottom details.

The design of encased composite compression and tension members is presented in **Examples I-3** and **I-4**. There are no tables in the Manual for the design of these members.

The design of filled composite compression and tension members is presented in **Examples I-2** and **I-5**. The Manual includes tables for the design of filled composite members in compression.

I3. FLEXURAL MEMBERS

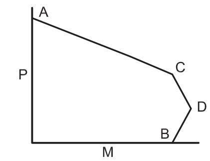
Because a plastic stress distribution is used for both LRFD and ASD designs, the flexural strength of composite beams is generally greater than that of former ASD designs. Shear connectors, in many cases, have lower horizontal shear strength than was permitted in past LRFD specifications. Designers are encouraged to read the discussion on this subject in Commentary Chapter I. The design of a typical composite beam member is illustrated in **Example I-1**.

I4. COMBINED AXIAL FORCE AND FLEXURE

Design for combined axial force and flexure may be accomplished in any of the three methods outlined in the Commentary. **Example I-7** illustrates the plastic-distribution method.

To assist in developing this curve, a series of equations is provided in Figure I-1. These equations define selected points on the interaction curve, without consideration of slenderness effects. Figures I-1a through I-1d outline specific cases, and the applicability of the equations to a cross-section that differs should be carefully considered. As an example, the equations in Figure I-1a are appropriate for the case of side bars located at the centerline, but not for other side bar locations. In contrast, these equations are appropriate for any amount of reinforcing at the extreme reinforcing bar location. In Figure I-1b, the equations are appropriate only for the case of 4 reinforcing bars at the corners of the encased section. When design cases deviate from those presented the appropriate interaction equations can be derived from first principles.

PLASTIC CAPACITIES FOR RECTANGULAR, ENCASED W-SHAPES BENT ABOUT THE X-X AXIS



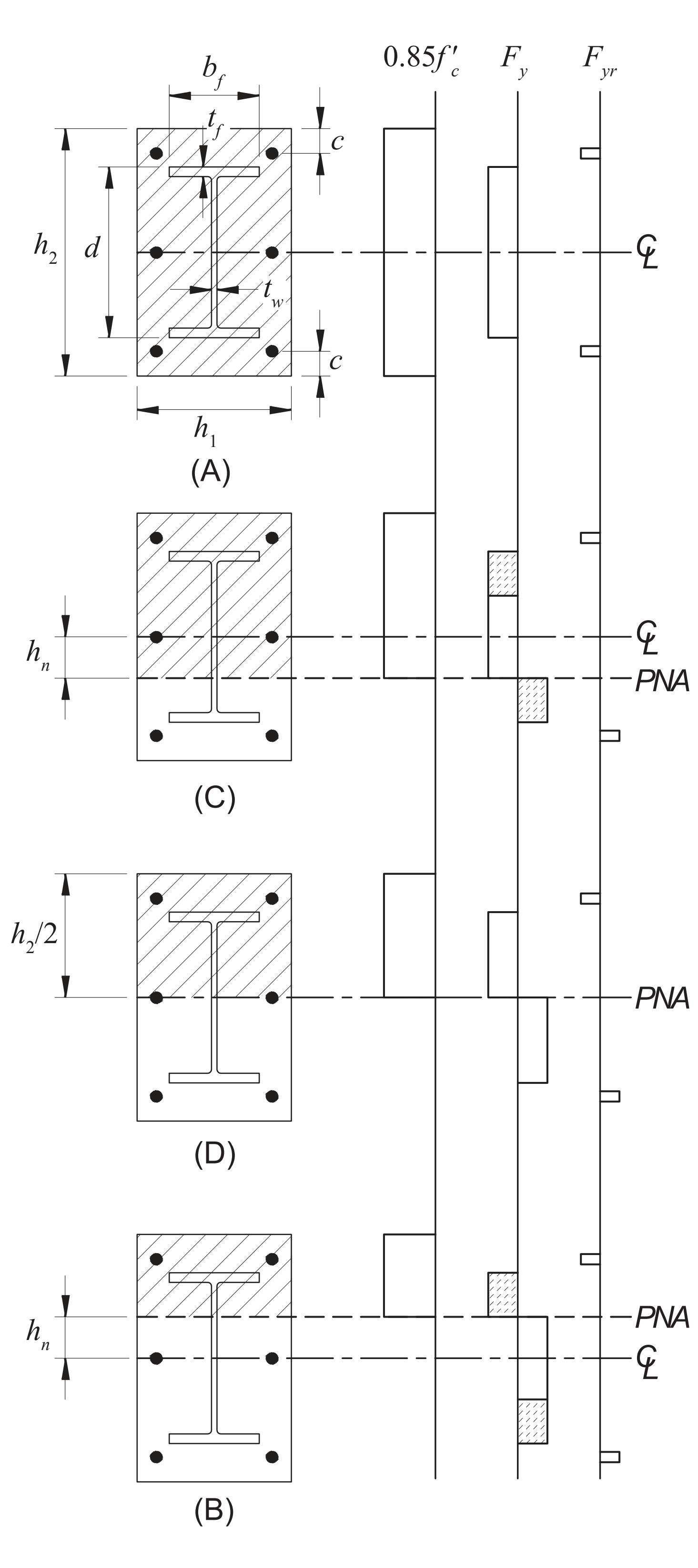
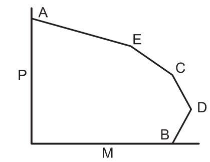
Section	Stress Distribution	Point	Defining Equations
		A	$P_A = A_s F_y + A_{sr} F_{yr} + 0.85 f'_c A_c$ $M_A = 0$ A_s = area of steel shape A_{sr} = area of all continuous reinforcing bars $A_c = h_1 h_2 - A_s - A_{sr}$
		C	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
		D	$P_D = \dfrac{0.85 f'_c A_c}{2}$ $M_D = Z_s F_y + Z_r F_{yr} + \dfrac{Z_c}{2}\left(0.85 f'_c\right)$ Z_s = full x-axis plastic section modulus of steel shape A_{srs} = area of continuous reinforcing bars at the centerline $Z_r = \left(A_{sr} - A_{srs}\right)\left(\dfrac{h_2}{2} - c\right)$ $Z_c = \dfrac{h_1 h_2^2}{4} - Z_s - Z_r$
		B	$P_B = 0$ $M_B = M_D - Z_{sn} F_y - \tfrac{1}{2} Z_{cn}\left(0.85 f'_c\right)$ $Z_{cn} = h_1 h_n^2 - Z_{sn}$ For h_n below the flange $\left(h_n \le \dfrac{d}{2} - t_f\right)$ $h_n = \dfrac{0.85 f'_c\left(A_c + A_{srs}\right) - 2F_{yr} A_{srs}}{2\left[0.85 f'_c\left(h_1 - t_w\right) + 2F_y t_w\right]}$ $Z_{sn} = t_w h_n^2$ For h_n within the flange $\left(\dfrac{d}{2} - t_f < h_n \le \dfrac{d}{2}\right)$ $h_n = \dfrac{0.85 f'_c\left(A_c + A_s - d b_f + A_{srs}\right) - 2F_y\left(A_s - d b_f\right) - 2F_{yr} A_{srs}}{2\left[0.85 f'_c\left(h_1 - b_f\right) + 2F_y b_f\right]}$ $Z_{sn} = Z_s - b_f\left(\dfrac{d}{2} - h_n\right)\left(\dfrac{d}{2} + h_n\right)$ For h_n above the flange $\left(h_n > \dfrac{d}{2}\right)$ $h_n = \dfrac{0.85 f'_c\left(A_c + A_s + A_{srs}\right) - 2F_y A_s - 2F_{yr} A_{srs}}{2\left(0.85 f'_c h_1\right)}$ $Z_{sn} = Z_{sx}$ = full x-axis plastic section modulus of steel shape

Figure I-1a. W-Shapes, Strong-Axis Anchor Points.

PLASTIC CAPACITIES FOR RECTANGULAR, ENCASED W-SHAPES BENT ABOUT THE Y-Y AXIS



Section	Stress Distribution	Point	Defining Equations
		A	$P_A = A_s F_y + A_{sr} F_{yr} + 0.85 f'_c A_c$ $M_A = 0$ A_s = area of steel shape A_{sr} = area of continuous reinforcing bars $A_c = h_1 h_2 - A_s - A_{sr}$
		E	$P_E = A_s F_y + (0.85 f'_c)\left[A_c - \frac{h_1}{2}(h_2 - b_f) + \frac{A_{sr}}{2}\right]$ $M_E = M_D - Z_{sE} F_y - \frac{1}{2} Z_{cE}(0.85 f'_c)$ $Z_{sE} = Z_{sy}$ = full y-axis plastic section modulus of steel shape $Z_{cE} = \frac{h_1 b_f^2}{4} - Z_{sE}$
		C	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
		D	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = Z_s F_y + Z_r F_{sr} + \frac{1}{2} Z_c (0.85 f'_c)$ Z_s = full y-axis plastic section modulus of steel shape $Z_r = A_{sr}\left(\frac{h_2}{2} - c\right)$ $Z_c = \frac{h_1 h_2^2}{4} - Z_s - Z_r$
		B	$P_B = 0$ $M_B = M_D - Z_{sn} F_y - \frac{1}{2} Z_{cn}(0.85 f'_c)$ $Z_{cn} = h_1 h_n^2 - Z_{sn}$ For h_n below the flange $\left(\frac{t_w}{2} < h_n \le \frac{b_f}{2}\right)$ $h_n = \frac{0.85 f'_c (A_c + A_s - 2t_f b_f) - 2F_y (A_s - 2t_f b_f)}{2\left[4t_f F_y + (h_1 - 2t_f) 0.85 f'_c\right]}$ $Z_{sn} = Z_s - 2t_f\left(\frac{b_f}{2} + h_n\right)\left(\frac{b_f}{2} - h_n\right)$ For h_n above the flange $\left(h_n > \frac{b_f}{2}\right)$ $h_n = \frac{0.85 f'_c (A_c + A_s) - 2F_y A_s}{2\left[0.85 f'_c h_1\right]}$ $Z_{sn} = Z_{sy}$ = full y-axis plastic section modulus of steel shape

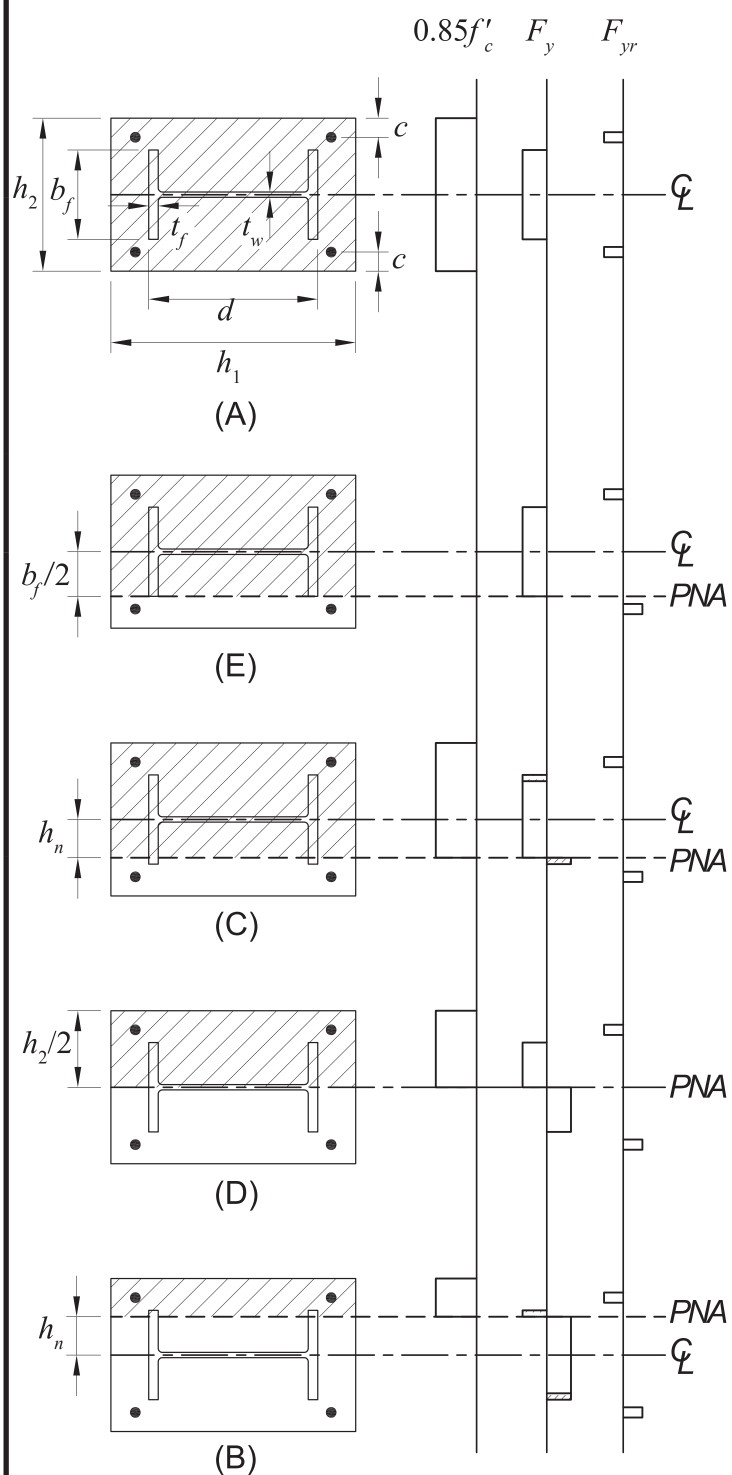
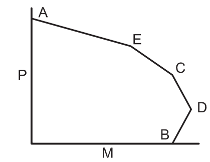
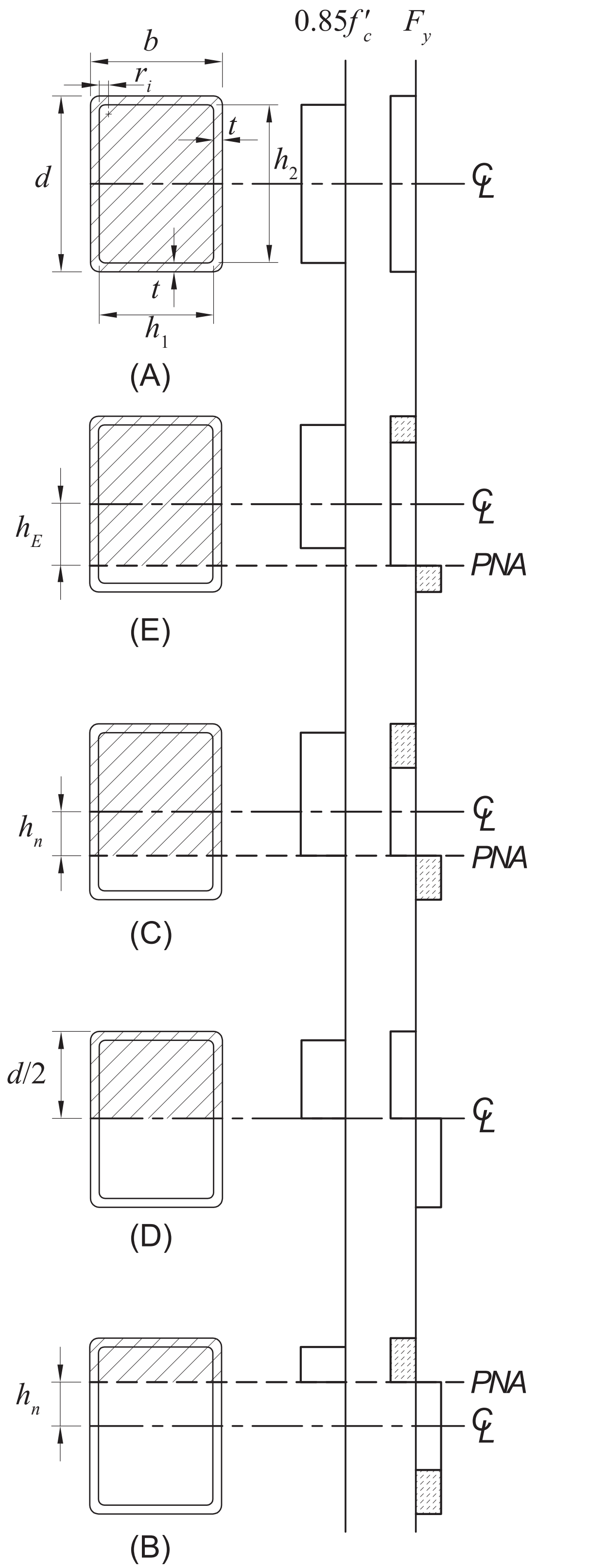


Figure I-1b. W-Shapes, Weak-Axis Anchor Points.

PLASTIC CAPACITIES FOR COMPOSITE, FILLED HSS BENT ABOUT THE X-X AXIS

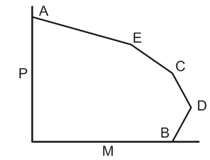




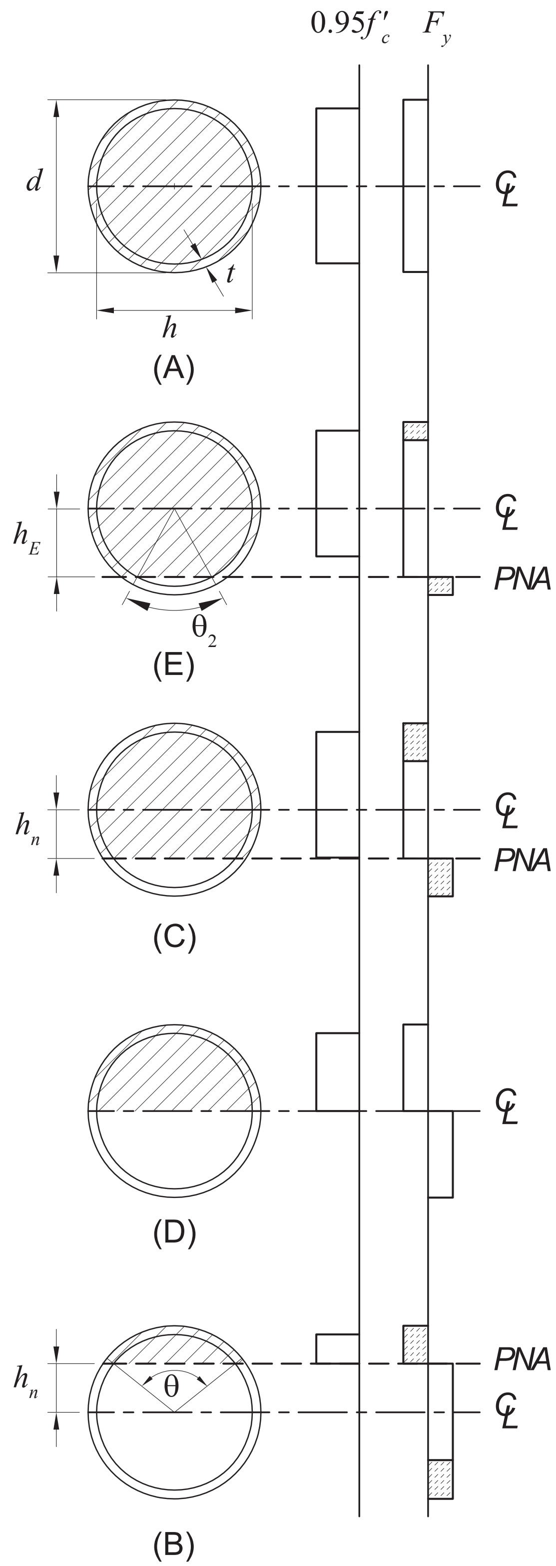
Point	Defining Equations
A	$P_A = F_y A_s + 0.85 f'_c A_c$ $M_A = 0$ A_s = area of steel shape $A_c = h_1 h_2 - 0.858 r_i^2$ $h_1 = b - 2t$ $h_2 = d - 2t$
E	$P_E = \frac{1}{2}\left(0.85 f'_c A_c\right) + 0.85 f'_c h_1 h_E + 4 F_y t h_E$ $M_E = M_D - F_y Z_{sE} - \frac{1}{2}\left(0.85 f'_c Z_{cE}\right)$ $Z_{cE} = h_1 h_E^2$ $Z_{sE} = 2 t h_E^2$ $h_E = \frac{h_n}{2} + \frac{d}{4}$
C	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
D	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = F_y Z_s + \frac{1}{2}\left(0.85 f'_c Z_c\right)$ Z_s = full x-axis plastic section modulus of HSS $Z_c = \frac{h_1 h_2^2}{4} - 0.192 r_i^3$
B	$P_B = 0$ $M_B = M_D - F_y Z_{sn} - \frac{1}{2}\left(0.85 f'_c Z_{cn}\right)$ $Z_{sn} = 2 t h_n^2$ $Z_{cn} = h_1 h_n^2$ $h_n = \frac{0.85 f'_c A_c}{2\left[0.85 f'_c h_1 + 4 t F_y\right]} \le \frac{h_2}{2}$

Figure I-1c. Filled Rectangular or Square HSS, Strong- or Weak-Axis Anchor Points.

PLASTIC CAPACITIES FOR COMPOSITE, FILLED ROUND HSS BENT ABOUT ANY AXIS



Section	Stress Distribution



Point	Defining Equations
A	$P_A = F_y A_s + 0.95 f'_c A_c$ * $M_A = 0$ $A_s = \pi(dt - t^2)$ $A_c = \dfrac{\pi h^2}{4}$
E	$P_E = P_A - \tfrac{1}{4}\left[F_y\left(d^2 - h^2\right) + \tfrac{1}{2}\left(0.95 f'_c\right)h^2\right]\left(\theta_2 - \sin\theta_2\right)$ $M_E = F_y Z_{sE} + \tfrac{1}{2}\left(0.95 f'_c Z_{cE}\right)$ $Z_{cE} = \dfrac{h^3}{6}\sin^3\left(\dfrac{\theta_2}{2}\right)$ $Z_{sE} = \dfrac{\left(d^3 - h^3\right)}{6}\sin\left(\dfrac{\theta_2}{2}\right)$ $h_E = \dfrac{h_n}{2} + \dfrac{h}{4}$ $\theta_2 = \pi - 2\arcsin\left(\dfrac{2h_E}{h}\right)$
C	$P_C = 0.95 f'_c A_c$ $M_C = M_B$
D	$P_D = \dfrac{0.95 f'_c A_c}{2}$ $M_D = F_y Z_s + \tfrac{1}{2}\left(0.95 f'_c Z_c\right)$ Z_s = plastic section modulus of steel shape $= \dfrac{d^3}{6} - Z_c$ $Z_c = \dfrac{h^3}{6}$
B	$P_B = 0$ $M_B = F_y Z_{sB} + \tfrac{1}{2}\left(0.95 f'_c Z_{cB}\right)$ $Z_{sB} = \dfrac{\left(d^3 - h^3\right)}{6}\sin\left(\dfrac{\theta}{2}\right)$ $Z_{cB} = \dfrac{h^3\sin^3\left(\dfrac{\theta}{2}\right)}{6}$ $\theta = \dfrac{0.0260K_c - 2K_s}{0.0848K_c} + \dfrac{\sqrt{\left(0.0260K_c + 2K_s\right)^2 + 0.857K_cK_s}}{0.0848K_c}$ (rad) $K_c = f'_c h^2$ $K_s = F_y\left(\dfrac{d-t}{2}\right)t$ ("thin" HSS wall assumed) $h_n = \dfrac{h}{2}\sin\left(\dfrac{\pi - \theta}{2}\right) \le \dfrac{h}{2}$

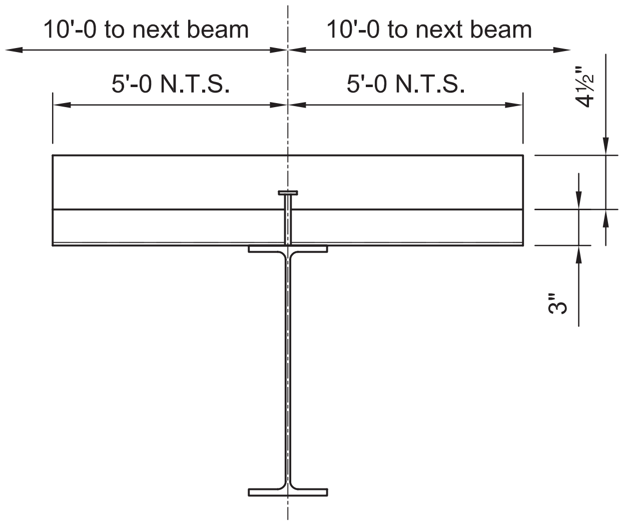
*$0.95f'_c$ may be used for concrete filled round HSS.

Figure I-1d. Filled Round HSS Anchor Points.

Example I-1 Composite Beam Design

Given:

A series of 45-ft span composite beams at 10 ft o.c. are carrying the loads shown below. The beams are ASTM A992 and are unshored. The concrete has f'_c = 4 ksi. Design a typical floor beam with 3 in. 18 gage composite deck, and 4½ in. normal weight concrete above the deck, for fire protection and mass. Select an appropriate beam and determine the required number of ¾-in.-diameter shear studs.



Solution:

Material Properties:

Concrete f'_c = 4 ksi

Beam ASTM A992 F_y = 50 ksi F_u = 65 ksi

Manual Table 2-3

Loads:

Dead load:
- Slab = 75 psf
- Beam weight = 8 psf (assumed)
- Miscellaneous = 10 psf (ceiling etc.)
- total = 93 psf

Live load:
- Non-reduced = 100 psf
- Construction = 20 psf

Total dead load W_D = 93 psf (1 kip/1000 lb)(10 ft) = 0.930 kip/ft
Total live load W_L = 100 psf (1 kip/1000 lb)(10 ft) = 1.00 kip/ft

Construction dead load (unshored) = 83 psf(1 kip/1000 lb)(10 ft) = 0.830 kip/ft
Construction live load (unshored) = 20 psf(1 kip/1000 lb)(10 ft) = 0.200 kip/ft

Determine the required flexural strength

LRFD	ASD
$w_u = 1.2(0.930 \text{ kip/ft}) + 1.6(1.00 \text{ kip/ft})$ $= 2.72 \text{ kips/ft}$ $M_u = \dfrac{(2.72 \text{ kips/ft})(45 \text{ ft})^2}{8} = 689 \text{ kip-ft}$	$w_a = 0.930 \text{ kip/ft} + 1.00 \text{ kip/ft}$ $= 1.93 \text{ kips/ft}$ $M_a = \dfrac{(1.93 \text{ kips/ft})(45 \text{ ft})^2}{8} = 489 \text{ kip-ft}$

Limitations:

1. Normal weight concrete   10 ksi $\geq f'_c \geq$ 3 ksi f'_c = 4 ksi **o.k.** Section I1.2
2. Minimum length of shear studs = 4d_{stud} = 4(0.75 in.) = 3.00 in. Section I1.3
3. $h/t_w \leq 3.76\sqrt{E/F_y}$ required to determine M_n from the plastic stress distribution on the composite section for the limit state of yielding (plastic moment). User note: All current ASTM W shapes satisfy the limit for $F_y \leq$ 50 ksi. Section I3.2a
4. Normal rib height of formed steel deck $\leq$ 3 in. **o.k.** Section I3.2c
5. $d_{stud} \leq$ 3/4 in. **ok.** Section I3.2c
6. The slab thickness above the deck shall be not less than 2 in. **o.k.**
7. Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 in.

Use Tables 3-19, 3-20 and 3-21 from the Manual to select an appropriate member.

Determine b_{eff}

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, which shall not exceed: Section I3.1.1a

(1) one-eighth of the beam span, center to center of supports

$$\frac{45 \text{ ft.}}{8}(2) = 11.3 \text{ ft}$$

(2) one-half the distance to center-line of the adjacent beam

$$\frac{10 \text{ ft.}}{2}(2) = 10.0 \text{ ft} \quad \textbf{Controls}$$

(3) the distance to the edge of the slab
Not applicable

Calculate the moment arm for the concrete force measured from the top of the steel shape, Y2

Assume a = 1.0 in. (Some assumption must be made to start the design process. An assumption of 1.0 in. has proven to be a reasonable starting point in many design problems.)

$Y2 = t_{slab} - a/2 = 7.5 - ½ = 7.0$ in.

Enter Manual Table 3-19 with the required strength and $Y2$=7.0 in. Select a beam and plastic neutral axis location that indicates sufficient available strength.

Select a W21×50 as a trial beam. Manual Table 3-19

With PNA location 5 (BFL) and $\sum Q_n$ = 386 kips, this composite shape has an available strength of:

Manual Table 3-19

LRFD	ASD
$\phi_b M_n$ = 770 kip-ft > 689 kip-ft **o.k.**	M_n / Ω_b = 512 kip-ft > 489 kip-ft **o.k.**

Manual Table 3-19

Note that based on the available flexural strength provided in Table 3-19, the required PNA location for ASD and LRFD differ. This is because the live to dead load ratio in this example is not equal to 3. Thus, the PNA location requiring the most shear transfer is selected to be acceptable for ASD. It will be conservative for LRFD.

Check a

$$a = \frac{\sum Q_n}{0.85 f'_c b} = \frac{386 \text{ kips}}{0.85(4 \text{ ksi})(10 \text{ ft})(12 \text{ in./ft})} = 0.946 \text{ in.}$$

0.946 in. < 1.0 in. assumed **o.k.**

Check the beam deflections and available strength

Check selected member strength as an unshored beam under construction loads assuming adequate lateral bracing through the deck attachment to the beam flange.

LRFD	ASD
Calculate the required strength 1.4 DL = 1.4 (0.830 kip/ft) = 1.16 kip/ft 1.2DL+1.6LL = 1.2(0.830 kip/ft) + 1.6(0.200 kip/ft) = 1.32 kips/ft $M_{u\,(\text{unshored})} = \dfrac{1.32 \text{ kips/ft}(45 \text{ ft})^2}{8}$ = 334 kip-ft The design strength for a W21×50 is: 413 kip-ft > 334 kip-ft **o.k.**	*Calculate the required strength* DL+LL = 0.830 kip/ft + 0.200 kip/ft = 1.03 kip/ft $M_{a\,(\text{unshored})} = \dfrac{1.03 \text{ kips/ft}(45 \text{ ft})^2}{8}$ = 261 kip-ft The allowable strength for a W21× 50 is: 274 kip-ft > 261 kip-ft **o.k.**

Manual Table 3-2

Check the deflection of the beam under construction, considering only the weight of the slab and the self-weight of the beam as contributing to the construction dead load.

From Manual Table 1-1, a W21×50 has I_x = 984 in.^{4}

$$\Delta = \frac{5}{384}\frac{w_{DL} l^4}{EI} = \frac{5(0.830 \text{ kip/ft})(45 \text{ ft})^4 (1728 \text{ in.}^3/\text{ft}^3)}{384(29{,}000 \text{ ksi})(984 \text{ in.}^4)} = 2.68 \text{ in.}$$

Camber the beam for 80% of the calculated deflection

camber = (0.8)(2.68 in.) = 2.14 in.

Round the calculated value down to the nearest 1/4 in.; therefore, specify 2 in. of camber.

Check live load deflection

$\Delta_{LL} < l/360 = (45\text{ ft})(12\text{ in./ft})/360 = 1.5\text{ in.}$

A lower bound moment of inertia for composite beams is tabulated in Manual Table 3-20. Manual Table 3-20

For a W21×50 with $Y2$=7.0 and the PNA at location 5(BFL), $I_{LB} = 2520\text{ in.}^4$

$$\Delta_{LL} = \frac{5}{384}\frac{w_{LL}l^4}{EI_{LB}} = \frac{5(1.0\text{ kip/ft})(45\text{ ft})^4(1728\text{ in.}^3/\text{ft}^3)}{384(29{,}000\text{ ksi})(2520\text{ in.}^4)} = 1.26\text{ in.}$$

1.26 in. < 1.5 in. **o.k.**

A check for vibration serviceability should also be done. See Design Guide 11.

Determine if the beam has sufficient available shear strength

LRFD	ASD
$V_u = \frac{45\text{ft}}{2}(2.72\text{kip/ft}) = 61.2\text{ kips}$ $\phi V_n = 237\text{ kips} > 61.2\text{ kips}$ **o.k.**	$V_a = \frac{45\text{ft}}{2}(1.93\text{kip/ft}) = 43.4\text{ kips}$ $V_n / \Omega = 158\text{ kips} > 43.4\text{ kips}$ **o.k.**

Manual Table 3-2

Determine the required number of shear stud connectors

Using perpendicular deck with one ¾-in.-diameter weak stud per rib in normal weight, 4 ksi concrete: $Q_n = 17.2$ kips/stud. Manual Table 3-21

$$\frac{\Sigma Q_n}{Q_n} = \frac{386\text{ kips}}{17.2\text{ kips}} = 22.4\text{; therefore, use 23 studs on each side of the beam.}$$

Section I3.2d(5)

Total number of shear connectors; use 2(23) = 46 shear connectors; therefore, more than one stud per rib will be required in some locations. Section I3.2d(6)

Using perpendicular deck with two ¾- in. diameter weak studs per rib in normal weight, 4 ksi concrete: $Q_n = 14.6$ kips/stud Manual Table 3-21

Each deck flute is at 12 in. on center; therefore, the minimum number of flutes available on each side of the beam center is:

$$\frac{l}{2} - 1 = \frac{45}{2} - 1 = 21.5\text{ : Assume 21 flutes on each side of the beam.}$$

Solve for the number of flutes where two studs are required on each side of the beam.

$$\left(\frac{17.2\text{ kips}}{\text{stud}}\right)\left(\frac{1\text{ stud}}{\text{flute}}\right)(n\text{ flutes}) + \left(\frac{14.6\text{ kips}}{\text{stud}}\right)\left(\frac{2\text{ stud}}{\text{flute}}\right)(21 - n\text{ flutes}) = 386\text{ kips}$$

$n = 18.9$ flutes

Therefore, use 18 flutes with one stud:

$$\left(\frac{17.2\text{ kips}}{\text{stud}}\right)\left(\frac{1\text{ stud}}{\text{flute}}\right)(18\text{ flutes}) = 310\text{ kips}$$

And 3 flutes with two studs:

Section

I3.2c(6)

$$\left(\frac{14.6\,\text{kips}}{\text{stud}}\right)\left(\frac{2\,\text{stud}}{\text{flute}}\right)(3\,\text{flutes}) = 87.6\,\text{kips}$$

Total shear strength

310 kips + 87.6 kips = 398 kips > 386 kips **o.k.**

Provide a minimum of one stud in each flute along the entire length of the beam and provide two studs in each of the first three flutes starting from both ends of the beam.

Section I3.2d(6)

The studs are to be 5 in. long, so that they will extend a minimum of 1 1/2 in. above the top of the deck into the slab and will have at least 1/2 in. of concrete cover over the top of the installed studs.

Section I3.2c(1b)

Check shear stud placement and spacing (with flutes spaced 12 in. o.c.) according to Specification Section I3.2d(6):

Manual Table 1-1

t_f = 0.535 in. for a W21x50

$d_{stud} \leq 2.5t_f = 2.5(0.535\text{ in.}) = 1.34\text{ in.}$

The center-to-center spacing in any direction is limited as follows:

$4d_{stud}$ < 12 in. < ($8t_{slab}$ or 36 in.)

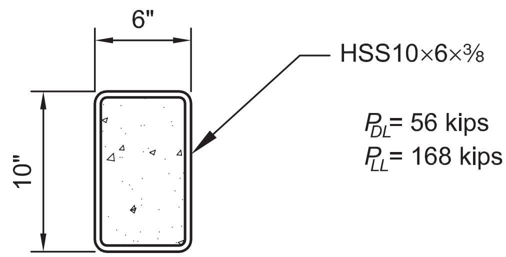
Therefore, the shear stud spacing requirements along the longitudinal axis of the supporting composite beam are met.

The minimum center to center spacing of the shear studs transverse to the longitudinal axis of the supporting composite beam is $4d_{stud}$ = 4(0.750 in.) = 3.00 in.

Example I-2 Filled Composite Column in Axial Compression

Given:

Determine if a 14-ft long HSS10×6×⅜ ASTM A500 Gr. B column filled with f'_c = 5 ksi normal weight concrete can support a dead load of 56 kips and a live load of 168 kips in axial compression. The column is pinned at both ends in a braced frame and the concrete at the base bears directly on the base plate. At the top, the load is transferred to the concrete in direct bearing.



Solution:

Calculate the required compressive strength

LRFD	ASD
P_u = 1.2(56 kips) + 1.6(168 kips) = 336 kips	P_a = 56 kips + 168 kips = 224 kips

The available strength in axial compression can be determined directly from the Manual at KL = 14 ft as:

LRFD	ASD
$\phi_c P_n$ = 353 kips 353 kips > 336 kips **o.k.**	P_n / Ω_c = 236 kips 236 kips > 224 kips **o.k.**

Manual Table 4-14

Supporting Calculations:

The available strength of this filled composite section can be most easily determined by using Table 4-14 of the Manual. Alternatively, the available strength can be determined by direct application of the Specification requirements, as illustrated below.

Material Properties:

Column ASTM A500 Gr. B F_y = 46 ksi F_u = 58 ksi

Manual Table 2-3

Concrete w_c = 145 pcf f'_c = 5 ksi

$$E_c = w^{1.5}\sqrt{f'_c} = (145 \text{ pcf})^{1.5}\sqrt{5 \text{ ksi}} = 3{,}900 \text{ ksi}$$

Section I2.1b

Geometric Properties:

HSS 10×6×⅜ A = 10.4 in.^{2} I_y = 61.8 in.^{4} h/t = 25.7 Manual Table 1-11
t = 0.375 in. t_{des} = 0.349 in. B= 6.00 in.
H = 10.0 in.

Concrete:
The concrete area is calculated as follows:

$r = 1.5t_{des}$ = 1.5(0.349 in.) = 0.524 in. (outside radius) Section B4.2(d)

$b = B - 2r$ = 6.00 in. – 2(0.524 in.) = 4.95 in.

$h = H - 2r$ = 10.0 in. – 2(0.524 in.) = 8.95 in.

$r - t$ = 0.524 in. – 0.349 in. = 0.175 in.

$$A_c = bh + \pi(r-t)^2 + 2b(r-t) + 2h(r-t)$$

$$= (4.95\text{ in.})(8.95\text{ in.}) + \pi(0.175\text{ in.})^2 + 2(4.95\text{ in.})(0.175\text{ in.}) + 2(8.95\text{in.})(0.175\text{ in.})$$

$$= 49.3\text{ in.}^2$$

$$I_c = \frac{b_1 h_1^3}{12} + \frac{2(b_2)(h_2)^3}{12} + 2(r-t)^4\left(\frac{\pi}{8} - \frac{8}{9\pi}\right) + 2\left(\frac{\pi(r-t)^2}{2}\right)\left(\frac{h_2}{2} + \frac{4(r-t)}{3\pi}\right)^2$$

For this shape, buckling will take place about the weak axis; thus,

$$h_1 = B - 2t = 6.00\text{ in.} - 2(0.349\text{ in.}) = 5.30\text{ in.}$$
$$b_1 = H - 2r = 10.0\text{ in.} - 2(0.524\text{ in.}) = 8.95\text{ in.}$$
$$h_2 = B - 2r = 6.00\text{ in.} - 2(0.524\text{ in.}) = 4.95\text{ in.}$$
$$b_2 = r - t = 0.524\text{ in.} - 0.349\text{ in.} = 0.175\text{ in.}$$

$$I_c = \frac{(8.95\text{ in.})(5.30\text{ in.})^3}{12} + \frac{2(0.175\text{ in.})(4.95\text{ in.})^3}{12} + 2(0.175\text{ in.})^4\left(\frac{\pi}{8} - \frac{8}{9\pi}\right)$$
$$+\ 2\left(\frac{\pi(0.175\text{ in.})^2}{2}\right)\left(\frac{4.95\text{ in.}}{2} + \frac{4(0.175\text{ in.})}{3\pi}\right)^2 = 115\text{ in.}^4$$

Limitations applicable to rectangular sections:

1) Normal weight concrete 10 ksi ≥ f'_c ≥ 3 ksi f'_c = 5 ksi **o.k.** Section I1.2

2) $F_y \le 75$ ksi $F_y = 46$ ksi **o.k.**

3) The cross-sectional area of the steel HSS shall comprise at least one percent of the total composite cross section. Section I2.2a

   10.4 in.^{2} > (0.01)(49.3 in.^{2} + 10.4 in.^{2}) = 0.597 in.^{2} **o.k.**

4) The maximum width-thickness ratio for a rectangular HSS used as a composite column shall be equal to $2.26\sqrt{E/F_y}$. Section

$$h/t = 25.7 \le 2.26\sqrt{E/F_y} = 2.26\sqrt{29{,}000\text{ ksi}/46\text{ ksi}} = 56.7 \quad \textbf{o.k.}$$ I2.2a

Calculate the available compressive strength

C_2 = 0.85 for rectangular sections — Sec. I2.2b

$$P_o = A_sF_y + A_{sr}F_{yr} + C_2A_cf'_c$$
$$= (10.4\text{ in.}^2)(46\text{ ksi}) + 0.0 + 0.85(49.3\text{ in.}^2)(5\text{ ksi}) = 688\text{ kips}$$ Eqn. I2-13

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) = 0.6 + 2\left(\frac{10.4\text{ in.}^2}{49.3\text{ in.}^2 + 10.4\text{ in.}^2}\right) = 0.948 > 0.90$$ Eqn. I2-15

Therefore, use C_3 = 0.90

$$EI_{eff} = E_sI_s + E_sI_{sr} + C_3E_cI_c$$
$$= (29{,}000\text{ ksi})(61.8\text{ in.}^4) + 0.0 + (0.90)(3{,}900\text{ ksi})(115\text{ in.}^4)$$
$$= 2{,}200{,}000\text{ kip-in.}^2$$ Eqn. I2-14

User note: K value is determined from Chapter C and for this case K = 1.0.

$$P_e = \pi^2\left(EI_{eff}\right)/\left(KL\right)^2 = \pi^2\left(2{,}200{,}000\text{ kip-in.}^2\right)/\left[(1.0)(14\text{ ft})(12\text{ in./ft})\right]^2 = 769\text{ kips}$$ Eqn. I2-5

$P_e = 769\text{ kips} \ge 0.44P_o = (0.44)(688\text{ kips}) = 303\text{ kips}$

Therefore, use Eqn. I2-2 to solve for P_n

$$\frac{P_o}{P_e} = \frac{688\text{ kips}}{769\text{ kips}} = 0.895$$ Section I2.1b

$$P_n = P_o\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] = (688\text{ kips})\left[0.658^{(0.895)}\right] = 473\text{ kips}$$ Eqn. I2-2

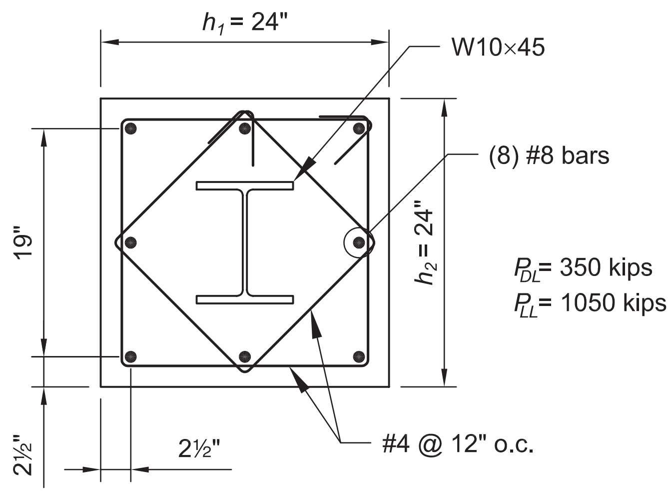
LRFD	ASD
$\phi_c = 0.75$ $\phi_cP_n = 0.75(473\text{ kips}) = 355\text{ kips}$ 355 kips > 336 kips **o.k.**	$\Omega_c = 2.00$ $P_n/\Omega_c = 473\text{ kips} / 2.00 = 237\text{ kips}$ 237 kips > 224 kips **o.k.**

Section I2.1b

Example I-3a Encased Composite Column in Axial Compression

Given:

Determine if a 14 ft tall W10×45 steel section encased in a 24 in.×24in. concrete column with f'_c = 5 ksi, is adequate to support a dead load of 350 kips and a live load of 1050 kips in axial compression. The concrete section has 8-#8 longitudinal reinforcing bars and #4 transverse ties @ 12 in. o.c. The column is pinned at both ends and the load is applied directly to the concrete encasement.



Solution:

Material Properties:

Column ASTM A992	F_y = 50 ksi	F_u = 65 ksi		Manual Table 2-3
Concrete	w_c = 145 pcf	f'_c = 5 ksi	E_c = 3,900 ksi	ACI-318
Reinforcement	F_{yr} = 60 ksi			

Geometric Properties:

W10×45	A_s = 13.3 in.^{2}	I_y = 53.4 in.^{4}	Manual Table 1-1

Reinforcing steel:

A_{sr} = 6.32 in.^{2} (the area of 1-#8 bar is 0.79 in.^{2}, per ACI)

$$I_{sr} = \frac{\pi r^4}{4} + Ad^2 = 8\frac{\pi(0.50\text{ in.})^4}{4} + 6\left(0.79\text{ in.}^2\right)\left(9.50\text{ in.}\right)^2 = 428\text{ in.}^4$$

Concrete:

$$A_c = A_g - A_s - A_{sr} = 576\text{ in.}^2 - 13.3\text{ in.}^2 - 6.32\text{ in.}^2 = 556\text{ in.}^2$$

$$I_c = I_g - I_s - I_{sr} = \frac{(24.0\text{ in.})^4}{12} - 53.4\text{ in.}^4 - 428\text{ in.}^4 = 27{,}200\text{ in.}^4$$

Note: The weak axis moment of inertia is used in the slenderness check.

Limitations:

1) Normal weight concrete 10 ksi $\geq f'_c \geq$ 3 ksi $\quad f'_c$ = 5 ksi **o.k.** — Section I1.2

2) $F_{yst} \leq 75$ ksi $\quad F_y = 50$ ksi $\quad F_{yr} = 60$ ksi **o.k.**

3) The cross-sectional area of the steel core shall comprise at least one percent of the total composite cross section. — Section I2.1a

   13.3 in.^{2} > (0.01)(576 in.^{2}) = 5.76 in.^{2} **o.k.**

4) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum transverse reinforcement shall be at least 0.009 in.^{2} per inch of tie spacing.

   2(0.20 in.^{2})/12 in. = 0.0333 in.^{2}/in. > 0.009 in.^{2}/in. conservatively, using only the perimeter ties. **o.k.**

5) The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004.

   Eqn. I2-1

$$\rho_{sr} = \frac{A_{sr}}{A_g} = \frac{6.32\text{ in.}^2}{576\text{ in.}^2} = 0.011 > 0.004 \quad \textbf{o.k.}$$

Calculate the total required compressive strength

LRFD	ASD
P_u = 1.2(350 kips) + 1.6(1,050 kips) = 2100 kips	P_a = 350 kips + 1,050 kips = 1400 kips

Calculate the available compressive strength

$$P_o = A_sF_y + A_{sr}F_{yr} + 0.85A_c f'_c$$
$$= (13.3\text{ in.}^2)(50\text{ ksi}) + (6.32\text{ in.}^2)(60\text{ ksi}) + 0.85(556\text{ in.}^2)(5\text{ ksi}) = 3{,}410\text{ kips}$$
Eqn. I2-4

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) = 0.1 + 2\left(\frac{13.3\text{ in.}^2}{556\text{ in.}^2 + 13.3\text{ in.}^2}\right) = 0.147 \leq 0.3$$
Eqn. I2-7

$$EI_{eff} = E_sI_s + 0.5E_sI_{sr} + C_1E_cI_c$$
$$= (29{,}000\text{ ksi})(53.4\text{ in.}^4) + 0.5(29{,}000\text{ ksi})(428\text{ in.}^4) + (0.147)(3{,}900\text{ ksi})(27{,}200\text{ in.}^4)$$
$$= 23{,}300{,}000\text{ kip-in.}^2$$
Eqn. I2-6

User note: The K value is determined from Chapter C and for this case K = 1.0.

$$P_e = \pi^2(EI_{eff})/(KL)^2 = \pi^2(23{,}300{,}000\text{ kip-in.}^2) / [(1.0)(14\text{ ft})(12\text{ in./ft})]^2 = 8{,}150\text{ kips}$$
Eqn. I2-5

P_e = 8,150 kips $\geq 0.44P_o$ = 0.44(3,410 kips) = 1,500 kips; therefore, use Eqn. I2-2 to determine P_n

$$\frac{P_o}{P_e} = \frac{3{,}410\text{ kips}}{8{,}150\text{ kips}} = 0.418$$
Section I2.1b

$$P_n = P_o\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] = (3{,}410\text{ kips})\left[0.658^{(0.418)}\right] = 2{,}860\text{ kips}$$ Eqn. I2-2

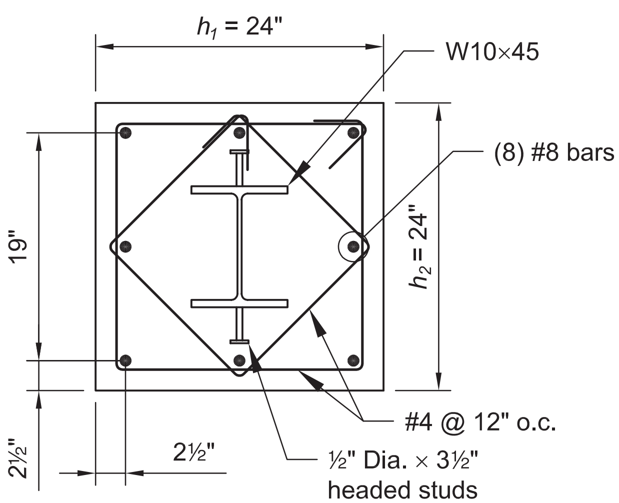
LRFD	ASD
$\phi_c = 0.75$ $\phi_c P_n = 0.75(2{,}860\text{kips}) = 2{,}150\text{ kips}$ 2,150 kips > 2,100 kips **o.k.**	$\Omega_c = 2.00$ $P_n / \Omega_c = 2{,}860\text{ kips} / 2.00 = 1{,}430\text{ kips}$ 1,440 kips > 1,400 kips **o.k.**

Section I2.1b

Because the entire load in the column was applied directly to the concrete, accommodations must be made to transfer an appropriate portion of the axial force to the steel column. This force is transferred as a shear force at the interface between the two materials.

Example I-3b Encased Composite Column in Axial Compression

Determine the number and spacing of ½-in. diameter headed shear studs to transfer the axial force in the composite column of Example I-3a.



Solution:

Material Properties:

Concrete	$w_c = 145$ pcf	$f'_c = 5$ ksi	$E_c = 3{,}900$ ksi
Shear Studs	$F_u = 65$ ksi		

Geometric Properties:

W10×45	$A_s = 13.3\text{ in.}^2$	$d = 10.1$ in.
Shear Studs	$A_{sc} = 0.196\text{ in.}^2$	
Concrete	$d_c = 24$ in.	

Manual Table 1-1

Calculate the shear force to be transferred

LRFD	ASD

$V=\dfrac{P_u}{\phi_c}=\dfrac{2{,}100}{0.75}=2{,}800\text{ kips}$	$V=P_a\Omega_c=1{,}400(2)=2{,}800\text{ kips}$

$V'=V\left(A_sF_y/P_o\right)=2{,}800\,\text{kips}\left[\left(13.3\text{ in.}^2\right)\left(50\text{ ksi}\right)/3{,}410\text{ kips}\right]=546\,\text{kips}$ Eqn. I2-10

Calculate the nominal strength of one ½ in. diameter shear stud connector

$Q_n = 0.5A_{sc}\sqrt{f_c'E_c} \leq A_{sc}F_u$ Eqn. I2-12

$0.5A_{sc}\sqrt{f_c'E_c} = 0.5(0.196\text{ in.}^2)\sqrt{(5\,\text{ksi})(3{,}900\,\text{ksi})} = 13.7\text{ kips}$

$A_{sc}F_u = (0.196\text{ in.}^2)(65\text{ ksi}) = 12.7\text{ kips}$

Therefore, use 12.7 kips.

Calculate the number of shear studs required to transfer the total force, V′

$V'/Q_n = 546\text{ kips} / 12.7\text{ kips} = 43$

An even number of studs are required to be placed symmetrically on two faces. Therefore use 22 studs minimum per flange

Determine the spacing for the shear studs

The maximum stud spacing is 16 in. Section I2.1f

The available column length is 14 ft (12 in./ft) = 168 in.

The maximum spacing = 168 in./(22+1) = 7.30 in.

Therefore, on the flanges, use single studs @ 7 in.

Stud placement is to start 10.5 in. from one end.

Determine the available clear distance for the studs for the flanges

$$\left(\frac{d_c-d}{2}\right)-3.00\,\text{in.}=\left(\frac{24.0\text{ in.}-10.1\text{ in.}}{2}\right)-3.00\,\text{in.}=3.95\text{ in.}$$

Therefore use 3 ½ in. long shear studs on each of the flanges.

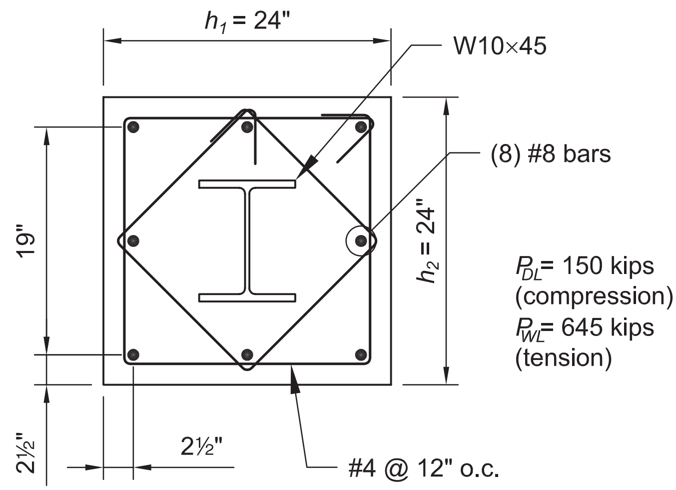
Note: The subtraction of 3 in. is to ensure sufficient cover.

Summary: Use (22) ½-in. diameter x 3 ½ in. shear stud connectors as shown on each flange, spaced @ 7 in.

Example I-4 Encased Composite Column in Axial Tension

Given:

Determine if the composite column in Example I-3 can support a dead load compression of 150 kips and a wind load tension of 645 kips. The column is pinned at both ends. The steel W-shape and the reinforcing are developed at each end in order to transfer any tensile force.



Solution:

Material Properties:

Column ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi		Manual Table 2-3
Concrete	$w_c = 145$ pcf	$f'_c = 5$ ksi	$E_c = 3{,}900$ ksi	ACI 318
Reinforcement	$F_{yr} = 60$ ksi			

Geometric Properties:

W10×45:	$A_s = 13.3$ in.^{2}	$I_y = 53.4$ in.^{4}	Manual Table 1-1
Reinforcing steel:	$A_{sr} = 6.32$ in.^{2}	$I_{sr} = 428$ in.^{4}	
Concrete:	$A_c = 556$ in.^{2}	$I_c = 27{,}200$ in.^{4}	

Limitations: Section I1.2

1) Normal weight concrete 10 ksi $\geq f'_c \geq$ 3 ksi; $f'_c = 5$ ksi **o.k.**

2) $F_{yst} \leq 75$ ksi; $F_y = 50$ ksi $F_{yr} = 60$ ksi **o.k.**

3) The cross-sectional area of the steel core shall comprise at least one percent of the total composite cross section. Section I2.1a

   13.3 in.^{2} > (0.01)(576 in.^{2}) = 5.76 in.^{2} **o.k.**

4) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum transverse reinforcement shall be at least 0.009 in.^{2} per inch of tie spacing.

$2(0.20 \text{ in.}^2)/12 \text{ in.} = 0.0333 \text{ in.}^2/\text{in.} > 0.009 \text{ in.}^2/\text{in.}$, conservatively using only the perimeter ties **o.k.**

5) The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004.

$$\rho_{sr} = \frac{A_{sr}}{A_g} = \frac{6.32 \text{ in.}^2}{576 \text{ in.}^2} = 0.011 > 0.004 \quad \textbf{o.k.}$$ Eqn. I2-1

Calculate the required tensile strength

LRFD	ASD
P_u =0.9(-150 kips) + 1.6(645 kips) = 897 kips	P_a = 0.6(-150 kips) + 645 kips = 555 kips

Calculate the available tensile strength

$P_n = P_t = A_sF_y + A_{sr}F_{yr}$ Eqn. I2-8
$= (13.3 \text{ in.}^2)(50 \text{ ksi}) + (6.32 \text{ in.}^2)(60 \text{ ksi}) = 1040 \text{ kips}$

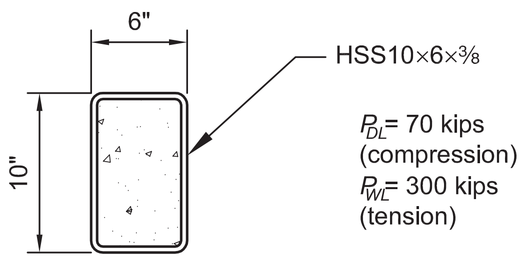
LRFD	ASD
$\phi_t = 0.90$ $\phi_tP_n = 0.90(1{,}040 \text{ kips}) = 936 \text{ kips}$ 936 kips > 897 kips **o.k.**	$\Omega_t = 1.67$ $P_n / \Omega_t = 1{,}040 \text{ kips} / 1.67 = 623 \text{ kips}$ 623 kips > 555 kips **o.k.**

Section I2.1c

Example I-5 Filled Composite Column in Axial Tension

Given:

Determine if the filled composite column in Example I-2 is adequate to support a dead load compression of 70 kips and a wind load tension of 300 kips. The column is pinned at both ends and all of the load is transferred by the base and cap plates.



Solution:

Material Properties:

HSS10×6×⅜	ASTM A500 Gr. B	F_y= 46 ksi	F_u = 58 ksi	Manual Table 2-3
Concrete	w_c = 145 pcf	f'_c = 5 ksi	E_c = 3,900 ksi	ACI-318

Geometric Properties:

HSS10×6×⅜ A = 10.4 in.^{2} — Manual Table 1-11

Note: See Example I-2 for a check of the limitations given in Section I1.2 and Section I2.2a.

Calculate the required tensile strength

LRFD	ASD
P_u = 0.9(-70 kips) + 1.6(300 kips) = 417 kips	P_a = 0.6(-70 kips) + 300 kips = 258 kips

Determine the available tensile strength

$P_n = A_sF_y + A_{sr}F_{yr}$ — Eqn. I2-16
= (10.4 in.^{2})(46 ksi) + 0.0 = 478 kips

LRFD	ASD
ϕ_t = 0.90 ϕ_tP_n =0.90(478 kips) = 430 kips 430 kips > 417 kips **o.k.**	Ω_t = 1.67 P_n / Ω_t = 478 kips / 1.67 = 286 kips 286 kips >258 kips **o.k.**

Section I2.2c

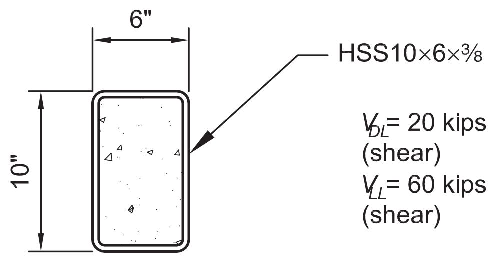
Note: The concrete is not considered to contribute to the available tensile strength, therefore no shear transfer between the encasing steel and the concrete fill is required for this load case.

Example I-6 Filled Composite Member Design for Shear

Given:

This composite member ASTM A500 Gr. B with 5 ksi normal weight concrete has a dead load end shear of 20 kips and a live load end shear of 60 kips. Verify that this end shear can be safely carried using both LRFD and ASD analysis.

Note: According to Specification Section I2.2d, shear strength is calculated based on either the shear strength of the steel section alone as specified in Specification Chapter G or the shear strength of the reinforced concrete portion alone. Section I2.2d



Solution:

Material Properties:

HSS 10×6×⅜ ASTM A500 Gr. B $F_y = 46$ ksi $F_u = 58$ ksi Manual Table 2-3

Concrete $w_c = 145$ pcf $f'_c = 5$ ksi ACI-318

Geometric Properties:

HSS 10×6×⅜ $d = 10$ in. $t = t_w = 0.349$ in. Manual Table 1-11

Note: See example I-2 for a check of the limitations indicated in Section I1.2 and Section I2.2a.

Calculate the required shear strength

LRFD	ASD
$V_u = 1.2(20 \text{ kips}) + 1.6(60 \text{ kips})$ $= 120$ kips	$V_a = 20 \text{ kips} + 60 \text{ kips}$ $= 80$ kips

Calculate the available shear strength

The available shear area for rectangular HSS and box members is $2ht$, where h is the outside dimension minus the top and bottom outside radii. If the exact corner radius is not known, h shall be taken as the outside dimension minus three times the design wall thickness. Section G5

Note: $k_V = 5.0$ for all sizes of HSS rectangular or box members listed in the Manual. Most HSS sections listed in the Manual have a $C_v = 1.0$ at $F_y \leq 46$ ksi. Section G2 & G5

Calculate slenderness

$h = d - (3t_w) = 10 \text{ in.} - (3)(0.349 \text{ in.}) = 8.95 \text{ in.}$ Section G5

$h/t_w = 8.95 \text{ in.}/0.349 \text{ in.} = 25.6$

$1.10\sqrt{k_y E/F_y} = 1.10\sqrt{5(29{,}000 \text{ ksi}/46 \text{ ksi})} = 61.8$ Section G5

$25.6 \leq 61.8$; therefore, $C_v = 1.0$ Eqn. G2-3

Calculate A_w

$A_w = 2ht_w = 2(8.95 \text{ in.})(0.349 \text{ in.}) = 6.25 \text{ in.}^2$

Calculate V_n

$V_n = 0.6F_yA_wC_v = 0.6(46 \text{ ksi})(6.25 \text{ in.}^2)(1.0) = 173 \text{ kips}$ Eqn. G2-1

LRFD	ASD
$\phi_v = 0.90$ $V_u = \phi_v V_n = 0.90(173 \text{ kips}) = 156 \text{ kips}$ $156 \text{ kips} \geq 120 \text{ kips}$ **o.k.**	$\Omega_v = 1.67$ $V_a = V_n/\Omega_v = 173 \text{ kips} / 1.67 = 104 \text{ kips}$ $104 \text{ kips} \geq 80 \text{ kips}$ **o.k.**

Section G1

The available shear strength of the steel section alone has been shown to be sufficient, but the available shear strength of the reinforced concrete portion alone has been calculated below for the purpose of demonstration in this example. Section I2.2d

Note: The shear strength of reinforced concrete may be determined by ACI 318, Chapter 11. Since there is no reinforcement in this case, ACI 318 Chapter 11 does not apply, but ACI 318 Chapter 22 does apply.

$\phi V_n \geq V_u$ ACI Eqn. 22-8

$\phi = 0.55$ ACI Section 9.3.5

$V_n = 4/3\sqrt{f_c'}b_w h$ ACI Eqn. 22-9

$b_w = 6 \text{ in.} - 2(0.375 \text{ in.}) = 5.25 \text{ in.}$

$h = 10 \text{ in.} - 2(0.375 \text{ in.}) = 9.25 \text{ in.}$

$$V_n = 4/3\sqrt{5{,}000 \text{ psi}}\left(\frac{1 \text{ ksi}}{1{,}000 \text{ psi}}\right)(5.25 \text{ in.})(9.25 \text{ in.}) = 4.58 \text{ kips}$$

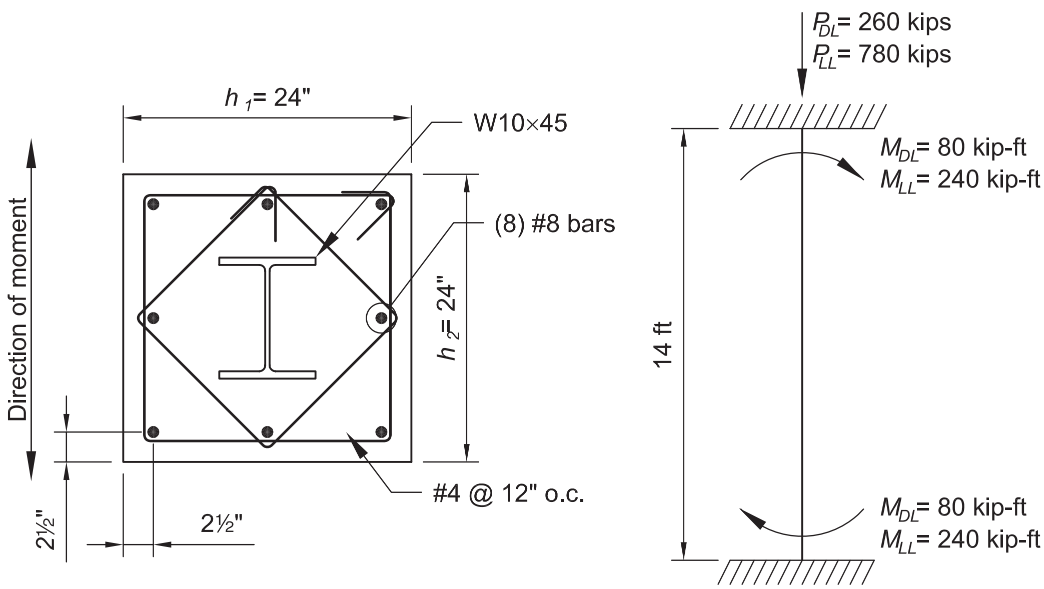
$\phi V_n = 0.55(4.58 \text{ kips}) = 2.52 \text{ kips} < 120 \text{ kips}$

Use the available shear strength provided by the steel section alone.

Example I-7 Combined Axial and Flexural Strength

Given:

A 14 ft long composite column consists of a W10×45 steel section encased in a 24 in.×24in. concrete section. The concrete is reinforced with 8-#8 longitudinal bars and #4 transverse ties @ 12 in. o.c., as illustrated below. Determine if the member has sufficient available strength to support an axial dead load of 260 kips and an axial live load of 780 kips in compression, as well as a dead load moment of 80 kip-ft and a live load moment of 240 kip-ft. The load is applied directly to the concrete encasement.



Solution:

Material Properties:

W10×45	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Concrete	w_c = 145 pcf	f'_c = 5 ksi	E_c = 3,900 ksi	ACI-318
Reinforcement	F_{yr} = 60 ksi			

Geometric Properties:

W10×45	A_s = 13.3 in.^{2}	I_y = 53.4 in.^{4}	Z_x =54.9 in.^{3}	d = 10.1 in.	Manual Table 1-1
	b_f = 8.02 in.	t_w = 0.350 in.	t_f= 0.620 in.		
Reinforcement	A_{sr} = 6.32 in.^{2}	A_{srs} = 1.58 in.^{2}	I_{sr} = 428 in.^{4}		
Concrete	A_c = 556 in.^{2}	I_{cy} = 27,200 in.^{4}			

Limitations:

Section I1.2

1) Normal weight concrete 10 ksi ≥ f'_c ≥ 3 ksi; f'_c = 5 ksi **o.k.**

2) F_y and F_{yr} ≤ 75 ksi F_y = 50 ksi and F_{yr} = 60 ksi **o.k.**

Section I2.1a

3) The cross-sectional area of the steel core shall comprise at least one percent of the total composite cross section.

13.3 in.^{2} > (0.01)(576 in.^{2}) = 5.76 in.^{2} **o.k.**

4) Concrete encasement of steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum transverse reinforcement shall be at least 0.009 in.^{2} per inch of tie spacing.

   2(0.20 in.^{2})12 in. = 0.0333 in.^{2}/in. > 0.009 in.^{2}/in., conservatively considering only the perimeter ties **o.k.**

5) The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004.

$$\rho_{sr} = \frac{A_{sr}}{A_g} = \frac{6.32\text{in.}^2}{576\text{in.}^2} = 0.011 > 0.004 \quad \textbf{o.k.}$$ Eqn. I2-1

Calculate the required strengths

LRFD	ASD
P_u = 1.2(260 kips) + 1.6(780 kips) = 1560 kips M_{ux} = 1.2(80 kip-ft) + 1.6(240 kip-ft) = 480 kip-ft	P_a = 260 kips + 780 kips = 1040 kips M_{ax} = 80 kip-ft + 240 kip-ft = 320 kip-ft

The available strength of the composite section subjected to combined axial and flexural loads is determined by constructing an interaction curve. The curve is generated by calculating the available strength of the section at a series of points on the interaction curve and reducing the strength for slenderness effects and multiplying by the resistance factor for LRFD or dividing by the safety factor for ASD. The defining equations of the interaction curve were given earlier in Figure I-1a, and will be used to construct the curve illustrated in Commentary Figure C-I4.1, and repeated here.

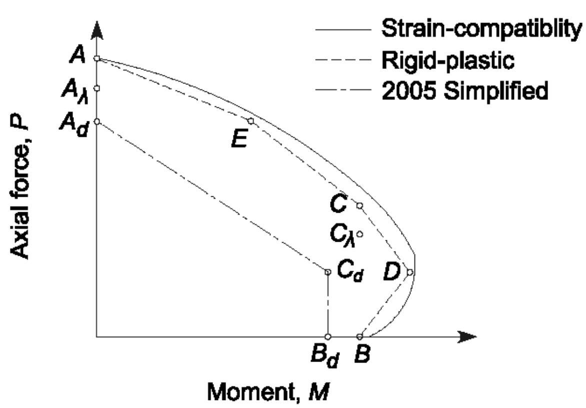


Fig. C-I4.1 Interaction diagram for composite beam-column design

Determine the available axial compressive strength and flexural strength

Point A (M_{nA} = 0)

$P_o = A_sF_y + A_{sr}F_{yr} + 0.85A_c f'_c$
$= (13.3 \text{ in.}^2)(50 \text{ ksi}) + (6.32 \text{ in.}^2)(60 \text{ ksi}) + 0.85(556 \text{ in.}^2)(5 \text{ ksi}) = 3{,}410 \text{ kips}$ Eqn. I2-4

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) = 0.1 + 2\left(\frac{13.3\text{in.}^2}{556\text{in.}^2 + 13.3\text{in.}^2}\right) = 0.147 \le 0.3$$ Eqn. I2-7

$EI_{eff} = E_sI_s + 0.5E_sI_{sr} + C_1E_cI_c$ Eqn. I2-6
$= (29{,}000 \text{ ksi})(53.4 \text{ in.}^4) + 0.5(29{,}000 \text{ ksi})(428 \text{ in.}^4) + (0.147)(3{,}900 \text{ ksi})(27{,}200 \text{ in.}^4)$
$= 23{,}300{,}000 \text{ kip-in.}^2$

Note: The K value is determined from Chapter C and for this case K = 1.0.

Eqn. I2-5

$P_e = \pi^2(EI_{eff})/(KL)^2 = \pi^2(23{,}300{,}000 \text{ kip-in.}^2) / [(1.0)(14 \text{ ft})(12\text{in./ft})]^2 = 8{,}150 \text{ kips}$

$P_e = 8{,}150 \text{ kips} \ge 0.44P_o = 0.44(3{,}410 \text{ kips}) = 1{,}500 \text{ kips}$; therefore, use Eqn. I2-2 to determine P_n

$$\frac{P_o}{P_e} = \frac{3{,}410 \text{ kips}}{8{,}150 \text{ kips}} = 0.418$$

$$P_{nA} = P_o\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] = (3{,}410 \text{ kips})\left[0.658^{(0.418)}\right] = 2{,}860 \text{ kips}$$ Eqn. I2-2

Point D

Commentary Figure C-I4.1 and Figure I-1a

$$P_D = \frac{0.85 f'_c A_c}{2} = \frac{(0.85)(5 \text{ ksi})(556 \text{ in.}^2)}{2} = 1{,}180 \text{ kips}$$

Z_s = full x-axis plastic section modulus of steel shape

$$Z_s = Z_x = 54.9 \text{ in.}^3$$

$$Z_r = (A_{sr} - A_{srs})\left(\frac{h_2}{2} - c\right) = (6.32 \text{ in.}^2 - 1.58 \text{ in.}^2)\left(\frac{24.0 \text{ in.}}{2} - 2.50 \text{ in.}\right) = 45.0 \text{ in.}^3$$

$$Z_c = \frac{h_1 h_2^2}{4} - Z_s - Z_r = \frac{(24.0 \text{ in.})(24.0 \text{ in.})^2}{4} - 54.9 \text{ in.}^3 - 45.0 \text{ in.}^3 = 3{,}360 \text{ in.}^3$$

$$M_{nD} = Z_sF_y + Z_rF_{yr} + \frac{Z_c}{2}(0.85 f'_c)$$

$$M_{nD} = (54.9 \text{ in.}^3)(50 \text{ ksi}) + (45.0 \text{ in.}^3)(60.0 \text{ ksi}) + \frac{3{,}360 \text{ in.}^3}{2}(0.85)(5 \text{ ksi})$$
$$= 12{,}600 \text{ kip-in. or } 1050 \text{ kip-ft}$$

$$P_{nD} = P_D\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] = (1{,}180 \text{ kips})\left[0.658^{(0.418)}\right] = 991 \text{ kips}$$ Eqn. I2-2 and Commentary I4

Point B (P_{nB} = 0)

For h_n within the flange, Figure I-1a

$$h_n = \frac{0.85 f_c'\left(A_c + A_s - db_f + A_{srs}\right) - 2F_y\left(A_s - db_f\right) - 2F_{yr}A_{srs}}{2\left[0.85 f_c'\left(h_1 - b_f\right) + 2F_y b_f\right]}$$

$$= \frac{\left[\begin{array}{l} 0.85\left(5\text{ ksi}\right)\left(556\text{ in.}^2 + 13.3\text{ in.}^2 - 10.1\text{ in.}(8.02\text{ in.}) + 1.58\text{ in.}^2\right) \\ -2\left(50\text{ ksi}\right)\left(13.3\text{ in.}^2 - 10.1\text{ in.}(8.02\text{ in.})\right) - 2\left(60\text{ ksi}\right)\left(1.58\text{ in.}^2\right) \end{array}\right]}{2\left[\left(0.85\right)\left(5\text{ ksi}\right)\left(24\text{ in.} - 8.02\text{ in.}\right) + 2(50\text{ ksi})(8.02\text{ in.})\right]} = 4.98\text{ in.}$$

$$\left(\frac{d}{2} - t_f\right) < h_n \le \left(\frac{d}{2}\right)$$

$$\frac{d}{2} - t_f = \frac{10.1\text{ in.}}{2} - 0.620\text{ in.} = 4.43\text{ in.}$$

$$\frac{d}{2} = \frac{10.1\text{ in.}}{2} = 5.05\text{ in}$$

4.43 in. < 4.98 in. ≤ 5.05 in. Therefore h_n is within the flange

$$Z_{sn} = Z_s - b_f\left(\frac{d}{2} - h_n\right)\left(\frac{d}{2} + h_n\right) = 54.9\text{ in.}^3 - 8.02\text{ in.}\left(\frac{10.1\text{ in.}}{2} - 4.98\text{ in.}\right)\left(\frac{10.1\text{ in.}}{2} + 4.98\text{ in.}\right)$$

$$= 49.3\text{ in.}^3$$

$$Z_{cn} = h_1 h_n^2 - Z_{sn} = \left(24\text{ in.}\right)\left(4.98\text{ in.}\right)^2 - 49.3\text{ in.}^3 = 546\text{ in.}^3$$

$$M_{nB} = M_D - Z_{sn}F_y - \frac{1}{2}Z_{cn}\left(0.85 f_c'\right)$$

$$= \left(12{,}600\text{ kip-in.}\right) - \left(49.3\text{ in.}^3\right)\left(50\text{ ksi}\right) - \frac{1}{2}\left(546\text{ in.}^3\right)\left(0.85\right)\left(5\text{ ksi}\right)$$

$$= 8{,}970\text{ kip-in. or } 748\text{ kip-ft}$$

Point C ($M_C = M_B$; $P_C = 0.85 f_c' A_c$)

$$P_C = 0.85 f_c' A_c = \left(0.85\right)\left(5\text{ ksi}\right)\left(556\text{ in.}^2\right) = 2{,}360\text{ kips}$$

$$M_{nC} = M_{nB} = 748\text{ kip-ft}$$

Eqn. I2-2 and Commentary Section I4

$$P_{nC} = P_C\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] = 2{,}360\text{ kips}\left[0.658^{0.418}\right] = 1{,}980\text{ kips}$$

Summary of nominal strengths including length effects

$$
\begin{aligned}
P_{nA} &= 2{,}860 \text{ kips} \\
M_{nxA} &= 0 \\
P_{nB} &= 0 \\
M_{nxB} &= 748 \text{ kip-ft} \\
P_{nC} &= 1{,}980 \text{ kips} \\
M_{nxC} &= 748 \text{ kip-ft} \\
P_{nD} &= 991 \text{ kips} \\
M_{nxD} &= 1{,}050 \text{ kip-ft}
\end{aligned}
$$

LRFD	ASD	
$\phi_c = 0.75 \quad \phi_b = 0.90$	$\Omega_c = 2.00 \quad \Omega_b = 1.67$	Section I4
$\phi_c P_{nA} = 2{,}150$ kips $\phi_b M_{nxA} = 0$ $\phi_c P_{nB} = 0$ $\phi_b M_{nxB} = 673$ kip-ft $\phi_c P_{nC} = 1{,}490$ kips $\phi_b M_{nxC} = 673$ kip-ft $\phi_c P_{nD} = 743$ kips $\phi_b M_{nxD} = 945$ kip-ft	$P_{nA} / \Omega_c = 1{,}430$ kips $M_{nxA} / \Omega_b = 0$ $P_{nB} / \Omega_c = 0$ $M_{nxB} / \Omega_b = 448$ kip-ft $P_{nC} / \Omega_c = 990$ kips $M_{nxC} / \Omega_b = 448$ kip-ft $P_{nD} / \Omega_c = 496$ kips $M_{nxD} / \Omega_b = 629$ kip-ft	
Interaction	**Interaction**	
$P_u \geq \phi_c P_{nC}$ $1{,}560 \text{ kips} > 1{,}490 \text{ kips}$; thus,	$P_a \geq \dfrac{P_{nC}}{\Omega_c}$ $1{,}040 \text{ kips} > 990 \text{ kips}$; thus,	Commentary Section I4
$\dfrac{P_u - \phi_c P_{nC}}{\phi_c P_{nA} - \phi_c P_{nC}} + \dfrac{M_{ux}}{\phi_b M_{nxC}} \leq 1.0$	$\dfrac{P_a - P_{nC}/\Omega_c}{P_{nA}/\Omega_c - P_{nC}/\Omega_c} + \dfrac{M_{ax}}{M_{axC}/\Omega_b} \leq 1.0$	
$\dfrac{1{,}560 \text{ kips} - 1{,}490 \text{ kips}}{2{,}150 \text{ kips} - 1{,}490 \text{ kips}} + \dfrac{480 \text{ kip-ft}}{673 \text{ kip-ft}} = 0.819$ $0.819 < 1.0$	$\dfrac{1{,}040 \text{ kips} - 990 \text{ kips}}{1{,}430 \text{ kips} - 990 \text{ kips}} + \dfrac{320 \text{ kip-ft}}{448 \text{ kip-ft}} = 0.828$ $0.828 < 1.0$	
Therefore, this column is adequate for the specified loading.	Therefore, this column is adequate for the specified loading.	

The shear connector requirements for this encased composite column are similar to that of Example I-3.

CHAPTER J
DESIGN OF CONNECTIONS

INTRODUCTION

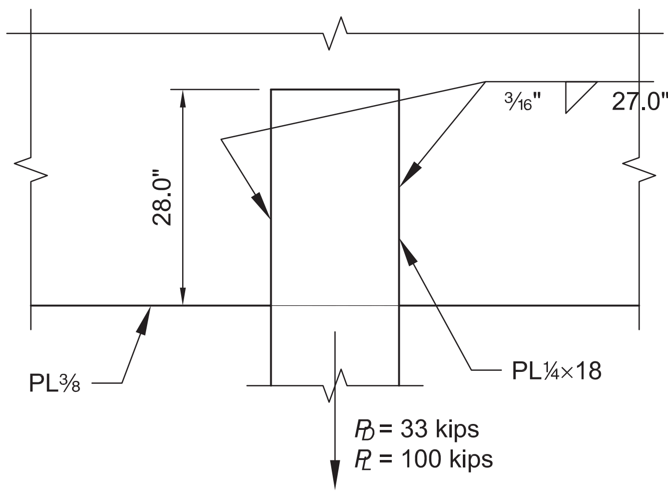
Chapter J of the Specification addresses the design and checking of connections. The chapter's primary focus is the design of welded and bolted connections. Design requirements for fillers, splices, column bases, concentrated forces, anchors rods, and other threaded parts are also covered. Special requirements for connections subject to fatigue are not covered in this chapter.

Example J.1 Fillet Weld in Longitudinal Shear

Given:

A ¼ in.×18-in. wide plate is fillet welded to a ⅜-in. plate. Assume that the plates are ASTM A572 Grade 50 and have been properly sized. Assume F_{EXX} = 70 ksi. Note that the plates would normally be specified as ASTM A36, but F_y = 50 ksi plate has been used here to demonstrate the requirements for long welds.

Size the welds for the loads shown.



Solution:

Determine the maximum weld size

Because the thickness of the overlapping plate is ¼ in., the maximum fillet weld size that can be used without special notation per Section J2.2b, is a 3⁄16-in. fillet weld. A 3⁄16-in. fillet weld can be deposited in the flat or horizontal position in a single pass (true up to 5⁄16-in).

Note that the minimum size of fillet weld, based on a material thickness of ¼ in. is ⅛ in. Specification Table J2.4

Determine the required strength

LRFD	ASD
P_u = 1.2(33 kips) + 1.6(100 kips) = 200 kips	P_a = 33 kips + 100 kips = 133 kips

Calculate the nominal strength per inch of a 3⁄16-in. weld

Eqn. J2-4

$$R_n = (F_w)(A_w)$$
$$=(0.60\ F_{exx})(A_w)$$
$$= (0.60)(70\text{ ksi})\left(0.188\text{ in.}/\sqrt{2}\right)$$
$$= 5.58\text{ kips / in.}$$

Determine the length of weld required

LRFD	ASD
$\dfrac{P_u}{\phi R_n} = \dfrac{200 \text{ kips}}{0.75\left(5.58 \text{ kips/in.}\right)} = 47.8 \text{ in.}$ or 24 in. of weld on each side	$\dfrac{P_a \Omega}{R_n} = \dfrac{133 \text{ kips}\left(2.00\right)}{5.58 \text{ kips/in.}} = 47.7 \text{ in.}$ or 24 in. of weld on each side.

Note that for longitudinal fillet welds used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. Section J2.2b

$24 \text{ in.} \geq 18 \text{ in.}$ **o.k.**

Check the weld for length to weld size ratio, since this is an end loaded fillet weld. Section J2.2b

$\dfrac{L}{w} = \dfrac{24 \text{ in.}}{0.188 \text{ in.}} = 128 > 100$; therefore, Specification Equation J2-1 must be applied, and the length of weld increased, since the resulting β will reduce the available strength below the required strength.

Try a weld length of 27 in.

The new length to weld size ratio is 27 in. /0.188 in. = 144

For this ratio

$\beta = 1.2 - 0.002(L/w) \leq 1.0$;
$\quad = 1.2 - 0.002(144) = 0.912$ Eqn. J2-1

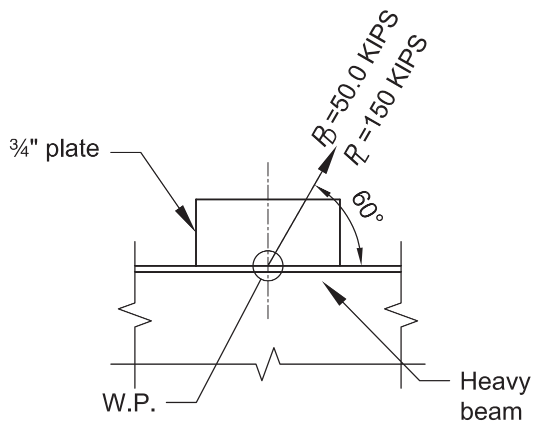
Recheck the weld at its reduced strength

LRFD	ASD
$\phi R_n = \left(0.912\right)\left(0.75\right)\left(5.58 \text{ kips/in.}\right)\left(54.0 \text{ in.}\right)$ $= 206 \text{ kips} > P_u = 200 \text{ kips}$ **o.k.** Therefore, use 27 in. of weld on each side	$\dfrac{R_n}{\Omega} = \dfrac{(0.912)\left(5.58 \text{ kips/in.}\right)\left(54.0 \text{ in.}\right)}{2.00}$ $= 137 \text{ kips} > P_a = 133 \text{ kips}$ **o.k.** Therefore, use 27 in. of weld on each side

Example J.2 Fillet Weld Loaded at an Angle

Given:

Design a fillet weld at the edge of a gusset plate to carry a force of 50 kips due to dead load and a force of 150 kips due to live load, at an angle of 60 degrees relative to the weld. Assume the beam and the gusset plate thickness and length have been properly sized.



Solution:

Calculate the required strength

LRFD	ASD
P_u = 1.2(50 kips) + 1.6(150 kips) = 300 kips	P_a = 50 kips + 150 kips = 200 kips

Assume a 5⁄16-in. fillet weld is used on each side.

Note that the minimum size of fillet weld, based on a material thickness of ¾ in. is ¼ in. Specification Table J2.4

Calculate the nominal shear strength of a 5⁄16-in. fillet weld per inch of length

$$A_w = \frac{0.313\text{ in.}}{\sqrt{2}} = 0.221\text{ in.}$$

Eqn. J2-5

$$\begin{aligned} F_w &= 0.60F_{exx}\left(1.0 + 0.5\sin^{1.5}\theta\right) \\ &= (0.60)(70\text{ ksi})\left(1.0 + 0.5\sin^{1.5}60^\circ\right) \\ &= 58.9\text{ ksi} \end{aligned}$$

Eqn. J2-4

$$\begin{aligned} R_n &= F_w A_w \\ &= (58.9\text{ ksi})(0.221\text{ in.}) = 13.0\text{ kip/in.} \end{aligned}$$

Calculate the available strength per inch of weld length

LRFD	ASD
ϕR_n = (0.75)(13.0 kip/in.) = 9.75 kip/in. For 2 sides ϕR_n = 2(9.75 kip/in.) = 19.5 kip/in.	R_n/Ω = (13.0 kip/in.)/(2.00) = 6.50 kip/in. For 2 sides R_n/Ω = (2)(13.0 kip/in.) = 13.0 kip/in.

Find the required length of weld

LRFD	ASD
300 kips / 19.5 kip/in. = 15.4 in. Use 16 in. **o.k.**	200 kips / 13.0 kip/in. = 15.4 in. Use 16 in. **o.k.**

Example J.3 Combined Tension and Shear in Bearing Type Connections

Given:

A ¾-in. diameter, ASTM A325-N bolt is subjected to a tension force of 3.5 kips due to dead load and 12 kips due to live load, and a shear force of 1.33 kips due to dead load and 4 kips due to live load.

Check the combined stresses according to the Equations J3-3a and J3-3b.

Solution:

Calculate the required tensile and shear strengths

LRFD	ASD
Tension T_u = 1.2(3.50 kips) +1.6(12.0 kips) = 23.4 kips	Tension T_a = 3.50 kips + 12.0 kips = 15.5 kips
Shear V_u = 1.2(1.33 kips) + 1.6(4.00 kips) = 8.00 kips	Shear V_a= 1.33 kips + 4.00 kips = 5.33 kips

Calculate the available tensile strength

LRFD	ASD	
Calculate f_v	*Calculate f_v*	
8.00 kips / 0.442 in.^{2} = 18.1 ksi $\leq \phi F_{nv}$	5.33 kips / 0.442 in.^{2} = 12.1 ksi $\leq F_{nv} / \Omega$	
Check combined tension and shear	*Check combined tension and shear*	
$F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}$	$F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt}$	
F_{nt} = 90 ksi, F_{nv} = 48 ksi	F_{nt} = 90 ksi, F_{nv} = 48 ksi	
$F'_{nt} = 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}}{0.75(48 \text{ ksi})}(18.1 \text{ ksi})$ $= 71.8 \text{ ksi} < 90 \text{ ksi}$	$F'_{nt} = 1.3(90 \text{ ksi}) - \dfrac{2.00(90 \text{ ksi})}{48 \text{ ksi}}(12.1 \text{ ksi})$ $= 71.6 \text{ ksi} < 90 \text{ ksi}$	Eq. J3-3a and J3-3b Table J3.2
$R_n = F'_{nt} A_b = 71.8 \text{ ksi}(0.442 \text{ in.}^2)$ $= 31.7$ kips	$R_n = F'_{nt} A_b = 71.6 \text{ ksi}(0.442 \text{ in.}^2)$ $= 31.6$ kips	
For combined tension and shear ϕ = 0.75	For combined tension and shear Ω = 2.00	Eqn. J3-2
Design tensile strength	*Allowable tensile strength*	Section J3.7
$\phi R_n = 0.75(31.7 \text{ kips})$ = 23.8 kips > 23.4 kips **o.k.**	$R_n / \Omega = 31.6 \text{ kips}/2.00$ = 15.8 kips > 15.5 kips **o.k.**	

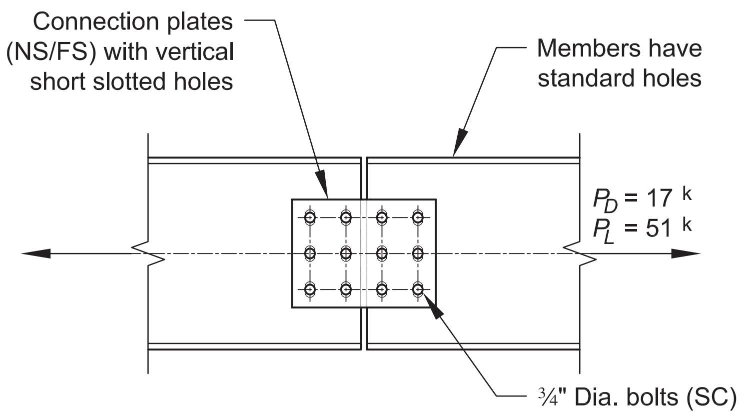
Example J.4a Slip-Critical Connection with Short-Slotted Holes

High-strength bolts in slip-critical connections are permitted to be designed to prevent slip either as a serviceability limit state or as a strength limit state. The most common design case is to design for slip as a serviceability limit state. The design for slip as a strength limit state should only be applied when bolt slip can result in a connection geometry that will increase the required strength beyond that of a strength limit state, such as bearing or bolt shear. Such considerations occur only when oversized holes or slots parallel to the load are used, and when the slipped geometry increases the demand on the connection. Examples include the case of ponding in flat-roofed long span trusses, or the case of shallow, short lateral bracing.

Given:

Select the number of ¾-in. ASTM A325 slip-critical bolts with a Class A faying surface that are required to support the loads shown when the connection plates have short slots transverse to the load. Select the number of bolts required for slip resistance only.

Assume that the connected pieces have short slots transverse to the load. Use a mean slip coefficient of 0.35, which corresponds to a Class A surface.



Solution:

Calculate the required strength

LRFD	ASD
P_u = 1.2(17 kips) + 1.6(51 kips) = 102 kips	P_a = 17 kips + 51 kips = 68.0 kips

For standard holes or slots transverse to the direction of the load, a connection can be designed on the basis of the serviceability limit state. For the serviceability limit state:

ϕ = 1.00 Ω = 1.50

Section J3.8

Find R_n, where:

μ = 0.35 for Class A surface
D_u = 1.13
h_{sc} = 0.85 (short slotted holes) Table J3.1
T_b = 28 kips
N_s = 2, number of slip planes

$R_n = \mu D_u h_{sc} T_b N_s$ Eqn. J3-4

$= 0.35(1.13)(0.85)(28 \text{ kips})(2) = 18.8 \text{ kips/bolt}$

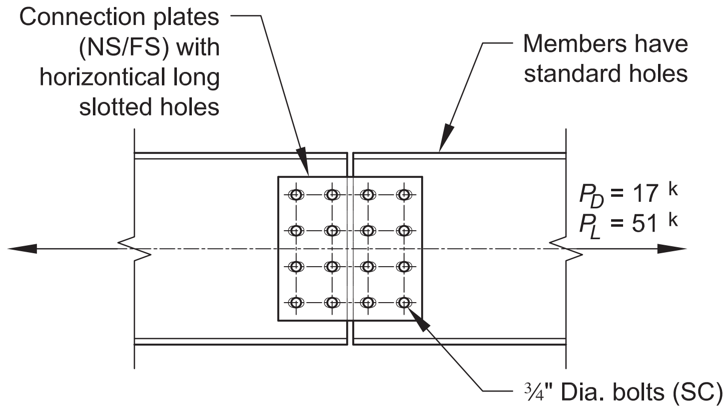
Determine the required number of bolts.

LRFD	ASD
$n = 102 \text{ kips}/\left[1.00\left(18.8 \text{ kips/bolt}\right)\right]$ $= 5.43 \text{ bolts}$ Use 6 bolts **o.k.**	$n = 68.0 \text{ kips}/\left(\dfrac{18.8 \text{ kips/bolt}}{1.50}\right)$ $= 5.43 \text{ bolts}$ Use 6 bolts **o.k.**

Example J.4b Slip-Critical Connection with Short-Slotted Holes

Given:

Repeat the problem with the same loads, but assuming that the connected pieces have long slotted holes in the direction of the load and that the deformed geometry of the connection would result in a critical load increase.



Solution:

Required strength from above

LRFD	ASD
$P_u = 102$ kips	$P_a = 68.0$ kips

For this connection, the designer has determined that oversized holes or slots parallel to the direction of the load will result in a deformed geometry of the connection that creates a critical load case. Therefore, the connection is designed to prevent slip at the required strength level.

$$\phi = 0.85 \quad \Omega = 1.76$$ Section J3.8

In addition, h_{sc} will change because we now have long slotted holes.

Find R_n

$\mu = 0.35$ for Class A surface
$D_u = 1.13$
$h_{sc} = 0.70$ (long slotted holes)
$T_b = 28$ kips Table J3.1

N_s = 2, number of slip planes

$R_n = \mu D_u h_{sc} T_b N_s$ Eqn. J3-4

$= 0.35(1.13)(0.70)(28 \text{ kips})(2) = 15.5 \text{ kips/bolt}$

Determine the required number of bolts

LRFD	ASD
$\dfrac{102 \text{ kips}}{0.85(15.5 \text{ kips/bolt})} = 7.74 \text{ bolts}$	$\dfrac{68.0 \text{ kips } (1.76)}{15.5 \text{ kips/bolt}} = 7.72 \text{ bolts}$
Use 8 bolts **o.k.**	Use 8 bolts **o.k.**

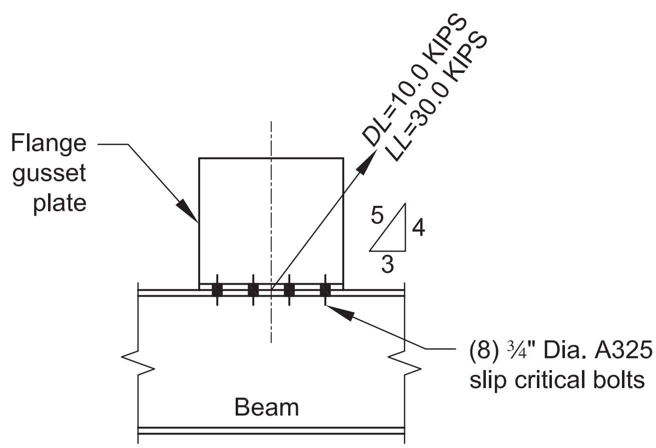
Manual Table 7-4

Example J.5 Combined Tension and Shear in a Slip-Critical Connection

Because the pretension of a bolt in a slip-critical connection is used to create the clamping force that produces the shear strength of the connection, the available shear strength must be reduced for any load that produces tension in the connection.

Given:

The slip-critical bolt group shown below is subjected to tension and shear. This connection is designed for slip as a serviceability limit state. Use ¾-in. diameter ASTM A325 slip-critical Class A bolts in standard holes. This example shows the design for bolt slip resistance only, and assumes that the beams and plates are adequate to transmit the loads. Determine if the bolts are adequate.



Solution:

The fastener pretension for a ¾-in. diameter ASTM A325 bolt is 28 kips. Specification Table J3.1

D_u = 1.13 per Specification Section J3.8.
N_b = 8 (number of bolts carrying the applied tension)
h_{sc} = 1.0 (standard holes)
T_b = 28 kips
N_s = 1 (number of slip planes)

Calculate the required strength

LRFD	ASD
P_u = 1.2(10.0 kips)+1.6(30.0 kips) = 60.0 kips *By geometry,* $T_u = \frac{4}{5}$(60.0 kips) = 48.0 kips = 6.00 kips/bolt $V_u = \frac{3}{5}$(60.0 kips) = 36.0 kips	P_a = 10.0 kips + 30.0 kips = 40.0 kips *By geometry,* $T_a = \frac{4}{5}$(40.0 kips) = 32.0 kips = 4.00 kips/bolt $V_a = \frac{3}{5}$(40.0 kips) = 24.0 kips

Calculate the nominal tensile strength of the bolts

Specification Table J3.2

$F_n = 90$ ksi

$$A_b = \frac{\pi(0.75 \text{ in.})^2}{4} = 0.442 \text{ in.}^2$$

$$R_n = F_nA_b = (90 \text{ ksi})(0.442 \text{ in.}^2) = 39.8 \text{ kips}$$

Check bolt tension

LRFD	ASD
$\phi R_n = (0.75)\left(\frac{39.8 \text{ kips}}{\text{bolt}}\right) > \frac{48.0 \text{ kips}}{8 \text{ bolts}}$ $= 29.9$ kips/bolt > 6.00 kips/bolt **o.k.** *Determine the available slip resistance (T_u = 0) of a bolt.* $\phi R_n = \mu D_u h_{sc} T_b N_s$ $\mu = 0.35$ (class A surface) $D_u = 1.13$ $h_{sc} = 1.00$ (standard holes) $T_b = 28$ kips (Table J3.1) $N_s = 1$ $\phi R_n = 1.00(0.35)(1.13)(1.00)(28 \text{ kips})(1)$ $= 11.1$ kips/bolt	$\frac{R_n}{\Omega} = \left(\frac{39.8 \text{ kips/bolt}}{2.00}\right) > \frac{32.0 \text{ kips}}{8 \text{ bolts}}$ $= 19.9$ kips/bolt > 4.00 kips/bolt **o.k.** *Determine the available slip resistance (T_a = 0) of a bolt.* $\frac{R_n}{\Omega} = \frac{\mu D_u h_{sc} T_b N_s}{\Omega}$ $\mu = 0.35$ (Class A surface) $D_u = 1.13$ $h_{sc} = 1.00$ (standard holes) $T_b = 28$ kips (Table J3.1) $N_s = 1$ $\frac{R_n}{\Omega} = \frac{(0.35)(1.13)(1.00)(28 \text{ kips})(1)}{1.50}$ $= 7.40$ kips/bolt

Eqn. J3-4

Calculate the available slip resistance of the connection.

LRFD	ASD
Slip-critical combined tension and shear coefficient. $k_s = 1 - \frac{T_u}{D_u T_b N_b} = 1 - \frac{48.0 \text{ kips}}{1.13(28 \text{ kips})(8)}$ $= 0.810$ $\phi = 1.0$ $\phi R_n = \phi R_n k_s N_b$ $= (11.1 \text{ kips/bolt})(0.810)(8 \text{ bolts})$ $= 71.9$ kips > 36.0 kips **o.k.**	*Slip-critical combined tension and shear coefficient.* $k_s = 1 - \frac{1.5T_a}{D_u T_b N_b} = 1 - \frac{1.5(32.0 \text{ kips})}{1.13(28 \text{ kips})(8)}$ $= 0.810$ $\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{R_n}{\Omega} k_s N_b$ $= (7.40 \text{ kips/bolt})(0.810)(8 \text{ bolts})$ $= 48.0$ kips > 24.0 kips **o.k.**

Eqn. J3-5a and J3-5b

Note: In this example, slip has been checked as a serviceability limit state and the bolt group must still be checked for all applicable strength limit states for a bearing-type connection.

Example J.6 Bearing Strength of a Pin in a Drilled Hole

Given:

A 1-in. diameter pin is placed in a drilled hole in a 1½-in. thick steel plate.

Determine the available bearing strength of the pinned connection, assuming the pin is stronger than the plate.

Material Properties:

Plate ASTM A36 F_y = 36 ksi F_u = 58 ksi — Manual Table 2-4

Solution:

Calculate the projected bearing area

$$A_{pb} = dt_p = (1 \text{ in.})(1.50 \text{ in.}) = 1.50 \text{ in.}^2$$

Calculate nominal bearing strength

$$R_n = 1.8 F_y A_{pb} = 1.8(36 \text{ ksi})(1.50 \text{ in.}^2) = 97.2 \text{ kips}$$ Eqn. J7-1

Calculate the available bearing strength — Section J7

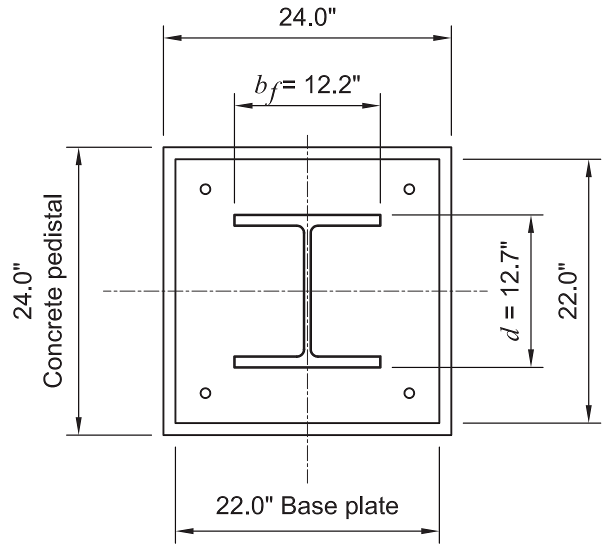
LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(97.2 \text{ kips}) = 72.9 \text{ kips}$	$\Omega = 2.00$ $R_n / \Omega = \dfrac{97.2 \text{ kips}}{2.00} = 48.6 \text{ kips}$

Example J.7 Base Plate Bearing on Concrete

Given:

A W12×96 column bears on a 24 in. × 24 in. concrete pedestal. The space between the base plate and the concrete pedestal is grouted. Design the base plate to support the following loads in axial compression:

P_D = 115 kips
P_L = 345 kips



Material Properties:

Column W12×96	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3 Table 2-4
Base Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	
Concrete Pedestal	$f_c' = 3$ ksi			
Grout	$f_c' = 4$ ksi			

Geometric Properties:

Column W12×96 d = 12.7 in. b_f = 12.2 in. t_f = 0.900 in. t_w = 0.550 in. — Manual Table 1-1

Solution:

Calculate the required strength

LRFD	ASD
$P_u = 1.2(115 \text{ kips}) + 1.6(345 \text{ kips}) = 690 \text{ kips}$	$P_a = 115 \text{ kips} + 345 \text{ kips} = 460 \text{ kips}$

Calculate the base plate area

LRFD	ASD
$\phi_c = 0.60$ $A_{1(req)} = \dfrac{P_u}{\phi_c 0.85 f'_c}$ $= \dfrac{690 \text{ kips}}{0.6(0.85)(3 \text{ ksi})} = 451 \text{ in.}^2$	$\Omega_c = 2.50$ $A_{1(req)} = \dfrac{P_a \Omega_c}{0.85 f'_c}$ $= \dfrac{(460 \text{ kips})(2.50)}{(0.85)(3 \text{ ksi})} = 451 \text{ in.}^2$

Specification Section J8

Note: The strength of the grout has conservatively been neglected, as its strength is greater than that of the concrete pedestal.

Try a 22.0 in.×22.0 in. base plate

Check base plate dimensions

Verify $N \geq d + 2(3.00 \text{ in.})$ and $B \geq b_f + 2(3.00 \text{ in.})$

$d + 2(3.00 \text{ in.}) = 12.7 \text{ in.} + 2(3.00 \text{ in.}) = 18.7 \text{ in.} < 22 \text{ in.}$ **o.k.**

$b_f + 2(3.00 \text{ in.}) = 12.2 \text{ in.} + 2(3.00 \text{ in.}) = 18.2 \text{ in.} < 22 \text{ in.}$ **o.k.**

Base plate area, $A_1 = NB = (22.0 \text{ in.})(22.0 \text{ in.}) = 484 \text{ in.}^2 > 451 \text{ in.}^2$ **o.k.**

Note: A square base plate with a square anchor rod pattern will be used to minimize the chance for field and shop problems.

Calculate the geometrically similar concrete bearing area

Since the pedestal is square and the base plate is a concentrically located square, the full pedestal area is also the geometrically similar area. Therefore,

$A_2 = (24.0 \text{ in.})(24.0 \text{ in.}) = 576 \text{ in.}^2$

Verify the concrete bearing strength

Use Eqn. J8-2 because the base plate covers less than the full area of the concrete support.

LRFD	ASD
$\phi_c = 0.60$ $\phi_c P_p = \phi_c 0.85 f_c' A_1 \sqrt{\dfrac{A_2}{A_1}} \leq \phi_c 1.7 f_c' A_1$ $= 0.6(0.85)(3 \text{ ksi})(484 \text{ in.}^2)\sqrt{\dfrac{576 \text{ in.}^2}{484 \text{ in.}^2}}$ $\leq (0.60)(1.7)(3 \text{ ksi})(484 \text{ in.}^2)$ $= 808 \text{ kips} \leq 1{,}480 \text{ kips}$, use 808 kips	$\Omega_c = 2.50$ $P_p / \Omega_c = \dfrac{0.85 f'_c A_1}{\Omega_c}\sqrt{\dfrac{A_2}{A_1}} \leq \dfrac{1.7 f'_c A_1}{\Omega_c}$ $= \dfrac{(0.85)(3 \text{ ksi})(484 \text{ in.}^2)}{2.50}\sqrt{\dfrac{576 \text{ in.}^2}{484 \text{ in.}^2}}$ $\leq \dfrac{(1.7)(3 \text{ ksi})(484 \text{ in.}^2)}{2.50}$ $= 539 \text{ kips} \leq 987 \text{ kips}$, use 539 kips

Section J8

Eqn. J8-2

808 kips > 690 kips **o.k.**	539 kips > 460 kips **o.k.**

Notes:

1) $A_2/A_1 \le 4$ so the upper limit in Eqn. J8-2 does not control.
2) It is permitted to take ϕ_c = 0.65 per ACI 318-05
3) As the area of the base plate approaches the area of concrete, the modifying ratio, $\sqrt{\dfrac{A_2}{A_1}}$, approaches unity and Specification Eqn. J8-2 converges to Specification Eqn. J8-1.

Calculate the required base plate thickness

Manual Part 14

$$m = \frac{N - 0.95d}{2} = \frac{22.0\text{ in.} - 0.95(12.7\text{ in.})}{2} = 4.97\text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} = \frac{22.0\text{ in.} - 0.8(12.2\text{ in.})}{2} = 6.12\text{ in.}$$

$$n' = \frac{\sqrt{db_f}}{4} = \frac{\sqrt{(12.7\text{ in.})(12.2\text{ in.})}}{4} = 3.11\text{ in.}$$

LRFD	ASD
$X = \dfrac{4db_fP_u}{(d+b_f)^2\,\phi_cP_p}$ $= \dfrac{4(12.7\text{ in.})(12.2\text{ in.})(690\text{ kips})}{(12.7\text{ in.}+12.2\text{ in.})^2\,(808\text{ kips})}$ $= 0.854$ $\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$ $= \dfrac{2\sqrt{0.854}}{1+\sqrt{1-0.854}}$ $= 1.34 > 1,\text{ use } \lambda = 1.$	$X = \dfrac{4db_fP_a\Omega_c}{(d+b_f)^2\,P_p}$ $= \dfrac{4(12.7\text{ in.})(12.2\text{ in.})(460\text{ kips})}{(12.7\text{ in.}+12.2\text{ in.})^2\,(539\text{ kips})}$ $= 0.853$ $\lambda = \dfrac{2\sqrt{X}}{1+\sqrt{1-X}} \le 1$ $= \dfrac{2\sqrt{0.853}}{1+\sqrt{1-0.853}}$ $= 1.34 > 1,\text{ use } \lambda = 1.$

Note: λ can always be conservatively taken as being equal to 1

$$\lambda n' = (1)(3.11\text{ in.}) = 3.11\text{ in.}$$

$$l = \max(m,\ n,\ \lambda n') = \max(4.97\text{ in., } 6.12\text{ in., } 3.11\text{ in.}) = 6.12\text{ in.}$$

LRFD	ASD
$f_{pu} = \dfrac{P_u}{BN} = \dfrac{690\text{ kips}}{(22.0\text{ in.})(22.0\text{ in.})} = 1.43\text{ ksi}$	$f_{pa} = \dfrac{P_a}{BN} = \dfrac{460\text{ kips}}{(22.0\text{ in.})(22.0\text{ in.})} = 0.950\text{ ksi}$

LRFD	ASD
$t_{p(req)} = l\sqrt{\dfrac{2f_{pu}}{0.9F_y}}$ $= (6.12 \text{ in.})\sqrt{\dfrac{2(1.43 \text{ ksi})}{0.9(36 \text{ ksi})}}$ $= 1.82 \text{ in.}$	$t_{p(req)} = l\sqrt{\dfrac{3.33f_{pa}}{F_y}}$ $= (6.12 \text{ in.})\sqrt{\dfrac{3.33(0.950 \text{ ksi})}{(36 \text{ ksi})}}$ $= 1.81 \text{ in.}$

Use a 2.00 in. thick base plate.

CHAPTER K
Design of HSS and Box Member Connections

Examples K.1 through **K.6** illustrate common beam to column shear connections that have been adapted for use with HSS columns. **Example K.7** illustrates a through plate shear connection, which is unique to HSS columns. Calculations for transverse and longitudinal forces applied to HSS are illustrated in **Examples K.8** and **K.9**. An example of an HSS truss connection is given in **Example K.10**. Examples on HSS cap plate and base plate connections are given in **Examples K.11** through **K.13**.

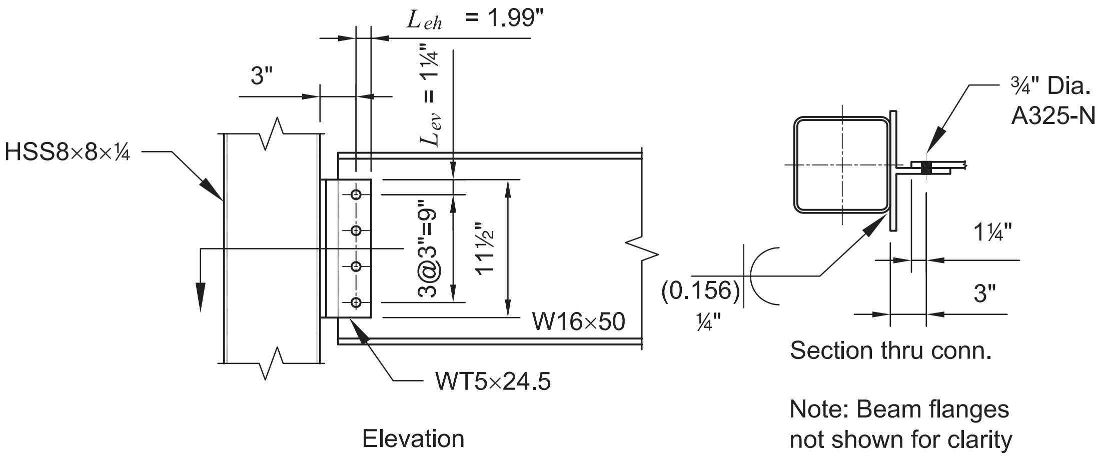
Example K.1 Welded/bolted Wide Tee Connection to an HSS Column

Given:

Design a connection between a W16×50 beam and a HSS8×8×¼ column using a WT5×24.5. Use ¾-in. diameter ASTM A325-N bolts in standard holes with a bolt spacing, s, of 3 in., vertical edge distance L_{ev} of 1¼ in. and 3 in. from the weld line to the bolt line. Design as a flexible connection.

P_D = 6.2 kips
P_L = 18.5 kips

Note: A tee with a flange width wider than 8 in. was selected to provide sufficient surface for flare bevel groove welds on both sides of the column, since the tee will be slightly offset from the column centerline.



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Tee	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Table 2-3
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W16×50	t_w = 0.380 in.	d = 16.3 in.	t_f = 0.630 in.	Manual
		T = 13 ⅝ in.			Tables
Tee	WT5×24.5	$t_s = t_w$ = 0.340 in.	d = 4.99 in.	t_f = 0.560 in.	1-1, 1-8, and
		b_f = 10.0 in.	k_1 = 13⁄16 in.		1-12
Column	HSS8×8×¼	t = 0.233 in.	B = 8.00 in.		

Calculate the required strength

LRFD	ASD
P_u = 1.2(6.2 kips) + 1.6(18.5 kips) = 37.0 kips	P_a = 6.2 kips + 18.5 kips = 24.7 kips

Calculate the available strength assuming the connection is flexible

LRFD	ASD	
Determine the number of bolts	*Determine the number of bolts*	
Determine the single bolt shear strength ϕr_n = 15.9 kips	*Determine the single bolt shear strength* r_n / Ω = 10.6 kips	Manual Table 7-1
Determine single bolt bearing strength based on edge distance	*Determine single bolt bearing strength based on edge distance*	
L_{ev} = 1¼ in. ≥ 1.25 in. **o.k.**	L_{ev} = 1¼ in. ≥ 1.25 in. **o.k.**	Table J3.4
ϕr_n = 49.4 kips/in.(0.340 in.) = 16.8 kips	r_n / Ω = 32.9 kips/in.(0.340 in.) = 11.2 kips	Manual Table 7-6
Determine single bolt bearing capacity based on spacing	*Determine single bolt bearing capacity based on spacing*	
s = 3.00 in. > 3(¾ in.) = 2.25 in.	s = 3.00 in. > 3(¾ in.) = 2.25 in.	Section J3.3
ϕr_n = 87.8 kips/in.(0.340 in.) = 29.9 kips	r_n / Ω = 58.5 kips/in.(0.340 in.) = 19.9 kips	Manual Table 7-5
Therefore bolt shear controls, $C_{min} = \frac{P_u}{\phi r_n} = \frac{37.0 \text{ kips}}{15.9 \text{ kips}} = 2.33$	Therefore bolt shear controls, $C_{min} = \frac{P_a}{r_n / \Omega} = \frac{24.7 \text{ kips}}{10.6 \text{ kips}} = 2.33$	
Using e = 3.00 in. and s = 3.00 in., determine C.	Using e = 3.00 in. and s = 3.00 in., determine C.	
Try 4 bolts, C = 2.81 > 2.33 **o.k.**	Try 4 bolts, C = 2.81 > 2.33 **o.k.**	Manual Table 7-7

Check WT stem thickness limit — Manual Part 9

$$t_{max} = \frac{d_b}{2} + \tfrac{1}{16} \text{ in.} = \frac{\left(\tfrac{3}{4} \text{ in.}\right)}{2} + \tfrac{1}{16} \text{ in.} = 0.438 \text{ in.} > 0.340 \text{ in.} \qquad \textbf{o.k.}$$

Note: The beam web thickness is greater than the WT stem thickness. If the beam web were thinner than the WT stem, this check could be satisfied by checking the thickness of the beam web.

Determine WT length required — Manual Part 10

A W16×50 has a T-dimension of 13⅝ in.

L_{min} = T/2 = (13⅝ in.)/2 = 6.81 in.

Determine WT length required for bolt spacing and edge distances

L = 3(3.00 in.) + 2 (1.25 in.) = 11.5 in. < T = 13⅝ in. **o.k.**

Try L = 11.5 in.

Calculate the stem shear yielding strength

$$R_n = 0.6F_yA_g = 0.6(50\text{ ksi})(11.5\text{ in.})(0.340\text{ in.}) = 117\text{ kips}$$ Eqn. J4-3

LRFD	ASD
ϕR_n = 1.00(117 kips) = 117 kips	$R_n / \Omega = \dfrac{117\text{ kips}}{1.50}$ = 78.0 kips
117 kips > 37.0 kips **o.k.**	78.0 kips > 24.7 kips **o.k.**

Calculate the stem shear rupture strength

Section J4.2

$$R_n = [L - n(d_h + 1/16)](t)(0.6F_u)$$
= [11.5 in. – 4(0.875 in.)](0.340 in.)(0.6)(65 ksi)
= 106 kips

LRFD	ASD
ϕR_n = 0.75(106 kips) = 79.5 kips	$R_n / \Omega = \dfrac{106\text{ kips}}{2.00}$ = 53.0 kips
79.5 kips > 37.0 kips **o.k.**	53.0 kips > 24.7 kips **o.k.**

Calculate the stem available block shear rupture strength Section J4.3

For this case U_{bs} = 1.0

LRFD	ASD
$\dfrac{\phi F_u A_{nt}}{t}$ = 76.2 kips/in.	$\dfrac{F_u A_{nt}}{t\Omega}$ = 50.8 kips/in.
$\dfrac{\phi 0.6F_y A_{gv}}{t}$ = 231 kips/in.	$\dfrac{0.6F_y A_{gv}}{t\Omega}$ = 154 kips/in.
$\dfrac{\phi 0.6F_u A_{nv}}{t}$ = 210 kips/in.	$\dfrac{0.6F_u A_{nv}}{t\Omega}$ = 140 kips/in.
$\phi R_n = \phi 0.6F_uA_{nv} + \phi U_{bs}F_uA_{nt}$ $\leq \phi 0.6F_yA_{gv} + \phi U_{bs}F_uA_{nt}$	$\dfrac{R_n}{\Omega} = \dfrac{0.6F_uA_{nv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega} \leq \dfrac{0.6F_yA_{gv}}{\Omega} + \dfrac{U_{bs}F_uA_{nt}}{\Omega}$
ϕR_n=0.340 in.(210 kips/in + 76.2 kips/in) $\leq$ 0.340 in.(231kips/in.+76.2 kips/in.) =97.3 kips $\leq$ 104 kips	R_n / Ω = 0.340 in.(140 kips/in. +50.8 kips/in.) $\leq$ 0.340 in.(154 kips/in.+50.8 kips/in.) = 64.9 kips $\leq$ 69.6 kips
97.3 kips > 37.0 kips **o.k.**	64.9 kips > 24.7 kips **o.k.**

Manual Table 9-3a

Manual Table 9-3b

Manual Table 9-3c

Eqn. J4-5

Check stem bending

Calculate the required flexural strength

LRFD	ASD
$M_u = P_u e$ = 37.0 kips(3.00 in.) = 111 kip-in.	$M_a = P_a e$ = 24.7 kips(3.00 in.) = 74.1 kip-in.

Calculate the stem nominal flexural yielding strength

$$Z = \frac{td^2}{4} = \frac{0.340\text{ in.}\left(11.5\text{ in.}\right)^2}{4} = 11.2\text{ in.}^3$$

$$S = \frac{td^2}{6} = \frac{0.340\text{ in.}\left(11.5\text{ in.}\right)^2}{6} = 7.49\text{ in.}^3$$

$$M_n = M_p = F_y Z \le 1.6 M_y$$

$$= 50\text{ ksi}\left(11.2\text{ in.}^3\right) \le 1.6\left(50\text{ ksi}\right)\left(7.49\text{ in.}^3\right)$$

= 560 kip-in. < 599 kip-in. **o.k.**

Note: The 1.6 limit will never control, because the shape factor for a plate is 1.5.

Calculate the stem available flexural yielding strength

Section F1

LRFD	ASD
ϕM_n=0.90(560 kip-in.) = 504 kip-in. > 111 kip-in. **o.k.**	$M_n / \Omega = \dfrac{560\text{ kip-in.}}{1.67}$ = 335 kip-in. > 74.1 kip-in. **o.k.**

Calculate the stem flexural rupture strength

Manual Part 9

$$Z_{net} = \frac{td^2}{4} - 2t\left(d_h + \tfrac{1}{16}\text{ in.}\right)\left(1.5\text{ in.} + 4.5\text{ in.}\right)$$

$$= \frac{0.340\text{ in.}\left(11.5\text{ in.}\right)^2}{4} - 2\left(0.340\text{ in.}\right)\left(\tfrac{13}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}\right)\left(1.5\text{ in.} + 4.5\text{ in.}\right) = 7.67\text{ in.}^3$$

$$M_n = F_u Z_{net} = 65\text{ ksi}\left(7.67\text{ in.}^3\right) = 499\text{ kip-in.}$$

LRFD	ASD
ϕM_n=0.75(499 kip-in.) = 374 kip-in. > 111 kip-in. **o.k.**	$M_n / \Omega = \dfrac{499\text{ kip-in.}}{2.00}$ = 250 kip-in. > 74.1 kip-in. **o.k.**

Because the WT flange is thicker than the stem and carries only half of the beam reaction, shear yielding of the flange is not a critical limit state.

Check beam web bearing

$t_w > t_s \rightarrow$ 0.380 in. > 0.340 in.

Beam web is satisfactory for bearing by comparison with WT.

Calculate weld size

Since the flange width of the WT is larger than the width of the HSS, a flare bevel groove weld is required. Taking the outside radius as 2(¼ in.) = ½ in. and using AISC Specification Table J2.2, the effective throat thickness of the flare bevel weld is E = 5⁄16(½ in.) = 0.156 in. Table J2.2

The minimum effective throat thickness of the flare bevel weld, based on the 0.233 in. thickness of the HSS column, is ⅛ in. Table J2.3

E = 0.156 in. > ⅛ in.

The equivalent fillet weld that provides the same throat dimension is

$$\left(\frac{D}{16}\right)\left(\frac{1}{\sqrt{2}}\right) = 0.156 \rightarrow D = 16\sqrt{2}\left(0.156\right) = 3.53 \text{ sixteenths of an inch}$$

The equivalent fillet weld size is used in the following calculations

Check weld ductility Manual Part 9

Let $b_f = B = 8$ in.

$$b = \frac{b_f - 2k_1}{2} = \frac{8.00\text{ in.} - 2\left(\text{13⁄16 in.}\right)}{2} = 3.19\text{ in.}$$

$$w_{\min} = \left(0.0158\right)\frac{F_y t_f^{\,2}}{b}\left(\frac{b^2}{L^2} + 2\right) \leq \left(\text{⅝}\right)t_s$$

$$= \left(0.0158\right)\frac{\left(50\text{ ksi}\right)\left(0.560\text{ in.}\right)^2}{3.19\text{ in.}}\left[\frac{\left(3.19\text{ in.}\right)^2}{\left(11.5\text{ in.}\right)^2} + 2\right] \leq \left(\text{⅝}\right)\left(0.340\text{ in.}\right)$$

$$= 0.161\text{ in.} < 0.213\text{ in.}$$

0.161 in. = 2.58 sixteenths of an inch

$D_{\min} = 2.58 < 3.53$ sixteenths of an inch **o.k.**

Calculate the nominal weld shear strength

The load is assumed to act concentrically with the weld group (flexible connection).

a = 0, therefore, C = 3.71

R_n=CC_1Dl=3.71(1.00)(3.53 sixteenths of an in.)(11.5 in.) = 151 kips Manual Table 8-4

Check shear rupture of the HSS at the weld Manual Part 9

$$t_{\min} = \frac{3.09D}{F_u} = \frac{3.09\left(3.53\text{ sixteenths}\right)}{58\text{ ksi}} = 0.188\text{ in.}$$

By inspection, shear rupture of the WT flange at the welds will not control.

Therefore, the weld controls.

Calculate the available weld strength

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(151 \text{ kips})$ $\quad = 113 \text{ kips} > 37.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $R_n / \Omega = 151 \text{ kips} / 2.00$ $\quad = 75.5 \text{ kips} > 24.7 \text{ kips}$ **o.k.**

Section J2.4

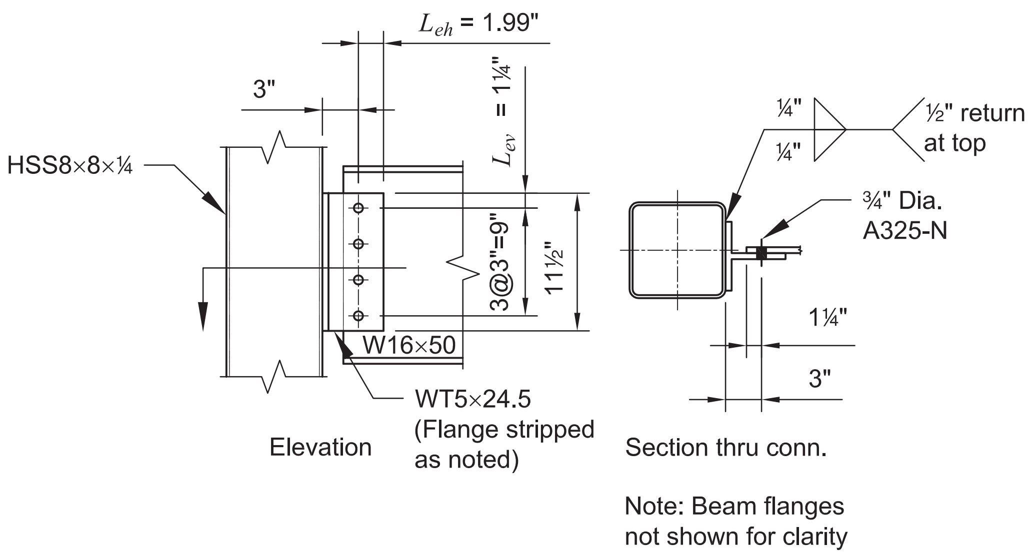
Example K.2 Welded/bolted Narrow Tee Connection to an HSS Column

Given:

Design a connection for a W16×50 beam to an HSS8×8×¼ column using a tee with fillet welds against the flat width of the HSS. Use ¾-in. diameter A325-N bolts in standard holes with a bolt spacing, *s*, of 3.00 in., vertical edge distance L_{ev} of 1¼ in. and 3.00 in. from the weld line to the center of the bolt line. Design this as a connection to a flexible support. Assume that, for architectural purposes, the flanges of the WT from the previous example have been stripped down to a width of 5.5 in.

P_D = 6.2 kips
P_L = 18.5 kips

Note: This is the same problem as **Example K.1** with the exception that a narrow tee will be selected which will permit fillet welds on the flat of the column. The beam will still be centered; therefore the tee will be slightly offset.



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Tee	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Table 2-3
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W16×50	t_w = 0.380 in.	d = 16.3 in.	t_f = 0.630 in.	Manual
Column	HSS8×8×¼	t = 0.233 in.	B = 8.00 in.		Tables 1-1,
Tee	WT5×24.5	t_s = t_w = 0.340 in.	d = 4.99 in.	t_f = 0.560 in.	1-8 and 1-12
		k_1 = 13⁄16 in.			

Calculate the required strength

LRFD	ASD
$P_u = 1.2(6.2 \text{ kips}) + 1.6(18.5 \text{ kips})$ $= 37.0 \text{ kips}$	$P_a = 6.2 \text{ kips} + 18.5 \text{ kips}$ $= 24.7 \text{ kips}$

Verify the strength of the WT

Maximum flange width assuming ¼-in. welds and HSS corner radius equal to 2.25 times the nominal thickness (2.25)(¼ in.) = 9/16 in.

$$\text{Connection offset} = \frac{0.380 \text{ in.}}{2} + \frac{0.340 \text{ in.}}{2} = 0.360 \text{ in.}$$

$$b_f \leq 8.00 \text{ in.} - 2\left(\tfrac{9}{16} \text{ in.}\right) - 2\left(\tfrac{1}{4} \text{ in.}\right) - 2\left(0.360 \text{ in.}\right) = 5.66 \text{ in.}$$

The strength of the stem thickness was verified in Example K.1.

Determine the number of bolts

LRFD	ASD	
Determine the bolt shear strength ϕr_n = 15.9 kips	*Determine the bolt shear strength* r_n / Ω = 10.6 kips	Manual Table 7-1
Determine bolt bearing strength based on edge distance L_{ev} = 1¼-in. ≥ 1¼ in.	*Determine bolt bearing strength based on edge distance* L_{ev} = 1¼-in. ≥ 1¼ in.	Table J3.4
ϕr_n = 49.4 kips/in.(0.340 in.) = 16.8 kips	r_n / Ω = 32.9 kips/in.(0.340 in.) = 11.2 kips	Manual Table 7-6
Determine single bolt bearing capacity based on spacing	*Determine single bolt bearing capacity based on spacing*	
s = 3.00 in. > 3(¾ in.) = 2.25 in.	s = 3.00 in. > 3(¾ in.) = 2.25 in.	Section J3.3
ϕr_n = 87.8 kips/in.(0.340 in.) = 29.9 kips	r_n / Ω = 58.5 kips/in.(0.340 in.) = 19.9 kips	Manual Table 7-5
Therefore, bolt shear strength controls	Therefore, bolt shear strength controls	

Determine the coefficient for the eccentrically loaded bolt group

LRFD	ASD	
ϕr_n = 15.9 kips	r_n / Ω = 10.6 kips	Manual Table 7-7
$C_{min} = \dfrac{P_u}{\phi r_n} = \dfrac{37.0 \text{ kips}}{15.9 \text{ kips}} = 2.33$	$C_{min} = \dfrac{P_a}{r_n / \Omega} = \dfrac{24.7 \text{ kips}}{10.6 \text{ kips}} = 2.33$	
Using e = 3.00 in. and s = 3.00 in., enter Manual Table 7-7	Using e = 3.00 in. and s = 3.00 in., enter Manual Table 7-7	
Try 4 bolts, C = 2.81 > 2.33 **o.k.**	Try 4 bolts, C = 2.81 > 2.33 **o.k.**	

Determine the minimum fillet weld size based on the thinner part

Minimum size = ⅛ in. (D = 2) for welding to ¼ in. material | Table J2.4

Check weld ductility

Manual Part 9

$$b = \frac{\left(b_f - 2k_1\right)}{2} = \frac{\left[5.50\text{ in.} - 2\left(\tfrac{13}{16}\text{ in.}\right)\right]}{2} = 1.94\text{ in.}$$

$$w_{\min} = \left(0.0158\right)\frac{F_y t_f^{\,2}}{b}\left(\frac{b^2}{L^2} + 2\right) \le \left(\tfrac{5}{8} t_s\right)$$

$$= \left(0.0158\right)\frac{\left(50\text{ ksi}\right)\left(0.560\text{ in.}\right)^2}{1.94\text{ in.}}\left[\frac{\left(1.94\text{ in.}\right)^2}{\left(11.5\text{ in.}\right)^2} + 2\right] \le \left(\tfrac{5}{8}\right)\left(0.340\text{ in.}\right)$$

$$= 0.259\text{ in.} \le 0.213\text{ in.}\text{ , use } 0.213\text{ in.}$$

D_{min} = (0.213 in.)16 = 3.41 sixteenths of an inch.

Try a ¼-in. fillet weld as a practical minimum, which is less than the maximum permitted weld size of t_f – ¹⁄₁₆ in. = 0.560 in. – ¹⁄₁₆ in. = 0.498 in. Provide ½ in. return welds at the top of the WT to meet the criteria listed in Section J2.2b. | Section J2.2b

Calculate minimum wall thickness in HSS to match weld strength | Manual Section 9

$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09\left(4\right)}{58\text{ ksi}} = 0.213\text{ in.} < 0.233\text{ in.}$$

By inspection, shear rupture of the flange of the WT at the welds will not control.

Therefore, the weld controls

Calculate the weld nominal shear strength

The load is assumed to act concentrically with the weld group (flexible connection).

a = 0; therefore, C = 3.71

$R_n = CC_1Dl$ = 3.71(1.00)(4 sixteenths of an in.)(11.5 in.) = 171 kips | Manual Table 8-4

Calculate the available weld shear strength

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
ϕR_n = 0.75(171 kips) = 128 kips	$R_n / \Omega = \dfrac{171\text{ kips}}{2.00} = 85.5\text{ kips}$
128 kips > 37.0 kips **o.k.**	85.5 kips > 24.7 kips **o.k.**

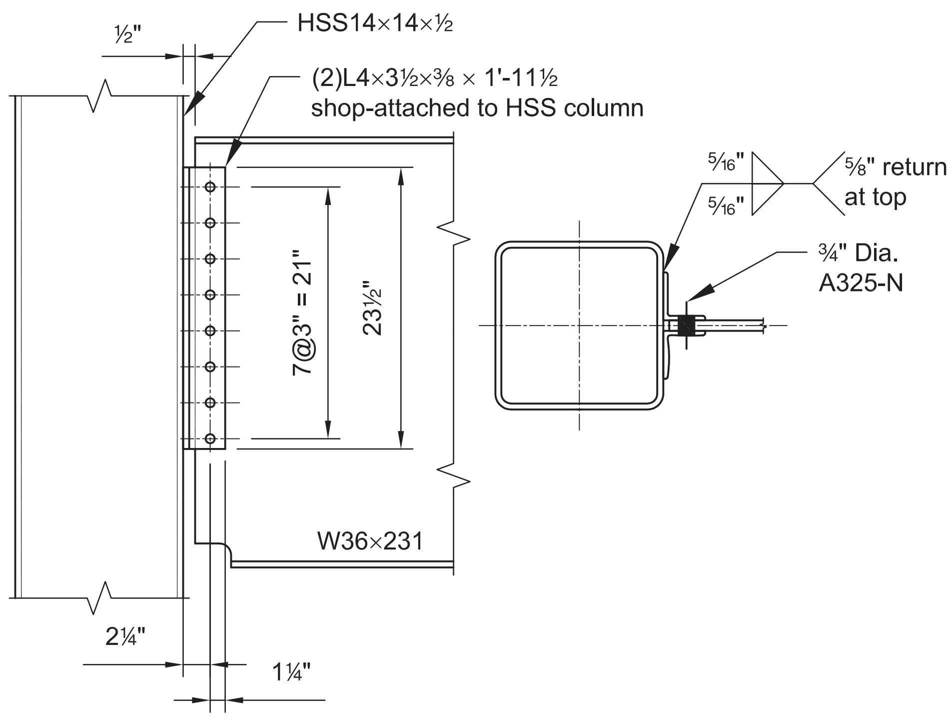
Section J2.4

Example K.3 Double Angle Connection to an HSS Column

Given:

Use Tables 10-1 and 10-2 to design a double-angle connection for a W36×231 beam to an HSS14×14×½ column. Use ¾-in. diameter ASTM A325-N bolts in standard holes. The bottom flange cope is required for erection.

P_D = 37.5 kips
P_L = 113 kips



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Table 2-3
Angles	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W36×231	t_w = 0.760 in.		Manual
Column	HSS14×14×½	t = 0.465 in.	B = 14.0 in.	Tables 1-1 and 1-12

Compute the required strength

LRFD	ASD
R_u = 1.2(37.5 kips) + 1.6(113 kips) = 226 kips	R_a = 37.5 kips + 113 kips = 151 kips

Design bolts and welds

Try 8 rows of bolts and ⁵⁄₁₆-in. welds

Obtain the bolt group and angle available strengths from Table 10-1

LRFD	ASD
ϕR_n = 254 kips > 226 kips **o.k.**	R_n/Ω = 170 kips > 151 kips **o.k.**

Manual Table 10-1

Obtain the available weld strength from Table 10-2 (welds B)

LRFD	ASD
ϕR_n = 279 kips > 226 kips **o.k.**	R_n/Ω= 186 kips > 151 kips **o.k.**

Manual Table 10-2

Check the minimum support thickness

Manual Table 10-2

Minimum support thickness = $0.238\text{ in.}\left(\frac{65\text{ ksi}}{58\text{ ksi}}\right)$ = 0.267 in. < 0.465 in. **o.k.**

Calculate the minimum angle thickness

Section J2.2b

$t_{min} = w + 1/16 = 5/16 + 1/16 = 3/8$ in.

Use ⅜-in. angle thickness to accommodate the welded legs of the double angle connection.

Use 2L4×3½ ×⅜×1'-11½".

L = 23.5 in. > T/2 **o.k.**

The workable flat for the HSS column is 14.0 in. – 2(2.25)(0.500 in.) = 11.8 in.

Manual Part 1

The minimum acceptable width to accommodate the connection is
2(4 in.) + 0.760 in. + 2(5/16 in.) = 9.39 in. < 11.8 in. **o.k.**

Calculate the available beam web strength

LRFD	ASD
ϕR_n = (702 kips/in.)(0.760 in.) = 534 kips 534 kips > 226 kips **o.k.**	R_n/Ω = (468 kips/in.)(0.760 in.) = 356 kips 356 kips > 151 kips **o.k.**

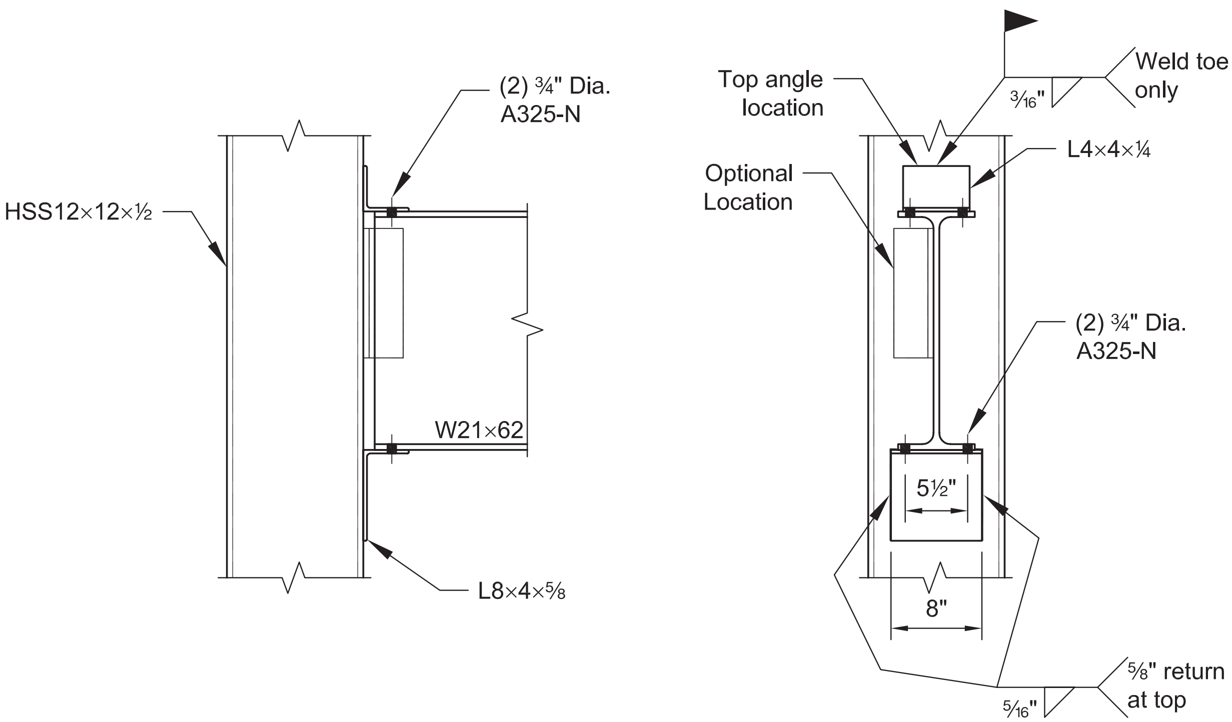
Manual Table 10-1

Example K.4 Unstiffened Seated Connection to an HSS Column

Given:

Use Table 10-6 to design an unstiffened seated connection for a W21×62 beam to an HSS12×12×½ column.

P_D = 9 kips
P_L = 27 kips



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Table 2-3
Angles	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W21×62	t_w = 0.400 in.	d = 21.0 in.	k_{des} = 1.12 in.	Manual
Column	HSS12×12×½	t = 0.465 in.	B = 12.0 in.		Tables 1-1 and 1-12

Calculate the required strength

LRFD	ASD
R_u = 1.2(9 kips) + 1.6(27 kips) = 54.0 kips	R_a = 9 kips + 27 kips = 36.0 kips

Design the seat angle and weld

LRFD	ASD
Check local web yielding $N_{min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{54.0 \text{ kips} - 55.8 \text{ kips}}{20.0 \text{ kips/in.}} \geq 1.12 \text{ in.}$ Use N_{min} = 1.12 in. *Check web crippling when N / d ≤ 0.2,* $N_{min} = \dfrac{R_u - \phi R_3}{\phi R_4} = \dfrac{54.0 \text{ kips} - 71.7 \text{ kips}}{5.37 \text{ kips/in.}}$ which results in a negative quantity. *Check web crippling when N / d > 0.2,* $N_{min} = \dfrac{R_u - \phi R_5}{\phi R_6} = \dfrac{54.0 \text{ kips} - 64.2 \text{ kips}}{7.16 \text{ kips/in.}}$ which results in a negative quantity.	*Check local web yielding* $N_{min} = \dfrac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k_{des}$ $= \dfrac{36.0 \text{ kips} - 37.2 \text{ kips}}{13.3 \text{ kips/in.}} \geq 1.12 \text{ in.}$ Use N_{min} = 1.12 in. *Check web crippling when N / d ≤ 0.2,* $N_{min} = \dfrac{R_a - R_3 / \Omega}{R_4 / \Omega} = \dfrac{36.0 \text{ kips} - 47.8 \text{ kips}}{3.58 \text{ kips/in.}}$ which results in a negative quantity. *Check web crippling when N / d > 0.2,* $N_{min} = \dfrac{R_a - R_5 / \Omega}{R_6 / \Omega} = \dfrac{36.0 \text{ kips} - 42.8 \text{ kips}}{4.78 \text{ kips/in.}}$ which results in a negative quantity.

Manual Table 9-4
Manual Part 10

Note: Generally, the value of N/d is not initially known and the larger value determined from the web crippling equations above can be used conservatively to determine the bearing length required for web crippling.

For this beam and end reaction, the beam web strength exceeds the required strength (hence the negative bearing lengths) and the lower-bound bearing length controls (N_{req} = k = 1.12 in.). Thus, N_{min} = 1.12 in.

Try a L8×4×⅝ seat with 5⁄16 in. fillet welds.

Determine outstanding angle leg available strength

LRFD	ASD
$\phi R_n = 81.0$ kips 81.0 kips > 54.0 kips **o.k.**	$R_n / \Omega = 53.9$ kips 53.9 kips > 36.0 kips **o.k.**

Manual Table 10-6

Determine weld available strength

LRFD	ASD
$\phi R_n = 66.7$ kips > 54.0 kips **o.k.**	$R_n / \Omega = 44.5$ kips > 36.0 kips **o.k.**

Manual Table 10-6

Since the t of the HSS is greater than the t_{min} for the 5⁄16-in. weld, no reduction in the weld strength is required to account for the shear in the HSS.

Connection to beam and top angle

Use a L4×4×¼ in. top angle. Use a 3⁄16-in. fillet weld across the toe of the angle for attachment to the HSS. Attach both the seat and top angles to the beam flanges with two ¾-in. diameter

Manual Part 10

ASTM A325-N bolts.

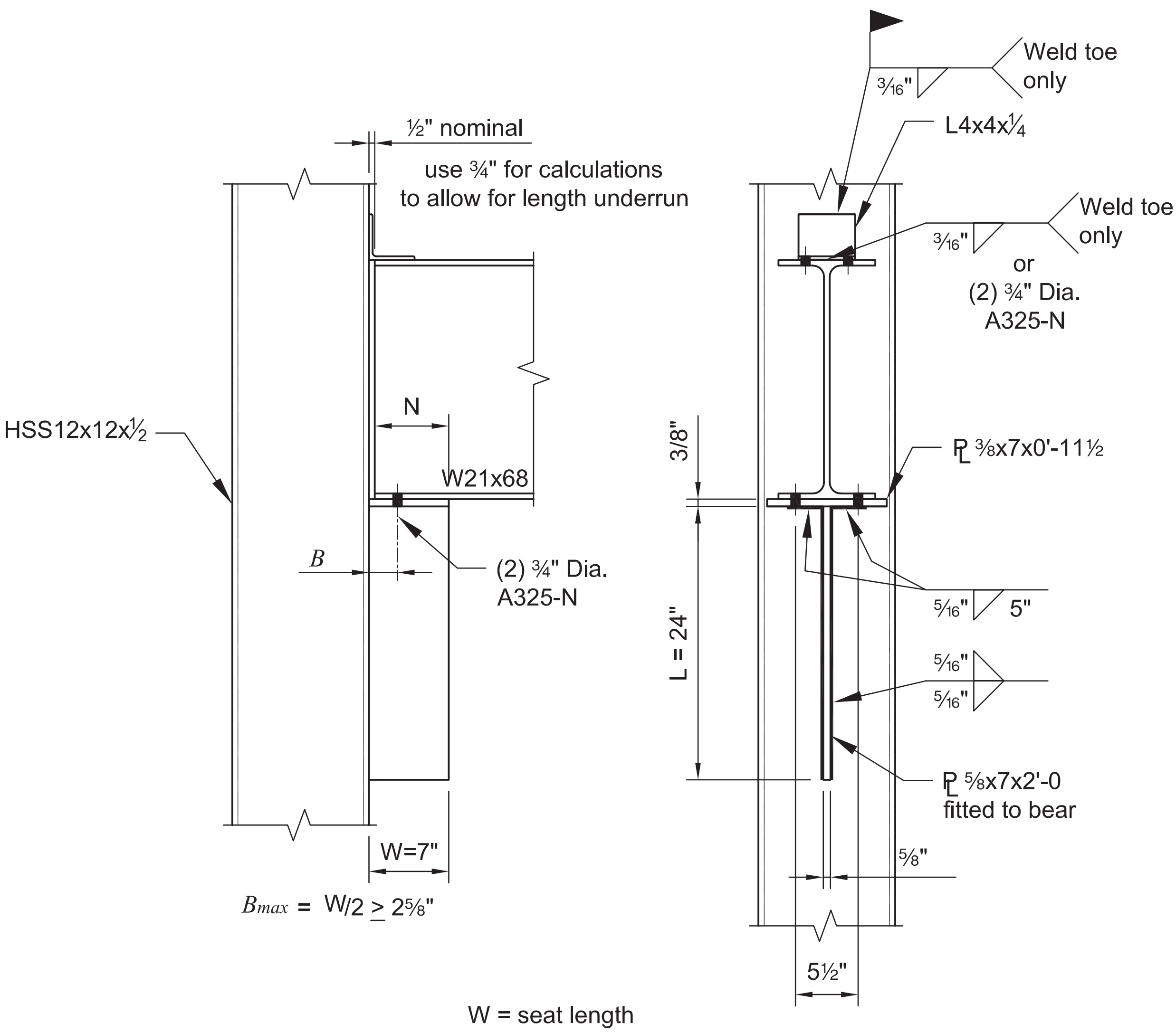
Example K.5 Stiffened Seated Connection to an HSS Column

Given:

Use Manual Tables 10-8 and 10-14 to design a stiffened seated connection for a W21×68 beam to a HSS12×12×½ column.

P_D = 20 kips
P_L = 60 kips

Use ¾-in. diameter ASTM A325-N bolts in standard holes to connect the beam to the seat plate. Use 70-ksi electrode welds to connect the stiffener, seat plate and top angle to the HSS.



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Tables 2-3
Angles and Plates	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	& 2-4

Geometric Properties:

Manual Tables 1-1 and 1-12

Beam	W21×68	t_w = 0.430 in.	d = 21.1 in.	k_{des} = 1.19 in.
Column	HSS12×12×½	t = 0.465 in.	B = 12.0 in.	

Check limits of applicability for Specification Section K1

Strength: $F_y = 46\text{ ksi} \leq 52\text{ ksi}$ **o.k.** Section K1.2

Ductility: $\dfrac{F_y}{F_u} = \dfrac{46\text{ ksi}}{58\text{ ksi}} = 0.793 \leq 0.8$ **o.k.**

Calculate the required strength

LRFD	ASD
$P_u = 1.2(20.0\text{ kips}) + 1.6(60.0\text{ kips})$ $= 120\text{ kips}$	$P_a = 20.0\text{ kips} + 60.0\text{ kips}$ $= 80.0\text{ kips}$

Determine stiffener width W required for web crippling and local web yielding

For web crippling, assume N/d > 0.2 and use constants R_5 and R_6 from Manual Table 9-4. Assume a ¾-in. setback. Manual Part 10

LRFD	ASD
$W_{min} = \dfrac{R_u - \phi R_5}{\phi R_6} + \text{setback} \geq k + \text{setback}$ $= \dfrac{120\text{ kips} - 75.9\text{ kips}}{7.94\text{ kips/in.}} + \frac{3}{4}\text{ in.}$ $= 6.30\text{ in.}$	$W_{min} = \dfrac{R_a - R_5/\Omega}{R_6/\Omega} + \text{setback} \geq k + \text{setback}$ $= \dfrac{80.0\text{ kips} - 50.6\text{ kips}}{5.29\text{ kips/in.}} + \frac{3}{4}\text{ in.}$ $= 6.31\text{ in.}$

Manual Table 9-4

For local web yielding, use constants R_1 and R_2 from Manual Table 9-4. Assume a ¾-in. setback.

LRFD	ASD
$W_{min} = \dfrac{R_u - \phi R_1}{\phi R_2} + \text{setback}$ $= \dfrac{120\text{ kips} - 63.7\text{ kips}}{21.5\text{ kips/in.}} + \frac{3}{4}\text{ in.}$ $= 3.37\text{ in.}$	$W_{min} = \dfrac{R_a - R_1/\Omega}{R_2/\Omega} + \text{setback}$ $= \dfrac{80.0\text{ kips} - 42.5\text{ kips}}{14.3\text{ kips/in.}} + \frac{3}{4}\text{ in.}$ $= 3.37\text{ in.}$

Manual Table 9-4

The minimum stiffener width, W, for web crippling controls. Use W = 7.00 in.

Check bearing width assumption

$$N = 7.00\text{ in.} - \tfrac{3}{4}\text{ in.} = 6.25\text{ in.}$$

$$N/d = \frac{6.25\text{ in.}}{21.1\text{ in.}} = 0.296 > 0.2,\ \text{as assumed.}$$

Determine the weld strength requirements for the seat plate

Try a stiffener of length, L = 24 in. with 5⁄16 in. fillet welds. Enter Manual Table 10-8 using W

= 7 in. as determined above.

LRFD	ASD
$\phi R_n = 293 \text{ kips} > 120 \text{ kips}$ **o.k.**	$R_n / \Omega = 195 \text{ kips} > 80.0 \text{ kips}$ **o.k.**

Manual Table 10-8

Because t of the HSS is greater than t_{min} for the ⁵⁄₁₆-in. fillet weld (from Specification Section J2.2), no reduction in the weld strength to account for shear in the HSS is required.

The minimum length of the seat-plate-to-HSS weld on each side of the stiffener is $0.2L = 4.8$ in. This establishes the minimum weld between the seat plate and stiffener; use 5 in. of ⁵⁄₁₆-in. weld on each side of the stiffener. Manual Part 10

Determine the stiffener plate thickness

To develop the stiffener-to-seat plate welds, the minimum stiffener thickness is

$$t_{pmin} = 2w = 2\left(\tfrac{5}{16} \text{ in.}\right) = \tfrac{5}{8} \text{ in.}$$

For a stiffener with F_y = 36 ksi and a beam with F_y = 50 ksi, the minimum stiffener thickness,

$$t_{pmin} = \left(\frac{F_{y\,beam}}{F_{y\,stiffener}}\right)t_w = \left(\frac{50\text{ ksi}}{36\text{ ksi}}\right)(0.430 \text{ in.}) = 0.597 \text{ in.}$$

For a stiffener with F_y = 36 ksi and a column with F_u = 58 ksi, the maximum stiffener thickness is

$$t_{pmax} = \frac{F_u t}{F_{yp}} = \frac{58 \text{ ksi}(0.465 \text{ in.})}{36 \text{ ksi}} = 0.749 \text{ in.}$$

Eqn. K1-10

Use stiffener thickness of ⅝ in.

Determine the stiffener length using Manual Table 10-14

LRFD	ASD
$\left(\dfrac{R_u W}{t^2}\right)_{req} = \dfrac{120 \text{ kips}(7.00 \text{ in.})}{(0.465 \text{ in.})^2}$ $= 3{,}880$ kips/in. To satisfy the minimum above, select a stiffener L = 24 in. from Table 10-14 $\dfrac{R_u W}{t^2} = 4{,}310 \text{ kips/in.} > 3{,}880 \text{ kips/in.}$ **o.k.**	$\left(\dfrac{R_a W}{t^2}\right)_{req} = \dfrac{80.0 \text{ kips}(7.00 \text{ in.})}{(0.465 \text{ in.})^2}$ $= 2{,}590$ kips/in. To satisfy the minimum above, select a stiffener L = 24 in. from Table 10-14 $\dfrac{R_a W}{t^2} = 2{,}870 \text{ kips/in.} > 2{,}590 \text{ kips/in.}$ **o.k.**

Manual Table 10-14

Use PL⅝×7×2’-0 for the stiffener.

Check the HSS width

The minimum width is $0.4L + t_p + 3t$

$$B = 12.0 \text{ in.} > 0.4(24.0 \text{ in.}) + \tfrac{5}{8} \text{ in.} + 3(0.465 \text{ in.}) = 11.6 \text{ in.}$$

Determine the seat plate dimensions

To accommodate two ¾-in. diameter ASTM A325-N bolts on a 5½ in. gage connecting the beam flange to the seat plate, a width of 8 in. is required. To accommodate the seat-plate-to-HSS weld, the required width is:

$$2(5.00\text{ in.}) + 0.625\text{ in.} = 10.6\text{ in.}$$

Manual Part 10

Note: To allow room to start and stop welds, an 11.5 in. width is used.

Use PL⅜ in. ×7×0 ft 11½ in. for the seat plate.

Select the top angle, bolts and welds

The minimum weld size for the HSS thickness according to Specification Table J2.4 is 3⁄16 in. The angle thickness should be 1⁄16 in. larger.

Use L4×4×¼ with 3⁄16-in. fillet welds along the toes of the angle to the beam flange and HSS. Alternatively two ¾-in. diameter ASTM A325-N bolts may be used to connect the leg of the angle to the beam flange.

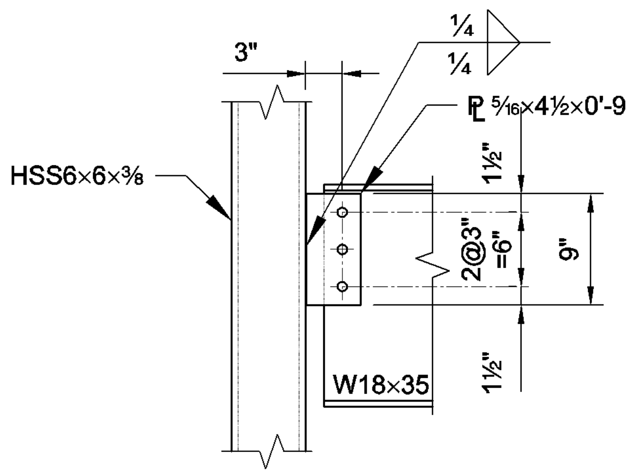
Example K.6 Single-Plate Connection to a Rectangular HSS Column

Given:

Use Manual Table 10-9 to design a single-plate connection for a W18×35 beam framing into a HSS6×6×⅜ column.

P_D = 6.5 kips
P_L = 19.5 kips

Use ¾-in. diameter ASTM A325-N bolts in standard holes and 70 ksi weld electrode.



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Tables 2-3
Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	and 2-4

Geometric Properties:

					Manual
Beam	W18×35	d = 17.7 in.	t_w = 0.300 in.	T = 15½ in.	Tables 1-1
Column	HSS6×6×⅜	$B = H$ = 6.00 in.	t = 0.349 in.	b/t = 14.2	and 1-12

Determine applicability of Specification Section K1

Strength: $F_y = 46 \text{ ksi} \leq 52 \text{ ksi}$ **o.k.**

Section K1.2

Ductility: $\dfrac{F_y}{F_u} = \dfrac{46 \text{ ksi}}{58 \text{ ksi}} = 0.793 \leq 0.8$ **o.k.**

Determine if a single plate connection is suitable (the HSS wall is not slender)

Manual Part 10

Slenderness: $\lambda = 1.40\sqrt{\dfrac{E}{F_y}} = 1.40\sqrt{\dfrac{29{,}000 \text{ ksi}}{46 \text{ ksi}}} = 35.2 > 14.2$ **o.k.**

Table B4.1 Case 12

Calculate the required strength

LRFD	ASD
$R_u = 1.2(6.5\text{ kips}) + 1.6(19.5\text{ kips})$ $= 39.0\text{ kips}$	$R_a = 6.5\text{ kips} + 19.5\text{ kips}$ $= 26.0\text{ kips}$

Calculate maximum single-plate thickness

$$t_{pmax} = \frac{F_u t}{F_{yp}} = \frac{58\text{ ksi}(0.349\text{ in.})}{36\text{ ksi}} = 0.562\text{ in.}$$ Eqn. K1-10

Note: Limiting the single-plate thickness precludes a shear yielding failure of the HSS wall. Section K1.5

Design the single-plate connection

Try 3 bolts and a 5⁄16-in. plate thickness with ¼-in. fillet welds.

$t_p = \frac{5}{16}\text{ in.} < 0.562\text{ in.}$ **o.k.**

Note: Either the plate or the beam web must satisfy: Manual Part 10

$t \le d_b/2 + \frac{1}{16} = 0.750\text{ in.}/2 + \frac{1}{16}\text{ in.} = 0.438\text{ in.}$

$\frac{5}{16}\text{ in.} \le 0.438\text{ in.}$ **o.k.**

Obtain the available single plate connection strength from Manual Table 10-9

Manual Table 10-9

LRFD	ASD
$\phi R_n = 47.7\text{ kips} > 39.0\text{ kips}$ **o.k.**	$R_n / \Omega = 31.8\text{ kips} > 26.0\text{ kips}$ **o.k.**

Use a PL5⁄16×4½×0’-9”

Check HSS shear rupture strength at welds

Manual Part 9

$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09(4)}{58\text{ ksi}} = 0.213\text{ in.} < t = 0.349\text{ in.} \quad \textbf{o.k.}$$

Calculate the available beam web bearing strength from Manual Table 10-1

For three ¾-in. diameter bolts

Manual Table 10-1

LRFD	ASD
$\phi R_n = (263\text{ kips/in.})(0.300\text{ in.})$ $= 78.9\text{ kips} > 39.0\text{ kips}$ **o.k.**	$R_n / \Omega = (175\text{ kips/in.})(0.300\text{ in.})$ $= 52.5\text{ kips} > 26.0\text{ kips}$ **o.k.**

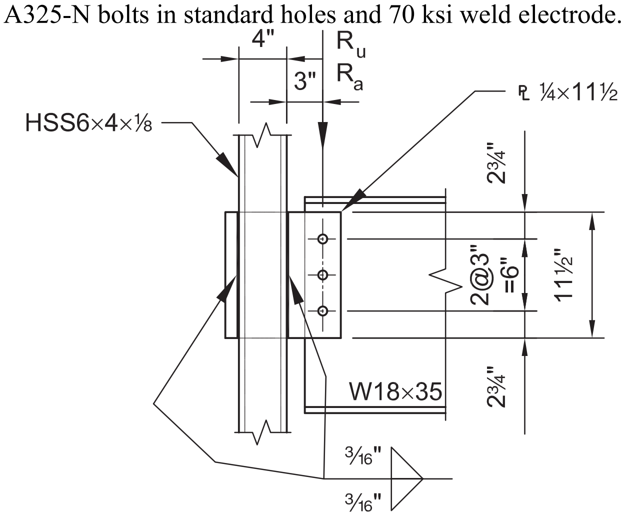
Example K.7 Through-Plate Connection

Given:

Use Table 10-9 to design a through-plate connection between a W18×35 beam and a HSS6×4×⅛ with the connection to one of the 6 in. faces. A thin-walled column is used to illustrate the design of a through-plate connection.

P_D = 3.3 kips
P_L = 9.9 kips

Use ¾-in. diameter ASTM A325-N bolts in standard holes and 70 ksi weld electrode.



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Tables 2-3
Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	and 2-4

Geometric Properties:

					Manual
Beam	W18×35	d = 17.7 in.	t_w = 0.300 in.	T = 15½ in.	Tables 1-1
Column	HSS6×4×⅛	B = 4.00 in.	H = 6.00 in.	t = 0.116 in.	and 1-11
		h/t = 48.7			

Determine applicability of Specification Section K1

Strength: $F_y = 46\text{ ksi} \le 52\text{ ksi}$ **o.k.** Section K1.2

Ductility: $\dfrac{F_y}{F_u} = \dfrac{46\text{ ksi}}{58\text{ ksi}} = 0.793 \le 0.8$ **o.k.**

Determine if a single-plate connection is suitable (HSS wall is not slender)

Manual Part 10 Table B4.1 Case 12

Slenderness: $\lambda = 1.40\sqrt{\dfrac{E}{F_y}} = 1.40\sqrt{\dfrac{29{,}000\text{ ksi}}{46\text{ ksi}}} = 35.2 < 48.7$ **n.g.**

Since the HSS6×4×⅛ is slender, a through-plate connection should be used instead of a single-plate connection. Through plate connections are typically very expensive. When a single-plate connection is not adequate, another type of connection, such as a double-angle connection may be preferable to a through-plate connection.

Calculate the required strength

LRFD	ASD
$R_u = 1.2(3.3 \text{ kips}) + 1.6(9.9 \text{ kips})$ $= 19.8 \text{ kips}$	$R_a = 3.3 \text{ kips} + 9.9 \text{ kips}$ $= 13.2 \text{ kips}$

Design the portion of the through-plate connection that resembles a single-plate

Try 3 rows of bolts (L=8½) and a ¼ in. plate thickness with 3⁄16-in. fillet welds.

Note: Either the plate or the beam web must satisfy: Manual Part 10

$t \le d_b/2 + 1/16 = 0.750 \text{ in.}/2 + 1/16 \text{ in.} = 0.438 \text{ in.}$

$1/4 \text{ in.} \le 0.438 \text{ in.}$ **o.k.**

Obtain the available single plate connection strength from Manual Table 10-9

Manual Table 10-9a

LRFD	ASD
$\phi R_n = 38.3 \text{ kips} > 19.8 \text{ kips}$ **o.k.**	$R_n / \Omega = 25.6 \text{ kips} > 13.2 \text{ kips}$ **o.k.**

Check weld size for through-plate connection

$e = 3.00 \text{ in.}$

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(3)}{58\text{ksi}} = 0.320 \text{ in.} > 0.250 \text{ in.}$$

Manual Part 9

Calculate the required weld strength

LRFD	ASD
$V_{fu} = \frac{R_u(B+e)}{B}$ $= \frac{(19.8 \text{ kips})(4.00 \text{ in.} + 3.00 \text{ in.})}{4.00 \text{ in.}}$ $= 34.7 \text{ kips}$ $v_{fu} = \frac{V_{fu}}{L} = \frac{34.7 \text{ kips}}{8.50 \text{ in.}} = 4.08 \text{ kips/in.}$	$V_{fa} = \frac{R_a(B+e)}{B}$ $= \frac{(13.2 \text{ kips})(4.00 \text{ in.} + 3.00 \text{ in.})}{4.00 \text{ in.}}$ $= 23.1 \text{ kips}$ $v_{fa} = \frac{V_{fa}}{L} = \frac{23.1 \text{ kips}}{8.50 \text{ in.}} = 2.72 \text{ kips/in.}$

Calculate the available weld strength

Manual Part 8

LRFD	ASD
$\phi r_n = 1.392 D n_w \left(\frac{t}{t_{\min}}\right)$ $= 1.392(3)(2 \text{ welds})\left(\frac{0.116 \text{ in.}}{0.320 \text{ in.}}\right)$ $= 3.03 \text{ kips/in.} < 4.08 \text{ kips/in.}$ **n.g.** A deeper plate is required.	$r_n/\Omega = 0.928 D n_w \left(\frac{t}{t_{\min}}\right)$ $= 0.928(3)(2 \text{ welds})\left(\frac{0.116 \text{ in.}}{0.320 \text{ in.}}\right)$ $= 2.02 \text{ kips/in.} < 2.72 \text{ kips/in.}$ **n.g.** A deeper plate is required.

LRFD	ASD
$L_{req} = \dfrac{V_{fu}}{\phi r_n} = \dfrac{34.7 \text{ kips}}{3.03 \text{ kips/in.}} = 11.5 \text{ in.}$	$L_{req} = \dfrac{V_{fa}}{r_n / \Omega} = \dfrac{23.1 \text{ kips}}{2.02 \text{ kips/in.}} = 11.4 \text{ in.}$

Use an 11½ in. long plate and increase the vertical edge distance to 2¾ in.

Recheck the plate length

L = 11½ in. < T = 15½ in. **o.k.**

Calculate the available beam web bearing strength

For three ¾ in. diameter bolts

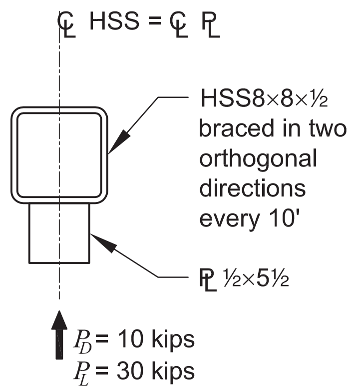
LRFD	ASD
$\phi R_n = (263 \text{ kips/in.})(0.300 \text{ in.})$ = 78.9 kips > 19.8 kips **o.k.**	$R_n / \Omega = (175 \text{ kips/in.})(0.300 \text{ in.})$ = 52.5 kips > 13.2 kips **o.k.**

Manual Table 10-1

Example K.8 Transverse Plate Loaded Perpendicular to the HSS Axis on a Rectangular HSS

Given:

Verify the local strength of the HSS column subject to the transverse loadings given below, applied through a 5½ in. wide plate. The HSS 8×8×½ is in compression with nominal axial loads of $P_{D\ column}$ = 54 kips and $P_{L\ column}$ = 162 kips. The HSS has negligible required flexural strength.



Solution:

Material Properties:

Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Manual
Plate	ASTM A36	F_{yp} = 36 ksi	F_u = 58 ksi	Table 2-3
				Table 2-4

Geometric Properties:

Column	HSS8×8×½	B = 8.00 in.	t = 0.465 in.	Manual
Plate		B_p = 5.50 in.	t_p = 0.500 in.	Table 1-12

Check the limits of applicability of Specification Section K1

Strength: $F_y = 46$ ksi ≤ 52 ksi for HSS	**o.k.**	Section K1.2
Ductility: $F_y/F_u = 0.793 \le 0.8$ for HSS	**o.k.**	
$0.25 < B_p/B \le 1.0$; $B_p/B = 0.688$	**o.k.**	Section K1.3b
$B/t = 17.2 \le 35$	**o.k.**	

Calculate the required strength

LRFD	ASD
Transverse force from the plate P_u = 1.2(10 kips) + 1.6(30 kips) = 60.0 kips *Column axial force* $P_r = P_{u\ column}$ = 1.2(54 kips) + 1.6(162 kips) = 324 kips	*Transverse force from the plate* P_a = 10 kips + 30 kips = 40.0 kips *Column axial force* $P_r = P_{a\ column}$ = 54 kips + 162 kips = 216 kips

Calculate available local yielding strength for uneven load distribution in the loaded plate

$$R_n = \left[\frac{10F_y t}{\left(\frac{B}{t}\right)}\right] B_p \le F_{yp} t_p B_p$$ Eqn. K1-2

$$R_n = \left[\frac{10(46 \text{ ksi})(0.465 \text{ in.})}{\left(\frac{8.00 \text{ in.}}{0.465 \text{ in.}}\right)}\right] 5.50 \text{ in.} \le 36 \text{ ksi}(0.500 \text{ in.})(5.50 \text{ in.})$$

$= 68.4$ kips $\le$ 99.0 kips **o.k.**

LRFD	ASD
$\phi = 0.95$	$\Omega = 1.58$
$\phi R_n = 0.95(68.4 \text{ kips}) = 65.0$ kips	$R_n / \Omega = \dfrac{68.4 \text{ kips}}{1.58} = 43.3$ kips
65.0 kips > 60.0 kips **o.k.**	43.3 kips > 40.0 kips **o.k.**

Section K1.3b

Check shear yielding (punching)

This limit state need not be checked when $B_p > B\text{-}2t$, nor when $B_p < 0.85B$. Section K1.3b(b)

$B\text{-}2t = 8.00 \text{ in.} – 2(0.465 \text{ in.}) = 7.07$ in.

$0.85B = 0.85(8.00 \text{ in.}) = 6.80$ in.

Therefore, since $B_p < 6.80$ in. this limit state does not control.

Check sidewall strength

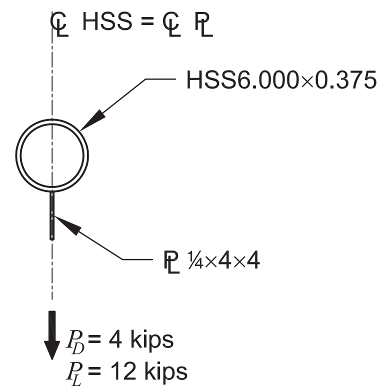
This limit state does not control unless the chord member and branch member (connecting element) have the same width. Section K1.3b(c)

Therefore, since $B_p / B < 1.0$ this limit state does not control.

Example K.9 Longitudinal Plate Loaded Perpendicular to the HSS Axis on a Round HSS

Given:

Verify the local strength of the HSS 6.000×0.375 tension chord subject to transverse loads, $P_D = 4$ kips and $P_L = 12$ kips, applied through a 4 in. wide plate.



Solution:

Material Properties:

Chord	ASTM A500 Gr. B	F_y = 42 ksi	F_u = 58 ksi	Manual Table 2-3
Plate	ASTM A36	F_{yp} = 36 ksi	F_u = 58 ksi	Table 2-4

Geometric Properties:

Chord	HSS6.000×0.375	D = 6.00 in.	t = 0.349 in.	Manual Table 1-13
Plate	PL¼ in.×4 in.× 4 in.			

Check the limits of applicability of Specification Section K1

1) Strength: F_y = 42 ksi $\leq$ 52 ksi for HSS **o.k.** Section K1.2
2) Ductility: F_y/F_u= 0.724 $\leq$ 0.8 for HSS **o.k.**
3) D/t =17.2 $\leq$ 50 for T-connections **o.k.** Section K1.4a

Calculate the required strength

LRFD	ASD
P_u = 1.2(4 kips) + 1.6(12 kips) = 24.0 kips	P_a = 4 kips + 12 kips = 16.0 kips

Check the limit state of chord plastification

$$R_n = 5.5F_y t^2 \left(1 + \frac{0.25N}{D}\right) Q_f$$ Eqn. K1-8

Since the chord is in tension, Q_f = 1.0 Section K2.2

$$R_n = (5.5)(42 \text{ ksi})(0.349 \text{ in.})^2 \left(1 + \frac{0.25(4.00 \text{ in.})}{6.00 \text{ in.}}\right)(1.0) = 32.8 \text{ kips}$$ Eqn. K1-8

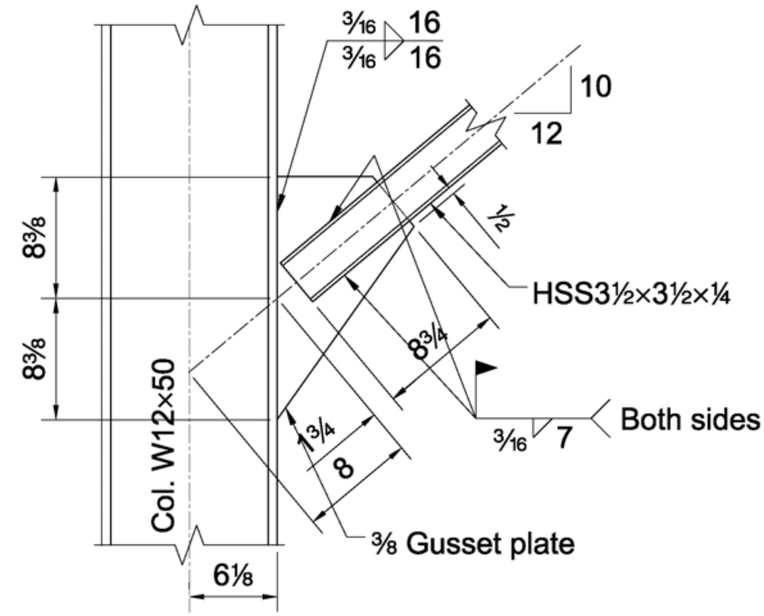
LRFD	ASD
$\phi R_n = 0.90(32.8\text{ kips}) = 29.5\text{ kips}$	$R_n / \Omega = \dfrac{32.8\text{ kips}}{1.67} = 19.6\text{ kips}$
29.5 kips > 24.0 kips **o.k.**	19.6 kips > 16.0 kips **o.k.**

Section K1.4a

Example K.10 HSS Brace Connection to a W-shape Column

Given:

Verify the strength of a HSS 3½×3½×¼ brace for required axial forces of 80 kips (LRFD) and 52 kips (ASD). The axial force may be either tension or compression. The length of the brace is 6 ft. Design the connection of the HSS brace to the gusset plate.



Solution:

Material Properties:

Brace	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Manual
Plate	ASTM A36	F_{yp} = 36 ksi	F_u = 58 ksi	Tables 2-3 & 2-4

Geometric Properties:

Brace	HSS 3½×3½×¼	A = 2.91 in.^{2}	r = 1.32 in.	t = 0.233 in.	Manual
Plate		t = 0.375 in.			Table 1-12

Obtain the available axial compression strength of the brace from Manual Table 4-4

K = 1.0
L_b = 6.00 ft

LRFD	ASD
$\phi_c P_n$ = 98.4 kips 98.4 kips > 80.0 kips **o.k.**	P_n/Ω_c = 65.4 kips 65.4 kips > 52.0 kips **o.k.**

Manual Table 4-4

Obtain the available tension yielding strength of the brace from Manual Table 5-5

LRFD	ASD
$\phi_t P_n$ = 120 kips 120 kips > 80.0 kips **o.k.**	P_n/Ω_t = 80.2 kips 80.2 kips > 52.0 kips **o.k.**

Manual Table 5-5

Calculate the tensile rupture strength of the brace

$A_n = A_g - 2(t)(\text{slot}) = 2.91 - 2(0.233)\left(\tfrac{3}{8} + \tfrac{1}{16}\right) = 2.71 \text{ in.}^2$

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{(3\tfrac{1}{2}\text{ in.})^2 + 2(3\tfrac{1}{2}\text{ in.})(3\tfrac{1}{2}\text{ in.})}{4(3\tfrac{1}{2}\text{ in.} + 3\tfrac{1}{2}\text{ in.})} = 1.31 \text{ in.}$$ Table D3.1

8¾ in. of overlap occurs. Try four $\tfrac{3}{16}$-in. fillet welds, each 7-in. long.

Based on Table J2.4 and the HSS thickness of ¼ in., the minimum weld is a ⅛ in. fillet weld.

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.31\text{ in.}}{7.00\text{ in.}} = 0.813$$

$A_e = UA_n = 0.813(2.71 \text{ in.}^2) = 2.20 \text{ in.}^2$ Eqn. D3-1

$P_n = F_u A_e = 58 \text{ ksi}(2.20 \text{ in.}^2) = 128 \text{ kips}$ Eqn. D2-2

Calculate the available tensile rupture strength

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(128 \text{ kips}) = 96.0 \text{ kips}$ $96.0 \text{ kips} > 80.0 \text{ kips}$ **o.k.**	$\Omega_t = 2.00$ $P_n/\Omega_t = (128 \text{ kips})/2.00 = 64.0 \text{ kips}$ $64.0 \text{ kips} > 52.0 \text{ kips}$ **o.k.**

Section D2

Calculate available strength of $\tfrac{3}{16}$-in. weld of HSS to plate

LRFD	ASD
$\phi R_n = 4(1.392Dl) = 4(1.392)(3 \text{ sixteenths})(7 \text{ in.})$ $= 117 \text{ kips} > 80.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 4(0.928Dl) = 4(0.928)(3 \text{ sixteenths})(7\text{in.})$ $= 78.0 \text{ kips} > 52.0 \text{ kips}$ **o.k.**

Manual Part 8

Check HSS shear rupture strength at welds

$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09(3)}{58} = 0.160 \text{ in.} < 0.233 \text{ in.}$$ **o.k.**

Check gusset plate shear rupture strength at welds

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(3)}{58} = 0.320 \text{ in.} < 0.375 \text{ in.}$$ **o.k.**

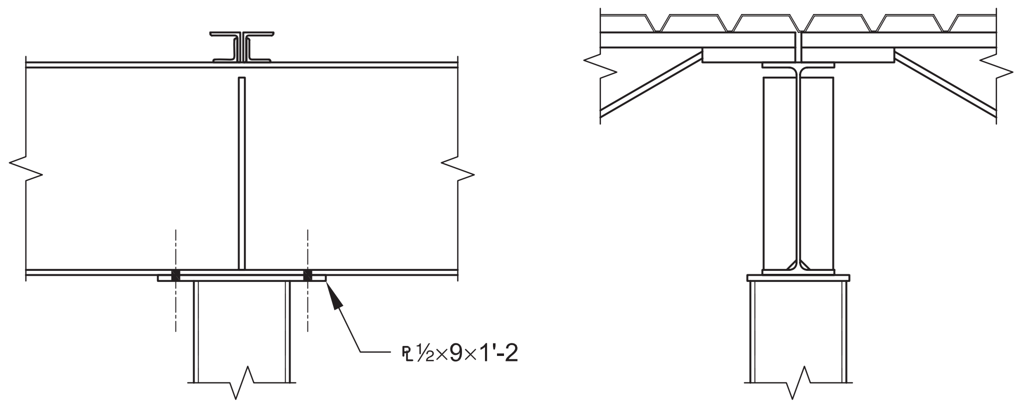
A complete check of the connection would also require consideration of the limit states of the other connection elements, such as:

- Whitmore buckling
- Local capacity of column web yield and crippling
- Yielding of gusset plate at gusset-to-column intersection

Example K.11 Rectangular HSS Column with a Cap Plate, Supporting a Continuous Beam

Given:

Verify the local strength of the HSS column subject to the given gravity beam reactions through the cap plate. Out of plane stability of the column top is provided by the beam web stiffeners; however, the stiffeners will be neglected in the column strength calculations. The column axial forces are R_D = 24 kips and R_L = 30 kips.



Solution:

Material Properties:

Beam	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Tables 2-3
Cap Plate	ASTM A36	F_{yp} = 36 ksi	F_u = 58 ksi	& 2-4

Geometric Properties:

Beam	W18×35	d = 17.7 in.	b_f = 6.00 in.	t_w = 0.300 in.	Manual
		t_f = 0.425 in.	k_1 = ¾ in.		Table 1-1
Column	HSS8×8×¼	t = 0.233 in.			Manual
Cap Plate		t = 0.500 in.			Table 1-12

Calculate the required strength

LRFD	ASD
R_u = 1.2(24.0 kips) + 1.6(30.0 kips) = 76.8 kips	R_a = 24.0 kips + 30.0 kips = 54.0 kips

Assume the vertical beam reaction is transmitted to the HSS through bearing of the cap plate at the two column faces perpendicular to the beam.

Calculate bearing length, N, at bottom of W18×35 — Section K1.6

Assume $N = 2k_1$ and the dispersed load width = $5t_p + N$. With $t_p = t_f$,

Dispersed load width = $5t_f + 2k_1$ = 5(0.425 in.) + 2(¾ in.) = 3.63 in.

Check limit for number of HSS faces contributing

$5t_p + N = 5(0.500\text{ in.}) + 3.63\text{ in.} = 6.13\text{ in.}$ < 8.00 in. therefore, only 2 walls contribute

Calculate the nominal local wall yielding strength of the HSS

For each of the two walls:

Eqn. K1-11

$$R_n = F_y t(5t_p + N) \le BF_y t$$
$$= 46\text{ ksi}(0.233\text{ in.})\left[5(0.500\text{ in.}) + 3.63\text{ in}\right] \le 8.00\text{ in.}(46\text{ ksi})(0.233\text{ in.})$$
$$65.7\text{ kips} \le 85.7\text{ kips}$$

Use R_n = 65.7 kips per wall

Calculate the available local wall yielding strength of the HSS

LRFD	ASD
ϕ= 1.00 ϕR_n = 1.00(65.7 kips)(2 walls) = 131 kips > 76.8 kips **o.k.**	Ω = 1.50 R_n/Ω = (65.7 kips/1.50)(2 walls) = 87.6 kips > 54.0 kips **o.k.**

Section K1.6

Calculate the nominal wall local crippling strength of the HSS

Section K1.6

For each of the two walls:

Eqn. K1-12

$$R_n = 0.8t^2\left[1 + (6N/B)(t/t_p)^{1.5}\right]\left[EF_y t_p / t\right]^{0.5}$$

$$R_n = 0.8(0.233\text{ in.})^2\left[1 + \left(\frac{6(3.63\text{ in.})}{8.00\text{ in.}}\right)\left(\frac{0.233\text{ in.}}{0.500\text{ in.}}\right)^{1.5}\right]\left[\frac{29{,}000\text{ ksi}(46\text{ ksi})(0.500\text{ in.})}{0.233\text{ in.}}\right]^{0.5}$$

= 137 kips per wall

Calculate the available wall local crippling strength of the HSS

LRFD	ASD
ϕ = 0.75 ϕR_n = 0.75(137 kips)(2 walls) = 206 kips > 76.8 kips **o.k.**	Ω = 2.00 R_n/Ω = (137 kips/2.00)(2 walls) = 137 kips > 54.0 kips **o.k.**

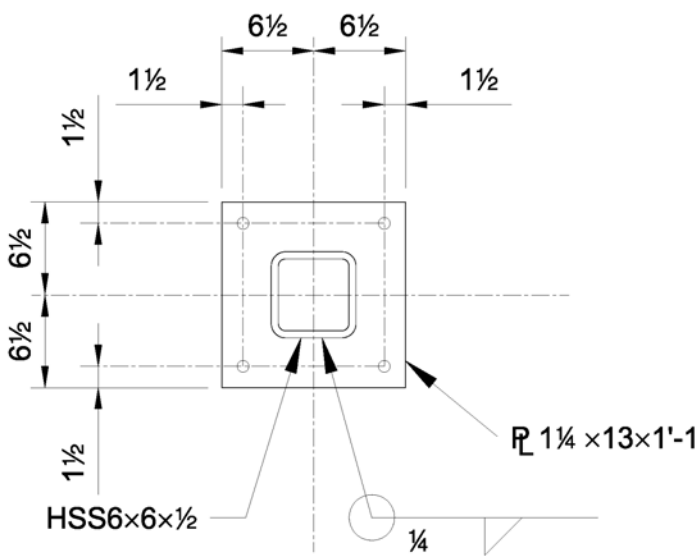
Section K1.6

Note: This example illustrates the application of the relevant provisions of Chapter K of the Specification. Other limit states should also be checked to complete the design.

Example K.12 Rectangular HSS Column Base Plate

Given:

A HSS 6×6×½ column is supporting nominal loads of 40 kips of dead load and 120 kips of live load. The column is supported by a 7 ft-6 in. × 7 ft-6 in. concrete spread footing with f'_c = 3,000 psi. Size a base plate for this column.



Solution:

Material Properties:

Column	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Manual
Base Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	Tables 2-3 & 2-4

Geometric Properties:

HSS 6×6×½ B = H = 6.00 in.

Calculate the required strength

LRFD	ASD
P_u = 1.2(40 kips) + 1.6(120 kips) = 240 kips	P_a = 40 kips + 120 kips = 160 kips

Note: The procedure illustrated here is similar to that presented in *AISC Design Guide 1-Column Base Plates* and Part 14 of the Manual.

Try a base plate which extends 3½ in. from each face of the HSS column, or 13 in. by 13 in.

Calculate the available strength for the limit state of concrete crushing

On less than the full area of a concrete support, Eqn. J8-2

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \;\leq\; 1.7 f_c' A_1$$

A_1 = (13.0.in.)(13.0 in.) = 169 in.^{2}

A_2 = (90.0.in.)(90.0 in.) = 8,100 in.^{2}

$$P_p = 0.85(3 \text{ ksi})(169 \text{ in.}^2)\sqrt{\frac{8{,}100 \text{ in.}^2}{169 \text{ in.}^2}} \leq 1.7(3 \text{ ksi})(169 \text{ in.}^2)$$

$$= 2{,}980 \text{ kips} \leq 862 \text{ kips}$$

Use P_p = 862 kips

Note: The limit on the right side of Equation J8-2 will control when A_2 / A_1 exceeds 4.0.

LRFD	ASD	
ϕ_c = 0.60 $\phi_c P_p$ = 0.60(862 kips) = 517 kips > 240 kips **o.k.**	Ω_c = 2.50 P_p / Ω_c = 862 kips/2.50 = 345 kips > 160 kips **o.k.**	Section J8

Calculate the pressure under the bearing plate and determine the required thickness

For a rectangular HSS, the distance m or n is determined using 0.95 times the depth and width of the HSS.

$$m = n = \frac{N \text{ - } 0.95(\text{outside dimension})}{2}$$

The critical bending moment is the cantilever moment outside the HSS perimeter. Therefore, $m = n = l$.

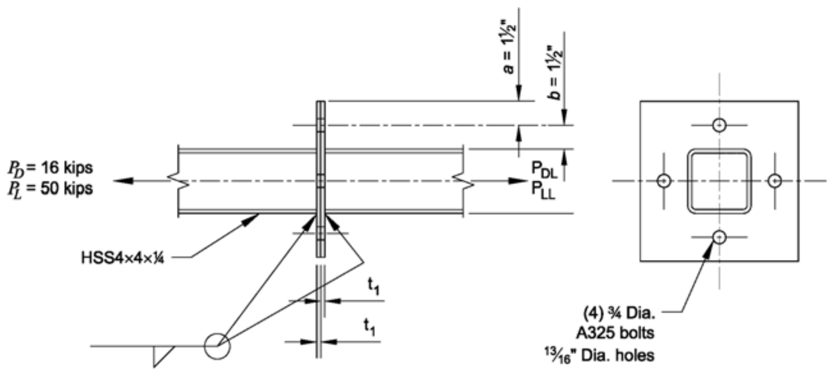
LRFD	ASD	
$f_{pu} = \frac{P_u}{A} = \frac{240 \text{ kips}}{169 \text{ in.}^2} = 1.42 \text{ ksi}$	$f_{pa} = \frac{P_a}{A} = \frac{160 \text{ kips}}{169 \text{ in.}^2} = 0.947 \text{ ksi}$	
$M_u = \frac{f_{pu} l^2}{2}$	$M_a = \frac{f_{pa} l^2}{2}$	
$Z = \frac{t_p^{\,2}}{4}$	$Z = \frac{t_p^{\,2}}{4}$	
ϕ_b = 0.90	Ω_b = 1.67	
$M_n = M_p = F_y Z$	$M_n = M_p = F_y Z$	Eqn. F11-1
Equating:	Equating:	
$M_u = \phi_b M_n$ and solving for t_p gives: $t_{p(req)} = \sqrt{\frac{2 f_{pu} l^2}{\phi_b F_y}}$ $= \sqrt{\frac{2(1.42 \text{ ksi})(3.65 \text{ in.})^2}{0.90(36 \text{ ksi})}} = 1.08$ in.	$M_a = M_n / \Omega_b$ and solving for t_p gives: $t_{p(req)} = \sqrt{\frac{2 f_{pa} l^2}{F_y / \Omega_b}}$ $= \sqrt{\frac{2(0.947 \text{ ksi})(3.65 \text{ in.})^2}{(36 \text{ ksi})/1.67}} = 1.08$ in.	

Therefore, use a 1¼ in. thick base plate.

Example K.13 Rectangular HSS Strut End Plate

Given:

Determine the weld leg size, end plate thickness, and the size of ASTM A325 bolts required to resist nominal forces of 16 kips from dead load and 50 kips from live load on an ASTM A500 Gr. B HSS4×4×¼ section. The end plate is ASTM A36. Use 70-ksi weld electrodes.



Solution:

Material Properties:

Strut	ASTM A500 Gr. B	F_y = 46 ksi	F_u = 58 ksi	Manual
End Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	Tables 2-3 & 2-4

Geometric Properties:

Strut	HSS 4×4×¼	t = 0.233 in.	A = 3.37 in.^{2}	Manual
End Plate		$b = a$ = 1.50 in.		Table 1-12

Calculate the required tensile strength

LRFD	ASD
P_u = 1.2(16 kips) + 1.6(50 kips) = 99.2 kips	P_a = 16 kips + 50 kips = 66.0 kips

Preliminary size of the (4) ASTM A325 bolts

LRFD	ASD
$r_{ut} = \dfrac{P_u}{n} = \dfrac{99.2\text{ kips}}{4} = 24.8\text{ kips}$ Try ¾-in. diameter ASTM A325 bolts ϕr_n = 29.8 kips	$r_{at} = \dfrac{P_a}{n} = \dfrac{66.0\text{ kips}}{4} = 16.5\text{ kips}$ Try ¾-in. diameter ASTM A325 bolts r_n / Ω = 19.9 kips

Manual Table 7-2

Calculate required end-plate thickness with consideration of prying action

$$a' = a + \frac{d_b}{2} \le 1.25b + \frac{d_b}{2}$$

Manual Part 9

$$= 1.50 \text{ in.} + \frac{3/4 \text{ in.}}{2} \le 1.25(1.50 \text{ in.}) + \frac{3/4 \text{ in.}}{2}$$

$$= 1.88 \text{ in.} \le 2.25 \text{ in.} \qquad \textbf{o.k.}$$

$$b' = b - \frac{d_b}{2} = 1.5 \text{ in.} - \frac{3/4 \text{ in.}}{2} = 1.13 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.13}{1.88} = 0.601$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{13/16 \text{ in.}}{4.00 \text{ in.}} = 0.797$$

LRFD	ASD
$\beta = \frac{1}{\rho}\left(\frac{\phi r_n}{r_{ut}} - 1\right) = \frac{1}{0.601}\left(\frac{29.8}{24.8} - 1\right) = 0.336$ $\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right)$ $= \frac{1}{0.797}\left(\frac{0.336}{1-0.336}\right) = 0.635 \le 1.0$	$\beta = \frac{1}{\rho}\left(\frac{r_n/\Omega}{r_{at}} - 1\right) = \frac{1}{0.601}\left(\frac{19.9}{16.5} - 1\right) = 0.343$ $\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right)$ $= \frac{1}{0.797}\left(\frac{0.343}{1-0.343}\right) = 0.655 \le 1.0$

Manual Part 9

Use the equation for t_{req} in Chapter 9 of the Manual, except that F_u is replaced by F_y per recommendation of Willibald, Packer and Puthli (2003)[1]. The tributary length per bolt,

$$p = (\text{full plate width})/(\text{number of bolts per side})$$

$$= 10.0 \text{ in}/1 = 10.0 \text{ in.}$$

LRFD	ASD
$t_{req} = \sqrt{\frac{4.44 r_{ut} b'}{pF_y(1+\delta\alpha')}}$ $= \sqrt{\frac{4.44(24.8 \text{ kips})(1.13 \text{ in.})}{10.0 \text{ in.}(36 \text{ ksi})(1+0.797(0.635))}}$ $= 0.479$ in. Use $1/2$ in. end plate, $t_1 > 0.479$, further bolt check for prying not required. Use (4) $3/4$ in. diameter A325 bolts	$t_{req} = \sqrt{\frac{6.66 r_{at} b'}{pF_y(1+\delta\alpha')}}$ $= \sqrt{\frac{6.66(16.5 \text{ kips})(1.13 \text{ in.})}{10.0 \text{ in.}(36 \text{ ksi})(1+0.797(0.655))}}$ $= 0.476$ in. Use $1/2$ in. end plate, $t_1 > 0.476$, further bolt check for prying not required. Use (4) $3/4$ in. diameter A325 bolts

Manual Part 9

Calculate the weld size required

$R_n = F_w A_w$ Eqn. J2-3

[1] *Willibald, S., Packer, J.A., and Puthli (2003), "Design recommendations for Bolted Rectangular HSS Flange-PlateConnections in Axial Tension", Engineering Journal, AISC,*

$$F_w = 0.60F_{exx}$$
$$= 0.60(70 \text{ ksi})$$
$$= 42.0 \text{ ksi}$$

Table J2.5

$$l = 4(4.00 \text{ in.}) = 16.0 \text{ in.}$$

LRFD	ASD
For shear load on fillet welds $\phi = 0.75$ $w \geq \dfrac{P_u}{\phi F_w (0.707) l}$ $\geq \dfrac{99.2 \text{ kips}}{0.75(42.0 \text{ ksi})(0.707)(16.0 \text{ in.})}$ $= 0.278$ in.	For shear load on fillet welds $\Omega = 2.00$ $w \geq \dfrac{\Omega P_a}{F_w (0.707) l}$ $\geq \dfrac{2.00(66.0 \text{ kips})}{(42.0 \text{ ksi})(0.707)(16.0 \text{ in.})}$ $= 0.278$ in.

Table J2.5

Manual Part 8

Try $w = \frac{5}{16}$ in.

Check minimum weld size requirements

For t = ¼ in., the minimum weld size = ⅛ in.

Table J2.4

LRFD	ASD
5⁄16 in. > 0.278 in. Use $\frac{5}{16}$-in. weld leg size	5⁄16 in. > 0.278 in. Use $\frac{5}{16}$-in. weld leg size

Results

Use $\frac{5}{16}$-in. weld with $\frac{1}{2}$-in. end plate and (4) ¾-in.-diameter ASTM A325 bolts.

Chapter IIA
Simple Shear Connections

The design of simple shear connections is covered in Part 10 of the AISC *Steel Construction Manual*.

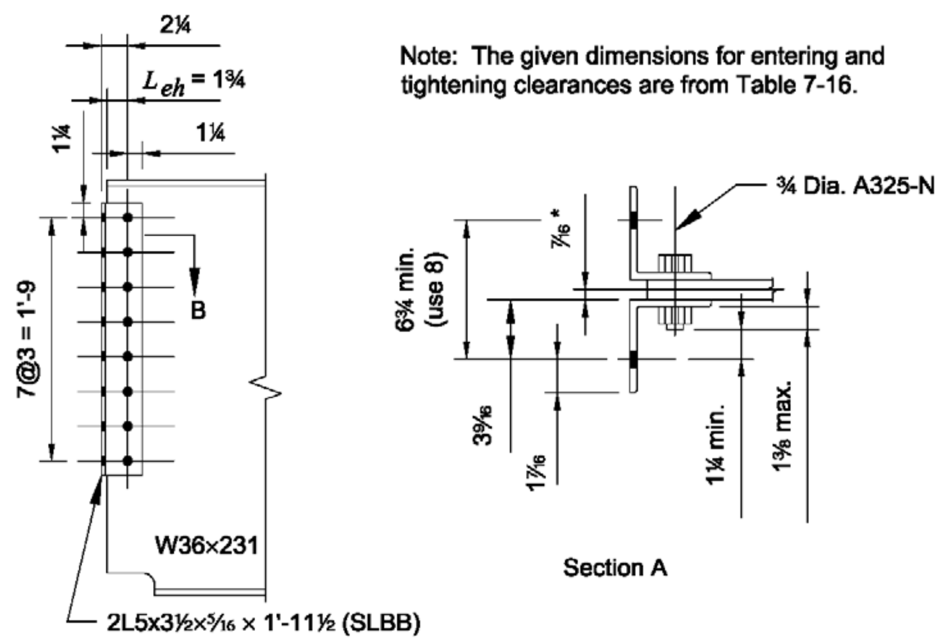
Example II.A-1 All-Bolted Double-Angle Connection

Given:

Select an all-bolted double-angle connection between a W36×231 beam and a W14×90 column flange to support the following beam end reactions:

R_D = 37.5 kips
R_L = 113 kips

Use ¾-in. diameter ASTM A325-N or F1852-N bolts in standard holes.



* This dimension (see sketch, Section A) is determined as one-half of the decimal web thickness rounded to the next higher ¹⁄₁₆ in. Example: 0.355/2 = 0.178; use ³⁄₁₆ in. This will produce spacing of holes in the supporting beam slightly larger than detailed in the angles to permit spreading of angles (angles can be spread but not closed) at time of erection to supporting member. Alternatively, consider using horizontal short slots in the support legs of the angles.

Manual Figure 10-5

Material Properties:

Beam	W36×231	ASTM	A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Column	W14×90	ASTM	A992	F_y = 50 ksi	F_u = 65 ksi	
Angles	2L5×3½	ASTM	A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W36×231	t_w = 0.760 in.	Manual Table 1-1
Column	W14×90	t_f = 0.710 in.	
Angles	2L5×3½ SLBB		

Solution:

LRFD	ASD	
Calculate the required strength R_u = 1.2(37.5 kips) + 1.6(113 kips) = 226 kips *Manual Table 10-1 includes checks for the limit states of bearing, shear yielding, shear rupture, and block shear rupture on the angles, and shear on the bolts.* Try 8 rows of bolts and $^5/_{16}$–in. angle thickness. ϕR_n = 247 kips > 226 kips **o.k.**	*Calculate the required strength* R_a = 37.5 kips + 113 kips = 151 kips *Manual Table 10-1 includes checks for the limit states of bearing, shear yielding, shear rupture, and block shear rupture on the angles, and shear on the bolts.* Try 8 rows of bolts and $^5/_{16}$–in. angle thickness. $R_n / \Omega = 165$ kips > 151 kips **o.k.**	Manual Table 10-1
Check the beam web for bolt bearing Uncoped, L_{eh} = 1¾ in. ϕR_n = (702 kips/in.)(0.760 in.) = 534 kips > 226 kips **o.k.**	*Check the beam web for bolt bearing* Uncoped, L_{eh} = 1¾ in. R_n / Ω = (468 kips/in.)(0.760 in.) = 356 kips > 151 kips **o.k.**	Manual Table 10-1
Check supporting member flange for bolt bearing ϕR_n = (1400 kips/in.)(0.710 in.) = 994 kips > 226 kips **o.k.**	*Check supporting member flange for bolt bearing* R_n / Ω = (936 kips/in.)(0.710 in.) = 665 kips > 151 kips **o.k.**	Manual Table 10-1

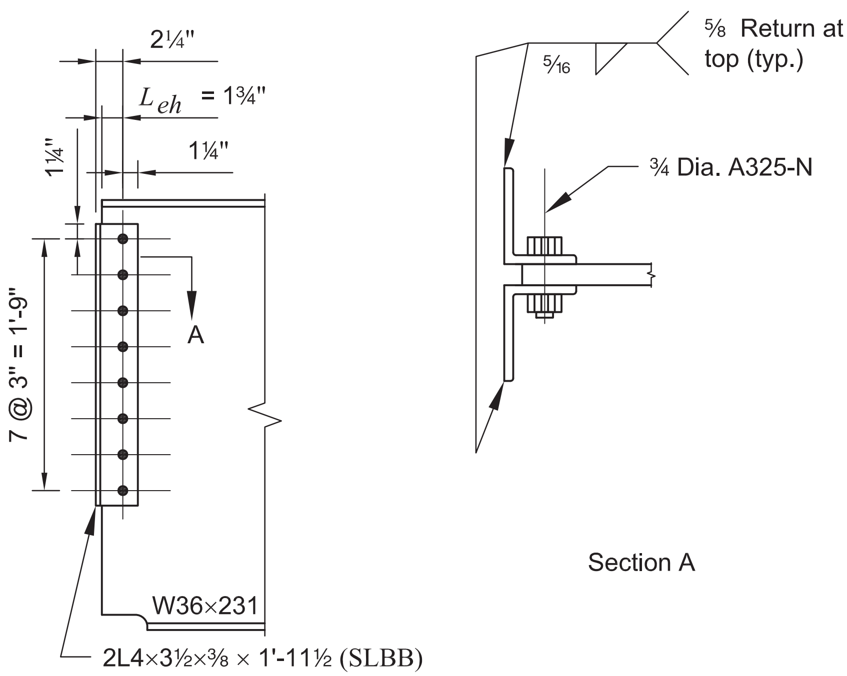
Note: In this example, because of the relative size of the cope to the overall beam size, the coped section does not control. When this cannot be determined by inspection, see Manual Part 9 for the design of the coped section.

See **Example II.A-2** for a bolted/welded double angle connection.

Example II.A-2 Bolted/Welded Double-Angle Connection

Given:

Repeat Example II.A-1 using Manual Table 10-2 to substitute welds for bolts in the support legs of the double-angle connection (welds B). Use 70 ksi electrodes.



Note: Bottom flange coped for erection.

Material Properties:

Beam	W36×231	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Column	W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles	2L4×3½	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W36×231	t_w = 0.760 in.	Manual Table 1-1
Column	W14×90	t_f = 0.710 in.	
Angles	2L4×3½ SLBB		

Solution:

LRFD	ASD
Calculate the required strength R_u = 1.2(37.5 kips) + 1.6(113 kips) = 226 kips *Design welds (welds B)* Try $\frac{5}{16}$-in. weld size, L = 23½ in. $t_{f\,min}$ = 0.238 in. < 0.710 in. **o.k.** ϕR_n = 335 kips > 226 kips **o.k.**	*Calculate the required strength* R_a = 37.5 kips + 113 kips = 151 kips *Design welds (welds B)* Try $\frac{5}{16}$-in. weld size, L = 23½ in. $t_{f\,min}$ = 0.238 in. < 0.710 in. **o.k.** $R_n / \Omega = 223$ kips > 151 kips **o.k.**
Check minimum angle thickness for weld $t_{min} = w + \frac{1}{16}$-in. = $\frac{5}{16}$ in. + $\frac{1}{16}$ in. = ⅜ in. > $\frac{5}{16}$ in. **o.k.** Use 2L4×3½×⅜	*Check minimum angle thickness for weld* $t_{min} = w + \frac{1}{16}$-in. = $\frac{5}{16}$ in. + $\frac{1}{16}$ in. = ⅜ in. > $\frac{5}{16}$ in. **o.k.** Use 2L4×3½×⅜

Manual Table 10-2

Section J2.2b

Note: The support for the connection must meet the minimum thickness indicated in Table 10-2 for the weld.

LRFD	ASD
Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture and block shear rupture Check 8 rows of bolts and ⅜ in. angle thickness ϕR_n = 254 kips > 226 kips **o.k.** *Check beam web for bolt bearing.* Uncoped, L_{eh} = 1¾ in. ϕR_n = (702 kips/in.)(0.760 in.) = 534 kips > 226 kips **o.k.**	*Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture and block shear rupture* Check 8 rows of bolts and ⅜ in. angle thickness $R_n / \Omega = 170$ kips > 151 kips **o.k.** *Check beam web for bolt bearing.* Uncoped, L_{eh} = 1¾ in. $R_n / \Omega = (468$ kips/in.)(0.760 in.) = 356 kips > 151 kips **o.k.**

Manual Table 10-1

Manual Table 10-1

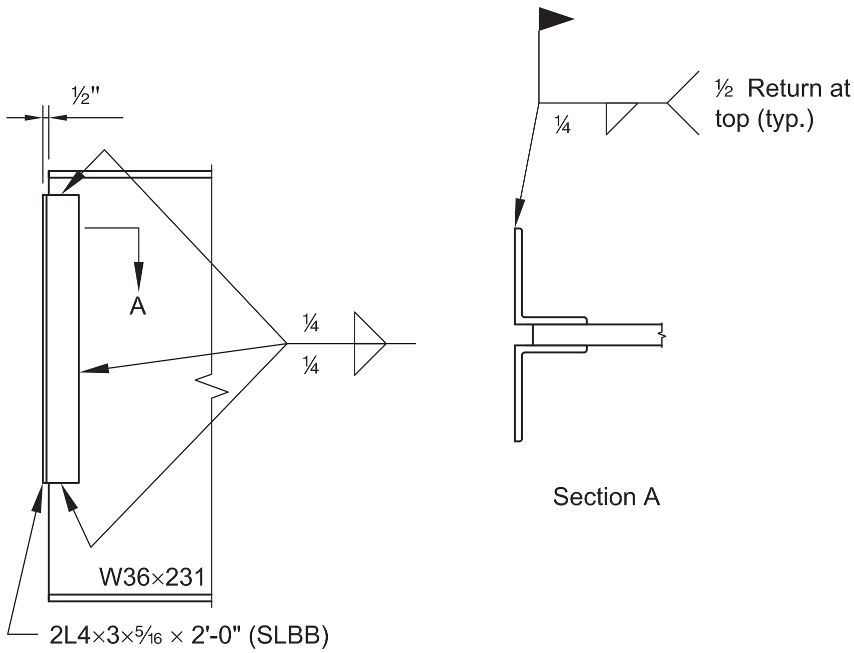
Note: In this example, because of the relative size of the cope to the overall beam size, the coped section does not control. When this cannot be determined by inspection, see Manual Part 9 for the design of the coped section.

See **Example II.A-1** for an all-bolted double-angle connection (beam-to-column flange).

Example II.A-3 All-Welded Double-Angle Connection

Given:

Repeat Example II.A-1 using Manual Table 10-3 to design an all-welded double-angle connection between a W36×231 beam and a W14×90 column flange.
Use 70 ksi electrodes.



Material Properties:

Beam	W36×231	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Column	W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles	2L4×3	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W36×231	t_w = 0.760 in.	Manual Tables 1-1
Column	W14×90	t_f = 0.710 in.	
Angles	2L4×3 SLBB		

Solution:

LRFD	ASD	
Calculate the required strength R_u = 1.2(37.5 kips) + 1.6(113 kips) = 226 kips *Design the weld between the beam-web and the angle leg (welds A)* Try $^3\!/_{16}$ in. weld size, L = 24 in. $t_{w\,min}$ = 0.286 in. < 0.760 in. **o.k.** ϕR_n = 258 kips > 226 kips **o.k.**	*Calculate the required strength* R_a = 37.5 kips + 113 kips = 151 kips *Design the weld between the beam-web and the angle leg (welds A)* Try $^3\!/_{16}$ in. weld size, L = 24 in. $t_{w\,min}$ = 0.286 in. < 0.760 in. **o.k.** R_n / Ω = 172 kips > 151 kips **o.k.**	Manual Table 10-3
Design the welds between support and the angle leg (welds B) Try ¼ in. weld size, L = 24 in. $t_{f\,min}$ = 0.190 in. < 0.710 in. **o.k.** ϕR_n = 229 kips > 226 kips **o.k.**	*Design the welds between support and the angle leg (welds B)* Try ¼ in. weld size, L = 24 in. $t_{f\,min}$ = 0.190 in. < 0.710 in. **o.k.** R_n / Ω = 153 kips > 151 kips **o.k.**	Manual Table 10-3
Check the minimum angle thickness for weld $t_{min} = w + {}^1\!/_{16}$ in. = ¼ in. + $^1\!/_{16}$ in. = $^5\!/_{16}$ in. **o.k.** Use 2L4×3×$^5\!/_{16}$	*Check the minimum angle thickness for weld* $t_{min} = w + {}^1\!/_{16}$ in. = ¼ in. + $^1\!/_{16}$ in. = $^5\!/_{16}$ in. **o.k.** Use 2L4×3×$^5\!/_{16}$	Section J2.2b

Note: The support for the connection must meet the minimum thickness indicated in Table 10-2 for the weld.

Check angles for shear yielding — Section J4.2

$$A_g = 2(24.0\text{ in.})(0.313\text{ in.}) = 15.0\text{ in.}^2$$

$$R_n = 0.6F_yA_g = 0.6(36\text{ ksi})(15.0\text{ in.}^2) = 324\text{ kips}$$ Eqn. J4-3

LRFD	ASD
ϕ = 1.00 ϕR_n = 1.00(324 kips) = 324 kips > 226 kips **o.k.**	Ω = 1.5 R_n/Ω = 324 kips/1.50 = 216 kips > 151 kips **o.k.**

Comment:

See **Example II.A-1** for an all-bolted double-angle connection and **Example II.A-2** for a bolted/welded double-angle connection.

Example II.A-4 All-Bolted Double-Angle Connection in a Coped Beam

Given:

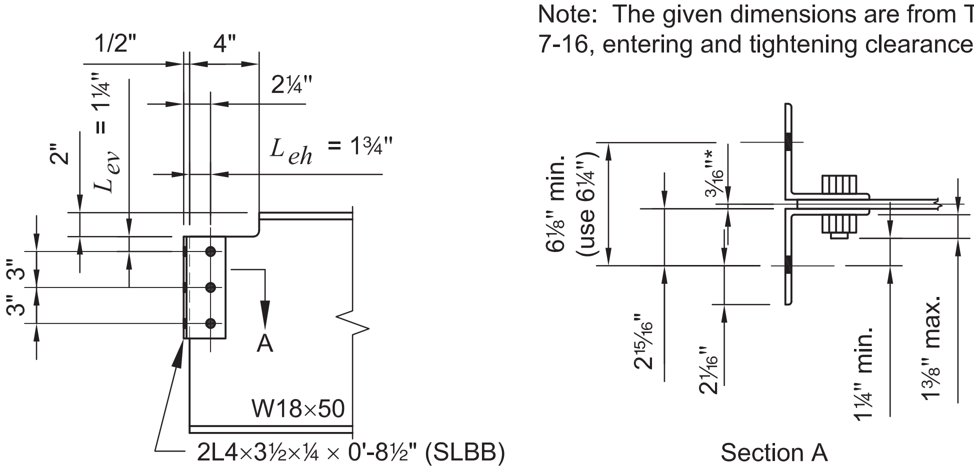
Use Manual Table 10-1 to select an all-bolted double-angle connection between a W18×50 beam and a W21×62 girder web to support the following beam end reactions:

R_D = 10 kips
R_L = 30 kips

The beam top flange is coped 2-in. deep by 4-in. long, L_{ev} = 1¼ in., L_{eh} = 1¾ in.

Use ¾-in. diameter ASTM A325-N or F1852-N bolts in standard holes.

Note: The given dimensions are from Table 7-16, entering and tightening clearances.



* This dimension is one-half decimal web thickness rounded to the next higher ⅟16 in., as in Example II.A-1.

Material Properties:

Beam	W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Girder	W21×62	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles	2L4×3½	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W18×50	d = 18.0 in.	t_w = 0.355 in.	S_{net} = 23.4 in.^{3}	Manual Tables 1-1 and 9-2
Girder	W21×62	t_w = 0.400 in.			
Angles	2L4×3 ½ SLBB				
Beam W18×50 Cope		c = 4.00 in. h_0 = 16.0 in.	d_c = 2.00 in.	e = 4.00 in. + 0.500 in. = 4.50 in.	Manual Figure 9-2

Solution:

LRFD	ASD
Calculate required strength R_u = 1.2(10 kips) + 1.6(30 kips) = 60.0 kips *Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture and block shear rupture* Try 3 rows of bolts and ¼ in. angle thickness ϕR_n = 76.4 kips > 60.0 kips **o.k.**	*Calculate required strength* R_a = 10 kips + 30 kips = 40.0 kips *Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture and block shear rupture* Try 3 rows of bolts and ¼ in. angle thickness R_n / Ω = 50.9 kips > 40.0 kips **o.k.**
Check beam web for bolt bearing and block shear rupture Top flange coped, L_{ev} = 1¼ in, L_{eh} = 1¾ in ϕR_n = (200 kips/in.)(0.355 in.) = 71.0 kips > 60.0 kips **o.k.**	*Check beam web for bolt bearing and block shear rupture* Top flange coped, L_{ev} = 1¼ in, L_{eh} = 1¾ in R_n / Ω = (133 kips/in.)(0.355 in.) = 47.2 kips > 40.0 kips **o.k.**
Check supporting member web for bolt bearing ϕR_n = (526 kips/in.)(0.400) = 210 kips > 60.0 kips **o.k.**	*Check supporting member web for bolt bearing* R_n / Ω = (351 kips/in.)(0.400 in.) = 140 kips > 40.0 kips **o.k.**

Manual Table 10-1

Manual Table 10-1

Manual Table 10-1

Note: The middle portion of Manual Table 10-1 includes checks of the limit-state of bolt bearing on the beam web and the limit-state of block shear rupture on coped beams. Manual Tables 9-3a, 9-3b and 9-3c may be used to determine the available block shear strength for values of L_{ev} and L_{eh} beyond the limits of Table 10-1. For coped members, the limit states of flexural yielding and local buckling must be checked independently per Part 9.

Manual Table 10-1

Check flexural rupture on the coped section

S_{net} = 23.4 in^3

Manual Table 9-2

$$R_n = \frac{F_u S_{net}}{e} = \frac{(65\text{ ksi})(23.4\text{ in.}^3)}{4.50\text{ in.}} = 338\text{ kips}$$

Manual Part 9

LRFD	ASD
$\phi = 0.75$ ϕR_n = 0.75(338 kips) = 254 kips > 60.0 kips **o.k.**	$\Omega = 2.00$ $R_n / \Omega = 338\text{ kips}/2.00$ $= 169\text{ kips} > 40.0\text{ kips}$ **o.k.**

Check yielding and local web buckling at the coped section

Verify $c \le 2d$ and $d_c \le \dfrac{d}{2}$

Manual Part 9 Page 9-7

$c = 4.00\text{ in.} \le 2(18.0\text{ in.}) = 36.0\text{ in.}$ **o.k.**

$d_c = 2.00\text{ in.} \le \dfrac{18.0\text{ in.}}{2} = 9.00\text{ in.}$ **o.k.**

$$\frac{c}{d} = \frac{4.00 \text{ in.}}{18.0 \text{ in.}} = 0.222;\ \ \frac{c}{h_0} = \frac{4.00 \text{ in.}}{16.0 \text{ in.}} = 0.250$$

Since $\frac{c}{d} \le 1.0$,

$$f = 2\left(\frac{c}{d}\right) = 2\left(0.222\right) = 0.444$$

Since $\frac{c}{h_0} \le 1.0$,

$$k = 2.2\left(\frac{h_0}{c}\right)^{1.65} = 2.2\left(\frac{16.0 \text{ in.}}{4.00 \text{ in.}}\right)^{1.65} = 21.7$$

$$F_{cr} = 26{,}210\left(\frac{t_w}{h_0}\right)^2 fk = 26{,}210\left(\frac{0.355 \text{ in.}}{16.0 \text{ in.}}\right)^2 \left(0.444\right)\left(21.7\right) = 124 \text{ ksi} \le F_y$$

Use $F_{cr} = F_y = 50$ ksi

$$R_n = \frac{F_{cr} S_{net}}{e} = \frac{\left(50 \text{ ksi}\right)\left(23.4 \text{ in.}^3\right)}{4.50 \text{ in.}} = 260 \text{ kips}$$ Manual Part 9

LRFD	ASD
$\phi = 0.90$ $\phi R_n =$ 0.90(260 kips) = 234 kips > 60.0 kips **o.k.**	$\Omega = 1.67$ $R_n / \Omega = 260 \text{ kips}/1.67$ $= 156 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Check shear yielding on beam web

$R_n = 0.6\ F_y\ A_g$ = 0.6(50 ksi)(0.355 in.)(16.0 in) = 170 kips Eqn J4-3

LRFD	ASD
$\phi = 1.00$ $\phi R_n =$ 1.00(170 kips) = 170 kips > 60.0 kips **o.k.**	$\Omega = 1.50$ $R_n / \Omega = 170 \text{ kips}/1.50$ $= 113 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Section J4.2

Check shear rupture on beam web

$$A_{nv} = t_w\left[h_o - 3\left(0.875 \text{ in.}\right)\right] = \left(0.355 \text{ in.}\right)\left(16.0 \text{ in.} - 2.63 \text{ in.}\right) = 4.75 \text{in.}^2$$ Eqn J4-4

$$R_n = 0.6\ F_u A_{nv} = 0.6\left(65.0 \text{ ksi}\right)\left(4.75 \text{ in.}^2\right) = 185 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n =$ 0.75(185 kips) = 139 kips > 60.0 kips **o.k.**	$\Omega = 2.00$ $R_n / \Omega = 185 \text{ kips}/2.00$ = 92.5 kips > 40.0 kips **o.k.**

Section J4.2

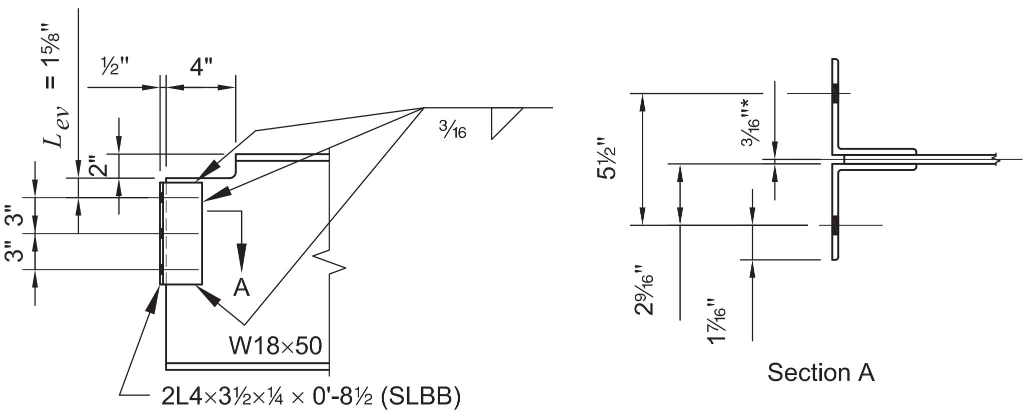
Note: see **Example II.A-5** for a bolted/welded double-angle connection.

Example II.A-5 Welded/Bolted Double-Angle Connection (beam-to-girder web)

Given:

Repeat Example II.A-4 using Manual Table 10-2 to substitute welds for bolts in the supported-beam-web legs of the double-angle connection (welds A).

Use 70 ksi electrodes.
Use ¾-in. diameter ASTM A325-N or F1852-N bolts in standard holes.



* This dimension is one-half decimal web thickness rounded to the next higher 1⁄16 in., as in Example II.A-1.

Material Properties:

Beam	W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Girder	W21×62	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles	2L4×3½	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W18×50	d = 18.0 in.	t_w = 0.355 in.	S_{net} = 23.4 in.^{3}	Manual Tables 1-1 & 9-2
Girder	W21×62	t_w = 0.400 in.			
Angles	2L4×3½ SLBB				
Beam	W18×50 Cope	c = 4.00 in. h_0 = 16.0 in.	d_c = 2.00 in.	e = 4.00 in. + 0.500 in. = 4.50 in.	Manual Figure 9-2

Solution:

LRFD	ASD	
Calculate required strength R_u = 1.2(10 kips) + 1.6(30 kips) = 60.0 kips *Design welds (welds A)* Try 3/16-in. weld size, L = 8½ in. $t_{w\,min}$ = 0.286 in. < 0.355 in. **o.k.** ϕR_n = 110 kips > 60.0 kips **o.k.**	*Calculate required strength* R_a = 10 kips + 30 kips = 40.0 kips *Design welds (welds A)* Try 3/16-in. weld size, L = 8½ in. $t_{w\,min}$ = 0.286 in. < 0.355 in. **o.k.** R_n / Ω = 73.4 kips > 40.0 kips **o.k.**	Manual Table 10-2
Check minimum angle thickness for weld w = weld size $t_{min} = w$ + 1/16-in. = 3/16 in. + 1/16 in. = ¼ in. **o.k.**	*Check minimum angle thickness for weld* w = weld size $t_{min} = w$ + 1/16-in. = 3/16 in. + 1/16 in. = ¼ in. **o.k.**	Section J2.2b

LRFD	ASD	
Check supporting member web for bolt bearing $\phi R_n = (526\text{ kips/in.})(0.400\text{ in.})$ $= 210\text{ kips} > 60.0\text{ kips}$ **o.k.**	*Check supporting member web for bolt bearing* $R_n / \Omega = (351\text{ kips/in.})(0.400\text{ in.})$ $= 140\text{ kips} > 40.0\text{ kips}$ **o.k.**	Manual Table 10-1
Check bolts and angle for bearing, shear and block shear $\phi R_n = 76.4\text{ kips} > 60.0\text{ kips}$ **o.k.**	*Check bolts and angle for bearing, shear and block shear* $\phi R_n = 50.9\text{ kips} > 40.0\text{ kips}$ **o.k.**	Manual Table 10-1

Note: The middle portion of Manual Table 10-1 includes checks of the limit-state of bolt bearing on the beam web and the limit-state of block shear rupture on coped beams. Manual Tables 9-3a, 9-3b and 9-3c may be used to determine the available block shear strength for values of L_{ev} and L_{eh} beyond the limits of Table 10-1. For coped members, the limit states of flexural yielding and local buckling must be checked independently per Part 9. — Manual Table 10-1

Check flexural rupture on the coped section

S_{net} = 23.4 in^3 — Manual Table 9-2

$$R_n = \frac{F_u S_{net}}{e} = \frac{(65\text{ ksi})(23.4\text{ in.}^3)}{4.50\text{ in.}} = 338\text{ kips}$$

Manual Part 9

LRFD	ASD
$\phi = 0.75$ ϕR_n = 0.75(338 kips) = 254 kips > 60.0 kips **o.k.**	$\Omega = 2.00$ $R_n / \Omega = 338\text{ kips}/2.00$ $= 169\text{ kips} > 40.0\text{ kips}$ **o.k.**

Check yielding and local web buckling at the coped section

Verify $c \le 2d$ and $d_c \le \dfrac{d}{2}$ — Manual Part 9 Page 9-7

$$c = 4.00 \text{ in.} \le 2(18.0 \text{ in.}) = 36.0 \text{ in.} \qquad \textbf{o.k.}$$

$$d_c = 2.00 \text{ in.} \le \frac{18.0 \text{ in.}}{2} = 9.00 \text{ in.} \qquad \textbf{o.k.}$$

$$\frac{c}{d} = \frac{4.00 \text{ in.}}{18.0 \text{ in.}} = 0.222; \; \frac{c}{h_0} = \frac{4.00 \text{ in.}}{16.0 \text{ in.}} = 0.250$$

Since $\dfrac{c}{d} \le 1.0$,

$$f = 2\left(\frac{c}{d}\right) = 2(0.222) = 0.444$$

Since $\dfrac{c}{h_0} \le 1.0$,

$$k = 2.2\left(\frac{h_0}{c}\right)^{1.65} = 2.2\left(\frac{16.0 \text{ in.}}{4.00 \text{ in.}}\right)^{1.65} = 21.7$$

$$F_{cr} = 26{,}210\left(\frac{t_w}{h_0}\right)^2 fk = 26{,}210\left(\frac{0.355 \text{ in.}}{16.0 \text{ in.}}\right)^2 (0.444)(21.7) = 124 \text{ ksi} \le F_y$$

Use $F_{cr} = F_y = 50$ ksi

$$R_n = \frac{F_{cr}S_{net}}{e} = \frac{(50 \text{ ksi})(23.4 \text{ in.}^3)}{4.50 \text{ in.}} = 260 \text{ kips}$$ — Manual Part 9

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(260 \text{ kips})$ $= 234 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $R_n / \Omega = 260 \text{ kips}/1.67$ $= 156 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Check shear yielding on beam web

$$R_n = 0.6F_yA_g = 0.6(50.0 \text{ ksi})(0.355 \text{ in.})(16.0 \text{ in.}) = 170 \text{ kips}$$ — Eqn J4-3

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(170 \text{ kips})$ $= 170 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $R_n / \Omega = 170 \text{ kips}/1.50$ $= 113 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Section J4.2

Check shear rupture on beam web

$$R_n = 0.6\, F_uA_{nv} = 0.6(65.0 \text{ ksi})(0.355 \text{ in.})(16.0 \text{ in.}) = 222 \text{ kips}$$ — Eqn J4-4

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(222 \text{ kips})$ $= 167 \text{ kips} > 60.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $R_n / \Omega = 222 \text{ kips}/2.00$ $= 111 \text{ kips} > 40.0 \text{ kips}$ **o.k.**

Section J4.2

Note: see **Example II.A-4** for an all-bolted double-angle connection (beam-to-girder web).

Example II.A-6 Beam End Coped at the Top Flange Only

Given:

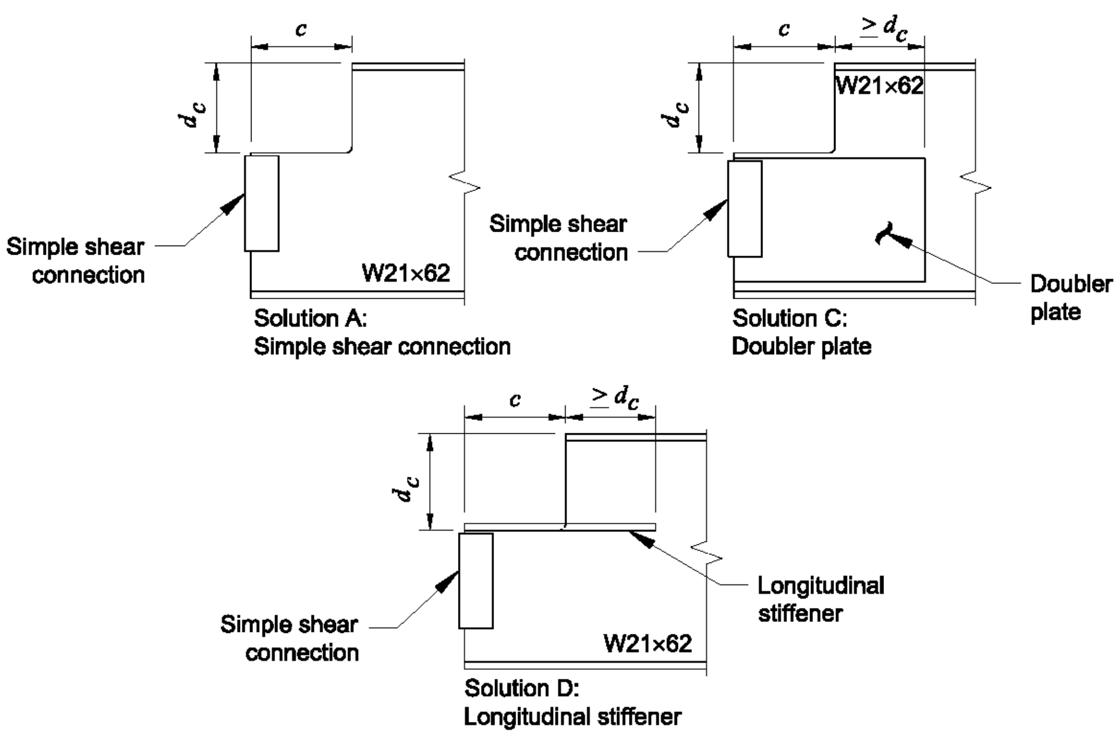
For a W21×62 coped 8-in. deep by 9-in. long at the top flange only:

A. Calculate the available strength of the beam end, considering the limit states of flexural yielding and local buckling. Assume e = 9½ in.

B. Choose an alternate W21 shape to eliminate the need for stiffening for an end reaction of R_D = 16.5 kips and R_L = 47 kips.

C. Determine the size of doubler plate needed to stiffen the W21×62 for the given end reaction.

D. Determine the size of longitudinal stiffeners needed to stiffen the W21×62 for the given end reaction.



Material Properties:

Beam W21×62	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	Tables 2-3 and 2-4

Geometric Properties:

Beam W21×62	d = 21.0 in.	t_w = 0.400 in.	S_{net} = 17.8 in.^{3}	Manual Tables 1-1
Cope	c = 9.00 in. h_0 = 13.0 in.	d_c = 8.00 in.	e = 9.00 in. + 0.500 in. = 9.50 in.	and 9-2 Manual

Figure 9-2

Solution A:

Check yielding and local buckling

Manual Part 9

Verify parameters

$c \leq 2d$

$c = 9.00 \text{ in.} \leq 2(21.0 \text{ in.}) = 42.0 \text{ in.}$ **o.k.**

$d_c \leq d/2$

$d_c = 8.00 \text{ in.} \leq \dfrac{21.0 \text{ in.}}{2} = 10.5 \text{ in.}$ **o.k.**

$\dfrac{c}{d} = \dfrac{9.00 \text{ in.}}{21.0 \text{ in.}} = 0.429; \quad \dfrac{c}{h_0} = \dfrac{9.00 \text{ in.}}{13.0 \text{ in.}} = 0.692$

Since $\dfrac{c}{d} \leq 1.0$,

$$f = 2\left(\frac{c}{d}\right) = 2(0.429) = 0.858$$

Since $\dfrac{c}{h_0} \leq 1.0$,

$$k = 2.2\left(\frac{h_0}{c}\right)^{1.65} = 2.2\left(\frac{13.0 \text{ in.}}{9.00 \text{ in.}}\right)^{1.65} = 4.04$$

For a top cope only, the critical buckling stress is

$$F_{cr} = 26{,}210\left(\frac{t_w}{h_0}\right)^2 fk \leq F_y$$

$$= 26{,}210\left(\frac{0.400 \text{ in.}}{13.0 \text{ in.}}\right)^2 (0.858)(4.04) \leq F_y$$

$$= 86.0 \text{ ksi} \leq F_y$$

Use $F_{cr} = F_y = 50$ ksi

$$R_n = \frac{F_{cr} S_{net}}{e} = \frac{(50 \text{ ksi})(17.8 \text{ in.}^3)}{9.50 \text{ in.}} = 93.7 \text{ kips}$$

Manual Part 9

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(93.7 \text{ kips}) = 84.3 \text{ kips}$	$\Omega = 1.67$ $R_n / \Omega = 93.7 \text{ kips}/1.67 = 56.1 \text{ kips}$

Check shear yielding on beam web

$R_n = 0.6 F_y A_g = 0.6(50 \text{ ksi})(0.400 \text{ in.})(13.0 \text{ in.}) = 156 \text{ kips}$ Eqn J4-3

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(156 \text{ kips}) = 156 \text{ kips}$	$\Omega = 1.50$ $R_n / \Omega = 156 \text{ kips}/1.50 = 104 \text{ kips}$

Sect J4.2

Check shear rupture on beam web

$R_n = 0.6\, F_u\, A_{nv} = 0.6(65 \text{ ksi})(0.400 \text{ in.})(13.0 \text{ in.}) = 203 \text{ kips}$ Eqn J4-4

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(203 \text{ kips}) = 152 \text{ kips}$	$\Omega = 2.00$ $R_n / \Omega = 203 \text{ kips}/2.00 = 102 \text{ kips}$

Section J4.2

Thus, the available strength is controlled by local bucking, with

LRFD	ASD
$\phi R_n = 84.3 \text{ kips}$	$R_n/\Omega = 56.1 \text{ kips}$

Solution B:

Calculate required strength

LRFD	ASD
$R_u = 1.2(16.5 \text{ kips}) + 1.6(47 \text{ kips}) = 95.0 \text{ kips}$	$R_a = 16.5 \text{ kips} + 47 \text{ kips} = 63.5 \text{ kips}$

Calculate required section modulus based on flexural yielding

LRFD	ASD
$S_{req} = \dfrac{R_u e}{\phi F_y} = \dfrac{(95.0 \text{ kips})(9.50 \text{ in.})}{0.90(50.0 \text{ ksi})}$ $= 20.1 \text{ in.}^3$	$S_{req} = \dfrac{R_a e\, \Omega}{F_y} = \dfrac{63.5 \text{ kips}(9.50 \text{ in.})(1.67)}{50.0 \text{ ksi}}$ $= 20.1 \text{ in.}^3$

Try W21×73 with an 8-in. deep cope

$S_{net} = 21.0 \text{ in.}^3 > 20.1 \text{ in.}^3$ **o.k.**

Manual Table 9-2

Check local buckling

As determined in Solution A for the W21×62, the available critical stress due to local buckling for a W21×73 with an 8-in. deep cope is limited to the yield stress.

Manual Part 9

Therefore,

$$R_n = \frac{F_{cr} S_{net}}{e} = \frac{(50 \text{ ksi})(21.0 \text{ in.}^3)}{9.50 \text{ in.}} = 111 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(111 \text{ kips})$ $= 99.9 \text{ kips} > 95.0 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $R_n / \Omega = 111 \text{ kips}/1.67$ $= 66.5 \text{ kips} > 63.5 \text{ kips}$ **o.k.**

Note: By inspection the W21x73 has sufficient shear strength.

Solution C:

Design doubler plate

LRFD	ASD
Doubler plate must provide a required strength of 95.0 kips – 84.3 kips = 10.7 kips. $S_{req} = \dfrac{\left(R_u - \phi R_{n\ beam}\right)e}{\phi F_y}$ $= \dfrac{\left(95.0\text{ kips - }84.3\text{ kips}\right)\left(9.50\text{ in.}\right)}{0.90\left(50\text{ ksi}\right)}$ $= 2.26\text{ in.}^3$ For an 8-in. deep plate, $t_{req} = \dfrac{6S_{req}}{d^2} = \dfrac{6\left(2.26\text{ in.}^3\right)}{\left(8.00\text{ in.}\right)^2} = 0.212\text{ in.}$	Doubler plate must provide a required strength of 63.5 kips – 56.1 kips = 7.40 kips. $S_{req} = \dfrac{\left(R_a - R_{n\ beam}/\Omega\right)e\ \Omega}{F_y}$ $= \dfrac{\left(63.5\text{ kips-}56.1\text{ kips}\right)\left(9.50\text{ in.}\right)\left(1.67\right)}{50\text{ ksi}}$ $= 2.35\text{ in.}^3$ For an 8-in. deep plate, $t_{req} = \dfrac{6S_{req}}{d^2} = \dfrac{6\left(2.35\text{ in.}^3\right)}{\left(8.00\text{ in.}\right)^2} = 0.220\text{ in.}$

Manual Part 9

Note: ASTM A572 grade 50 plate is recommended in order to match the beam yield strength.

Thus, since the doubler plate must extend at least d_c beyond the cope, use a PL ¼ in.×8 in. ×1’-5” with 3⁄16 in. welds top and bottom.

Solution D:

Design longitudinal stiffeners

Try PL ¼ in.×4 in. slotted to fit over the beam web with F_y = 50 ksi.

From section property calculations for the neutral axis and moment of inertia, conservatively ignoring the beam fillets, the neutral axis is located 4.39 in. from the bottom flange (8.86 in. from the top of the stiffener).

	I_0 **(in.⁴)**	Ad^2 **(in.⁴)**	$I_0 + Ad^2$ **(in.⁴)**
Stiffener	0.00521	76.3	76.3
W21×62 web	63.3	28.9	92.2
W21×62 bottom flange	0.160	84.5	84.7
			$\Sigma = I_x = 253\text{ in.}^4$

Check the slenderness of the longitudinal stiffener.

Table B4.1 Case 2

$$\lambda_r = 0.95\sqrt{k_c E/F_L}$$

$$k_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{11.9\text{ in.}/0.400\text{ in.}}} = 0.733 \quad \text{where } 0.35 \le k_c \le 0.76$$

use $k_c = 0.733$

$$S_c = \frac{I_x}{c} = \frac{253 \text{ in.}^4}{8.86 \text{ in.}} = 28.6 \text{ in.}^3$$

$$S_{xt} = \frac{253 \text{ in.}^4}{4.39 \text{ in.}} = 57.6 \text{ in.}^3$$

$$\frac{S_{xt}}{S_{xc}} = \frac{57.6 \text{ in.}^3}{28.6 \text{ in.}^3} = 2.01 \geq 0.7 \text{; therefore, } F_L = 0.7F_y = 0.7(50 \text{ ksi}) = 35.0 \text{ ksi}$$

$$\lambda_r = 0.95\sqrt{0.733(29{,}000 \text{ ksi})/35.0 \text{ ksi}} = 23.4$$

$$\frac{b}{t} = \frac{4.00 \text{ in.}}{2(0.250 \text{ in.})} = 8.00 < 23.4 \text{ ; therefore, the stiffener is not slender}$$

$$S_{net} = S_c$$

and the nominal strength of the reinforced section is

$$R_n = \frac{F_y S_{net}}{e} = \frac{(50 \text{ ksi})(28.6 \text{ in.}^3)}{9.50 \text{ in.}} = 151 \text{ kips}$$

Manual Part 9

LRFD	ASD
Calculate design strength	*Calculate allowable strength*
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = 0.90(151 \text{ kips})$	$R_n / \Omega = 151 \text{ kips}/1.67$
$= 136 \text{ kips} > 95.0 \text{ kips}$ **o.k.**	$= 90.4 \text{ kips} > 63.5 \text{ kips}$ **o.k.**

Note: ASTM A572 Grade 50 plate is recommended in order to match the beam yield strength.

Plate dimensions

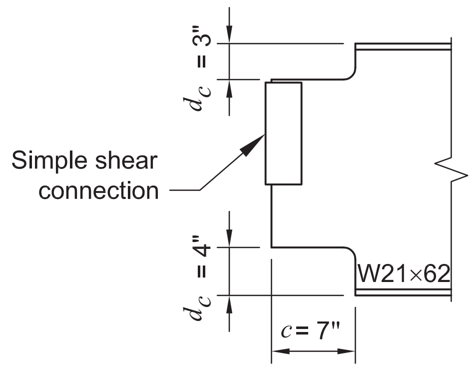
Since the longitudinal stiffening must extend at least d_c beyond cope,

Use PL ¼ in.×4 in.×1’-5” with ¼ in. welds.

Example II.A-7 Beam End Coped at the Top and Bottom Flanges

Given:

A W21×62 is coped 3-in. deep by 7-in. long at the top flange and 4-in. deep by 7-in. long at the bottom flange. Calculate the available strength of the beam end considering the limit states of flexural yielding, local buckling and shear yielding. Assume e = 7½ in.



Material Properties:

Beam W21×62 ASTM A992 F_y = 50 ksi F_u = 65 ksi — Manual Table 2-3

Geometric Properties:

Beam W21×62 d = 21.0 in. t_w = 0.400 in.
Cope c = 7.00 in. d_c = 4.00 in. e = 7.50 in. h_0 = 14.0 in.

Manual Tables 1-1 & Figure 9-3

Solution:

Check yielding and local buckling

Manual Part 9

$$\lambda = \frac{h_o\sqrt{F_y}}{10t_w\sqrt{475+280\left(\frac{h_o}{c}\right)^2}}$$

$$= \frac{(14.0\text{ in.})\sqrt{(50\text{ ksi})}}{10(0.400\text{ in.})\sqrt{475+280\left(\frac{14.0\text{ in.}}{7.00\text{ in.}}\right)^2}}$$

$$= 0.620 \le 0.7\text{ ; therefore, Q} = 1.0$$

$$F_{cr} = F_yQ = (50\text{ ksi})(1.0) = 50\text{ ksi}$$

Therefore, yielding governs.

Use F_{cr} = 50 ksi

Manual Part 9

$$S_{net} = \frac{t_w h_o^2}{6} = \frac{(0.400\text{ in.})(14.0\text{ in.})^2}{6} = 13.1\text{ in.}^3$$

$$R_n = \frac{F_{cr}S_{net}}{e} = \frac{(50\text{ ksi})(13.1\text{ in.}^3)}{7.50\text{ in.}} = 87.3\text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(87.3\text{ kips}) = 78.6\text{ kips}$	$\Omega = 1.67$ $R_n / \Omega = \dfrac{(87.3\text{ kips})}{1.67} = 52.3\text{ kips}$

Check shear yielding on beam web

$R_n = 0.6\, F_y\, A_g = 0.6(50\text{ ksi})(0.400\text{ in.})(14.0\text{ in.}) = 168\text{ kips}$ Eqn J4-3

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(168\text{ kips}) = 168\text{ kips}$	$\Omega = 1.50$ $R_n / \Omega = \dfrac{168\text{ kips}}{1.50} = 112\text{ kips}$

Section J4.2

Check shear rupture on beam web

$R_n = 0.6\, F_u\, A_{nv} = 0.6(65\text{ ksi})(0.400\text{ in.})(14.0\text{ in.}) = 218\text{ kips}$ Eqn J4-4

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(218\text{ kips}) = 164\text{ kips}$	$\Omega = 2.00$ $R_n / \Omega = \dfrac{218\text{ kips}}{2.00} = 109\text{ kips}$

Specification Sect J4.2

Thus, the available strength is controlled by yielding, with

LRFD	ASD
ϕR_n = 78.6 kips	R_n/Ω = 52.3 kips

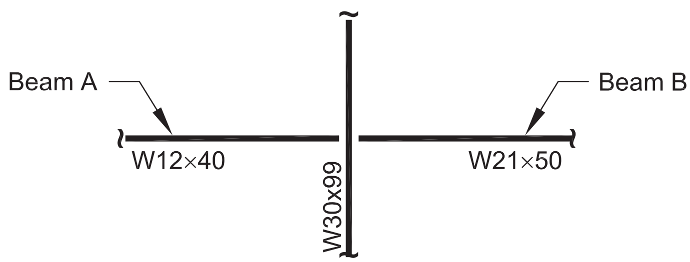
Example II.A-8 All-Bolted Double-Angle Connections (beams-to-girder web)

Given:

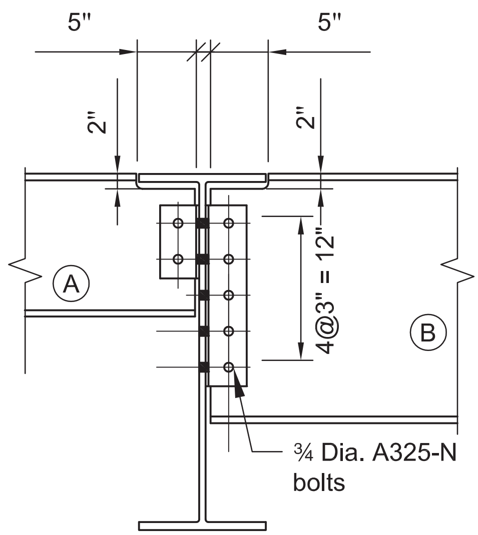
Design the all-bolted double-angle connections between the W12×40 beam (A) and W21×50 beam (B) and the W30×99 girder-web to support the following beam end reactions:

Beam A	Beam B
R_{DA} = 4.17 kips	R_{DB} = 18.3 kips
R_{LA} = 12.5 kips	R_{LB} = 55.0 kips

Use ¾-in. diameter ASTM A325-N or F1852-N bolts in standard holes.



Part Plan



Section

Material Properties:

W12×40	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
W21×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
W30×99	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angle	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Manual Table 1-1 and Manual Table 9-2

Beam W12×40 t_w = 0.295 in. d = 11.9 in. h_0 = 9.9 in. S_{net} = 8.03 in.^{3}
top flange cope d_c = 2.00 in. c = 5.00 in. e = 5.50 in.

Beam W21×50 t_w = 0.380 in. d = 20.8 in. h_0 = 18.8 in. S_{net} = 32.5 in.^{3}
top flange cope d_c = 2.00 in. c = 5.00 in. e = 5.50 in.

Girder W30×99 t_w = 0.520 in. d = 29.7 in.

Solution:

Beam A:

LRFD	ASD
Calculate required strength R_{Au} = (1.2)(4.17 kips) + (1.6)(12.5 kips) = 25.0 kips *Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture (beam A)* Try two rows of bolts and ¼-in. angle thickness ϕR_n = 48.9 kips > 25.0 kips **o.k.** *Check beam web for bolt bearing and block shear rupture (beam A)* From Table 10-1, for two rows of bolts and L_{ev} = 1¼ in. and L_{eh} = 1½ in. ϕR_n = (126 kips/in.)(0.295 in.) = 37.2 kips > 25.0 kips **o.k.**	*Calculate required strength* R_{Aa} = 4.17 kips + 12.5 kips = 16.7 kips *Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture (beam A)* Try two rows of bolts and ¼-in. angle thickness R_n / Ω = 32.6 kips > 16.7 kips **o.k.** *Check beam web for bolt bearing and block shear rupture (beam A)* From Table 10-1, for two rows of bolts and L_{ev} = 1¼ in. and L_{eh} = 1½ in. R_n / Ω = (83.7 kips/in.)(0.295 in.) = 24.7 kips > 16.7 kips **o.k.**
Check flexural rupture of the coped section (beam A) S_{net} = 8.03 in.^{3} $\phi = 0.75$ $\phi R_n = \dfrac{\phi F_u S_{net}}{e} = \dfrac{0.75(65\text{ ksi})(8.03\text{ in.}^3)}{(5.50\text{ in.})}$ = 71.2 kips > 25.0 kips **o.k.**	*Check flexural rupture of the coped section (beam A)* S_{net} = 8.03 in.^{3} $\Omega = 2.00$ $R_n / \Omega = \dfrac{F_u S_{net}}{e\Omega} = \dfrac{(65\text{ ksi})(8.03\text{ in.}^3)}{(5.50\text{ in.})(2.00)}$ = 47.5 kips > 16.7 kips **o.k.**

Manual Table 10-1

Manual Table 10-1

Manual Table 9-2

Manual Part 9

Check yielding and local buckling of coped section

S_{net} = 8.03 in.^{3}

Manual Table 9-2

Verify parameters

$c \le 2d$

Manual

5.00 in. ≤ 2(11.9 in.) = 23.8 in. **o.k.** — Part 9

$d_c \le d/2$
2.00 in. ≤ 11.9 in. / 2 = 5.95 in. **o.k.**

$$\frac{c}{d}=\frac{5.00\text{ in.}}{11.9\text{ in.}}=0.420\le 1.0\ ;\ \frac{c}{h_0}=\frac{5.00\text{ in.}}{9.90\text{ in.}}=0.505\le 1.0$$

Calculate plate buckling model adjustment factor

$$f=2\left(\frac{c}{d}\right)=2(0.420)=0.840$$

Calculate plate buckling coefficient

$$k=2.2\left(\frac{h_0}{c}\right)^{1.65}=2.2\left(\frac{9.90\text{ in.}}{5.00\text{ in.}}\right)^{1.65}=6.79$$

$$F_{cr}=26{,}210\left(\frac{t_w}{h_0}\right)^2 fk\le F_y$$

$$=26{,}210\left(\frac{0.295\text{ in.}}{9.90\text{ in.}}\right)^2(0.840)(6.79)=133\text{ ksi}\le F_y$$

Use $F_{cr} = F_y$ = 50 ksi

LRFD	ASD	
$\phi = 0.90$ $\phi R_n=\dfrac{\phi F_{cr}S_{net}}{e}=\dfrac{0.90(50\text{ ksi})(8.03\text{ in.}^3)}{(5.50\text{ in.})}$ = 65.7 kips > 25.0 kips **o.k.**	$\Omega = 1.67$ $R_n/\Omega=\dfrac{(F_{cr}/\Omega)S_{net}}{e}=\dfrac{(50\text{ ksi})(8.03\text{ in.}^3)}{1.67(5.50\text{ in.})}$ = 43.7 kips > 16.7 kips **o.k.**	Manual Part 9

Check shear yielding on beam web

$R_n = 0.6\, F_y\, A_g$ = 0.6(50 ksi)(0.295 in.)(9.90 in.) = 87.6 kips — Eqn J4-3

LRFD	ASD	
$\phi = 1.00$ $\phi R_n = 1.00(87.6\text{ kips})$ = 87.6 kips > 25.0 kips **o.k.**	$\Omega = 1.50$ $R_n/\Omega=\dfrac{87.6\text{ kips}}{1.50}$ = 58.4 kips > 16.7 kips **o.k.**	Section J4.2

Check shear rupture on beam web

$A_{nv} = t_w\,[(h_o - 2(0.875\text{ in.})]$ = 0.295 in. (9.90 in. – 1.75 in.) = 2.40 in^2

$R_n = 0.6\, F_u\, A_{nv}$ = 0.6(65 ksi)(2.40 in.^{2}) = 93.6 kips — Eqn J4-4

LRFD	ASD	
$\phi = 0.75$	$\Omega = 2.00$	Section J4.2

$\phi R_n = 0.75(93.6\text{ kips}) = 70.2\text{ kips}$ 70.2 kips > 25.0 kips **o.k.**	$R_n / \Omega = \dfrac{93.6\text{ kips}}{2.00} = 46.8\text{ kips}$ 46.8 kips > 16.7 kips **o.k.**

Beam B:

LRFD	ASD	
Calculate required strength R_{Bu} = (1.2)(18.3kips) + (1.6)(55.0 kips) = 110 kips *Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture (beam B)* Try five rows of bolts and ¼ in. angle thickness. ϕR_n = 125 kips > 110 kips **o.k.** *Check beam web for bolt bearing and block shear rupture (beam B)* From Table 10-1, for five rows of bolts and L_{ev} = 1¼ in. and L_{eh} = 1½ in. ϕR_n = (312 kips/in.)(0.380 in.) = 119 kips > 110 kips **o.k.**	*Calculate required strength* R_{Ba} = 18.3 kips + 55.0 kips = 73.3 kips *Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture (beam B)* Try five rows of bolts and ¼ in. angle thickness. R_n / Ω = 83.3 kips > 73.3 kips **o.k.** *Check beam web for bolt bearing and block shear rupture (beam B)* From Table 10-1, for five rows of bolts and L_{ev} = 1¼ in. and L_{eh} = 1½ in. R_n / Ω = (208 kips/in.)(0.380 in.) = 79.0 kips > 73.3 kips **o.k.**	Manual Table 10-1 Manual Table 10-1
Check flexural rupture of the coped section (beam B) S_{net} = 32.5 in^3 $\phi = 0.75$ $\phi R_n = \dfrac{\phi F_u S_{net}}{e} = \dfrac{0.75(65\text{ ksi})(32.5\text{ in.}^3)}{(5.50\text{ in.})}$ = 288 kips > 110 kips **o.k.**	*Check flexural rupture of the coped section (beam B)* S_{net} = 32.5 in^3 $\Omega = 2.00$ $R_n / \Omega = \dfrac{F_u S_{net}}{e\,\Omega} = \dfrac{(65\text{ ksi})(32.5\text{ in}^3)}{(5.50\text{ in.})(2.00)}$ = 192 kips > 73.3 kips **o.k.**	Manual Table 9-2 Manual Part 9

Check yielding and local buckling of coped section — Manual Table 9-2

Verify parameters — Manual Part 9

$c \le 2d$
5.00 in. ≤ 2(20.8 in.) = 41.6 in. **o.k.**

$d_c \le d/2$
2.00 in. ≤ 20.8 in. / 2 = 10.4 in. **o.k.**

$$\frac{c}{d} = \frac{5.00\text{ in.}}{20.8\text{ in.}} = 0.240 \le 1.0\ ;\ \frac{c}{h_0} = \frac{5.00\text{ in.}}{18.8\text{ in.}} = 0.266 \le 1.0$$

Calculate plate buckling model adjustment factor

$$f = 2\left(\frac{c}{d}\right) = 2(0.240) = 0.480$$

Calculate plate buckling coefficient

$$k = 2.2\left(\frac{h_0}{c}\right)^{1.65} = 2.2\left(\frac{18.8\text{ in.}}{5.00\text{ in.}}\right)^{1.65} = 19.6$$

$$F_{cr} = 26{,}210\left(\frac{t_w}{h_0}\right)^2 fk \le F_y$$

$$= 26{,}210\left(\frac{0.380\text{ in.}}{18.8\text{ in.}}\right)^2 (0.480)(19.6) = 101\text{ ksi} \le F_y$$

Use $F_{cr} = F_y = 50$ ksi

$S_{net} = 32.5$ in.^{3}

LRFD	ASD
$\phi = 0.90$ $\phi R_n = \dfrac{\phi F_{cr} S_{net}}{e} = \dfrac{0.90(50\text{ ksi})(32.5\text{ in.}^3)}{(5.50\text{ in.})}$ $= 266$ kips > 110 kips **o.k.**	$\Omega = 1.67$ $R_n / \Omega = \dfrac{(F_{cr}/\Omega) S_{net}}{e} = \dfrac{(50\text{ ksi})(32.5\text{ in.}^3)}{1.67(5.50\text{ in.})}$ $= 177$ kips > 73.3 kips **o.k.**

Check shear yielding on beam web

$R_n = 0.6 F_y A_g = 0.6(50\text{ ksi})(0.380\text{ in.})(18.8\text{ in.}) = 214$ kips — Eqn J4-3

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(214\text{ kips})$ $= 214$ kips > 110 kips **o.k.**	$\Omega = 1.50$ $R_n / \Omega = \dfrac{214\text{ kips}}{1.50}$ $= 143$kips > 73.3 kips **o.k.**

Section J4.2

Check shear rupture on beam web

$A_{nv} = t_w [h_o - (5)(0.875\text{ in.})] = 0.380\text{ in.}(18.8\text{ in.} - 4.38\text{ in.}) = 5.48\text{ in.}^2$

$R_n = 0.6 F_u A_{nv} = 0.6(65\text{ ksi})(5.48\text{ in.}^2) = 214$ kips — Eqn J4-4

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(214\text{ kips}) = 161$ kips 161 kips > 110 kips **o.k.**	$\Omega = 2.00$ $R_n / \Omega = \dfrac{214\text{ kips}}{2.00} = 107$ kips 107 kips > 73.3 kips **o.k.**

Section J4.2

Supporting Girder

Check the supporting girder web

The required bearing strength per bolt is greatest for the bolts that are loaded by both connections. Thus, for the design of these 4 critical bolts, the required strength is determined as

LRFD	ASD
From Beam A, each bolt must support one-fourth of 25.0 kips or 6.25 kips/bolt. From Beam B, each bolt must support one-tenth of 110 kips or 11.0 kips/bolt. Thus, R_u = 6.25 kips/bolt + 11.0 kips/bolt = 17.3 kips/bolt The design bearing strength per bolt is ϕr_n = (87.8 kips/in.)(0.520 in.) = 45.7 kips/bolt > 17.3 kips/bolt **o.k.**	From Beam A, each bolt must support one-fourth of 16.7 kips or 4.18 kips/bolt. From Beam B, each bolt must support one-tenth of 73.3 kips or 7.33 kips/bolt. Thus, R_a = 4.18 kips/bolt + 7.33 kips/bolt = 11.5 kips/bolt The allowable bearing strength per bolt is r_n / Ω = (58.5 kips/in.)(0.520 in.) = 30.4 kips/bolt > 11.5 kips/bolt **o.k.**

Manual Table 7-5

Although not required for design, the tabulated values may be verified by hand calculations, as follows:

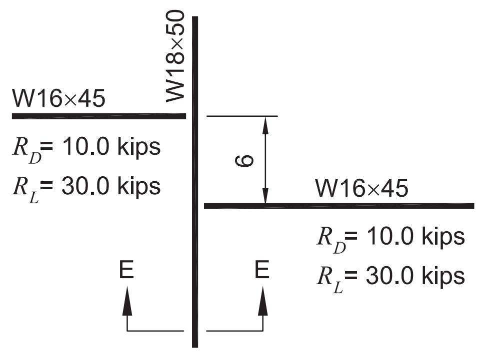
LRFD	ASD
$\phi r_n = \phi(1.2L_c tF_u) \le \phi(2.4dtF_u)$ $L_c = 3.00\text{ in.} - 0.895\text{ in.} = 2.11\text{ in.}$ $\phi 1.2L_c tF_u = (0.75)(1.2)(2.11\text{ in.})(0.520\text{ in.})(65\text{ksi})$ = 64.2 kips $\phi(2.4dtF_u) = (0.75)(2.4)(0.750\text{ in.})(0.520\text{ in.})(65\text{ ksi})$ = 45.6 kips < 64.2 kips ϕr_n = 45.6 kips/bolt > 17.3 kips/bolt **o.k.**	$r_n/\Omega = (1.2L_c tF_u)/\Omega \le (2.4dtF_u)/\Omega$ $L_c = 3.00\text{ in.} - 0.895\text{ in.} = 2.11\text{ in.}$ $1.2L_c tF_u / \Omega = \dfrac{(1.2)(2.11\text{ in.})(0.520\text{ in.})(65\text{ ksi})}{2.00}$ = 43.1 kips $(2.4dtF_u)/\Omega = \dfrac{(2.4)(0.750\text{ in.})(0.520\text{ in.})(65.0\text{ ksi})}{2.00}$ = 30.4 kips < 43.1 kips r_n / Ω = 30.4 kips/bolt > 11.5 kips/bolt **o.k.**

Eqn. J3-6a

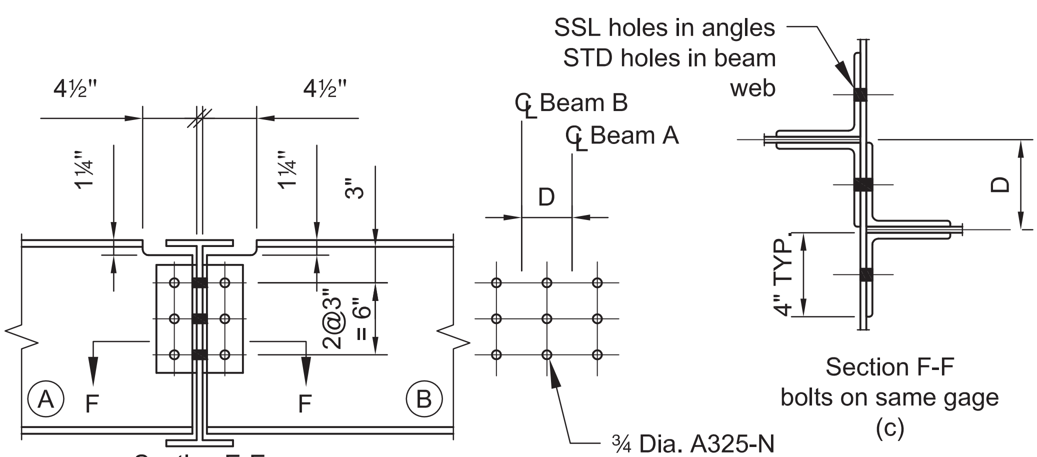
Example II.A-9 Offset All-Bolted Double-Angle Connections (beams-to-girder web)

Given:

Two all-bolted double-angle connections are made back-to-back with offset beams. Design the connections to accommodate an offset of 6 in.



Part Plan
(Beam flush top)
(a)



Section E-E
bolts on same gage
(b)

Section F-F
bolts on same gage
(c)

Material Properties:

Girder	W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Beam	W16×45	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angle		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Girder	W18×50	t_w = 0.355 in.	d = 18.0 in.	Manual Table 1-1
Beam	W16×45	t_w = 0.345 in.	d = 16.1 in.	

Solution:

Modify the 2L4×3½×¼ SLBB connection designed in Example II.A-4 to work in the configuration shown above. The offset dimension (6 in.) is approximately equal to the gage on the support from the previous example (6 ¼ in.) and, therefore, is not recalculated below.

Thus, the bearing strength of the middle vertical row of bolts (through both connections), which now carry a portion of the reaction for both connections, must be verified for this new configuration.

For each beam,

R_D = 10 kips
R_L = 30 kips

LRFD	ASD
Calculate the required strength	*Calculate the required strength*
R_u = (1.2)(10 kips) + (1.6)(30 kips) = 60.0 kips	R_a = 10 kips + 30 kips = 40.0 kips
The required bearing strength per bolt is	The required bearing strength per bolt is
$r_u = \dfrac{(2\text{ connections})(60.0\text{ kips/connection})}{6\text{ bolts}}$	$r_a = \dfrac{(2\text{ connections})(40.0\text{ kips/connection})}{6\text{ bolts}}$
= 20.0 kips/bolt < 31.8 kips/bolt	= 13.3 kips/bolt < 21.2 kips/bolt
Check supporting girder web	*Check supporting girder web*
The design bearing strength per bolt is	The allowable bearing strength per bolt is
ϕr_u = (87.8 kips/in)(0.355 in.)	r_a / Ω = (58.5 kips/in.)(0.355 in.)
= 31.2 kips/bolt > 20.0 kips/bolt **o.k.**	= 20.8 kips/bolt > 13.3 kips/bolt **o.k.**

Manual Table 7-1

Manual Table 7-5

Note: If the bolts are not spaced equally from the supported beam web, the force in each column of bolts should be determined by using a simple beam analogy between the bolts, and applying the laws of statics.

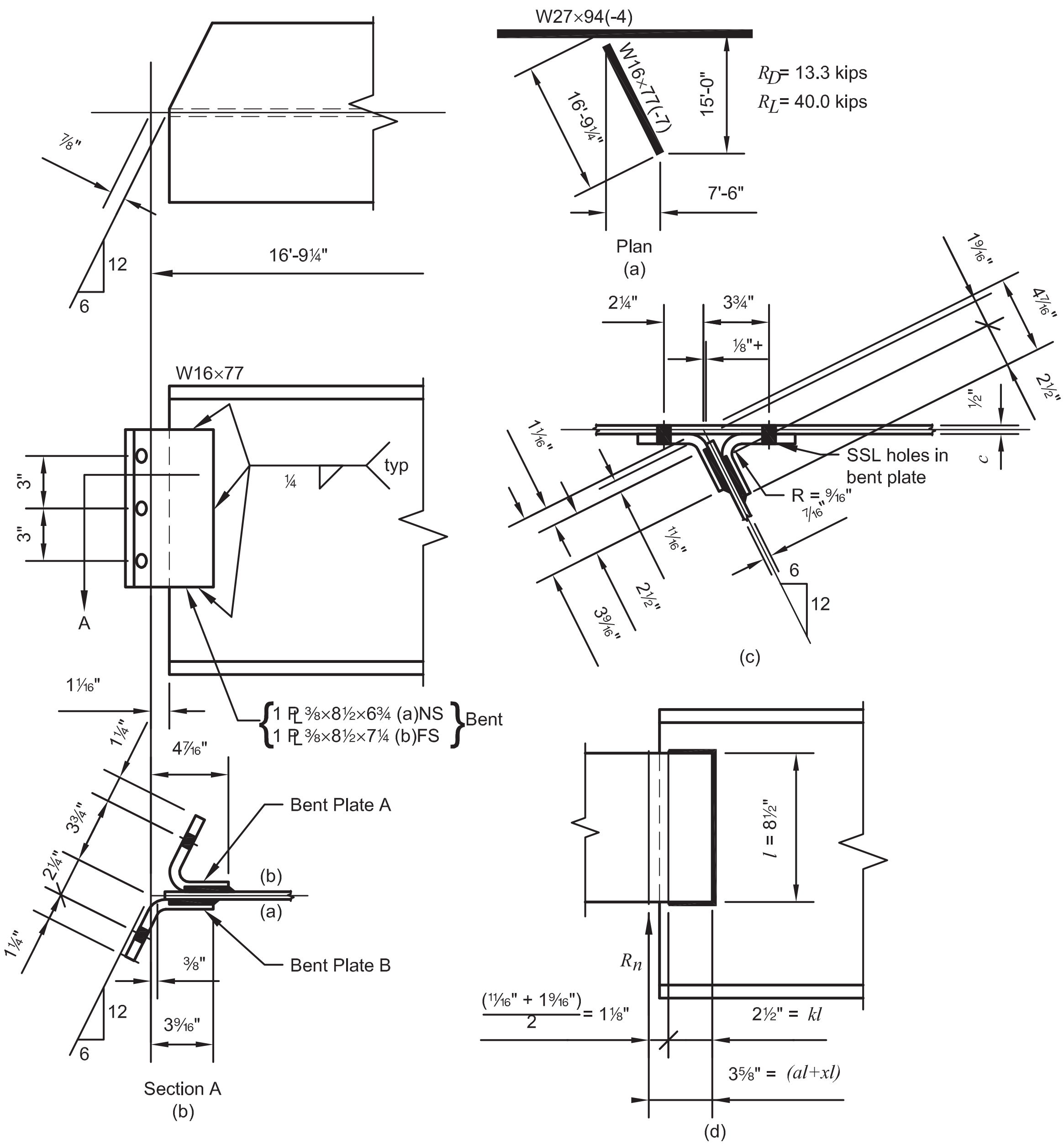
Example II.A-10 Skewed Double Bent-Plate Connection (beam-to-girder web)

Given:

Design the skewed double bent-plate connection between the W16×77 beam to W27×94 girder-web to support the following beam end reactions:

R_D = 13.3 kips
R_L = 40.0 kips

Use ⅞–in. diameter ASTM A325-N or F1852-N bolts in standard holes through the support. Use 70 ksi electrode welds to the supported beam.



Material Properties:

W16×77	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
W27×94	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Plate Material	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

W16×77	t_w = 0.455 in.	d = 16.5 in.	Manual Table 1-1
W27×94	t_w = 0.490 in.		

Solution:

Calculate the required strength

LRFD	ASD
R_u = 1.2 (13.3 kips) + 1.6 (40.0 kips) = 80.0 kips	R_a = 13.3 kips + 40.0 kips = 53.3 kips

See the scaled layout (c) of the connection. Assign load to each vertical row of bolts by assuming a simple beam analogy between bolts and applying the laws of statics.

LRFD	ASD
Required strength of bent plate A $R_u = \dfrac{80.0 \text{ kips (2.25 in.)}}{6.00 \text{ in.}} = 30.0 \text{ kips}$ *Required strength for bent plate B* $R_u = 80.0 \text{ kips} - 30.0 \text{ kips} = 50.0 \text{ kips}$	*Required strength for bent plate A* $R_a = \dfrac{53.3 \text{ kips (2.25 in.)}}{6.00 \text{ in.}} = 20.0 \text{ kips}$ *Required strength for bent plate B* $R_a = 53.3 \text{ kips} - 20.0 \text{ kips} = 33.3 \text{ kips}$

Assume that the welds across the top and bottom of the plates will be 2½ in. long, and that the load acts at the intersection of the beam centerline and the support face.

While the welds do not coincide on opposite faces of the beam web and the weld groups are offset, the locations of the weld groups will be averaged and considered identical see Figure (d).

Design welds

Assume plate length of 8½ in.

$$k = \frac{kl}{l} = \frac{2.50 \text{ in.}}{8.50 \text{ in.}} = 0.294 \text{ in.}$$

From tables, with θ = 0° and k = 0.294 — Manual Table 8-8

$$xl = \frac{2.50 \text{ in.}(1.25 \text{ in.})(2)}{2.50 \text{ in.}(2) + 8.50 \text{ in.}} = 0.463 \text{ in.}$$

Thus,

$$a = \frac{(al + xl) - xl}{l} = \frac{3.63 \text{ in} - 0.463 \text{ in.}}{8.50 \text{ in.}} = 0.373$$

Interpolating from tables, with θ = 0°, a = 0.373, and k = 0.294, — Manual Table 8-8

C = 2.52

The required weld size for two such welds is

LRFD	ASD
$\phi = 0.75$ $D_{req} = \frac{R_u}{\phi C C_1 l}$ $= \frac{(50.0 \text{ kips})}{0.75(2.52)(1.0)(8.50 \text{ in.})}$ $= 3.11 \rightarrow$ 4 sixteenths	$\Omega = 2.00$ $D_{req} = \frac{\Omega R_a}{C C_1 l}$ $= \frac{2.0(33.3 \text{ kips})}{(2.52)(1.0)(8.50 \text{ in.})}$ $= 3.11 \rightarrow$ 4 sixteenths

Manual Table 8-8

Use ¼–in. fillet welds and at least 5/16 in. thick bent plates to allow for the welds.

Check beam web thickness

According to Part 9 of the Manual, with F_{EXX} = 70 ksi on both sides of the connection, the minimum thickness required to match the available shear rupture strength of the connection element to the available shear rupture strength of the base metal is — Manual Part 9

$$t_{\min} = \frac{6.19D}{F_u} = \frac{(6.19)(3.11 \text{ sixteenths})}{65 \text{ ksi}} = 0.296 \text{ in.} < 0.455 \text{ in.} \qquad \textbf{o.k.}$$

LRFD	ASD
Design bolts	*Design bolts*
Maximum shear to bent plate = 50.0 kips	Maximum shear to bent plate = 33.3 kips
Use 3 rows of 7/8 in. diameter ASTM A325-N bolts.	Use 3 rows of 7/8 in. diameter ASTM A325-N bolts.
Check shear on bolts	*Check shear on bolts*
$\phi R_n = n(\phi r_n)$ = (3 bolts)(21.6 kips/bolt) = 64.8 kips > 50.0 kips **o.k.**	$R_n / \Omega = n(r_n / \Omega)$ = (3 bolts)(14.4 kips/bolt) = 43.2 kips > 33.3 kips **o.k.**
Check bearing on support	*Check bearing on support*
ϕr_n = 102 kips/in. = (102 kips/in.)(0.490 in.)(3 bolts) = 150 kips > 50.0 kips **o.k.**	R_n / Ω = 68.3 kips/in. = (68.3 kips/in.)(0.490 in.)(3 bolts) = 100 kips > 33.3 kips **o.k.**
Design bent plates	*Design bent plates*
Try a PL 5⁄16 in.	Try a PL 5⁄16 in.

Manual Table 7-1

Manual Table 7-5

LRFD	ASD	
Check bearing on plates $\phi_v r_{ni}$ = 91.4 kips/in. $\phi_v r_{no}$= 40.8 kips/in. $\phi R_n = \left[\begin{array}{l}(91.4\text{ kips/in.})(2\text{ bolts})\\+(40.8\text{ kips / in.})(1\text{ bolt})\end{array}\right](0.313\text{ in.})$ = 70.0 kips > 50.0 kips **o.k.**	*Check bearing on plates* r_{ni} / Ω_v = 60.9 kips/in. r_{no} / Ω_v = 27.2 kips/in. $R_n/\Omega = \left[\begin{array}{l}(60.9\text{ kips/in.})(2\text{ bolts})\\+(27.2\text{ kips/in.})(1\text{ bolt})\end{array}\right](0.313\text{ in.})$ = 46.6 kips > 33.3 kips **o.k.**	Manual Table 7-5 Manual Table 7-6
Check shear yielding of plates ϕ = 1.00 $\phi R_n = \phi(0.6F_y)\, A_g$ = (1.00)(0.6)(36 ksi)(8.50 in.)(0.313 in.) = 57.6 kips > 50.0 kips **o.k.**	*Check shear yielding of plates* Ω = 1.50 $R_n / \Omega = (0.6F_y)\, A_g / \Omega$ = (0.6)(36 ksi)(8.50 in.)(0.313 in.)/1.50 = 38.3 kips > 33.3 kips **o.k.**	Eqn. J4-3
Check shear rupture of the plates $A_n = \left[(8.50\text{ in.})\text{-}(3)(1.00\text{ in.})\right](0.313\text{ in.})$ = 1.72 in.^{2} ϕ = 0.75 $\phi R_n = \phi\,(0.6F_u)\, A_n$ = (0.75)(0.6)(58 ksi)(1.72 in.^{2}) = 44.9 kips < 50.0 kips **n.g.**	*Check shear rupture of the plates* $A_n = \left[(8.50\text{ in.})\text{-}(3)(1.00\text{ in.})\right](0.313\text{ in.})$ = 1.72 in.^{2} Ω = 2.00 $R_n / \Omega = (0.6F_u)\, A_n / \Omega$ = (0.6)(58 ksi)(1.72 in.^{2})/2.00 = 29.9 kips < 33.3 kips **n.g.**	Eqn. J4-4
Increase the plate thickness to ⅜ in. $A_n = \left[(8.50\text{ in.})\text{-}(3)(1.00\text{ in.})\right](0.375\text{ in.})$ = 2.06 in.^{2} ϕ = 0.75 ϕR_n = (0.75)(0.6)(58 ksi)(2.06 in.^{2}) = 53.8 kips > 50.0 kips **o.k.**	**Increase the plate thickness to ⅜ in.** $A_n = \left[(8.50\text{ in.})\text{-}(3)(1.00\text{ in.})\right](0.375\text{ in.})$ = 2.06 in.^{2} ϕ = 0.75 R_n / Ω = (0.6)(58 ksi)(2.06 in.^{2})/2.00 = 35.8 kips > 33.3 kips **o.k.**	
Check block shear rupture of the plate with n = 3, $L_{ev} = L_{eh}$ = 1¼ in., $\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	*Check block shear rupture of the plate* with n = 3, $L_{ev} = L_{eh}$ = 1¼ in., $\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6F_y A_{gv}}{\Omega},\ \dfrac{0.6F_u A_{nv}}{\Omega}\right)$	Eqn. J4-5
Tension rupture component $\phi F_u A_{nt}$ = 32.6 kips/in.(0.375 in.) Shear yielding component $\phi 0.6 F_y A_{gv}$ = 117 kips/in.(0.375 in.)	Tension rupture component $\dfrac{F_u A_{nt}}{\Omega}$ = 21.8 kips/in.(0.375 in.) Shear yielding component $\dfrac{0.6F_y A_{gv}}{\Omega}$ = 78.3 kips/in.(0.375 in.)	Manual Table 9-3a Manual Table 9-3b Manual Table 9-3c

LRFD	ASD
Shear rupture component $\phi 0.6F_uA_{nv}$ = 124 kips/in.(0.375 in.) ϕR_n = (117 kips/in. + 32.6 kips/in.)(0.375 in.) = 56.1 kips > 50.0 kips **o.k.**	Shear rupture component $\dfrac{0.6F_uA_{nv}}{\Omega}$ = 82.6 kips/in.(0.375 in.) $\dfrac{R_n}{\Omega}$ = (78.3 kips/in.+21.8 kips/in.)(0.375 in.) = 37.5 kips > 33.3 kips **o.k.**

Thus, the configuration shown in Figure II.A-10 can be supported using ⅜-in. bent plates, and ¼-in. fillet welds.

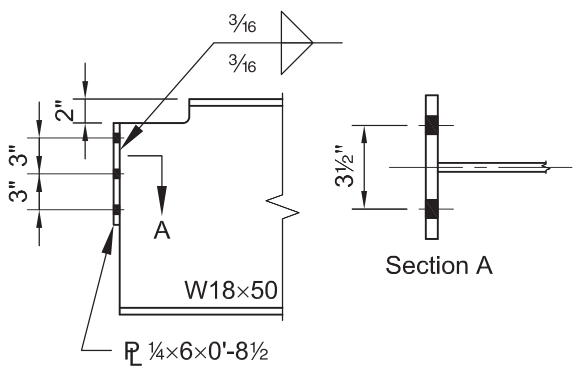
Example II.A-11 Shear End-Plate Connection (beam to girder web)

Given:

Design a shear end-plate connection to connect a W18×50 beam to W21×62 girder web, to support the following beam end reactions:

R_D = 10 kips
R_L = 30 kips

Use ¾–in. diameter ASTM A325-N or F1852-N bolts in standard holes and 70 ksi electrodes.



Material Properties:

Beam W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Girder W21×62	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Tables 2-3
Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	and 2-4

Geometric Properties:

Beam W18×50	d = 18.0 in.	t_w = 0.355 in.	S_{net} = 23.4 in.^{3}		Manual
Cope	c = 4 ¼ in.	d_c = 2 in.	e = 4½ in.	h_0 = 16.0 in.	Tables 1-1
Girder W21×62	t_w = 0.400 in.				and 9-2 &
					Figure 9-2

Solution:

LRFD	ASD
Calculate required strength	*Calculate required strength*
R_u = 1.2(10 kips) + 1.6(30 kips) = 60.0 kips	R_a = 10 kips + 30 kips = 40.0 kips
Check bolt shear. Check end plate for bolt bearing, shear yielding, shear rupture and block shear rupture.	*Check bolt shear. Check end plate for bolt bearing, shear yielding, shear rupture and block shear rupture.*
Try 3 rows of bolts and ¼ in. plate thickness	Try 3 rows of bolts and ¼ in. plate thickness
ϕR_n = 76.4 kips > 60.0 kips **o.k.**	R_n / Ω = 50.9 kips > 40.0 kips **o.k.**

Manual Table 10-4

LRFD	ASD	
Check weld shear, check beam web shear rupture Try 3⁄16-in. weld. t_{wmin} = 0.286 in. < 0.355 in. **o.k.** ϕR_n = 67.9 kips > 60.0 kips **o.k.**	*Check weld shear, check beam web shear rupture* Try 3⁄16-in. weld. t_{wmin} = 0.286 in. < 0.355 in. **o.k.** R_n/Ω = 45.2 kips > 40.0 kips **o.k.**	Manual Table 10-4
Check supporting member web or flange for bolt bearing ϕR_n = (526 kips/in.)(0.400 in.) = 210 kips > 60.0 kips **o.k.**	*Check supporting member web or flange for bolt bearing* R_n / Ω = (351 kips/in.)(0.400 in.) = 140 kips > 40.0 kips **o.k.**	Manual Table 10-4

Check coped section

As was shown in **Example II.A-4**, coped section does not control design. **o.k.**

Check web shear

As was shown in **Example II.A-4**, web shear does not control design. **o.k.**

Note: See **Example II.A-4** for an all-bolted double-angle connection and **Example II.A-5** for a bolted/welded double-angle connection.

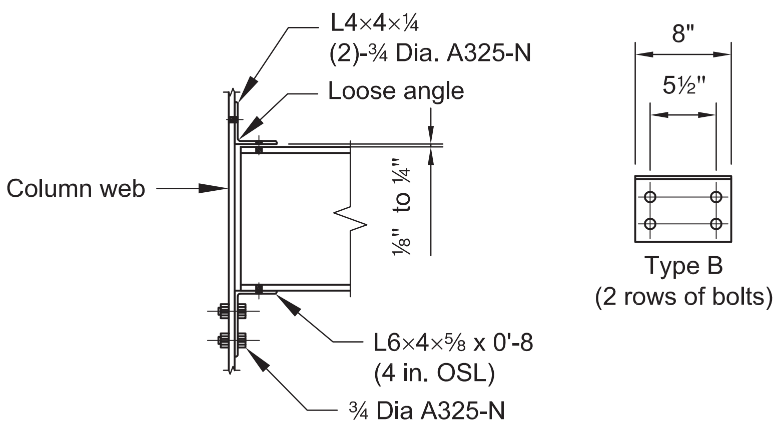
Example II.A-12 All-Bolted Unstiffened Seated Connection (beam-to-column web)

Given:

Design an all-bolted unstiffened seated connection between a W16×50 beam and W14×90 column web to support the following end reactions:

R_D = 9.0 kips
R_L = 27.5 kips

Use ¾-in. diameter ASTM A325-N or F1852-N bolts in standard holes.



Material Properties:

Beam W16×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Column W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam W16×50	t_w = 0.380 in.	d = 16.3 in.	b_f = 7.07 in.	Manual Table 1-1
	t_f = 0.630 in.	k_{des} = 1.03 in.		
Column W14×90	t_w = 0.440 in.			

Solution:

LRFD	ASD
Calculate required strength	*Calculate required strength*
$R_u = 1.2(9.0 \text{ kips}) + 1.6(27.5 \text{ kips}) = 54.8 \text{ kips}$	$R_a = 9.0 \text{ kips} + 27.5 \text{ kips} = 36.5 \text{ kips}$
Check beam web	*Check beam web*
N_{min} is the N-distance required for the limit states of local web yielding and local web crippling, but not less than k_{des}	*N_{min} is the N-distance required for the limit states of local web yielding and local web crippling, but not less than k_{des}*

Section J10.2 & J10.3

LRFD	ASD	
For local web yielding, $N_{min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{54.8 \text{ kips} - 49.0 \text{ kips}}{19.0 \text{ kips}} \geq 1.03$ $= 0.305 \text{ in.} < 1.03 \text{ in.}$ $= 1.03 \text{ in.}$ For web crippling, When $\dfrac{N}{d} \leq 0.2$ $N_{min} = \dfrac{R_u - \phi R_3}{\phi R_4}$ $= \dfrac{54.8 \text{ kips} - 67.2 \text{ kips}}{5.81 \text{ kips}}$ which results in a negative quantity. When $\dfrac{N}{d} > 0.2$ $N_{min} = \dfrac{R_u - \phi R_5}{\phi R_6}$ $= \dfrac{54.8 \text{ kips} - 60.9 \text{ kips}}{7.74 \text{ kips/in.}}$ which results in a negative quantity. Thus, $N_{min} = k_{des} = 1.03$ in.	For local web yielding, $N_{min} = \dfrac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k_{des}$ $= \dfrac{36.5 \text{ kips} - 32.7 \text{ kips}}{12.7 \text{ kips}} \geq 1.03$ $= 0.299 \text{ in.} < 1.03 \text{ in.}$ $= 1.03 \text{ in.}$ For web crippling, When $\dfrac{N}{d} \leq 0.2$ $N_{min} = \dfrac{R_a - R_3 / \Omega}{R_4 / \Omega}$ $= \dfrac{36.5 \text{ kips} - 44.8 \text{ kips}}{3.87 \text{ kips}}$ which results in a negative quantity. When $\dfrac{N}{d} > 0.2$ $N_{min} = \dfrac{R_a - R_5 / \Omega}{R_6 / \Omega}$ $= \dfrac{36.5 \text{ kips} - 40.6 \text{ kips}}{5.16 \text{ kips/in.}}$ which results in a negative quantity. Thus, $N_{min} = k_{des} = 1.03$ in.	Manual Table 9-4
Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web Try an 8 in. angle length with a ⅝ in. thickness, a 3½ in. minimum outstanding leg and $N_{req.}$ = 1.03 in. $N_{req.}$ < 11⁄16 in. Conservatively, for N =11⁄16 in. ϕR_n = 90.0 kips > 54.8 kips **o.k.** Try L6×4×⅝ (4-in. OSL), 8-in. long with 5½-in. bolt gage, connection type B (four bolts).	*Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web* Try an 8 in. angle length with a ⅝ in. thickness, a 3½ in. minimum outstanding leg and $N_{req.}$ = 1.03 in. $N_{req.}$ < 11⁄16 in. Conservatively, for N =11⁄16 in. R_n / Ω = 59.9 kips > 36.5 kips **o.k.** Try L6×4×⅝ (4-in. OSL), 8-in. long with 5½-in. bolt gage, connection type B four bolts).	Manual Table 10-5

LRFD	ASD	
For ¾-in. diameter ASTM A325-N bolts, $\phi R_n = \phi_v r_n n = 63.6 \text{ kips} > 54.8 \text{ kips}$ **o.k.**	For ¾-in. diameter ASTM A325-N bolts, $R_n/\Omega = (r_n/\Omega)n = 42.4 \text{ kips} > 36.5 \text{ kips}$ **o.k.**	Manual Table 10-5
Check bolt bearing on the angle *Required bearing strength* $r_u = \dfrac{54.8 \text{ kips}}{4 \text{ bolts}} = 13.7 \ \dfrac{\text{kips}}{\text{bolt}}$ By inspection, tear-out does not control. $\phi R_n = \phi(2.4dtF_u)$ $= 0.75(2.4)(0.750 \text{ in.})(0.625 \text{ in.})(58 \text{ ksi})$ $= 48.9 \text{ kips} > 13.7 \text{ kips}$ **o.k.**	*Check bolt bearing on the angle* *Required bearing strength* $r_a = \dfrac{36.5 \text{ kips}}{4 \text{ bolts}} = 9.13 \ \dfrac{\text{kips}}{\text{bolt}}$ By inspection, tear-out does not control. $R_n / \Omega = \dfrac{(2.4dtF_u)}{\Omega}$ $= \dfrac{(2.4)(0.750 \text{ in.})(0.625 \text{ in.})(58 \text{ ksi})}{2.00}$ $= 32.6 \text{ kips} > 9.13 \text{ kips}$ **o.k.**	Eqn J3-6a
Check supporting column Required bearing strength $r_u = \dfrac{54.8 \text{ kips}}{4 \text{ bolts}} = 13.7 \ \dfrac{\text{kips}}{\text{bolt}}$ $\phi R_n = \phi(2.4dtF_u)$ $= 0.75(2.4)(0.750 \text{ in.})(0.440 \text{ in.})(65 \text{ ksi})$ $= 38.6 \text{ kips} > 13.7 \text{ kips}$ **o.k.**	*Check supporting column* Required bearing strength $r_a = \dfrac{36.5 \text{ kips}}{4 \text{ bolts}} = 9.13 \ \dfrac{\text{kips}}{\text{bolt}}$ $R_n / \Omega = \dfrac{(2.4dtF_u)}{\Omega}$ $= \dfrac{(2.4)(0.750 \text{ in.})(0.440 \text{ in.})(65 \text{ ksi})}{2.00}$ $= 25.7 \text{ kips} > 9.13 \text{ kips}$ **o.k.**	Eqn J3-6a

Select top angle and bolts
Use an L4×4×¼ with two ¾–in. diameter ASTM A325-N or F1852-N bolts through each leg.

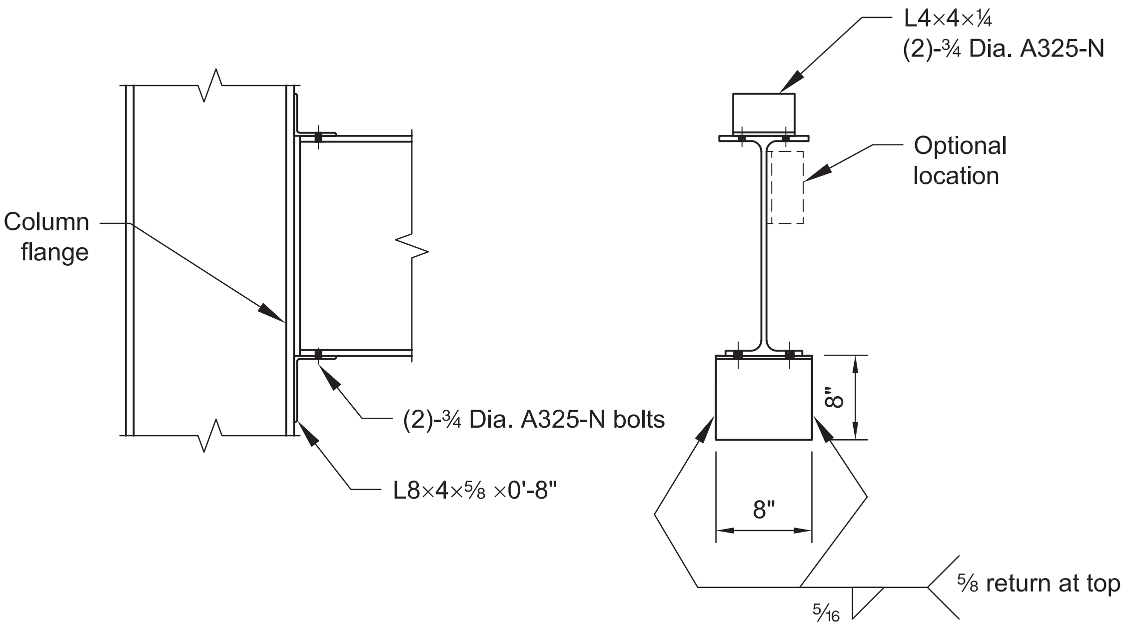
Example II.A-13 Bolted/Welded Unstiffened Seated Connection (beam-to-column flange)

Given:

Design an unstiffened seated connection between a W21×62 beam and a W14×61 column flange to support the following beam end reactions:

R_D = 9.0 kips
R_L = 27.5 kips

Use ¾–in. diameter ASTM A325-N or F1852-N bolts in standard holes to connect the supported beam to the seat and top angles. Use 70 ksi electrode welds to connect the seat and top angles to the column flange.



Note: For calculation purposes, assume setback is equal to ¾ in. to account for possible beam underrun.

Material Properties:

Beam	W21×62	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Column	W14×61	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W21×62	t_w = 0.400 in.	d = 21.0 in.	b_f = 8.24 in.	t_f = 0.615 in.	Manual Table 1-1
		k_{des} = 1.12 in.				
Column	W14×61	t_f = 0.645 in.				

Solution:

LRFD	ASD	
Calculate the required strength $R_u = 1.2(9.0 \text{ kips}) + 1.6(27.5 \text{ kips}) = 54.8 \text{ kips}$	*Calculate the required strength* R_a = 9.0 kips + 27.5 kips = 36.5 kips	
Check the strength of the beam web *N_{min} is the N-distance required for the limit states of local web yielding and local web crippling, but not less than k_{des}*	*Check the strength of the beam web* *N_{min} is the N-distance required for the limit states of local web yielding and local web crippling, but not less than k_{des}*	Section J10.2 & J10.3
For local web yielding $N_{\min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k$ $= \dfrac{54.8 \text{ kips} - 55.8 \text{ kips}}{20 \text{ kips/in.}} \geq 1.12 \text{ in.}$ which results in a negative quantity $N_{\min} = 1.12 \text{ in.}$	For local web yielding $N_{\min} = \dfrac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k$ $= \dfrac{36.5 \text{ kips} - 37.2 \text{ kips}}{13.3 \text{ kips/in.}} \geq 1.12 \text{ in.}$ which results in a negative quantity $N_{\min} = 1.12 \text{ in.}$	Manual Table 9-4
For web crippling, $\left(\dfrac{N}{d}\right)_{\max} = \dfrac{3.25 \text{ in.}}{21.0 \text{ in.}}$ $= 0.155 < 0.2$ When $\dfrac{N}{d} \leq 0.2$ $N_{\min} = \dfrac{R_u - \phi R_3}{\phi R_4}$ $= \dfrac{54.8 \text{ kips} - 71.7 \text{ kips}}{5.37 \text{ kips}}$ which results in a negative quantity. Therefore, $N_{\min} = k_{des} = 1.12 \text{ in.}$	For web crippling, $\left(\dfrac{N}{d}\right)_{\max} = \dfrac{3.25 \text{ in.}}{21.0 \text{ in.}}$ $= 0.155 < 0.2$ When $\dfrac{N}{d} \leq 0.2$ $N_{\min} = \dfrac{R_a - R_3 / \Omega}{R_4 / \Omega}$ $= \dfrac{36.5 \text{ kips} - 47.8 \text{ kips}}{3.58 \text{ kips}}$ which results in a negative quantity. Therefore, $N_{\min} = k_{des} = 1.12 \text{ in.}$	
Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web Try an 8 in. angle length with a ⅝–in. thickness and a 3½ in. minimum outstanding leg. N_{req} < 1⅛ in. Conservatively, for N = 1⅛ in.	*Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web* Try an 8 in. angle length with a ⅝–in. thickness and a 3½ in. minimum outstanding leg. N_{req} < 1⅛ in. Conservatively, for N = 1⅛ in.	

LRFD	ASD
ϕR_n =81.0 kips > 54.8 kips **o.k.** Try an L8×4×⅝ (4 in. OSL), 8 in. long with 5⁄16 in. fillet welds. ϕR_n = 66.7 kips > 54.8 kips **o.k.** Use two ¾–in. diameter ASTM A325-N bolts to connect the beam to the seat angle.	R_n / Ω = 53.9 kips > 36.5 kips **o.k.** Try an L8×4×⅝ (4 in. OSL), 8 in. long with 5⁄16 in. fillet welds. R_n / Ω = 44.5 kips > 36.5 kips **o.k.** Use two ¾–in. diameter ASTM A325-N bolts to connect the beam to the seat angle.

Manual Table 10-6

Manual Table 10-6

The strength of the bolts, welds and angles must be verified if horizontal forces are added to the connection.

Select top angle, bolts, and welds

Use L4×4×¼ with two ¾ in. diameter ASTM A325-N or F1852-N bolts through the supported-beam leg of the angle. Use a 3⁄16–in. fillet weld along the toe of the angle to the column flange. See the discussion in Manual Part 10.

Specification Sec. J2.2b

Note: See **Example II.A-12** for an all-bolted unstiffened seat connection.

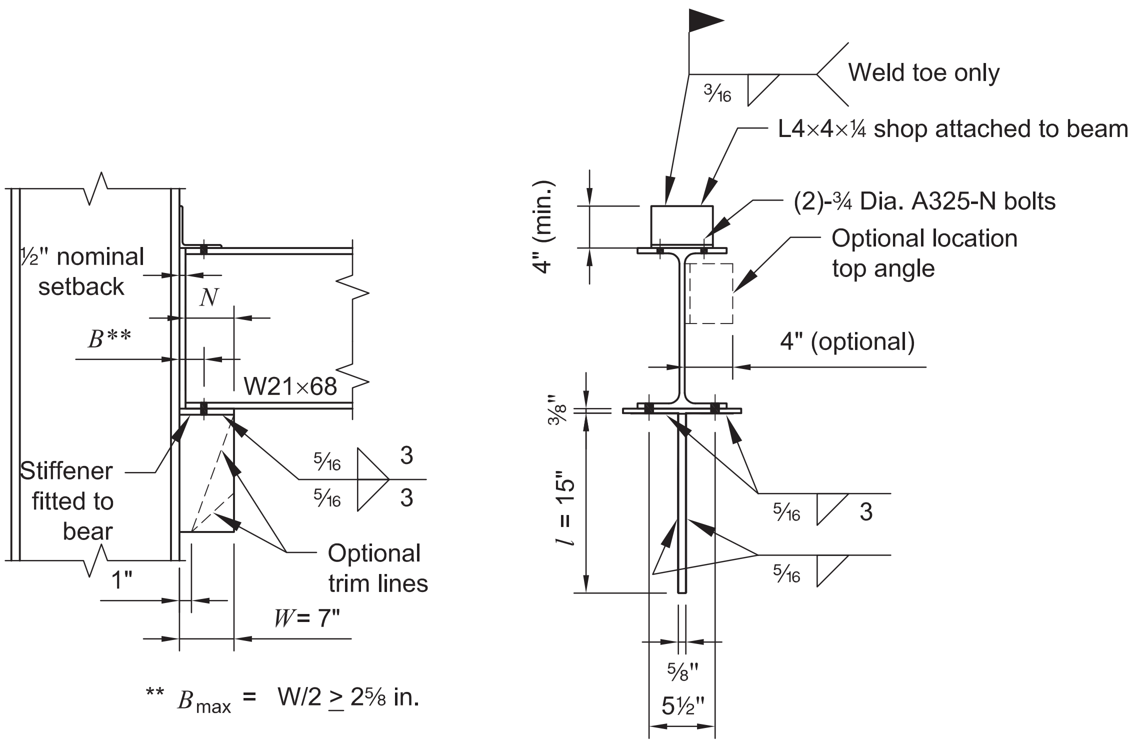
Example II.A-14 Stiffened Seated Connection (beam-to-column flange)

Given:

Design a stiffened seated connection between a W21×68 beam and a W14×90 column flange, to support the following end reactions:

R_D = 21 kips
R_L = 62.5 kips

Use ¾ in. diameter ASTM A325-N or F1852-N bolts in standard holes to connect the supported beam to the seat plate and top angle. Use 70 ksi electrode welds to connect the stiffener and top angle to the column flange.



Note: For calculation purposes, assume setback is equal to ¾ in. to account for possible beam underrun.

Material Properties:

Beam W21×68	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Column W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles and plates	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam W21×68	t_w = 0.430 in.	d = 21.1 in.	b_f = 8.27 in.	t_f = 0.685 in.	Manual Table 1-1
	k_{des} = 1.19 in.				
Column W14×90	t_f = 0.710 in.				

Solution:

LRFD	ASD
Calculate required strength R_u = 1.2(21 kips) + 1.6(62.5 kips) = 125 kips *Determine stiffener width W required* For web crippling, assume $N/d > 0.2$ $W_{min} = \dfrac{R_u - \phi R_5}{\phi R_6} + \text{ setback}$ $= \dfrac{125 \text{ kips} - 75.9 \text{ kips}}{7.94 \text{ kips/in.}} + 0.750 \text{ in.}$ = 6.93 in. For local web yielding, $W_{min} = \dfrac{R_u - \phi R_1}{\phi R_2} + \text{ setback}$ $= \dfrac{125 \text{ kips} - 63.7 \text{ kips}}{21.5 \text{ kips/in.}} + 0.750 \text{ in.}$ = 3.60 in. < 6.93 in. Use W = 7 in. *Check assumption* $\dfrac{N}{d} = \dfrac{6.93 \text{ in.} - 0.750 \text{ in.}}{21.1 \text{ in.}}$ = 0.293 > 0.2 **o.k.**	*Calculate required strength* R_a = 21 kips + 62.5 kips = 83.5 kips *Determine stiffener width W required* For web crippling, assume $N/d > 0.2$ $W_{min} = \dfrac{R_a - R_5 / \Omega}{R_6 / \Omega} + \text{ setback}$ $= \dfrac{83.5 \text{ kips} - 50.6 \text{ kips}}{5.29 \text{ kips/in.}} + 0.750 \text{ in.}$ = 6.97 in. For local web yielding, $W_{min} = \dfrac{R_a - R_1 / \Omega}{R_2 / \Omega} + \text{ setback}$ $= \dfrac{83.5 \text{ kips} - 42.5 \text{ kips}}{14.3 \text{ kips/in.}} + 0.750 \text{ in.}$ = 3.62 in. < 6.97 in. Use W = 7 in. *Check assumption* $\dfrac{N}{d} = \dfrac{6.97 \text{ in.} - 0.750 \text{ in.}}{21.1 \text{ in.}}$ = 0.295 > 0.2 **o.k.**
Determine stiffener length L and stiffener to column flange weld size Try a stiffener with L = 15 in. and 5⁄16 in. weld ϕR_n = 139 kips > 125 kips **o.k.**	*Determine stiffener length L and stiffener to column flange weld size* Try a stiffener with L = 15 in. and 5⁄16 in. weld R_n / Ω = 93.0 kips > 83.5 kips **o.k.**

Manual Table 9-4

Manual Table 9-4

Manual Table 10-8

Determine weld requirements for seat plate

Use 5⁄16–in. fillet welds on each side of the stiffener. Minimum length of seat-plate-to-column flange weld is 0.2(L) = 3 in. The weld between the seat plate and stiffener plate is required to have a strength equal to or greater than the weld between the seat plate and the column flange, use 5⁄16–in. fillet welds on each side of the stiffener to the seat plate, length of weld = 6 in. **o.k.**

Manual Part 10

Determine the seat plate dimensions

A width of 9 in. is adequate to accommodate two ¾–in. diameter ASTM A325-N bolts on a 5½ in. gage connecting the beam flange to the seat plate.

Manual Part 10

Use a PL⅜ in.×7 in.×9 in. for the seat plate.

Determine the stiffener plate thickness

Determine minimum plate thickness to develop the stiffener-to-seat-plate weld.

$$t_{min} = 2w = 2(5/16\text{ in.}) = 5/8\text{ in.}$$

Manual Section 10

Determine minimum plate thickness for a stiffener with F_y = 36 ksi and beam with F_y = 50 ksi.

$$t_{min} = \frac{50}{36}t_w = \frac{50}{36}(0.430\text{ in.})$$

$$= 0.597\text{ in.} < 5/8\text{ in.}$$

Use a PL ⅝ in.×7 in.×15 in.

Select top angle, bolts and welds

Use a L4×4×¼ with two ¾-in. diameter A325-N or F1852-N bolts through the supported-beam leg of the angle. Use a 3⁄16 in. fillet weld along the toe of the supported leg of the angle.

Section J2.2b

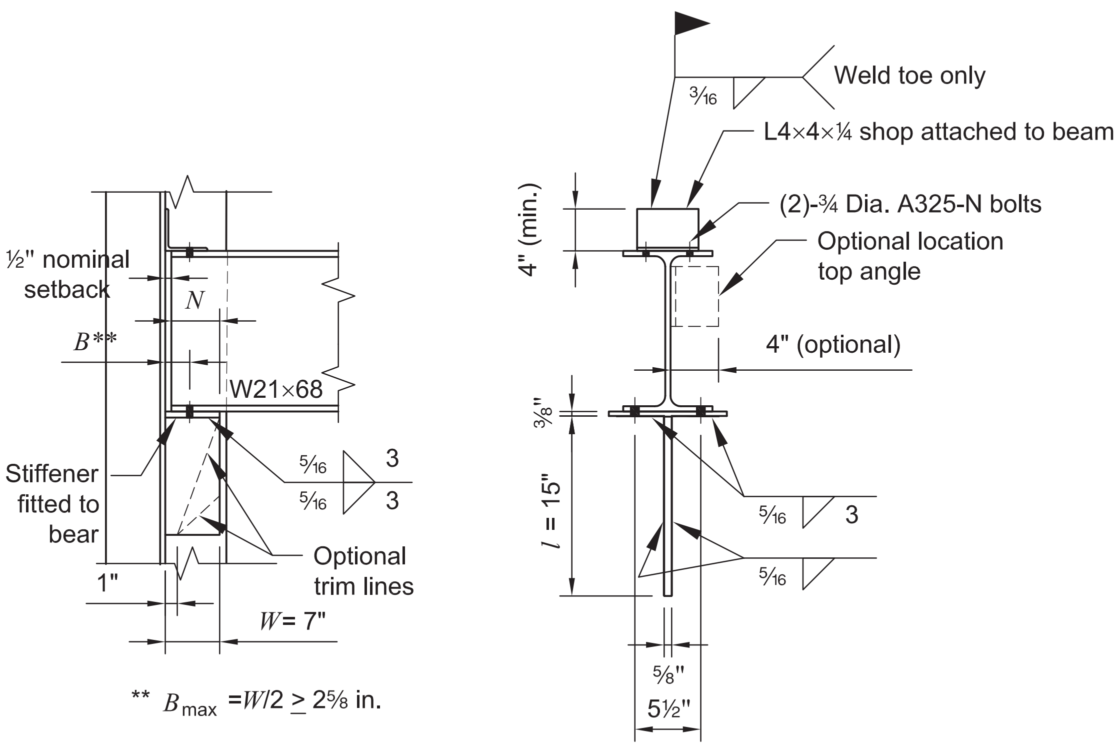
Example II.A-15 Stiffened Seated Connection (beam-to-column web)

Given:

Design a stiffened seated connection between a W21×68 beam and a W14×90 column web to support the following beam end reactions:

R_D = 21 kips
R_D = 62.5 kips

Use ¾ in. diameter ASTM A325-N or F1852-N bolts in standard holes to connect the supported beam to the seat plate and top angle. Use 70 ksi electrode welds to connect the stiffener and top angle to the column web.



Material Properties:

Beam W21×68	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Column W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles and Plates	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam W21×68	t_w = 0.430 in.	d = 21.1 in.	t_f = 0.685 in.	Manual Table 1-1
	b_f = 8.27 in.	k_{des} = 1.19 in.		
Column W14×90	t_w = 0.440 in.	T = 10 in.		

Solution:

LRFD	ASD
Calculate required strength R_u = 1.2(21 kips) + 1.6(62.5 kips) = 125 kips *Determine stiffener width W required* As previously calculated in **Example II.A-14**, use W = 7 in.	*Calculate required strength* R_a = 21 kips + 62.5 kips = 83.5 kips *Determine stiffener width W required* As previously calculated in **Example II.A-14**, use W = 7 in.
Determine stiffener length L and stiffener to column web weld size As previously calculated in **Example II.A-14**, use L = 15 in. and 5⁄16 in. weld size.	*Determine stiffener length L and stiffener to column web weld size* As previously calculated in **Example II.A-14**, use L = 15 in. and 5⁄16 in. weld size.

Determine weld requirements for seat plate

As previously calculated in **Example II.A-14**, use 3 in. of 5⁄16 in. weld on both sides of the seat plate for the seat-plate-to-column-web welds and for the seat-plate-to-stiffener welds.

Determine seat plate dimensions

For a column-web support, the maximum distance from the face of the support to the line of the bolts between the beam flange and seat plate is 3½ in. The PL ⅜ in.×7 in.×9 in. previously selected in **Example II.A-14** will accommodate these bolts.

Determine stiffener plate thickness

As previously calculated in **Example II.A-14,** use a PL ⅝ in.×7 in.×15 in.

Select top angle, bolts, and welds

Use L4×4×¼ with two ¾ in. diameter ASTM A325-N bolts through the supported-beam leg of the angle. Use a 3⁄16 in. fillet weld along the toe of the supported leg of the angle. Section J2.2b

Check the strength of the column web

If only one side of the column web has a stiffened seated connection, then Manual Part 9

$$t_{w\,\min} = \frac{3.09D}{F_u} = \frac{3.09(5\,\text{sixteenths})}{65\,\text{ksi}} = 0.238 \text{ in.}$$

If both sides of the column web have a stiffened seated connection, then

$$t_{w\,\min} = \frac{6.19D}{F_u} = \frac{6.19(5\,\text{sixteenths})}{65\,\text{ksi}} = 0.476 \text{ in.}$$

Column t_w = 0.440 in., which is sufficient for the one-sided stiffened seated connection shown.

Note: Additional detailing considerations for stiffened-seated connections are given on page 10-94 of the Manual.

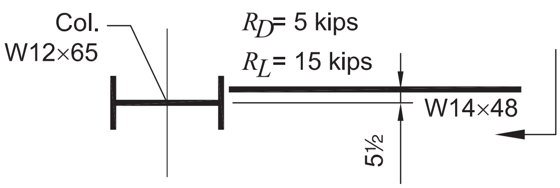
Example II.A-16 Offset Unstiffened Seated Connection (beam-to-column flange)

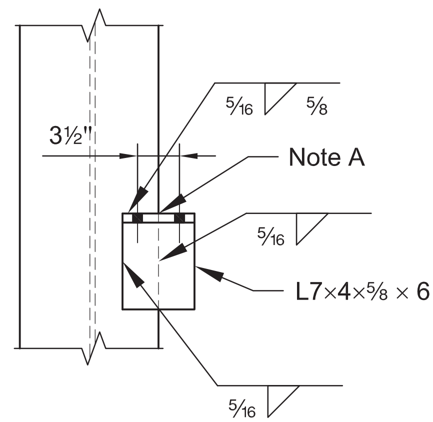
Given:

Determine the seat angle and weld size required for the unstiffened seated connection between a W14×48 beam and a W12×65 column-flange connection with an offset of 5½ in, to support the following beam end reactions:

R_D = 5.0 kips
R_L = 15 kips

Use 70 ksi electrode welds to connect the seat angle to the column flange.





Note A: End return is omitted because the AWS Code does not permit weld returns to be carried around the corner formed by the column flange toe and seat angle heel.

Note B: Beam and top angle not shown for clarity.

Note C: The nominal setback of the beam from the face of the flange is 1/2 in. A setback of 3/4 in. is used in the calculations to accommodate potential beam underrun.

Material Properties:

W14×48	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
W12×65	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angle	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

W14×48	t_w = 0.340 in.	d = 13.8 in.	b_f = 8.03 in.	t_f = 0.595 in.	Manual Table 1-1
	k_{des} = 1.19 in.				
W12×65	t_f = 0.605 in.	b_f = 12.0 in.			

Solution:

LRFD	ASD	
Calculate required strength	*Calculate required strength*	
$R_u = (1.2)(5.0\text{ kips}) + (1.6)(15\text{ kips}) = 30.0\text{ kips}$	R_a = 5.0 kips + 15 kips = 20.0 kips	
N_{min} is the N-distance required for the limit states of local web yielding and local web crippling, but not less than k_{des}	*N_{min} is the N-distance required for the limit states of local web yielding and local web crippling, but not less than k_{des}*	
For local web yielding,	For local web yielding,	
$N_{\min} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k_{des}$ $= \dfrac{(30.0\text{ kips}) - (50.4\text{ kips})}{17.0\text{ kips/in.}} > 1.19\text{ in.}$	$N_{\min} = \dfrac{R_a - (R_1/\Omega)}{(R_2/\Omega)} \geq k_{des}$ $= \dfrac{(20.0\text{ kips}) - (33.6\text{ kips})}{11.3\text{ kips/in.}} > 1.19\text{ in.}$	Manual Table 9-4
which results in a negative quantity	which results in a negative quantity	
N_{min} = 1.19 in.	N_{min} = 1.19 in.	
For web crippling,	For web crippling,	
When $\dfrac{N}{d} \leq 0.2$ $N_{\min} = \dfrac{R_u - \phi R_3}{\phi R_4}$ $= \dfrac{(30.0\text{ kips}) - (55.2\text{ kips})}{5.19\text{ kips/in.}}$	When $\dfrac{N}{d} \leq 0.2$ $N_{\min} = \dfrac{R_a - (R_3/\Omega)}{(R_4/\Omega)}$ $= \dfrac{(20.0\text{ kips}) - (36.8\text{ kips})}{3.46\text{ kips/in.}}$	Manual Table 9-4
which results in a negative quantity.	which results in a negative quantity	
Thus, $N_{req} = k_{des}$ = 1.19 in.	Thus, $N_{req} = k_{des}$ = 1.19 in.	
Design the seat angle and welds	*Design the seat angle and welds*	
The required strength for the right-hand weld can be determined by summing moments about the left-hand weld.	The required strength for the right-hand weld can be determined by summing moments about the left-hand weld.	
$R_{uR} = \dfrac{(30.0\text{ kips})(3.00\text{ in.})}{3.50\text{ in.}} = 25.7\text{ kips}$	$R_{aR} = \dfrac{(20.0\text{ kips})(3.00\text{ in.})}{3.50\text{ in.}} = 17.1\text{ kips}$	
Conservatively design the seat for twice the force in the more highly loaded weld. Therefore design the seat for	Conservatively design the seat for twice the force in the more highly loaded weld. Therefore design the seat for	Manual Part 10
R_u = 2(25.7 kips) = 51.4 kips	R_a = 2(17.1 kips) = 34.2 kips	
A 6 in. angle length with a ⅝–in. thickness provides	A 6 in. angle length with a ⅝–in. thickness provides	Manual Table 10-6

LRFD	ASD
ϕR_n = 55.2 kips > 51.4 kips **o.k.**	R_n / Ω= 36.7 kips > 34.2 kips **o.k.**
With a L7×4 (OSL) angle and 5⁄16-in. fillet welds, the weld strength from the tables is	With a L7×4 (OSL) angle and 5⁄16-in. fillet welds, the weld strength from the tables is
ϕR_n = 53.4 kips > 51.4 kips **o.k.**	R_n / Ω= 35.6 kips > 34.2 kips **o.k.**
Use L7×4× 5⁄8×6 in. for the seat angle. Use two ¾ in. diameter ASTM A325-N or F1852-N bolts to connect the beam to the seat angle and weld the angle to the column with 5⁄16-in. fillet welds.	Use L7×4×5⁄8×6 in. for the seat angle. Use two ¾ in. diameter ASTM A325-N or F1852-N bolts to connect the beam to the seat angle and weld the angle to the column with 5⁄16-in. fillet welds.
Select top angle, bolts, and welds	*Select top angle, bolts, and welds*
Use L4×4×¼ with two ¾–in. diameter ASTM A325-N or F1852-N bolts through the outstanding leg of the angle.	Use L4×4×¼ with two ¾–in. diameter ASTM A325-N or F1852-N bolts through the outstanding leg of the angle.
Use a 3⁄16–in. fillet weld along the toe of the angle to the column flange (maximum size permitted by the Specification).	Use a 3⁄16–in. fillet weld along the toe of the angle to the column flange (maximum size permitted by the Specification).

Section J2.2b

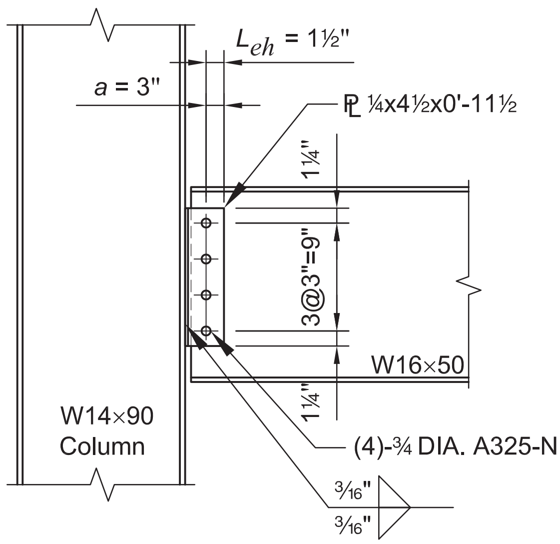
Example II.A-17 Single-Plate Connection (conventional – beam-to-column flange)

Given:

Design a single-plate connection between a W16×50 beam and a W14×90 column flange to support the following beam end reactions:

R_D = 8.0 kips
R_L = 25 kips

Use ¾–in. diameter ASTM A325-N or F1852-N bolts in standard holes and 70 ksi electrode welds.



Material Properties:

Beam	W16×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3 and 2-4
Column	W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W16×50	t_w = 0.380 in.	d = 16.3 in.	t_f = 0.630 in.	Manual Table 1-1
Column	W14×90	t_f = 0.710 in.			

Solution:

LRFD	ASD	
Calculate required strength R_u = 1.2(8.0 kips) + 1.6(25 kips) = 49.6 kips *Check bolt shear. Check plate for bolt bearing, shear yielding, shear rupture, and block shear rupture. Check weld shear.* Try four rows of bolts, ¼ in. single plate thickness, and 3⁄16 in. fillet weld size. ϕR_n = 52.2 kips > 49.6 kips **o.k.**	*Calculate required strength* R_a = 8.0 kips + 25 kips = 33.0 kips *Check bolt shear. Check plate for bolt bearing, shear yielding, shear rupture, and block shear rupture. Check weld shear.* Try four rows of bolts, ¼ in. single plate thickness, and 3⁄16 in. fillet weld size. $R_n / \Omega = 34.8 \text{ kips} > 33.0 \text{ kips}$ **o.k.**	Manual Table 10-9
Check beam web for bolt bearing. Block shear rupture, shear yielding and shear rupture will not control for an uncoped section. For an uncoped section, ϕR_n = (351 kips/in.)(0.380 in.) = 133 kips > 49.6 kips **o.k.**	*Check beam web for bolt bearing. Block shear rupture, shear yielding and shear rupture will not control for an uncoped section.* For an uncoped section, $R_n / \Omega = (234 \text{ kips/in.})(0.380 \text{ in.})$ $= 88.9 \text{ kips} > 33.0 \text{ kips}$ **o.k.**	Manual Table 10-1

Note: To provide for stability during erection, it is recommended that the minimum plate length be one-half the T-dimension of the beam to be supported. Table 10-1 may be used as a reference to determine the recommended maximum and minimum connection lengths for a supported beam.

Example II.A-18 Single-Plate Connection (beam-to-girder web)

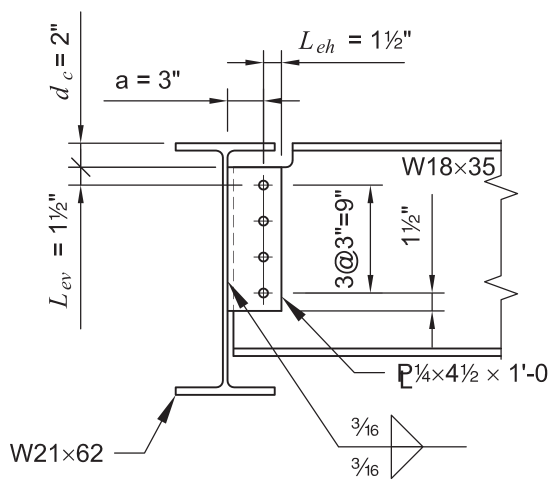
Given:

Design a single-plate connection between a W18×35 beam to a W21×62 girder web to support the following beam end reactions:

R_D = 6.5 kips
R_L = 20 kips

Top flange coped 2-in. deep by 4 -in. long, L_{ev} = 1½ in., L_{eh} = 1½ in.

Use ¾–in. diameter ASTM A325-N or F1852-N bolts in standard holes and 70 ksi electrode welds.



Material Properties:

Beam W18×35	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Girder W21×62	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Tables 2-3
Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	and 2-4

Geometric Properties:

Beam W18×35	t_w = 0.300 in.	d = 17.7 in.	t_f = 0.425 in.	Manual
Cope	c = 4.00 in.	d_c = 2.00 in.	e = 4.50 in.	Table 1-1
	h_0 = 15.7 in.			Manual
Girder W21×62	t_w = 0.400 in.			Figure 9-2

Solution:

Note: The connection plate dimensions for a ¾ in. diameter bolt in Manual Table 10-9 are based on L_{eh} = 1½ in. min. and L_{ev} (top & bottom) = 1¼ in. min.

Manual Part 10

LRFD	ASD
Calculate required strength R_u = 1.2(6.5 kips) + 1.6(20 kips) = 39.8 kips *Check bolt shear. Check plate for bolt bearing, shear yielding, shear rupture, and block shear rupture. Check weld shears.* Try four rows of bolts, ¼ in. single plate thickness, and ³⁄₁₆ in. fillet weld size. ϕR_n = 52.2 kips > 39.8 kips **o.k.**	*Calculate required strength* R_a = 6.5 kips + 20 kips = 26.5 kips *Check bolt shear. Check plate for bolt bearing, shear yielding, shear rupture, and block shear rupture. Check weld shears.* Try four rows of bolts, ¼ in. single plate thickness, and ³⁄₁₆ in. fillet weld size. $R_n / \Omega = 34.8$ kips > 26.5 kips **o.k.**
Check beam web for bolt bearing and block shear rupture. For coped section, n = 4, L_{ev} = 1½ in., L_{eh} > 1¾ in. ϕR_n = (269 kips/in.)(0.300 in.) = 80.7 kips > 40 kips **o.k.**	*Check beam web for bolt bearing and block shear rupture* For coped section, n = 4, L_{ev} = 1½ in. L_{eh} > 1¾ in. $R_n / \Omega = (180\text{ kips/in.})(0.300\text{ in.})$ = 54.0 kips > 26.5 kips **o.k.**

Manual Table 10-9

Manual Table 10-1

Check shear rupture of the girder web at the weld (W21×62)

$$t_{\min} = \frac{3.09D}{F_u} = \frac{3.09(3\,\text{sixteenths})}{65\text{ ksi}} = 0.143\text{ in.} < 0.400\text{ in.} \quad \textbf{o.k.}$$

Manual Part 9

Note: For coped beam sections, the limit states of flexural rupture, yielding and local buckling should be checked independently per Part 9. The supported beam web should also be checked for shear yielding and shear rupture per Section J4.2. However, for the shallow cope in this example, these limit states do not govern. For a illustration of these checks, see **Example II.A-4.**

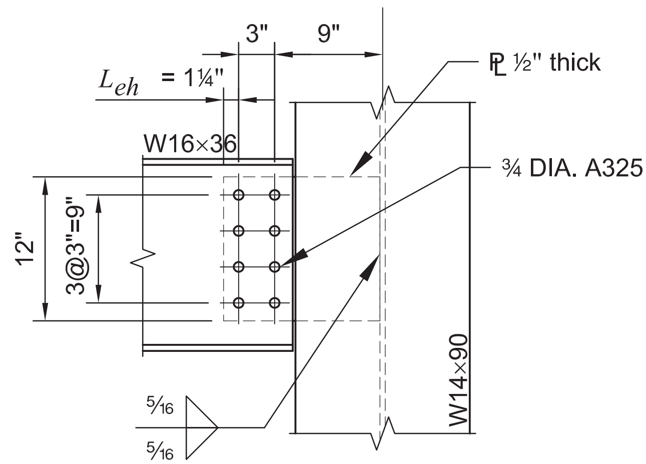
Example II.A-19 Extended Single-Plate Connection (beam-to-column web)

Given:

Design the connection between a W16×36 beam and the web of a W14×90 column, to support the following beam end reactions:

R_D = 6.0 kips
R_L = 18 kips

Use ¾-in. diameter ASTM A325-N or F1852-N bolts in standard holes. The beam is braced by the floor diaphragm. The plate is assumed to be thermally cut.



Note: all dimensional limitations are satisfied.

Material Properties:

W16×36	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Plate Material	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

W16×36	t_w = 0.295 in.	d = 15.9 in.	Manual Table 1-1
W14×90	t_w = 0.440 in.	b_f = 14.5 in.	

Solution:

Determine the required strength

LRFD	ASD
Calculate required strength R_u = 1.2(6.0 kips) + 1.6(18 kips) = 36.0 kips	*Calculate required strength* R_a = 6.0 kips + 18 kips = 24.0 kips

Determine the distance from the support to the first line of bolts and the distance to the center of gravity of the bolt group.

a = 9.00 in.
e = 9.00 in. +1.50 in. = 10.5 in.

LRFD	ASD	
Determine the bearing strength of one bolt on the beam web (tear-out does not control by inspection) $\phi r_n = (87.8 \text{ kips/in.})(0.295 \text{ in.})$ $= 25.9 \text{ kips}$	*Determine the bearing strength of one bolt on the beam web (tear-out does not control by inspection)* $\dfrac{r_n}{\Omega} = (58.5 \text{ kips/in.})(0.295 \text{ in.})$ $= 17.3 \text{ kips}$	Manual Table 7-5
Determine the shear strength of one bolt $\phi r_n = 15.9 \text{ kips}$ Therefore, shear controls over bearing *Determine the strength of the bolt group* For e = 10.5 in., C = 2.33 $\phi R_n = C\phi r_n = 2.33(15.9 \text{ kips})$ $= 37.0 \text{ kips} > 36.0 \text{ kips}$ **o.k.**	*Determine the shear strength of one bolt* $\dfrac{r_n}{\Omega} = 10.6 \text{ kips}$ Therefore, shear controls over bearing *Determine the strength of the bolt group* For e = 10.5 in., C = 2.33 $\dfrac{R_n}{\Omega} = \dfrac{Cr_n}{\Omega} = 2.33(10.6 \text{ kips})$ $= 24.7 \text{ kips} > 24.0 \text{ kips}$ **o.k.**	Manual Table 7-1 Manual Table 7-8

Determine the maximum plate thickness such that the plate will yield before the bolts shear.

$M_{max} = 1.25F_{nv}A_bC'$ — Manual Page 10-103

$1.25F_{nv} = 1.25(48 \text{ ksi}) = 60.0 \text{ ksi}$ — Manual Table J3.2

$A_b = 0.442 \text{ in.}^2$

$C' = 26.0$ — Manual Table 7-8

$$M_{max} = (60.0 \text{ ksi})(0.442 \text{ in.}^2)(26.0 \text{ in.})$$
$$= 690 \text{ kip-in.}$$

$$t_{max} = \frac{6M_{max}}{F_y d^2} = \frac{6(690 \text{ kip-in.})}{36.0 \text{ ksi}(12.0 \text{ in.})^2} = 0.799 \text{ in.}$$

Try a plate thickness of ½ in.

Check bolt bearing on the plate. Edge distance = 1.50 in.

$$R_n = 1.2L_ctF_u \le 2.4dtF_u$$ Eqn J3-6a

$$L_c = 1.50 \text{ in.} - \frac{0.813 \text{ in.}}{2} = 1.09 \text{ in.}$$

$1.2(1.09 \text{ in.})(0.500 \text{ in.})(58 \text{ ksi}) \leq 2.4(0.750 \text{ in.})(0.500 \text{ in.})(58 \text{ ksi})$
38.1 kips/bolt ≤ 52.2 kips/bolt

Use 38.1 kips/bolt

LRFD	ASD
$\phi R_n = 0.75(38.1 \text{ kips/bolt}) = 28.6 \text{ kips/bolt}$ 28.6 kips/bolt > 15.9 kip/bolt **Therefore, bolt shear controls** *Check shear yielding of the plate* $\phi = 1.00$ $\phi R_n = \phi 0.6 F_y A_g$ $= (1.00)(0.6)(36 \text{ ksi})(12.0 \text{ in.})(0.500 \text{ in.})$ $= 130 \text{ kips} > 36.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{(38.1 \text{ kips/bolt})}{2} = 19.1 \text{ kips/bolt}$ 19.1 kips/bolt > 10.6 kip/bolt **Therefore, bolt shear controls** *Check shear yielding of the plate* $\Omega = 1.50$ $R_n / \Omega = 0.6 F_y A_g / \Omega$ $= (0.6)(36 \text{ ksi})(12.0 \text{ in.})(0.500 \text{ in.})/1.50$ $= 86.4 \text{ kips} > 24.0 \text{ kips}$ **o.k.**

Eqn. J4-3

Check shear rupture of the plate

$$A_n = (t_p)\left[d - n(d_b + 0.125 \text{ in.})\right]$$
$$= (0.500 \text{ in.})\left[(12.0 \text{ in.}) - (4)(0.750 \text{ in.} + 0.125 \text{ in.})\right]$$
$$= 4.25 \text{ in.}^2$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = \phi 0.6 F_u A_n$ $= 0.75(0.6)(58 \text{ ksi})(4.25 \text{ in.}^2)$ $= 111 \text{ kips} > 36.0 \text{ kips}$ **o.k.** *Check block shear rupture of the plate* $n = 4$, $L_{ev} = 1\frac{1}{2}$ in. $L_{eh} = 4\frac{1}{4}$ in., $\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$ Tension rupture component $U_{bs} = 0.5$ $A_{nt} = (0.500 \text{ in.})(4.25 \text{ in.} - 1.5(0.875 \text{ in.}))$ $= 1.47 \text{ in.}^2$	$\Omega = 2.00$ $R_n / \Omega = \dfrac{0.6 F_u A_n}{\Omega}$ $= \dfrac{0.6(58 \text{ ksi})(4.25 \text{ in.}^2)}{2.00}$ $= 74.0 \text{ kips} > 24.0 \text{ kips}$ **o.k.** *Check block shear rupture of the plate* $n = 4$, $L_{ev} = 1\frac{1}{2}$ in. $L_{eh} = 4\frac{1}{4}$ in., $\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$ Tension rupture component $U_{bs} = 0.5$ $A_{nt} = (0.500 \text{ in.})(4.25 \text{ in.} - 1.5(0.875 \text{ in.}))$ $= 1.47 \text{ in.}^2$

Eqn. J4-4

Eqn. J4-5

Section J4.3

LRFD	ASD
$\phi F_u A_{nt} U_{bs} = (0.75)(58\text{ ksi})(1.47\text{ in.}^2)(0.5)$ $= 32.0\text{ kips}$	$\dfrac{F_u A_{nt} U_{bs}}{\Omega} = \dfrac{(58\text{ ksi})(1.47\text{ in.}^2)(0.5)}{2.00}$ $= 21.3\text{ kips}$
Shear yielding component	Shear yielding component
$\phi 0.6 F_y A_{gv} = (170\text{ kips/in.})(0.500\text{ in.})$ $= 85.0\text{ kips}$	$0.6 F_y A_{gv} / \Omega = 113\text{ kips/in.}(0.500)$ $= 56.5\text{ kips}$
Shear rupture component	Shear rupture component
$\phi 0.6 F_u A_{nv} = (194\text{ kips/in.})(0.500\text{ in.})$ $= 97.0\text{ kips}$	$0.6 F_u A_{nv} / \Omega = 129\text{ kips/in.}(0.500)$ $= 64.5\text{ kips}$
$\phi R_n = (32.0\text{ kips} + 85.0\text{ kips})$ $= 117\text{ kips} > 36.0\text{ kips}$ **o.k.**	$R_n / \Omega = (21.3\text{ kips} + 56.5\text{ kips}) = 77.8\text{ kips}$ $= 77.8\text{ kips} > 24.0\text{ kips}$ **o.k.**
Check flexural strength of the plate	*Check flexural strength of the plate*
The required strength is	The required strength is
$M_u = R_u\, a = (36.0\text{ kips})(9.00\text{ in.})$ $= 324\text{ kip-in.}$	$M_a = R_a\, a = (24.0\text{ kips})(9.00\text{ in.})$ $= 216\text{ kip-in.}$
Determine critical flexural stress in presence of shear stress, f_v	*Determine critical flexural stress in presence of shear stress, f_v*
$f_v = \dfrac{36.0\text{ kips}}{(0.500\text{ in.})(12.0\text{ in.})} = 6.00\text{ ksi}$	$f_v = \dfrac{24.0\text{ kips}}{(0.500\text{ in.})(12.0\text{ in.})} = 4.00\text{ ksi}$

Manual Table 9-3b

Manual Table 9-3c

Check local buckling of the plate

This check is analogous to the local buckling check for doubly coped beams as illustrated in the Manual Part 9 where c = 9 in. and h_o = 12 in.

$$\lambda = \frac{h_o\sqrt{F_y}}{10t_w\sqrt{475 + 280\left(\dfrac{h_o}{c}\right)^2}}$$

$$= \frac{(12.0\text{ in.})\sqrt{36\text{ ksi}}}{10(0.500\text{ in.})\sqrt{475 + 280\left(\dfrac{12.0\text{ in.}}{9.00\text{ in.}}\right)^2}} = 0.462$$

Manual Part 9

$\lambda \leq 0.7$, therefore, $Q = 1.0$

$QF_y = F_y$

Therefore, plate buckling is not a controlling limit state.

LRFD	ASD
$\phi F_{cr} = \sqrt{\left(\phi Q F_y\right)^2 - 3 f_v^2}$ $= \sqrt{\left(0.90\left(36 \text{ ksi}\right)\right)^2 - 3\left(6.00 \text{ ksi}\right)^2}$ $= 30.7 \text{ ksi}$ $\phi M_n = \phi F_{cr} S = 32.1 \text{ ksi} \dfrac{\left(0.500 \text{ in.}\right)\left(12.0 \text{ in.}\right)^2}{6}$ $= 385 \text{ kip-in.} > 324 \text{ kip-in.}$ **o.k.**	$\dfrac{F_{cr}}{\Omega} = \sqrt{\left(\dfrac{QF_y}{\Omega}\right)^2 - 3 f_v^2}$ $= \sqrt{\left(\dfrac{36 \text{ ksi}}{1.67}\right)^2 - 3\left(4.00 \text{ ksi}\right)^2}$ $= 20.4 \text{ ksi}$ $\dfrac{M_n}{\Omega} = \dfrac{F_{cr}}{\Omega} S = 20.4 \text{ ksi} \dfrac{\left(0.500 \text{ in.}\right)\left(12.0 \text{ in.}\right)^2}{6}$ $= 245 \text{ kip-in.} > 216 \text{ kip-in.}$ **o.k.**

Manual Part 10

$Z_{net} = 12.8 \text{ in.}^3$

Manual Table 15-2

LRFD	ASD
For flexural rupture $\phi = 0.75$ $\phi M_n = \phi F_u Z_{net}$ $= 0.75\left(58 \text{ ksi}\right)\left(12.8 \text{ in.}^3\right)$ $= 557 \text{ kip-in.} > 324 \text{ kip-in.}$ **o.k.**	For flexural rupture $\Omega = 2.00$ $M_n/\Omega = F_u Z_{net}/\Omega$ $= \dfrac{\left(58 \text{ ksi}\right)\left(12.8 \text{ in.}^3\right)}{2.00}$ $= 371 \text{ kip-in.} > 216 \text{ kip-in.}$ **o.k.**

Determine the weld between the plate and the column web.

$w = \dfrac{5}{8} t_p = \dfrac{5}{8}\left(0.500 \text{ in.}\right) = 0.313 \text{ in.}$; therefore, use a 5⁄16 in. fillet weld on both sides of the plate.

Manual Part 10

Check the Strength of the column web

$t_{\min} = \dfrac{3.09\text{D}}{F_u} = \dfrac{3.09\left(5 \text{sixteenths}\right)}{65 \text{ ksi}} = 0.238 \text{ in.} < 0.440 \text{ in.}$ **o.k.**

Manual Part 9

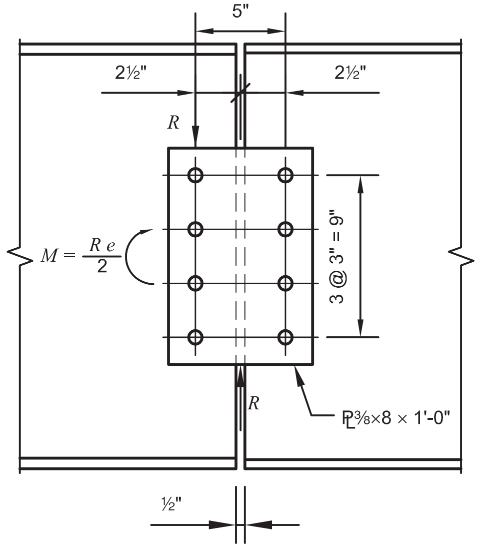
Example II.A-20 All-Bolted Single-Plate Shear Splice

Given:

Design an all-bolted single-plate shear splice between a W24×55 beam and a W24×68 beam.

R_D = 10 kips
R_L = 30 kips

Use ⅞-in. diameter ASTM A325-N or F1852-N bolts in standard holes with 5 in. between vertical bolt rows.



Material Properties:

W24×55	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
W24×68	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Splice Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam W24×55	t_w = 0.395 in.	Manual Table 1-1
Beam W24×68	t_w = 0.415 in.	

Solution:

Design the bolt groups

Note: When the splice is symmetrical, the eccentricity of the shear to the center of gravity of either bolt group is equal to half the distance between the centroids of the bolt groups. Therefore, each bolt group can be designed for the shear, R_u or R_a, and one-half the eccentric moment, R_ue or R_ae.

Using a symmetrical splice, each bolt group will carry one-half the eccentric moment. Thus, the eccentricity on each bolt group, $e/2$ = 2½ in.

LRFD	ASD
Calculate the required strength R_u = 1.2(10 kips) + 1.6(30 kips) = 60.0 kips For bolt shear, ϕr_n = 21.6 kips/bolt	*Calculate the required strength* R_a = 10 kips + 30 kips = 40.0 kips For bolt shear, r_n/Ω = 14.4 kips/bolt

Manual Table 7-1

For bearing on a ⅜ in. splice plate

Note: The available bearing strength based on edge distance will conservatively be used for all of the bolts.

$$r_n = 1.2L_c tF_u \;\leq\; 2.4dtF_u$$

Eqn. J3-6a

$$L_c = 1.50\text{ in.} - \frac{0.938\text{ in.}}{2} = 1.03\text{ in.}$$

$$1.2(1.03\text{ in.})(0.375\text{ in.})(58\text{ ksi}) \leq 2.4(0.875\text{ in.})(0.375\text{ in.})(58.0\text{ ksi})$$

$$26.9\text{ kips/bolt} \leq 45.7\text{ kips/bolt}$$

Use r_n = 26.9 kips/bolt

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(26.9\text{ kips}) = 20.2\text{ kips/bolt}$ Note: By inspection bearing on the webs of the W24 beams will not govern.	$\Omega = 2.00$ $r_n/\Omega = 26.9\text{ kips}/2.00 = 13.5\text{ kips/bolt}$ Note: By inspection bearing on the webs of the W24 beams will not govern.
Since bearing is more critical, $C_{min} = \dfrac{R_u}{\phi r_n} = \dfrac{60.0\text{ kips}}{20.2\text{ kips/bolt}} = 2.97$ with θ = 0° and e_x = 2½ in., a four-bolt connection provides C = 3.07 > 2.97 **o.k.**	Since bearing is more critical, $C_{min} = \dfrac{R_a}{r_n/\Omega} = \dfrac{40.0\text{ kips}}{13.5\text{ kips/bolt}} = 2.96$ with θ = 0° and e_x = 2½ in., a four-bolt connection provides C = 3.07 > 2.96 **o.k.**
Design splice plate Try PL ⅜ in. ×8 in. ×1'-0". *Required flexural strength of the plate* $M_u = \dfrac{R_u e}{2} = \dfrac{(60.0\text{ kips})(5.00\text{ in.})}{2}$ = 150 kip-in. For flexural yielding,	*Design splice plate* Try PL ⅜ in. ×8 in. ×1'-0". *Required flexure strength of the plate* $M_a = \dfrac{R_a e}{2} = \dfrac{(40.0\text{ kips})(5.00\text{ in.})}{2}$ = 100 kip-in. For flexural yielding,

LRFD	ASD	
$\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= (0.90)(36\text{ ksi})\left[\dfrac{(0.375\text{ in.})(12.0\text{ in.})^2}{4}\right]$ $= 437\text{ kip-in.} > 150\text{ kip-in.}$	$\Omega = 1.67$ $M_n/\Omega = F_y Z_x / \Omega$ $= \dfrac{36\text{ ksi}}{1.67}\left[\dfrac{(0.375\text{ in.})(12.0\text{ in.})^2}{4}\right]$ $= 291\text{ kip-in.} > 100\text{ kip-in.}$	Manual Part. 15

For flexure rupture,

Manual
Table 15-2

$Z_{net} = 9.00\text{ in.}^3$

LRFD	ASD	
$\phi = 0.75$ $\phi M_n = \phi F_u Z_{net}$ $= (0.75)(58\text{ ksi})(9.00\text{ in}^3)$ $= 392\text{ kip-in.} > 150\text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $M_n / \Omega = F_u Z_{net} / \Omega$ $= (58\text{ ksi})(9.00\text{ in}^3) / 2.00$ $= 261\text{ kip-in.} > 100\text{ kip-in.}$ **o.k.**	Manual Part 15
Check shear yielding of the plate	*Check shear yielding of the plate*	
$\phi = 1.00$ $\phi R_n = \phi(0.6 F_y A_g)$ $= 1.00(0.6(36\text{ ksi})(12.0\text{ in.})(0.375\text{ in.}))$ $= 97.2\text{ kips} > 60.0\text{ kips}$ **o.k.**	$\Omega = 1.50$ $R_n / \Omega = (0.6 F_y A_g) / \Omega$ $= 0.6(36\text{ ksi})(12.0\text{ in.})(0.375\text{ in.}) / 1.50$ $= 64.8\text{ kips} > 40.0\text{ kips}$ **o.k.**	Eqn. J4-3
Check shear rupture of the plate	*Check shear rupture of the plate*	
$A_n = [12.0\text{ in.} - 4(1.00\text{ in.})](0.375\text{ in.})$ $= 3.00\text{ in.}^2$	$A_n = [12.0\text{ in.} - 4(1.00\text{ in.})](0.375\text{ in.})$ $= 3.00\text{ in.}^2$	
$\phi = 0.75$ $\phi R_n = \phi(0.6 F_u A_n)$ $= 0.75(0.6)(58\text{ ksi})(3.00\text{ in.}^2)$ $= 78.3\text{ kips} > 60\text{ kips}$ **o.k.**	$\Omega = 2.00$ $R_n / \Omega = (0.6 F_u A_{nv}) / \Omega$ $= 0.6(58\text{ ksi})(3.00\text{ in.}^2)/2.00$ $= 52.2\text{ kips} > 40\text{ kips}$ **o.k.**	Eqn. J4-4
Check block shear rupture of the plate	*Check block shear rupture of the plate*	
$L_{eh} = L_{ev} = 1½\text{ in.}$	$L_{eh} = L_{ev} = 1½\text{ in.}$	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv})$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn. J4-5
Because maximum shear stress occurs at one row of bolts	Because maximum shear stress occurs at one row of bolts	
$U_{bs} = 1.00$	$U_{bs} = 1.00$	Commentary Section J4.3
Tension rupture component	Tension rupture component	

LRFD	ASD	
$\phi F_u A_{nt}$ = 43.5 kips/in.(0.375 in.)	$F_u A_{nt} / \Omega$ = 29.0 kips/in.(0.375 in.)	Manual Table 9-3a
Shear yielding component	Shear yielding component	
$\phi 0.6 F_y A_{gv}$ = 170 kips/in.(0.375 in.)	$0.6 F_y A_{gv} / \Omega$ = 113 kips/in.(0.375 in.)	Manual Table 9-3b
Shear rupture component	Shear rupture component	
$\phi 0.6 F_u A_{nv}$ = 183 kips/in.(0.375 in.)	$0.6 F_u A_{nv} / \Omega$ = 122 kips/in.(0.375 in.)	Manual Table 9-3c
ϕR_n = (170 kips/in. + 43.5 kips/in.)(0.375in.) = 80.1 kips > 60.0 kips **o.k.**	$R_n/\Omega = \left(113 \text{ kips/in.} + 29.0 \text{ kips/in.}\right) 0.375 \text{ in.}$ = 53.3 kips > 40.0 kips **o.k.**	
Use PL ⅜ in.×8 in.×1'-0"	Use PL ⅜ in.×8 in.×1'-0"	

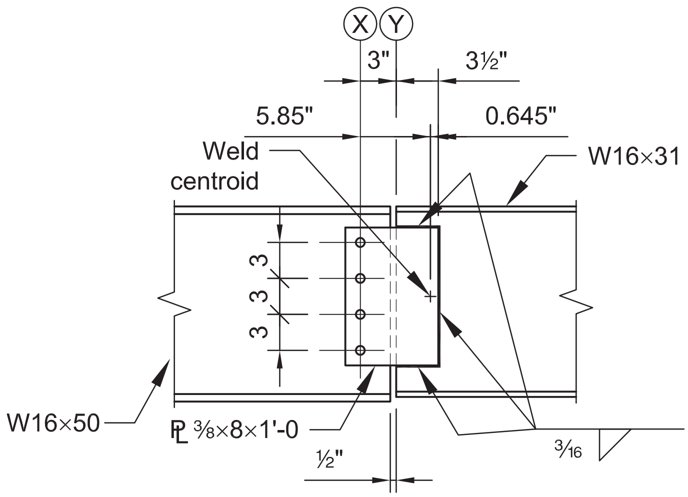
Example II.A-21 Bolted/Welded Single-Plate Shear Splice

Given:

Design a single-plate shear splice between a W16×31 beam and W16×50 beam to support the following beam end reactions:

R_D = 8.0 kips
R_L = 24.0 kips

Use ¾-in. diameter ASTM A325-N or F1852-N bolts through the web of the W16×50 and 70 ksi electrode welds to the web of the W16×31.



Material Properties:

W16×31	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
W16×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Splice Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

W16×31	t_w = 0.275 in.	Manual Table 1-1
W16×50	t_w = 0.380 in.	

Solution:

Calculate the required strength

LRFD	ASD
R_u = 1.2(8.0 kips) + 1.6(24 kips) = 48.0 kips	R_a = 8.0 kips + 24 kips = 32.0 kips

Design the weld group

Since the splice is unsymmetrical and the weld group is more rigid, it will be designed for the full moment from the eccentric shear.

Assume PL ⅜ in.×8 in.×1'-0"

$$k = \frac{kl}{l} = \frac{3.50 \text{ in.}}{12.0 \text{ in.}} = 0.292$$

$$xl = \frac{(kl)^2}{2kl + l} = \frac{(3.50)^2}{2(3.50 \text{ in.}) + 12.0 \text{ in.}} = 0.645 \text{ in.}$$

$$al = 6.50 \text{ in.} - 0.645 \text{ in.} = 5.86 \text{ in.}, \; a = \frac{al}{l} = \frac{5.86 \text{ in.}}{12.0 \text{ in.}} = 0.488$$

By interpolation, with $\theta = 0°$, $C = 2.15$ and the required weld size is — Manual Table 8-8

LRFD	ASD
$D_{req} = \frac{P_u}{\phi C C_1 l} = \frac{(48.0 \text{ kips})}{0.75(2.15)(1.0)(12.0 \text{ in.})}$ $= 2.48 \rightarrow 3$ sixteenths	$D_{req} = \frac{P_a \Omega}{C C_1 l} = \frac{(32.0 \text{ kips})(2.00)}{(2.15)(1.0)(12.0 \text{ in.})}$ $= 2.48 \rightarrow 3$ sixteenths
The minimum weld size is $^3/_{16}$ in.	The minimum weld size is $^3/_{16}$ in.
Use a $^3/_{16}$–in. fillet weld.	Use a $^3/_{16}$–in. fillet weld.

Table J2.4

Check shear rupture of beam web at the weld (W16×31)

For fillet welds with F_{exx} = 70 ksi on one side of the connection, the minimum thickness required to match the available shear rupture strength of the connection element to the available shear rupture strength of the base metal is:

$$t_{min} = \frac{3.09D}{F_u} = \frac{(3.09)(2.48 \text{ sixteenths})}{65 \text{ ksi}} = 0.118 < 0.275 \text{ in.} \quad \textbf{o.k.}$$

Manual Part 9

Design the bolt group

Since the weld group was designed for the full eccentric moment, the bolt group will be designed for shear only.

LRFD	ASD
For bolt shear	For bolt shear
ϕr_n = 15.9 kips/bolt	r_n / Ω = 10.6 kips/bolt
For bearing on the ⅜-in. thick single plate, conservatively use the design values provided for L_e = 1¼ in.	For bearing on the ⅜-in. thick single plate, conservatively use the design values provided for L_e = 1 ¼ in.
Note: By inspection, bearing on the web of the W16x50 beam will not govern.	Note: By inspection, bearing on the web of the W16x50 beam will not govern.
$\phi r_n = (44.0 \text{ kips/in./bolt})(0.375 \text{ in.})$ $= 16.5$ kips/bolt	$r_n / \Omega = (29.4 \text{ kips/in./bolt})(0.375 \text{ in.})$ $= 11.0$ kips/bolt

Manual Table 7-1

Manual Table 7-6

LRFD	ASD
Since bolt shear is more critical than bearing, $n_{min} = \dfrac{R_u}{\phi r_n}$ $= \dfrac{48.0 \text{ kips}}{15.9 \text{ kips/bolt}}$ $= 3.02 \rightarrow 4 \text{ bolts}$	Since bolt shear is more critical than bearing, $n_{min} = \dfrac{R_u}{r_n / \Omega}$ $= \dfrac{32.0 \text{ kips}}{10.6 \text{ kips/bolt}}$ $= 3.02 \rightarrow 4 \text{ bolts}$

Design the single plate

As before, try a PL ⅜ in.×8 in.×1'-0"

LRFD	ASD	
Check flexure of the plate $M_u = R_u e = (48.0 \text{ kips})(5.86 \text{ in.})$ $= 281 \text{ kip-in.}$ For flexural yielding $\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= (0.9)(36 \text{ ksi})\left[\dfrac{(0.375 \text{ in.})(12.0 \text{ in.})^2}{4}\right]$ $= 437 \text{ kip-in.} > 281 \text{ kip-in.}$ **o.k.**	*Check flexure of the plate* $M_a = R_a e = (32.0 \text{ kips})(5.86 \text{ in.})$ $= 188 \text{ kip-in.}$ For flexural yielding $\Omega = 1.67$ $M_n / \Omega = F_y Z_x / \Omega$ $= \dfrac{36 \text{ ksi}}{1.67}\left[\dfrac{(0.375 \text{ in.})(12.0 \text{ in.})^2}{4}\right]$ $= 291 \text{ kip-in.} > 187 \text{ kip-in.}$ **o.k.**	Manual Part 15
Check shear yielding of the plate $\phi = 1.00$ $\phi R_n = \phi(0.6 F_y A_g)$ $= 1.00[0.6(36 \text{ ksi})(12.0 \text{ in.})(0.375 \text{ in.})]$ $= 97.2 \text{ kips} > 48.0 \text{ kips}$ **o.k.** *Check shear rupture of the plate* $A_n = [12.0 \text{ in.} - 4(0.875 \text{ in.})](0.375 \text{ in.})$ $= 3.19 \text{ in.}^2$ $\phi = 0.75$ $\phi R_n = \phi(0.6 F_u A_n)$ $= 0.75(0.6)(58 \text{ ksi})(3.19 \text{ in.}^2)$ $= 83.3 \text{ kips} > 48.0 \text{ kip}$ **o.k.** *Check block shear rupture of the plate* $L_{eh} = L_{ev} = 1\frac{1}{2} \text{ in.}$	*Check shear yielding of the plate* $\Omega = 1.50$ $R_n / \Omega = (0.6 F_y A_g) / \Omega$ $= \dfrac{0.6(36 \text{ ksi})(12.0 \text{ in.})(0.375 \text{ in.})}{1.50}$ $= 64.8 \text{ kips} > 32.0 \text{ kips}$ **o.k.** *Check shear rupture of the plate* $A_n = [12.0 \text{ in.} - 4(0.875 \text{ in.})](0.375 \text{ in.})$ $= 3.19 \text{ in.}^2$ $\Omega = 2.00$ $R_n / \Omega = (0.6 F_u A_n) / \Omega$ $= \dfrac{0.6(58 \text{ ksi})(3.19 \text{ in.}^2)}{2.00}$ $= 55.5 \text{ kips} > 32.0 \text{ kips}$ **o.k.** *Check block shear rupture of the plate* $L_{eh} = L_{ev} = 1\frac{1}{2} \text{ in.}$	Manual Part 9 Eqn. J4-3 Eqn. J4-4

LRFD	ASD	
Thus,	Thus,	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn. J4-5
U_{bs} = 1.0	U_{bs} = 1.0	
Tension rupture component	Tension rupture component	
$\phi F_u A_{nt}$ = 46.2 kips/in.(0.375 in.)	$F_u A_{nt} / \Omega$ = 30.8 kips/in.(0.375 in.)	Manual Table 9-3a
Shear yielding component	Shear yielding component	
$\phi 0.6 F_y A_{gv}$ = 170 kips/in.(0.375 in.)	$0.6 F_y A_{gv} / \Omega$ = 113 kips/in.(0.375 in.)	Manual Table 9-3b
Shear rupture component	Shear rupture component	
$\phi 0.6 F_u A_{nv}$ = 194 kips/in.(0.375 in.)	$0.6 F_u A_{nv} / \Omega$ = 129 kips/in.(0.375 in.)	Manual Table 9-3c
ϕR_n = (170 kips/in. + 46.2 kips/in.)(0.375in.) = 81.1 kips > 48.0 kips **o.k.**	R_n / Ω = (113 kips/in. + 30.8 kips/in.)0.375in. = 53.9 kips > 32.0 kips **o.k.**	
Use PL ⅜ in.×8 in.×1'-0"	Use PL ⅜ in.×8 in.×1'-0"	

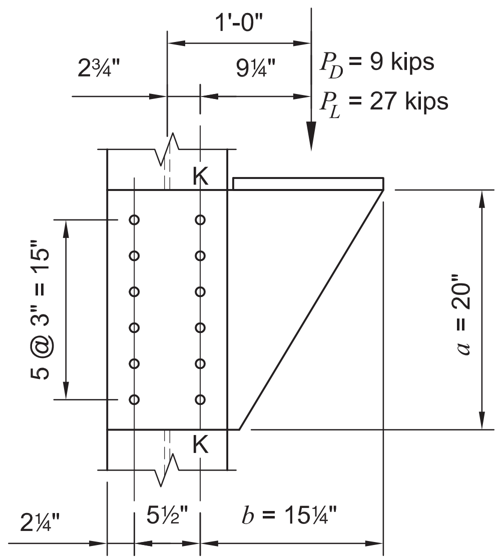
Example II.A-22 Bolted Bracket Plate Design

Given:

Design a bracket plate to support the following loads:

$P_D = 9.0$ kips
$P_L = 27$ kips

Use ¾–in. diameter ASTM A325-N or 1852-N bolts in standard holes
Assume the column has sufficient available strength for the connection.



Material Properties:

Plate Material	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	Manual Table 2-4

Solution:

LRFD	ASD	
Calculate required strength	*Calculate required strength*	
$R_u = 1.2(9.0 \text{ kips}) + 1.6(27 \text{ kips}) = 54.0 \text{ kips}$	$R_a = 9.0 \text{ kips} + 27 \text{ kips} = 36.0 \text{ kips}$	
Design bolts	*Design bolts*	
For bolt shear,	For bolt shear,	
$\phi r_n = 15.9$ kips	$r_n/\Omega = 10.6$ kips	Manual Table 7-1
For bearing on the bracket plate	For bearing on the bracket plate	
Try PL⅜ in. × 20 in., $L_e \geq 1^{15}/_{16}$ in.	Try PL⅜ in. × 20 in., $L_e \geq 1^{15}/_{16}$ in.	
$\phi r_n = (78.3 \text{ kips/bolt})(0.375 \text{ in.})$ $= 29.4$ kips/bolt	$r_n / \Omega = (52.2 \text{ kips/bolt})(0.375 \text{ in.})$ $= 19.6$ kips/bolt	Manual Table 7-6

LRFD	ASD
Since this is greater than the single-shear strength of one bolt, bolt bearing is not critical. $C_{min} = \dfrac{R_u}{\phi r_n}$ $= \dfrac{54.0 \text{ kips}}{15.9 \text{ kips/bolt}}$ $= 3.40$ For $\theta = 0°$, a 5½ in. gage with s = 3 in., e_x = 12.0 in., and n = 6 $C = 4.53 > 3.40$ **o.k.**	Since this is greater than the single-shear strength of one bolt, bolt bearing is not critical. $C_{min} = \dfrac{R_a}{r_n / \Omega}$ $= \dfrac{36.0 \text{ kips}}{10.6 \text{ kips/bolt}}$ $= 3.40$ For $\theta = 0°$, a 5½ in. gage with s = 3 in., e_x = 12.0 in., and n = 6 $C = 4.53 > 3.40$ **o.k.**
Check flexure in the bracket plate On line K, the required strength M_u is $M_u = P_u e_b$ = (54.0 kips)(12.0 in. – 2.75 in.) = 500 kip-in. For flexural yielding on line K, $\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(36 \text{ ksi})\left(\dfrac{(0.375 \text{ in.})(20.0 \text{ in.})^2}{4}\right)$ = 1220 kip-in. > 500 kip-in. **o.k.**	*Check flexure in the bracket plate* On line K, the required strength M_a is $M_a = P_a e_b$ = (36.0 kips)(12.0 in. – 2.75 in.) = 333 kip-in. For flexural yielding on line K, $\Omega = 1.67$ $M_n / \Omega = F_y Z_x / \Omega$ $= \dfrac{(36 \text{ ksi})\left(\dfrac{(0.375 \text{ in.})(20.0 \text{ in.})^2}{4}\right)}{1.67}$ = 808 kip-in. > 333 kip-in. **o.k.**

Manual Table 7-9

For flexural rupture on line K,

$$Z_{net} = 0.375 \text{ in.}\left[\frac{(20.0 \text{ in.})^2}{4} - 2(1.50 \text{ in.} + 4.50 \text{ in.} + 7.50 \text{ in.})(0.875 \text{ in.})\right]$$
$$= 28.6 \text{ in.}^3$$

LRFD	ASD
$\phi = 0.75$ $\phi M_n = \phi F_u Z_{net} = 0.75(58 \text{ ksi})(28.6 \text{ in.}^3)$ = 1240 kip-in. > 500 kip-in. **o.k.**	$\Omega = 2.00$ $M_n / \Omega = \dfrac{F_u Z_{net}}{\Omega} = \dfrac{(58 \text{ ksi})(28.6 \text{ in.}^3)}{2.00}$ = 829 kip-in. > 333 kip-in. **o.k.**

Manual Part 15

For flexural yielding on the free edge of the triangular plate,

Manual Part 15

$$z = 1.39 - 2.2\left(\frac{b}{a}\right) + 1.27\left(\frac{b}{a}\right)^2 - 0.25\left(\frac{b}{a}\right)^3$$
$$= 1.39 - 2.2\left(\frac{15.3 \text{ in.}}{20 \text{ in.}}\right) + 1.27\left(\frac{15.3 \text{ in.}}{20 \text{ in.}}\right)^2 - 0.25\left(\frac{15.3 \text{ in.}}{20 \text{ in.}}\right)^3 = 0.338$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = \phi F_y zbt$ $= 0.90(36 \text{ ksi})(0.338)(15.3 \text{ in.})(0.375 \text{ in.})$ $= 62.8 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $P_n / \Omega = \dfrac{F_y zbt}{\Omega}$ $= \dfrac{(36 \text{ ksi})(0.338)(15.3 \text{ in.})(0.375 \text{ in.})}{1.67}$ $= 41.8 \text{ kips} > 36.0 \text{ kips}$ **o.k.**

Check local buckling of the bracket plate

Note: Manual Part 5

0.6b=0.6(15.3 in.) = 9.18 in. ≈ 9.25 in. and lateral movement of the outstanding portion of the bracket plate is prevented, therefore, the following limit is applicable.

$$\frac{b}{a} = \frac{15.3 \text{ in.}}{20.0 \text{ in.}} = 0.765$$

Since $0.5 < \dfrac{b}{a} \leq 1.0$,

$$t_{min} = b\left(\frac{\sqrt{F_y}}{250}\right) = (15.3 \text{ in.})\left(\frac{\sqrt{36 \text{ ksi}}}{250}\right) = 0.367 \text{ in.} < \tfrac{3}{8} \text{ in} \quad \textbf{o.k.}$$

LRFD	ASD
Check shear yielding of the bracket plate $\phi = 1.00$ $\phi R_n = \phi(0.6F_y)A_g$ $= 1.0(0.6)(36 \text{ ksi})(20.0 \text{ in.})(0.375 \text{ in.})$ $= 162 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	*Check shear yielding of the bracket plate* $\Omega = 1.50$ $R_n / \Omega = \dfrac{(0.6F_y)A_g}{\Omega}$ $= \dfrac{0.6(36 \text{ ksi})(20.0 \text{ in.})(0.375 \text{ in.})}{1.50}$ $= 108 \text{ kips} > 36.0 \text{ kips}$ **o.k.**
Check shear rupture of the bracket plate $A_{nv} = [20.0 \text{ in.} - 6(0.875 \text{ in.})](0.375 \text{ in.})$ $= 5.53 \text{ in.}^2$ $\phi = 0.75$ $\phi R_n = \phi(0.6F_u)A_{nv}$ $= 0.75(0.6)(58 \text{ ksi})(5.53 \text{ in.}^2)$ $= 144 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	*Check shear rupture of the bracket plate* $A_{nv} = [20.0 \text{ in.} - 6(0.875 \text{ in.})](0.375 \text{ in.})$ $= 5.53 \text{ in.}^2$ $\Omega = 2.00$ $R_n / \Omega = \dfrac{0.6F_u A_{nv}}{\Omega}$ $= \dfrac{0.6(58 \text{ ksi})(5.53 \text{ in.}^2)}{2.00}$ $= 96.2 \text{ kips} > 36.0 \text{ kips}$ **o.k.**

Eqn. J4-3

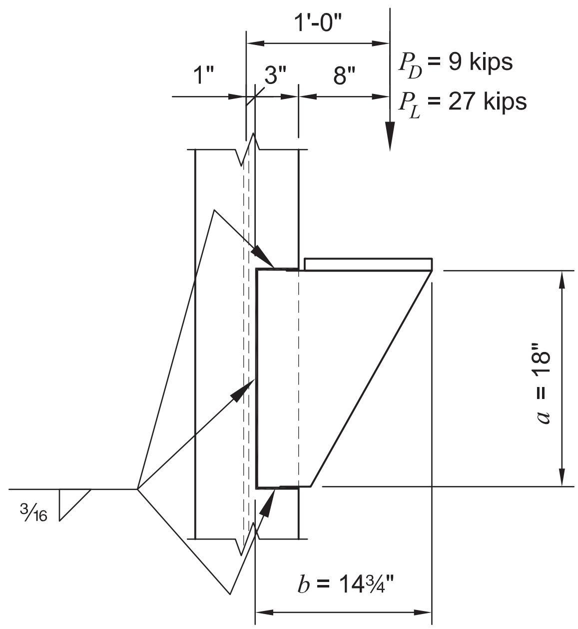
Eqn. J4-4

Example II.A-23 Welded Bracket Plate Design

Given:

Design a welded bracket plate, using 70 ksi electrodes, to support the following loads:

P_D = 9.0 kips
P_L = 27 kips
Assume the column has sufficient available strength for the connection.



Material Properties:

Plate Material ASTM A36 F_y = 36 ksi F_u = 58 ksi Manual Table 2-4

Solution:

Calculate required strength

LRFD	ASD
R_u = 1.2(9.0 kips) + 1.6(27 kips) = 54.0 kips	R_a = 9.0 kips + 27 kips = 36.0 kips

Assume PL½ in.×18 in.

Try a C-shaped weld with kl = 3 in. and l = 18 in. Manual Table 8-8

$k = kl/l$ = 3.00 in / 18.0 in = 0.167

$$xl = \frac{(3.00\text{ in.})^2}{\left[2(3.00\text{ in.})+18.00\text{ in.}\right]} = 0.375\text{ in.}$$

and

$al + xl$ = 11.0 in.

a(18.0 in.) + 0.375 in. = 11.0 in.

a = 0.590

Interpolate using θ = 0°, k = 0.167, and a = 0.590

C = 1.49

C_1 = 1.0 for E70XX electrode. Manual Table 8-3

LRFD	ASD
$D_{req} = \dfrac{P_u}{\phi CC_1 l} = \dfrac{54.0 \text{ kips}}{0.75(1.49)(1.0)(18.0 \text{ in.})}$ $= 2.68 \rightarrow 3$ sixteenths < ½ in. – 1⁄16 in. = 7⁄16 **o.k.** D_{min} = 3 **o.k.** Use a 3⁄16-in. fillet weld. *Check the flexural strength of the bracket plate* Conservatively taking the required moment strength of the plate as equal to the moment strength of the weld group, $M_u = P_u(al)$ = (54.0 kips)(0.590)(18.0 in.) = 573 kip-in.	$D_{req} = \dfrac{P_a\Omega}{CC_1 l} = \dfrac{(36.0 \text{ kips})(2.00)}{(1.49)(1.0)(18.0 \text{ in.})}$ $= 2.68 \rightarrow 3$ sixteenths < ½ in. – 1⁄16 in. = 7⁄16 **o.k.** D_{min} = 3 **o.k.** Use a 3⁄16-in. fillet weld. *Check the flexural strength of the bracket plate* Conservatively taking the required moment strength of the plate as equal to the moment strength of the weld group, $M_a = P_a(al)$ = (36.0 kips)(0.590)(18.0 in.) = 382 kip-in.

Table J2.4

For flexural yielding of the plate,

$$M_n = F_y Z_x = (36 \text{ ksi})\frac{(0.500 \text{ in.})(18.0 \text{ in.})^2}{4} = 1{,}460 \text{ kip-in.}$$

Table 17-27

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(1{,}460 \text{ kip-in.}) = 1{,}310 \text{ kip-in.}$ $1{,}310 \text{ kip-in.} > 573 \text{ kip-in.}$ **o.k.**	$\Omega = 1.67$ $M_n / \Omega = \dfrac{(1{,}460 \text{ kip-in.})}{1.67} = 874 \text{ kip-in.}$ $874 \text{ kip-in.} > 382 \text{ kip-in.}$ **o.k.**

Manual Part 15

For yielding on the free edge of the triangular plate,

$$\frac{b}{a} = \frac{14.8 \text{ in.}}{18.0 \text{ in.}} = 0.822$$

$$z = 1.39 - 2.2\left(\frac{b}{a}\right) + 1.27\left(\frac{b}{a}\right)^2 - 0.25\left(\frac{b}{a}\right)^3 = 1.39 - 2.2(0.822) + 1.27(0.822)^2 - 0.25(0.822)^3$$
$$= 0.301$$

Manual Part 15

$$P_n = F_y zbt = (36 \text{ ksi})(0.301)(14.8 \text{ in.})(0.500 \text{ in.}) = 80.2 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = 0.90(80.2\text{ kips}) = 72.2\text{ kips}$ 72.2 kips > 54.0 kips **o.k.**	$\Omega = 1.67$ $P_n / \Omega = \dfrac{(80.2\text{ kips})}{1.67} = 48.0\text{ kips}$ 48.0 kips > 36.0 kips **o.k.**

Check local buckling of the bracket plate

Manual Part 15

Note:
0.6b = 0.6(14.8 in.) = 8.88 in. ≈ 8.00 in. and lateral movement of the outstanding portion of the bracket plate is prevented, therefore, the following limit is applicable.

Since $0.5 < \dfrac{b}{a} \leq 1.0$

$$t_{min} = b\left(\frac{\sqrt{F_y}}{250}\right) = (14.8\text{ in.})\left(\frac{\sqrt{36\text{ ksi}}}{250}\right) = 0.355\text{ in.} < ½\text{ in.}$$ **o.k.**

Eqn. J4-3

Check shear yielding of the bracket plate

$$R_n = 0.6F_yA_g = 0.6(36\text{ ksi})(18.0\text{ in.})(0.500\text{ in.}) = 194\text{ kips}$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(194\text{ kips}) = 194\text{ kips}$ 194 kips > 54.0 kips **o.k.**	$\Omega = 1.50$ $P_n / \Omega = \dfrac{(194\text{ kips})}{1.50} = 129\text{ kips} > 36.0\text{ kips}$ 129 kips > 36.0 kips **o.k.**

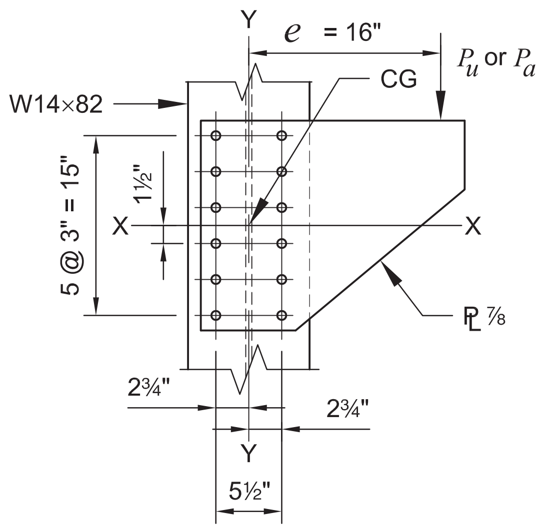
Example II.A-24 Eccentrically-Loaded Bolt Group (IC method)

Given:

Determine the largest eccentric force that can be supported by the available shear strength of the bolts using the instantaneous center of rotation method. Use ⅞-in. diameter ASTM A325-N or F1852-N bolts in standard holes. Assume that bolt shear controls over bearing. Use Manual Table 7-9.

Solution A:

Assume the load is vertical ($\theta = 0°$) as illustrated below



With $\theta = 0°$, with s = 3.00 in., e_x = 16.0 in., and n = 6:

$C = 3.55$ Manual Table 7-9

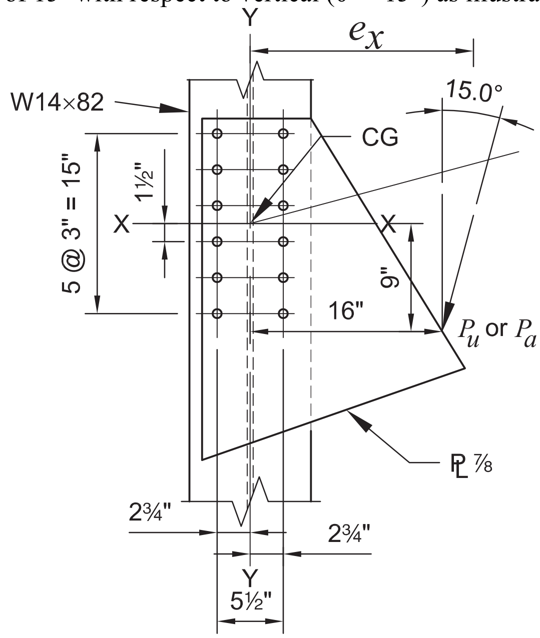
LRFD	ASD
ϕr_n = 21.6 kips $\phi R_n = C\phi r_n$ = 3.55(21.6 kips) = 76.7 kips Thus, P_u must be less than or equal to 76.7 kips.	r_n / Ω = 14.4 kips $R_n / \Omega = C(r_n / \Omega)$ = 3.55(14.4 kips) = 51.1 kips Thus, P_a must be less than or equal to 51.1 kips.

Manual Table 7-1

Note: The eccentricity of the load significantly reduces the shear strength of the bolt group.

Solution B:

Assume the load acts at an angle of 15^0 with respect to vertical ($\theta = 15°$) as illustrated below



e_x = 16.0 in. + (9.00 in.)(tan 15°) = 18.4 in.

With θ = 15°, s = 3.00 in., e_x = 18.4 in., and n = 6:

Manual Table 7-9

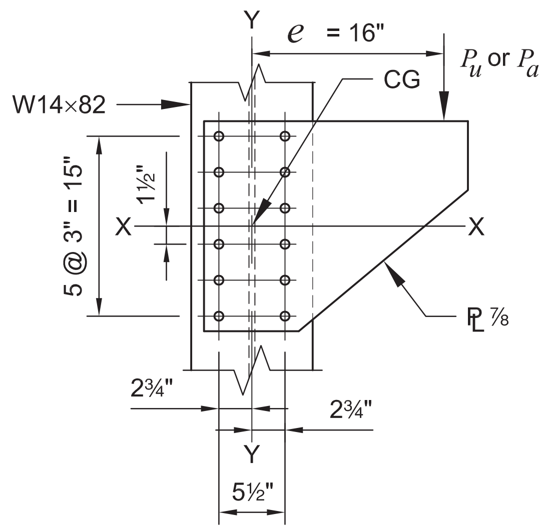
$C = 3.21$

LRFD	ASD
$\phi R_n = C\phi r_n$ = 3.21(21.6 kips) = 69.3 kips Thus, P_u must be less than or equal to 69.3 kips.	$R_n / \Omega = Cr_n / \Omega$ = 3.21(14.4 kips)= 46.2kips Thus, P_a must be less than or equal to 46.2 kips.

Example II.A-25 Eccentrically Loaded Bolt Group (elastic method)

Given:

Determine the largest eccentric force that can be supported by the available shear strength of the bolts using the elastic method for $\theta = 0°$. Compare the result with that of the previous example. Use ⅞ in. diameter ASTM A325-N or F1852-N bolts in standard holes. Assume that bolt shear controls over bearing.



Solution:

LRFD	ASD
Direct shear force per bolt $r_{px} = 0,\ r_{py} = \dfrac{P_u}{n} = \dfrac{P_u}{12}$ *Additional shear force due to eccentricity* Determine the polar moment of inertia $I_x \approx \Sigma y^2 = \begin{bmatrix} 4(7.50\text{ in.})^2 \\ +4(4.50\text{ in.})^2 \\ +4(1.50\text{ in.})^2 \end{bmatrix} = 315\dfrac{\text{in.}^4}{\text{in.}^2}$ $I_y \approx \Sigma x^2 = 12(2.75\text{ in.})^2 = 90.8\ \dfrac{\text{in.}^4}{\text{in.}^2}$ $I_p \approx \text{I}_x + I_y = 315\dfrac{\text{in.}^4}{\text{in.}^2} + 90.8\dfrac{\text{in.}^4}{\text{in.}^2} = 406\dfrac{\text{in.}^4}{\text{in.}^2}$	*Direct shear force per bolt* $r_{px} = 0,\ r_{py} = \dfrac{P_a}{n} = \dfrac{P_a}{12}$ *Additional shear force due to eccentricity* Determine the polar moment of inertia $I_x \approx \Sigma y^2 = \begin{bmatrix} 4(7.50\text{ in.})^2 \\ +4(4.50\text{ in.})^2 \\ +4(1.50\text{ in.})^2 \end{bmatrix} = 315\dfrac{\text{in.}^4}{\text{in.}^2}$ $I_y \approx \Sigma x^2 = 12(2.75\text{ in.})^2 = 90.8\ \dfrac{\text{in.}^4}{\text{in.}^2}$ $I_p \approx \text{I}_x + I_y = 315\dfrac{\text{in.}^4}{\text{in.}^2} + 90.8\dfrac{\text{in.}^4}{\text{in.}^2} = 406\dfrac{\text{in.}^4}{\text{in.}^2}$

Manual Part 7

LRFD	ASD
$r_{mx}=\dfrac{P_u e c_y}{I_p}=\dfrac{P_u\left(16.0\text{ in.}\right)\left(7.50\text{ in.}\right)}{406\text{ in.}^4/\text{in.}^2}$ $=0.296\ P_u$	$r_{mx}=\dfrac{P_a e c_y}{I_p}=\dfrac{P_a\left(16.0\text{ in.}\right)\left(7.50\text{-in.}\right)}{406\text{ in.}^4/\text{in.}^2}$ $=0.296\ \text{P}_a$
$r_{my}=\dfrac{P_u e c_x}{I_p}=\dfrac{P_u\left(16.0\text{ in.}\right)\left(\dfrac{5.50\text{ in.}}{2}\right)}{406\text{ in.}^4/\text{in.}^2}$ $=0.108\ P_u$	$r_{my}=\dfrac{P_a e c_x}{I_p}=\dfrac{P_a\left(16.0\text{ in.}\right)\left(\dfrac{5.50\text{-in.}}{2}\right)}{406\text{ in.}^4/\text{in.}^2}$ $=0.108\ P_a$
Resultant shear force	*Resultant shear force*
$r_u=\sqrt{\left(r_{px}+r_{mx}\right)^2+\left(r_{py}+r_{my}\right)^2}$ $=\sqrt{\left(0+0.296P_u\right)^2+\left(\dfrac{P_u}{12}+0.108P_u\right)^2}$ $=0.352P_u$	$r_a=\sqrt{\left(r_{px}+r_{mx}\right)^2+\left(r_{py}+r_{my}\right)^2}$ $=\sqrt{\left(0+0.296P_a\right)^2+\left(\dfrac{P_a}{12}+0.108P_a\right)^2}$ $=0.352P_a$
Since r_u must be less than or equal to the available strength,	Since r_a must be less than or equal to the available strength,
$P_u\le\dfrac{\phi r_n}{0.352}=\dfrac{21.6\text{ kips}}{0.352}=61.4\text{ kips}$	$P_a\le\dfrac{r_n/\Omega}{0.352}=\dfrac{14.4\text{ kips}}{0.352}=40.9\text{ kips}$

Note: the elastic method, shown here, is more conservative than the instantaneous center of rotation method, shown in **Example II.A-24a.**

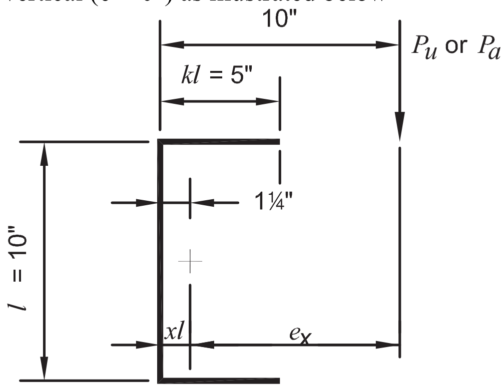
Example II.A-26 Eccentrically-Loaded Weld Group (IC method)

Given:

Determine the largest eccentric force that can be supported by the available shear strength of the weld group, using the instantaneous center of rotation method. Use a ⅜-in. fillet weld and 70 ksi electrode. Use Manual Table 8-8.

Solution A:

Assume that the load is vertical (θ = 0°) as illustrated below



l = 10.0 in., kl = 5.00 in., therefore $k = \dfrac{kl}{l} = \dfrac{5.00\text{ in.}}{10.0\text{ in.}} = 0.500$

$$xl = \frac{(5.00\text{ in.})^2}{2(5.00\text{ in.})+10.0\text{ in.}} = 1.25\text{ in.}$$

$$xl + al = 10.0\text{ in.}$$
$$1.25 + a(10.0\text{ in.}) = 10.0\text{ in.}$$
$$a = 0.875$$

By interpolation, with θ = 0°, a = 0.875 and k = 0.500

Manual Table 8-8

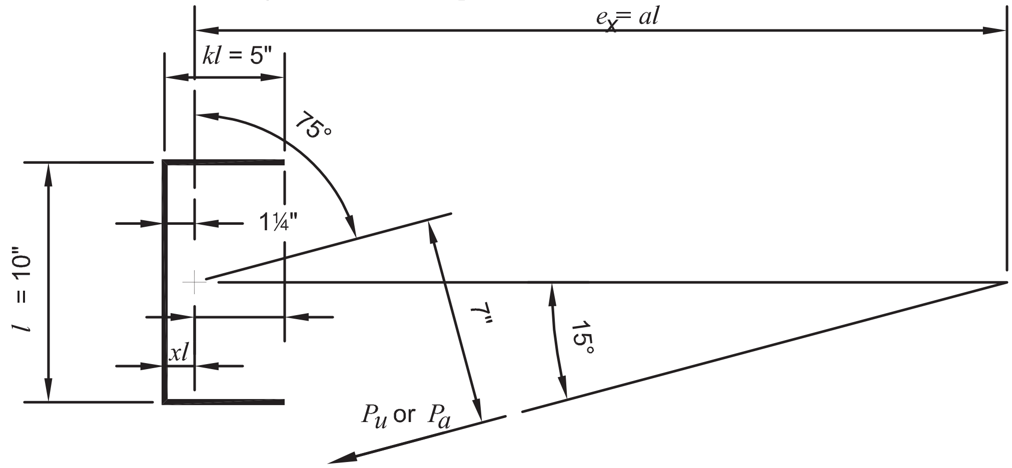
$C = 1.88$

LRFD	ASD
$\phi R_n = \phi C C_1 D l$ $= 0.75(1.88)(1.0)(6\text{ sixteenths})(10.0\text{ in.})$ $= 84.6\text{ kips}$ Thus, P_u must be less than or equal to 84.6 kips.	$R_n / \Omega = \dfrac{C C_1 D l}{\Omega}$ $= \dfrac{1.88(1.0)(6\text{ sixteenths})(10.0\text{ in.})}{2.00}$ $= 56.4\text{ kips}$ Thus, P_a must be less than or equal to 56.4 kips.

Note: The eccentricity of the load significantly reduces the shear strength of this weld group as compared to the concentrically loaded case.

Solution B:

Assume that the load acts at an angle of 75° with respect to vertical (θ = 75°) as illustrated below



As determined in Solution A,

k = 0.500 and xl = 1.25 in., Manual Table 8-8

$$e_x = al = 7.00\text{ in.}/\sin 15° = 27.0\text{ in.}$$

$$a = \frac{e_x}{l} = \frac{27.0\text{ in.}}{10.0\text{ in.}} = 2.70$$

By interpolation, with θ = 75°, a = 2.70 and k = 0.500

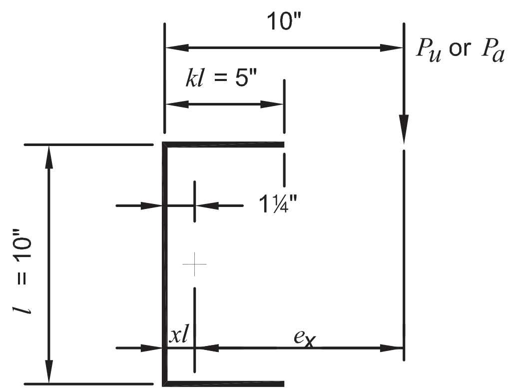
$$C = 1.98$$

LRFD	ASD
$\phi R_n = \phi CC_1Dl$ $= 0.75(1.98)(1.0)(6\text{ sixteenths})(10.0\text{ in.})$ $= 89.1\text{ kips}$ Thus, P_u must be less than or equal to 89.1 kips.	$R_n / \Omega = \dfrac{CC_1Dl}{\Omega}$ $= \dfrac{1.98(1.0)(6\text{ sixteenths})(10.0\text{ in.})}{2.00}$ $= 59.4\text{ kips}$ Thus, P_a must be less than or equal to 59.4 kips.

Example II.A-27 Eccentrically-Loaded Weld Group (elastic method)

Given:

Determine the largest eccentric force that can be supported by the available shear strength of the welds in the connection, using the elastic method. Compare the result with that of the previous example. Use ⅜ in. fillet welds made with E70 electrodes.



Solution:

Calculate the required strength

LRFD	ASD
Direct shear force per inch of weld $r_{px}=0,\ r_{py}=\dfrac{P_u}{l}=\dfrac{P_u}{20.0\text{ in.}}=0.0500\dfrac{P_u}{\text{in.}}$	*Direct shear force per inch of weld* $r_{px}=0,\ r_{py}=\dfrac{P_a}{l}=\dfrac{P_a}{20.0\text{ in.}}=0.0500\dfrac{P_a}{\text{in.}}$

Manual Part 8

Additional shear force due to eccentricity
Determine the polar moment of inertia

$$I_x=\frac{l^3}{12}+ly^2$$
$$=\frac{(10.0\text{ in.})^3}{12}+2(5.00\text{ in.})(5.00\text{ in.})^2$$
$$=333\text{in.}^4/\text{in.}$$

$$I_y=\frac{l^3}{12}+lx^2$$
$$=\frac{2(5.00\text{ in.})^3}{12}+2(5.00\text{ in.})(2.50\text{ in.}-1.25\text{ in.})^2+(10.0\text{ in.})(1.25\text{ in.})^2$$
$$=52.1\text{in.}^4/\text{in.}$$

$$I_p=I_x+I_y$$
$$=333\text{ in.}^4/\text{in.}+52.1\text{ in.}^4/\text{in.}$$
$$=385\text{ in.}^4/\text{in.}$$

LRFD	ASD
$r_{mx} = \dfrac{P_u e c_y}{I_p} = \dfrac{P_u(8.75\text{ in.})(5.00\text{ in.})}{385\text{in.}^4/\text{in.}}$ $= \dfrac{0.114\ P_u}{\text{in.}}$ $r_{my} = \dfrac{P_u e c_x}{I_p} = \dfrac{P_u(8.75\text{ in.})(3.75\text{ in.})}{385\text{in.}^4/\text{in.}}$ $= \dfrac{0.0852\ P_u}{\text{in.}}$ *Resultant shear force* $r_u = \sqrt{(r_{px} + r_{mx})^2 + (r_{py} + r_{my})^2}$ $= \sqrt{\left(\dfrac{0.114P_u}{\text{in.}}\right)^2 + \left(\dfrac{0.0500P_u}{\text{in.}} + \dfrac{0.0852P_u}{\text{in.}}\right)^2}$ $= \dfrac{0.177P_u}{\text{in.}}$ Since r_u must be less than or equal to the available strength, $r_u = 0.177P_u \leq \phi r_n$ $P_u \leq \dfrac{\phi r_n}{0.177}$ $\leq \left(\dfrac{1.392\text{ kips/in.}}{\text{sixteenth}}\right)(6\text{ sixteenths})\left(\dfrac{\text{in.}}{0.177}\right)$ $\leq 47.2\text{ kips}$	$r_{mx} = \dfrac{P_a e c_y}{I_p} = \dfrac{P_a(8.75\text{ in.})(5.00\text{ in.})}{385\text{ in.}^4/\text{in.}}$ $= \dfrac{0.114\ P_a}{\text{in.}}$ $r_{my} = \dfrac{P_a e c_x}{I_p} = \dfrac{P_a(8.75\text{ in.})(3.75\text{ in.})}{385\text{ in.}^4/\text{in.}}$ $= \dfrac{0.0852\ P_a}{\text{in.}}$ *Resultant shear force* $r_a = \sqrt{(r_{px} + r_{mx})^2 + (r_{py} + r_{my})^2}$ $= \sqrt{\left(\dfrac{0.114P_a}{\text{in.}}\right)^2 + \left(\dfrac{0.0500P_a}{\text{in.}} + 0.0852P_a\right)^2}$ $= \dfrac{0.177P_a}{\text{in.}}$ Since r_a must be less than or equal to the available strength, $r_a = 0.177P_a \leq r_n/\Omega$ $P_a \leq \dfrac{r_n/\Omega}{0.177}$ $\leq \left(\dfrac{0.928\text{ kip/in.}}{\text{sixteeth}}\right)(6\text{ sixteenths})\left(\dfrac{\text{in.}}{0.177}\right)$ $\leq 31.5\text{ kips}$

Manual Part 8

Note: The strength of the weld group predicted by the elastic method, as shown here, is significantly less than the predicted by the instantaneous center of rotation method in **Example II.A-26a**.

Example II.A-28 All-Bolted Single-Angle Connection (beam-to-girder web)

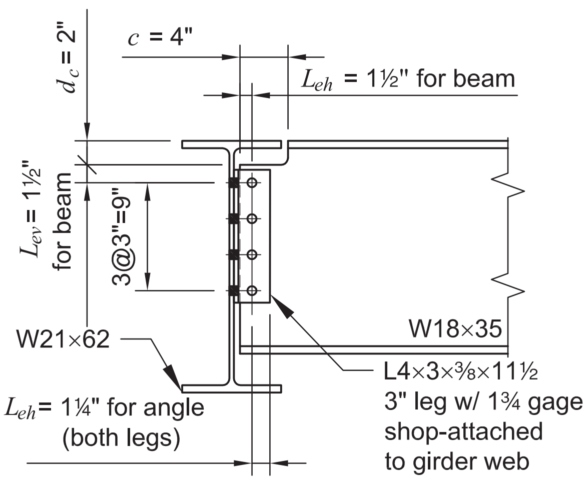
Given:

Design an all-bolted single-angle connection (case I in Table 10-10) between a W18×35 beam and a W21×62 girder-web, to support the following beam end reactions:

R_D = 6.5 kips
R_L = 20 kips

Top flange coped 2 in. deep by 4 in. long, L_{ev} = 1½ in., L_{eh} = 1½ in. (assumed to be 1¼ in. for calculation purposes to account for possible underrun in beam length),

Use ¾-in. diameter A325-N or F1852-N bolts in standard holes.



Material Properties:

Beam W18×35	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Girder W21×62	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angle	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam W18×35	t_w = 0.300 in.	d = 17.7 in.	t_f = 0.425 in.	Manual Table 1-1
Cope	c = 4.00 in.	d_c = 2.00 in.	e = 4.50 in.	Manual Figure 9-2
	h_0 = 15.7 in.			
Girder W21×62	t_w = 0.400 in.			

Solution:

LRFD	ASD	
Calculate the required strength R_u = 1.2(6.5 kips) + 1.6(20 kips) = 39.8 kips *Design the bolts and angle* Check eccentricity of connection For the 4 in. angle leg attached to the supported beam (W18x35) $e = 2.75 \leq 3.00$ in. Therefore, eccentricity does not need to be considered for this leg. For the 3 in. angle leg attached to the supporting girder (W21x62) $e = 1.75 \text{ in.} + \dfrac{0.300 \text{ in.}}{2} = 1.90 \text{ in.} \leq 2.50 \text{ in.}$ Therefore, Table 10-10 may conservatively be used for bolt shear. $\phi r_n = 15.9 \text{ kips}$ For bearing on the angle with a ⅜ in. thick angle, $\phi r_n = \left(\dfrac{44.0 \text{ kips}}{\text{in.}}\right)(0.375 \text{ in.}) = 16.5 \text{ kips}$ Bolt shear is more critical than bolt bearing in this example; thus, $\phi_v r_n$ = 15.9 kips. $C_{min} = \dfrac{R_u}{\phi r_n} = \dfrac{39.8 \text{ kips}}{15.9 \text{ kips/bolt}} = 2.50$ Try a four-bolt connection. $C = 3.07 > 2.50$ **o.k.** The 3-in. leg will be shop bolted to the girder web and the 4-in. leg will be field bolted to the beam web.	*Calculate the required strength* R_a = 6.5 kips + 20 kips = 26.5 kips *Design the bolts and angle* Check eccentricity of connection For the 4 in. angle leg attached to the supported beam (W18x35) $e = 2.75 \leq 3.00$ in. Therefore, eccentricity does not need to be considered for this leg. For the 3 in. angle leg attached to the supporting girder (W21x62) $e = 1.75 \text{ in.} + \dfrac{0.300 \text{ in.}}{2} = 1.90 \text{ in.} \leq 2.50 \text{ in.}$ Therefore, Table 10-10 may conservatively be used for bolt shear. $\dfrac{r_n}{\Omega} = 10.6 \text{ kips}$ For bearing on the angle with a ⅜ in. thick angle, $\dfrac{r_n}{\Omega} = \left(\dfrac{29.4 \text{ kips}}{\text{in.}}\right)(0.375 \text{ in.}) = 11.0 \text{ kips}$ Bolt shear is more critical than bolt bearing in this example; thus, r_n / Ω_v = 10.6 kips. $C_{min} = \dfrac{R_a}{r_n / \Omega} = \dfrac{26.5 \text{ kips}}{10.6 \text{ kips/bolt}} = 2.50$ Try a four-bolt connection. $C = 3.07 > 2.50$ **o.k.** The 3-in. leg will be shop bolted to the girder web and the 4-in. leg will be field bolted to the beam web.	Manual Table 7-1 Manual Table 7-6
Check shear yielding of the angle $\phi R_n = \phi(0.6F_yA_g)$ = 1.0(0.6)(36 ksi)(11.5 in.)(0.375 in.) = 93.2 kips > 39.8 kips **o.k.**	*Check shear yielding of the angle* $R_n / \Omega = 0.6F_yA_g / \Omega$ = 0.6(36 ksi)(11.5 in.)(0.375 in.)/1.50 = 62.1 kips > 26.5 kips **o.k.**	Section J4.2 Eqn J4-3

LRFD	ASD	
Check shear rupture of the angle $\phi R_n = \phi(0.6F_uA_{nv})$ $= 0.75\left\{0.6(58\text{ ksi})\left[\begin{array}{l}(11.5\text{ in.})(0.375\text{ in.}) -\\ 4(0.875\text{ in.})(0.375\text{ in.})\end{array}\right]\right\}$ $= 78.3$ kips > 39.8 kips **o.k.**	*Check shear rupture of the angle* $R_n / \Omega = (0.6F_uA_{nv})/ \Omega$ $= \dfrac{0.6(58\text{ ksi})\left[\begin{array}{l}(11.5\text{ in.})(0.375\text{ in.}) -\\ 4(0.875\text{ in.})(0.375\text{ in.})\end{array}\right]}{2.00}$ $= 52.2$ kips > 26.5 kips **o.k.**	Section J4.2 Eqn J4-4
Check block shear rupture of the angle From the tables, with n = 4, L_{ev} = 1¼ in. L_{eh} = 1¼ in., $\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$ Tension rupture component $\phi F_u A_{nt}$ = 35.3 kips/in.(0.375 in.) Shear yielding component $\phi 0.6 F_y A_{gv}$ = 166 kips/in.(0.375 in.) Shear rupture component $\phi 0.6 F_u A_{nv}$ = 188 kips/in.(0.375 in.) Tension stress is uniform, therefore U_{bs} = 1.0 ϕR_n = (166 kips/in. + 35.3 kips/in.)(0.375 in.) = 75.5 kips > 39.8 kips **o.k.**	*Check block shear rupture of the angle* From the tables, with n = 4, L_{ev} = 1¼ in. L_{eh} = 1¼ in., $\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$ Tension rupture component $F_u A_{nt} / \Omega$ = 23.6 kips/in.(0.375 in.) Shear yielding component $0.6 F_y A_{gv} / \Omega$ = 111 kips/in.(0.375 in.) Shear rupture component $0.6 F_u A_{nv} / \Omega$ = 125 kips/in.(0.375 in.) Tension stress is uniform, therefore U_{bs} = 1.0 R_n/Ω = (111 kips/in.+23.6 kips/in.)(0.375 in.) = 50.5 kips > 26.5 kips **o.k.**	Eqn. J4-5 Manual Table 9-3a Manual Table 9-3b Manual Table 9-3c Commentary Section J4.3
Check flexure of the support-leg of the angle The required strength is $M_u = R_u e$ $= (39.8\text{ kips})\left(1.75\text{ in.} + \dfrac{0.300\text{ in.}}{2}\right)$ $= 75.6$ kip-in. For flexural yielding $\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(36\text{ ksi})\left[\dfrac{(0.375\text{ in.})(11.5\text{ in.})^2}{4}\right]$ $= 402$ kip-in. > 75.6 kip-in. **o.k.**	*Check flexure of the support-leg of the angle* The required strength is $M_a = R_a e$ $= (26.5\text{ kips})\left(1.75\text{ in.} + \dfrac{0.300\text{ in.}}{2}\right)$ $= 50.4$ kip-in. For flexural yielding $\Omega = 1.67$ $M_n/\Omega = F_y Z_x/\Omega$ $= \dfrac{(36\text{ ksi})}{1.67}\left[\dfrac{(0.375\text{ in.})(11.5\text{ in.})^2}{4}\right]$ $= 267$ kip-in. > 50.4 kip-in. **o.k.**	Manual Part 15

For flexural rupture,

$$Z_{net} = 0.375\left[\frac{(11.5\text{ in.})^2}{4} - 2(0.875\text{ in.})(4.50\text{ in.}) - 2(0.875\text{ in.})(1.50\text{ in.})\right]$$
$$= 8.46\text{ in.}^3$$

LRFD	ASD
$\phi = 0.75$ $\phi M_n = \phi F_u Z_{net} = 0.75(58\text{ ksi})(8.46\text{ in.}^3)$ $= 368\text{ kip-in.} > 75.6\text{ kip-in.}$ **o.k.**	$\Omega = 2.00$ $M_n / \Omega = \dfrac{F_u Z_{net}}{\Omega} = \dfrac{(58\text{ ksi})(8.46\text{ in.}^3)}{2.00}$ $= 245\text{ kip-in.} > 50.4\text{ kip-in.}$ **o.k.**
Check beam web for bolt bearing and block shear rupture. $n = 4$, $L_{ev} = 1\frac{1}{2}$ in., and $L_{eh} = 1\frac{1}{2}$ in. (L_{eh} assumed to be $1\frac{1}{4}$ in. for calculation purposes to provide for possible underrun in beam length), $\phi R_n = (257\text{ kips/in.})(0.300\text{ in.})$ $= 77.1\text{ kips} > 39.8\text{ kips}$ **o.k.**	*Check beam web for bolt bearing and block shear rupture.* $n = 4$, $L_{ev} = 1\frac{1}{2}$ in., and $L_{eh} = 1\frac{1}{2}$ in. (L_{eh} assumed to be $1\frac{1}{4}$ in. for calculation purposes to provide for possible underrun in beam length), $R_n / \Omega = (171\text{ kips/in.})(0.300\text{ in.})$ $= 51.3\text{ kips} > 26.5\text{ kips}$ **o.k.**

Manual Table 10-1

Note: For coped beam sections, the limit states of flexural rupture, yielding and local buckling should be checked independently per Part 9. The supported beam web should also be checked for shear yielding and shear rupture per Section J4.2. However, for the shallow cope in this example, these limit states do not govern. For a illustration of these checks, see Example II.A-4.

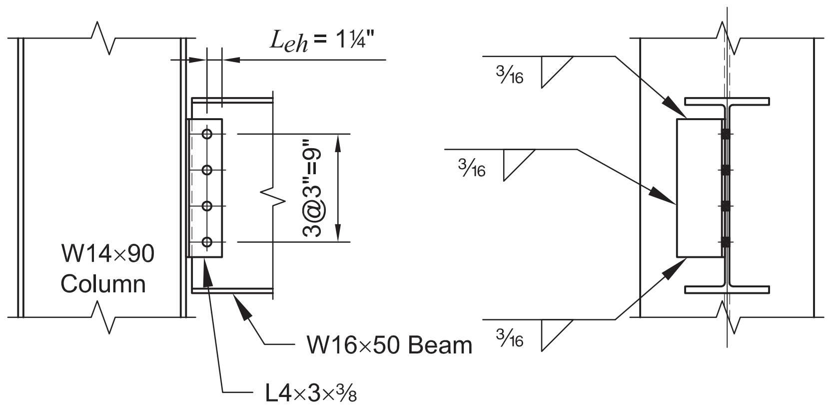
Example II.A-29 Bolted/Welded Single-Angle Connection (beam-to-column flange)

Given:

Design a single-angle connection between a W16×50 beam and a W14×90 column flange to support the following beam end reactions:

R_D = 9.0 kips
R_L = 27 kips

Use ¾ in. diameter ASTM A325-N or F1852-N bolts to connect the supported beam to the single angle. Use 70 ksi electrode welds to connect the single angle to the column flange.



Material Properties:

Manual Tables 2-3

W16×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
Angle	ASTM A36	F_y = 36 ksi	F_u = 58 ksi

Geometric Properties:

Manual Table 1-1

Beam	W16×50	t_w = 0.380 in.	d = 16.3in.	t_f = 0.630 in.
Column	W14×90	t_f = 0.710		

Solution:

LRFD	ASD
Calculate required strength R_u = 1.2 (9.0 kips) + 1.6 (27 kips) = 54.0 kips *Design single angle, bolts, and welds* Check eccentricity of the connection For the 4 in. angle leg attached to the support beam (W16x50) e = 2.75 in. ≤ 3.00 in. Therefore, eccentricity does not need to be	*Calculate required strength* R_a = 9.0 kips + 27 kips = 36.0 kips *Design single angle, bolts, and welds* Check eccentricity of the connection For the 4 in. angle leg attached to the support beam (W16x50) e = 2.75 in. ≤ 3.00 in. Therefore, eccentricity does not need to be

LRFD	ASD	
considered for this leg.	considered for this leg.	
For the 3 in. angle leg attached to the supporting column flange (W14x90). Since the half-web dimension of the W16x50 supported beam is less than ¼ in., Table 10-11 may conservatively be used.	For the 3 in. angle leg attached to the supporting column flange (W14x90). Since the half-web dimension of the W16x50 supported beam is less than ¼ in., Table 10-11 may conservatively be used.	
Try a four bolt single angle (L4×3×3/8).	Try a four bolt single angle (L4×3×3/8).	
ϕR_n = 63.6 kips > 54.0 kips **o.k.**	R_n / Ω = 42.4 kips > 36.0 kips **o.k.**	Manual Table 10-11
Also with a 3/16-in fillet weld size	Also with a 3/16–in. fillet weld size	
ϕR_n = 56.6 kips > 54.0 kips **o.k.**	R_n / Ω = 37.7 kips > 36.0 kips **o.k.**	Manual Table 10-11
Check support	*Check support*	
Minimum support thickness for the 3/16–in. welds is	Minimum support thickness for the 3/16–in. welds is	
$t_{min} = \dfrac{3.090}{F_u} = \dfrac{3.09(3\text{ sixteenths})}{65\text{ ksi}} = 0.143\text{ in.}$	$t_{min} = \dfrac{3.090}{F_u} = \dfrac{3.09(3\text{ sixteenths})}{65\text{ ksi}} = 0.143\text{ in.}$	Manual Part 9
t_f = 0.710 in > 0.143 in **o.k.**	t_f = 0.710 in > 0.143 in **o.k.**	
Note: The minimum thickness values listed in Table 10-11 are for conditions with angles on both sides of the web.	Note: The minimum thickness values listed in Table 10-11 are for conditions with angles on both sides of the web.	
Use four-bolt single angle L4×3×3/8. The 3-in. leg will be shop welded to the column flange and the 4 in. leg will be field bolted to the beam web.	Use four-bolt single angle L4×3×3/8. The 3-in. leg will be shop welded to the column flange and the 4 in. leg will be field bolted to the beam web.	
Check supported beam web	*Check supported beam web*	
The bearing strength of the beam web is	The bearing strength of the beam web is	
s = 3.00 in., ¾ in. diameter bolts, standard holes	s = 3.00 in., ¾ in. diameter bolts, standard holes	
$\phi R_n = \phi r_n t_w n$ = (87.8 kips/in.)(0.380 in.)(4 bolts) = 133 kips > 54.0 kips **o.k.**	$R_n / \Omega = \dfrac{r_n t_w n}{\Omega}$ = (58.5 kips/in.)(0.380 in.)(4 bolts) = 88.9 kips > 36.0 kips **o.k.**	Manual Table 7-5

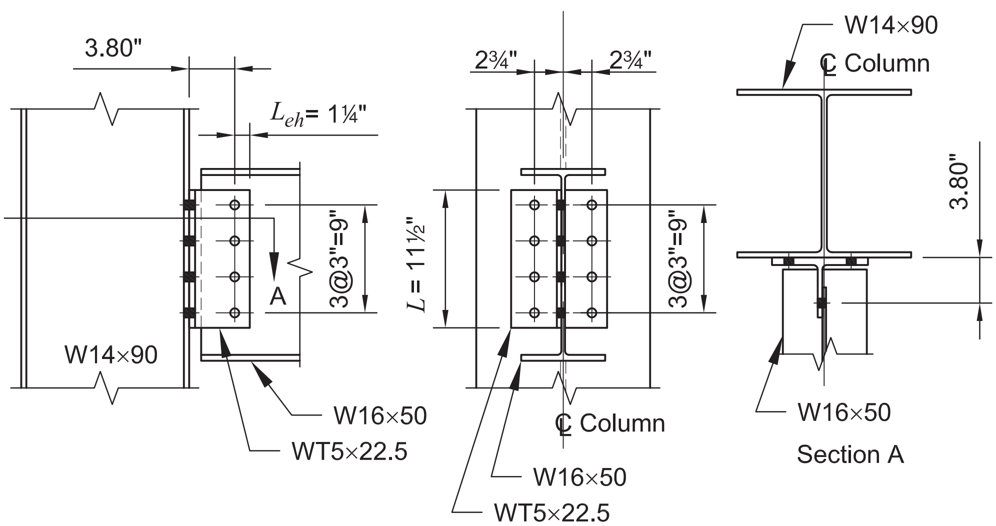
Example II.A-30 All-Bolted Tee Connection (beam-to-column flange)

Given:

Design an all-bolted tee connection between a W16×50 beam and a W14×90 column flange to support the following beam end reactions:

R_D = 9.0 kips
R_L = 27 kips

Use ¾ in. diameter ASTM A325-N or F1852-N bolts in standard holes. Try a WT5×22.5 with a four-bolt connection.



Material Properties:

Manual Table 2-3

W16×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
WT5×22.5	ASTM A992	F_y = 50 ksi	F_u = 65 ksi

Geometric Properties:

Manual Tables 1-1 and 1-8

Beam	W16×50	t_w = 0.380 in.	d = 16.3 in.	t_f = 0.630 in.
Column	W14×90	t_f = 0.710 in.		
Tee	WT5×22.5	d = 5.05 in.	b_f = 8.02 in.	t_f = 0.620 in.
		t_s = 0.350 in.	k_1 = 13⁄16 in.	k_{des} = 1.12 in.
			(see W10×45 Manual Table 1-1)	

Solution:

LRFD	ASD
R_u = 1.2 (9.0 kips) + 1.6 (27 kips) = 54.0 kips	R_a = 9.0 kips + 27 kips = 36.0 kips

Check limitation on tee stem thickness

Manual Part 9

See Rotational Ductility discussion at the beginning of the Manual Part 9

$$t_{s\,max} = \frac{d_b}{2} + \tfrac{1}{16}\text{ in.} = \frac{0.750\text{ in.}}{2} + \tfrac{1}{16}\text{ in.} = 0.438\text{ in.} > 0.350\text{ in.} \qquad \textbf{o.k.}$$

Check limitation on bolt diameter for bolts through tee flange

Note: The bolts are not located symmetrically with respect to the centerline of the tee.

b = flexible width in connection element
= 2.39 in. – 0.813 in. = 1.58 in.

$$d_{b\,min} = 0.163 t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2}+2\right)} \le 0.69\sqrt{t_s}$$

$$= 0.163(0.620\text{ in.})\sqrt{\frac{50.0\text{ ksi}}{1.58\text{ in.}}\left[\frac{(1.58\text{ in.})^2}{(11.5\text{ in})^2}+2\right]} \le 0.69\sqrt{0.350\text{ in.}}$$

$$= 0.808\text{ in.} \le 0.408\text{ in.}$$

Use $d_{b\,min}$ = 0.408 in.

Since d_b = 0.75 in. > $d_{b\,min}$ = 0.408 in. **o.k.**

Since the connection is rigid at the support, the bolts through tee stem must be designed for shear, but do not need to be designed for eccentric moment.

Check bolt group through beam web for shear and bearing

LRFD	ASD
Since bolt shear is more critical than bolt bearing in this example, ϕr_n = 15.9 kips, Thus, $\phi R_n = n\phi r_n$ = (4 bolts)(15.9 kips) = 63.6 kips > 54.0 kips **o.k.** *Check flexure in the stem of the tee at the junction of the stem and the fillet* $M_u = P_u e$ = (54.0 kips)(3.80 in. – 1.12 in.) = 145 kip-in. *For flexural yielding* ϕ = 0.90	Since bolt shear is more critical than bolt bearing in this example, r_n / Ω = 10.6 kips, Thus, $R_n / \Omega = nr_n/\Omega$ = (4 bolts)(10.6 kips) = 42.4 kips > 36.0 kips **o.k.** *Check flexure in the stem of the tee at the junction of the stem and the fillet* M_a = (36.0 kips)(3.80 in. – 1.12 in.) = 96.5 kip-in. *For flexural yielding* Ω = 1.67

Manual Table 7-1

Manual Part 5

LRFD	ASD	
$\phi M_n = \phi F_y Z_x$ $= 0.90(50\text{ ksi})\left(\dfrac{(0.350\text{ in.})(11.5\text{ in.})^2}{4}\right)$ $= 521\text{ kip-in.} > 145\text{ kip-in.}$ **o.k.**	$\dfrac{M_u}{\Omega} = \dfrac{F_y Z_x}{\Omega}$ $= \dfrac{(50\text{ ksi})\left(\dfrac{(0.350\text{ in.})(11.5\text{ in.})^2}{4}\right)}{1.67}$ $= 346\text{ kip-in.} > 96.5\text{ kip-in.}$ **o.k.**	
Check shear yielding of the tee stem $\phi = 1.00$ $\phi R_n = \phi 0.6 F_y A_g$ $= 1.00\left[0.6(50\text{ ksi})(11.5\text{ in.})(0.350\text{ in.})\right]$ $= 121\text{ kips} > 54.0\text{ kips}$ **o.k.**	*Check shear yielding of the tee stem* $\Omega = 1.50$ $R_n / \Omega = 0.6 F_y A_g / \Omega$ $= \dfrac{0.6(50\text{ ksi})(11.5\text{ in.})(0.350\text{ in.})}{1.50}$ $= 80.5\text{ kips} > 36.0\text{ kips}$ **o.k.**	Eqn. J4-3
Check shear rupture of the tee stem $A_{nv} = \left[11.5\text{ in.} - 4(0.875\text{ in.})\right]0.350\text{ in.}$ $= 2.80\text{ in.}^2$ $\phi = 0.75$ $\phi R_n = \phi 0.6 F_u A_{nv}$ $= 0.75(0.6)(65\text{ ksi})(2.80\text{ in.}^2)$ $= 81.9\text{ kips} > 54.0\text{ kips}$ **o.k.**	*Check shear rupture of the tee stem* $A_{nv} = \left[11.5\text{ in.} - 4(0.875\text{ in.})\right]0.350\text{ in.}$ $= 2.80\text{ in.}^2$ $\Omega = 2.00$ $R_n / \Omega = 0.6 F_u A_{nv} / \Omega$ $= (0.6)(65\text{ ksi})(2.80\text{ in.}^2)/2.00$ $= 54.6\text{ kips} > 36.0\text{ kips}$ **o.k.**	Eqn. J4-4
Check block shear rupture of the tee stem $L_{eh} = L_{ev} = 1\frac{1}{4}$ in., Thus, $\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	*Check block shear rupture of the tee stem* $L_{eh} = L_{ev} = 1\frac{1}{4}$ in., Thus, $\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn. J4-5
Tension rupture component $\phi F_u A_{nt} = 39.6\text{ kips/in.}\ (0.350\text{ in.})$	Tension rupture component $F_u A_{nt} / \Omega = 26.4\text{ kips/in.}\ (0.350\text{ in.})$	Manual Table 9-3a
Shear yielding component $\phi 0.6 F_y A_{gv} = 231\text{ kips/in.}\ (0.350\text{ in.})$	Shear yielding component $0.6 F_y A_{gv} / \Omega = 154\text{ kips/in.}\ (0.350\text{ in.})$	Manual Table 9-3b
Shear rupture component $\phi 0.6 F_u A_{nv} = 210\text{ kips/in.}\ (0.350\text{ in.})$	Shear rupture component $0.6 F_u A_{nv} / \Omega = 140\text{ kips/in.}\ (0.350\text{ in.})$	
$\phi R_n = (210\text{ kips/in.} + 39.6\text{ kips/in.})(0.350\text{in.})$ $= 87.4\text{ kips} > 54.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (140\text{ kips/in.} + 26.4\text{ kips/in.})(0.350\text{ in.})$ $= 58.2\text{ kips} > 36.0\text{ kips}$ **o.k.**	Manual Table 9-3c

Since the connection is rigid at the support, the bolts attaching the tee flange to the support must be designed for the shear and the eccentric moment. — Manual Part 10

LRFD	ASD
Check bolt group through support for shear and bearing combined with tension due to eccentricity The following calculation follows the Case II approach in the Section "Eccentricity Normal to the Plane of the Faying Surface" in the introduction of Part 7 of the Manual Calculate tensile force per bolt r_{ut}. $r_{ut} = \dfrac{R_u e}{n' d_m}$ $= \dfrac{54.0 \text{ kips } (5.05 \text{ in.} - 1.25 \text{ in.})}{4 \text{ bolts}(6.00 \text{ in.})}$ = 8.55 kips/bolts *Check design strength of bolts for tension-shear interaction* When threads are not excluded from the shear planes of ASTM A325 bolts $r_{uv} = \dfrac{54.0 \text{ kips}}{8 \text{ bolts}}$ = 6.75 kips/bolt < 15.9 kips/bolt **o.k.** $f_v = \dfrac{6.75 \text{ kips/bolt}}{0.442 \text{ in.}^2} = 15.3 \text{ ksi}$ F_{nt} = 90 ksi, F_{nv} = 48 ksi $\phi = 0.75$ $F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \left(\dfrac{90 \text{ ksi}}{(0.75)(48 \text{ ksi})}\right)(15.3 \text{ ksi})$ = 78.8 ksi ≤ 90 ksi $\phi R_n = \phi F'_{nt} A_b$ = 0.75(78.8 ksi)(0.442 in.^{2}) = 26.1 kips/bolt > 8.55 kips/bolt **o.k.**	*Check bolt group through support for shear and bearing combined with tension due to eccentricity* The following calculation follows the Case II approach in the Section "Eccentricity Normal to the Plane of the Faying Surface" in the introduction of Part 7 of the Manual Calculate tensile force per bolt r_{at}. $r_{at} = \dfrac{R_a e}{n' d_m}$ $= \dfrac{36.0 \text{ kips } (5.05 \text{ in.} - 1.25 \text{ in.})}{4 \text{ bolts}(6.00 \text{ in.})}$ = 5.70 kips/bolts *Check allowable strength of bolts for tension-shear interaction* When threads are not excluded from the shear planes of ASTM A325 bolts $r_{av} = \dfrac{36.0 \text{ kips}}{8 \text{ bolts}}$ = 4.50 kips/bolt < 10.6 kips/bolt **o.k.** $f_v = \dfrac{4.50 \text{ kips/bolt}}{0.442 \text{ in.}^2} = 10.2 \text{ ksi}$ F_{nt} = 90 ksi, F_{nv} = 48 ksi $\Omega = 2.00$ $F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \left(\dfrac{2.00(90 \text{ ksi})}{(48 \text{ ksi})}\right)(10.2 \text{ ksi})$ = 78.8 ksi < 90 ksi $R_n/\Omega = F'_{nt} A_b/\Omega$ = (78.8 ksi)(0.442 in.^{2})/2.00 = 17.4 kips/bolt > 5.7 kips/bolt **o.k.**

Manual Table 7-1

Table J3.2

Eqn. J3-3a and J3-3b

Eqn. J3-2

Check bearing strength at bolt holes

With L_e = 1¼ in. and s = 3 in., the bearing strength of the tee flange exceeds the single shear strength of the bolts. Therefore, bearing strength is o.k.

Check prying action — Manual Part 9

By inspection, prying action in the tee will control over prying action in the column.

Note: The bolts are not located symmetrically with respect to the centerline of the tee.

$$b = 3.12 \text{ in.} - \frac{0.350 \text{ in.}}{2} = 2.95 \text{ in.}$$

$$a = \frac{8.02 \text{ in.}}{2} - 3.12 \text{ in.} = 0.890 \text{ in.}$$

$$b' = b - \frac{d_b}{2} = 2.95 \text{ in.} - \left(\frac{0.750 \text{ in.}}{2}\right) = 2.58 \text{ in.}$$

Since a = 0.890 in. is less than 1.25b = 3.69 in., use a = 0.890 in. for calculation purposes

$$a' = a + \left(\frac{d_b}{2}\right) = 0.890 \text{ in.} + \left(\frac{0.750 \text{ in.}}{2}\right) = 1.27 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{2.58 \text{ in.}}{1.27 \text{ in.}} = 2.03$$

LRFD	ASD
$T = r_{ut} = 8.55$ kips/bolt $B = \phi r_n = 26.1$ kips/bolt $\beta = \frac{1}{\rho}\left(\frac{B}{T} - 1\right)$ $= \frac{1}{2.03}\left(\frac{26.1 \text{ kips/bolts}}{8.55 \text{ kips/bolts}} - 1\right) = 1.01$	$T = r_{at} = 5.70$ kips/bolt $B = r_n / \Omega = 17.4$ kips/bolt $\beta = \frac{1}{\rho}\left(\frac{B}{T} - 1\right)$ $= \frac{1}{2.03}\left(\frac{17.4 \text{ kips/bolts}}{5.70 \text{ kips/bolts}} - 1\right) = 1.01$

Manual Part 9

Since $\beta \geq 1$, set $\alpha' = 1.0$

Manual Part 9

$$p = \frac{11.5 \text{ in.}}{4 \text{ bolts}} = 2.88 \text{ in./bolt}$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{0.813 \text{ in.}}{2.88 \text{ in.}} = 0.718$$

LRFD	ASD
$t_{req} = \sqrt{\frac{4.44 r_{ut} b'}{p F_u (1 + \delta\alpha')}}$ $= \sqrt{\frac{4.44(8.55 \text{ kips/bolt})(2.58 \text{ in.})}{(2.88 \text{ in./bolt})(65 \text{ ksi})[1 + (0.718)(1.0)]}}$ = 0.552 in. < 0.620 in. **o.k.**	$t_{req} = \sqrt{\frac{6.66 r_{at} b'}{p F_u (1 + \delta\alpha')}}$ $= \sqrt{\frac{6.66(5.70 \text{ kips/bolt})(2.58 \text{ in.})}{(2.88 \text{ in./bolt})(65 \text{ ksi})[1 + (0.718)(1.0)]}}$ = 0.552 in. < 0.620 in. **o.k.**

Manual Part 9

Similarly, checks of the tee flange for shear yielding, shear rupture, and block shear rupture will show that the tee flange is o.k.

LRFD	ASD	
Check beam web for bolt bearing. For four rows of ¾ in. diameter bolts and an uncoped beam with F_y = 50 ksi and F_u = 65 ksi, ϕR_n = (351 kips/in.)(0.380 in.) = 133 kips > 54.0 kips **o.k.**	*Check beam web for bolt bearing.* For four rows of ¾ in. diameter bolts and an uncoped beam with F_y = 50 ksi and F_u = 65 ksi, R_n / Ω= (234 kips/in.)(0.380 in.) = 88.9 kips > 36.0 kips **o.k.**	Manual Table 10-1
Check supporting member web or flange for bolt bearing For four rows of ¾ in. diameter bolts with F_y = 50 ksi and F_u = 65 ksi, ϕR_n = (702 kips/in.)(0.710 in.) = 498 kips > 54.0 kips **o.k.**	*Check supporting member web or flange for bolt bearing* For four rows of ¾ in. diameter bolts with F_y = 50 ksi and F_u = 65 ksi, R_n / Ω = (468 kips/in.)(0.710 in.) = 332 kips > 36.0 kips **o.k.**	Manual Table 10-1

Note: Although the edge distance for one row of bolts in the tee flange does not meet the minimum value indicated in Table J3.4, based on footnote [a] for Table J3.4 the edge distance provided is acceptable because the provisions of Section J3.10 have been met in this case.

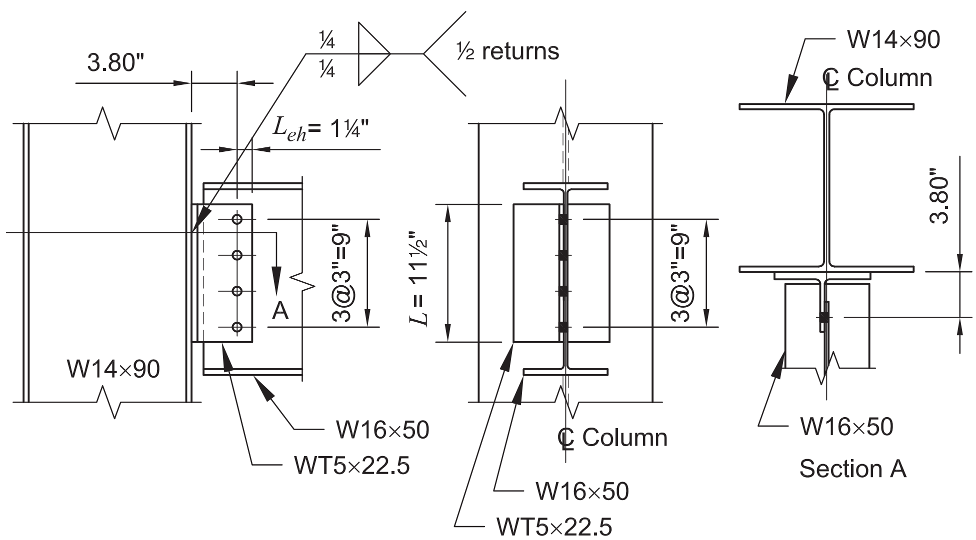
Example II.A-31 Bolted/Welded Tee Connection (beam-to-column flange)

Given:

Design a tee connection bolted to a W16×50 supported beam and welded to a W14×90 supporting column flange, to support the following beam end reactions:

R_D = 6 kips
R_L = 18 kips

Use ¾ in. diameter ASTM A325-N or F1852-N bolts in standard holes and E70 electrode welds. Try a WT5×22.5 with four-bolts.



Material Properties:

W16×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
W14×90	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
WT5×22.5	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	

Geometric Properties:

Beam	W16×50	t_w = 0.380 in.	d = 16.3 in.	t_f = 0.630 in.	Manual Tables 1-1 and 1-8
Column	W14×90	t_f = 0.710 in.			
Tee	WT5×22.5	d = 5.05 in.	b_f = 8.02 in.	t_f = 0.620 in.	
		t_s = 0.350 in.	k_1 = 13⁄16 in.	k_{des} = 1.12 in.	
			(see W10×45 Manual Table 1-1)		

Solution:

Calculate required strength

LRFD	ASD
R_u = 1.2(6.0 kips) + 1.6(18 kips) = 36.0 kips	R_a = 6.0 kips + 18 kips = 24.0 kips

Check limitation on tee stem thickness — Manual Part 9

See Rotational Ductility discussion at the beginning of the Manual Part 9

$$t_{s\,max} = \frac{d_b}{2} + \tfrac{1}{16}\text{ in.} = \frac{0.750\text{ in.}}{2} + \tfrac{1}{16}\text{ in.} = 0.438\text{ in.} > 0.350\text{ in.} \qquad \textbf{o.k.}$$

Design the welds connecting the tee flange to the column flange

b = flexible width in connection element

$$b = \frac{b_f - 2k_1}{2} = \frac{8.02\text{ in.} - 2(0.813\text{ in.})}{2} = 3.20\text{ in.}$$

Manual Part 9

$$w_{min} = 0.0158\frac{F_y t_f^{\,2}}{b}\left(\frac{b^2}{L^2} + 2\right) \leq \tfrac{5}{8} t_s$$

$$= 0.0158\left[\frac{(50\text{ ksi})(0.620\text{ in.})^2}{(3.20\text{ in.})}\right]\left[\left(\frac{(3.20\text{ in.})^2}{(11.5\text{ in.})^2}\right) + 2\right] \leq (\tfrac{5}{8})(0.350\text{ in.})$$

$$= 0.197\text{ in.} \leq 0.219\text{ in.}$$

Manual Part 10

Since the connection is flexible at the support, the welds attaching the tee flange to the support must be designed for the shear, but do not need to be designed for the eccentric moment.

Design welds for direct load only with no eccentricity

The minimum weld size is ¼–in. per Table J2.4

Manual Part 8

Try ¼–in. fillet welds.

LRFD	ASD
$\phi R_n = 1.392Dl$ $= 1.392$ (4 sixteenths)(2 sides)(11.5 in.) $= 128$ kips > 36.0 kips **o.k.** Use ¼–in. fillet welds. *Check the supporting column flange* From the table, for column flange material with $F_y = 50$ ksi, $n = 4$, $L = 11½$, Welds B, Weld size = ¼-in. the minimum support thickness is 0.190 in. $t_f = 0.710$ in. > 0.190 in. **o.k.**	$R_n / \Omega = 0.928Dl$ $= 0.928$ (4 sixteenths)(2 sides)(11.5 in.) $= 85.4$ kips > 24.0 kips **o.k.** Use ¼–in. fillet welds. *Check the supporting column flange* From the table, for column flange material with $F_y = 50$ ksi, $n = 4$, $L = 11½$, Welds B, Weld size = ¼-in. the minimum support thickness is 0.190 in. $t_f = 0.710$ in. > 0.190 in. **o.k.**

Manual Table 10-2

Check the stem side of the connection

Since the connection is flexible at the support, the tee stem and bolts must be designed for eccentric shear, where the eccentricity, e_b, is

$a = d - L_{eh} = 5.05$ in. – 1.25 in. = 3.80 in.

$e_b = a = 3.80$ in.

LRFD	ASD
Thus the tee stem and bolts must be designed for R_u = 36.0 kips and $R_u e_b$ = (36.0 kips)(3.80 in.) = 137 kip-in.	Thus the tee stem and bolts must be designed for R_a = 24.0 kips and $R_a e_b$ = (24.0 kips)(3.80 in.) = 91.2 kip-in.
Check bolt group through tee stem for shear and bearing	*Check bolt group through tee stem for shear and bearing*
Check tee stem for bolt bearing	*Check tee stem for bolt bearing*
For a 1¼ in. edge distance.	*For a* 1¼ in. edge distance.
ϕR_n = (49.4 kips/in.)(0.350 in.) = 17.3 kips > 15.9 kips **Bolt shear controls**	R_n / Ω = (32.9 kips/in.)(0.350 in.) = 11.5 kips > 10.6 kips **Bolt shear controls**
Note: By inspection, bolt bearing on the beam web does not control.	Note: By inspection, bolt bearing on the beam web does not control.
For $\theta = 0°$, with s = 3 in., $e_x = e_b$ = 3.80 in., and n = 4 bolts,	For $\theta = 0°$, with s = 3 in., $e_x = e_b$ = 3.80 in., and n = 4 bolts,
C = 2.45	C = 2.45
and, since bolt shear is more critical than bolt bearing in this example,	and, since bolt shear is more critical than bolt bearing in this example,
$\phi R_n = C\phi r_n$ = 2.45 (15.9 kips/bolt) = 39.0 kips > 36.0 kips **o.k.**	$R_n / \Omega = Cr_n / \Omega$ = 2.45 (10.6 kips/bolt) = 26.0 kips > 24.0 kips **o.k.**
Check flexure on the tee stem	*Check flexure on the tee stem*
For flexural yielding	For flexural yielding
$\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= (0.9)(50\text{ ksi})\left[\dfrac{(0.350\text{ in.})(11.5\text{ in.})^2}{4}\right]$ = 521 kip-in. > 137 kip-in. **o.k.**	$\Omega = 1.67$ $M_n / \Omega = F_y Z_x / \Omega$ $= \left(\dfrac{50\text{ ksi}}{1.67}\right)\left[\dfrac{(0.350\text{ in.})(11.5\text{ in.})^2}{4}\right]$ = 346 kip-in. > 91.2 kip-in. **o.k.**

Manual Table 7-6

Manual Table 7-1

Manual Table 7-7

Manual Table 7-1

Manual Part 15

For flexural rupture,

$$Z_{net} = 0.375\left[\frac{(11.5\text{ in.})^2}{4} - 2(0.875\text{ in.})(4.50\text{ in.}) - 2(0.875\text{ in.})(1.50\text{ in.})\right]$$
$$= 7.90\text{ in.}^3$$

LRFD	ASD
$\phi = 0.75$ $\phi M_n = \phi F_u Z_{net}$ = (0.75)(65 ksi)(7.98 in.^{3}) = 389 kip-in. > 137 kip-in. **o.k.**	$\Omega = 2.00$ $M_n/\Omega = F_u Z_{net} / \Omega$ = (65 ksi)(7.98 in.^{3})/2.00 = 259 kip-in. > 91.2 kip-in. **o.k.**

LRFD	ASD	
Check shear yielding of the tee stem	*Check shear yielding of the tee stem*	
$\phi = 1.00$ $\phi R_n = \phi 0.60 F_y A_g$ $= 1.00(0.6)(50 \text{ ksi})(11.5 \text{ in.})(0.350 \text{ in.})$ $= 121 \text{ kips} > 36.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $R_n / \Omega = 0.60 F_y A_g / \Omega$ $= 0.6(50 \text{ ksi})(11.5 \text{ in.})(0.350 \text{ in.}) / 1.50$ $= 80.5 \text{ kips} > 24.0 \text{ kips}$ **o.k.**	Eqn. J4-3
Check shear rupture of the tee stem	*Check shear rupture of the tee stem*	
$A_{nv} = [11.5 - 4(0.875 \text{ in.})](0.350 \text{ in.})$ $= 2.80 \text{ in.}^2$	$A_{nv} = [11.5 - 4(0.875 \text{ in.})](0.350 \text{ in.})$ $= 2.80 \text{ in.}^2$	
$\phi = 0.75$ $\phi R_n = \phi 0.6 F_u A_{nv}$ $= 0.75(0.6)(65 \text{ ksi})(2.80 \text{ in.}^2)$ $= 81.9 \text{ kips} > 36.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $R_n / \Omega = 0.6 F_u A_{nv} / \Omega$ $= \dfrac{0.6(65 \text{ ksi})(2.80 \text{ in.}^2)}{2.00}$ $= 54.6 \text{ kips} > 24.0 \text{ kips}$ **o.k.**	Eqn. J4-4
Check block shear rupture of the tee stem	*Check block shear rupture of the tee stem*	
$L_{eh} = L_{ev} = 1\frac{1}{4}$ in.,	$L_{eh} = L_{ev} = 1\frac{1}{4}$ in.,	
Thus, $\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	Thus, $\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn. J4-5
Tension rupture component $\phi F_u A_{nt}$ = 39.6 kips/in. (0.350 in.)	Tension rupture component $F_u A_{nt} / \Omega$ = 26.4 kips/in. (0.350 in.)	Manual Table 9-3a
Shear yielding component $\phi 0.6 F_y A_{gv}$ = 231 kips/in. (0.350 in.)	Shear yielding component $0.6 F_y A_{gv} / \Omega$ = 154 kips/in. (0.350 in.)	Manual Table 9-3b
Shear rupture component $\phi 0.6 F_u A_{nv}$ = 210 kips/in. (0.350 in.)	Shear rupture component $0.6 F_u A_{nv} / \Omega$ = 140 kips/in. (0.350 in.)	Manual Table 9-3c
ϕR_n = (210 kips/in. + 39.6 kips/in.)(0.350in.) = 87.4 kips > 36.0 kips **o.k.**	$\dfrac{R_n}{\Omega}$ = (140 kips/in. + 26.4 kips/in.)(0.350 in.) = 58.2 kips > 24.0 kips **o.k.**	

Chapter IIB
Fully-Restrained (FR) Moment Connections

The design of fully restrained (FR) moment connections is covered in Part 12 of the AISC *Steel Construction Manual.*

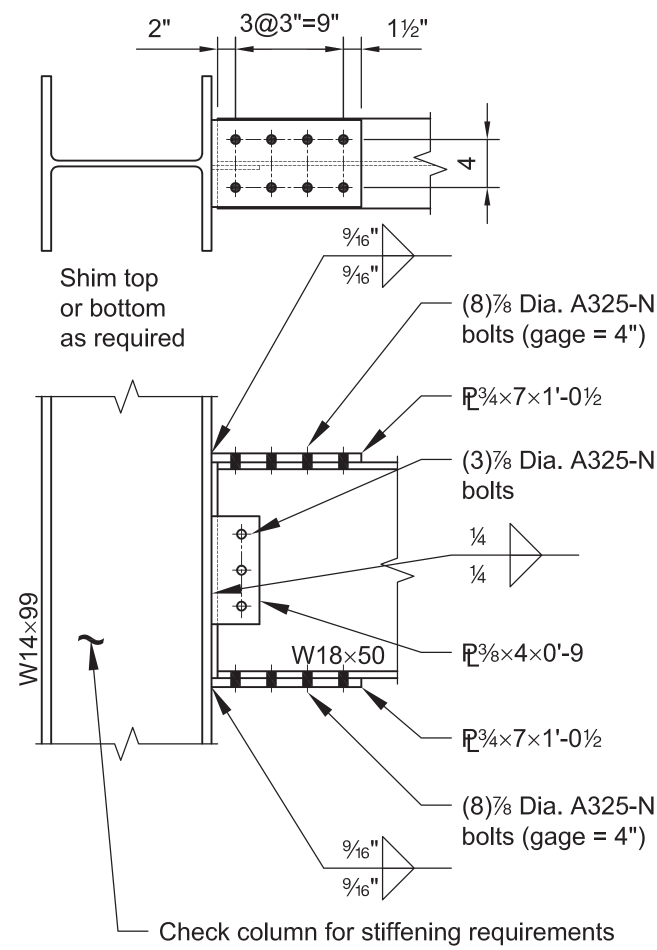
Example II.B-1 Bolted Flange-Plate FR Moment Connection (beam-to-column flange)

Given:

Design a bolted flange-plated FR moment connection between a W18×50 beam and a W14×99 column flange to transfer the following forces:

R_D = 7.0 kips M_D = 42 kip-ft
R_L = 21 kips M_L = 126 kip-ft

Use ⅞-in. diameter ASTM A325-N or F1852-N bolts in standard holes and E70 electrodes.



Material Properties:

Beam	W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	W14×99	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Table 2-3
Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	Table 2-4

Geometric Properties:

Manual Table 1-1

Beam W18×50 d = 18.0 in. b_f = 7.50 in. t_f = 0.570 in. t_w = 0.355 in. S_x = 88.9 in.^{3}
Column W14×99 d = 14.2 in. b_f = 14.6 in. t_f = 0.780 in. t_w = 0.485 in. k_{des} = 1.38 in.

Solution:

Calculate requried strength

LRFD	ASD
$R_u = 1.2(7.0 \text{ kips}) + 1.6(21 \text{ kips})$ $= 42.0 \text{ kips}$ $M_u = 1.2(42 \text{ kip-ft}) + 1.6(126 \text{ kip-ft})$ $= 252 \text{ kip-ft}$	$R_a = 7.0 \text{ kips} + 21 \text{ kips}$ $= 28.0 \text{ kips}$ $M_a = 42 \text{ kip-ft} + 126 \text{ kip-ft}$ $= 168 \text{ kip-ft}$

Check the beam available flexural strength

Assume two rows of bolts in standard holes.

$$A_{fg} = b_f t_f = (7.50 \text{ in.})(0.570 \text{ in.}) = 4.28 \text{ in.}^2$$

Section F13.1

$$A_{fn} = A_{fg} - 2(d_b + \tfrac{1}{8} \text{ in.})t_f = 4.28 \text{ in.}^2 - 2(0.875 \text{ in.} + 0.125 \text{ in.})(0.570 \text{ in.}) = 3.14 \text{ in.}^2$$

$$\frac{F_y}{F_u} = \frac{50 \text{ ksi}}{65 \text{ ksi}} = 0.769 \leq 0.8 \text{, therefore } Y_t = 1.0.$$

$$F_u A_{fn} = (65 \text{ ksi})(3.14 \text{ in.}^2) = 204 \text{ kips}$$

$$Y_t F_y A_{fg} = (1.0)(50 \text{ ksi})(4.28 \text{ in.}^2) = 214 \text{ kips} > 204 \text{ kips}$$

Therefore the nominal flexural strength, M_n, at the location of the holes in the tension flange is not greater than:

$$M_n = \frac{F_u A_{fn} S_x}{A_{fg}} = \frac{(65 \text{ ksi})(3.14 \text{ in.}^2)(88.9 \text{ in.}^3)}{4.28 \text{ in.}^2} = 4240 \text{ kip-in. or } 353 \text{ kip-ft}$$

Eqn F13-1

LRFD	ASD
$\phi = 0.90$ $\phi_b M_n = 0.90(353 \text{ kip-ft}) = 318 \text{ kip-ft}$ 318 kip-ft > 252 kip-ft **o.k.**	$\Omega = 1.67$ $M_n / \Omega_b = \dfrac{353 \text{ kip-ft}}{1.67} = 211 \text{ kip-ft}$ 211 kip-ft > 168 kip-ft **o.k.**

Note: The available flexural strength of the beam may be less than determined based on Eqn. F13-1. Other applicable provisions in Section F could be checked to possibly determine a lower value for the available flexural strength of the beam.

Design single-plate web connection

Try a PL ⅜×4×0′-9″, with three ⅞-in. diameter ASTM A325-N bolts and ¼-in. fillet welds.

LRFD	ASD	
Design shear strength of the bolts	*Allowable shear strength of bolts*	
Single shear;	Single shear;	
ϕr_n = 21.6 kips/bolt	r_n/Ω = 14.4 kips/bolt	Manual Table 7-1
Bearing strength of bolts	*Bearing strength of bolts*	
Bearing on the plate controls over bearing on the beam web.	Bearing on the plate controls over bearing on the beam web.	
Edge distance = 1.50 in.	Edge distance = 1.50 in.	
$L_c = 1.50\text{ in.} - \dfrac{0.938\text{ in.}}{2} = 1.03\text{ in.}$	$L_c = 1.50\text{ in.} - \dfrac{0.938\text{ in.}}{2} = 1.03\text{ in.}$	
$\phi = 0.75$ $\phi r_n = \phi 1.2 L_c t F_u \le \phi 2.4 d t F_u$	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{1.2 L_c t F_u}{\Omega} \le \dfrac{2.4 d t F_u}{\Omega}$	Eqn. J3-6a
$0.75(1.2)(1.03\text{ in.})(0.375\text{ in.})(58\text{ ksi}) \le 0.75(2.4)(0.875\text{ in.})(0.375\text{ in.})(58\text{ ksi})$	$\dfrac{1.2(1.03\text{ in.})(0.375\text{in.})(58\text{ ksi})}{2.00} \le \dfrac{2.4(0.875\text{ in.})(0.375\text{ in.})(58\text{ ksi})}{2.00}$	
$20.2\text{ kips} \le 34.3\text{ kips}$ $\phi r_n = 20.2\text{ kips}$	$13.5\text{kips} \le 22.8\text{ kips}$ $r_n/\Omega = 13.5\text{kips}$	
Bolt spacing = 3.00 in.	Bolt spacing = 3.00 in.	
ϕr_n = (91.4 kips/in./bolt)(0.375 in.) = 34.3 kips/bolt	r_n/Ω = (60.9 kips/in./bolt)(0.375 in.) = 22.8 kips/bolt	Manual Table 7-5
The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is:	The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is:	
$\phi R_n = n\phi r_n$ $= 1(20.2\text{ kips/bolt}) + 2(21.6\text{ kips/bolt})$ $= 63.4\text{ kips} > 42.0\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{nr_n}{\Omega}$ $= 1(13.5\text{ kips/bolt}) + 2(14.4\text{ kips/bolt})$ $= 42.3\text{ kips} > 28.0\text{ kips}$ **o.k.**	
Plate shear yielding	*Plate shear yielding*	
ϕ = 1.00 $\phi R_n = 0.60\,\phi F_y A_g$	Ω = 1.50 $R_n/\Omega = 0.60\,F_y A_g/\Omega$	Eqn J4-3

LRFD	ASD
$= 0.60(1.00)(36\text{ ksi})(9.00\text{ in.})(0.375\text{ in.})$ $= 72.9\text{ kips} > 42.0\text{ kips}$ **o.k.**	$= \dfrac{0.60(36\text{ ksi})(9.00\text{ in.})(0.375\text{ in.})}{1.50}$ $= 48.6\text{ kips} > 28.0\text{ kips}$ **o.k.**
Plate shear rupture Total length of bolt holes (3 bolts)(0.875 in. + 0.0625 in. + 0.0625in.) = 3.00 in. A_{nv} = (9.00 in. -3.00 in.)(0.375 in.) = 2.25 in.^{2} $\phi = 0.75$ $\phi R_n = 0.60\ \phi F_u A_{nv}$ $= 0.60(0.75)(58\text{ ksi})(2.25\text{ in.}^2)$ $= 58.7\text{ kips} > 42.0\text{ kips}$ **o.k.**	*Plate shear rupture* Total length of bolt holes (3 bolts)(0.875 in. + 0.0625 in. + 0.0625in.) = 3.00 in. A_{nv} = (9.00 in. –3.00 in.)(0.375 in.) = 2.25 in.^{2} $\Omega = 2.00$ $R_n / \Omega = 0.60\ F_u A_{nv} / \Omega$ $= 0.60(58\text{ ksi})(2.25\text{ in.}^2)/(2.00)$ $= 39.2\text{ kips} > 28.0\text{ kips}$ **o.k.**

Eqn J4-4

Block shear rupture strength of the plate

L_{eh} = 1¼ in.; L_{ev} = 1½ in.; U_{bs} = 1.0; n = 3

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$ Tension rupture component $\phi F_u A_{nt}$ = 32.6 kips/in.(0.375 in.) Shear yielding component $\phi 0.6 F_y A_{gv}$ = 121 kips/in.(0.375 in.) Shear rupture component $\phi 0.6 F_u A_{nv}$ = 131 kips/in.(0.375 in.) ϕR_n = (121 kips/in. + 32.6 kips/in.)(0.375 in.) = 57.6 kips > 42 kips **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$ Tension rupture component $F_u A_{nt} / \Omega$ = 21.8 kips/in.(0.375 in.) Shear yielding component $0.6 F_y A_{gv} / \Omega$ = 81.0 kips/in.(0.375 in.) Shear rupture component $0.6 F_u A_{nv} / \Omega$ = 87.0 kips/in.(0.375 in.) R_n / Ω = (81.0 kips/in. + 21.8 kips/in.)(0.375 in.) = 38.6 kips > 24.0 kips **o.k.**
Weld Strength $\phi R_n = 1.392 D\, l\, (2)$ = 1.392(4 sixteenths)(9.00 in.)(2) = 100 kips > 42.0 kips **o.k.**	*Weld Strength* $R_n / \Omega = 0.928 Dl\, (2)$ = 0.928(4 sixteenths)(9.00 in.)(2) = 66.8 kips > 28.0 kips **o.k.**

Eqn.J4-5

Manual Table 9-3a

Manual Table 9-3b

Manual Table 9-3c

Manual Part 8

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

Section J4.2
Manual Part 9

$$t_{\min} = \frac{0.6F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \quad \text{for } F_{exx} = 70.0 \text{ ksi}$$

Column flange; $t_f = 0.780$ in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(4 \text{ sixteenths})}{65 \text{ ksi}} = 0.190 \text{ in.} < 0.780 \text{ in.} \qquad \textbf{o.k.}$$

Note: By inspection, the available shear yielding and shear rupture strength of the beam web is o.k.

Design tension flange plate and connection

Design of bolts

LRFD	ASD	
$P_{uf} = \frac{M_u}{d} = \frac{(252 \text{ kip-ft})(12 \text{ in./ft})}{18.0 \text{ in.}} = 168 \text{ kips}$	$P_{af} = \frac{M_a}{d} = \frac{(168 \text{ kip-ft})(12 \text{ in./ft})}{18.0 \text{ in.}} = 112 \text{ kips}$	Manual Part 12
Try a PL ¾× 7	Try a PL ¾×7	
Determine critical bolt strength	Determine critical bolt strength	
For shear, $\phi r_n = 21.6$ kips/bolt	For shear, $r_n / \Omega = 14.4$ kips/bolt	Manual Table 7-1
For bearing on flange;	For bearing on flange;	
Edge distance = 1½ in. (use 1¼ in. to account for possible underrun in beam length.)	Edge distance = 1½ in. (use 1¼ in. to account for possible underrun in beam length.)	
$\phi r_n = (45.7 \text{ kips/bolt})t_f$ $= (45.7 \text{ kips/bolt})(0.570 \text{ in.})$ $= 26.0$ kips/bolt	$r_n / \Omega = (30.5 \text{ kips/bolt})t_f$ $= (30.5 \text{ kips/bolt})(0.570 \text{ in.})$ $= 17.4$ kips/bolt	
For bearing in plate;	For bearing in plate;	
Edge distance = 1½ in. (Conservatively, use 1¼ in. value from table)	Edge distance = 1½ in. (Conservatively, use 1¼ in. value from table)	
$\phi r_n = (40.8 \text{ kips/bolt})t_p$ $= (40.8 \text{ kips/bolt})(0.750 \text{ in.})$ $= 30.6$ kips/bolt	$r_n / \Omega = (27.2 \text{ kips/bolt})t_p$ $= (27.2 \text{ kips/bolt})(0.750 \text{ in.})$ $= 20.4$ kips/bolt	Manual Table 7-6
Shear controls, therefore the number of bolts required is as follows:	Shear controls, therefore the number of bolts required is as follows:	
$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{168 \text{ kips}}{21.6 \text{ kips/bolt}} = 7.78 \text{ bolts}$	$n_{\min} = \frac{P_{af}}{r_n / \Omega} = \frac{112 \text{ kips}}{14.4 \text{ kips/bolt}} = 7.78 \text{ bolts}$	
Use eight bolts.	Use eight bolts.	

Check flange plate tension yielding

$P_n = F_y A_g = (36 \text{ ksi})(7.00 \text{ in.})(0.750 \text{ in.}) = 189 \text{ kips}$ Eqn. J4-1

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d+t_p} = \dfrac{(252 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.} + 0.750 \text{ in.})}$ $= 161 \text{ kips}$ $\phi = 0.90$ $\phi P_n = 0.90(189 \text{ kips}) = 170 \text{ kips}$ 170 kips > 161 kips **o.k.**	$P_{uf} = \dfrac{M_u}{d+t_p} = \dfrac{(168 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.} + 0.750 \text{ in.})}$ $= 108 \text{ kips}$ $\Omega = 1.67$ $P_n / \Omega = 189 \text{ kips}/1.67 = 113 \text{ kips}$ 113 kips > 108 kips **o.k.**

Check flange plate tension rupture

$0.85\,A_g = 0.85(7.00 \text{ in.})(0.750 \text{ in.}) = 4.46 \text{ in.}^2$ Section J4.1b

$A_n = \left[B - 2\left(d_b + \tfrac{1}{8} \text{ in.}\right)\right]t_p = \left[(7.00 \text{ in.}) - 2(0.875 \text{ in.} + 0.125 \text{ in.})\right](0.750 \text{ in.}) = 3.75 \text{ in.}^2$

$A_e = A_n \le 0.85A_g,\ A_e = 3.75 \text{ in.}^2$

$P_n = F_u A_e = (58 \text{ ksi})(3.75 \text{ in.}^2) = 218 \text{ kips}$ Eqn. J4-2

LRFD	ASD
$\phi = 0.75$ $\phi P_n = 0.75(218 \text{ kips}) = 164 \text{ kips}$ 164 kips > 161 kips **o.k.**	$\Omega = 2.00$ $P_n / \Omega = 218 \text{ kips}/2.0 = 109 \text{ kips}$ 109 kips > 108 kips **o.k.**

Check flange plate block shear rupture

There are two cases for which block shear rupture must be checked. The first case involves the tearout of the two blocks outside the two rows of bolt holes in the flange plate; for this case L_{eh} = 1½ in. and L_{ev} = 1½ in. The second case involves the tearout of the block between the two rows of the holes in the flange plate. Manual Tables 9-3a, 9-3b, and 9-3c may be adapted for this calculation by considering the 4 in. width to be comprised of two, 2 in. wide blocks where L_{eh} = 2 in. and L_{ev} = 1½ in. Thus, the former case is the more critical.

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6F_y A_{gv},\ \phi 0.6F_u A_{nv}\right)$ Tension component $\phi U_{bs} F_u A_{nt} = 43.5 \text{ kips/in.}\ (0.75 \text{ in.})(2)$ Shear yielding component $\phi 0.6F_y A_{gv} = 170 \text{ kips/in.}\ (0.75 \text{ in.})(2)$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6F_y A_{gv}}{\Omega},\ \dfrac{0.6F_u A_{nv}}{\Omega}\right)$ Tension component $F_u A_{nt} / \Omega = 29.0 \text{ kips/in.}\ (0.75 \text{ in.})(2)$ Shear yielding component $0.6F_y A_{gv} / \Omega = 113 \text{ kips/in.}\ (0.75 \text{ in.})(2)$

Eqn. J4-5

Manual Table 9-3a

Manual Table 9-3b

LRFD	ASD
Shear rupture component	Shear rupture component
$\phi 0.6F_uA_{nv} = 183\text{ kips/in.}\left(0.75\text{ in.}\right)\left(2\right)$	$0.6F_uA_{nv}/\Omega = 122\text{ kips/in.}\left(0.75\text{ in.}\right)\left(2\right)$
Shear yielding controls, thus	Shear yielding controls, thus
$\phi R_n = \left(\dfrac{170\text{ kips}}{\text{in.}} + \dfrac{43.5\text{ kips}}{\text{in.}}\right)\left(0.75\text{ in.}\right)\left(2\right)$ $= 320\text{ kips} > 161\text{ kips}$ **o.k.**	$R_n/\Omega = \left(\dfrac{113\text{ kips}}{\text{in.}} + \dfrac{29.0\text{ kips}}{\text{in.}}\right)\left(0.75\text{ in.}\right)\left(2\right)$ $= 213\text{ kips} > 108\text{ kips}$ **o.k.**

Manual Table 9-3c

Check block shear rupture of the beam flange

Eqns. J2-4, and J2-5

This case involves the tearout of the two blocks outside the two rows of bolt holes in the flanges; for this case L_{eh} = 1¾ in. and L_{ev} = 1½ in. (use 1¼ in. to account for possible underrun in beam length.)

LRFD	ASD
$\phi R_n = \phi F_uA_{nt}U_{bs} + \min\left(\phi 0.6F_yA_{gv},\ \phi 0.6F_uA_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_uA_{nt}U_{bs}}{\Omega} + \min\left(\dfrac{0.6F_yA_{gv}}{\Omega},\ \dfrac{0.6F_uA_{nv}}{\Omega}\right)$
Tension component	Tension component
$\phi U_{bs}F_uA_{nt} = 54.4\text{ kips/in.}\left(0.570\text{ in.}\right)\left(2\right)$	$F_uA_{nt}/\Omega = 36.3\text{ kips/in.}\left(0.570\text{ in.}\right)\left(2\right)$
Shear yielding component	Shear yielding component
$\phi 0.6F_yA_{gv} = 231\text{ kips/in.}\left(0.570\text{ in.}\right)\left(2\right)$	$0.6F_yA_{gv}/\Omega = 154\text{ kips/in.}\left(0.570\text{ in.}\right)\left(2\right)$
Shear rupture component	Shear rupture component
$\phi 0.6F_uA_{nv} = 197\text{ kips/in.}\left(0.570\text{ in.}\right)\left(2\right)$	$0.6F_uA_{nv}/\Omega = 132\text{ kips/in.}\left(0.570\text{ in.}\right)\left(2\right)$
Shear rupture controls, thus	Shear rupture controls, thus
$\phi R_n = \left(\dfrac{197\text{ kips}}{\text{in.}} + \dfrac{54.4\text{ kips}}{\text{in.}}\right)\left(0.570\text{ in.}\right)\left(2\right)$ $= 287\text{ kips} > 168\text{ kips}$ **o.k.**	$R_n/\Omega = \left(\dfrac{132\text{ kips}}{\text{in.}} + \dfrac{36.3\text{ kips}}{\text{in.}}\right)\left(0.570\text{ in.}\right)\left(2\right)$ $= 192\text{ kips} > 112\text{ kips}$ **o.k.**

Eqn. J4-5

Manual Table 9-3a

Manual Table 9-3b

Manual Table 9-3c

Determine the required size of the fillet weld to supporting column flange

Manual Part 8

The applied load is perpendicular to the weld length; therefore $\theta = 90°$ and $1.0 + \sin^{1.5}\theta = 1.5$.

LRFD	ASD
$D_{\min} = \dfrac{P_{uf}}{2\left(1.5\right)\left(1.392\right)l}$ $= \dfrac{161\text{ kips}}{2\left(1.5\right)\left(1.392\right)\left(7.00\text{ in.}\right)}$ $= 5.51$ sixteenths	$D_{\min} = \dfrac{P_{af}}{2\left(1.5\right)\left(0.928\right)l}$ $= \dfrac{108\text{ kips}}{2\left(0.928\right)\left(7.00\text{ in.}\right)}$ $= 5.54$ sixteenths

Use ⅜-in. fillet welds, 6 > 5.51 **o.k.**	Use ⅜-in. fillet welds, 6 > 5.54 **o.k.**

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

Manual Part 9

$$t_{\min} = \frac{0.6F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \text{ for } F_{exx} = 70 \text{ ksi}$$

Column flange; t_f = 0.780 in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5.54 \text{ sixteenths})}{65 \text{ ksi}} = 0.263 \text{ in.} < 0.780 \text{ in.} \quad \textbf{o.k.}$$

Design compression flange plate and connection

Try PL ¾×7

Assume K = 0.65 and l = 2.00 in. (1½ in. edge distance and ½ in. setback).

Table C-C2.2

$$\frac{Kl}{r} = \frac{0.65(2.00 \text{ in.})}{\left(\frac{0.750 \text{ in.}}{\sqrt{12}}\right)} = 6.00 \le 25$$

Therefore, $F_{cr} = F_y$

Section J4.4

$$A = (7.00 \text{ in.})(0.750 \text{ in.}) = 5.25 \text{ in.}^2$$

LRFD	ASD
$\phi = 0.90$ $\phi P_n = \phi F_y A_g = 0.90(36 \text{ ksi})(5.25 \text{ in.}^2)$ $= 170 \text{ kips} > 161 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $P_n / \Omega = \frac{F_y A_g}{\Omega} = \frac{(36 \text{ ksi})(5.25 \text{ in.}^2)}{1.67}$ $= 113 \text{ kips} > 108 \text{ kips}$ **o.k.**

Eqn. J4-6

The compression flange plate will be identical to the tension flange plate; a ¾ in.×7 in. plate with eight bolts in two rows of four bolts on a 4 in. gage and ⁹⁄₁₆ in. fillet welds to the supporting column flange.

Note: The bolt bearing and shear checks are the same as for the tension flange plate and are o.k. by inspection. Tension due to load reversal must also be considered in the design of the fillet weld to the supporting column flange.

Check the column for concentrated forces

Check flange local bending

Assume the concentrated force to be resisted is applied at a distance from the member end that is greater than $10t_f$. $10t_f$ = 10(0.780 in.) = 7.80 in.

LRFD	ASD
$R_n = 6.25t_f{}^2F_{yf}$ $= 6.25(0.780\text{ in.})^2(50\text{ ksi}) = 190\text{ kips}$ $\phi = 0.90$ $\phi R_n = 0.90(190\text{ kips}) = 171\text{ kips} > 161\text{ kips}$ **o.k.**	$R_n = 6.25t_f{}^2F_{yf}$ $= 6.25(0.780\text{ in.})^2(50\text{ ksi}) = 190\text{ kips}$ $\Omega = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{190\text{ kips}}{1.67} = 114\text{ kips} > 108\text{ kips}$ **o.k.**

Eqn. J10-1

Check web local yielding

Assume the concentrated force to be resisted is applied at a distance from the member that is greater than the depth of the member d (d = 14.2 in.)

Section J10.2

LRFD	ASD
$\phi R_n = 2(\phi R_1) + N(\phi R_2)$ $= 2(83.5\text{ kips}) + (0.750\text{ in.})(24.3\text{ kips/in.})$ $= 185\text{ kips} > 161\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 2\left(\dfrac{R_1}{\Omega}\right) + N\left(\dfrac{R_2}{\Omega}\right)$ $= 2(55.7\text{ kips}) + (6.750\text{ in.})(16.2\text{ kips/in.})$ $= 124\text{ kips} > 108\text{ kips}$ **o.k.**

Manual Table 9-4

Check web crippling

Assume the concentrated force to be resisted is applied at a distance from the member end that is greater than or equal to d/2 [d/2 = (14.2 in.)/2 = 7.10 in.]

Section J10.3

LRFD	ASD
$\phi R_n = 2(\phi R_3) + 2N(\phi R_4)$ $= 2(108\text{ kips}) + 2(0.750\text{ in.})(11.2\text{ kips/in.})$ $= 233\text{ kips} > 161\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 2\left(\dfrac{R_3}{\Omega}\right) + 2N\left(\dfrac{R_4}{\Omega}\right)$ $= 2(71.8\text{ kips}) + 2(0.750\text{ in.})(7.46\text{ kips/in.})$ $= 155\text{ kips} > 108\text{ kips}$ **o.k.**

Manual Table 9-4

Note: Web compression buckling (Section J10.5) would also need to be checked if another beam framed into the opposite side of the column at this location.

Web panel zone shear (Section J10.6) should also be checked for this column.

For further information, see AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections – Wind and Seismic Applications* (Carter, 1999).

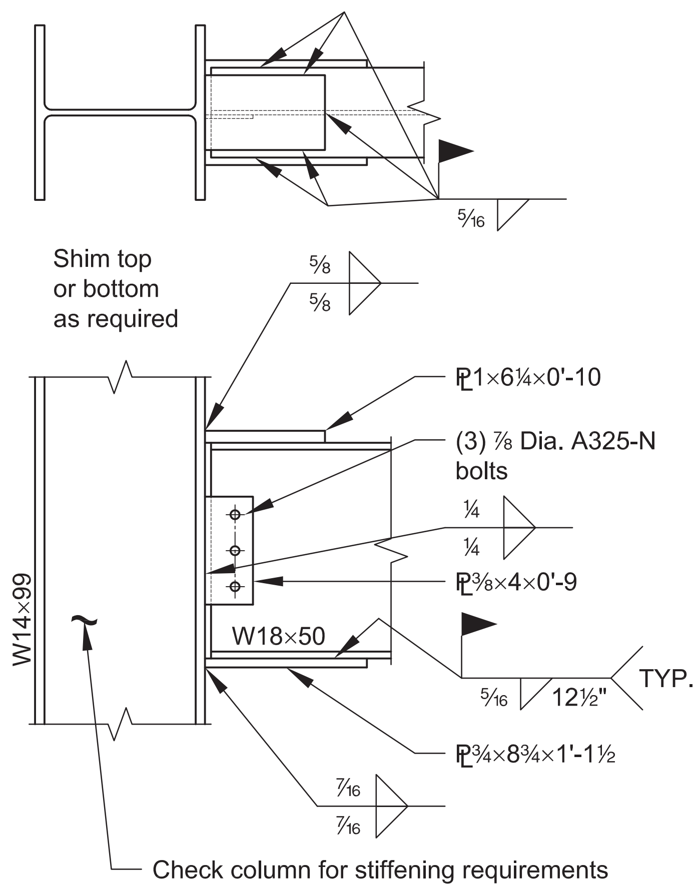
Example II.B-2 Welded Flange-Plated FR Moment Connection (beam-to-column flange)

Given:

Design a welded flange-plated FR moment connection between a W18×50 beam and a W14×99 column flange to transfer the following forces:

R_D = 7.0 kips M_D = 42 kip-ft
R_L = 21 kips M_L = 126 kip-ft

Use ⅞-in. diameter ASTM A325-N or F1852-N bolts in standard holes and E70 electrodes.



Material Properties:

Beam	W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	W14×99	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Table 2-3
Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	Table 2-4

Geometric Properties:

Beam	W18×50	d = 18.0 in.	b_f = 7.50 in.	t_f = 0.570 in.	t_w = 0.355 in.	Z_x = 101 in.^{3}	Manual
Column	W14×99	d = 14.2 in.	b_f = 14.6 in.	t_f = 0.780 in.	t_w = 0.485 in.	k_{des} = 1.38 in.	Table 1-1

Solution:

LRFD	ASD
R_u = 1.2(7.0 kips) + 1.6(21 kips) = 42.0 kips M_u = 1.2(42 kip-ft) + 1.6(126 kip-ft) = 252 kip-ft	R_a = 7.0 kips + 21 kips = 28.0 kips M_a = 42 kip-ft + 126 kip-ft = 168 kip-ft

Design the single-plate web connection

Try a PL ⅜×4×0′-9″, with three ⅞–in. diameter ASTM A325-N bolts and ¼ in. fillet welds.

LRFD	ASD	
Design shear strength of the bolts Single shear; ϕr_n = 21.6 kips/bolt	*Allowable shear strength of bolts* Single shear; r_n/Ω = 14.4 kips/bolt	Manual Table 7-1
Bearing strength of bolts Bearing on the plate controls over bearing on the beam web. Edge distance = 1.50 in. $L_c = 1.50\text{ in.} - \dfrac{0.938\text{ in.}}{2} = 1.03\text{ in.}$ $\phi = 0.75$ $\phi r_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d t F_u$ $0.75(1.2)(1.03\text{ in.})(0.375\text{ in.})(58\text{ ksi}) \leq 0.75(2.4)(0.875\text{ in.})(0.375\text{ in.})(58\text{ ksi})$ $20.2\text{ kips} \leq 34.3\text{ kips}$ $\phi r_n = 20.2\text{ kips}$	*Bearing strength of bolts* Bearing on the plate controls over bearing on the beam web. Edge distance = 1.50 in. $L_c = 1.50\text{ in.} - \dfrac{0.938\text{ in.}}{2} = 1.03\text{ in.}$ $\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{1.2 L_c t F_u}{\Omega} \leq \dfrac{2.4 d t F_u}{\Omega}$ $\dfrac{1.2(1.03\text{ in.})(0.375\text{ in.})(58\text{ ksi})}{2.00} \leq \dfrac{2.4(0.875\text{ in.})(0.375\text{ in.})(58\text{ ksi})}{2.00}$ $13.5\text{ kips} \leq 22.8\text{ kips}$ $r_n/\Omega = 13.5\text{ kips}$	Eqn. J3-6a
Bolt spacing = 3.00 in. ϕr_n = (91.4 kips/in./bolt)(0.375 in.) = 34.3 kips/bolt	Bolt spacing = 3.00 in. r_n/Ω = (60.9 kips/in./bolt)(0.375 in.) = 22.8 kips/bolt	Manual Table 7-5
The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is:	The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is: $\dfrac{R_n}{\Omega} = \dfrac{nr_n}{\Omega}$ $= 1(13.5\text{ kips/bolt}) + 2(14.4\text{ kips/bolt})$	

LRFD	ASD	
$\phi R_n = n\phi r_n$ $= 1(20.2 \text{ kips/bolt}) + 2(21.6 \text{ kips/bolt})$ $= 63.4$ kips > 42.0 kips **o.k.** *Plate shear yielding* $\phi = 1.00$ $\phi R_n = 0.60\ \phi F_y A_g$ $= 0.60(1.00)(36 \text{ ksi})(9.00 \text{ in.})(0.375 \text{ in.})$ $= 72.9$ kips > 42.0 kips **o.k.** *Plate shear rupture* Total length of bolt holes (3 bolts)(0.875 in. + 0.0625 in. + 0.0625in.) = 3.00 in. $A_{nv} = (9.00 \text{ in.} - 3.00 \text{ in.})(0.375 \text{ in.}) = 2.25 \text{ in}^2$ $\phi = 0.75$ $\phi R_n = 0.60\ \phi F_u A_{nv}$ $= 0.60(0.75)(58 \text{ ksi})(2.25 \text{ in.}^2)$ $= 58.7$ kips > 42.0 kips **o.k.**	$= 42.3$ kips > 28.0 kips **o.k.** *Plate shear yielding* $\Omega = 1.50$ $R_n / \Omega = 0.60\ F_y A_g / \Omega$ $= \dfrac{0.60(36 \text{ ksi})(9.00 \text{ in.})(0.375 \text{ in.})}{1.50}$ $= 48.6$ kips > 28.0 kips **o.k.** *Plate shear rupture* Total length of bolt holes (3 bolts)(0.875 in. + 0.0625 in. + 0.0625in.) = 3.00 in. $A_{nv} = (9.00 \text{ in.} - 3.00 \text{ in.})(0.375 \text{ in.}) = 2.25 \text{ in}^2$ $\Omega = 2.00$ $R_n / \Omega = 0.60\ F_u A_{nv} / \Omega$ $= 0.60(58 \text{ ksi})(2.25 \text{ in.}^2)/(2.00)$ $= 39.2$ kips > 28.0 kips **o.k.**	Eqn J4-3 Eqn J4-4

Block shear rupture strength of the plate
$L_{eh} = 1\frac{1}{4}$ in.; $L_{ev} = 1\frac{1}{2}$ in.; $U_{bs} = 1.0$; $n = 3$

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn.J4-5
Tension rupture component $\phi F_u A_{nt} = 32.6$ kips/in.(0.375 in.)	Tension rupture component $F_u A_{nt} / \Omega = 21.8$ kips/in.(0.375 in.)	Manual Table 9-3a
Shear yielding component $\phi 0.6 F_y A_{gv} = 121$ kips/in.(0.375 in.)	Shear yielding component $0.6 F_y A_{gv} / \Omega = 81.0$ kips/in.(0.375 in.)	Manual Table 9-3b
Shear rupture component $\phi 0.6 F_u A_{nv} = 131$ kips/in.(0.375 in.)	Shear rupture component $0.6 F_u A_{nv} / \Omega = 87.0$ kips/in.(0.375 in.)	Manual Table 9-3c
$\phi R_n = (121 \text{ kips/in.} + 32.6 \text{ kips/in.})(0.375 \text{ in.})$ $= 57.6$ kips > 42 kips **o.k.**	$R_n / \Omega = (81.0 \text{ kips/in.} + 21.8 \text{ kips/in.})(0.375 \text{ in.})$ $= 38.6$ kips > 24.0 kips **o.k.**	
Weld Strength $\phi R_n = 1.392 D\, l\ (2)$ $= 1.392(4 \text{ sixteenths})(9.00 \text{ in.})(2)$ $= 100$ kips > 42.0 kips **o.k.**	*Weld Strength* $R_n / \Omega = 0.928 Dl\ (2)$ $= 0.928(4 \text{ sixteenths})(9.00 \text{ in.})(2)$ $= 66.8$ kips > 28.0 kips **o.k.**	Manual Part 8

Design top tension flange plate and connection

Determine the flange force

The top flange width b_f = 7.50 in. Assume a shelf dimension of ⅝ in. on both sides of the plate. The plate width, then, is 7.50 in. – 2(0.625 in.) = 6.25 in. Try a 1 in.×6¼ in. flange plate.

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d+t_p}$ $= \dfrac{(252\text{ kip-ft})(12\text{ in./ft})}{(18.0\text{ in.}+0.875\text{ in.})} = 160\text{ kips}$	$P_{af} = \dfrac{M_a}{d+t_p}$ $= \dfrac{(168\text{ kip-ft})(12\text{ in./ft})}{(18.0\text{ in.}+0.875\text{ in.})} = 107\text{ kips}$

Manual Part 12

Check top flange plate tension yielding

$$R_n = F_y A_g = (36\text{ ksi})(6.25\text{ in.})(1.00\text{ in.}) = 225\text{ kips}$$ Eqn. J4-1

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(225\text{ kips}) = 203\text{ kips}$ 203 kips > 160 kips **o.k.**	$\Omega = 1.67$ $R_n / \Omega = 225\text{ kips}/1.67 = 135\text{ kips}$ 135 kips > 107 kips **o.k.**

Determine the force in the welds

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d}$ $= \dfrac{(252\text{ kip-ft})(12\text{ in./ft})}{(18.0\text{ in.})} = 168\text{ kips}$	$P_{af} = \dfrac{M_a}{d}$ $= \dfrac{(168\text{ kip-ft})(12\text{ in./ft})}{(18.0\text{ in.})} = 112\text{ kips}$

Manual Part 12

Determine the required weld size and length for fillet welds to beam flange. Try a 5⁄16-in. fillet weld. The minimum length of weld l_{min} is as follows:

For weld compatibility, disregard the increased capacity due to perpendicular loading of the end weld.

LRFD	ASD
$l_{\min} = \dfrac{P_{uf}}{1.392D} = \dfrac{168\text{ kips}}{1.392(5)} = 24.1\text{ in.}$ Use 9 in. of weld along each side and 6¼ in. of weld along the end of the flange plate. $l = 2(9.00\text{ in.}) + 6.25\text{ in.}$ $= 24.3\text{ in.} > 24.1\text{ in.}$ **o.k.**	$l_{\min} = \dfrac{P_{af}}{0.928D} = \dfrac{112\text{ kips}}{0.928(5)} = 24.1\text{ in.}$ Use 9 in. of weld along each side and 6¼ in. of weld along the end of the flange plate. $l = 2(9.00\text{ in.}) + 6.25\text{ in.}$ $= 24.3\text{ in.} > 24.1\text{ in.}$ **o.k.**

Manual Part 8

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

$$t_{\min} = \frac{0.6F_{exx}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \quad \text{for } F_{exx} = 70 \text{ ksi}$$

Manual Part 9

Beam flange; t_f = 0.570 in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5 \text{ sixteenths})}{65 \text{ ksi}} = 0.238 \text{ in.} < 0.570 \text{ in.} \quad \textbf{o.k.}$$

Flange plate; t_p = 1.00 in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5 \text{ sixteenths})}{58 \text{ ksi}} = 0.266 \text{ in.} < 1.00 \text{ in.} \quad \textbf{o.k.}$$

Determine the required fillet weld size to supporting column flange

The applied tension load is perpendicular to the weld; therefore $\theta = 90°$ and $1.0 + \sin^{1.5}\theta = 1.5$.

Manual Part 8

LRFD	ASD
$D_{\min} = \dfrac{P_{uf}}{2(1.5)(1.392)l}$ $= \dfrac{160 \text{ kips}}{2(1.5)(1.392)(6.25 \text{ in.})}$ $= 6.13$ sixteenths Use 7⁄16 in. fillet welds, 7 > 6.13 **o.k.**	$D_{\min} = \dfrac{P_{af}}{2(1.5)(0.928)l}$ $= \dfrac{107 \text{ kips}}{2(1.5)(0.928)(6.25 \text{ in.})}$ $= 6.15$ sixteenths Use 7⁄16 in. fillet welds, 7 > 6.15 **o.k.**

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

$$t_{\min} = \frac{0.6F_{exx}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \quad \text{for } F_{exx} = 70 \text{ ksi}$$

Manual Part 9

Column flange; t_f = 0.780 in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(9.14 \text{ sixteenths})}{65 \text{ ksi}} = 0.435 \text{ in.} < 0.780 \text{ in.} \quad \textbf{o.k.}$$

Design bottom compression flange plate and connection

Assume a shelf dimension of ⅝ in. The plate width, then, is 7.50 in. + 0.625 in. = 8.75 in. Try a ¾ in. × 8¾ in. compression flange plate.

Assume k = 0.65 and l = 1.00 in (1 in. setback)

$$\frac{Kl}{r} = \frac{0.65(1.00\text{ in.})}{\frac{0.750\text{ in.}}{\sqrt{12}}} = 3.00 < 25$$

$$A = (8.75\text{ in.})(0.750\text{ in.}) = 6.56\text{ in.}^2$$

LRFD	ASD	
$\phi = 0.90$ $\phi R_n = \phi F_y A_g = 0.90(36\text{ ksi})(6.56\text{ in.}^2)$ $= 213\text{ kips} > 160\text{ kips}$ **o.k.**	$\Omega = 1.67$ $R_n / \Omega = \dfrac{F_y A_g}{\Omega} = \dfrac{(36\text{ ksi})(6.56\text{ in.}^2)}{1.67}$ $= 141\text{ kips} > 107\text{ kips}$ **o.k.**	Eqn. J4-6

Determine the required weld size and length for fillet welds to beam flange

Based upon the weld length required for the tension flange plate, use 5⁄16 in. fillet weld and 12½ in. of weld along each side of the beam flange.

Note: Tension due to load reversal must also be considered in the design of the fillet weld to the supporting column flange.

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

$$t_{\min} = \frac{0.6F_{exx}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \quad \text{for } F_{exx} = 70\text{ ksi}$$

Manual Part 9

Beam flange; $t_f = 0.570$ in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5\text{ sixteenths})}{65\text{ ksi}} = 0.238\text{ in.} < 0.570\text{ in.} \quad \textbf{o.k.}$$

Bottom flange plate; $t_p = 0.750$ in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5\text{ sixteenths})}{58\text{ ksi}} = 0.266\text{ in.} < 0.750\text{ in.} \quad \textbf{o.k.}$$

See example II.B-1 for checks of the column under concentric forces. For further information, see AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections – Wind and Seismic Applications*. (Carter, 1999).

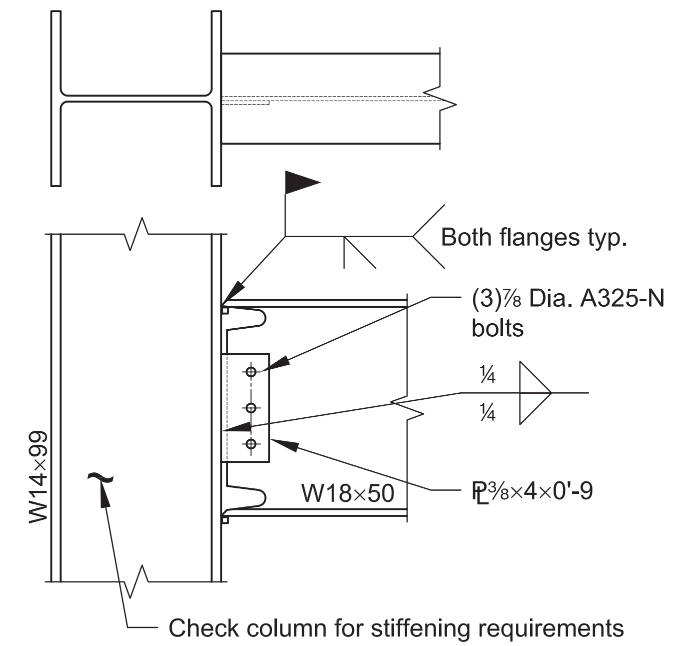
Example II.B-3 Directly-Welded Flange FR Moment Connection (beam-to-column flange).

Given:

Design a directly welded flange FR moment connection between a W18×50 beam and a W14×99 column flange to transfer the following forces:

R_D = 7.0 kips M_D = 42 kip-ft
R_L = 21 kips M_L = 126 kip-ft

Use ⅞-in. diameter ASTM A325-N or F1852-N bolts in standard holes and E70 electrodes.



Material Properties:

Beam	W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column	W14×99	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Table 2-3
Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	Table 2-4

Geometric Properties:

Beam	W18×50	d = 18.0 in.	b_f = 7.50 in.	t_f = 0.570 in.	t_w = 0.355 in.	Z_x = 101 in.^{3}	Manual
Column	W14×99	d = 14.2 in.	b_f = 14.6 in.	t_f = 0.780 in.	t_w = 0.485 in.	k_{des} = 1.38 in.	Table 1-1

Solution:

LRFD	ASD
Calculate required strength R_u = 1.2(7.0 kips) + 1.6(21 kips) = 42.0 kips M_u = 1.2(42 kip-ft) + 1.6(126 kip-ft) = 252 kip-ft	*Calculate required strength* R_a = 7.0 kips + 21 kips = 28.0 kips M_a = 42 kip-ft + 126 kip-ft = 168 kip-ft

Design single-plate web connection
Note: The available flexural strength of the beam may be less than determined based on Eqn. F13-1. All other applicable provisions in Section F should be checked to determine the governing value for the available flexural strength of the beam.

Design single-plate web connection

Try a PL ⅜×4×0'-9, with three ⅞-in. diameter ASTM A325-N bolts and ¼-in. fillet welds.

LRFD	ASD	
Design shear strength of the bolts Single shear; ϕr_n = 21.6 kips/bolt = 42 kips/(21.6 kips/bolt) = 1.94 bolts	*Allowable shear strength of bolts* Single shear; r_n/Ω = 14.4 kips/bolt = 28 kips/(14.4 kips/bolt) = 1.94 bolts	Manual Table 7-1
Bearing strength of bolts Bearing on the plate controls over bearing on the beam web. Edge distance = 1.50 in.	*Bearing strength of bolts* Bearing on the plate controls over bearing on the beam web. Edge distance = 1.50 in.	Manual Table 7-5 Eqn. J3-6a
Bolt spacing = 3.00 in. ϕr_n = (91.4 kips/in./bolt)(0.375 in.) = 34.3 kips/bolt The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is:	Bolt spacing = 3.00 in. r_n/Ω = (60.9 kips/in./bolt)(0.375 in.) = 22.8 kips/bolt The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is: $\frac{R_n}{\Omega} = \frac{nr_n}{\Omega} = 1(13.5\text{ kips/bolt}) + 2(14.4\text{ kips/bolt})$ = 42.3 kips > 28.0 kips **o**.k. *Plate shear yielding*	

$\phi R_n = n\phi r_n = 1(20.2\text{ kips/bolt}) + 2(21.6\text{ kips/bolt})$ $= 63.4\text{ kips} > 42.0\text{ kips}$ **o**.k. *Plate shear yielding* $\phi = 1.00$ $\phi R_n = 0.60\,\phi F_y A_g$ = 0.60(1.00)(36.0 ksi)(9.00 in.)(0.375 in.) = 72.9 kips > 42.0 kips **o.k.** *Plate shear rupture* Where $\phi = 0.75$ $\phi R_n = 0.60\,\phi F_u A_{nv}$ (3 bolts)(0.875 in. + 0.0625 in. + 0.0625in.) = 3.00 in. A_{nv} = (9.00 in. – 3.00 in.)(0.375 in.) = 2.25 in^2 = 0.60(0.75)(58.0 ksi)(2.25 in^2) = 58.7 kips > 42.0 kips **o.k.** $\phi r_n = \phi 1.2 L_c t F_u \le \phi 2.4 d_t F_u$ $= 0.75(1.2)\left(1.50\text{ in.-}\frac{0.938\text{ in.}}{2}\right)(0.375\text{ in.})(58.$ $\le 0.75(2.4)(0.875\text{ in.})(0.375\text{ in.})(58.0\text{ ksi})$ $= 20.2\text{ kips} \le 34.3\text{ kips}$	$\Omega = 1.50$ $R_n/\Omega = 0.60\,F_y A_g/\Omega =$ $= \frac{0.60(36.0\text{ ksi})(9.00\text{ in.})(0.375\text{ in.})}{1.50}$ = 48.6 kips > 28.0 kips **o.k.** *Plate shear rupture* $\Omega = 2.00$ $R_n/\Omega = 0.60\,F_u A_{nv}/\Omega =$ (3 bolts)(0.875 in. + 0.0625 in. + 0.0625 in.) = 3.00 in. A_{nv} = (9.00 in. – 3.00 in.)(0.375 in.) = 2.25 in^2 = 0.60(58.0 ksi)(2.25 in^2)/(2.00) = 39.2 kips > 28.0 kips **o.k.** $\frac{r_n}{\Omega} = \frac{1.2 L_c t F_u}{\Omega} \le \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2\left(1.50\text{ in.-}\frac{0.938\text{ in.}}{2}\right)(0.375\text{in.})(58.0\text{ ksi})}{2.00}$ $\le \frac{2.4(0.875\text{ in.})(0.375\text{ in.})(58.0\text{ ksi})}{2.00}$ $= 13.5\text{ kips} \le 22.8\text{ kips}$	Eqn J4-3 Eqn J4-4

Block shear rupture strength of the plate
L_{eh} = 1¼ in.; L_{ev} = 1½ in.; U_{bs} = 1.0; n = 3

LRFD	ASD	
Tension rupture component $\phi F_u A_{nt}$ = 32.6 kips/in.(0.375 in.) Shear yielding component	Tension rupture component $F_u A_{nt}/\,\Omega$ = 21.8 kips/in.(0.375 in.) Shear yielding component	Eqn.J4-5 Manual Table 9-3a

$\phi 0.6F_yA_{gv}$ = 121 kips/in.(0.375 in.) Shear rupture component $\phi 0.6F_uA_{nv}$ = 131 kips/in.(0.375 in.) ϕR_n = (121 kips/in. + 32.6 kips/in.)(0.375 in.) = 57.6 kips > 42 kips **o.k.** $\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$0.6F_yA_{gv} / \Omega$ = 81.0 kips/in.(0.375 in.) Shear rupture component $0.6F_uA_{nv} / \Omega$ = 87.0 kips/in.(0.375 in.) R_n/Ω = (81.0 kips/in. + 21.8 kips/in.)(0.375 in.) = 38.6 kips > 24.0 kips **o.k.** $\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Manual Table 9-3b Manual Table 9-3c
Weld Strength $\phi R_n = 1.392D\, l\,(2)$ = 1.392(4 sixteenths)(9.00 in.)(2) = 100 kips > 42.0 kips **o.k.**	*Weld Strength* $R_n / \Omega = 0.928Dl\ (2)$ = 0.928(4 sixteenths)(9.00 in.)(2) = 66.8 kips > 28.0 kips **o.k.**	Manual Part 8

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

for F_{exx} = 70.0 ksi

Column flange; t_f = 0.780 in.

Section J4.2
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$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(4\text{ sixteenths})}{65.0\text{ ksi}} = 0.190\text{ in.} < 0.780\text{ in. } \textbf{o}\text{.k.}$$

Note: By inspection, the available shear yielding and shear rupture strength of the beam web is o.k.

$$t_{\min} = \frac{0.6F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u}$$

Try a PL ⅜×4×0'-9, with three ⅞-in. diameter ASTM A325-N bolts and ¼-in. fillet welds.

LRFD	ASD	
Design shear strength of the bolts Single shear; ϕr_n = 21.6 kips/bolt *Bearing strength of bolts* Bearing on the plate controls over bearing on the beam web.	*Allowable shear strength of bolts* Single shear; r_n/Ω = 14.4 kips/bolt *Bearing strength of bolts* Bearing on the plate controls over bearing on the beam web.	Manual Table 7-1

LRFD	ASD	
Edge distance = 1.50 in. $L_c = 1.50 \text{ in.} - \dfrac{0.938 \text{ in.}}{2} = 1.03 \text{ in.}$ $\phi = 0.75$ $\phi r_n = \phi 1.2 L_c t F_u \le \phi 2.4 d t F_u$ $0.75(1.2)(1.03 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi}) \le 0.75(2.4)(0.875 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})$ $20.2 \text{ kips} \le 34.3 \text{ kips}$ $\phi r_n = 20.2 \text{ kips}$	Edge distance = 1.50 in. $L_c = 1.50 \text{ in.} - \dfrac{0.938 \text{ in.}}{2} = 1.03 \text{ in.}$ $\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{1.2 L_c t F_u}{\Omega} \le \dfrac{2.4 d t F_u}{\Omega}$ $\dfrac{1.2(1.03 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})}{2.00} \le \dfrac{2.4(0.875 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})}{2.00}$ $13.5 \text{ kips} \le 22.8 \text{ kips}$ $r_n / \Omega = 13.5 \text{ kips}$	Eqn. J3-6a
Bolt spacing = 3.00 in. ϕr_n = (91.4 kips/in./bolt)(0.375 in.) = 34.3 kips/bolt	Bolt spacing = 3.00 in. r_n / Ω = (60.9 kips/in./bolt)(0.375 in.) = 22.8 kips/bolt	Manual Table 7-5
The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is: $\phi R_n = n \phi r_n$ $= 1(20.2 \text{ kips/bolt}) + 2(21.6 \text{ kips/bolt})$ $= 63.4 \text{ kips} > 42.0 \text{ kips}$ **o.k.**	The strength of each bolt is the lesser of the available shear and bearing strength. The total available strength for the bolt group considering shear and bearing is: $\dfrac{R_n}{\Omega} = \dfrac{n r_n}{\Omega}$ $= 1(13.5 \text{ kips/bolt}) + 2(14.4 \text{ kips/bolt})$ $= 42.3 \text{ kips} > 28.0 \text{ kips}$ **o.k.**	
Plate shear yielding $\phi = 1.00$ $\phi R_n = 0.60 \phi F_y A_g$ = 0.60(1.00)(36 ksi)(9.00 in.)(0.375 in.) = 72.9 kips > 42.0 kips **o.k.**	*Plate shear yielding* $\Omega = 1.50$ $R_n / \Omega = 0.60 F_y A_g / \Omega$ $= \dfrac{0.60(36 \text{ ksi})(9.00 \text{ in.})(0.375 \text{ in.})}{1.50}$ = 48.6 kips > 28.0 kips **o.k.**	Eqn J4-3
Plate shear rupture Total length of bolt holes (3 bolts)(0.875 in. + 0.0625 in. + 0.0625in.) = 3.00 in. A_{nv} = (9.00 in. -3.00 in.)(0.375 in.) = 2.25 in.^{2} $\phi = 0.75$ $\phi R_n = 0.60 \phi F_u A_{nv}$	*Plate shear rupture* Total length of bolt holes (3 bolts)(0.875 in. + 0.0625 in. + 0.0625in.) = 3.00 in. A_{nv} = (9.00 in. –3.00 in.)(0.375 in.) = 2.25 in.^{2} $\Omega = 2.00$ $R_n / \Omega = 0.60 F_u A_{nv} / \Omega$	

LRFD	ASD	
$= 0.60(0.75)(58 \text{ ksi})(2.25 \text{ in.}^2)$ $= 58.7 \text{ kips} > 42.0 \text{ kips}$ **o.k.**	$= 0.60(58 \text{ ksi})(2.25 \text{ in.}^2)/(2.00)$ $= 39.2 \text{ kips} > 28.0 \text{ kips}$ **o.k.**	Eqn J4-4

Block shear rupture strength of the plate
L_{eh} = 1¼ in.; L_{ev} = 1½ in.; U_{bs} = 1.0; n = 3

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega},\ \frac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn.J4-5
Tension rupture component $\phi F_u A_{nt}$ = 32.6 kips/in.(0.375 in.)	Tension rupture component $F_u A_{nt} / \Omega$ = 21.8 kips/in.(0.375 in.)	Manual Table 9-3a
Shear yielding component $\phi 0.6 F_y A_{gv}$ = 121 kips/in.(0.375 in.)	Shear yielding component $0.6 F_y A_{gv} / \Omega$ = 81.0 kips/in.(0.375 in.)	Manual Table 9-3b
Shear rupture component $\phi 0.6 F_u A_{nv}$ = 131 kips/in.(0.375 in.)	Shear rupture component $0.6 F_u A_{nv} / \Omega$ = 87.0 kips/in.(0.375 in.)	Manual Table 9-3c
ϕR_n = (121 kips/in. + 32.6 kips/in.)(0.375 in.) = 57.6 kips > 42 kips **o.k.**	R_n / Ω = (81.0 kips/in. + 21.8 kips/in.)(0.375 in.) = 38.6 kips > 24.0 kips **o.k.**	
Weld Strength $\phi R_n = 1.392 D\, l\, (2)$ = 1.392(4 sixteenths)(9.00 in.)(2) = 100 kips > 42.0 kips **o.k.**	*Weld Strength* $R_n / \Omega = 0.928 D l\, (2)$ = 0.928(4 sixteenths)(9.00 in.)(2) = 66.8 kips > 28.0 kips **o.k.**	Manual Part 8

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

$$t_{\min} = \frac{0.6 F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6 F_u} = \frac{3.09 D}{F_u} \quad \text{for } F_{exx} = 70 \text{ ksi}$$

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Column flange; t_f = 0.780 in.

$$t_{\min} = \frac{3.09 D}{F_u} = \frac{(3.09)(4 \text{ sixteenths})}{65 \text{ ksi}} = 0.190 \text{ in.} < 0.780 \text{ in.} \qquad \textbf{o.k.}$$

Note: By inspection, the available shear yielding and shear rupture strength of the beam web is o.k.

Weld of beam flange to column

A complete-joint penetration groove weld will transfer the entire flange force in tension and compression. Table J2.5

Note: See Example II.B-1 for checks of the column under concentric forces. For further information, see AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections – Wind and Seismic Applications*. (Carter, 1999).

Example II.B-4 Four-Bolt Unstiffened Extended End-Plate FR Moment Connection (beam-to-column flange).

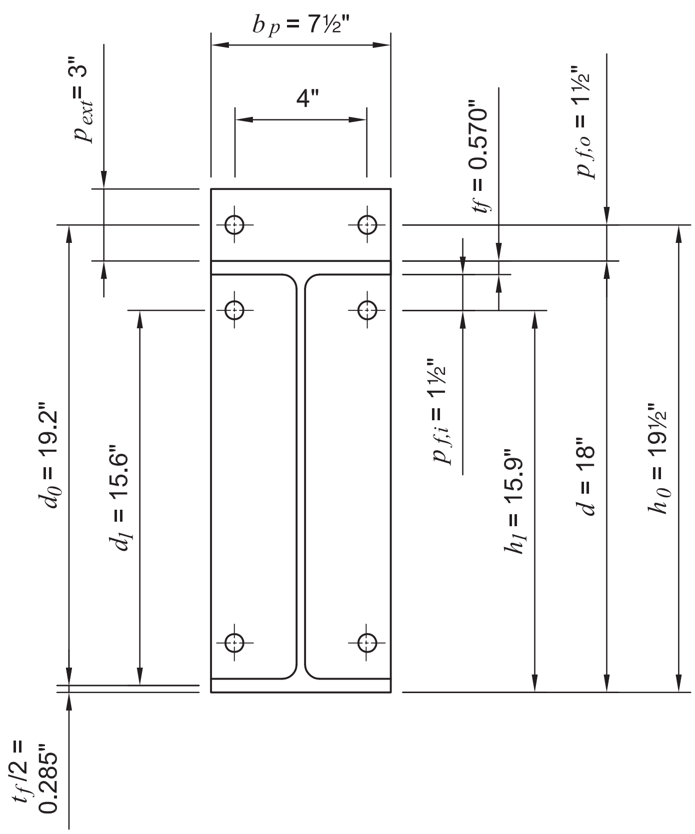
Given:

Design a four-bolt unstiffened extended end-plate FR moment connection between a W18×50 beam and a W14×99 column-flange to transfer the following forces:

R_D = 7 kips	M_D = 42 kip-ft
R_L = 21 kips	M_L = 126 kip-ft

Use ASTM A325-N snug-tight bolts in standard holes and E70 electrodes.

a. Use design procedure 1 (thick end-plate and smaller diameter bolts) from AISC Steel Design Guide 16 *Flush and Extended Multiple-Row Moment End-Plate Connections*.
b. Use design procedure 2 (thin end-plate and larger diameter bolts) from AISC Steel Design Guide 16 *Flush and Extended Multiple-Row Moment End-Plate Connections*.



Material Properties:

Beam W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual
Column W14×99	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Table 2-3
Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	Table 2-4

Geometric Properties:

Beam W18×50	d = 18.0 in.	b_f = 7.50 in.	t_f = 0.570 in.	t_w = 0.355 in.	S_x = 88.9 in.^{3}	Manual
Column W14×99	d = 14.2 in.	b_f = 14.6 in.	t_f = 0.780 in.	t_w = 0.485 in.	k_{des} = 1.38 in.	Table 1-1

Solution A:

LRFD	ASD
R_u = 1.2(7.0 kips) + 1.6(21 kips) = 42.0 kips M_u = 1.2(42 kip-ft) + 1.6(126 kip-ft) = 252 kip-ft	R_a = 7.0 kips + 21 kips = 28.0 kips M_a = 42 kip-ft + 126 kip-ft = 168 kip-ft

Extended end-plate geometric properties:

b_p = 7½ in. g = 4 in. $p_{f,i}$ = 1½ in. $p_{f,o}$ = 1½ in. p_{ext} = 3 in.

Calculate secondary dimensions

$$h_0 = d + p_{f,o} = 18.0 \text{ in.} + 1.50 \text{ in.} = 19.5 \text{ in.}$$

$$d_o = h_0 - \frac{t_f}{2} = 19.5 \text{ in.} - \frac{0.570 \text{ in.}}{2} = 19.2 \text{ in.}$$

$$h_1 = d - p_{f,i} - t_f = 18.0 \text{ in.} - 1.50 \text{ in.} - 0.570 \text{ in.} = 15.9 \text{ in.}$$

$$d_1 = h_1 - \frac{t_f}{2} = 15.9 \text{ in.} - \frac{0.570 \text{ in.}}{2} = 15.6 \text{ in.}$$

$\gamma_r = 1.0$ for extended end-plates

Determine the required bolt diameter assuming no prying action

For ASTM A325-N bolts, $F_{nt} = 90$ ksi Table J3.2

LRFD	ASD
$d_{breq} = \sqrt{\dfrac{2M_u}{\pi\phi F_{nt}\left(\sum d_n\right)}}$ $= \sqrt{\dfrac{2(252 \text{ kip-ft})(12 \text{ in./ft})}{\pi(0.75)(90 \text{ ksi})(19.2 \text{ in.} + 15.6 \text{ in.})}}$ $= 0.905$ in. Use 1-in. diameter ASTM A325-N snug-tightened bolts.	$d_{breq} = \sqrt{\dfrac{2M_a\Omega}{\pi F_{nt}\left(\sum d_n\right)}}$ $= \sqrt{\dfrac{2(168 \text{ kip-ft})(12 \text{ in./ft})(2.00)}{\pi(90 \text{ ksi})(19.2 \text{ in.} + 15.6 \text{ in.})}}$ $= 0.905$ in. Use 1-in. diameter ASTM A325-N snug-tightened bolts.

Determine the required end-plate thickness

$$s = \frac{\sqrt{b_p g}}{2} = \frac{\sqrt{(7.50 \text{ in.})(4.00 \text{ in.})}}{2} = 2.74 \text{ in.}$$

$p_{f,i} = 1.50 \text{ in.} \leq s = 2.74$ in. therefore use $s = 2.74$ in.

$$Y = \frac{b_p}{2}\left[h_1\left(\frac{1}{p_{f,i}} + \frac{1}{s}\right) + h_0\left(\frac{1}{p_{f,o}}\right) - \tfrac{1}{2}\right] + \frac{2}{g}\left[h_1\left(p_{f,i} + s\right)\right]$$

$$= \frac{7.50\text{ in.}}{2}\left[(15.9\text{ in.})\left(\frac{1}{1.50\text{ in.}} + \frac{1}{2.74\text{ in.}}\right) + (19.5\text{ in.})\left(\frac{1}{1.50\text{ in.}}\right) - \tfrac{1}{2}\right]$$
$$+ \frac{2}{4.00\text{ in.}}\left[(15.9\text{ in.})(1.50\text{ in.} + 2.74\text{ in.})\right]$$
$$= 142\text{ in.}$$

$$P_t = \frac{\pi d_b^{\,2} F_{nt}}{4} = \frac{\pi(1.00\text{ in.})^2(90\text{ ksi})}{4} = 70.7\text{ kips}$$

$$M_{np} = 2P_t\left(\sum d_n\right) = 2(70.7\text{ kips})(19.2\text{ in.} + 15.6\text{ in.}) = 4920\text{ kip-in.}$$

LRFD	ASD
$\phi = 0.75$ $\phi M_{np} = 0.75(4920\text{ kip-in.}) = 3690\text{ kip-in.}$ $\phi = 0.90$ $t_{p\ req'd} = \sqrt{\dfrac{1.11\gamma_r \phi M_{np}}{\phi_b F_{py} Y}}$ $= \sqrt{\dfrac{1.11(1.0)(3690\text{ kip-in.})}{(0.90)(36\text{ ksi})(142\text{ in.})}}$ $= 0.944\text{ in.}$ Use a 1-in. thick end-plate.	$\Omega = 2.00$ $M_{np}/\Omega = \dfrac{4920\text{ kip-in.}}{2.00} = 2460\text{ kip-in.}$ $\Omega = 1.67$ $t_{p\ req'd} = \sqrt{\dfrac{1.11\gamma_r\left(\dfrac{M_{np}}{\Omega}\right)}{\left(\dfrac{F_{py}}{\Omega_b}\right)Y}}$ $= \sqrt{\dfrac{1.11(1.0)(2460\text{ kip-in.})(1.67)}{(36\text{ ksi})(142\text{ in.})}}$ $= 0.944\text{ in.}$ Use a 1-in. thick end-plate.

LRFD	ASD
Calculate end-plate design strength From above, $\phi M_{np} = 3690\text{ kip-in.}$ $\phi = 0.90$ $\phi_b M_{pl} = \phi_b F_{py} t_p^{\,2} Y$ $= 0.90(36\text{ ksi})(1.00\text{ in.})^2(142\text{ in.})$ $= 4600\text{ kip-in.}$ $\phi M_n = \min\left(\phi M_{np},\ \phi_b M_{pl}\right)$ $= 3690\text{ kip-in. or } 308\text{ kip-ft}$ 308 kip-ft > 252 kip-ft **o.k.**	*Calculate end-plate allowable strength* From above, $M_{np}/\Omega = 2460\text{ kip-in.}$ $\Omega = 1.67$ $\dfrac{M_{pl}}{\Omega_b} = \dfrac{F_{py} t_p^{\,2} Y}{\Omega_b}$ $= \dfrac{(36\text{ ksi})(1.00\text{ in.})^2(133\text{ in.})}{1.67}$ $= 2870\text{ kip-in.}$ $M_n/\Omega_b = \min\left(M_{np}/\Omega_b,\ \dfrac{M_{pl}}{\Omega_b}\right)$ $= 2460\text{ kip-in. or } 205\text{ kip-ft}$ 205 kip-ft > 168 kip-ft **o.k.**

Check bolt shear and bearing

Try the minimum of four bolts at tension flange and two bolts at compression flange.

Note: Based on common practice, the compression bolts are assumed to resist all of the shear force.

LRFD	ASD	
For bolt shear $\phi R_n = n\phi r_n = (2\text{ bolts})(28.3\text{ kips/bolt})$ $= 56.6\text{ kips} > 42.0\text{ kips}$ **o.k.** For bolt bearing on endplate ($L_e \geq L_{e\,full}$) $\phi R_n = n\phi r_n = (104\text{ kip/in./bolt})(1.00\text{ in.})$ $= 104\text{ kips/bolt} > 28.3\text{ kips/bolt}$ Bolt shear governs For bolt bearing on column flange ($L_e \geq L_{e\,full}$) $\phi R_n = n\phi r_n = (117\text{ kip/in./bolt})(0.780\text{ in.})$ $= 91.3\text{ kips/bolt} > 28.3\text{ kips/bolt}$ Bolt shear governs	For bolt shear $R_n / \Omega = nr_n / \Omega = (2\text{ bolts})(18.8\text{ kips/bolt})$ $= 37.6\text{ kips} > 28.0\text{ kips}$ **o.k.** For bolt bearing on endplate ($L_e \geq L_{e\,full}$) $\dfrac{R_n}{\Omega} = \dfrac{nr_n}{\Omega} = (69.6\text{ kips/in./bolt})(1.00\text{ in.})$ $= 69.6\text{ kips/bolt} > 18.8\text{ kips/bolt}$ Bolt shear governs For bolt bearing on column flange ($L_e \geq L_{e\,full}$) $\dfrac{R_n}{\Omega} = \dfrac{nr_n}{\Omega} = (78.0\text{ kips/in./bolt})(0.780\text{ in.})$ $= 60.8\text{ kips/bolt} > 18.8\text{ kips/bolt}$ Bolt shear governs	Manual Table 7-1 Manual Table 7-6 Manual Table 7-6
Determine the required size of the beam web-to end-plate fillet weld $D_{min} = \dfrac{\phi F_y t_w}{2(1.392)} = \dfrac{0.90(50\text{ ksi})(0.355\text{ in.})}{2(1.392)}$ $= 5.74\text{ sixteenths}$	*Determine the required size of the beam web-to end-plate fillet weld* $D_{min} = \dfrac{F_y t_w}{2\Omega(0.928)} = \dfrac{(50\text{ ksi})(0.355\text{ in.})}{2(1.67)(0.928)}$ $= 5.73\text{ sixteenths}$	Manual Part 8

Use ⅜ in. fillet welds on both sides of the beam web from the inside face of the beam tension flange to the centerline of the inside bolt holes plus two bolt diameters.

Determine weld size required for the end reaction

The end reaction , R_u or R_a, is resisted by weld between the mid-depth of the beam and the inside face of the compression flange, or between the inner row of tension bolts plus two bolt diameters and the inside face of the beam compression flange, which ever is smaller. By inspection the former governs for this example.

$$l = \frac{d}{2} - t_f = \frac{18.0\text{ in.}}{2} - 0.570\text{ in.} = 8.43\text{ in.}$$

LRFD	ASD
$D_{min} = \frac{R_u}{2(1.392)l} = \frac{42.0\text{ kips}}{2(1.392)(8.43\text{ in.})}$ $= 1.79 \rightarrow 3$ sixteenths (minimum size) Use 3⁄16-in. fillet weld on both sides of the beam web below the tension-bolt region.	$D_{min} = \frac{R_a}{2(0.928)l} = \frac{28.0\text{ kips}}{2(0.928)(8.43\text{ in.})}$ $= 1.79 \rightarrow 3$ sixteenths (minimum size) Use 3⁄16-in. fillet weld on both sides of the beam web below the tension-bolt region.

Manual Part 8

Table J2.4

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

$$t_{min} = \frac{0.6F_{exx}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \qquad \text{for } F_{exx} = 70\text{ ksi}$$

Manual Part 9

Beam web $t_w = 0.355$ in.

$$t_{min} = \frac{3.09(2)D}{F_u} = \frac{(6.19)(1.79\text{ sixteenths})}{65\text{ ksi}} = 0.170\text{ in.} < 0.355\text{ in.} \quad \textbf{o.k.}$$

End plate $t_p = 1.00$ in.

$$t_{min} = \frac{3.09D}{F_u} = \frac{(3.09)(1.79\text{ sixteenths})}{58\text{ ksi}} = 0.0954\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}$$

Determine required fillet weld size for the beam flange to end-plate connection

$$l = 2(b_f) - t_w = 2(7.50\text{ in.}) - 0.355\text{ in.} = 14.6\text{ in.}$$

LRFD	ASD
$P_{uf} = \frac{2M_u}{\sum d_n} = \frac{2(252\text{ kip-ft})(12\text{ in./ft})}{19.2\text{ in.} + 15.6\text{ in.}}$ $= 174$ kips $D_{min} = \frac{P_{uf}}{1.50(1.392)l} = \frac{174\text{ kips}}{1.50(1.392)(14.6\text{ in.})}$ $= 5.71 \rightarrow 6$ sixteenths	$P_{af} = \frac{2M_a}{\sum d_n} = \frac{2(168\text{ kip-ft})(12\text{ in./ft})}{19.2\text{ in.} + 15.6\text{ in.}}$ $= 116$ kips $D_{min} = \frac{P_{af}}{1.50(0.928)l} = \frac{116\text{ kips}}{1.50(0.928)(14.6\text{ in.})}$ $= 5.71 \rightarrow 6$ sixteenths

Note that the 1.5 factor is from Specification J2.4 and accounts for the increased strngth of a transversely loaded fillet weld.

Use ⅜-in. fillet welds at beam tension flange. Welds at compression flange may be ¼-in. fillet welds (minimum size per Specification Table J2.4).

Connecting Elements Rupture Strength at Welds

Shear rupture strength of base metal

$$t_{\min} = \frac{0.6F_{exx}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \quad \text{for } F_{exx} = 70 \text{ ksi}$$

Manual
Part 9

End plate; t_p = 1.00 in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5.71 \text{ sixteenths})}{58 \text{ ksi}} = 0.304 \text{ in.} < 1.00 \text{ in.} \quad \textbf{o.k.}$$

Solution B:

Only those portions of the design that vary from the solution "A" calculations are presented here.

LRFD	ASD
Determine required end-plate thickness $\phi = 0.90$ $t_{preq} = \sqrt{\dfrac{\gamma_r M_u}{\phi_b F_{py} Y}}$ $= \sqrt{\dfrac{1.0(252\text{ kip-ft})(12\text{ in./ft})}{0.90(36\text{ ksi})(142\text{ in.})}}$ $= 0.811\text{ in.}$ Use t_p = ⅞ in.	*Determine required end-plate thickness* $\Omega = 1.67$ $t_{preq} = \sqrt{\dfrac{\gamma_r M_a \Omega_b}{F_{py} Y}}$ $= \sqrt{\dfrac{1.0(168\text{ kip-ft})(12\text{ in./ft})(1.67)}{(36\text{ ksi})(142\text{ in.})}}$ $= 0.812\text{ in.}$ Use t_p = ⅞ in.

Select a trial bolt diameter and calculate the maximum prying forces

Try 1-in. diameter bolts.

$$w' = \frac{b_p}{2} - \left(d_b + \tfrac{1}{16}\text{ in.}\right) = \frac{7.50\text{ in.}}{2} - (1.00\text{ in.} + 0.0625\text{ in.}) = 2.69\text{ in.}$$

$$a_i = 3.682\left(\frac{t_p}{d_b}\right)^3 - 0.085 = 3.682\left(\frac{0.875\text{ in.}}{1.00\text{ in.}}\right)^3 - 0.085 = 2.38\text{ in.}$$

$$F_i' = \frac{t_p^{\,2} F_{py}\left[0.85\left(\dfrac{b_p}{2}\right) + 0.80w'\right] + \dfrac{\pi d_b^{\,3} F_{nt}}{8}}{4p_{f,i}}$$

$$= \frac{(0.875)^2(36\text{ ksi})\left[0.85\left(\dfrac{7.50\text{ in.}}{2}\right) + 0.80(2.69\text{ in.})\right]}{4(1.50\text{ in.})} + \frac{\dfrac{\pi(1.00\text{ in.})^3(90\text{ ksi})}{8}}{4(1.50\text{ in.})}$$

$$= 30.4\text{ kips}$$

$$Q_{\max i} = \frac{w' t_p^{\,2}}{4a_i}\sqrt{F_{py}^{\,2} - 3\left(\frac{F_i'}{w' t_p}\right)^2}$$

$$= \frac{(2.69\text{ in.})(0.875\text{ in.})^2}{4(2.38\text{ in.})}\sqrt{(36\text{ ksi})^2 - 3\left(\frac{30.4\text{ kips}}{(2.69\text{ in.})(0.875\text{ in.})}\right)^2}$$

$$= 6.10\text{ kips}$$

$$a_o = \min\left[a_i,\ p_{ext} - p_{f,o}\right] = \min\left[2.38\text{ in., (3.00 in. - 1.50 in. = 1.50 in.)}\right] = 1.50\text{ in.}$$

$$F_o' = F_i'\left(\frac{p_{f,i}}{p_{f,o}}\right) = (30.4\text{ kips})\left(\frac{1.50\text{ in.}}{1.50\text{ in.}}\right)$$
$$= 30.4\text{ kips}$$

$$Q_{\max o} = \frac{w't_p^{\,2}}{4a_o}\sqrt{F_{py}^{\,2} - 3\left(\frac{F_o'}{w't_p}\right)^2}$$
$$= \frac{(2.69\text{ in.})(0.875\text{ in.})^2}{4(1.50\text{ in.})}\sqrt{(36\text{ ksi})^2 - 3\left(\frac{30.4\text{ kips}}{(2.69\text{ in.})(0.875\text{ in.})}\right)^2}$$
$$= 9.68\text{ kips}$$

Calculate the connection available strength for the limit state of bolt rupture with prying action

$$P_t = \frac{\pi d_b^{\,2} F_{nt}}{4} = \frac{\pi(1.00\text{ in.})^2(90\text{ ksi})}{4} = 70.7\text{ kips}$$

Table J3.1

Unmodified Bolt Pretension, $T_{b0} = 51$ kips

Modify bolt pretension for the snug-tight condition.

AISC *Design Guide 16* Table 4-1

$$T_b = \frac{T_{b0}}{4} = \frac{51\text{ kips}}{4} = 12.8\text{ kips}$$

LRFD	ASD
$\phi M_q =$ $\max\begin{Bmatrix}\phi\left[2(P_t - Q_{\max o})d_0 + 2(P_t - Q_{\max i})d_1\right]\\ \phi\left[2(P_t - Q_{\max o})d_0 + 2(T_b)d_1\right]\\ \phi\left[2(P_t - Q_{\max i})d_1 + 2(T_b)d_0\right]\\ \phi\left[2(T_b)(d_0 + d_1)\right]\end{Bmatrix}$ $=$ $\max\begin{Bmatrix}0.75\begin{bmatrix}2(70.7\text{ kips - }9.68\text{ kips})(19.2\text{ in.})\\ +2(70.7\text{ kips - }6.10\text{ kips})(15.6\text{ in.})\end{bmatrix}\\ 0.75\begin{bmatrix}2(70.7\text{ kips - }9.68\text{ kips})(19.2\text{ in.})\\ +2(12.8\text{ kips})(15.6\text{ in.})\end{bmatrix}\\ 0.75\begin{bmatrix}2(70.7\text{ kips - }6.10\text{ kips})(15.6\text{ in.})\\ +2(12.8\text{ kips})(19.2\text{ in.})\end{bmatrix}\\ 0.75\left[2(12.8\text{ kips})(19.2\text{ in.} + 15.6\text{ in.})\right]\end{Bmatrix}$	$\frac{M_q}{\Omega} =$ $\max\begin{Bmatrix}\frac{1}{\Omega}\left[(P_t - Q_{\max o})d_0 + 2(P_t - Q_{\max i})d_1\right]\\ \frac{1}{\Omega}\left[2(P_t - Q_{\max o})d_0 + 2(T_b)d_1\right]\\ \frac{1}{\Omega}\left[2(P_t - Q_{\max i})d_1 + 2(T_b)d_0\right]\\ \frac{1}{\Omega}\left[2(T_b)(d_0 + d_1)\right]\end{Bmatrix}$ $=$ $\max\begin{Bmatrix}\frac{1}{2.00}\begin{bmatrix}2(70.7\text{ kips - }9.68\text{ kips})(19.2\text{ in.})\\ +2(70.7\text{ kips - }6.10\text{ kips})(15.6\text{ in.})\end{bmatrix}\\ \frac{1}{2.00}\begin{bmatrix}2(70.7\text{ kips - }9.68\text{ kips})(19.2\text{ in.})\\ +2(12.8\text{ kips})(15.6\text{ in.})\end{bmatrix}\\ \frac{1}{2.00}\begin{bmatrix}2(70.7\text{ kips - }6.10\text{ kips})(15.6\text{ in.})\\ +2(12.8\text{ kips})(19.2\text{ in.})\end{bmatrix}\\ \frac{1}{2.00}\left[2(12.8\text{ kips})(19.2\text{ in.} + 15.6\text{ in.})\right]\end{Bmatrix}$

LRFD	ASD
$= \max\begin{Bmatrix} 3270 \text{ kip-in.} \\ 2060 \text{ kip-in.} \\ 1880 \text{ kip-in.} \\ 668 \text{ kip-in.} \end{Bmatrix} = 3270 \text{ kip-in.}$	$= \max\begin{Bmatrix} 2180 \text{ kip-in.} \\ 1370 \text{ kip-in.} \\ 1250 \text{ kip-in.} \\ 445 \text{ kip-in.} \end{Bmatrix} = 2180 \text{ kip-in.}$
$\phi M_q = 3270$ kip-in. $= 273$ kip-ft > 252 kip-ft **o.k.**	$M_q / \Omega = 2180$ kip-in. $= 182$ kip-ft > 168 kip-ft **o.k.**

For **Example IIB-4**, design procedure 1 produced a design with a 1-in. thick end-plate and 1-in. diameter bolts. Design procedure 2 produced a design with a ⅞-in. thick end-plate and 1-in. diameter bolts. Either design is acceptable. Design procedure 1 did not produce a smaller bolt diameter for this example, although in general it should result in a thicker plate and smaller diameter bolt than design procedure 2. It will be noted that the bolt stress is lower in design procedure 1 than in design procedure 2.

Chapter IIC
Bracing and Truss Connections

The design of bracing and truss connections is covered in Part 13 of the AISC *Steel Construction Manual*.

Example II.C-1 Truss Support Connection

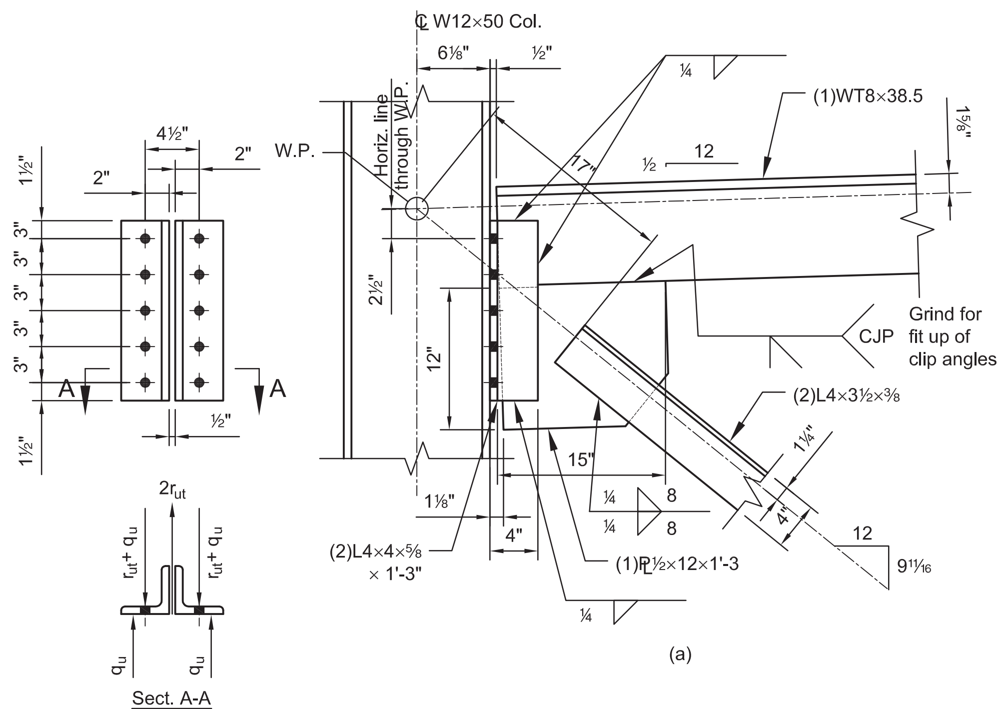
Given:

Determine:

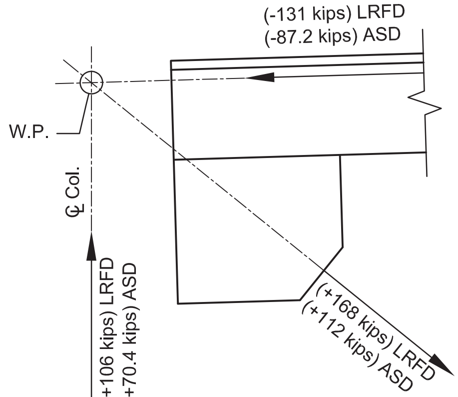
a. the connection requirements between gusset and column,
b. the required gusset size and the weld requirements connecting the diagonal to the gusset.

R_D = 16.6 kips
R_L = 53.8 kips

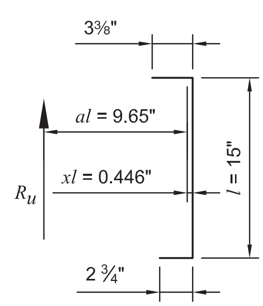
Use ⅞-in. diameter ASTM A325-N or F1852-N bolts in standard holes and E70 electrodes.



(a)



(b)



(c)

Material Properties:

Top Chord WT8×38.5	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Column W12×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Brace 2L4×3½×⅜	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	
Gusset Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	
Clip Angles 2L4×4	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Top Chord WT8×38.5	t_w = 0.455 in.				Manual Table 1-8, Table 1-15 & Table 1-1
Column W12×50	d = 12.2 in.	t_f = 0.640 in.	b_f = 8.08 in.	t_w = 0.370 in.	
Brace 2L4×3½×⅜	t = 0.375 in.	A = 5.35 in.^{2}	$\bar{x}$ = 0.947 in.		

Solution:

Calculate the required strengths

LRFD	ASD
Brace axial load $R_u = 168$ kips Truss end reaction $R_u = 1.2(16.6) + 1.6(53.8) = 106$ kips Top chord axial load $R_u = 131$ kips	Brace axial load $R_a = 112$ kips Truss end reaction $R_a = 16.6 + 53.8 = 70.4$ kips Top chord axial load $R_a = 87.2$ kips

Design the weld connecting the diagonal to the gusset

Note: Specification Section J1.7 requiring that the center of gravity of the weld group coincide with the center of gravity of the member does not apply to end connections of statically loaded single angle, double angle and similar members. — Section J1.7

For ⅜-in. angles, $D_{\min} = 3$, try ¼-in. fillet welds, $D = 4$. — Table J2.4

LRFD	ASD
$L_{req} = \dfrac{R_u}{4D(1.392)} = \dfrac{168 \text{ kips}}{4(4)(1.392)} = 7.54$ in. Use 8 in. at the heel and 8 in. at the toe of each angle.	$L_{req} = \dfrac{R_a}{4D(1.392)} = \dfrac{112 \text{ kips}}{4(4)(0.928)} = 7.54$ in. Use 8 in. at the heel and 8 in. at the toe of each angle.

Check tensile yielding of the 2L4×3½×⅜ LLBB *truss diagonal*

$$R_n = F_y A_g = (36 \text{ ksi})(5.35 \text{ in.}^2) = 193 \text{ kips}$$ Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.90(193 \text{ kips}) = 174 \text{ kips} > 168 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{193 \text{ kips}}{1.67} = 116 \text{ kips} > 112 \text{ kips}$ **o.k.**

Check tensile rupture of the truss diagonal

$A_n = A_g = 5.35 \text{ in.}^2$

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.947 \text{ in.}}{8.00 \text{ in.}} = 0.882$$

$A_e = A_nU = (5.35 \text{ in.}^2)(0.882) = 4.72 \text{ in.}^2$

$R_n = F_uA_e = (58 \text{ ksi})(4.72 \text{ in.}^2) = 274 \text{ kips}$

LRFD	ASD
$\phi R_n = 0.75(274 \text{ kips}) = 206 \text{ kips} > 168 \text{ kips}$ **o.k.**	$\frac{R_n}{\Omega} = \frac{274 \text{ kips}}{2.00} = 137 \text{ kips} > 112 \text{ kips}$ **o.k.**

Use a ½ in. gusset plate. With the diagonal to gusset welds determined, a gusset plate layout as shown in Figure (c) can be made.

LRFD	ASD
Design bolts connecting clip angles to column (shear and tension) The number of ⅞-in. diameter ASTM A325-N bolts required for shear only is as follows: $n_{min} = \frac{R_u}{\phi r_n} = \frac{106 \text{ kips}}{21.6 \text{ kips/bolt}} = 4.91$	*Design bolts connecting clip angles to column (shear and tension)* The number of ⅞-in. diameter ASTM A325-N bolts required for shear only is as follows: $n_{min} = \frac{R_u}{\phi r_n} = \frac{70.4 \text{ kips}}{14.4 \text{ kips/bolt}} = 4.89$

Manual Table 7-1

Try a clip angle thickness of ⅝ in. For a trial calculation, the number of bolts was increased to 10 in pairs at 3-in. spacing; this is done to "square off" the connection as shown

LRFD	ASD
With 10 bolts, $f_v = \frac{R_u}{nA_b} = \frac{106 \text{ kips}}{(10 \text{ bolts})(0.601 \text{ in.}^2)} = 17.6 \text{ ksi}$	With 10 bolts, $f_v = \frac{R_a}{nA_b} = \frac{70.4 \text{ kips}}{(10 \text{ bolts})(0.601 \text{ in.}^2)} = 11.7 \text{ ksi}$

LRFD	ASD
The eccentric moment at the faying surface is as follows: $M_u = R_ue = (106 \text{ kips})(6.10 \text{ in.})$ $= 647 \text{ kip-in.}$	The eccentric moment at the faying surface is as follows: $M_a = R_ae = (70.4 \text{ kips})(6.10 \text{ in.})$ $= 429 \text{ kip-in.}$

For the bolt group, the Case II approach of Manual Page 7-12, can be used. Thus, the maximum tensile force per bolt, T, is given by:

n' = number of bolts above the neutral axis = 4 bolts
d_m = moment arm between resultant tensile force and resultant compressive force = 9.00 in.

LRFD	ASD
$T_u = \dfrac{M_u}{n'd_m} = \dfrac{647 \text{ kip-in.}}{(4 \text{ bolts})(9.00 \text{ in.})}$ $= 18.0 \text{ kips/bolt}$	$T_a = \dfrac{M_a}{n'd_m} = \dfrac{429 \text{ kip-in.}}{(4 \text{ bolts})(9.00 \text{ in.})}$ $= 11.9 \text{ kips/bolt}$

LRFD	ASD
Check design tensile strength of bolts $F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}}{0.75(48 \text{ ksi})}(17.6 \text{ ksi})$ $= 73.0 \text{ ksi} < 90 \text{ ksi}$ **o.k.** $B = \phi F'_{nt} A_b = 0.75(73.0 \text{ ksi})(0.601 \text{ in.}^2)$ $= 32.9 \text{ kips} > 18.0 \text{ kips}$ **o.k.**	*Check design tensile strength of bolts* $F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \dfrac{(2.00)(90 \text{ ksi})}{48 \text{ ksi}}(11.7 \text{ ksi})$ $= 73.1 \text{ ksi} < 90 \text{ ksi}$ **o.k.** $B = \dfrac{F'_{nt}}{\Omega} A_b = \dfrac{73.1 \text{ ksi}}{2.00}(0.601 \text{ in.}^2)$ $= 22.0 \text{ kips} > 11.7 \text{ kips}$ **o.k.**

Eqn. J3-3a (LRFD) and J3-3b (ASD)

Table J3.2

Check the clip angles

Check prying action.

$p = 3.00$ in.

Manual Part 9

$$b = 2.00 \text{ in.} - \frac{0.625 \text{ in.}}{2} = 1.69 \text{ in.}$$

Note: 1⅜ in. entering and tightening clearance accommodated and the column fillet toe is cleared, **o.k.**

Manual Table 7-16

$$a = \frac{8.08 \text{ in.} - 4.50 \text{ in.}}{2} = 1.79 \text{ in.}$$

Note: “a” was calculated based on the column flange width in this case.

$$b' = b - \frac{d_b}{2} = 1.69 \text{ in.} - \frac{0.875 \text{ in.}}{2} = 1.25 \text{ in.}$$

$$a' = a + \frac{d_b}{2} \le \left(1.25b + \frac{d_b}{2}\right) = 1.79 \text{ in.} + \frac{0.875 \text{ in.}}{2} \le \left(1.25(1.69 \text{ in.}) + \frac{0.875 \text{ in.}}{2}\right)$$
$$= 2.23 \text{ in.} \le 2.55 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.25 \text{ in.}}{2.23 \text{ in.}} = 0.561$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{0.938 \text{ in.}}{3.00 \text{ in.}} = 0.687$$

LRFD	ASD
$t_c = \sqrt{\dfrac{4.44Bb'}{pF_u}} = \sqrt{\dfrac{4.44(32.9 \text{ kips})(1.25 \text{ in.})}{(3.00 \text{ in.})(58 \text{ ksi})}}$ $= 1.02 \text{ in.}$	$t_c = \sqrt{\dfrac{6.66Bb'}{pF_u}} = \sqrt{\dfrac{6.66(22.0 \text{ kips})(1.25 \text{ in.})}{(3.00 \text{ in.})(58 \text{ ksi})}}$ $= 1.03 \text{ in.}$ Use 1.03

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right]$$
$$= \frac{1}{0.687(1+0.561)}\left[\left(\frac{1.03 \text{ in.}}{0.625 \text{ in.}}\right)^2 - 1\right]$$
$$= 1.60$$

Since $\alpha' > 1$,

$$Q = \left(\frac{t}{t_L}\right)^2(1+\delta) = \left(\frac{0.625 \text{ in.}}{1.03 \text{ in.}}\right)^2(1+0.687) = 0.621$$

LRFD	ASD
$T_{avail} = BQ$ $= (32.9 \text{ kips})(0.621)$ $= 20.4 \text{ kips} > 18.0 \text{ kips}$ **o.k.**	$T_{avail} = BQ$ $= (22.0 \text{ kips})(0.621)$ $= 13.7 \text{ kips} > 11.7 \text{ kips}$ **o.k.**
Check shear yielding of the clip angles $\phi R_n = \phi(0.6F_y)A_g$ $= 1.0(0.6)(36 \text{ ksi})[2(15.0 \text{ in.})(0.625 \text{ in.})]$ $= 405 \text{ kips} > 106 \text{ kips}$ **o.k.**	*Check shear yielding of the clip angles* $R_n / \Omega = \dfrac{(0.6F_y)A_g}{\Omega}$ $= \dfrac{(0.6)(36 \text{ ksi})[2(15.0 \text{ in.})(0.625 \text{ in.})]}{1.50}$ $= 270 \text{ kips} > 70.4 \text{ kips}$ **o.k.**
Check shear rupture of the angles $A_{nv} = 2[15.0 \text{ in.} - 5(1.00 \text{ in.})](0.625 \text{ in.})$ $= 12.5 \text{ in.}^2$ $\phi R_n = \phi(0.6F_u)A_{nv}$ $= 0.75(0.6)(58 \text{ ksi})(12.5 \text{ in.}^2)$ $= 326 \text{ kips} > 106 \text{ kips}$ **o.k.**	*Check shear rupture of the angles* $A_{nv} = 2[15.0 \text{ in.} - 5(1.00 \text{ in.})](0.625 \text{ in.})$ $= 12.5 \text{ in.}^2$ $R_n / \Omega = \dfrac{(0.6F_u)A_{nv}}{\Omega}$ $= \dfrac{(0.6)(58 \text{ ksi})(12.5 \text{ in.}^2)}{2.00}$ $= 218 \text{ kips} > 70.4 \text{ kips}$ **o.k.**

Eqn. J4-3

Eqn. J4-4

Check block shear rupture of the clip angles

Section J4.3

Assume uniform tension stress, use U_{bs} = 1.0.

Gross area subject to shear, $A_{gv} = 2(15.0 \text{ in.} - 1.50 \text{ in.})(0.625 \text{ in.}) = 16.9 \text{ in.}^2$

Net area subject to shear, $A_{nv} = 16.9 \text{ in.}^2 - 2[4.50(1.00 \text{ in.})(0.625 \text{ in.})] = 11.3 \text{ in.}^2$

Net area subject to tension, $A_{nt} = 2\left[(2.00\text{ in.})(0.625\text{ in.}) - 0.5(1.00\text{ in.})(0.625\text{ in.})\right] = 1.88\text{ in.}^2$

LRFD	ASD	
$\phi R_n = \phi\left[U_{bs}F_uA_{nt} + \min\left\{0.6F_yA_{gv}, 0.6F_uA_{nv}\right\}\right]$ $= 0.75\begin{bmatrix}1.0(58\text{ ksi})(1.88\text{ in.}^2)+\\ \min\begin{Bmatrix}0.6(36\text{ ksi})(16.9\text{ in.}^2),\\ 0.6(58\text{ ksi})(11.3\text{ in.}^2)\end{Bmatrix}\end{bmatrix}$ $= 356\text{ kips} > 106\text{ kips}$ **o.k.**	$R_n/\Omega = \dfrac{\left[U_{bs}F_uA_{nt} + \min\left\{0.6F_yA_{gv}, 0.6F_uA_{nv}\right\}\right]}{\Omega}$ $= \dfrac{\begin{bmatrix}1.0(58\text{ ksi})(1.88\text{ in.}^2)+\\ \min\begin{Bmatrix}0.6(36\text{ ksi})(16.9\text{ in.}^2),\\ 0.6(58\text{ ksi})(11.3\text{ in.}^2)\end{Bmatrix}\end{bmatrix}}{2.00}$ $= 237\text{ kips} > 70.4\text{ kips}$ **o.k.**	Eqn. J4-5

Check bearing and tearout on the clip angles

The clear edge distance, L_c, for the top bolts is $L_c = L_e - \dfrac{d'}{2}$, where L_e is the distance to the center of the hole. Thus,

$$L_c = 1.50\text{ in.} - \frac{0.938\text{ in.}}{2} = 1.03\text{ in.}$$

The bearing/ tearout capacity of the top bolt is as follows:

LRFD	ASD	
$\phi r_n = \phi 1.2L_ctF_u \le \phi 2.4dtF_u$ $= 0.75(1.2)(1.03\text{ in.})(0.625\text{ in.})(58\text{ ksi})$ $\le 0.75(2.4)(0.875\text{ in.})(0.625\text{ in.})(58\text{ ksi})$ $= 33.6\text{ kips} \le 57.1\text{ kips}$ 33.6 kips/bolt > 21.6 kips/bolt **Bolt shear controls**	$r_n/\Omega = \dfrac{1.2L_ctF}{\Omega} \le \dfrac{2.4dtF_u}{\Omega}$ $= \dfrac{1.2(1.03\text{ in.})(0.625\text{ in.})(58\text{ ksi})}{2.00}$ $\le \dfrac{2.4(0.875\text{ in.})(0.625\text{ in.})(58\text{ ksi})}{2.00}$ $= 22.4\text{ kips} \le 38.1\text{ kips}$ 22.4 kips/bolt >14.4 kips/bolt **Bolt shear controls**	Eqn. J3-6a

The bearing/ tearout capacity of each of the remaining bolts is as follows:
$L_c = 3.00\text{ in.} - 1(0.938\text{ in.}) = 2.06\text{ in.}$

LRFD	ASD	
$\phi r_n = \phi 1.2L_ctF_u \le \phi 2.4dtF_u$ $= 0.75(1.2)(2.06\text{ in.})(0.625\text{ in.})(58\text{ ksi})$ $\le 0.75(2.4)(0.875\text{ in.})(0.625\text{ in.})(58\text{ ksi})$ $= 67.2\text{ kips} \le 57.1\text{ kips}$ 57.1 kips/bolt > 21.6 kips/bolt **Bolt shear controls**	$r_n/\Omega = \dfrac{1.2L_ctF}{\Omega} \le \dfrac{2.4dtF_u}{\Omega}$ $= \dfrac{1.2(2.06\text{ in.})(0.625\text{ in.})(58\text{ ksi})}{2.00}$ $\le \dfrac{2.4(0.875\text{ in.})(0.625\text{ in.})(58\text{ ksi})}{2.00}$ $= 44.8\text{ kips} \le 38.1\text{ kips}$ 38.1 kips/bolt > 14.4 kips/bolt **Bolt shear controls**	Eqn. J3-6a

Use 2L4x4x⅝

Check the column flange for prying action

$p = 3.00$ in.

$$b = \frac{4.50 \text{ in.} - 0.370 \text{ in.}}{2} = 2.07 \text{ in.}$$

Manual Part 9

$$a = \frac{8.08 \text{ in.} - 4.50 \text{ in.}}{2} = 1.79 \text{ in.}$$

$$b' = b - \frac{d_b}{2} = 2.07 \text{ in.} - \frac{0.875 \text{ in.}}{2} = 1.63 \text{ in.}$$

$$a' = \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) = 1.79 \text{ in.} + \frac{0.875 \text{ in.}}{2} \le 1.25\left(2.07 \text{ in.}\right) + \frac{0.875 \text{ in.}}{2}$$
$$= 2.23 \text{ in.} \le 3.03 \text{ in.}$$

$$p = \frac{b'}{a'} = \frac{1.63 \text{ in.}}{2.23 \text{ in.}} = 0.731$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{0.938 \text{ in.}}{3.00 \text{ in.}} = 0.687$$

LRFD	ASD
$t_c = \sqrt{\frac{4.44Bb'}{pF_u}}$ $= \sqrt{\frac{4.44(32.9 \text{ kips})(1.63 \text{ in.})}{(3.00 \text{ in.})(65 \text{ ksi})}} = 1.11 \text{ in.}$ $a' = \frac{1}{\delta(1+p)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right]$ $= \frac{1}{0.687\left(1+0.731\right)}\left[\left(\frac{1.11 \text{ in.}}{0.640 \text{ in.}}\right)^2 - 1\right] = 1.69$ Since $\alpha' > 1$ $Q = \left(\frac{t}{t_c}\right)^2\left(1+\delta\right)$ $= \left(\frac{0.640 \text{ in.}}{1.11 \text{ in.}}\right)^2\left(1+0.687\right) = 0.561$ $T_{avail} = BQ$	$t_c = \sqrt{\frac{6.66Bb'}{pF_u}}$ $= \sqrt{\frac{6.66(22.0 \text{ kips})(1.63 \text{ in.})}{(3.00 \text{ in.})(65 \text{ ksi})}} = 1.11 \text{ in.}$ $a' = \frac{1}{\delta(1+p)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right]$ $= \frac{1}{0.687(1+0.731)}\left[\left(\frac{1.11 \text{ in.}}{0.640 \text{ in.}}\right)^2 - 1\right] = 1.69$ Since $\alpha' > 1$ $Q = \left(\frac{t}{t_c}\right)^2\left(1+\delta\right)$ $= \left(\frac{0.640 \text{ in.}}{1.11 \text{ in.}}\right)^2\left(1+0.687\right) = 0.561$

LRFD	ASD
$= (32.9 \text{ kips})(0.561)$ $= 18.4 \text{ kips} > 18.0 \text{ kips}$ **o.k.** *Check the column flange for bearing/tearout.* By inspection, these limit states will not control, since the column flange is thicker than the plate and the column material strength is greater than the plate material strength	$T_{avail} = BQ$ $= (22.0 \text{ kips})(0.561)$ $= 12.3 \text{ kips} > 11.7 \text{ kips}$ **o.k.** *Check the column flange bearing/tearout.* By inspection, these limit states will not control, since the column flange is thicker than the plate and the column material strength is greater than the plate material strength

Design clip angle-to-gusset connection

The minimum weld size is 3⁄16 in. with top chord slope being ½ on 12, the horizontal welds are as shown due to the square cut end. Use the average length. Then, Table J2.4

$l = 15.0 \text{ in.}$

Manual Table 8-8

$$kl = \frac{3.38 \text{ in.} + 2.75 \text{ in.}}{2} = 3.07 \text{ in.}$$

$$k = \frac{kl}{l} = \frac{3.07 \text{ in.}}{15.0 \text{ in.}} = 0.205$$

$$xl = \frac{(3.07 \text{ in.})^2}{2(3.07 \text{ in.}) + 15.0 \text{ in.}} = 0.446 \text{ in.}$$

$$al + xl = 6.10 \text{ in.} + 4.00 \text{ in.} = 10.1 \text{ in.}$$

$$al = 10.1 \text{ in.} - xl = 10.1 \text{ in.} - 0.446 \text{ in.} = 9.65 \text{ in.}$$

$$a = \frac{al}{l} = \frac{9.65 \text{ in.}}{15.0 \text{ in.}} = 0.643$$

By interpolation, with $\theta = 0°$ $C = 1.50$

LRFD	ASD
$D_{req} = \dfrac{R_u}{2(\phi C C_1 l)}$ $= \dfrac{106 \text{ kips}}{2(0.75)(1.50)(1.0)(15.0 \text{ in.})}$ $= 3.14 \rightarrow 4 \text{ sixteenths}$ Use ¼-in. fillet welds.	$D_{req} = \dfrac{\Omega R_a}{2(C C_1 l)}$ $= \dfrac{(2.0)(70.4 \text{ kips})}{2(1.50)(1.0)(15.0 \text{ in.})}$ $= 3.13 \rightarrow 4 \text{ sixteenths}$ Use ¼-in. fillet welds.

Note: Using the average of the horizontal weld lengths provides a reasonable solution when the horizontal welds are close in length. A conservative solution can be determined by using the smallers of the horizontal weld lengths as effective for both horizontal welds. For this example, using kl = 2.75 in., C = 1.43 and D_{req} = 3.45 sixteenths.

Design the gusset plate

The gusset plate thickness should match or slightly exceed that of the tee stem; use ½-in. plate.

Check tension yielding on the Whitmore section

$$L_w = 4.00 \text{ in.} + 2(8.00 \text{ in.})\tan 30° = 13.2 \text{ in.}$$

Manual Part 9

LRFD	ASD
$\phi R_n = \phi F_y A_g$ $= 0.9(36 \text{ ksi})(13.2 \text{ in.})(0.500 \text{ in.})$ $= 214 \text{ kips} > 168 \text{ kips}$ **o.k.**	$R_n / \Omega = \dfrac{F_y A_g}{\Omega} = \dfrac{(36 \text{ ksi})(13.2 \text{ in.})(0.500 \text{ in.})}{1.67}$ $= 142 \text{ kips} > 112 \text{ kips}$ **o.k.**

Eqn. J4-1

Check block shear rupture of the gusset plate

Assume uniform tension stress, use $U_{bs} = 1.0$

Section J4.3

Gross area subject to shear, $A_{gv} = 2(8.00 \text{ in.})(0.500 \text{ in.}) = 8.00 \text{ in.}^2$

Net area subject to shear, $A_{nv} = A_{gv} = 8.00 \text{ in.}^2$

Net area subject to tension, $A_t = (4.00 \text{ in.})(0.500 \text{ in.}) = 2.00 \text{ in.}^2$

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv})$ $= 0.75(58 \text{ ksi})(2.00 \text{ in.}^2)(1.0)$ $+ \min\begin{Bmatrix} 0.75(0.6)(36 \text{ ksi})(8.00 \text{ in.}^2), \\ 0.75(0.6)(58 \text{ ksi})(8.00 \text{ in.}^2) \end{Bmatrix}$ $= 217 \text{ kips} > 168 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$ $= \dfrac{(58 \text{ ksi})(2.00 \text{ in.}^2)(1.0)}{2.00}$ $+ \min\begin{Bmatrix} \dfrac{0.6(36 \text{ ksi})(8.00 \text{ in.}^2)}{2.00}, \\ \dfrac{0.6(58 \text{ ksi})(8.00 \text{ in.}^2)}{2.00} \end{Bmatrix}$ $= 144 \text{ kips} > 112 \text{ kips}$ **o.k.**

Eqn. J4-5

The gusset width must be such that the groove weld connecting it to the stem of the tee can transfer the tee axial force between the gusset and the top chord (note that the slight slope of the top chord has been ignored). The required length is

LRFD	ASD
$L_{req} = \dfrac{R_u}{\phi(0.6F_y)t}$ $= \dfrac{131 \text{ kips}}{1.00(0.6)(36 \text{ ksi})(0.500 \text{ in.})} = 12.1 \text{ in.}$ Use L = 15 in. *Check shear rupture of the tee stem*	$L_{req} = \dfrac{\Omega R_a}{(0.6F_y)t}$ $= \dfrac{(1.50)(2.00)(87.2 \text{ kips})}{(0.6)(36 \text{ ksi})(0.500 \text{ in.})} = 12.1 \text{ in.}$ Use L = 15 in. *Check shear rupture of the tee stem*

LRFD	ASD
$\phi Rn = \phi 0.6 F_u A_{nv}$ $= 0.75(0.6)(65\text{ ksi})(15.0\text{ in.})(0.455\text{ in.})$ $= 200\text{ kips} > 131\text{ kips}$	$\dfrac{R_n}{\Omega} = \dfrac{0.6 F_u A_{nv}}{\Omega}$ $= \dfrac{0.6(65\text{ ksi})(15.0\text{ in.})(0.455\text{ in.})}{2.00}$ $= 133\text{ kips} > 87.2\text{ kips}$

Eqn. J4-4

The gusset depth depends upon the connection angles. From a scaled layout, the gusset must extend 1'-0" below the tee stem.

Use PL½×12×1'-3".

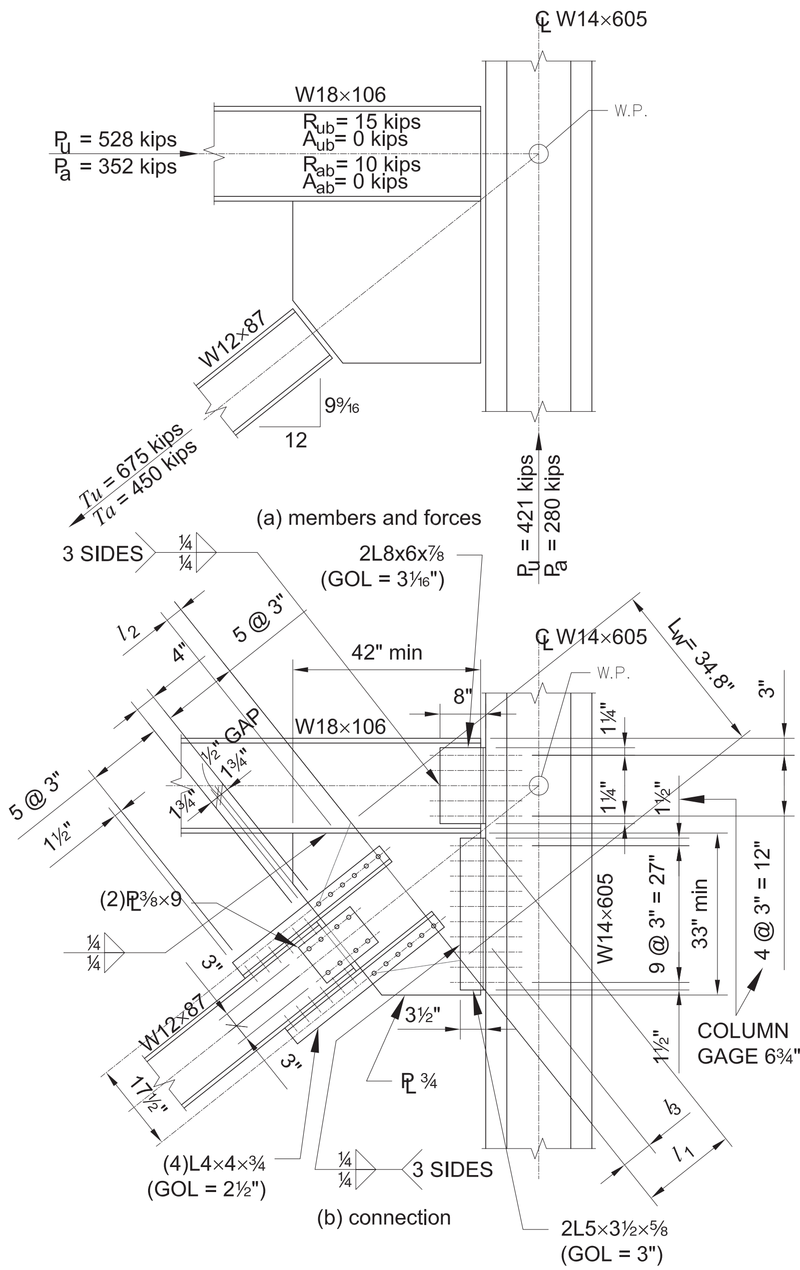
Example II.C-2 Bracing Connection

Given:

Design the diagonal bracing connection between the W12×87 brace and the W18×106 beam and the W14×605 column.

Brace Axial Load	T_u = 675 kips	T_a = 450 kips
Beam End Reaction	R_u = 15 kips	R_a = 10 kips
Column Axial Load	P_u = 421 kips	P_a = 280 kips
Beam Axial Load	P_u = 528 kips	P_a = 352 kips

Use ⅞-in. diameter ASTM A325-N or F1852-N bolts in standard holes and E70 electrodes.



Material Properties:

Brace	W12×87	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Beam	W18×106	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Column	W14×605	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Gusset Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	
Angles		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Brace	W12×87	A=25.6 in.^{2}	d =12.5 in.	t_w=0.515 in.	b_f=12.1 in.	t_f=0.810 in.	Manual Table 1-1
Beam	W18×106	d=18.7 in.	t_w=0.590 in.	b_f=11.2 in.	t_f=0.940 in.	k_{des}=1.34	
Column	W14×605	d=20.9 in.	t_w=2.60 in.	b_f=17.4 in.	t_f=4.16 in.		

Solution:

Brace-to-gusset connection

Distribute brace force in proportion to web and flange areas.

LRFD	ASD
Force in flange $P_{uf} = \dfrac{P_u b_f t_f}{A} = \dfrac{(675\text{ kips})(12.1\text{ in.})(0.810\text{ in.})}{25.6\text{ in.}^2}$ = 258 kips Force in web $P_{uw} = P_u - 2P_{uf} = 675\text{ kips} - 2(258\text{ kips})$ = 159 kips	Force in flange $P_{af} = \dfrac{P_a b_f t_f}{A} = \dfrac{(450\text{ kips})(12.1\text{ in.})(0.810\text{ in.})}{25.6\text{ in.}^2}$ = 172 kips Force in web $P_{aw} = P_a - 2P_{af} = 450\text{ kips} - 2(172\text{ kips})$ = 106 kips

Design brace-flange-to-gusset connection.

For short claw angle connections, eccentricity may be an issue. See AISC Engineering Journal, Vol. 33, No. 4, pp 123-128, 1996.

Determine number of ⅞-in. diameter ASTM A325-N bolts required for the flanges on the brace side for single shear.

LRFD	ASD
$n_{\min} = \dfrac{P_{uf}}{\phi r_n} = \dfrac{258\text{ kips}}{21.6\text{ kips/bolt}}$ $= 11.9 \rightarrow 12\text{ bolts}$	$n_{\min} = \dfrac{P_{af}}{r_n/\Omega} = \dfrac{172\text{ kips}}{14.4\text{ kips/bolt}}$ $= 11.9 \rightarrow 12\text{ bolts}$

Manual Table 7-1

On the gusset side, since the bolts are in double shear, half as many bolts will be required. Try six rows of two bolts each through the flange, six bolts through the gusset, and 2L4x4x¾ angles (A = 10.9 in.^{2} $\bar{x} = 1.27$ in.).

Manual Table 1-7

LRFD	ASD
Check tension yielding of the angles $\phi R_n = \phi F_y A_g = 0.90(36\text{ ksi})(10.9\text{ in.}^2)$ = 353 kips > 258 kips **o.k.**	*Check tension yielding of the angles* $R_n / \Omega = \dfrac{F_y A_g}{1.67} = \dfrac{(36\text{ ksi})(10.9\text{ in.}^2)}{1.67}$ = 235 kips > 172 kips **o.k.**

Eqn. J4-1

Check tension rupture of the angles

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.27\text{ in.}}{15\text{ in.}} = 0.915$$ Table D3.1

$$A_e = UA_n = 0.915\left[10.9\text{ in.}^2 - 2(0.750\text{ in.})(1.00\text{ in.})\right] = 8.60\text{ in.}^2$$ Eqn. D3-1

Eqn. J4-2

LRFD	ASD
$\phi R_n = \phi F_u A_e = 0.75(58\text{ ksi})(8.60\text{ in.}^2)$ $= 374\text{ kips} > 258\text{ kips}$ **o.k.**	$R_n / \Omega = \dfrac{F_u A_e}{\Omega} = \dfrac{(58\text{ ksi})(8.60\text{ in.}^2)}{2.00}$ $= 249\text{ kips} > 172\text{ kips}$ **o.k.**

Check block shear rupture of the angles.

Use n = 6, L_{ev} = 1½ in., L_{eh} = 1½ in. and U_{bs} = *1.0*

Eqn.J4-5; Manual Table 9-3a; Manual Table 9-3b; Manual Table 9-3c; Eqn. J4-5

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$
Tension Rupture Component	Tension Rupture Component
$\phi F_u A_{nt} = (43.5\text{ kips/in.})(0.750\text{ in.})(2)$	$F_u A_{nt} / \Omega = (29.0\text{ kips/in.})(0.750\text{ in.})(2)$
Shear Yielding Component	Shear Yielding Component
$\phi 0.6 F_y A_{gv} = (267\text{ kips/in.})(0.750\text{ in.})(2)$	$0.6 F_y A_{gv} / \Omega = (178\text{ kips/in.})(0.750\text{ in.})(2)$
Shear Rupture Component	Shear Rupture Component
$\phi 0.6 F_u A_{nv} = (287\text{ kips/in.})(0.750\text{ in.})(2)$	$0.6 F_u A_{nv} / \Omega = (191\text{ kips/in.})(0.750\text{ in.})(2)$
$\phi R_n = (43.5\text{ kips} + 267\text{ kips})(0.750\text{ in.})(2)$ $= 466\text{ kips} > 258\text{ kips}$ **o.k.**	$R_n / \Omega = (29.0\text{ kips} + 178\text{ kips})(0.750\text{ in.})(2)$ $= 311\text{ kips} > 172\text{ kips}$ **o.k.**

The flange thickness is greater than the angle thickness, the tensile strength of the flange is greater than the tensile strength of the angles, and L_{ev} = 1¾ in. for the brace is greater than 1½ in. for the angles.

Therefore, by inspection, the block shear rupture strength of the brace flange is o.k.

Design brace-web-to-gusset connection

Determine number of ⅞-in. diameter ASTM A325-N bolts required on the brace side (double shear) for shear.

Manual Table 7-1

LRFD	ASD
$n_{\min} = \dfrac{P_{uw}}{\phi r_n} = \dfrac{159\text{ kips}}{43.3\text{ kips/bolt}} = 3.67 \rightarrow 4\text{ bolts}$	$n_{\min} = \dfrac{P_{aw}}{r_n/\Omega} = \dfrac{106\text{ kips}}{28.9\text{ kips/bolt}} = 3.67 \rightarrow 4\text{ bolts}$

On the gusset side, the same number of bolts are required. Try two rows of two bolts and two PL⅜×9.

LRFD	ASD
Check tension yielding of the plates $\phi R_n = \phi F_y A_g$ $= 0.90(36 \text{ ksi})(2)(0.375 \text{ in.})(9.00 \text{ in.})$ $= 219 \text{ kips} > 159 \text{ kips}$ **o.k.**	*Check tension yielding of the plates* $R_n / \Omega = F_y A_g / \Omega$ $= \dfrac{(36 \text{ ksi})(2)(0.375 \text{ in.})(9.00 \text{ in.})}{1.67}$ $= 146 \text{ kips} > 106 \text{ kips}$ **o.k.**

Eqn. J4-1

Check tension rupture of the plates

Take A_e as the lesser of A_n and $0.85A_g$,

Section J4.18

$$A_e = \min\left(A_n, 0.85A_g\right)$$
$$= \min\left\{(0.375 \text{ in.})\left[2(9.00 \text{ in.}) - 4(1.00 \text{ in.})\right], 0.85(2)(0.375 \text{ in.})(9.00 \text{ in.})\right\} = 5.25 \text{ in.}^2$$

LRFD	ASD
$\phi R_n = \phi F_u A_e = 0.75(58 \text{ ksi})(5.25 \text{ in.}^2)$ $= 228 \text{ kips} > 159 \text{ kips}$ **o.k.**	$R_n / \Omega = F_u A_e / \Omega = \dfrac{(58 \text{ ksi})(5.25 \text{ in.}^2)}{2.00}$ $=152 \text{ kips} > 106 \text{ kips}$ **o.k.**

Eqn. J4-2

Check block shear rupture of the plates (outer blocks).

Use n = 2, L_{ev} = 1½ in., and L_{eh} = 1½ in. The calculations here are done in the same manner as those for the angles. Thus,

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$
Tension Rupture Component $\phi F_u A_{nt} = (43.5 \text{ kips/in.})(0.375 \text{ in.})(4)$	Tension Rupture Component $F_u A_{nt} / \Omega = (29.0 \text{ kips/in.})(0.375 \text{ in.})(4)$
Shear Yielding Component $\phi 0.6 F_y A_{gv} = (72.9 \text{ kips/in.})(0.375 \text{ in.})(4)$	Shear Yielding Component $0.6 F_y A_{gv} / \Omega = (48.6 \text{ kips/in.})(0.375 \text{ in.})(4)$
Shear Rupture Component $\phi 0.6 F_u A_{nv} = (78.3 \text{ kips/in.})(0.375 \text{ in.})(4)$	Shear Rupture Component $0.6 F_u A_{nv} / \Omega = (52.2 \text{ kips/in.})(0.375 \text{ in.})(4)$
$\phi R_n = (43.5 \text{ kips} + 72.9 \text{ kips})(0.375 \text{ in.})(4)$ $= 175 \text{ kips} > 159 \text{ kips}$ **o.k.**	$R_n / \Omega = (29.0 \text{ kips} + 48.6 \text{ kips})(0.375 \text{ in.})(4)$ $= 116 \text{ kips} > 106 \text{ kips}$ **o.k.**

Eqn. J4-5

Manual Table 9-3a

Manual Table 9-3b

Manual Table 9-3c

Eqn. J4-5

Similarly, by inspection, because the tension area is larger for the interior flange path, the block shear rupture strength of the interior blocks of the brace-web plates is o.k.

Check block shear rupture of the interior brace web

Use n = 2, L_{ev} = 1¾ in., but use 1½ in. for calculations to account for possible underrun in brace length and L_{eh} = 6 in.

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn. J4-5
Tension Rupture Component $\phi F_u A_{nt} = (122 \text{ kips/in.})(0.515 \text{ in.})(2)$	Tension Rupture Component $F_u A_{nt} / \Omega = (81.3 \text{ kips/in.})(0.515 \text{ in.})(2)$	Manual Table 9-3a
Shear Yielding Component $\phi 0.6 F_y A_{gv} = (101 \text{ kips/in.})(0.515 \text{ in.})(2)$	Shear Yielding Component $0.6 F_y A_{gv} / \Omega = (67.5 \text{ kips/in.})(0.515 \text{ in.})(2)$	Manual Table 9-3b
Shear Rupture Component $\phi 0.6 F_u A_{nv} = (87.8 \text{ kips/in.})(0.515 \text{ in.})(2)$	Shear Rupture Component $0.6 F_u A_{nv} / \Omega = (58.5 \text{ kips/in.})(0.515 \text{ in.})(2)$	Manual Table 9-3c
$\phi R_n = (122 \text{ kips} + 87.8 \text{ kips})(0.515 \text{ in.})(2)$ $= 216 \text{ kips} > 159 \text{ kips}$ **o.k.**	$R_n / \Omega = (81.3 \text{ kips} + 58.5 \text{ kips})(0.515 \text{ in.})(2)$ $= 144 \text{ kips} > 106 \text{ kips}$ **o.k.**	Eqn. J4-5

LRFD	ASD	
Check tension yielding of the brace $\phi R_n = \phi F_y A_g = 0.90(50 \text{ ksi})(25.6 \text{ in.}^2)$ $= 1150 \text{ kips} > 675 \text{ kips}$ **o.k.**	*Check tension yielding of the brace* $R_n / \Omega = F_y A_g / \Omega = \dfrac{(50 \text{ ksi})(25.6 \text{ in.}^2)}{1.67}$ $= 766 \text{ kips} > 450 \text{ kips}$ **o.k.**	Eqn. D2-1

Check tension rupture of the brace

Take A_e as A_n,

$$A_e = A_n = 25.6 \text{ in.}^2 - \left[4(0.810 \text{ in.}) + 2(0.515 \text{ in.})\right](1.00 \text{ in.}) = 21.3 \text{ in.}^2$$

LRFD	ASD	
$\phi R_n = \phi F_u A_e = 0.75(65 \text{ ksi})(21.3 \text{ in.}^2)$ $= 1040 \text{ kips} > 675 \text{ kips}$ **o.k.**	$R_n / \Omega = F_u A_e / \Omega = \dfrac{(65 \text{ ksi})(21.3 \text{ in.}^2)}{2.00}$ $= 692 \text{ kips} > 450 \text{ kips}$ **o.k.**	Eqn. D2-2

Design the gusset

From edge distance, spacing, and thickness requirements of the angles and web plates, try PL ¾.

Check block shear rupture for the force transmitted through web. Use $n = 2$, $L_{ev} = 1\frac{3}{4}$ in. and $L_{eh} = 3$ in.

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv},\ \phi 0.6 F_u A_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6 F_y A_{gv}}{\Omega},\ \dfrac{0.6 F_u A_{nv}}{\Omega}\right)$	Eqn. J4-5
Tension Rupture Component $\phi F_u A_{nt} = (109 \text{ kips/in.})(0.750 \text{ in.})(2)$	Tension Rupture Component $F_u A_{nt} / \Omega = (72.5 \text{ kips/in.})(0.750 \text{ in.})(2)$	Manual Table 9-3a
Shear Yielding Component		

LRFD	ASD	
	Shear Yielding Component	
$\phi 0.6F_yA_{gv} = (76.9\text{ kips/in.})(0.750\text{ in.})(2)$	$0.6F_yA_{gv}/\Omega = (51.3\text{ kips/in.})(0.750\text{ in.})(2)$	Manual Table 9-3b
Shear Rupture Component	Shear Rupture Component	
$\phi 0.6F_uA_{nv} = (84.8\text{ kips/in.})(0.750\text{ in.})(2)$	$0.6F_uA_{nv}/\Omega = (56.6\text{ kips/in.})(0.750\text{ in.})(2)$	Manual Table 9-3c
$\phi R_n = (109\text{ kips} + 76.9\text{ kips})(0.750\text{ in.})(2)$ $= 279\text{ kips} > 159\text{ kips}$ **o.k.**	$R_n/\Omega = (72.5\text{ kips} + 51.3\text{ kips})(0.750\text{ in.})(2)$ $= 186\text{ kips} > 106\text{ kips}$ **o.k.**	Eqn. J4-5

Check block shear rupture for total brace force

With A_{gv} = 25.1in.^{2}, A_{nv} = 16.9 in.^{2}, and A_{nt} = 12.4 in.^{2}. Thus, — Section J4.3

LRFD	ASD	
$\phi R_n = \phi F_uA_{nt}U_{bs} + \min\left(\phi 0.6F_yA_{gv},\ \phi 0.6F_uA_{nv}\right)$	$\dfrac{R_n}{\Omega} = \dfrac{F_uA_{nt}U_{bs}}{\Omega} + \min\left(\dfrac{0.6F_yA_{gv}}{\Omega},\ \dfrac{0.6F_uA_{nv}}{\Omega}\right)$	Eqn. J4-5
Tension Rupture Component	Tension Rupture Component	
$\phi F_uA_{nt} = 0.75(58\text{ ksi})(12.4\text{ in.}^2)$ $= 539\text{ kips}$	$\dfrac{F_uA_{nt}}{\Omega} = \dfrac{(58\text{ ksi})(12.4\text{ in.}^2)}{2.00}$ $= 360\text{ kips}$	
Shear Yielding Component	Shear Yielding Component	
$\phi 0.6F_yA_{gv} = 0.75(0.6)(36\text{ ksi})(25.1\text{ in.}^2)$ $= 407\text{ kips}$	$\dfrac{0.6F_yA_{gv}}{\Omega} = \dfrac{(0.6)(36\text{ ksi})(25.1\text{ in.}^2)}{2.00}$ $= 271\text{ kips}$	
Shear Rupture Component	Shear Rupture Component	
$\phi 0.6F_uA_{nv} = 0.75(0.6)(58\text{ ksi})(16.9\text{ in.}^2)$ $= 441\text{ kips}$	$\dfrac{0.6F_uA_{nv}}{\Omega} = \dfrac{(0.6)(58\text{ ksi})(16.9\text{ in.}^2)}{2.00}$ $= 294\text{ kips}$	
$\phi R_n = 539\text{ kips} + \min(407\text{ kips}, 441\text{ kips})$ $= 946\text{ kips} > 675\text{ kips}$ **o.k.**	$R_n/\Omega = 360\text{ kips} + \min(271\text{ kips}, 294\text{ kips})$ $= 631\text{ kips} > 450\text{ kips}$ **o.k.**	

Check tension yielding on the Whitmore section of the gusset.

The Whitmore section, as illustrated with dashed lines in Figure (b), is 34.8 in. long; 30.9 in. occurs in the gusset and 3.90 in. occurs in the beam web. Thus, — Manual Part 9

LRFD	ASD	
$\phi R_n = \phi F_yA_w$ $= 0.90\begin{bmatrix}(36\text{ ksi})(30.9\text{ in.})(0.750\text{ in.}) + \\ (50\text{ ksi})(3.90\text{ in.})(0.590\text{ in.})\end{bmatrix}$ $= 854\text{ kips} > 675\text{ kips}$ **o.k.**	$R_n/\Omega = F_yA_w/\Omega$ $= \dfrac{\begin{bmatrix}(36\text{ ksi})(30.9\text{ in.})(0.750\text{ in.}) + \\ (50\text{ ksi})(3.90\text{ in.})(0.590\text{ in.})\end{bmatrix}}{1.67}$ $= 568\text{ kips} > 450\text{ kips}$ **o.k.**	Eqn. J4-1

Note: The beam web thickness is used, conservatively ignoring the larger thickness in the beam-flange and the flange-to-web fillet area.

Check bearing strength of the angles, brace flange and gusset

The bearing strength per bolt is given by Specification Section J3.10 as:

$$r_n = 1.2L_c tF_u \le 2.4dtF_u$$ Eqn. J3-6a

Because of the edge distance requirement, the angles, brace flange, and gusset, must be considered simultaneously. The angles have edge bolts and interior bolts. For an edge bolt,

$$L_c = 1.50 \text{ in.} - (0.5)(0.938 \text{ in.}) = 1.03 \text{ in.}$$

LRFD	ASD
$\phi r_n = \phi 1.2L_c tF_u \le \phi 2.4dtF_u$ $= 0.75(1.2)(1.03 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})$ $\le (0.75)2.4(0.875 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})$ $= 40.3 \text{ kips} \le 68.5 \text{ kips}$ $= 40.3$ kips/bolt	$r_n / \Omega = \dfrac{1.2L_c tF_u}{\Omega} \le \dfrac{2.4dtF_u}{\Omega}$ $= \dfrac{1.2(1.03 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})}{2.00}$ $\le \dfrac{2.4(0.875 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})}{2.00}$ $= 26.9 \text{ kips} \le 45.7 \text{ kips}$ $= 26.9$ kips/bolt

Eqn. J3-6a

For an interior bolt,

$$L_c = 3.00 \text{ in.} - (1)(0.938 \text{ in.}) = 2.06 \text{ in.}$$

LRFD	ASD
$\phi r_n = \phi 1.2L_c tF_u \le \phi 2.4dtF_u$ $= 0.75(1.2)(2.06 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})$ $\le 0.75(2.4)(0.875 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})$ $= 80.6 \text{ kips} \le 68.5 \text{ kips}$ $= 68.5$ kips/bolt	$r_n / \Omega = \dfrac{1.2L_c tF_u}{\Omega} \le \dfrac{2.4dtF_u}{\Omega}$ $= \dfrac{1.2(2.06 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})}{2.00}$ $\le \dfrac{2.4(0.875 \text{ in.})(0.750 \text{ in.})(58 \text{ ksi})}{2.00}$ $= 53.8 \text{ kips} \le 45.7 \text{ kips}$ $= 45.7$ kips/bolt

Eqn. J3-6a

Note: The above strengths are per angle.

The gusset plate bolt strengths differ from the angle bolt strengths because although the gusset plate is the same thickness and material, and its edge distance is L_e = 1.75 in. and L_e = 1.28 in.

LRFD	ASD
For an edge bolt, $\phi r_n = 50.1$ kips/bolt For an interior bolt, ϕr_n = 68.5 kips/bolt	For an edge bolt, $r_n / \Omega = 33.4$ kips/bolt For an interior bolt, r_n/Ω = 45.7 kips/bolt

For the brace flange, edge bolt L_c = 1.28 in. and interior bolt L_c = 2.06 in.

LRFD	ASD
Brace edge bolt, $\phi r_n = \phi 1.2L_c tF_u \le \phi 2.4dtF_u$	Brace edge bolt, $r_n / \Omega = \dfrac{1.2L_c tF_u}{\Omega} \le \dfrac{2.4dtF_u}{\Omega}$

Eqn. J3-6a

LRFD	ASD
$= 0.75(1.2)(1.28\text{ in.})(0.810\text{ in.})(65\text{ ksi})$ $\leq 0.75(2.4)(0.875\text{ in.})(0.810\text{ in.})(65\text{ ksi})$ $= 60.7\text{ kips} \leq 82.9\text{ kips}$ $= 60.7$ kips/bolt	$= \dfrac{1.2(1.28\text{ in.})(0.810\text{ in.})(65\text{ ksi})}{2.00}$ $\leq \dfrac{2.4(0.875\text{ in.})(0.810\text{ in.})(65\text{ ksi})}{2.00}$ $= 40.4\text{ kips} \leq 55.3\text{ kips}$ $= 40.4$ kips/bolt

LRFD	ASD
Brace interior bolt, $\phi r_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d t F_u$ $= 0.75(1.2)(2.06\text{ in.})(0.810\text{ in.})(65\text{ ksi})$ $\leq 0.75(2.4)(0.875\text{ in.})(0.810\text{ in.})(65\text{ ksi})$ $= 97.6\text{ kips} \leq 82.9\text{ kips}$ $= 82.9$ kips/bolt	Brace interior bolt, $r_n / \Omega = \dfrac{1.2 L_c t F_u}{\Omega} \leq \dfrac{2.4 d t F_u}{\Omega}$ $= \dfrac{1.2(2.06\text{ in.})(0.810\text{ in.})(65\text{ ksi})}{2.00}$ $\leq \dfrac{2.4(0.875\text{ in.})(0.810\text{ in.})(65\text{ ksi})}{2.00}$ $= 65.1\text{ kips} \leq 55.3\text{ kips}$ $= 55.3$ kips/bolt

Eqn. J3-6a

The bearing strength of the flange connection can now be calculated. Summarizing the various strengths:

LRFD

Member	Edge	Interior
Angle (2)	80.6 kips	137 kips
Gusset	50.1 kips	68.5 kips
Brace Flange (2)	121 kips	166 kips

From the above table,

$$\phi R_n = (1\text{ bolt})(50.1\text{ kips/bolt}) + (5\text{ bolts})(68.5\text{ kips/bolt})$$

$= 393\text{ kips} > 258\text{ kips}$ **o.k.**

ASD

Member	Edge	Interior
Angle (2)	53.8 kips	91.4 kips
Gusset	33.4 kips	45.7 kips
Brace Flange (2)	80.8 kips	111 kips

From the above table,

$$R_n / \Omega = (1\text{ bolt})(33.4\text{ kips/bolt}) + (5\text{ bolts})(45.7\text{ kips/bolt})$$

$= 262\text{ kips} > 172\text{ kips}$ **o.k.**

Note: If any of these bearing strengths were less than the bolt double shear strength; the bolt shear strength would need to be re-checked.

Check bearing strength of the brace web, web plate, and gusset.

Because of the edge distance requirement, these must be checked simultaneously.

The edge distance for the brace web and gusset is L_e = 1.75 in., and the edge distance for the plates is L_e = 1.5 in.

For an edge bolt

Brace web and gusset L_c = 1.75 in. – 0.5(0.938 in.) = 1.28 in.
Plates L_c = 1.50 in. – 0.5(0.938 in.) = 1.03 in.

For interior bolts at 3.00 in. on center

Brace web, gusset and plates L_c = 3.00 in. – 1.0(0.938 in.) = 2.06 in.

For and edge bolt in the brace web with L_c = 1.28 in.

LRFD	ASD	
$\phi r_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d t F_u$ $= 0.75(1.2)(1.28 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})$ $\leq 0.75(2.4)(0.875 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})$ $= 38.6 \text{ kips} \leq 52.7 \text{ kips}$ $= 38.6 \text{ kips}$	$\frac{r_n}{\Omega} = \frac{1.2 L_c t F_u}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(1.28 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})}{2.00}$ $\leq \frac{2.4(0.875 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})}{2.00}$ $= 25.7 \text{ kips} \leq 35.1 \text{ kips}$ $= 25.7 \text{ kips}$	Eqn. J3-6a

For an interior bolt in the brace web with L_c = 2.06 in.

LRFD	ASD	
$\phi r_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d t F_u$ $= 0.75(1.2)(2.06 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})$ $\leq 0.75(2.4)(0.875 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})$ $= 62.1 \text{ kips} \leq 52.7 \text{ kips}$ $= 52.7 \text{ kips}$	$\frac{r_n}{\Omega} = \frac{1.2 L_c t F_u}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(2.06 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})}{2.00}$ $\leq \frac{2.4(0.875 \text{ in.})(0.515 \text{ in.})(65 \text{ ksi})}{2.00}$ $= 41.4 \text{ kips} \leq 35.1 \text{ kips}$ $= 35.1 \text{ kips}$	Eqn. J3-6a

The two web plates and the gusset plate are the same material and thickness

The web plates have a shorter edge distance and will limit the bolt bearing capacity.

For an edge bolt in a web plate with k = 1.03 in.

LRFD	ASD	
$\phi r_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d t F_u$ $= 0.75(1.2)(1.03 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})$ $\leq 0.75(2.4)(0.875 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})$ $= 20.2 \text{ kips} \leq 34.3 \text{ kips}$ $= 20.2 \text{ kips}$	$\frac{r_n}{\Omega} = \frac{1.2 L_c t F_u}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(1.03 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})}{2.00}$ $\leq \frac{2.4(0.875 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})}{2.00}$ $= 13.4 \text{ kips} \leq 22.8 \text{ kips}$ $= 13.4 \text{ kips}$	Eqn. J3-6a

For an interior bolt in a web plate with L_c = 2.06 in.

LRFD	ASD
$\phi r_n = \phi 1.2 L_c t F_u \le \phi 2.4 d t F_u$ $= 0.75(1.2)(2.06 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})$ $\le 0.75(2.4)(0.875 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})$ $= 40.3 \text{ kips} \le 34.3 \text{ kips}$ $= 34.3 \text{ kips}$	$\dfrac{r_n}{\Omega} = \dfrac{1.2 L_c t F_u}{\Omega} \le \dfrac{2.4 d t F_u}{\Omega}$ $= \dfrac{1.2(2.06 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})}{2.00}$ $\le \dfrac{2.4(0.875 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})}{2.00}$ $= 26.9 \text{ kips} \le 22.8 \text{ kips}$ $= 22.8 \text{ kips}$

Eqn. J3-6a

The bearing strength of the web connection can now be calculated. Summarizing the various strengths:

LRFD

Member	Edge	Interior
Brace web	38.6 kips	52.7 kips
Web plate(2)	40.4 kips	68.6 kips

From the above table,

$$\phi R_n = (2 \text{ bolts})(38.6 \text{ kips/bolt}) + (2 \text{ bolts})(52.7 \text{ kips/bolt})$$
$$= 183 \text{ kips} > 159 \text{ kips} \quad \textbf{o.k.}$$

ASD

Member	Edge	Interior
Brace web	25.7 kips	35.1 kips
Web plate(2)	26.8 kips	45.6 kips

From the above table,

$$\frac{R_n}{\Omega} = (2 \text{ bolts})(25.7 \text{ kips/bolt}) + (2 \text{ bolts})(35.1 \text{ kips/bolt})$$
$$= 122 \text{ kips} > 106 \text{ kips} \quad \textbf{o.k.}$$

Note: The bearing strength for the edge bolts is less than the double shear strength of the bolts; therefore, the bolt shear strength must be rechecked. The revised bolt shear strength is

LRFD	ASD
$\phi R_n = (2 \text{ bolts})(38.6 \text{ kips/bolt}) + (2 \text{ bolts})(43.3 \text{ kips/bolt})$ $= 164 \text{ kips} > 159 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = (2 \text{ bolts})(25.7 \text{ kips/bolt}) + (2 \text{ bolts})(28.9 \text{ kips/bolt})$ $= 109 \text{ kips} > 106 \text{ kips}$ **o.k.**

Note: When the brace force is compression gusset buckling would have to be checked; refer to the comments at the end of this example.

Distribution of the brace force to beam and column

From the members and frame geometry

Manual Part 13

$$e_b = \frac{d_{beam}}{2} = \frac{18.7 \text{ in.}}{2} = 9.35 \text{ in.}$$

$$e_c = \frac{d_{column}}{2} = \frac{20.9 \text{ in.}}{2} = 10.5 \text{ in.}$$

$$\tan\theta = \frac{12}{9\frac{9}{16}} = 1.25$$

and $e_b \tan\theta - e_c = (9.35 \text{ in.})(1.25) - (10.5 \text{ in.}) = 1.19 \text{ in.}$

Try gusset PL ¾ × 42 in. horizontally × 33 in. vertically (several intermediate gusset dimensions were inadequate). With connection centroids at the midpoint of the gusset edges

$$\bar{\alpha} = \frac{42.0 \text{ in.}}{2} + 0.500 \text{ in.} = 21.5 \text{ in.}$$

where ½ in. is allowed for the setback between the gusset and the column, and

$$\bar{\beta} = \frac{33.0 \text{ in.}}{2} = 16.5 \text{ in.}$$

Choosing $\beta = \bar{\beta}$, the α required for the uniform forces is

$$\alpha = e_b \tan\theta - e_c + \beta \tan\theta = 1.19 \text{ in.} + (16.5 \text{ in.})(1.25) = 21.8 \text{ in.}$$

The resulting eccentricity is $\alpha - \bar{\alpha}$, where

$$\alpha - \bar{\alpha} = 21.8 \text{ in.} - 21.5 \text{ in.} = 0.300 \text{ in.}$$

Since this slight eccentricity is negligible, use α = 21.8 in. and β = 16.5 in.

Calculate gusset interface forces

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(21.8 \text{ in.} + 10.5 \text{ in.})^2 + (16.5 \text{ in.} + 9.35 \text{ in.})^2} = 41.4 \text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$H_{uc} = \frac{e_c P_u}{r} = \frac{(10.5 \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 171 \text{ kips}$ $V_{uc} = \frac{\beta P_u}{r} = \frac{(16.5 \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 269 \text{ kips}$	$H_{ac} = \frac{e_c P_a}{r} = \frac{(10.5 \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 114 \text{ kips}$ $V_{ac} = \frac{\beta P_a}{r} = \frac{(16.5 \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 179 \text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \frac{\alpha P_u}{r} = \frac{(21.8 \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 355 \text{ kips}$ $V_{ub} = \frac{e_b P_u}{r} = \frac{(9.35 \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 152 \text{ kips}$	$H_{ab} = \frac{\alpha P_a}{r} = \frac{(21.8 \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 237 \text{ kips}$ $V_{ab} = \frac{e_b P_a}{r} = \frac{(9.35 \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 102 \text{ kips}$

Design gusset-to-column connection

The forces involved are V_{uc} = 269 kips (V_{ac} = 179 kips ASD) shear and H_{uc} = 171 kips (H_{ac} = 114 kips ASD) tension.

Try 2L5×3½×½×2'-6" welded to the gusset and bolted with 10 rows of ⅞ in. diameter A325-N bolts in standard holes to the column flange.

Calculate the required tensile strength per bolt

LRFD	ASD
$T_u = \dfrac{H_{uc}}{n} = \dfrac{171 \text{ kips}}{20 \text{ bolts}} = 8.55 \text{ kips/bolt}$	$T_a = \dfrac{H_{ac}}{n} = \dfrac{114 \text{ kips}}{20 \text{ bolts}} = 5.70 \text{ kips/bolt}$
Check design strength of bolts for tension-shear interaction.	*Check allowable strength of bolts for tension-shear interaction.*
$r_{uv} = \dfrac{V_{uc}}{n} = \dfrac{269 \text{ kips}}{20 \text{ bolts}} = 13.5 \text{ kips/bolt}$	$r_{av} = \dfrac{V_{ac}}{n} = \dfrac{179 \text{ kips}}{20 \text{ bolts}} = 8.95 \text{ kips/bolt}$
13.5 kips/bolt < 21.6 kips/bolt **o.k.**	8.95 kips/bolt < 14.4 kips/bolt **o.k.**
$f_{uv} = \dfrac{r_{uv}}{A_b} = \dfrac{13.5 \text{ kips}}{0.601 \text{ in.}^2} = 22.5 \text{ ksi}$	$f_{av} = \dfrac{r_{av}}{A_b} = \dfrac{8.95 \text{ kips}}{0.601 \text{ in.}^2} = 14.9 \text{ ksi}$
$F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{uv} \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}}{0.75(48 \text{ ksi})}(22.5 \text{ ksi})$ $= 60.8 \text{ ksi} \le 90 \text{ ksi}$	$F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}\Omega}{F_{nv}} f_{av} \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}(2.00)}{48 \text{ ksi}}(14.9 \text{ ksi})$ $= 61.1 \text{ ksi} \le 90 \text{ ksi}$
Use $F'_{nt} =$ 60.8 ksi	Use $F'_{nt} =$ 61.1 ksi
$B_u = \phi F'_{nt} A_b = 0.75(60.8 \text{ ksi})(0.601 \text{ in.}^2)$ $= 27.4 \text{ kips/bolt} > 8.55 \text{ kips/bolt}$ **o.k.**	$B_a = \dfrac{F'_{nt} A_b}{\Omega} = \dfrac{(61.1 \text{ ksi})(0.601 \text{ in.}^2)}{2.00}$ $= 18.4 \text{ kips/bolt} > 5.70 \text{ kips/bolt}$ **o.k.**

Manual Table 7-1

Eqn. J3-3a/b and

Table J3.2

Check bearing strength at bolt holes

The bearing strength per bolt is

LRFD	ASD
$\phi r_n = \phi 1.2 L_c t F_u \le \phi 2.4 d t F_u$ $= 0.75(1.2)(1.03 \text{ in.})(0.500 \text{ in.})(58 \text{ ksi})$ $\le 0.75(2.4)(0.875 \text{ in.})(0.500 \text{ in.})(58 \text{ ksi})$ $= 26.9 \text{ kips} \le 45.7 \text{ kips}$ $= 26.9 \text{ kips/bolt}$	$r_n / \Omega = \dfrac{1.2 L_c t F_u}{\Omega} \le \dfrac{2.4 d t F_u}{\Omega}$ $= \dfrac{1.2(1.03 \text{ in.})(0.500 \text{ in.})(58 \text{ ksi})}{2.00}$ $\le \dfrac{2.4(0.875 \text{ in.})(0.500 \text{ in.})(58 \text{ ksi})}{2.00}$ $= 17.9 \text{ kips} \le 30.5 \text{ kips}$ $= 17.9 \text{ kips/bolt}$
Since this edge bolt value exceeds the single-shear strength of the bolts 21.6 kips, and the actual shear per bolt of 13.5 kips, bearing strength is o.k.	Since this edge bolt value exceeds the single-shear strength of the bolts 14.4 kips, and the actual shear per bolt of 8.95 kips, bearing strength is o.k.

Eqn. J3-6a

The bearing strength of the interior bolts will not control.

Check prying action

$$b = g - \frac{t}{2} = 3.00 \text{ in.} - \frac{0.500 \text{ in.}}{2} = 2.75 \text{ in.}$$

Manual Part 9

Note: 1⅜ in. entering and tightening clearance is accommodated and the column fillet toe is cleared. **o.k.**

$$a = 5.00 \text{ in.} - g = 5.00 \text{ in.} - 3.00 \text{ in.} = 2.00 \text{ in.}$$

Manual Table 7-16

$$b' = b - \frac{d_b}{2} = 2.75 \text{ in.} - \frac{0.875 \text{ in.}}{2} = 2.31 \text{ in.}$$

$$a' = a + \frac{d_b}{2} \leq \left(1.25b \text{ in.} + \frac{d_b}{2}\right)$$

$$= 2.00 \text{ in.} + \frac{0.875 \text{ in.}}{2} \leq 1.25(2.75 \text{ in.}) + \frac{0.875 \text{ in.}}{2}$$

$$= 2.44 \text{ in.} \leq 3.88 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{2.31 \text{ in.}}{2.44 \text{ in.}} = 0.947$$

LRFD	ASD
$\beta = \frac{1}{\rho}\left(\frac{B_u}{T_u} - 1\right) = \frac{1}{0.947}\left(\frac{27.4 \text{ kips/bolt}}{8.55 \text{ kips/bolt}} - 1\right)$ $= 2.33$	$\beta = \frac{1}{\rho}\left(\frac{B_a}{T_a} - 1\right) = \frac{1}{0.947}\left(\frac{18.4 \text{ kips/bolt}}{5.70 \text{ kips/bolt}} - 1\right)$ $= 2.35$

Since β > 1, set $\alpha' = 1.0$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{0.938 \text{ in.}}{3.00 \text{ in.}} = 0.687$$

LRFD	ASD
$t_{req} = \sqrt{\frac{4.44 T_u b'}{p F_u (1 + \delta\alpha')}}$ $= \sqrt{\frac{4.44(8.55 \text{ kips/bolt})(2.31 \text{ in.})}{(3.00 \text{ in.})(58 \text{ ksi})[1 + 0.687(1.0)]}}$ $= 0.547$ in. Since t = ½ in. < 0.547 in., angles are **n.g.**	$t_{req} = \sqrt{\frac{6.66 T_a b'}{p F_u (1 + \delta\alpha')}}$ $= \sqrt{\frac{6.66(5.70 \text{ kips/bolt})(2.31 \text{ in.})}{(3.00 \text{ in.})(58 \text{ ksi})[1 + 0.687(1.0)]}}$ $= 0.547$ in. Since t = ½ in. < 0.547 in., angles are **n.g.**

Try 2L5x3½x⅝ angles. The previously completed bearing checks for the 2L5x3½x½ angles will still be satisfactory and need not be repeated.

Check prying action with the 2L5x3½x⅝ *angles*

Manual Part 9

$$b = g - \frac{t}{2} = 3.00\text{ in.} - \frac{0.625\text{ in.}}{2} = 2.69\text{ in.}$$

$$a = 5.00\text{ in.} - g = 5.00\text{ in.} - 3.00\text{ in.} = 2.00\text{ in.}$$

$$b' = b - \frac{d_b}{2} = 2.69\text{ in.} - \frac{0.875\text{ in.}}{2} = 2.25\text{ in.}$$

$$\begin{aligned} a' &= a + \frac{d_b}{2} \le \left(1.25b\text{ in.} + \frac{d_b}{2}\right) \\ &= 2.00\text{ in.} + \frac{0.875\text{ in.}}{2} \le 1.25(2.69\text{ in.}) + \frac{0.875\text{ in.}}{2} \\ &= 2.44\text{ in.} \le 3.80\text{ in.} \end{aligned}$$

$$\rho = \frac{b'}{a'} = \frac{2.25\text{ in.}}{2.44\text{ in.}} = 0.922$$

LRFD	ASD
$\beta = \frac{1}{\rho}\left(\frac{\beta_u}{T_u} - 1\right) = \frac{1}{0.922}\left(\frac{27.4\text{ kips/bolt}}{8.55\text{ kips/bolt}} - 1\right)$ $= 2.39$	$\beta = \frac{1}{\rho}\left(\frac{\beta_u}{T_a} - 1\right) = \frac{1}{0.922}\left(\frac{18.4\text{ kips/bolt}}{5.70\text{ kips/bolt}} - 1\right)$ $= 2.42$

Since $\beta > 1$, set $\alpha' = 1.0$

$$\delta = 1 - \frac{\alpha'}{\rho} = 1 - \frac{0.938\text{ in.}}{3.00\text{ in.}} = 0.687$$

LRFD	ASD
$t_{req} = \sqrt{\frac{4.44T_u b'}{pF_u(1+\delta\alpha')}}$ $= \sqrt{\frac{4.44(8.55\text{ kips/bolt})(2.25\text{ in.})}{(3.00\text{ in.})(58\text{ ksi})\left[1+0.687(1.0)\right]}}$ $= 0.539\text{ in.} < 0.625\text{ in.}$ **angles are o.k.**	$t_{req} = \sqrt{\frac{6.66T_a b'}{pF_u(1+\delta\alpha')}}$ $= \sqrt{\frac{6.66(5.70\text{ kips/bolt})(2.25\text{ in.})}{(3.00\text{ in.})(58\text{ ksi})\left[1+0.687(1.0)\right]}}$ $= 0.539\text{ in.} < 0.625\text{ in.}$ **angles are o.k.**

Use the 2L5x3½x⅝ for the gusset to column connection.

Design welds

Try fillet welds around the perimeter (three sides) of both angles.

LRFD	ASD
$P_{uc} = \sqrt{H_{uc}^2 + V_{uc}^2}$ $= \sqrt{(171\text{ kips})^2 + (269\text{ kips})^2} = 319\text{ kips}$	$P_{ac} = \sqrt{H_{ac}^2 + V_{ac}^2}$ $= \sqrt{(114\text{ kips})^2 + (179\text{ kips})^2} = 212\text{ kips}$

$\theta = \tan^{-1}\left(\frac{H_{uc}}{V_{uc}}\right) = \tan^{-1}\left(\frac{171\text{ kips}}{269\text{ kips}}\right) = 32.4°$	$\theta = \tan^{-1}\left(\frac{H_{ac}}{V_{ac}}\right) = \tan^{-1}\left(\frac{114\text{ kips}}{179\text{ kips}}\right) = 32.5°$

From Manual Table 8-8 with $\theta = 30°$,

$l = 30.0$ in., $kl = 3.00$ in., therefore $k = 0.100$

$$xl = \frac{(kl)^2}{(l+2kl)} = \frac{(3.00\text{ in.})^2}{30.0\text{ in.}+2(3.00\text{ in.})} = 0.250\text{ in.}$$

$$al = 3.50\text{ in.} - xl = 3.50\text{ in.} - 0.250\text{ in.} = 3.25\text{ in.}$$

$a = 0.108$

By interpolation

$C = 2.55$

LRFD	ASD
$D_{req} = \frac{P_{uc}}{\phi C C_1 l}$ $= \frac{319\text{ kips}}{0.75(2.55)(1.0)(2\text{ welds})(30.0\text{ in.})}$ $= 2.78 \rightarrow 3$ sixteenths	$D_{req} = \frac{P_{ac}\Omega}{C C_1 l}$ $= \frac{(212\text{ kips})(2.00)}{(2.55)(1.0)(2\text{ welds})(30.0\text{ in.})}$ $= 2.77 \rightarrow 3$ sixteenths

From Specification Table J2.4, minimum weld size is ¼ in. Use ¼-in. fillet welds.

Check gusset thickness against weld size required for strength

For two fillet welds, $t_{min} = \frac{6.19D}{F_u} = \frac{6.19(2.78\text{ sixteenths})}{58\text{ ksi}} = 0.297\text{ in.} < \frac{3}{4}\text{ in.}$ **o.k.**

Check strength of angles

Check shear yielding (due to V_{uc} or V_{ac})

$$A_g = 2(30.0\text{ in.})(0.625\text{ in.}) = 37.5\text{ in.}^2$$

LRFD	ASD
$\phi R_n = \phi(0.60F_yA_g)$ $= 1.00\left[0.60(36\text{ ksi})(37.5\text{ in.}^2)\right]$ $= 810\text{ kips} > 269\text{ kips}$ **o.k.**	$R_n/\Omega = \frac{0.60F_yA_g}{\Omega}$ $= \frac{0.60(36\text{ ksi})(37.5\text{ in.}^2)}{1.50}$ $= 540\text{ kips} > 179\text{ kips}$ **o.k.**

Eqn. J4-3

Similarly, shear yielding of the angles due to H_{uc} is not critical.

Check shear rupture

$$A_{nv} = \left[2(30.0\text{ in.}) - 20(1.00\text{ in.})\right](0.625\text{ in.}) = 25.0\text{ in.}^2$$

LRFD	ASD	
$\phi R_n = \phi(0.60F_u A_{nv})$ $= 0.75(0.60)(58\text{ ksi})(25.0\text{ in.}^2)$ $= 653\text{ kips} > 269\text{ kips}$ **o.k.**	$R_n / \Omega = \dfrac{0.60F_u A_{nv}}{\Omega}$ $= \dfrac{0.60(58\text{ ksi})(25.0\text{ in.}^2)}{2.00}$ $= 435\text{ kips} > 179\text{ kips}$ **o.k.**	Eqn. J4-4

Block shear rupture

Use n = 10, L_{ev} = 1½ in. and L_{eh} = 2 in. Thus,

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6F_y A_{gv},\ \phi 0.6F_u A_{nv})$	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\dfrac{0.6F_y A_{gv}}{\Omega},\ \dfrac{0.6F_u A_{nv}}{\Omega}\right)$	
Tension Rupture Component	Tension Rupture Component	
$\phi F_u A_{nt} = (65.3\text{ kips/in.})(0.625\text{ in.})(2)$	$F_u A_{nt} / \Omega = (43.5\text{ kips/in.})(0.625\text{ in.})(2)$	Manual Table 9-3a
Shear Yielding Component	Shear Yielding Component	
$\phi 0.6F_y A_{gv} = (462\text{ kips/in.})(0.625\text{ in.})(2)$	$0.6F_y A_{gv} / \Omega = (308\text{ kips/in.})(0.625\text{ in.})(2)$	Manual Table 9-3b
Shear Rupture Component	Shear Rupture Component	
$\phi 0.6F_u A_{nv} = (496\text{ kips/in.})(0.625\text{ in.})(2)$	$0.6F_u A_{nv} / \Omega = (331\text{ kips/in.})(0.625\text{ in.})(2)$	Manual Table 9-3c
$\phi R_n = (65.3\text{ kips} + 462\text{ kips})(0.625\text{ in.})(2)$ $= 659\text{ kips} > 269\text{ kips}$ **o.k.**	$R_n / \Omega = (43.5\text{ kips} + 308\text{ kips})(0.625\text{ in.})(2)$ $= 439\text{ kips} > 179\text{ kips}$ **o.k.**	Eqn. J4-5

Check column flange

By inspection, the 4.16 in. thick column flange has adequate flexural strength, stiffness, and bearing strength.

Design the gusset-to-beam connection

The forces involved are

LRFD	ASD
H_{ub} = 355 kips	H_{ab} = 237 kips
V_{ub} = 152 kips	V_{ab} = 102 kips

This edge of the gusset is welded to the beam. The distribution of force on the welded edge is known to be non-uniform. The Uniform Force Method used here assumes a uniform distribution of stress on this edge. Fillet welds are known to have limited ductility, especially if transversely loaded. To account for this, the required strength of the gusset edge weld is amplified by a factor of 1.25 to allow for the redistribution of forces on the weld. (Manual Part 13)

The stresses on the gusset plate at the welded edge are as follows:

LRFD	ASD	
$f_{ua} = \dfrac{V_{ub}}{tl} \le \phi F_y$	$f_{aa} = \dfrac{V_{ab}}{tl} \le \dfrac{F_y}{\Omega}$	Eqn. J4-1

$= \dfrac{152\text{ kips}}{(0.750\text{ in.})(42.0\text{ in.})} \le 0.90(36\text{ ksi})$ $= 4.83\text{ ksi} < 32.4\text{ ksi}$ **o.k.** $f_{uv} = \dfrac{H_{ub}}{tl} \le \phi(0.60F_y)$ $= \dfrac{355\text{ kips}}{(0.750\text{ in.})(42.0\text{ in.})} \le 1.00(0.60)(36\text{ ksi})$ $= 11.3\text{ ksi} < 21.6\text{ ksi}$ **o.k.**	$= \dfrac{102\text{ kips}}{(0.750\text{ in.})(42.0\text{ in.})} \le \dfrac{36\text{ ksi}}{1.67}$ $= 3.24\text{ ksi} < 21.6\text{ ksi}$ **o.k.** $f_{av} = \dfrac{H_{ab}}{tl} \le \dfrac{0.60F_y}{\Omega}$ $= \dfrac{237\text{ kips}}{(0.750\text{ in.})(42.0\text{ in.})} \le \dfrac{0.60(36\text{ ksi})}{1.50}$ $= 7.52\text{ ksi} < 14.4\text{ ksi}$ **o.k.**

Eqn. J4-3

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{V_{ub}}{H_{ub}}\right) = \tan^{-1}\left(\dfrac{152\text{ kips}}{355\text{ kips}}\right) = 23.2°$	$\theta = \tan^{-1}\left(\dfrac{V_{ab}}{H_{ab}}\right) = \tan^{-1}\left(\dfrac{102\text{ kips}}{237\text{ kips}}\right) = 23.3°$

$$\mu = 1.0 + 0.5\sin^{1.5}\theta = 1.0 + 0.5\sin^{1.5}(23.2°) = 1.12$$

Manual Part 8

The weld strength per 1⁄16 in. is as follows:

LRFD	ASD
$\phi r_w = (1.392\text{ kips/in.})(1.12) = 1.56\text{ kips/in.}$ The peak weld stress is given by $f_{u\,peak} = \left(\dfrac{t}{2}\right)\sqrt{(f_{ua}+f_{ub})^2 + f_{uv}^{\;2}}$ $= \left(\dfrac{0.75\text{ in.}}{2}\right)\sqrt{(4.83\text{ ksi}+0\text{ ksi})^2 + (11.3\text{ ksi})^2}$ $= 4.61\text{ kips/in.}$ The average stress is $f_{u\,ave} = \dfrac{\dfrac{t}{2}\left[\sqrt{(f_{ua}-f_{ub})^2 + f_{uv}^{\;2}} + \sqrt{(f_{ua}+f_{ub})^2 + f_{uv}^{\;2}}\right]}{2}$ Since f_{ub} = 0 ksi (there is no moment on the edge) $f_{u\,ave} = f_{u\,peak}$ = 4.61 kips/in. The design weld stress is $f_{u\,weld} = \max\left\{f_{u\,peak},\ 1.25f_{u\,ave}\right\}$ $= \max\left\{\begin{array}{l}4.61\text{ kips/in.,}\\1.25(4.61\text{ kips/in.})\end{array}\right\}$ $= 5.76\text{ kips/in.}$ The required weld size is $D_{req} = \dfrac{f_{u\,weld}}{\phi r_w}$	$r_w/\Omega = (0.928\text{ kips/in.})(1.12) = 1.04\text{ kips/in.}$ The peak weld stress is given by $f_{a\,peak} = \left(\dfrac{t}{2}\right)\sqrt{(f_{aa}+f_{ab})^2 + f_{av}^{\;2}}$ $= \left(\dfrac{0.750\text{ in.}}{2}\right)\sqrt{(3.24\text{ ksi}+0\text{ ksi})^2 + (7.52\text{ ksi})^2}$ $= 3.07\text{ kips/in.}$ The average stress is $f_{a\,ave} = \dfrac{\dfrac{t}{2}\left[\sqrt{(f_{aa}-f_{ab})^2 + f_{av}^{\;2}} + \sqrt{(f_{aa}+f_{ab})^2 + f_{av}^{\;2}}\right]}{2}$ Since f_{ab} = 0 ksi (there is no moment on the edge) $f_{a\,ave} = f_{a\,peak}$ = 4.61 kips/in. The design weld stress is $f_{a\,weld} = \max\left\{f_{a\,peak},\ 1.25f_{a\,ave}\right\}$ $= \max\left\{\begin{array}{l}3.07\text{ kips/in.,}\\1.25(3.07\text{ kips/in.})\end{array}\right\}$ $= 3.84\text{ kips/in.}$ The required weld size is $D_{req} = \dfrac{\Omega f_{a\,weld}}{r_w}$

LRFD	ASD
$= \dfrac{5.76\text{ kips/in.}}{1.56\text{ kips/in.}} = 3.69 \rightarrow 4\text{ sixteenths}$ Minimum weld size is ¼ in. Use a ¼ in. fillet weld.	$= \dfrac{3.84\text{ kips/in.}}{1.04\text{ kips/in.}} = 3.69 \rightarrow 4\text{ sixteenths}$ Minimum weld size is ¼ in. Use a ¼ in. fillet weld.

Manual Table J2.4

LRFD	ASD
Check local web yielding of the beam $\phi R_n = \phi R_1 + N(\phi R_2)$ $= 99.0\text{ kips} + 42.0\text{ in.}(29.5\text{ kips/in.})$ $= 1340\text{ kips} > 152\text{ kips}$ **o.k.**	Check local web yielding of the beam $R_n / \Omega = R_1/\Omega + N\left(R_2/\Omega\right)$ $= 66.0\text{ kips} + 42.0\text{ in.}(19.7\text{ kips/in.})$ $= 893\text{ kips} > 102\text{ kips}$ **o.k.**

Manual Table 9-4

Check web crippling of the beam

$$\frac{N}{d} = \frac{42.0\text{ in.}}{18.7\text{ in.}} = 2.25 > 0.2$$

Manual Table 9-4

LRFD	ASD
$\phi R_n = \phi R_5 + N(\phi R_6)$ $= 143\text{ kips} + 42.0\text{ in.}(16.9\text{ kips/in.})$ $= 853\text{ kips} > 152\text{ kips}$ **o.k.**	$R_n / \Omega = R_5/\Omega + N\left(R_6/\Omega\right)$ $= 95.3\text{ kips} + 42.0\text{ in.}(11.2\text{ kips/in.})$ $= 566\text{ kips} > 102\text{ kips}$ **o.k.**

Manual Table 9-4

Design beam-to-column connection

Since the brace may be in tension or compression, the required strength of the beam-to-column connection is as follows. The required shear strength is

LRFD	ASD
$R_{ub} \pm V_{ub} = 15\text{ kips} + 152\text{ kips} = 167\text{ kips}$ and the required axial strength is $A_{ub} \pm \left(H_u - H_{ub}\right) = 0 \pm 171\text{ kips} = 171\text{ kips}$	$R_{ab} \pm V_{ab} = 10\text{ kips} + 102\text{ kips} = 112\text{ kips}$ and the required axial strength is $A_{ab} \pm \left(H_a - H_{ab}\right) = 0 \pm 114\text{ kips} = 114\text{ kips}$

Try 2L8×6×⅞×1'-2½" (leg gage = 3 1⁄16 in.) welded to the beam web, bolted with five rows of ⅞ in. diameter A325-N bolts in standard holes to the column flange.

Calculate tensile force per bolt

LRFD	ASD
$T_u = \dfrac{A_{ub} \pm \left(H_u - H_{ub}\right)}{n} = \dfrac{171\text{ kips}}{10\text{ bolts}}$ $= 17.1\text{ kips/bolt}$	$T_a = \dfrac{A_{ab} \pm \left(H_a - H_{ab}\right)}{n} = \dfrac{114\text{ kips}}{10\text{ bolts}}$ $= 11.4\text{ kips/bolt}$

Check available strength of bolts for tension-shear interaction

LRFD	ASD
$V_u = \dfrac{R_{ub} \pm V_{ub}}{n} = \dfrac{167\text{ kips}}{10\text{ bolts}}$ $= 16.7\text{ kips/bolt} < 21.6\text{ kips/bolt}$ **o.k.**	$V_a = \dfrac{R_{ab} \pm V_{ab}}{n} = \dfrac{112\text{ kips}}{10\text{ bolts}}$ $= 11.2\text{ kips/bolt} < 14.4\text{ kips/bolt}$ **o.k.**

Manual

$f_{uv} = \dfrac{V_u}{A_b} = \dfrac{16.7 \text{ kips/bolt}}{0.601 \text{ in.}^2 \text{ / bolt}} = 27.8 \text{ ksi}$	$f_{av} = \dfrac{V_a}{A_b} = \dfrac{11.2 \text{ kips/bolt}}{0.601 \text{ in.}^2 \text{ / bolt}} = 18.6 \text{ ksi}$	Table 7-1
$F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{uv} \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}}{0.75(48 \text{ ksi})}(27.8 \text{ ksi})$ $= 47.5 \text{ ksi} \le 90 \text{ ksi}$ **o.k.** Use $F'_{nt} = 47.5 \text{ ksi}$	$F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{av} \le F_{nt}$ $= 1.3(90 \text{ ksi}) - \dfrac{2.00(90 \text{ ksi})}{48 \text{ ksi}}(18.6 \text{ ksi})$ $= 47.3 \text{ ksi} \le 90 \text{ ksi}$ **o.k.** Use $F'_{nt} = 47.3 \text{ ksi}$	Table J3.2, Eqns. J3-3a and J3-3b
$B_u = \phi F'_{nt} A_b = 0.75(47.5 \text{ ksi})(0.601 \text{ in.}^2)$ $= 21.4 \text{ kips/bolt} > 17.1 \text{ kips/bolt}$ **o.k.**	$B_a = \dfrac{F'_{nt} A_b}{\Omega} = \dfrac{(47.3 \text{ ksi})(0.601 \text{ in.}^2)}{2.00}$ $= 14.2 \text{ kips/bolt} > 11.4 \text{ kips/bolt}$ **o.k.**	

LRFD	ASD	
Check bearing strength $L_c = 1.25 \text{ in.} - \dfrac{0.938 \text{ in.}}{2} = 0.781 \text{ in.}$	*Check bearing strength* $L_c = 1.25 \text{ in.} - \dfrac{0.938 \text{ in.}}{2} = 0.781 \text{ in.}$	
$\phi r_n = \phi 1.2 L_c t F_u < \phi 2.4 d t F_u$ $= 0.75(1.2)(0.781 \text{ in.})(0.875 \text{ in.})(58 \text{ ksi})$ $\le 0.75(2.4)(0.875 \text{ in.})(0.875 \text{ in.})(58 \text{ ksi})$ $= 35.7 \text{ kips} < 79.9 \text{ kips}$ $= 35.7 \text{ kips/bolt}$ Since this edge bolt value exceeds the single-shear strength of the bolts 21.6 kips, bearing does not control. **o.k.**	$r_n / \Omega = \dfrac{1.2 L_c t F_u}{\Omega} \le \dfrac{2.4 d t F_u}{\Omega}$ $= \dfrac{1.2(0.781 \text{ in.})(0.875 \text{ in.})(58 \text{ ksi})}{2.00}$ $\le \dfrac{2.4(0.875 \text{ in.})(0.875 \text{ in.})(58 \text{ ksi})}{2.00}$ $= 23.8 \text{ kips} < 53.3 \text{ kips}$ $= 23.8 \text{ kips/bolt}$ Since this edge bolt value exceeds the single-shear strength of the bolts 14.4 kips, bearing does not control. **o.k.**	Eqn. J3-6a

Check prying action

$$b = g - \frac{t}{2} = 3.06 \text{ in.} - \frac{0.875 \text{ in.}}{2} = 2.62 \text{ in.}$$

Manual Part 9

Note: 1⅜ in. entering and tightening clearance is accommodated, **o.k.**

Manual Table 7-16

$$a = 6.00 \text{ in.} - g = 6.00 \text{ in.} - 3.06 \text{ in.} = 2.94 \text{ in.}$$

$$b' = b - \frac{d_b}{2} = 2.62 \text{ in.} - \frac{0.875 \text{ in.}}{2} = 2.18 \text{ in.}$$

$$a' = \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right)$$
$$= \left(2.94\text{ in.} + \frac{0.875\text{ in.}}{2}\right) \le 1.25\left(2.62\text{ in.}\right) + \frac{0.875\text{ in.}}{2}$$
$$= 3.38\text{ in.} \le 3.71\text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{2.18\text{ in.}}{3.38\text{ in.}} = 0.645$$

$$p = \frac{14.5\text{ in.}}{5\text{ bolts}} = 2.90\text{ in./bolt}$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{0.938\text{ in.}}{2.90\text{ in.}} = 0.677$$

LRFD	ASD
$\beta = \frac{1}{\rho}\left(\frac{B_u}{T_u} - 1\right) = \frac{1}{0.645}\left(\frac{21.4\text{ kips}}{17.1\text{ kips}} - 1\right)$ $= 0.390$ $\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \le 1.0$ $= \frac{1}{0.677}\left(\frac{0.390}{1-0.390}\right) \le 1.0$ $= 0.944 \le 1.0$ $t_{req} = \sqrt{\frac{4.44T_u b'}{pF_u\left(1+\delta\alpha'\right)}}$ $= \sqrt{\frac{4.44\left(17.1\text{ kips/bolt}\right)\left(2.18\text{ in.}\right)}{\left(2.90\text{ in.}\right)\left(58\text{ ksi}\right)\left[1+\left(0.677\right)\left(0.944\right)\right]}}$ $= 0.775\text{ in.} < 0.875\text{ in.}$ **o.k.**	$\beta = \frac{1}{\rho}\left(\frac{B_a}{T_a} - 1\right) = \frac{1}{0.645}\left(\frac{14.2\text{ kips}}{11.4\text{ kips}} - 1\right)$ $= 0.381$ $\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \le 1.0$ $= \frac{1}{0.677}\left(\frac{0.381}{1-0.381}\right) \le 1.0$ $= 0.909 \le 1.0$ $t_{req} = \sqrt{\frac{6.66T_a b'}{pF_u\left(1+\delta\alpha'\right)}}$ $= \sqrt{\frac{6.66\left(11.4\text{ kips/bolt}\right)\left(2.18\text{ in.}\right)}{\left(2.90\text{ in.}\right)\left(58\text{ ksi}\right)\left[1+\left(0.677\right)\left(0.909\right)\right]}}$ $= 0.780\text{ in.} < 0.875\text{ in.}$ **o.k**

Design welds

Try fillet welds around perimeter (three sides) of both angles.

LRFD	ASD
$P_{uc} = \sqrt{\left(171\text{ kips}\right)^2 + \left(167\text{ kips}\right)^2} = 239\text{ kips}$ $\theta = \tan^{-1}\left(\frac{171\text{ kips}}{167\text{ kips}}\right) = 45.7°$	$P_{ac} = \sqrt{\left(114\text{ kips}\right)^2 + \left(112\text{ kips}\right)^2} = 160\text{ kips}$ $\theta = \tan^{-1}\left(\frac{114\text{ kips}}{112\text{ kips}}\right) = 45.5°$

For $\theta = 45°$, $l = 14.5$ in., $kl = 7.50$ in., and $k = 0.517$ — Manual Table 8-8

$$xl = \frac{(kl)^2}{l + 2kl} = \frac{(7.50\text{ in.})^2}{14.5\text{ in.} + 2(7.50\text{ in.})} = 1.91\text{ in.}$$

$$al = 8.00\text{ in.} - xl = 8.00\text{ in.} - 1.91\text{ in.} = 6.09\text{ in.}$$

a = 0.420

By interpolation

C = 3.55

LRFD	ASD
$D_{req} = \dfrac{P_{uc}}{\phi C C_1 l}$ $= \dfrac{239 \text{ kips}}{0.75(3.55)(1.0)(2 \text{ welds})(14.5 \text{ in.})}$ $= 3.10 \rightarrow 4 \text{ sixteenths}$ Minimum weld size is ¼ in. Use ¼-in. fillet welds.	$D_{req} = \dfrac{\Omega P_{ac}}{C C_1 l}$ $= \dfrac{2.00(160 \text{ kips})}{(3.55)(1.0)(2 \text{ welds})(14.5 \text{ in.})}$ $= 3.11 \rightarrow 4 \text{ sixteenths}$ Minimum weld size is ¼ in. Use ¼-in. fillet welds.

Table J2.4

Check beam web thickness (against weld size required for strength)

For two fillet welds

$$t_{\min} = \frac{6.19D}{F_u} = \frac{6.19(3.11 \text{ sixteenths})}{65 \text{ ksi}} = 0.296 \text{ in.} < 0.590 \text{ in.} \qquad \textbf{o.k.}$$

Manual Part 9

Check the strength of the angles

Shear yielding

$$A_g = (2)(14.5 \text{ in.})(0.875 \text{ in.}) = 25.4 \text{ in.}^2$$

Eqn. J4-3

LRFD	ASD
$\phi R_n = \phi(0.60 F_y A_g)$ $= 1.0(0.60)(36 \text{ ksi})(25.4 \text{ in.}^2)$ = 549 kips > 167 kips **o.k.**	$R_n / \Omega = \dfrac{0.60 F_y A_g}{\Omega}$ $= \dfrac{0.60(36 \text{ ksi})(25.4 \text{ in.}^2)}{1.50}$ = 366 kips > 112 kips **o.k.**

Similarly, shear yielding of the angles due to H_{uc} is not critical.

Shear rupture

$$A_{nv} = (0.875 \text{ in.})\left[2(14.5 \text{ in.}) - 10(1.00 \text{ in.})\right] = 16.6 \text{ in.}^2$$

Eqn. J4-4

LRFD	ASD
$\phi R_n = \phi(0.60 F_u A_{nv})$ $= 0.75(0.60)(58 \text{ ksi})(16.6 \text{ in.}^2)$ = 433 kips > 167 kips **o.k.**	$R_n / \Omega = \dfrac{0.60 F_u A_{nv}}{\Omega}$ $= \dfrac{0.60(58.0 \text{ ksi})(16.6 \text{ in.}^2)}{2.00}$ = 289 kips > 112 kips **o.k.**

Block shear rupture

With n = 5, L_{ev} = 1.25 in., L_{eh} = 2.94 in. Section J4.3

$$A_{nt} = \left[(2.94\text{ in.}) - (0.5)(1.00\text{ in.})\right](0.875\text{ in.})(2) = 4.27\text{ in.}^2$$

$$A_{gv} = (13.3\text{ in.})(0.875\text{ in.})(2) = 23.3\text{ in.}^2$$

$$A_{nv} = \left[(13.3\text{ in.}) - (4.50)(1.00\text{ in.})\right](0.875\text{ in.})(2) = 15.4\text{ in.}^2$$

$$F_u A_{nt} = (58\text{ ksi})(4.27\text{ in.}^2) = 248\text{ kips}$$

$$0.60F_y A_{gv} = 0.60(36\text{ ksi})(23.3\text{ in.}^2) = 503\text{ kips} \quad \textbf{controls}$$

$$0.60F_u A_{nv} = 0.6(58\text{ ksi})(15.4\text{ in.}^2) = 536\text{ kips}$$

LRFD	ASD
$\phi R_n = 0.75(248\text{ kips} + 503\text{ kips})$ $= 563\text{ kips} > 167\text{ kips}$ **o.k.**	$R_n / \Omega = \dfrac{(248\text{ kips} + 503\text{ kips})}{2.00}$ $= 376\text{ kips} > 112\text{ kips}$ **o.k.**

Eqn. J4-5

Check column flange

By inspection, the 4.16 in. thick column flange has adequate flexural strength, stiffness, and bearing strength.

Note: When the brace is in compression, the buckling strength of the gusset would have to be checked, where

LRFD	ASD
$\phi R_n = \phi_c F_{cr} A_w$	$R_n / \Omega = F_{cr} A_w / \Omega_c$

In the above equation, $\phi_c F_{cr}$ or F_{cr}/Ω_c may be determined from $\dfrac{Kl_1}{r}$ with Specification Section E3 or J4.4, where l_1 is the perpendicular distance from the Whitmore section to the interior edge of the gusset. Alternatively, the average value of $l = \dfrac{l_1 + l_2 + l_3}{3}$ may be substituted, where these quantities are illustrated in the figure. Note that, for this example l_2 is negative since part of the Whitmore section is in the beam web.

The effective length factor K has been established as 0.5 by full scale tests on bracing connections (Gross, 1990). It assumes that the gusset is supported on both edges. In cases where the gusset is supported on one edge only, such as illustrated in Example 11.C-3, Figure (d), the brace can more readily move out-of-plane and a sidesway mode of buckling can occur in the gusset. For this case, K should be taken as 1.2.

Check gusset buckling

The area of the Whitmore section is

$$A_w = (30.9\text{ in.})(0.750\text{ in.}) + (3.90\text{ in.})(0.590\text{ in.})\left(\frac{50\text{ ksi}}{36\text{ ksi}}\right) = 26.4\text{ in.}^2$$

In the above equation, the area in the beam web is multiplied by the ratio 50/36 to convert the area to an equivalent area of A36 plate.

The slenderness ratio is $\dfrac{Kl_1}{r} = \dfrac{0.5(17.0\text{ in.})\left(\sqrt{12}\right)}{0.750\text{ in.}} = 39.3$

From Specification Section E3

$$F_e = \frac{\pi^2 E}{\left(\dfrac{Kl}{r}\right)^2} = \frac{\pi^2(29000\text{ ksi})}{39.3^2} = 185\text{ ksi}$$ Eqn. E3-4

Check $4.71\sqrt{\dfrac{E}{F_y}} = 4.71\sqrt{\dfrac{29000\text{ ksi}}{36\text{ ksi}}} = 134$

The buckling stress is given by $F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{36\text{ ksi}}{185\text{ ksi}}}\right](36\text{ ksi}) = 33.2\text{ ksi}$ Eqn. E3-2

$$R_n = F_{cr}A_g = (33.2\text{ ksi})(26.4\text{ in.}^2) = 876\text{ kips}$$ Eqn. E3-1

LRFD	ASD
$\phi_c = 0.90$ $\phi_c R_n = 0.90(876\text{ kips})$ $= 788\text{ kips} > 675\text{ kips}$ **o.k.**	$\Omega_c = 1.67$ $\dfrac{R_n}{\Omega_c} = \dfrac{876\text{ kips}}{1.67} = 525\text{ kips} > 450\text{ kips}$ **o.k.**

Section E1

Example II.C-3 Bracing Connection

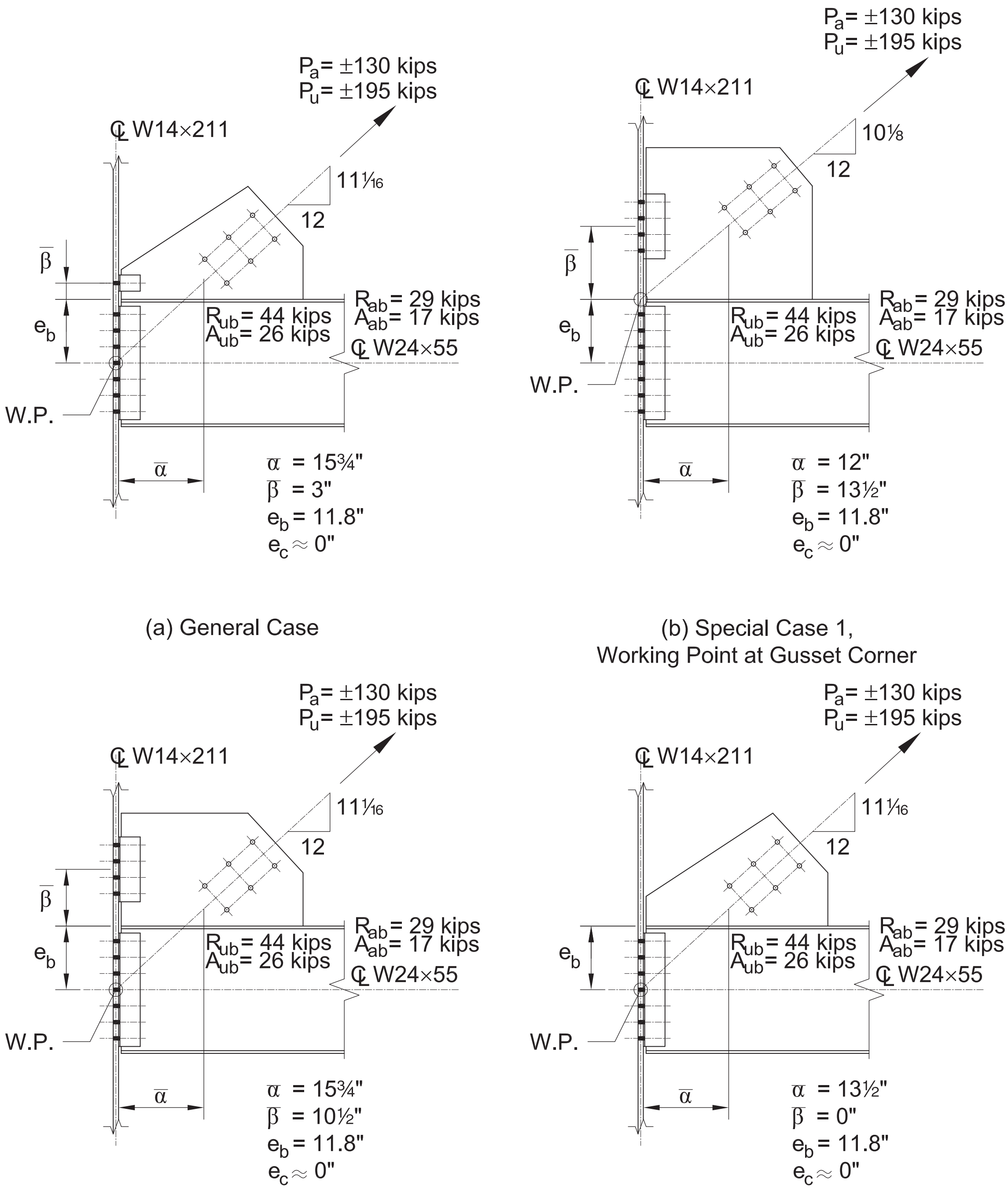
Given:

Each of the four designs shown for the diagonal bracing connection between the W14×68 brace, W24×55 beam, and W14×211 column web have been developed using the Uniform Force Method (the General Case and Special Cases 1, 2, and 3) for the load case of 1.2D + 1.3W for LRFD and D + W for ASD.

For the given values of α and β, determine the interface forces on the gusset-to-column and gusset-to-beam connections for

a. General Case of Figure (a)
b. Special Case 1 of Figure (b)
c. Special Case 2 of Figure (c)
d. Special Case 3 of Figure (d)

Brace Axial Load	P_u = ±195 kips	P_a = ±130 kips
Beam End Reaction	R_u = 44 kips	R_a = 29 kips
Beam Axial Load	A_u = 26 kips	A_a = 17 kips



(a) General Case

(b) Special Case 1,
Working Point at Gusset Corner

(c) Special Case 2, $\Delta V_{ub} = V_{ub}$ i.e.
Shear in Beam-to-Column
Connection Minimized

(d) Special Case 3,
No Gusset-to-Column Web Connection

Material Properties:

Brace	W14×68	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Beam	W24×55	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Column	W14×211	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Gusset Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Brace	W14×68	A=20.0 in.^{2}	d=14.0 in.	t_w=0.415 in.	b_f=10.0 in.	t_f=0.720 in.	Manual Table 1-1
Beam	W24×55	d=23.6 in.	t_w=0.395 in.	b_f=7.01 in.	t_f=0.505 in.	k_{des}=1.01 in.	
Column	W14×211	d=15.7 in.	t_w=0.980 in.	b_f=15.8 in.	t_f=1.56 in.		

Solution A (General Case):

Assume $\beta = \bar{\beta} = 3.00$ in.

$$\alpha = e_b \tan\theta - e_c + \beta\tan\theta = (11.8\text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) - 0 + (3.00\text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) = 16.1\text{ in.}$$

Manual Part 13

Since $\alpha \neq \bar{\alpha}$, an eccentricity exists on the gusset-to-beam connection.

Calculate the interface forces

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(16.1\text{ in.} + 0\text{ in.})^2 + (3.00\text{ in.} + 11.8\text{ in.})^2} = 21.9\text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = \dfrac{\beta}{r}P_u = \dfrac{3.00\text{ in.}}{21.9\text{ in.}}(195\text{ kips}) = 26.7\text{ kips}$ $H_{uc} = \dfrac{e_c}{r}P_u = 0\text{ kips}$	$V_{ac} = \dfrac{\beta}{r}P_a = \dfrac{3.00\text{ in.}}{21.9\text{ in.}}(130\text{ kips}) = 17.8\text{ kips}$ $H_{ac} = \dfrac{e_c}{r}P_a = 0\text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \dfrac{\alpha}{r}P_u = \dfrac{16.1\text{ in.}}{21.9\text{ in.}}(195\text{ kips}) = 143\text{ kips}$ $V_{ub} = \dfrac{e_b}{r}P_u = \dfrac{11.8\text{ in.}}{21.9\text{ in.}}(195\text{ kips}) = 105\text{ kips}$ $M_{ub} = V_{ub}\left(\alpha - \bar{\alpha}\right)$ $= \dfrac{(105\text{ kips})(15\frac{3}{4}\text{ in.} - 16.1\text{ in.})}{12\text{ in./ft}}$ = -3.06 kip-ft	$H_{ab} = \dfrac{\alpha}{r}P_a = \dfrac{16.1\text{ in.}}{21.9\text{ in.}}(130\text{ kips}) = 95.6\text{ kips}$ $V_{ab} = \dfrac{e_b}{r}P_a = \dfrac{11.8\text{ in.}}{21.9\text{ in.}}(130\text{ kips}) = 70.0\text{ kips}$ $M_{ab} = V_{ab}\left(\alpha - \bar{\alpha}\right)$ $= \dfrac{(70.0\text{ kips})(15\frac{3}{4}\text{ in.} - 16.1\text{ in.})}{12\text{ in./ft}}$ = -2.04 kip-ft

In this case, this small moment is negligible.

On the beam-to-column connection, the requried shear strength is

LRFD	ASD
$R_{ub} + V_{ub}$ = 44.0 kips + 105 kips = 149 kips	$R_{ab} + V_{ab}$ = 29.0 kips + 70.0 kips = 99.0 kips

and the requried axial strength is

LRFD	ASD
$A_{ub} + H_{uc}$ = 26.0 kips ± 0 kips = 26.0 kips	$A_{ab} + H_{ac}$ = 17.0 kips ± 0 kips = 17.0 kips

For a discussion of the sign use between A_{ub} and H_{uc} (A_{ab} and H_{ac} for ASD), refer to AISC (1992).

Solution B (Special Case 1):

In this case, the centroidal positions of the gusset-edge connections are irrelevant; $\bar{\alpha}$ and $\bar{\beta}$ are given to define the geometry of the connection, but are not needed to determine the gusset edge forces. Manual Part 13

The angle of the brace from the vertical is

$$\theta = \tan^{-1}\left(\frac{12}{10\frac{1}{8}}\right) = 49.8°$$

The horizontal and vertical components of the brace force are

LRFD	ASD
$H_u = P_u \sin\theta = (195 \text{ kips})\sin 49.8° = 149 \text{ kips}$ $V_u = P_u \cos\theta = (195 \text{ kips})\cos 49.8° = 126 \text{ kips}$	$H_a = P_a \sin\theta = (130 \text{ kips})\sin 49.8° = 99.3 \text{ kips}$ $V_a = P_a \cos\theta = (130 \text{ kips})\cos 49.8° = 83.9 \text{ kips}$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = V_u = 126 \text{ kips}$ $H_{uc} = 0 \text{ kips}$	$V_{ac} = V_a = 83.9 \text{ kips}$ $H_{ac} = 0 \text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$V_{ub} = 0 \text{ kips}$ $H_{ub} = H_u = 149 \text{ kips}$	$V_{ab} = 0 \text{ kips}$ $H_{ab} = H_a = 99.3 \text{ kips}$

On the beam-to-column connection

LRFD	ASD
R_{ub} = 44.0 kips (shear) A_{ub} = 26.0 kips (axial transfer force)	R_{ab} = 29.0 kips (shear) A_{ab} = 17.0 kips (axial transfer force)

In addition to the forces on the connection interfaces, the beam is subjected to a moment M_{ub} or M_{ab}, where

LRFD	ASD
$M_{ub} = H_{ub}e_b = \dfrac{(149 \text{ kips})(11.8 \text{ in.})}{12 \text{ in./ft}}$ $= 147 \text{ kip-ft}$	$M_{ab} = H_{ab}e_b = \dfrac{(99.3 \text{ kips})(11.8 \text{ in.})}{12 \text{ in./ft}}$ $= 97.6 \text{ kip-ft}$

This moment, as well as the beam axial load H_{ub} = 149 kips or H_{ab} = 99.3 kips and the moment and shear in the beam associated with the end reaction R_{ub} or R_{ab}, must be considered in the design of the beam.

Solution C (Special Case 2):

Assume $\beta = \bar{\beta} = 10.5$ in.

$$\alpha = e_b \tan\theta - e_c + \beta\tan\theta = (11.8 \text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) - 0 + (10.5 \text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) = 24.2 \text{ in.}$$

Manual Part 13

Calculate the interface forces for the general case before applying Special Case 2.

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(24.2 \text{ in.} + 0 \text{ in.})^2 + (10.5 \text{ in.} + 11.8 \text{ in.})^2} = 32.9 \text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = \frac{\beta}{r}P_u = \frac{10.5 \text{ in.}}{32.9 \text{ in.}}(195 \text{ kips}) = 62.2 \text{ kips}$ $H_{uc} = \frac{e_c}{r}P_u = 0 \text{ kips}$	$V_{ac} = \frac{\beta}{r}P_a = \frac{10.5 \text{ in.}}{32.9 \text{ in.}}(130 \text{ kips}) = 41.5 \text{ kips}$ $H_{ac} = \frac{e_c}{r}P_a = 0 \text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \frac{\alpha}{r}P_u = \frac{24.2 \text{ in.}}{32.9 \text{ in.}}(195 \text{ kips}) = 143 \text{ kips}$ $V_{ub} = \frac{e_b}{r}P_u = \frac{11.8 \text{ in.}}{32.9 \text{ in.}}(195 \text{ kips}) = 69.9 \text{ kips}$	$H_{ab} = \frac{\alpha}{r}P_a = \frac{24.2 \text{ in.}}{32.9 \text{ in.}}(130 \text{ kips}) = 95.6 \text{ kips}$ $V_{ab} = \frac{e_b}{r}P_a = \frac{11.8 \text{ in.}}{32.9 \text{ in.}}(130 \text{ kips}) = 46.6 \text{ kips}$

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub}$ = 44.0 kips + 69.9 kips = 114 kips	$R_{ab} + V_{ab}$ = 29.0 kips + 46.6 kips = 75.6 kips

and the axial force is

LRFD	ASD
$A_{ub} + H_{uc}$ = 26.0 kips ± 0 kips = 26.0 kips	$A_{ab} + H_{ac}$ = 17.0 kips ± 0 kips = 17.0 kips

Next, applying Special Case 2 with $\Delta V_{ub} = V_{ub}$ = 69.9 kips ($\Delta V_{ab} = V_{ab}$ = 46.6 kips for ASD), calculate the interface forces.

On the gusset-to-column connection (where V_{uc} is replaced by $V_{uc} + \Delta V_{ub}$) or (where V_{ac} is replaced by $V_{ac} + \Delta V_{ab}$ for ASD)

LRFD	ASD
$V_{uc} = 62.2 \text{ kips} + 69.9 \text{ kips} = 132 \text{ kips}$ $H_{uc} = 0 \text{ kips}$ (unchanged)	$V_{ac} = 41.5 \text{ kips} + 46.6 \text{ kips} = 88.1 \text{ kips}$ $H_{ac} = 0 \text{ kips}$ (unchanged)

On the gusset-to-beam connection (where V_{ub} is replaced by $V_{ub} - \Delta V_{ub}$) or (where V_{ab} is replaced by $V_{ab} - \Delta V_{ab}$)

LRFD	ASD
$H_{ub} = 143 \text{ kips (unchanged)}$ $V_{ub} = 69.9 \text{ kips} - 69.9 \text{ kips} = 0 \text{ kips}$ $M_{ub} = \left(\Delta V_{ub}\right)\alpha = \dfrac{(69.9 \text{ kips})(24.2 \text{ in.})}{12 \text{ in./ft}}$ $= 141 \text{ kip-ft}$	$H_{ab} = 95.6 \text{ kips (unchanged)}$ $V_{ab} = 46.6 \text{ kips} - 46.6 \text{ kips} = 0 \text{ kips}$ $M_{ab} = \left(\Delta V_{ab}\right)\alpha = \dfrac{(46.6 \text{ kips})(24.2 \text{ in.})}{12 \text{ in./ft}}$ $= 94.0 \text{ kip-ft}$

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub} - \Delta V_{ub}$ $= 44.0 \text{ kips} + 69.9 \text{ kips} - 69.9 \text{ kips}$ $= 44.0 \text{ kips}$	$R_{ab} + V_{ab} - \Delta V_{ab}$ $= 29.0 \text{ kips} + 46.6 \text{ kips} - 46.6 \text{ kips}$ $= 29.0 \text{ kips}$

and the axial force is

LRFD	ASD
$A_{ub} + H_{uc} = 26.0 \text{ kips} \pm 0 \text{ kips} = 26.0 \text{ kips}$	$A_{ab} + H_{ac} = 17.0 \text{ kips} \pm 0 \text{ kips} = 17.0 \text{ kips}$

Solution D (Special Case 3):

Set $\beta = \bar{\beta} = 0$ in.

$$\alpha = e_b \tan\theta = (11.8\text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) = 12.8\text{ in.}$$

Since, $\alpha \neq \bar{\alpha}$, an eccentricity exists on the gusset-to-beam connection.

Calculate the interface forces

$$r = \sqrt{\alpha^2 + e_b{}^2} = \sqrt{(12.8\text{ in.})^2 + (11.8\text{ in.})^2} = 17.4\text{ in.}$$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \dfrac{\alpha}{r}P_u = \dfrac{12.8\text{ in.}}{17.4\text{ in.}}(195\text{ kips}) = 143\text{ kips}$	$H_{ab} = \dfrac{\alpha}{r}P_a = \dfrac{12.8\text{ in.}}{17.4\text{ in.}}(130\text{ kips}) = 95.6\text{ kips}$
$V_{ub} = \dfrac{e_b}{r}P_u = \dfrac{11.8\text{ in.}}{17.4\text{ in.}}(195\text{ kips}) = 132\text{ kips}$	$V_{ab} = \dfrac{e_b}{r}P_a = \dfrac{11.8\text{ in.}}{17.4\text{ in.}}(130\text{ kips}) = 88.2\text{ kips}$
$M_{ub} = V_{ub}(\alpha - \bar{\alpha}) = \dfrac{(132\text{ kips})(12.8\text{ in.} - 13.5\text{ in.})}{12\text{ in./ft}}$ = -7.70 kip-ft	$M_{ab} = V_{ab}(\alpha - \bar{\alpha}) = \dfrac{(88.2\text{ kips})(12.8\text{ in.} - 13.5\text{ in.})}{12\text{ in./ft}}$ = -5.15 kip-ft

In this case, this small moment is negligible.

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub}$ = 44.0 kips + 132 kips = 176 kips	$R_{ab} + V_{ab}$ = 29.0 kips + 88.2 kips = 117 kips

And the axial force is

LRFD	ASD
$A_{ub} + H_{uc}$ = 26 kips ± 0 kips = 26.0 kips	$A_{ab} + H_{ac}$ = 17 kips ± 0 kips = 17.0 kips

Note: From the foregoing results, designs by Special Case 3 and the General Case of the Uniform Force Method provide the more economical designs. Additionally, note that designs by Special Case 1 and Special Case 2 result in moments on the beam and/ or column that must be considered.

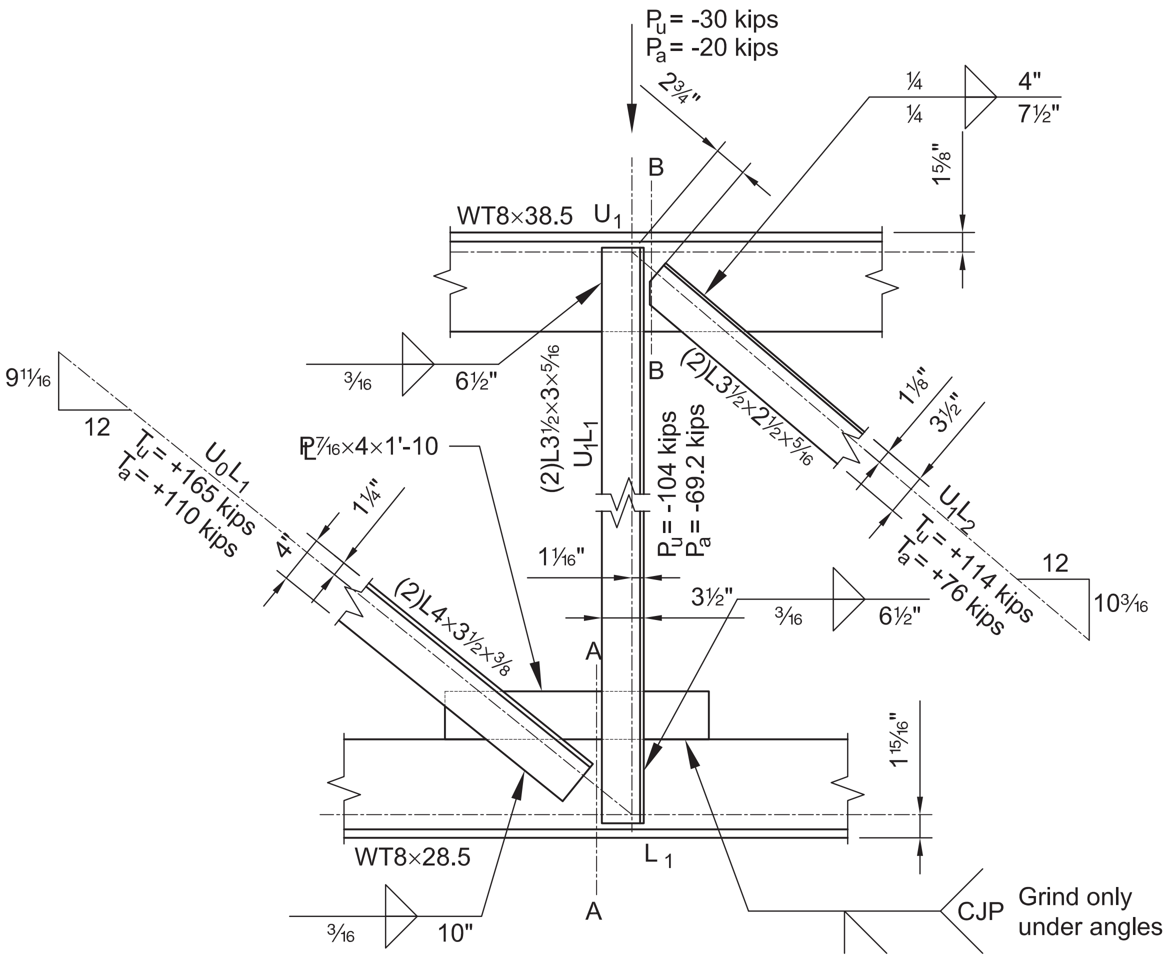
Example II.C-4 Truss Support Connection

Given:

Determine the requirements for the following cases:

a. Joint L_1
b. Joint U_1

Use E70 electrodes.



Material Properties:

Top Chord	WT8×38.5	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
Bottom Chord	WT8×28.5	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
Diagonal U_0L_1	2L4×3½×⅜	ASTM A36	F_y = 36 ksi	F_u = 58 ksi
Web U_1L_1	2L3½×3×5⁄16	ASTM A36	F_y = 36 ksi	F_u = 58 ksi
Diagonal U_1L_2	2L3½×2½×5⁄16	ASTM A36	F_y = 36 ksi	F_u = 58 ksi
Plate	PL7⁄16×4×1'–10"	ASTM A36	F_y = 36 ksi	F_u = 58 ksi

Manual Tables 2-3 and 2-4

Geometric Properties:

Top Chord	WT8×38.5	$t_w = 0.455$ in.	$d = 8.26$ in.	Manual Tables 1-7, 1-8 and 1-15
Bottom Chord	WT8×28.5	$t_w = 0.430$ in.	$d = 8.22$ in.	
Diagonal U_0L_1	2L4×3½×⅜	$A = 5.35$ in.^{2}	$\bar{x} = 0.947$ in.	
Web U_1L_1	2L3½×3×5⁄16	$A = 3.91$ in.^{2}		
Diagonal U_1L_2	2L3½×2½×5⁄16	$A = 3.58$ in.^{2}	$\bar{x} = 0.632$ in.	

LRFD	ASD
Web U_1L_1 load $R_u = -104$ kips Diagonal U_0L_1 load $R_u = +165$ kips Diagonal U_1L_2 load $R_u = +114$ kips	Web U_1L_1 load $R_a = -69.2$ kips Diagonal U_0L_1 load $R_a = +110$ kips Diagonal U_1L_2 load $R_a = +76.0$ kips

Solution a:

Check shear yielding of bottom chord tee stem (on Section A-A)

$$R_n = 0.6F_yA_w = 0.6(50 \text{ ksi})(8.22 \text{ in.})(0.430 \text{ in.}) = 106 \text{ kips}$$

Eqn. J4-3

LRFD	ASD
$\phi R_n = 1.00(106 \text{ kips}) = 106 \text{ kips}$ 106 kips > 104 kips **o.k.**	$R_n / \Omega = \dfrac{106 \text{ kips}}{1.50} = 70.7 \text{ kips}$ 70.7 kips > 69.2 kips **o.k.**

Design welds for member U_1L_1

User Note: Specification Section J1.7 requiring that the center of gravity of the weld group coincide with the center of gravity of the member does not apply to end connections of statically loaded single angle, double angle and similar members.

Section J1.7

The minimum weld size is $w_{min} = \frac{3}{16}$ in.

Table J2.4

The maximum weld size is $w_{max} =$ thickness - $\frac{1}{16}$ in. = $\frac{1}{4}$ in.

Calculate the minimum length of 3⁄16-in. fillet weld.

LRFD	ASD
$L_{min} = \dfrac{R_u}{1.392D} = \dfrac{104 \text{ kips}}{1.392(3 \text{ sixteenths})}$ $= 24.9$ in.	$L_{min} = \dfrac{R_a}{0.928D} = \dfrac{69.2 \text{ kips}}{0.928(3 \text{ sixteenths})}$ $= 24.9$ in.

Manual Part 8

Use 6½ in. of 3⁄16-in. weld at the heel and toe of both angles for a total of 26 in.

Check the minimum angle thickness to match the required shear rupture strength of the welds.

Manual Part 9

$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09(3\text{ sixteenths})}{58\text{ ksi}} = 0.160\text{ in.} < \tfrac{5}{16}\text{ in.} \qquad \textbf{o.k.}$$

Check the minimum stem thickness to match the required shear rupture strength of the welds. Manual Part 9

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(3\text{ sixteenths})}{65\text{ ksi}} = 0.286\text{ in.} < 0.430\text{ in.} \qquad \textbf{o.k.}$$

Top and bottom chords are o.k.

Table J2.4

Design welds for member U_0L_1

The minimum weld size is $w_{min} = \tfrac{3}{16}$ in.

The maximum weld size is $w_{max} =$ thickness - $\tfrac{1}{16}$ in. $= \tfrac{5}{16}$ in.

Calculate the minimum length of $\tfrac{3}{16}$ in. fillet weld.

LRFD	ASD
$L_{min} = \dfrac{R_u}{1.392D} = \dfrac{165\text{ kips}}{1.392(3\text{ sixteenths})}$ $= 39.5$ in.	$L_{min} = \dfrac{R_a}{0.928D} = \dfrac{110\text{ kips}}{0.928(3\text{ sixteenths})}$ $= 39.5$ in.

Use 10 in. of $\tfrac{3}{16}$ in. weld at the heel and toe of both angles for a total of 40 in.

Note: A plate will be welded to the stem of the WT to provide room for the connection. Based on the calculations above for the minimum angle and stem thicknesses, by inspection the angles, stems, and stem plate extension have adequate strength.

Eqn. J4-1

Check tension yielding of diagonal U_0L_1

$$R_n = F_yA_g = (36\text{ ksi})(5.35\text{ in.}^2) = 193\text{ kips}$$

LRFD	ASD
$\phi R_n = 0.90(193\text{ kips}) = 174\text{ kips}$ 174 kips > 165 kips **o.k.**	$R_n/\Omega = \dfrac{193\text{ kips}}{1.67} = 115\text{ kips}$ 116 kips > 110 kips **o.k.**

Check tension rupture of diagonal U_0L_1

Table D3.1 Case 2

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.947\text{ in.}}{10.0\text{ in.}} = 0.905$$

$$R_n = F_uA_e = (58\text{ ksi})(0.905)(5.35\text{ in.}^2) = 281\text{ kips}$$

Eqn. J4-2

LRFD	ASD
$\phi R_n = 0.75(281\text{ kips}) = 211\text{ kips}$ 211 kips > 165 kips **o.k.**	$R_n/\Omega = \dfrac{281\text{ kips}}{2.00} = 141\text{ kips}$ 141 kips > 110 kips **o.k.**

Check block shear rupture of the bottom chord

$A_{nt} = (4.0 \text{ in.})(0.430 \text{ in.}) = 1.72 \text{ in.}^2$ — Table D3.1 Case 2

$A_{nv} = (10.0 \text{ in.})(0.430 \text{ in.})(2) = 8.60 \text{ in.}^2$

$$\begin{aligned} R_n &= F_u A_{nt} + 0.6F_y A_{nv} \\ &= (65 \text{ ksi})(1.72 \text{ in.}^2) + 0.6(36 \text{ ksi})(8.60 \text{ in.}^2) \\ &= 112 \text{ kips} + 186 \text{ kips} \\ &= 298 \text{ kips} \end{aligned}$$ Eqn. J4-2

Because a A36 plate was used for the stem extension plate, use $F_y = 36$ ksi.

LRFD	ASD
$\phi R_n = 0.75(298 \text{ kips}) = 224 \text{ kips}$ 224 kips > 165 kips **o.k.**	$R_n / \Omega = \dfrac{298 \text{ kips}}{2.00} = 149 \text{ kips}$ 149 kips > 110 kips **o.k.**

Solution b:

Check shear yielding of top chord tee stem (on Section B-B)

$R_n = 0.6F_y A_w = 0.6(50 \text{ ksi})(8.26 \text{ in.})(0.455 \text{ in.}) = 113 \text{ kips}$ — Eqn. J4-3

LRFD	ASD
$\phi R_n = 1.00(113 \text{ kips}) = 113 \text{ kips}$ 113 kips > 74.0 kips **o.k.**	$R_n / \Omega = \dfrac{113 \text{ kips}}{1.50} = 75.3 \text{ kips}$ 75.3 kips > 49.2 kips **o.k.**

Design welds for member U_1L_1

As calculated previously in Solution a, use 6½ in. of 3⁄16-in. weld at the heel and toe of both angles for a total of 26 in.

Design welds for member U_1L_2

The minimum weld size is $w_{min} = \frac{3}{16}$ in. — Table J2.4

The maximum weld size is $w_{max} = \frac{1}{4}$ in.

Calculate the minimum length of ¼ in. fillet weld.

LRFD	ASD
$L_{min} = \dfrac{R_u}{1.392D} = \dfrac{114 \text{ kips}}{1.392(4 \text{ sixteenths})}$ $= 20.5$ in.	$L_{min} = \dfrac{R_a}{0.928D} = \dfrac{76.0 \text{ kips}}{0.928(4 \text{ sixteenths})}$ $= 20.5$ in.

Use 7½ in. of ¼-in. fillet weld at the heel and 4 in. of fillet weld at the toe of each angle for a total of 23 in.

Check the minimum angle thickness to match the required shear rupture strength of the welds.

Manual

$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09(4 \text{ sixteenths})}{58 \text{ ksi}} = 0.213 \text{ in.} < \tfrac{5}{16} \text{ in.} \quad \textbf{o.k.}$$

Part 9

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(4 \text{ sixteenths})}{58 \text{ ksi}} = 0.427 \text{ in.} < 0.455 \text{ in.} \quad \textbf{o.k.}$$

Manual Part 9

Check tension yielding of diagonal U_1L_2

$$R_n = F_y A_g = (36 \text{ ksi})(3.58 \text{ in.}^2) = 129 \text{ kips}$$

Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.90(129 \text{ kips}) = 116 \text{ kips}$ 116 kips > 114 kips **o.k.**	$R_n / \Omega = \dfrac{129 \text{ kips}}{1.67} = 77.2 \text{ kips}$ 77.2 kips > 76 kips **o.k.**

Check tension rupture of diagonal U_1L_2

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.632 \text{ in.}}{\left(\dfrac{7.50 \text{ in.} + 4.00 \text{ in.}}{2}\right)} = 0.890$$

Table D3.1 Case 2

$$R_n = F_u A_e = (58 \text{ ksi})(0.890)(3.58 \text{ in.}^2) = 185 \text{ kips}$$

Eqn. J4-2

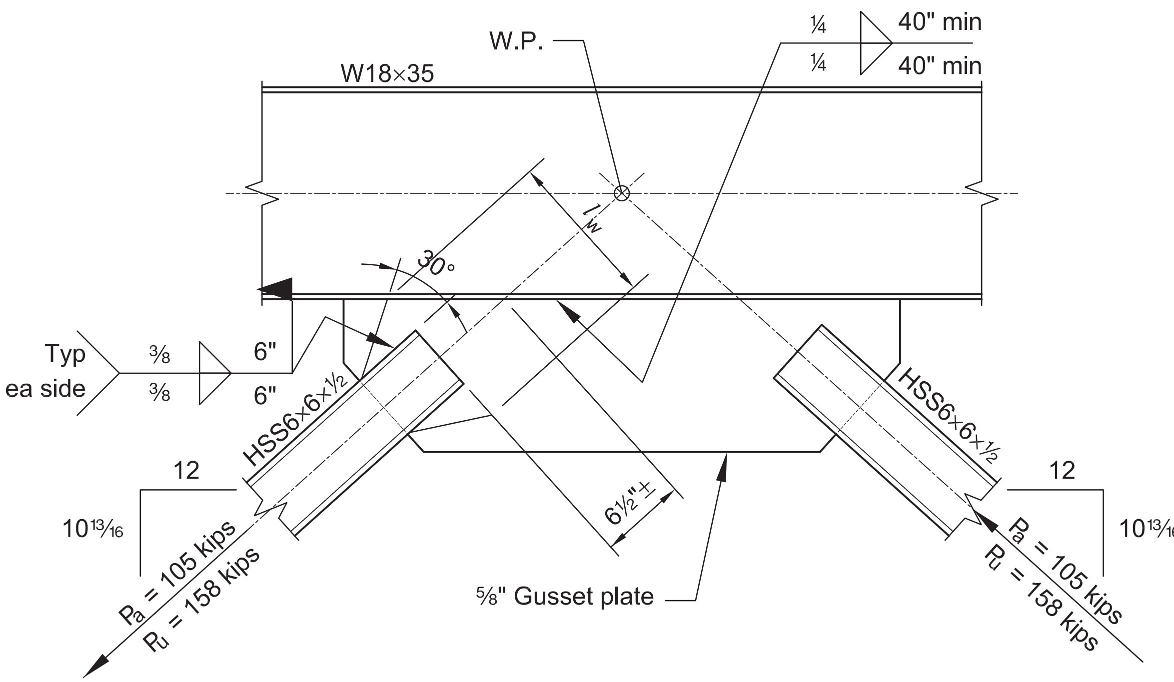
LRFD	ASD
$\phi R_n = 0.75(185 \text{ kips}) = 139 \text{ kips}$ 139 kips > 114 kips **o.k.**	$R_n / \Omega = \dfrac{185 \text{ kips}}{2.00} = 92.5 \text{ kips}$ 92.5 kips > 76.0 kips **o.k.**

Example II.C-5 HSS Chevron Brace Connection

Given:

Check the HSS braces for tension and compression and design the connection including welding, shear lag requirements and check to see if stiffening is required.

Use E70 electrodes.



Material Properties:

Beam W18×35	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Brace HSS6×6×½	ASTM A500 Gr.B	F_y = 46 ksi	F_u = 58 ksi	
Gusset Plate	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam W18×35	d = 17.7 in.	t_w = 0.300 in.	k_{des} = 0.827 in.		Manual Tables 1-1 and 1-12
Brace HSS6×6×½	H = 6.00 in.	B = 6.00 in.	A = 9.74 in.^{2}	t = 0.465 in.	

Solution:

Determine the required brace to gusset weld size.

LRFD	ASD
$D_{req} = \dfrac{R_u}{1.392l}$ $= \dfrac{158 \text{ kips}}{1.392(4)(6.00 \text{ in.})}$ $= 4.73$ sixteenths Increase the weld size by 1⁄16 in. to account	$D_{req} = \dfrac{R_a}{0.928l}$ $= \dfrac{105 \text{ kips}}{0.928(4)(6.00 \text{ in.})}$ $= 4.71$ sixteenths Increase the weld size by 1⁄16 in. to account

Manual Part 8

LRFD	ASD
for slot in the HSS D_{req} = 4.73 sixteenths + 1.00 sixteenths = 5.73 sixteenths	for the slot in the HSS D_{req} = 4.71 sixteenths + 1.00 sixteenth = 5.71 sixteenths

The minimum weld size for this connection is 3⁄16 in. Manual Table J2.4

Use 3⁄8 in. fillet welds

Determine the minimum gusset plate thickness to match the required shear rupture strength of the fillet welds.

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(4.73 \text{ sixteenths})}{58 \text{ ksi}} = 0.505 \text{ in.}$$ Manual Part 9

Try a 5⁄8 in. thick gusset plate.

Determine the minimum HSS brace thickness to match the required shear rupture strength of the fillet welds.

$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09(4.73 \text{ sixteenths})}{58 \text{ ksi}} = 0.252 \text{ in.} < \tfrac{1}{2} \text{ in.} \quad \textbf{o.k.}$$

Check gusset plate buckling (compression brace)

$$r = \frac{t_p}{\sqrt{12}} = \frac{0.625 \text{ in.}}{\sqrt{12}} = 0.180 \text{ in.}$$

From the figure, the distance l_1 = 6.50 in.

Since the gusset is attached by one edge only, the buckling mode could be a sidesway type as shown in Commentary Table C-C2.2. In this case, use K = 1.2.

$$\frac{Kl_1}{r} = \frac{1.2(6.50 \text{ in.})}{0.180 \text{ in.}} = 43.3$$

Limiting slenderness ratio $4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29,000 \text{ ksi}}{36 \text{ ksi}}} = 134 > 43.3$

$$F_e = \frac{\pi^2 E}{\left(\frac{Kl_1}{r}\right)^2} = \frac{\pi^2(29,000 \text{ ksi})}{(43.3)^2} = 153 \text{ ksi}$$ Eqn. E3-4

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{36 \text{ ksi}}{153 \text{ ksi}}}\right](36 \text{ ksi}) = 32.6 \text{ ksi}$$ Eqn. E3-2

$$l_w = B + 2\left[(\text{connection length})\tan 30°\right] = 6.00 \text{ in.} + 2(6.00 \text{ in.})\tan 30° = 12.9 \text{ in.}$$

Note: Here, the Whitmore section is assumed to be entirely in the gusset. The Whitmore section can spread across the joint into adjacent connected material of equal or greater thickness or adjacent connected material of lesser thickness provided that a rational analysis is performed.

$$A_w = l_w t_p = (12.9 \text{ in.})(0.625 \text{ in.}) = 8.06 \text{ in.}^2$$ Eqn. E3-1

$$P_n = F_{cr} A_w = (32.6 \text{ ksi})(8.06 \text{ in.}^2) = 263 \text{ kips}$$

LRFD	ASD
$\phi P_n = 0.90(263 \text{ kips}) = 237 \text{ kips}$ 237 kips > 158 kips **o.k.**	$P_n / \Omega = \dfrac{263 \text{ kips}}{1.67} = 157 \text{ kips}$ 157 kips > 105 kips **o.k.**

Check tension yielding of gusset plate (tension brace)

From above, A_w = 8.06 in.^{2}

$$R_n = F_y A_w = (36 \text{ ksi})(8.06 \text{ in.}^2) = 290 \text{ kips}$$ Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.90(290 \text{ kips}) = 261 \text{ kips}$ 261 kips > 158 kips **o.k.**	$R_n / \Omega = \dfrac{290 \text{ kips}}{1.67} = 174 \text{ kips}$ 174 kips > 105 kips **o.k.**

Check the available tensile yield strength of the HSS brace

$$R_n = F_y A_g = (46 \text{ ksi})(9.74 \text{ in.}^2) = 448 \text{ kips}$$ Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.9(448 \text{ kips}) = 403 \text{ kips} > 158 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{(448 \text{ kips})}{1.67} = 268 \text{ kips} > 105 \text{ kips}$ **o.k.**

Check the available tensile rupture strength of the HSS brace

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{(6.00 \text{ in.})^2 + 2(6.00 \text{ in.})(6.00 \text{ in.})}{4(6.00 \text{ in.}+6.00 \text{ in.})} = 2.25 \text{ in.}$$ Table D3.1 Case 6

$$U = 1 - \frac{\bar{x}}{L_w} = 1 - \frac{2.25 \text{ in.}}{6.00 \text{ in.}} = 0.625$$

Allowing for a $1/16$ in. gap in fit-up between the HSS and the gusset plate,

$$A_n = A_g - 2\left(t_p + \tfrac{1}{16} \text{ in.}\right)t = 9.74 \text{ in.}^2 - 2\left(0.625 \text{ in.} + \tfrac{1}{16} \text{ in.}\right)(0.465 \text{ in.}) = 9.10 \text{ in.}^2$$

$$A_e = UA_n = 0.625(9.10 \text{ in.}^2) = 5.69 \text{ in.}^2$$ Eqn. D3-1

$$R_n = F_u A_e = (58 \text{ ksi})(5.69 \text{ in.}^2) = 330 \text{ kips}$$ Eqn. J4-2

LRFD	ASD
$\phi R_n = 0.75(330\text{ kips}) = 248\text{ kips}$ 248 kips > 158 kips **o.k.**	$R_n / \Omega = \dfrac{330\text{ kips}}{2.00} = 165\text{ kips}$ 165 kips > 105 kips **o.k.**

Calculate interface forces

Design the gusset-to-beam connection as if each brace were the only brace and locate each brace's connection centroid at the ideal centroid locations to avoid inducing a moment on the gusset-beam interface, similarly to uniform force method special case 3.

$$e_b = \frac{d}{2} = \frac{17.7\text{ in.}}{2} = 8.85\text{ in.}$$

$$\theta = \tan^{-1}\left(\frac{12}{10^{13}\!/\!_{16}}\right) = 48.0°$$

Let $\bar{\alpha} = \alpha = e_b \tan\theta = (8.85\text{ in.})\tan 48.0° = 9.83\text{ in.} \rightarrow$ Use 10.0 in.

$$\beta = e_c = 0$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(10.0\text{ in.} + 0)^2 + (0 + 8.85\text{ in.})^2} = 13.4\text{ in.}$$

LRFD	ASD
$H_{ub} = \dfrac{\alpha P_u}{r} = \dfrac{(10.0\text{ in.})(158\text{ kips})}{13.4\text{ in.}} = 118\text{ kips}$ $V_{ub} = \dfrac{e_b P_u}{r} = \dfrac{(8.85\text{ in.})(158\text{ kips})}{13.4\text{ in.}} = 104\text{ kips}$	$H_{ab} = \dfrac{\alpha P_a}{r} = \dfrac{(10.0\text{ in.})(105\text{ kips})}{13.4\text{ in.}} = 78.4\text{ kips}$ $V_{ab} = \dfrac{e_b P_a}{r} = \dfrac{(8.85\text{ in.})(105\text{ kips})}{13.4\text{ in.}} = 69.3\text{ kips}$

Determine required gusset-to-beam weld size

The weld length is twice the horizontal distance from the work point to the centroid of the gusset-to-beam connection, α, for each brace. Therefore, $l = 2\alpha = 2(10.0\text{ in.}) = 20.0\text{ in.}$

Since the gusset straddles the work line of each brace, the weld is uniformly loaded. Therefore, the available strength is the average required strength and the fillet weld should be designed for 1.25 times the average strength. Manual Part 13

LRFD	ASD
$D_{req'd} = \dfrac{1.25P_u}{1.392l} = \dfrac{1.25(158\text{ kips})}{1.392(20.0\text{ in.})(2)} = 3.55$	$D_{req'd} = \dfrac{1.25P_a}{0.928l} = \dfrac{1.25(105\text{ kips})}{0.928(20.0\text{ in.})(2)} = 3.54$

The minimum fillet weld size is ¼ in. The required weld size is also ¼ in., use a ¼ in. fillet weld 40 in. long on each side of the gusset plate. Table J2.4

Check gusset thickness (against weld size required for strength) — Manual Part 9

$$t_{\min} = \frac{6.19D}{F_u} = \frac{6.19\left(3.55 \text{ sixteenths}\right)}{58 \text{ ksi}} = 0.379 \text{ in.} < \tfrac{5}{8} \text{ in.} \quad \textbf{o.k.}$$

Check local web yielding of the beam

Eqn. J10-2

$$R_n = \left(N + 5k\right)F_y t_w = \left[20.0 \text{ in.} + 5\left(0.827 \text{ in.}\right)\right]\left(50 \text{ ksi}\right)\left(0.300 \text{ in.}\right) = 362 \text{ kips}$$

LRFD	ASD
$R_u = 158 \text{ kips}(\cos\ 48.0°) = 106 \text{ kips}$ $\phi = 1.00$ $\phi R_n = 1.00\left(362 \text{ kips}\right) = 362 \text{ kips}$ 362 kips > 106 kips **o.k.**	$R_a = 105 \text{ kips}(\cos\ 48.0°) = 70.3 \text{ kips}$ $\Omega = 1.50$ $R_n / \Omega = \dfrac{362 \text{ kips}}{1.50} = 241 \text{ kips}$ 241 kips > 70.3 kips **o.k.**

Check web crippling

Eqn. J10-4

$$R_n = 0.80 t_w^{\,2}\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$= 0.80\left(0.300 \text{ in.}\right)^2\left[1 + 3\left(\frac{20.0 \text{ in.}}{17.7 \text{ in.}}\right)\left(\frac{0.300 \text{ in.}}{0.425 \text{ in.}}\right)^{1.5}\right]\sqrt{\frac{\left(29{,}000 \text{ ksi}\right)\left(50 \text{ ksi}\right)\left(0.425 \text{ in.}\right)}{\left(0.300 \text{ in.}\right)}}$$

$$= 311 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(311 \text{ kips}) = 233 \text{ kips}$ 233 kips > 106 kips **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{311 \text{ kips}}{2.00} = 156 \text{ kips}$ 156 kips > 70.3 kips **o.k.**

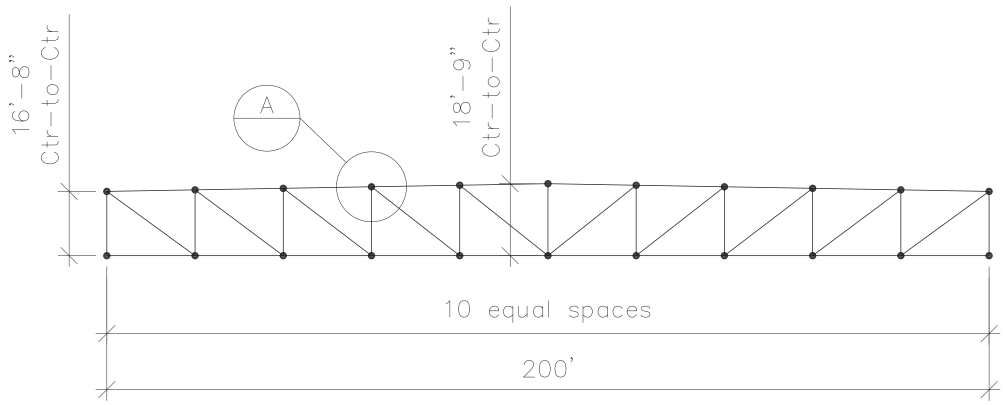
Example II.C-6 Heavy Wide Flange Compression Connection (flanges on the outside)

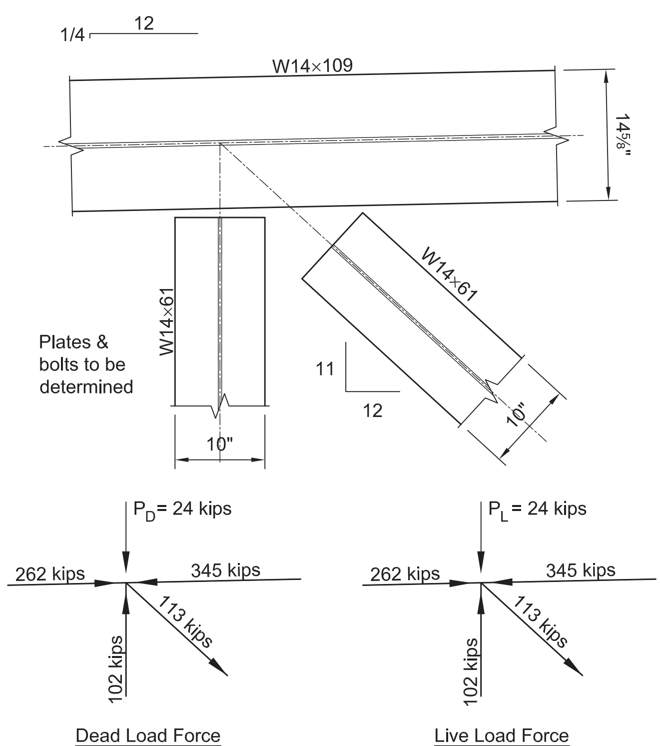
Given:

This truss has been designed with nominal 14 in. W-shapes, with the flanges to the outside of the truss. Beams framing into the top chord and lateral bracing are not shown but can be assumed to be adequate.

Based on multiple load cases, the critical dead and live load forces for this connection were determined. A typical top chord connection and the dead load and live load forces are shown below in detail A. Design this typical connection using 1-in. diameter ASTM A325 slip-critical bolts in oversized holes with a class-A faying surface.

Note: slip critical bolts in oversized holes were selected for this truss to facilitate field erection.





Detail A

Material Properties:

Beam W-shapes	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Gusset Plates	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

W14×109	d = 14. 3 in.	b_f = 14.6 in.	t_f = 0.860 in.	Manual Table 1-1
W14×61	d = 13.9 in.	b_f = 10.0 in.	t_f = 0.645 in.	

Solution:

Determine the required strengths of the members

LRFD	ASD
Left top chord: $P_u = 1.2(262 \text{ kips}) + 1.6(262 \text{ kips}) = 734 \text{ kips}$ Right top chord: $P_u = 1.2(345 \text{ kips}) + 1.6(345 \text{ kips}) = 966 \text{ kips}$ Vertical Web: $P_u = 1.2(102 \text{ kips}) + 1.6(102 \text{ kips}) = 286 \text{ kips}$ Diagonal Web: $T_u = 1.2(113 \text{ kips}) + 1.6(113 \text{ kips}) = 316 \text{ kips}$	Left top chord: $P_a = 262 \text{ kips} + 262 \text{ kips} = 524 \text{ kips}$ Right top chord: $P_a = 345 \text{ kips} + 345 \text{ kips} = 690 \text{ kips}$ Vertical Web: $P_a = 102 \text{ kips} + 102 \text{ kips} = 204 \text{ kips}$ Diagonal Web: $T_a = 113 \text{ kips} + 113 \text{ kips} = 226 \text{ kips}$

Determine the single shear strength

d_{bolt} = 1.00 in. — Section J3.8
ASTM A325-SC bolts
Class A faying surface
μ = 0.35 (class A)
d_{hg} = 1.25 in. (diameter of holes at gusset plates)
h_{sc} = 0.85 (oversized and short-slotted holes)
T_b = 51 kips — Table J3.1
D_u = 1.13

Slip prevention up to the required strength level is selected in this case to prevent truss deflections that could permit a significant ponding condition for which the truss was not designed.

$R_n = \mu D_u h_{sc} T_b N_s = 0.35(1.13)(0.85)(51 \text{ kips})(1) = 17.1 \text{ kips}$ — Eqn. J3-4

Section J3.8

LRFD	ASD
For connections designed to prevent slip at the required strength level; $\phi = 0.85$ $\phi R_n = 0.85(17.1 \text{ kips}) = 14.5 \text{ kips/bolt}$ *Bolt Shear Strength*	For connections designed to prevent slip at the required strength level; $\Omega = 1.76$ $R_n/\Omega = 17.1 \text{ kips} / 1.76 = 9.72 \text{ kips/bolt}$ *Bolt Shear Strength*

LRFD	ASD	
Based on ASTM A325-N; single shear $\phi r_n = 28.3 \text{ kips} > 14.5 \text{ kips}$	Based on ASTM A325-N; single shear $r_n / \Omega = 18.8 \text{ kips} > 9.72 \text{ kips}$	Manual Table 7-1

Diagonal connection

LRFD	ASD
Axial required strength P_u = 316 kips 316 kips / 14.5 kips/bolt = 21.8 bolts 2 rows both sides = 21.8 bolts / 4 = 5.45 Therefore use 6 rows @ min. 3 in. spacing	*Axial required strength* P_a = 226 kips 226 kips / 9.72 kips/bolt = 23.3 bolts 2 rows both sides = 23.3 bolts / 4 = 5.83 Therefore use 6 rows @ min. 3 in. spacing

Check the Whitmore section in the gusset plate (tension only)

Manual Part 9

$$\begin{aligned}\text{Whitmore section} &= \text{gage of the bolts} + \left(\tan 30^0\right)\left(\text{length of bolt group}\right)2 \\ &= \left(5.50 \text{ in.}\right) + \left(\tan 30^0\right)\left[\left(5 \text{ spaces}\right)\left(3.00 \text{ in.}\right)\right]2 \\ &= 22.8 \text{ in.}\end{aligned}$$

Try ⅜ in. thick plate

A_g = (0.375 in.)(22.8 in.) = 8.55 in.^{2}

LRFD	ASD	
Check tension yielding $\phi = 0.90$ $\phi R_n = \phi F_y A_g$ $= 0.90(36 \text{ ksi})(8.55 \text{ in.}^2)(2)$ $= 554 \text{ kips} > 316 \text{ kips}$ **o.k.**	*Check shear yielding* $\Omega = 1.67$ $R_n / \Omega = F_y A_g / \Omega$ $= (36 \text{ ksi})(8.55 \text{ in.}^2)(2) / 1.67$ $= 369 \text{ kips} > 226 \text{ kips}$ **o.k.**	Eqn. J4-1
Check block shear rupture in plate $\phi = 0.75$ Tension stress is uniform, Therefore; U_{bs} = 1.0 t_p = 0.375 in. A_{gv} / in. = (2 rows)[(6 bolts-1)(3 in.) + 2 in.] = 34.0 in.^{2}/in. A_{nv} / in. = 34.0 in. – (2 rows)(5.50 bolts) × (1.25 in.+ 0.0625 in.) = 19.6 in.^{2}/in. A_{nt} / in. = 5.50 in. – (1.25 in. + 0.0625 in.) = 4.19 in.^{2}/in.	*Check block shear rupture in plate* $\Omega = 2.00$ Tension stress is uniform, Therefore; U_{bs} = 1.0 Try Plate with, t_p = 0.375 in. A_{gv} / in. = (2 rows)[(6 bolts-1)(3 in.) + 2 in.] = 34.0 in.^{2}/in. A_{nv} / in. = 34.0 in. – (2 rows)(5.50 bolts) × (1.25 in.+ 0.0625 in.) = 19.6 in.^{2}/in. A_{nt} / in. = 5.50 in. – (1.25 in. + 0.0625 in.) = 4.19 in.^{2}/in.	Section J4.3

LRFD	ASD	
$\phi R_n = \phi(0.6F_uA_{nv} + U_{bs}F_uA_{nt}) \leq \phi(0.6F_yA_{gv} + U_{bs}F_uA_{nt})$	$R_n / \Omega = (0.6F_uA_{nv} + U_{bs}F_uA_{nt}) / \Omega \leq (0.6F_yA_{gv} + U_{bs}F_uA_{nt}) / \Omega$	Eqn. J4-5
Tension Rupture Component	Tension Rupture Component	
$\phi U_{bs}(F_u A_{nt}) / t$ $= 0.75(1.0)(58 \text{ ksi})(4.19 \text{ in.}^2/\text{in.})$ $= 182$ kips/in.	$U_{bs}(F_u A_{nt}) / (t\Omega)$ $= 1.0(58 \text{ ksi})(4.19 \text{ in.}^2/\text{in.}) / 2.00$ $= 122$ kips/in.	
Shear Yielding Component	Shear Yielding Component	
$\phi(0.6F_yA_{gv}) / t$ $= 0.75(0.6)(36 \text{ ksi})(34.0 \text{ in.}^2/\text{in.})$ $= 551$ kips/in.	$(0.6F_yA_{gv}) / (t\Omega)$ $= 0.6(36 \text{ ksi})(34.0 \text{ in.}^2/\text{in.}) / 2.00$ $= 367$ kips/in.	
Shear Rupture Component	Shear Rupture Component	
$\phi(0.6F_uA_{nv}) / t$ $= 0.75(0.6)(58 \text{ ksi})(19.6 \text{ in.}^2/\text{in.})$ $= 512$ kips/in.	$(0.6F_uA_{nv}) / (t\Omega)$ $= 0.6(58 \text{ ksi})(19.6 \text{ in.}^2/\text{in.}) / 2.00$ $= 341$ kips/in.	
$\phi R_n = (512 \text{ kips/in.} + 182 \text{ kips/in.})(0.375 \text{ in.})$ $= 260$ kips $\leq (551 \text{ kips/in.} + 182 \text{ kips/in.})(0.375 \text{ in.})$ $= 275$ kips $= 260 \text{ kips} < 275 \text{ kips}$	$R_n/\Omega = (341 \text{ kips/in.}+122 \text{ kips/in.})(0.375 \text{ in.})$ $= 174$ kips $\leq (367 \text{ kips/in.}+122 \text{ kips/in.})(0.375 \text{ in.})$ $= 183$ kips $= 174 \text{ kips} < 183 \text{ kips}$	
$260 \text{ kips} > 316 \text{ kips} / 2 = 158 \text{ kips}$ **o.k.**	$174 \text{ kips} > 226 \text{ kips}/2 = 113 \text{ kips}$ **o.k.**	
Check block shear rupture on beam flange By inspection, block shear rupture on the beam flange will not control.	*Check block shear rupture on beam flange* By inspection, block shear rupture on the beam flange will not control.	
Bolt bearing on plate	*Bolt bearing on plate*	
Based on Bolt Spacing = 3 in.; oversized holes, F_u = 58 ksi	Based on Bolt Spacing = 3 in.; oversized holes, F_u = 58 ksi	
$\phi r_n = (91.4 \text{ kips/in.})(0.375 \text{ in.})$ $= 34.3 \text{ kips} > 14.5 \text{ kips}$	$r_n / \Omega = (60.9 \text{ kips/in.})(0.375 \text{ in.})$ $=22.8 \text{ kips} > 9.72 \text{ kips}$	Manual Table 7-5
Based on Edge Distance = 2 in.; oversized holes, F_u = 58 ksi	Based on Edge Distance = 2 in.; oversized holes, F_u = 58 ksi	
$\phi r_n = (71.8 \text{ kips/in.})(0.375 \text{ in.})$ $= 26.9 \text{ kips} > 14.5 \text{ kips}$	$r_n / \Omega = (47.9 \text{ kips/in.})(0.375 \text{ in.})$ $= 18.0 \text{ kips} > 9.72 \text{ kips}$	Manual Table 7-6
Bolt bearing on flange	*Bolt bearing on flange*	
Based on Bolt Spacing = 3 in.; standard holes, F_u = 65 ksi	Based on Bolt Spacing = 3 in.; standard holes, F_u = 65 ksi	Manual

LRFD	ASD	
$\phi r_n = (113 \text{ kips/in.})(0.645 \text{ in.})$ $= 72.9 \text{ kips} > 14.5 \text{ kips}$	$r_n / \Omega = (75.6 \text{ kips/in.})(0.645)$ $= 48.8 \text{ kips} > 9.72 \text{ kips}$	Table 7-5
Based on Edge Distance = 2 in.; standard holes, F_u = 65 ksi	Based on Edge Distance = 2 in.; standard holes, F_u = 65 ksi	Manual Table 7-6
$\phi r_n = (85.9 \text{ kips/in.})(0.645 \text{ in.})$ $= 55.4 \text{ kips} > 14.5 \text{ kips}$	$r_n / \Omega = (57.3 \text{ kips/in.})(0.645 \text{ in.})$ $= 37.0 \text{ kips} > 9.72 \text{ kips}$	

Horizontal connection

LRFD	ASD	
Required axial strength	Required axial strength	
P_u = 966 kips – 734 kips = 232 kips	P_a = 690 kips – 524 kips = 166 kips	Section J3.8
232 kips / 14.5 kips/bolt = 16.0 bolts	166 kips / 9.72 kips/bolt = 17.1 bolts	
2 rows both sides = 16.0 bolts / 4 = 4.00	2 rows both sides = 17.1 bolts / 4 = 4.28	
Use 5 bolts per row	Use 5 bolts per row	
Check shear in plate	*Check shear in plate*	
Try Plate with, t_p = 0.375 in.	Try Plate with, t_p = 0.375 in.	
A_{gv} / t_p = 34.0 in.^{2}/in. (from sketch)	A_{gv} / t_p= 34.0 in.^{2}/in. (from sketch)	
A_{nv} / t_p = 34.0 in. – (1 row)(5 bolts) × (1.25 in.+ 0.0625 in.) = 27.4 in.^{2}/in.	A_{nv} / t_p = 34.0 in. – (1 row (5 bolts – 0.5) × (1.25 in.+ 0.0625 in.) = 27.4 in.^{2}/in.	
Check shear yielding	*Check shear yielding*	
$A_g = (34.0 \text{ in.})(0.375 \text{ in.}) = 12.8 \text{ in.}^2$	$A_g = (34.0 \text{ in.})(0.375 \text{ in.}) = 12.8 \text{ in.}^2$	
$\phi = 1.00$ $\phi R_n = \phi(0.6F_yA_g)$ $= 1.00(0.6)(36 \text{ ksi})(12.8 \text{ in.}^2)$ $= 276 \text{ kips}$	$\Omega = 1.50$ $R_n / \Omega = (0.6F_yA_g) / \Omega$ $= 0.6(36 \text{ ksi})(12.8 \text{ in.}^2)/1.50$ $= 184 \text{ kips}$	Eqn. J4-3
276 kips > 232 kips / 2 = 116 kips **o.k.**	184 kips > 166 kips/2 = 83.0 kips **o.k.**	
Check shear rupture	*Check shear rupture*	
$A_{nv} = (27.4 \text{ in.})(0.375 \text{ in.}) = 10.3 \text{ in.}^2$	$A_{nv} = (27.4 \text{ in.})(0.375 \text{ in.}) = 10.3 \text{ in.}^2$	
$\phi = 0.75$ $\phi R_n = \phi(0.6F_uA_{nv})$ $= 0.75(0.6)(58 \text{ ksi})(10.3 \text{ in.}^2)$ $= 268 \text{ kips}$	$\Omega = 2.00$ $R_n/\Omega = (0.6F_uA_{nv})/\Omega$ $= 0.6(58 \text{ ksi})(10.3 \text{ in.}^2)/2.00$ $= 179 \text{ kips}$	Eqn. J4-4
	179 kips > 166 kips / 2 = 83.0 kips **o.k.**	

LRFD	ASD	
268 kips > 232 kips / 2 = 116 kips **o.k.**		
Bolt bearing on plate	*Bolt bearing on plate*	
Based on Bolt Spacing = 5½ in.; oversized holes, F_u = 58 ksi	Based on Bolt Spacing = 5½ in.; oversized holes, F_u = 58 ksi	
$\phi r_n = (104\text{ kips/in.})(0.375\text{ in.})$ $= 39.0\text{ kips} > 14.5\text{ kips}$	$r_n / \Omega = (69.6\text{ kips/in.})(0.375\text{ in.})$ $= 26.1\text{ kips} > 9.72\text{ kips}$	Manual Table 7-5
Based on Edge Distance = 2 in.; oversized holes, F_u = 58 ksi	Based on Edge Distance = 2 in.; oversized holes, F_u = 58 ksi	
$\phi r_n = (71.8\text{ kips/in.})(0.375\text{ in.})$ $= 26.9\text{ kips} > 14.5\text{ kips}$	$r_n / \Omega = (47.9\text{ kips/in.})(0.375\text{ in.})$ $= 18.0\text{ kips} > 9.72\text{ kips}$	Manual Table 7-6
Bolt bearing on flange	*Bolt bearing on flange*	
Based on Bolt Spacing = 5½ in.; standard holes, F_u = 65 ksi	Based on Bolt Spacing = 5½ in.; standard holes, F_u = 65 ksi	
$\phi r_n = (117\text{ kips/in.})(0.860\text{ in.})$ $= 101\text{ kips} > 14.5\text{ kips}$	$r_n / \Omega = (78.0\text{ kips/in.})(0.860)$ $= 67.1\text{ kips} > 9.72\text{ kips}$	Manual Table 7-5
Based on edge distance = 2 in.; standard holes, F_u = 65 ksi	Based on edge distance = 2 in.; standard holes, F_u = 65 ksi	
$\phi r_n = (85.9\text{ kips/in.})(0.860\text{ in.})$ $= 73.9\text{ kips} > 14.5\text{ kips}$	$r_n / \Omega = (57.3\text{ kips/in.})(0.860\text{ in.})$ $= 49.3\text{ kips} > 9.72\text{ kips}$	Manual Table 7-6

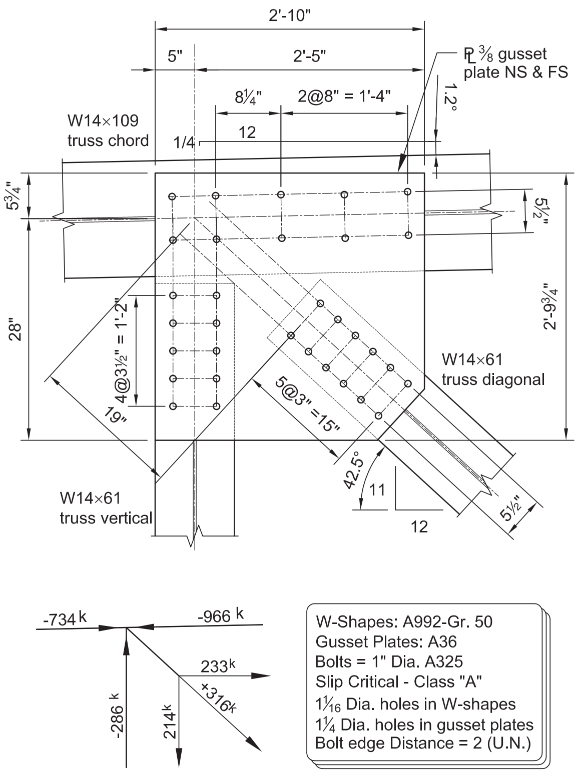
Vertical connection

LRFD	ASD
Required axial strength	Required axial strength
P_u = 286 kips	P_a = 204 kips
286 kips / 14.5 kips/bolt = 19.7 bolts	204 kips / 9.72 kips/bolt = 21.0 bolts
2 rows both sides = 19.7 bolts / 4 = 4.93	2 rows both sides = 21.0 bolts / 4 = 5.24
Therefore, use 5 rows	Therefore, use 6 rows
Check shear in plate	*Check shear in plate*
Try Plate with, t_p = 0.375 in.	Try Plate with, t_p = 0.375 in.
A_{gv} / t_p = 33.8 in.^{2}/in. (from sketch)	A_{gv} / t_p = 33.8 in.^{2}/in. (from sketch)
A_{nv} / t_p = 33.8 in. – (1 row)(7 bolts) × (1.25 in. + 0.0625 in.) = 24.6 in	A_{nv} / t_p = 33.8 in. – (1 row)(8 bolts) × (1.25 in. + 0.0625 in.) = 23.3 in.
Check shear yielding	

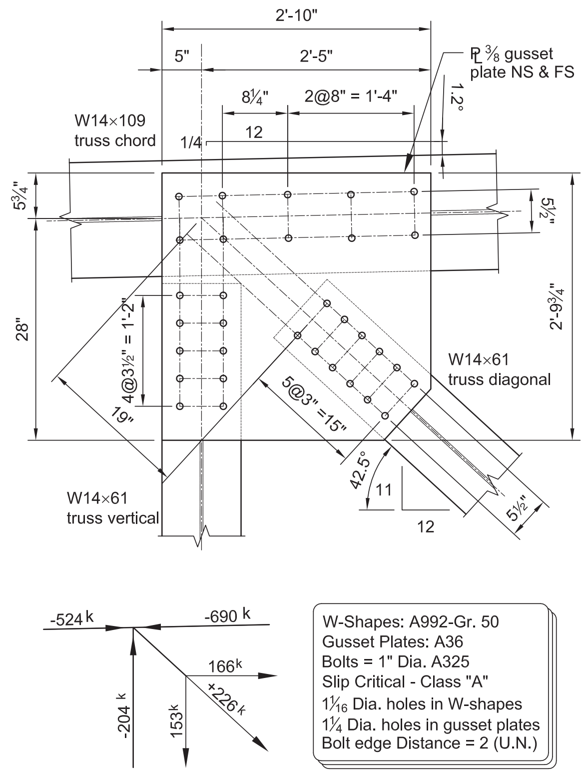
LRFD	ASD	
$A_g = (33.8\text{ in.})(0.375\text{ in.}) = 12.7\text{ in.}^2$ $\phi = 1.00$ $\phi R_n = \phi(0.6F_yA_g)$ $= 1.00(0.6)(36\text{ ksi})(12.7\text{ in.}^2)$ $= 274\text{ kips}$ 274 kips > 286 kips / 2 = 143 kips **o.k.**	*Check shear yielding* $A_g = (33.8\text{ in.})(0.375\text{ in.}) = 12.7\text{ in.}^2$ $\Omega = 1.50$ $R_n / \Omega = (0.6F_yA_g) / \Omega$ $= 0.6(36\text{ ksi})(12.7\text{ in.}^2)/1.50$ $= 183\text{ kips}$ 183 > 204 kips / 2 = 102 kips **o.k.**	Eqn. J4-3
Check shear rupture $A_{nv} = (24.6\text{ in.})(0.375\text{ in.}) = 9.23\text{ in.}^2$ $\phi = 0.75$ $\phi R_n = \phi(0.6F_uA_{nv})$ $= 0.75(0.6)(58\text{ ksi})(9.23\text{ in.}^2)$ $= 241\text{ kips}$ 241 kips > 286 kips / 2 = 143 kips **o.k.**	*Check shear rupture* $A_{nv} = (23.3\text{ in.})(0.375\text{ in.}) = 8.74\text{ in.}^2$ $\Omega = 2.00$ $R_n / \Omega = (0.6F_uA_{nv}) / \Omega$ $= 0.6(58\text{ ksi})(8.74\text{ in.}^2)/2.00$ $= 152\text{ kips}$ 152 kips > 204 kips / 2 = 102 kips **o.k.**	Eqn. J4-4
Bolt bearing on plate Conservatively based on Bolt Spacing = 3 in.; oversized holes, $\phi r_n = (91.4\text{ kips/in.})(0.375\text{ in.})$ $= 34.3\text{ kips} > 14.5\text{ kips}$	*Bolt bearing on plate* Conservatively based on Bolt Spacing = 3 in.; oversized holes, $r_n / \Omega = (60.9\text{ kips/in.})(0.375\text{ in.})$ $= 22.8\text{ kips} > 9.72\text{ kips}$	Manual Table 7-5
Based on Edge Distance = 2 in.; oversized holes, $\phi r_n = (71.8\text{ kips/in.})(0.375\text{ in.})$ $= 26.9\text{ kips} > 14.5\text{ kips}$	Based on Edge Distance = 2 in.; oversized holes, $r_n / \Omega = (47.9\text{ kips/in.})(0.375\text{ in.})$ $= 18.0\text{ kips} > 9.72\text{ kips}$	Manual Table 7-6
Bolt bearing on flange Conservatively based on Bolt Spacing = 3 in.; standard holes, $\phi r_n = (113\text{ kips/in.})(0.645\text{ in.})$ $= 72.9\text{ kips} > 14.5\text{ kips}$	*Bolt bearing on flange* Conservatively based on Bolt Spacing = 3 in.; standard holes, $r_n / \Omega = (75.6\text{ kips/in.})(0.645)$ $= 48.8\text{ kips} > 9.72\text{ kips}$	Manual Table 7-5
Based on edge distance = 2 in.; standard holes, $\phi r_n = (85.9\text{ kips/in.})(0.645\text{ in.})$ $= 55.4\text{ kips} > 14.5\text{ kips}$	Based on edge distance = 2 in.; standard holes, $r_n / \Omega = (57.3\text{ kips/in.})(0.645\text{ in.})$ $= 37.0\text{ kips} > 9.72\text{ kips}$	Manual Table 7-6

The final layout for the connection is as follows:

Example Connection Using Bolted Gusset Plates @
Top Chord Panel Point U3 of Example Truss Design
Case 1: LRFD Design Format



Example Connection Using Bolted Gusset Plates @
Top Chord Panel Point U3 of Example Truss Design
Case 2: ASD Design Format



Note that because the difference in depths between the top chord and the vertical and diagonal members, 3⁄16 in. loose shims are required on each side of the shallower members.

Chapter IID
Miscellaneous Connections

This section contains design examples on connections in the AISC *Steel Construction Manual*. that are not covered in other sections of AISC *Design Examples*.

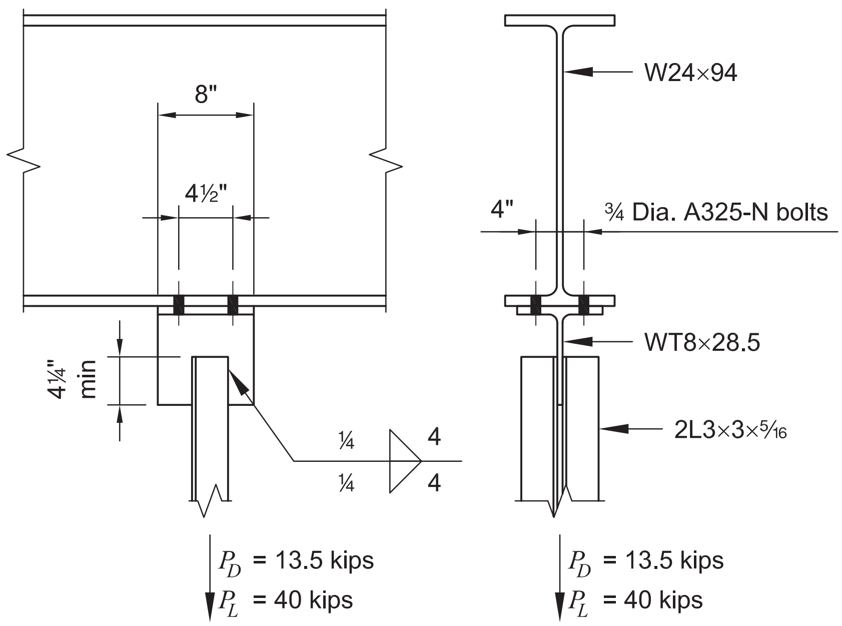
Example II.D-1 Prying Action in Tees and in Single Angles

Given:

Design a WT tension-hanger connection between a 2L3×3×5⁄16 tension member and a W24×94 beam to support the following loads:

P_D = 13.5 kips
P_L = 40 kips

Use ¾-in. diameter ASTM A325-N or F1852-N bolts and 70 ksi electrodes.



Material Properties:

Manual Table 2-3

Hanger WT	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
Beam W24×94	ASTM A992	F_y = 50 ksi	F_u = 65 ksi
Angles 2L3×3×5⁄16	ASTM A36	F_y = 36 ksi	F_u = 58 ksi

Geometric Properties:

Manual Tables 1-1, 1-7 & 1-15

Beam W24×94 d = 24.3 in. t_w = 0.515 in. b_f = 9.07 in. t_f = 0.875 in.
Angles 2L3×3×5⁄16 A = 3.55 in.^{2} $\bar{x}$ = 0.860 in.

Solution:

Calculate the required strength

LRFD	ASD
$P_u = 1.2(13.5 \text{ kips}) + 1.6(40 \text{ kips}) = 80.2 \text{ kips}$	$P_a = 13.5 \text{ kips} + 40 \text{ kips} = 53.5 \text{ kips}$

Check tension yielding of angles

$R_n = F_y A_g = (36 \text{ ksi})(3.55 \text{ in.}^2) = 128 \text{ kips}$ Eqn. D2-1

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(128 \text{ kips})$ $= 115 \text{ kips} > 80.2 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $R_n / \Omega = \dfrac{128 \text{ kips}}{1.67}$ $= 76.6 \text{ kips} > 53.5 \text{ kips}$ **o.k.**
The minimum weld size is: w_{min} = 3⁄16 in. The maximum weld size is: w_{max} = thickness - 1⁄16 in. = ¼ in. Try ¼–in. fillet welds $L_{min} = \dfrac{P_u}{1.392D} = \dfrac{80.2 \text{ kips}}{1.392(4 \text{ sixteenths})}$ $= 14.4 \text{ in.}$ Use four 4-in. welds (16 in. total), one at each toe and heel of each angle.	The minimum weld size is: w_{min} = 3⁄16 in. The maximum weld size is: w_{max} = thickness - 1⁄16 in. = ¼ in. Try ¼–in. fillet welds $L_{min} = \dfrac{P_a}{0.928D} = \dfrac{53.5 \text{ kips}}{0.928(4 \text{ sixteenths})}$ $= 14.4 \text{ in.}$ Use four 4-in. welds (16 in. total), one at each toe and heel of each angle.

Table J2.4

Manual Part 8

Check tension rupture of angles

Calculate the effective net area

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.860 \text{ in.}}{4.00 \text{ in.}} = 0.785$$

Table D3.1 Case 2

$A_e = A_n U = \left(3.55 \text{ in.}^2\right)\left(0.785\right) = 2.79 \text{ in.}^2$ Eqn. D3-1

LRFD	ASD
$R_n = F_u A_e = \left(58 \text{ ksi}\right)\left(2.79 \text{ in.}^2\right) = 162 \text{ kips}$ $\phi = 0.75$ $\phi_t R_n = 0.75\left(162 \text{ kips}\right)$ $= 122 \text{ kips} > 80.2 \text{ kips}$ **o.k.**	$R_n = F_u A_e = (58.0 \text{ ksi})(2.79 \text{ in.}^2) = 162 \text{ kips}$ $\Omega = 2.00$ $R_n / \Omega_t = \dfrac{162 \text{ kips}}{2.00}$ $= 122 \text{ kips} > 80.2 \text{ kips}$ **o.k.**
Select a preliminary WT *using beam gage* g = 4 in. With four ¾–in. diameter ASTM A325-N bolts,	*Select a preliminary* WT *using beam gage* g = 4 in. With four ¾–in. diameter ASTM A325-N bolts,

Eqn. D2-2

LRFD	ASD
$T = r_{ut} = \dfrac{P_u}{n} = \dfrac{80.2\text{ kips}}{4} = 20.1\text{ kips/bolt}$ $B = \phi r_n = 29.8\text{ kips} > 20.1\text{ kips}$ **o.k.**	$T = r_{at} = \dfrac{P_a}{n} = \dfrac{53.5\text{ kips}}{4} = 13.4\text{ kips/bolt}$ $B = r_n / \Omega = 19.9\text{ kips} > 13.4\text{ kips}$ **o.k.**

Manual Table 7-2

With four bolts, the maximum effective length is 2*g* = 8.00 in. Thus, there is 4.00 in. of tee length tributary to each pair of bolts and

LRFD	ASD
$\dfrac{2\text{ bolts}(20.1\text{ kips/bolt})}{4.00\text{ in.}} = 10.1\text{ kips/in.}$	$\dfrac{2\text{ bolts}(13.4\text{ kips/bolt})}{4.00\text{ in.}} = 6.70\text{ kips/in.}$

From Manual Table 15-16, with an assumed *b* = 4.00 in. / 2 = 2.00 in., the flange thickness of the WT hanger should be $t \approx 1\tfrac{1}{16}$ in.

The minimum depth WT that can be used is equal to the sum of the weld length plus the weld size plus the *k*-dimension for the selected section. From Manual Table 1-8 with an assumed *b* = 4 in./2 = 2 in., $t_0 \approx 1\tfrac{1}{16}$ in., and $d_{min} = 4\text{ in.} + \tfrac{1}{4}\text{ in.} + k \approx 6\text{ in.}$, appropriate selections include:

WT6×39.5	WT8×28.5
WT7×34	WT9×30

Manual Table 1-8

Try WT8×28.5; b_f = 7.12 in., t_f = 0.715 in., t_w = 0.430 in.

Manual Part 9

Check prying action

The beam flange is thicker than the WT flange; therefore, prying in the tee flange will control over prying in the beam flange.

$$b = \frac{g - t_w}{2} = \frac{(4.00\text{ in.} - 0.430\text{ in.})}{2} = 1.79\text{ in.} > 1\tfrac{1}{4}\text{-in. entering and tightening clearance, and}$$

the fillet toe is cleared. **o.k.**

$$a = \frac{b_f - g}{2} = \frac{(7.12\text{ in.} - 4.00\text{ in.})}{2} = 1.56\text{ in.}$$

$$b' = b - \frac{d_b}{2} = 1.79\text{ in.} - \left(\frac{0.750\text{ in.}}{2}\right) = 1.42\text{ in.}$$

$$\begin{aligned} a' &= \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \\ &= 1.56\text{ in.} + \left(\frac{0.750\text{ in.}}{2}\right) \le 1.25(1.79\text{ in.}) + \frac{0.750\text{ in.}}{2} \\ &= 1.94\text{ in.} \le 2.61\text{ in.} \end{aligned}$$

$$\rho = \frac{b'}{a'} = \frac{1.42\text{ in.}}{1.94\text{ in.}} = 0.732$$

LRFD	ASD
$\beta = \frac{1}{\rho}\left(\frac{B}{T}-1\right) = \frac{1}{0.732}\left(\frac{29.8 \text{ kips/bolt}}{20.1 \text{ kips / bolt}}-1\right)$ $= 0.659$	$\beta = \frac{1}{\rho}\left(\frac{B}{T}-1\right) = \frac{1}{0.732}\left(\frac{19.9 \text{ kips/bolt}}{13.4 \text{ kips / bolt}}-1\right)$ $= 0.663$

$$\delta = 1 - \frac{d'}{p} = 1 - \left(\frac{0.813 \text{ in.}}{4.00 \text{ in.}}\right) = 0.797$$

Since $\beta < 1.0$,

LRFD	ASD
$\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \le 1.0$ $= \frac{1}{0.797}\left(\frac{0.659}{1-0.659}\right)$ $= 2.42 \;\therefore\; \alpha' = 1.0$ $t_{\min} = \sqrt{\frac{4.44Tb'}{pF_u(1+\delta\alpha')}}$ $= \sqrt{\frac{4.44(20.1 \text{ kips/bolt})(1.42 \text{ in.})}{(4.00 \text{ in.})(65 \text{ ksi})\left[1+(0.797)(1.0)\right]}}$ $= 0.521 \text{ in.} < t_f = 0.715 \text{ in.}$ **o.k.**	$\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \le 1.0$ $= \frac{1}{0.797}\left(\frac{0.663}{1-0.663}\right)$ $= 2.47 \;\therefore\; \alpha' = 1.0$ $t_{\min} = \sqrt{\frac{6.66Tb'}{pF_u(1+\delta\alpha')}}$ $= \sqrt{\frac{6.66(13.4 \text{ kips/bolt})(1.42 \text{ in.})}{(4.00 \text{ in.})(65 \text{ ksi})\left[1+(0.797)(1.0)\right]}}$ $= 0.521 \text{ in.} < t_f = 0.715 \text{ in.}$ **o.k.**

Check tension yielding of the tee stem on the Whitmore section Manual Part 9

The effective width of the tee stem (which cannot exceed the actual width of 8 in.) is

$$L_w = 3.00 \text{ in.} + 2(4.00 \text{ in.})(\tan 30^\circ) \le 8.00 \text{ in.}$$
$$= 7.62 \text{ in.}$$

and the nominal strength is determined as

$$R_n = F_y A_{geff} = (50 \text{ ksi})(7.62 \text{ in.})(0.430 \text{ in.}) = 164 \text{ kips}$$ Eqn. J4-1

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(164 \text{ kips}) = 148 \text{ kips} > 80.2 \text{ kips}$ **o.k.**	$\Omega = 1.67$ $R_n / \Omega = \frac{164 \text{ kips}}{1.67} = 98.2 \text{ kips} > 53.5 \text{ kips}$ **o.k.**

Check shear rupture of the base metal along the toe and heel of each weld line

Manual Part 9

$$t_{min} = \frac{6.19D}{F_u}$$
$$= 6.19\left(\frac{4 \text{ sixteenths}}{65 \text{ ksi}}\right)$$
$$= 0.381 \text{ in.} < 0.430 \text{ in.} \quad \textbf{o.k.}$$

Check block shear rupture of the tee stem

Since the angles are welded to the WT-hanger the gross area shear yielding will control.

$$A_{gv} = (2 \text{ welds})(4.00 \text{ in.})(0.430 \text{ in.}) = 3.44 \text{ in.}^2$$

Tension stress is uniform, therefore $U_{bs} = 1.0$.

$$A_{nt} = A_g = 1.0(3.00 \text{ in.})(0.430 \text{ in.}) = 1.29 \text{ in.}^2$$

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

Eqn. J4-5

$$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(50 \text{ ksi})(3.44 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.29 \text{ in.}^2) = 187 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(187) = 140 \text{ kips} > 80.2 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $R_n / \Omega = \dfrac{187}{2.00} = 93.5 \text{ kips} > 53.5 \text{ kips}$ **o.k.**

Note: Alternately, a WT tension hanger could be selected with a flange thickness to reduce the effect of prying action to an insignificant amount, i.e., $q \approx 0$. Assuming $b' = 1.42$ in.

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4.44Tb'}{pF_u}}$ $= \sqrt{\dfrac{4.44(20.1 \text{ kips/bolt})(1.42 \text{ in.})}{(4.00 \text{ in./bolt})(65 \text{ ksi})}}$ $= 0.698$ in.	$t_{min} = \sqrt{\dfrac{6.66Tb'}{pF_u}}$ $= \sqrt{\dfrac{6.66(13.4 \text{ kips/bolt})(1.42 \text{ in.})}{(4.00 \text{ in./bolt})(65 \text{ ksi})}}$ $= 0.698$ in.

Manual Part 9

A WT8×28.5, with $t_f = 0.715$ in. > 0.698 in., provides adequate strength for the tension-hanger connection, and has sufficient flange thickness to reduce the effect of prying action to an insignificant amount.

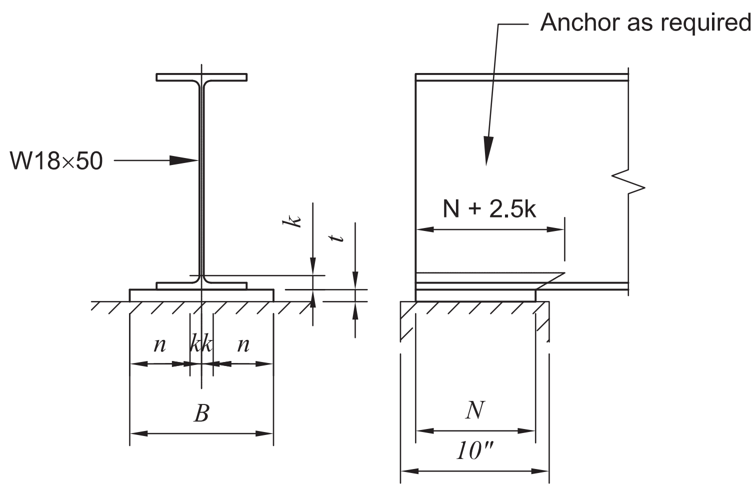
Example II.D-2 Beam Bearing Plate

Given:

A W18×50 beam with a dead load end reaction of 15 kips and a live load end reaction of 45 kips is supported by a 10-in. thick concrete wall.

If the beam has F_y = 50 ksi, the concrete has f_c' = 3 ksi, and the bearing plate has F_y = 36 ksi, determine:

a. if a bearing plate is required if the beam is supported by the full wall thickness,
b. the bearing plate required if N = 10 in. (the full wall thickness),
c. the bearing plate required if N = 6½ in. and the bearing plate is centered on the thickness of the wall.



Material Properties:

Manual Tables 2-3 and 2-4

Beam W18×50	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Bearing Plate (if required)	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	
Concrete Wall	f_c' = 3 ksi			

Geometric Properties:

Manual Table 1-1

Beam W18×50	d = 18.0 in.	t_w = 0.355 in.	b_f = 7.50 in.	t_f = 0.570 in.
	k_{des} = 0.972 in.	k_1 = 13⁄16 in.		
Concrete Wall	h = 10.0 in.			

Solution A:

LRFD	ASD	
Calculate required strength R_u = 1.2(15 kips) + 1.6(45 kips) = 90.0 kips *Check local web yielding* $N_{req} = \dfrac{R_u - \phi R_1}{\phi R_2} \geq k$ $= \dfrac{90.0 \text{ kips - } 43.1 \text{ kips}}{17.8 \text{ kips/in.}} \geq 0.972$ in. = 2.63 in. < 10.0 in. **o.k.**	*Calculate required strength* R_a = 15 kips + 45 kips = 60.0 kips *Check local web yielding* $N_{req} = \dfrac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k$ $= \dfrac{60.0 \text{ kips - } 28.8 \text{ kips}}{11.8 \text{ kips/in.}} \geq 0.972$ in. = 2.64 in. < 10.0 in. **o.k.**	Manual Table 9-4
Check web crippling $\dfrac{N}{d} = \dfrac{10.0 \text{ in.}}{18.0 \text{ in.}}$ = 0.556 Since $\dfrac{N}{d} > 0.2$, $N_{req} = \dfrac{R_u \text{ - } \phi R_5}{\phi R_6}$ $= \dfrac{90.0 \text{ kips - } 52.0 \text{ kips}}{6.30 \text{ kips/in.}}$ = 6.03 in. < 10.0 in. **o.k.** Verify $\dfrac{N}{d} > 0.2$, $\dfrac{N}{d} = 0.335 > 0.2$ **o.k.**	*Check web crippling* $\dfrac{N}{d} = \dfrac{10.0 \text{ in.}}{18.0 \text{ in.}}$ = 0.556 Since $\dfrac{N}{d} > 0.2$, $N_{req} = \dfrac{R_a \text{ - } R_5 / \Omega}{R_6 / \Omega}$ $= \dfrac{60.0 \text{ kips - } 34.7 \text{ kips}}{4.20 \text{ kips/in.}}$ = 6.02 in. < 10.0 in. **o.k.** Verify $\dfrac{N}{d} > 0.2$, $\dfrac{N}{d} = 0.335 > 0.2$ **o.k.**	Manual Table 9-4
Check the bearing strength of concrete $\phi_c = 0.60$ $\phi_c P_p = \phi_c(0.85f'_c)A_1$ = 0.60 (0.85)(3 ksi)(7.50 in.)(10.0 in.) = 115 kips > 90.0 kips **o.k.**	*Check the bearing strength of concrete* $\Omega_c = 2.50$ $P_p/\Omega_c = (0.85f'_c)\, A_1/\Omega_c$ = (0.85)(3 ksi)(7.50 in.)(10.0 in.)/2.50 = 76.5 kips > 60.0 kips **o.k.**	Eqn. J8-1

Check beam flange thickness

LRFD	ASD	
Determine cantilever length	*Determine cantilever length*	Manual Part 14

LRFD	ASD
$n = \dfrac{b_f}{2} - k_{des}$ $= \dfrac{7.50\text{ in.}}{2} - 0.972\text{ in.} = 2.78\text{ in.}$ *Determine bearing pressure* $f_p = \dfrac{R_u}{A_1}$ *Determine cantilever moment* $M_u = \dfrac{R_u n^2}{2A_1}$ $Z = \frac{1}{4}t^2$ $M_u \le \phi F_y Z \le \phi F_y\left(\dfrac{t^2}{4}\right)$ $t_{req} = \sqrt{\dfrac{4M_u}{\phi F_y}} = \sqrt{\dfrac{2R_u n^2}{\phi A_1 F_y}}$	$n = \dfrac{b_f}{2} - k_{des}$ $= \dfrac{7.50\text{ in.}}{2} - 0.972\text{ in.} = 2.78\text{ in.}$ *Determine bearing pressure* $f_p = \dfrac{R_a}{A_1}$ *Determine cantilever moment* $M_a = \dfrac{R_a n^2}{2A_1}$ $Z = \frac{1}{4}t^2$ $M_a \le \dfrac{F_y Z}{\Omega} \le \dfrac{F_y}{\Omega}\left(\dfrac{t^2}{4}\right)$ $t_{req} = \sqrt{\dfrac{\Omega 4M_a}{F_y}} = \sqrt{\dfrac{\Omega 2R_a n^2}{A_1 F_y}}$
$\phi = 0.90$ $t_{min} = \sqrt{\dfrac{2.22R_u n^2}{A_1 F_y}}$ $= \sqrt{\dfrac{2.22(90.0\text{ kips})(2.78\text{ in.})^2}{(7.50\text{ in.})(10.0\text{ in.})(50\text{ ksi})}}$ $= 0.642\text{ in.} > 0.570\text{ in.}$ **n.g.** **A bearing plate is required.**	$\Omega = 1.67$ $t_{min} = \sqrt{\dfrac{3.34R_a n^2}{A_1 F_y}}$ $= \sqrt{\dfrac{3.34(60.0\text{ kips})(2.78\text{ in.})^2}{(7.50\text{ in.})(10.0\text{ in.})(50\text{ ksi})}}$ $= 0.643\text{ in.} > 0.570\text{ in.}$ **n.g.** **A bearing plate is required.**

Solution B:

N = 10 in.

From Solution A, local web yielding and web crippling are not critical.

LRFD	ASD	
Calculate the required bearing-plate width.	*Calculate the required bearing-plate width.*	
$\phi_c = 0.60$	$\Omega_c = 2.50$	
$A_{1\,req} = \dfrac{R_u}{\phi_c(0.85 f_c')}$ $= \dfrac{90.0 \text{ kips}}{0.60(0.85)(3 \text{ ksi})}$ $= 58.8 \text{ in}^2$	$A_{1\,req} = \dfrac{R_a\Omega_c}{(0.85 f_c')}$ $= \dfrac{60.0 \text{ kips}(2.50)}{(0.85)(3 \text{ ksi})}$ $= 58.8 \text{ in}^2$	Eqn. J8-1
$B_{req} = \dfrac{A_{1\text{ req}}}{N}$ $= \dfrac{58.8 \text{ in.}^2}{10.0 \text{ in.}}$ $= 5.88 \text{ in.}$	$B_{req} = \dfrac{A_{1\text{ req}}}{N}$ $= \dfrac{58.8 \text{ in.}^2}{10.0 \text{ in.}}$ $= 5.88 \text{ in.}$	
Use B = 8 in. (least whole-inch dimension that exceeds b_f)	Use B = 8 in. (least whole-inch dimension that exceeds b_f)	
Calculate required bearing-plate thickness.	*Calculate required bearing-plate thickness.*	
$n = \dfrac{B}{2} - k_{des}$ $= \dfrac{8.00 \text{ in.}}{2} - 0.972 \text{ in.}$ $= 3.03 \text{ in.}$	$n = \dfrac{B}{2} - k_{des}$ $= \dfrac{8.00 \text{ in.}}{2} - 0.972 \text{ in.}$ $= 3.03 \text{ in.}$	Manual Part 14
$t_{min} = \sqrt{\dfrac{2.22 R_u n^2}{A_1 F_y}}$ $= \sqrt{\dfrac{2.22(90.0 \text{ kips})(3.03 \text{ in.})^2}{(10.0 \text{ in.})(8.00 \text{ in.})(36 \text{ ksi})}}$ $= 0.798 \text{ in.}$	$t_{min} = \sqrt{\dfrac{3.34 R_a n^2}{A_1 F_y}}$ $= \sqrt{\dfrac{3.34(60.0 \text{ kips})(3.03 \text{ in.})^2}{(10.0 \text{ in.})(8.00 \text{ in.})(36 \text{ ksi})}}$ $= 0.799 \text{ in.}$	
Use PL⅞×10×0’-8”	Use PL⅞×10×0’-8”	

Solution C:

N = 6.50 in.

From Solution A, local web yielding and web crippling are not critical.

Try B = 8 in.

$A_1 = B \times N$ = (8.00 in.)(6.50 in.) = 52.0 in.^{2}

To determine the dimensions of the area A_2, the load is spread into the concrete at a slope of 2:1 until an edge or the maximum condition $\sqrt{A_2 / A_1} \leq 2$ is met. There is also a requirement that the area A_2 be geometrically similar to A_1 or, in other words, have the same aspect ration as A_1.

N_1 = 6.50 in. + 2(1.75 in.) = 10.0 in.

$$\frac{B}{N} = \frac{8.00\text{ in.}}{6.50\text{ in.}} = 1.23$$

B_1 = 1.23(10.0 in.) = 12.3 in.

$A_2 = B_1 \times N_1$ = 12.3 in. (10.0 in.) = 123 in.^{2}

Check $\sqrt{\dfrac{A_2}{A_1}} = \sqrt{\dfrac{123\text{ in.}^2}{52.0\text{ in.}^2}} = 1.54 \leq 2$ **o.k.**

$$\begin{aligned} P_p &= 0.85 f'_c A_1 \sqrt{A_2 / A_1} \leq 1.7 f'_c A_1 \\ &= 0.85(3\text{ ksi})(52.0\text{ in.}^2)(1.54) \\ &= 204\text{ kips} \end{aligned}$$

LRFD	ASD	
$\phi_c = 0.60$ $\phi_c P_p = 0.60(204\text{ kips}) = 122\text{ kips}$ 122 kips > 90.0 kips **o.k.**	$\Omega_c = 2.50$ $\dfrac{P_p}{\Omega} = \dfrac{204\text{ kips}}{2.50} = 81.6\text{ kips}$ 81.6 kips > 60.0 kips **o.k.**	Eqn. J8-2
Calculate the required bearing-plate thickness.	*Calculate the required bearing-plate thickness.*	
$n = \dfrac{B}{2} - k$ $= \dfrac{8.00\text{ in.}}{2} - 0.972\text{ in.} = 3.03\text{ in.}$	$n = \dfrac{B}{2} - k$ $= \dfrac{8.00\text{ in.}}{2} - 0.972\text{ in.} = 3.03\text{ in.}$	
$t_{min} = \sqrt{\dfrac{2.22 R_u n^2}{A_1 F_y}}$	$t_{min} = \sqrt{\dfrac{3.33 R_a n^2}{A_1 F_y}}$	Manual Part 14

LRFD	ASD
$= \sqrt{\dfrac{2.22(90.0 \text{ kips})(3.03 \text{ in.})^2}{(6.50 \text{ in.})(8.00 \text{ in.})(36 \text{ ksi})}}$ $= 0.990$ in. Use PL1×6½×0’-8”	$= \sqrt{\dfrac{3.34(60.0 \text{ kips})(3.03 \text{ in.})^2}{(6.50 \text{ in.})(8.00 \text{ in.})(36 \text{ ksi})}}$ $= 0.991$ in. Use PL1×6½×0’-8”

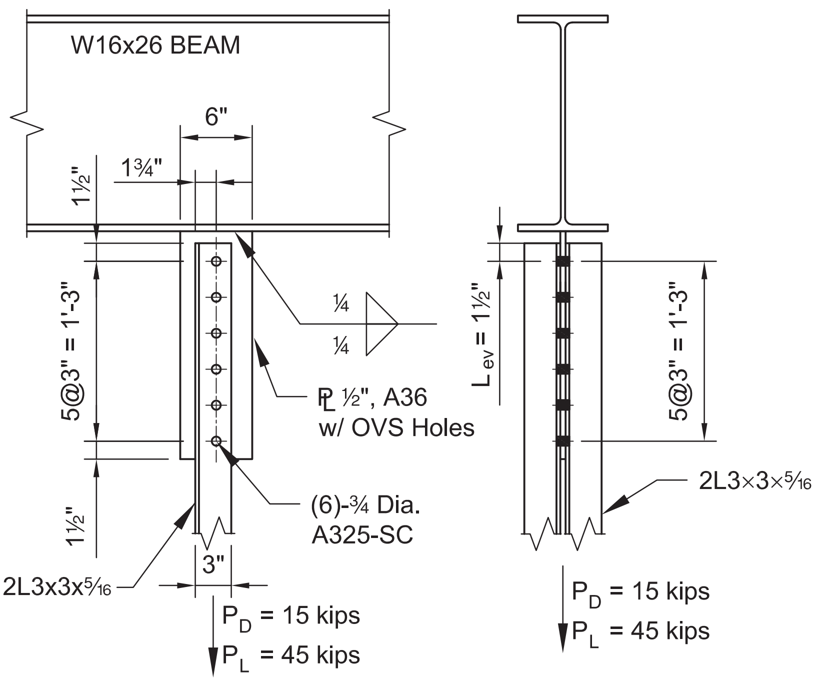
Example II.D-3 Slip-Critical Connection with Oversized Holes

Given:

Determine the number of bolts required to connect, 2L3×3×¼ tension member to a plate under a beam as shown. The angles have standard holes and the plate has oversized holes per Manual Table J3.3.

R_D = 15 kips
R_L = 45 kips

Use ¾-in. diameter ASTM A325-SC class A bolts.
Assume that the strength of the beam, angles, and plate have been checked.



Material Properties:

Beam	W16×26	ASTM A992	F_y = 36 ksi	F_u = 65 ksi	Manual Table 2-3 and 2-4
Hangers	2L3×3×5⁄16	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	
Plate Material		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W16×26	t_f = 0.345 in.	t_w = 0.250 in.	k_{des} = 0.747 in.	Manual Tables 1-1, 1-7 & 1-15
Hangers	2L3×3×5⁄16	A = 3.55 in.^{2}	$\bar{x}$ = 0.860 in.		
Plate Material		t_p = 0.500 in.			

Solution:

LRFD	ASD
R_u = (1.2)(15 kips) + (1.6)(45 kips) = 90.0 kips *Design of bolts* Because of oversized holes, the slip is a strength limit state, Class A faying surface, ¾ in. diameter ASTM A325-SC bolts in double shear ϕr_n = 16.0 kips/bolt $n = \dfrac{\phi R_n}{\phi r_n} = \dfrac{90.0 \text{ kips}}{16.0 \text{ kips/bolt}}$ $= 5.63 \rightarrow$ 6 bolts Try (5) ¾–in. dia. ASTM A325-SC bolts *Check bolt shear strength* $\phi r_n = \phi F_v A_b$ = 31.8 kips/bolt $\phi R_n = \phi r_n n$ = (31.8 kips/bolt)(6 bolts) = 191 kips > 90.0 kips **o.k.**	R_a = 15 kips + 45 kips = 60.0 kips *Design of bolts* Because of oversized holes, the slip is a strength limit state, Class A faying surface, ¾ in. diameter ASTM A325-SC bolts in double shear r_n/Ω = 10.8 kips/bolt $n = \dfrac{(R_n/\Omega)}{(r_n/\Omega)} = \dfrac{60.0 \text{ kips}}{10.8 \text{ kips/bolt}}$ $= 5.56 \rightarrow$ 6 bolts Try (5) ¾–in. dia. ASTM A325-SC bolts *Check bolt shear strength* $r_n/\Omega = \dfrac{F_v A_b}{\Omega}$ = 21.2 kips/bolt $R_n/\Omega = \dfrac{r_n}{\Omega} n$ = (21.2 kips/bolt)(6 bolts) = 127 kips > 60.0 kips **o.k.**

Section J3.8

Manual Table 7-4

Manual Table 7-1

Check tension yield strength of the angles

$$R_n = F_y A_g = (36 \text{ ksi})(3.55 \text{ in.}^2) = 128 \text{ kips}$$ Eqn. D2-1

LRFD	ASD
$\phi_t = 0.90$ $\phi_t R_n = 0.90(128 \text{ kips})$ $= 115 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\dfrac{R_n}{\Omega} = \dfrac{128 \text{ kips}}{1.67}$ = 76.6 kips > 60.0 kips **o.k.**

Check tension rupture strength of the angles

Calculate the effective net area

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.860 \text{ in.}}{15.0 \text{ in.}} = 0.943$$ Table D3.1 Case 2

$$A_e = UA_n = 0.943\left[3.55 \text{ in.}^2 \text{-} 2(0.313 \text{ in.})(0.875 \text{ in.})\right] = 2.83 \text{ in.}^2$$

$$R_n = F_u A_e = (58 \text{ ksi})(2.83 \text{ in.}^2) = 164 \text{ kips}$$ Eqn. D2-2

LRFD	ASD
$\phi_t = 0.75$ $\phi_t R_n = 0.75(164 \text{ kips}) = 123 \text{ kips} > 90.0$ **o.k.**	$\Omega_t = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{164 \text{ kips}}{2.00} = 82.0 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

Check the block shear strength of the angles

Use a single column of bolts $(U_{bs} = 1)$, $n = 6$, $L_{ev} = 1½$ in., and $L_{eh} = 1¼$ in.

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$ Eqn. J4-5

Shear Yielding Component

$$A_{gv} = \left[5(3.00 \text{ in.}) + 1.50 \text{ in.}\right](0.313 \text{ in.}) = 5.16 \text{ in.}^2$$

$$0.6F_y A_{gv} = 0.6(36 \text{ ksi})\left(5.16 \text{ in.}^2\right) = 111 \text{ kips}$$

Shear Rupture Component

$$A_{nv} = 5.16 \text{ in.}^2 - 5.5(0.875 \text{ in.})(0.313 \text{ in.}) = 3.65 \text{ in.}^2$$

$$0.6F_u A_{nv} = 0.6(58 \text{ ksi})\left(3.65 \text{ in.}^2\right) = 127 \text{ kips}$$

Tension Rupture Component

$$A_{nt} = \left[1.25 \text{ in.} - 0.5(0.875 \text{ in.})\right](0.313 \text{ in.}) = 0.254 \text{ in.}^2$$

$$U_{bs} F_u A_{nt} = 1.0(58 \text{ ksi})\left(0.254 \text{ in.}^2\right) = 14.7 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(111 \text{ kips} + 14.7 \text{ kips})$ $= 189 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{2(111 \text{ kips} + 14.7 \text{ kips})}{2.00}$ $= 126 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

Check the bearing / tear out of the angles (Holes are STD; 13/16 in. diameter),

Check strength for edge bolt.

$$L_c = 1.50 \text{ in.} - \frac{(0.813 \text{ in.})}{2} = 1.09 \text{ in.}$$

$$r_n = 1.2L_c t F_u \leq 2.4dtF_u$$ Eqn. J3-6a

$$= 1.2(1.09 \text{ in.})(0.313 \text{ in.})(58 \text{ ksi}) \leq 2.4(0.750 \text{ in.})(0.313 \text{ in.})(58 \text{ ksi})$$

$$= 23.7 \text{ kips} \leq 32.7 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(32.7 \text{ kips}) = 24.5 \text{ kips} > 16.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{32.7 \text{ kips}}{2.00} = 16.4 \text{ kips} > 10.8 \text{ kips}$ **o.k.**

Check tension yield strength of the ½" plate

Note: By inspection, the Whitmore section includes the entire width of the ½" plate.

$R_n = F_y A_g = (36 \text{ ksi})(0.500 \text{ in.})(6.00 \text{ in.}) = 108 \text{ kips}$ Eqn. J4-1

LRFD	ASD
$\phi_t = 0.90$ $\phi R_n = 0.90(108 \text{ kips})$ $= 97.2 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\Omega_t = 1.67$ $\dfrac{R_n}{\Omega_t} = \dfrac{108 \text{ kips}}{1.67}$ $= 64.7 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

Check tension rupture strength of the ½" plate (Holes are OVS; 15⁄16 in.)

Calculate the effective net area

$$A_e = \left[3.00 \text{ in.}^2 - (0.500 \text{ in.})(1.00 \text{ in.})\right] = 2.50 \text{ in.}^2$$

$R_n = F_u A_e = (58 \text{ ksi})(2.50 \text{ in.}^2) = 145 \text{ kips}$ Eqn. J4-2

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(145 \text{ kips})$ $= 109 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{145 \text{ kips}}{2.00}$ $= 72.5 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

Check the bearing / tear out strength of the ½" plate (Holes are OVS; 15⁄16 in.)

Check strength for edge bolt

$$L_c = 1.50 \text{ in.} - \frac{0.938 \text{ in.}}{2} = 1.03 \text{ in.}$$

$$\begin{aligned} r_n &= 1.2 L_c t F_u \le 2.4 d t F_u \\ &= 1.2(1.03 \text{ in.})(0.500 \text{ in.})(58 \text{ ksi}) \le 2.4(0.750)(0.500 \text{ in.})(58 \text{ ksi}) \\ &= 35.8 \text{ kips} \le 52.2 \text{ kips} \end{aligned}$$

Eqn. J3-6a

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(35.8 \text{ kips}) = 26.9 \text{ kips} > 16.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{35.8 \text{ kips}}{2.00} = 17.9 \text{ kips} > 10.8 \text{ kips}$ **o.k.**

Check strength for interior bolts.

$L_c = 3.00 \text{ in.} - 0.938 \text{ in.} = 2.06 \text{ in.}$

$$\begin{aligned} r_n &= 1.2L_c tF_u \le 2.4dtF_u \\ &= 1.2(2.06 \text{ in.})(0.500 \text{ in.})(58.0 \text{ ksi}) \le 2.4(0.750)(0.500 \text{ in.})(58.0 \text{ ksi}) \\ &= 71.7 \text{ kips} \le 52.2 \text{ kips} \end{aligned}$$ Eqn. J3-6a

LRFD	ASD
$\phi = 0.75$ $\phi r_n = 0.75(52.2 \text{ kips}) = 39.2 \text{ kips} > 16.0 \text{ kips}$ **o.k.**	$\Omega = 2.00$ $\dfrac{r_n}{\Omega} = \dfrac{52.2 \text{ kips}}{2.00} = 26.1 \text{ kips} > 10.8 \text{ kips}$ **o.k.**

Calculate the fillet weld required for the ½" plate to WF beam

Because the angle of the force relative to the axis of the weld is 90 degrees, the strength of the weld can be increased:

$$(1.0 + 0.50\sin^{1.5}\theta) = (1.0 + 0.50\sin^{1.5} 90) = 1.50$$

Determine the required weld size

LRFD	ASD
$D_{req} = \dfrac{R_u}{1.5(1.392l)}$ $= \dfrac{90.0 \text{ kips}}{1.5(1.392)(2)(6.00 \text{ in.})}$ $= 3.59 \text{ sixteenths}$	$D_{req} = \dfrac{P_a}{1.5(0.928l)}$ $= \dfrac{60.0 \text{ kips}}{1.5(0.928)(2)(6.00 \text{ in.})}$ $= 3.59 \text{ sixteenths}$

The minimum weld size is 3⁄16 in. Manual Table J2.4

Use a ¼ in. fillet weld on both sides of the plate.

Check the minimum thickness of the beam flange required to match the required shear rupture strength of the fillet welds. Manual Part 9

$$t_{\min} = \frac{3.09D}{F_u} = \frac{3.09(3.59 \text{ sixteenths})}{65 \text{ ksi}} = 0.171 \text{ in.} < 0.345 \text{ in.} \quad \textbf{o.k.}$$

Check the W16x26 beam for concentrated forces

Check web local yielding (Assume the connection is at a distance from the member end greater than the depth of the member d.)

$$\begin{aligned} R_n &= (5k + N)F_{yw}t_w \\ &= \left[5(0.747 \text{ in.}) + 6.00 \text{ in.}\right](50 \text{ ksi})(0.250 \text{ in.}) = 122 \text{ kips} \end{aligned}$$ Eqn. J10-2

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(122 \text{ kips})$ $= 122 \text{ kips} > 90.0 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{122 \text{ kips}}{1.50} = 81.3 \text{ kips} > 60.0 \text{ kips}$ **o.k.**

III. SYSTEM DESIGN EXAMPLES

DESIGN OF SELECTED MEMBERS AND LATERAL ANALYSIS OF A 4 STORY BUILDING

INTRODUCTION

This section illustrates the load determination and selection of representative members that are part of the gravity and lateral frame of a typical 4-story building. The design is completed in accordance with the 2010 AISC *Specification for Structural Steel Buildings* and the 14th Edition AISC *Steel Construction Manual*. Building code requirements are taken from International Building Code 2006 as the Design Code and loading criteria are based on SEI/ASCE 7-05.

This section includes:

- Analysis and design of a typical steel frame for gravity loads
- Analysis and design of a typical steel frame for lateral loads
- Examples illustrating three methods for satisfying the stability provisions of AISC Specification Chapter C.

The building being analyzed in this design example is located in a Midwestern city with moderate wind and seismic loads. The criteria for these minimum loads are given in the description of the design example.

CONVENTIONS

The following conventions are used throughout this example:

1. Beams or columns that have similar, but not necessarily identical, loads are grouped together. This is done to simplify the selection process, because such grouping is generally a more economical practice for design, fabrication, and erection.

2. Certain calculations, such as design loads for snow drift, which might typically be determined using a spreadsheet or structural analysis program, are summarized and then incorporated into the analysis. This simplifying feature allows the design example to illustrate concepts relevant to the member selection process.

3. Two commonly used deflection calculations, for uniform loads, have been rearranged so that the conventional units in the problem can be directly inserted into the equation for steel design. They are as follows:

Simple Beam:

$$\Delta = \frac{5\,w\text{ kip/in.}\left(L\text{ in.}\right)^4}{384\left(29{,}000\text{ ksi}\right)\left(I\text{ in.}\right)^4} = \frac{w\text{ kip/ft}\left(L\text{ ft}\right)^4}{1290\left(I\text{ in.}\right)^4}$$

Beam Fixed at both Ends:

$$\Delta = \frac{w\text{ kip/in.}\left(L\text{ in.}\right)^4}{384\left(29{,}000\text{ ksi}\right)\left(I\text{ in.}\right)^4} = \frac{w\text{ kip/ft}\left(L\text{ ft}\right)^4}{6440\left(I\text{ in.}\right)^4}$$

DESIGN SEQUENCE

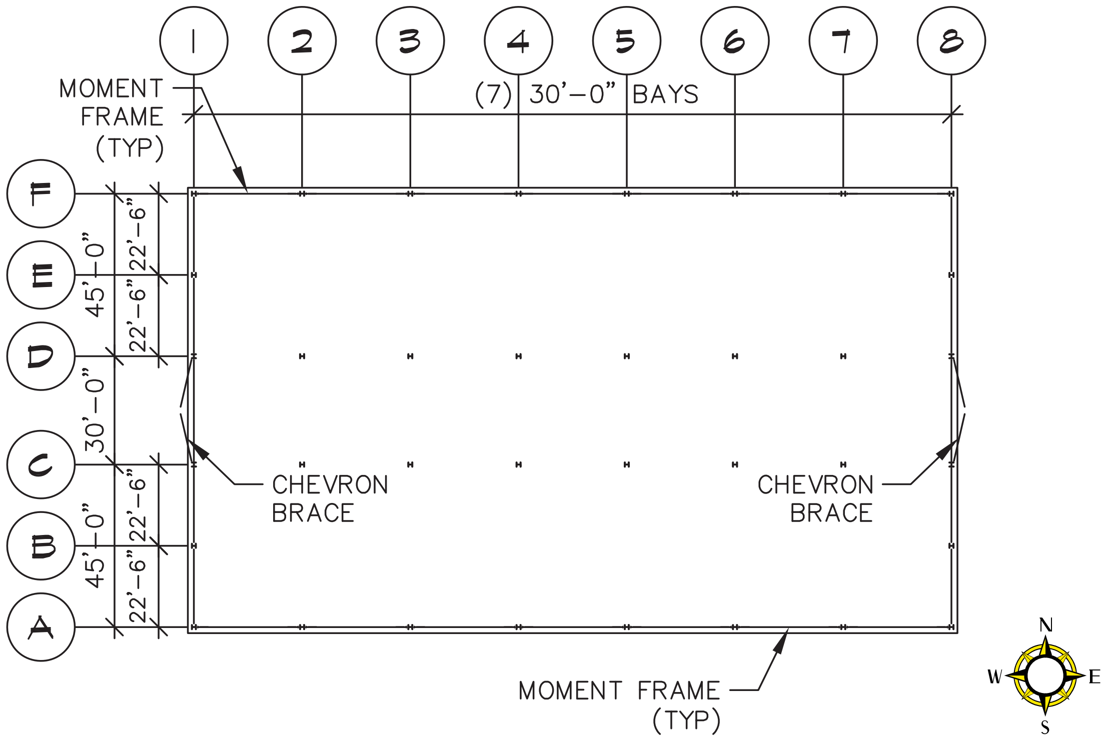
The design sequence is presented as follows:

1. General description of the building including geometry, gravity loads, and lateral loads.
2. Roof member design and selection.
3. Floor member design and selection.
4. Column design and selection for gravity loads.
5. Wind load determination.
6. Seismic load determination.
7. Horizontal force distribution to the lateral frames.
8. Preliminary column selection for the moment frames and braced frames.
9. Seismic load application to lateral systems.
10. Second order effects (*P*-delta) analysis

GENERAL DESCRIPTION OF THE BUILDING

Geometry

The design example is a 4-story building, comprised of 7 bays at 30 ft in the East-West (numbered grids) direction and bays of 45 ft, 30 ft and 45 ft in the North-South (lettered grids) direction. The floor-to-floor height for the 4 floors is 13 ft-6 in. and the height from the fourth floor to the roof (at the edge of the building) is 14 ft-6 in. Based on discussions with fabricators, the same column size will be used for the whole height of the building.



Basic Building Layout

The plans of these floors and the roof are shown on sheets S2.1 thru S2.3, found at the end of this Chapter. The exterior of the building is a ribbon window system with brick spandrels supported and back-braced with steel and in-filled with metal studs. The spandrel wall extends 2 ft above the elevation of the edge of the roof. The window and spandrel system is shown on design drawing Sheet S4.1.

The roof system is 1½-in. metal deck on bar joists. These bar joists are supported on steel beams as shown on Design Drawing Sheet S2.3. The roof slopes to interior drains. The middle 3 bays have a 6 ft tall screen wall around them and house the mechanical equipment and the elevator over run. This area has steel beams, in place of steel bar joists, to support the mechanical equipment.

The three elevated floors have 3 in. of normal weight concrete over 3 in. composite deck for a total slab thickness of 6 in. The supporting beams are spaced at 10 ft on center. These beams are carried by composite girders in the East-West direction to the columns. There is a 30 ft by 45 ft opening in the second floor, to create a 2 story atrium at the entrance. These floor layouts are shown on Drawings S2.1 and S2.2. The first floor is a slab on grade and the foundation consists of conventional spread footings.

The building includes both moment frames and braced frames for lateral resistance. The lateral system in the North-South direction consists of chevron braces at the end of the building, located adjacent to the stairways. In the East-West direction there are no locations in which chevron braces can be concealed; consequently, the lateral system in the East-West direction is composed of moment frames at the North and South faces of the building.

This building is sprinklered and has large open spaces around it, and consequently does not require fire proofing for the floors.

Wind Forces

The Basic Wind Speed is 90 miles per hour (3 second gust). Because it is sited in an open, rural area, it will be analyzed as Wind Exposure Category C. Because it is an ordinary (Category II) office occupancy, the wind importance factor is 1.0.

Seismic Forces

The sub-soil has been evaluated and the site class has been determined to be Category D. The area has a short period $S_s = 0.121g$ and a one-second period $S_1 = 0.060g$. The seismic importance factor is 1.0, that of an ordinary office occupancy (Category II).

Roof and Floor Loads

Roof loads:

The ground snow load (p_g) is 20 psf. The slope of the roof is ¼ in./ft or more at all locations, but not exceeding ½ in./ft; consequently, 5 psf rain-on-snow surcharge is to be considered, but ponding instability design calculations are not required. This roof can be designed as a fully exposed roof, but, per ASCE 7 Section 7.3, cannot be designed for less than $p_f = (I)p_g = 20$ psf uniform snow load. Snow drift will be applied at the edges of the roof and at the screen wall around the mechanical area. The roof live load for this building is 20 psf, but may be reduced per ASCE 7 section 4.9 where applicable.

Floor Loads:

The basic live load for the floor is 50 psf. An additional partition live load of 20 psf is specified. Because the locations of partitions and, consequently, corridors are not known, and will be subject to change, the entire floor will be designed for a live load of 80 psf. This live load will be reduced, based on type of member and area per the IBC provisions for live-load reduction.

Wall Loads:

A wall load of 55 psf will be used for the brick spandrels, supporting steel and metal stud back-up. A wall load of 15 psf will be used for the ribbon window glazing system.

ROOF MEMBER DESIGN AND SELECTION

Calculate dead load and snow load

Dead Load

Roofing	= 5 psf	
Insulation	= 2 psf	
Deck	= 2 psf	
Beams	= 3 psf	
Joists	= 3 psf	
Misc.	= 5 psf	
Total	= 20 psf	

Snow Load

Snow	= 20 psf	ASCE 7
Rain on Snow	= 5 psf	Section 7.3
Total	= 25 psf	and 7.10

Note: In this design, the Rain and Snow Load is greater than the Roof Live Load

The deck is 1½ in., wide rib, 22 gage, painted roof deck, placed in a pattern of 3 continuous spans minimum. The typical joist spacing is 6 ft on center. At 6 ft on center, this deck has an allowable total load capacity of 89 psf. The roof diaphragm and roof loads extend 6 in. past the centerline of grid as shown on Drawing S4.1.

Flat roof snow load = 20 psf, Density γ = 16.6 lbs/ft^3, h_b = 1.20 ft

ASCE 7 Section 7.7

Summary of Drifts

ASCE 7 Figure 7-8

	Upwind Roof Length (l_u)	**Proj. Height**	**Max. Drift Load**	**Max Drift Width (W)**
Side Parapet	121 ft	2 ft	13.2 psf	6.36 ft
End Parapet	211 ft	2 ft	13.2 psf	6.36 ft
Screen Wall	60.5 ft	6 ft	30.5 psf	7.35 ft

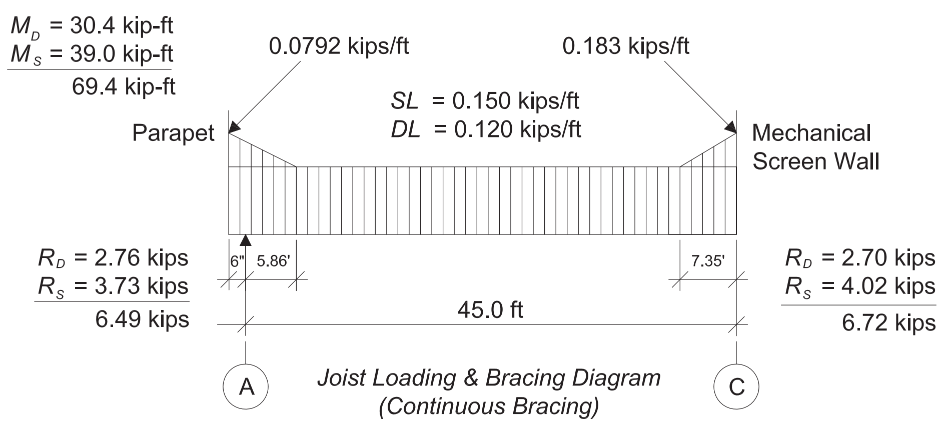
Summary from Spread Sheet

SELECT ROOF JOISTS

Layout loads and size joists

User Note: Joists are commonly designed by ASD and are designed and selected in the manner shown here.

The 45-ft side joist with the heaviest loads is shown below along with its end reactions and maximum moment:



Joist Loading & Bracing Diagram (Continuous Bracing)

Because the load is not uniform, select a 24KCS4 JOIST AT 16.5 plf, which has an allowable moment of 92.3 kip-ft, an allowable shear of 8.40 kips, and a gross moment of inertia of 453 in.^{4}

Steel Joist Institute Load Tables

The first joist away from the end of the building is loaded with snow drift along the length of the member. Based on analysis, a 24KCS4 joist is also acceptable for this uniform load case.

As an alternative to directly specifying the joist sizes on the design document, as done in this example, loading diagrams can be included on the design documents to allow the joist manufacturer to economically design the joists.

The typical 30 ft joist in the middle bay will have a uniform load of

w = (20 psf + 25 psf)(6 ft) = 270 plf

w_{SL} = (25 psf)(6 ft) = 150 plf

Per the Steel Joist Institute load tables, select an 18K5 joist which weighs approximately 7.7 plf and satisfies both strength and deflection requirements.

Note: the first joist away from the screen wall and the first joist away from the end of the building carry snow drift. Based on analysis, an 18K7 joist will be used in these locations.

SELECT ROOF BEAMS

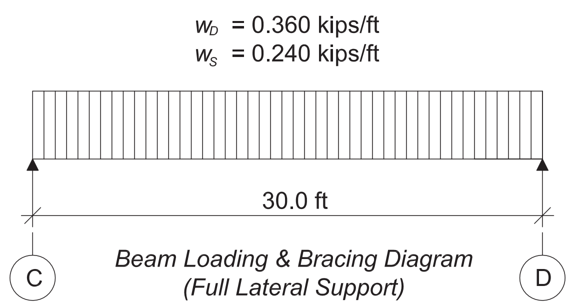
Calculate loads and select beams in the mechanical area

For the beams in the mechanical area, the mechanical units could weigh as much as 60 psf. Use 40 psf additional dead load, which will account for the mechanical units and the screen wall around the mechanical area. Use 15 psf additional snow load, which will account for any snow drift which could occur in the mechanical area. The beams in the mechanical area are spaced at 6 ft on center.

Calculate the minimum I_x to limit deflection to l/360 = 1.00 in. because a plaster ceiling will be used in the lobby area. Use 40 psf as an estimate of the snow load, including some drifting that could occur in this area, for deflection calculations. IBC Table 1604.3

Note: The beams and supporting girders in this area should be rechecked when the final weights and locations for the mechanical units have been determined.

$$I_{req}\text{ (Live Load)} = \frac{0.240\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(1.00\text{ in.}\right)} = 151\text{ in.}^4$$



Beam Loading & Bracing Diagram
(Full Lateral Support)

Calculate the required strengths and select the beams in the mechanical area

LRFD	ASD
W_u = 6.00 ft[1.2 (0.020 kip/ft^2 + 0.040 kip/ft^2) +1.6(0.025 kip/ft^2 + 0.015 kip/ft^2)] = 0.816 kip/ft	W_a = 6.00 ft[0.020 kip/ft^2 + 0.040 kip/ft^2 + 0.025 kip/ft^2 + 0.015 kip/ft^2] = 0.600 kip/ft
$R_u = \dfrac{30.0\text{ ft}}{2}\left(0.816\text{ kip/ft}\right) = 12.2\text{ kips}$	$R_a = \dfrac{30.0\text{ ft}}{2}\left(0.600\text{ kip/ft}\right) = 9.00\text{ kips}$
$M_u = \dfrac{0.816\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 91.8\text{ kip-ft}$	$M_a = \dfrac{0.600\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 67.5\text{ kip-ft}$
Select a W14×22, which has a design flexural strength of 125 kip-ft, an available shear of 94.8 kips, and an I_x of 199 in.^{4}	Select a W14×22, which has an allowable flexural strength of 82.8 kip-ft, an allowable shear strength of 63.2 kips and an I_x of 199 in.^{4}

Manual Table 3-2

Note: a W12×22 beam would also meet all criteria, but a 14 in. beam was selected to match the depth of other beams used throughout the building.

SELECT BEAMS AT THE END OF THE BUILDING

The beams at the ends of the building carry the brick spandrel panel and a small portion of roof load. For these beams, the cladding weight exceeds 25 percent of the total dead load on the beam. Therefore, per AISC Design Guide 3, Second Edition, limit the vertical deflection due to cladding and initial dead load to L/600 or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to L/600 or 0.3 in. max to accommodate the brick and L/360 or 0.25 in. max to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to L/600 or 0.25 in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. In calculating the wall loads, the spandrel panel weight is taken as 55 psf. The spandrel panel weight is approximately:

w_D = 7.50 ft(0.055 kip/ft^2) = 0.413 kip/ft

The dead load from the roof is equal to:

w_D = 3.50 ft(0.020 kip/ft^2) = 0.070 kip/ft

Use 8 psf for the initial dead load.

$w_{D(initial)}$ = 3.50 ft(0.008 kip/ft^2) = 0.0280 kip/ft

Use 12 psf for the superimposed dead load.

$w_{D(super)}$ = 3.50 ft(0.012 kip/ft^2) = 0.0420 kip/ft

The snow load from the roof can be conservatively taken as:

w_S = 3.50 ft(0.025 kip/ft^2 + 0.0132 kip/ft^2) = 0.134 kip/ft

to account for the maximum snow drift as a uniform load.

Assume the beams are simple spans of 22.5 ft.

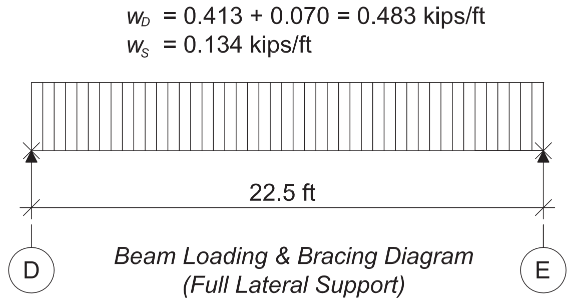
Calculate minimum I_x to limit the superimposed dead live load deflection to ¼ in.

$$I_{req} = \frac{0.176 \text{ kip/ft}\left(22.5 \text{ ft}\right)^4}{1290\left(0.250 \text{ in.}\right)} = 140 \text{ in.}^4$$

Calculate minimum I_x to limit the cladding and initial dead load deflection to ⅜ in.

$$I_{req} = \frac{0.441 \text{ kip/ft}\left(22.5 \text{ ft}\right)^4}{1290\left(0.375 \text{ in.}\right)} = 234 \text{ in.}^4$$

The loading diagram is as follows:



Calculate the required strengths and select the beams for the roof ends

LRFD	ASD
W_u =1.2(0.070 kip/ft + 0.413 kip/ft) +1.6(0.134 kip/ft) = 0.794 kip/ft	W_a = (0.070 kip/ft + 0.413 kip/ft) + 0.134 kip/ft = 0.617 kip/ft
$R_u = \dfrac{22.5\text{ ft}}{2}\left(0.794\text{ kip/ft}\right) = 8.93\text{ kips}$	$R_a = \dfrac{22.5\text{ ft}}{2}\left(0.617\text{ kip/ft}\right) = 6.94\text{ kips}$
$M_u = \dfrac{0.794\text{ kip/ft}\left(22.5\text{ ft}\right)^2}{8} = 50.2\text{ kip-ft}$	$M_a = \dfrac{0.617\text{ kip/ft}\left(22.5\text{ ft}\right)^2}{8} = 39.0\text{ kip-ft}$
Select W16×26, which has a design flexural strength of 166 kip-ft, a design shear strength of 106 kips, and an I_x of 301 in.^{4}	Select W16×26, which has an allowable flexural strength of 110 kip-ft, an allowable shear strength of 70.5 kips, and an I_x of 301 in.^{4}

Manual Table 3-2

SELECT THE BEAM ALONG THE SIDE OF THE BUILDING

The beams along the side of the building carry the spandrel panel and a substantial roof dead load and live load. For these beams, the cladding weight exceeds 25 percent of the total dead load on the beam. Therefore, per AISC Design Guide 3, Second Edition, limit the vertical deflection due to cladding and initial dead load to L/600 or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to L/600 or 0.3 in. max to accommodate the brick and L/360 or 0.25 in. max to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to L/600 or 0.25 in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. These beams will be part of the moment frames on the side of the building and therefore will be designed as fixed at both ends. The roof dead load and snow load to this edge beam is equal to the joist end dead load and snow load reaction. Treating this as a uniform load, divide this by the joist spacing.

w_D = 2.76 kips/6.00 ft = 0.460 kip/ft

w_S = 3.73 kips/6.00 ft = 0.622 kip/ft

$w_{D(initial)}$ = 23.0 ft (0.008 kip/ft^2) = 0.184 kip/ft

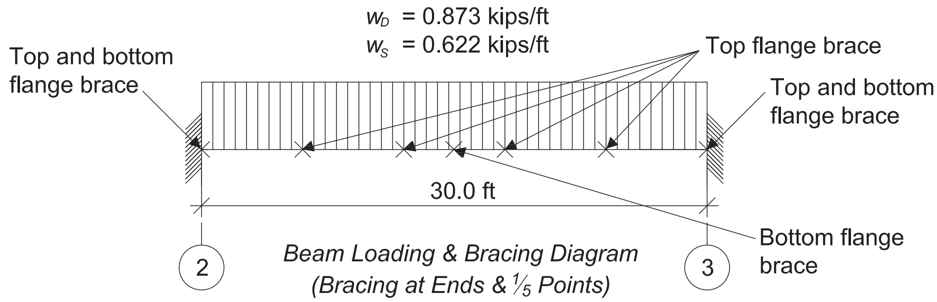
$w_{D(super)}$ = 23.0 ft (0.012 kip/ft^2) = 0.276 kip/ft

Calculate minimum I_x to limit the superimposed dead and live load deflection to ¼ in.

$$I_{req} = \frac{\left(0.898\text{ kip/ft}\right)\left(30.0\text{ ft}\right)^4}{6440\left(0.250\text{ in.}\right)} = 452\text{ in.}^4$$

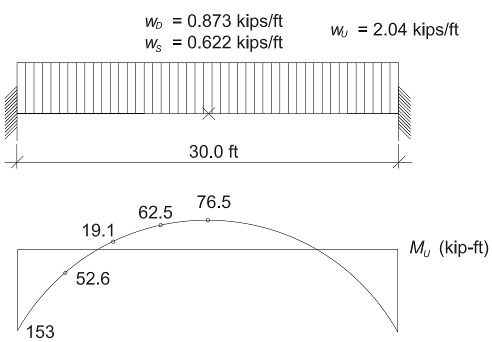
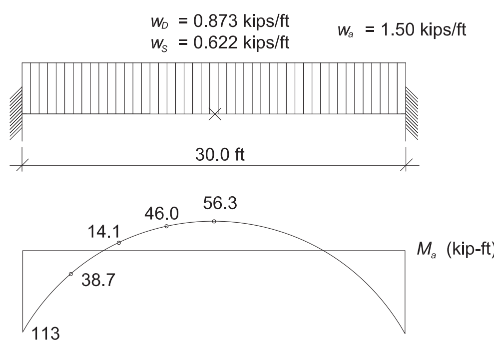
Calculate minimum I_x to limit the cladding and initial dead load deflection to ⅜ in.

$$I_{req} = \frac{(0.597\text{ kip/ft})(30.0\text{ ft})^4}{6440(0.375\text{ in.})} = 200\text{ in.}^4$$



Beam Loading & Bracing Diagram (Bracing at Ends & 1⁄5 Points)

Calculate the required strengths and select the beams for the roof sides

LRFD	ASD
$W_u = 1.2(0.460\text{ kip/ft} + 0.413\text{ kip/ft}) + 1.6(0.622\text{ kip/ft})$ $= 2.04\text{ kip/ft}$	$W_a = (0.460\text{ kip/ft} + 0.413\text{ kip/ft}) + 0.622\text{ kip/ft}$ $= 1.5\text{ kip/ft}$
$R_u = \frac{30.0\text{ ft}}{2}(2.04\text{ kip/ft}) = 30.6\text{ kips}$	$R_a = \frac{30.0\text{ ft}}{2}(1.50\text{ kip/ft}) = 22.5\text{ kips}$
Calculate C_b for compression in the bottom flange braced at the midpoint and supports.	*Calculate C_b for compression in the bottom flange braced at the midpoint and supports.*
 	
$M_{uMax} = \frac{2.04\text{ kip/ft}(30.0\text{ ft})^2}{12} = 153$ kip-ft at supports	$M_{aMax} = \frac{1.50\text{ kip/ft}(30.0\text{ ft})^2}{12} = 113$ kip-ft at supports
$M_u = \frac{2.04\text{ kip/ft}(30.0\text{ ft})^2}{24} = 76.5$ kip-ft at midpoint	$M_a = \frac{1.50\text{ kip/ft}(30.0\text{ ft})^2}{24} = 56.3$ kip-ft at midpoint

LRFD	ASD
$M_{uA} = \dfrac{2.04 \text{ kip/ft}}{12}\begin{pmatrix} 6(30.0 \text{ ft})(3.75 \text{ ft}) \\ -(30.0 \text{ ft})^2 - 6(3.75 \text{ ft})^2 \end{pmatrix}$ $= 52.6$ kip-ft at quarter point of unbraced segment	$M_{aA} = \dfrac{1.50 \text{ kip/ft}}{12}\begin{pmatrix} 6(30.0 \text{ ft})(3.75 \text{ ft}) \\ -(30.0 \text{ ft})^2 - 6(3.75 \text{ ft})^2 \end{pmatrix}$ $= 38.7$ kip-ft at quarter point of unbraced segment
$M_{uB} = \dfrac{2.04 \text{ kip/ft}}{12}\begin{pmatrix} 6(30.0 \text{ ft})(7.50 \text{ ft}) \\ -(30.0 \text{ ft})^2 - 6(3.25 \text{ ft})^2 \end{pmatrix}$ = 19.1 kip-ft at midpoint of unbraced segment	$M_{aB} = \dfrac{1.50 \text{ kip/ft}}{12}\begin{pmatrix} 6(30.0 \text{ ft})(7.50 \text{ ft}) \\ -(30.0 \text{ ft})^2 - 6(7.58 \text{ ft})^2 \end{pmatrix}$ = 14.1 kip-ft at midpoint of unbraced segment
$M_{uC} = \dfrac{2.04 \text{ kip/ft}}{12}\begin{pmatrix} 6(30.0 \text{ ft})(11.3 \text{ ft}) \\ -(30.0 \text{ ft})^2 - 6(11.3 \text{ ft})^2 \end{pmatrix}$ = 62.5 kip-ft at three quarter point of unbraced segment	$M_{aC} = \dfrac{1.50 \text{ kip/ft}}{12}\begin{pmatrix} 6(30.0 \text{ ft})(11.3 \text{ ft}) \\ -(30.0 \text{ ft})^2 - 6(11.3 \text{ ft})^2 \end{pmatrix}$ = 46.0 kip-ft at three quarter point of unbraced segment
$C_b = \dfrac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} R_m \le 3.0$ $= \dfrac{12.5(153 \text{ kip-ft})}{\begin{bmatrix} 2.5(153 \text{ kip-ft}) + 3(52.6 \text{ kip-ft}) \\ +4(19.1 \text{ kip-ft}) + 3(62.5 \text{ kip-ft}) \end{bmatrix}}$ $= 2.38$	$C_b = \dfrac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} R_m \le 3.0$ $= \dfrac{12.5(113 \text{ kip-ft})}{\begin{bmatrix} 2.5(113 \text{ kip-ft}) + 3(38.7 \text{ kip-ft}) \\ +4(14.1 \text{ kip-ft}) + 3(46.0 \text{ kip-ft}) \end{bmatrix}}$ $= 2.38$
Select W18×35	Select W18×35
For L_b = 6 ft and C_b = 1.0 ϕM_n = 229 kip-ft > 76.5 kip-ft **o.k.**	For L_b = 6 ft and C_b = 1.0 M_n / Ω = 152 kip-ft > 56.3 kip-ft **o.k.**
For L_b = 15 ft and C_b = 2.38 ϕM_n = (109 kip-ft)2.38 = 259 kip-ft ≤ 249 kip-ft = 249 kip-ft > 153 kip-ft **o.k.**	For L_b = 15 ft and C_b = 2.38 M_n / Ω = (72.7 kip-ft)2.38 = 173 kip-ft ≤ 166 kip-ft = 166 kip-ft > 113 kip-ft **o.k.**
A W18x35 has a design shear strength of 159 kips and an I_x of 510 in.^{4} **o.k.**	A W18x35 has an allowable shear strength of 106 kips and an I_x of 510 in.^{4} **o.k.**

Eqn F1-1

Manual Table 3-10

Manual Table 3-2

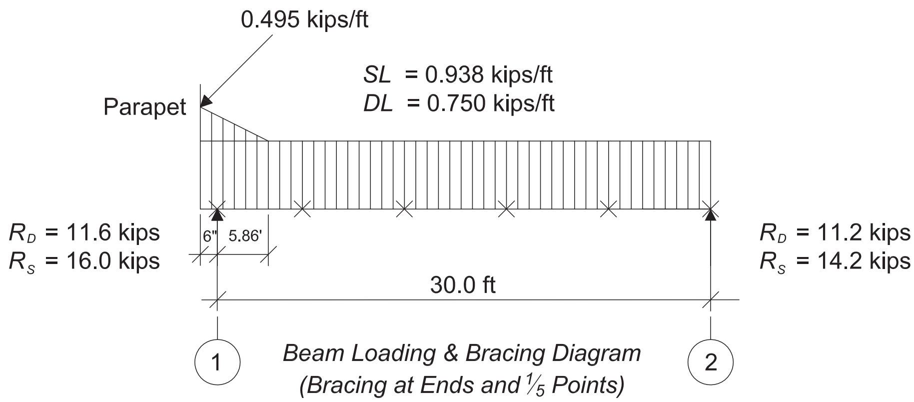
Note: This roof beam may need to be upsized during the lateral load analysis to increase the stiffness and strength of the member and improve lateral frame drift performance.

SELECT THE BEAMS ALONG THE INTERIOR LINES OF THE BUILDING

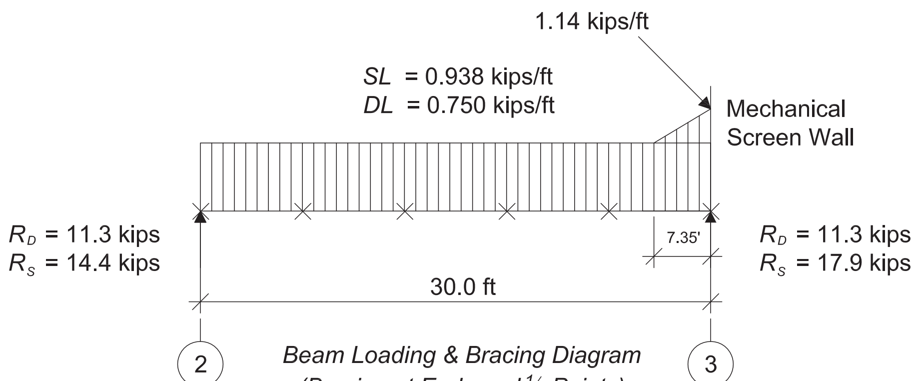
There are 3 individual beam loadings that occur along grids C and D. The beams from 1 to 2 and 7 to 8 have a uniform snow load except for the snow drift at the end at the parapet. The snow drift from the far ends of the 45 foot joists is negligible. The beams from 2 to 3 and 6 to 7 are the same as the first group, except they have snow drift at the screen wall. The loading diagrams are shown below. A summary of the moments, left and right reactions, and required I_x to keep the live load deflection to equal or less than the span divided by 240 (or 1.50 in.) is given below.

IBC Table 1604.3

Note: From footnote g of IBC Table 1604.3, for steel structural members, the dead load shall be taken as zero for D + S deflection calculation combination.



Beam Loading & Bracing Diagram
(Bracing at Ends and ⅕ Points)



Beam Loading & Bracing Diagram
(Bracing at Ends and ⅕ Points)

Summary of required strengths and required moment of inertia

LRFD	ASD
Grids 1 to 2 and 7 to 8 (opposite hand) R_u (left) $= 1.2(11.6 \text{ kips}) + 1.6(16.0 \text{ kips})$ $= 39.5$ kips R_u (right) $= 1.2(11.2 \text{ kips}) + 1.6(14.2 \text{ kips})$ $= 36.2$ kips $M_u = 1.2(84.3 \text{ kip-ft}) + 1.6(107 \text{ kip-ft})$ $= 272$ kip-ft	Grids 1 to 2 and 7 to 8 (opposite hand) R_a (left) $= 11.6 \text{ kips} + 16.0 \text{ kips}$ $= 27.6$ kips R_a (right) $= 11.2 \text{ kips} + 14.2 \text{ kips}$ $= 25.4$ kips $M_a = 84.3 \text{ kip-ft} + 107 \text{ kip-ft}$ $= 191$ kip-ft

LRFD	ASD	
$I_{x\,req'd} = \dfrac{(0.938\text{ klf})(30.0\text{ ft})^4}{1290(1.50\text{ in.})}$ $= 393\text{ in.}^4$	$I_{x\,req'd} = \dfrac{(0.938\text{ klf})(30.0\text{ ft})^4}{1290(1.50\text{ in.})}$ $= 393\text{ in.}^4$	
For L_b = 6 ft and C_b = 1.0, select W21×44 which has design flexural strength of 332 kip-ft, a design shear strength of 217 kips, and I_x = 843 in.^{4}	For L_b = 6 ft and C_b = 1.0, select W21×44 which has an allowable flexural strength of 221 kip-ft, an allowable shear strength of 145 kips, and I_x = 843 in.^{4}	Manual Table 3-2
Grids 2 to 3 and 6 to 7(opposite hand)	Grids 2 to 3 and 6 to 7(opposite hand)	
R_u (left) = 1.2(11.3 kips) + 1.6(14.4 kips) = 36.6 kips	R_a (left) = 11.3 kips + 14.4 kips = 25.7 kips	
R_u (right) = 1.2(11.3 kips) + 1.6(17.9) kips) = 42.2 kips	R_a (right) = 11.3 kips + 17.9 kips = 29.2 kips	
M_u = 1.2(84.4 kip-ft) + 1.6(111 kip-ft) = 279 kip-ft	M_a = 84.4 kip-ft + 111 kip-ft = 195 kip-ft	
$I_{x\,req'd} = \dfrac{(0.938\text{ klf})(30.0\text{ ft})^4}{1290(1.50\text{ in.})}$ $= 393\text{ in.}^4$	$I_{x\,req'd} = \dfrac{(0.938\text{ klf})(30.0\text{ ft})^4}{1290(1.50\text{ in.})}$ $= 393\text{ in.}^4$	
For L_b = 6 ft and C_b = 1.0, select W21×44 which has a design flexural strength of 332 kip-ft, a design shear strength of 217 kips and I_x = 843 in.^{4}	For L_b = 6 ft and C_b = 1.0, select W21×44 which has an allowable flexural strength of 221 kip-ft, an allowable shear strength of 145 kips, and I_x = 843 in.^{4}	Manual Table 3-2

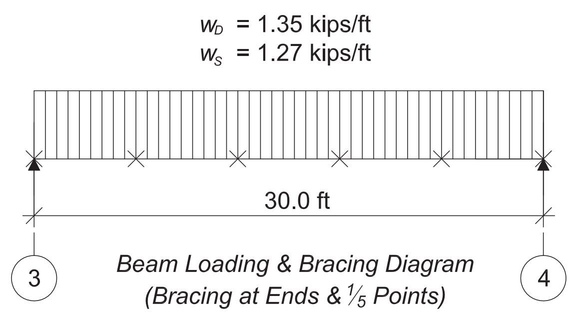
The third individual beam loading occurs at the beams from 3 to 4, 4 to 5, and 5 to 6. This is the heaviest load.

SELECT THE BEAMS ALONG THE SIDES OF THE MECHANICAL AREA

The beams from 3 to 4, 4 to 5, and 5 to 6 have a uniform snow load outside the screen walled area, except for the snow drift at the parapet ends and the screen wall ends of the 45 foot joists. Inside the screen walled area the beams support the mechanical equipment. A summary of the moments, left and right reactions, and required I_x to keep the live load deflection to equal or less than the span divided by 240 (or 1.50 in.) is given below.

IBC Table 1604.3

Note: From footnote g of IBC Table 1604.3, for steel structural members, the dead load shall be taken as zero for D + L deflection calculation load combination.



LRFD	ASD
W_u =1.2 (1.35 kip/ft) +1.6(1.27 kip/ft) = 3.65 kip/ft	W_a = 1.35 kip/ft + 1.27 kip/ft[2] = 2.62 kip/ft
$M_u = \dfrac{3.65\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 411\text{ kip-ft}$	$M_a = \dfrac{2.62\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 295\text{ kip-ft}$
$R_u = \dfrac{30.0\text{ ft}}{2}\left(3.65\text{ kip/ft}\right) = 54.8\text{ kips}$	$R_a = \dfrac{30.0\text{ ft}}{2}\left(2.62\text{ kip/ft}\right) = 39.3\text{ kips}$
$I_{x\text{ req'd}} = \dfrac{1.27\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(1.50\text{ in.}\right)} = 532\text{ in.}^4$	$I_{x\text{ req'd}} = \dfrac{1.27\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(1.50\text{ in.}\right)} = 532\text{ in.}^4$
For L_b = 6 ft and C_b = 1.0, select W21×55, which has a design flexural strength of 473 kip-ft, a design shear strength of 234 kips and an I_x of 1140 in.^{4}	For L_b = 6 ft and C_b = 1.0, select W21×55, which has an allowable flexural strength of 314 kip-ft, an allowable shear strength of 156 kips and an I_x of 1140 in.^{4}

Manual Table 3-2

FLOOR MEMBER DESIGN AND SELECTION

Calculate dead load and live load

Dead Load

Slab and Deck	= 57 psf
Beams (est.)	= 8 psf
Misc. (ceiling, mechanical, etc.)	= 10 psf
Total	= 75 psf

Note: The weight of the floor slab and deck was obtained from the manufacturer's literature.

Live Load

		IBC Section
Total (can be reduced for area per IBC 2003)	= 80 psf	1607.9

The floor and deck will be 3 in. of normal weight concrete, $f_c^{'}$ = 4 ksi, on 3 in. 20 gage, galvanized, composite deck, laid in a pattern of 3 or more continuous spans. The total depth of the slab is 6 in. The Steel Deck Institute maximum unshored span for construction with this deck and a 3 span condition is 10 ft-11 in. The general layout for the floor beams is 10 ft on center; therefore, the deck does not need to be shored during construction. At 10 ft on center, this deck has an allowable superimposed live load capacity of 143 psf. In addition, it can be shown that this deck can carry a 2,000 pound load over an area of 2.5 ft by 2.5 ft (per IBC Section 1607.4). The floor diaphragm and the floor loads extend 6 in. past the centerline of grid as shown on Drawing S4.1.

SELECT FLOOR BEAMS (composite and non-composite)

Note: There are two early and important checks in the design of composite beams. First, select a beam that either does not require camber, or establish a target camber and moment of inertia at the start of the design process. A reasonable approximation of the camber is between L/300 minimum and L/180 maximum (or a maximum of 1½ to 2 in.).

Second, check that the beam is strong enough to safely carry the wet concrete and a 20 psf construction live load (per ASCE 37-05), when designed by the SEI/ASCE 7 load combinations and the provisions of Chapter F of the Specification.

SELECT TYPICAL 45 FT INTERIOR COMPOSITE BEAM (10 FT ON CENTER)

Find a target moment of inertia for an un-shored beam

Hold deflection to around 2 in. maximum to facilitate pouring.

W_D = (0.057 kip/ft^2 + 0.008 kip/ft^2)(10.0 ft) = 0.650 kip/ft

$$I_{req} \approx \frac{0.650\text{ kip/ft}\left(45.0\text{ ft}\right)^4}{1290\left(2.00\text{ in.}\right)} = 1030\text{ in.}^4$$

Determine the required strength to carry wet concrete and construction live load

w_{DL} = 0.065 kip/ft^2(10.0 ft) = 0.650 kip/ft
w_{LL} = 0.020 kip/ft^2(10.0 ft) = 0.200 kip/ft

Determine the required flexural strength due to wet concrete only

LRFD	ASD
W_u = 1.4(0.650 kip/ft) = 0.910 kip/ft $M_u = \dfrac{0.910\text{ kip/ft}\left(45.0\text{ ft}\right)^2}{8}$ = 230 kip-ft	W_a = 0.650 kip/ft $M_a = \dfrac{0.650\text{ kip/ft}\left(45.0\text{ ft}\right)^2}{8}$ = 165 kip-ft

Determine the required flexural strength due to wet concrete and construction live load

LRFD	ASD
W_u = 1.2(0.650 kip/ft) + 1.6(0.200 kip/ft) = 1.10 kip/ft $M_u = \dfrac{1.10\text{ kip/ft}\left(45.0\text{ ft}\right)^2}{8}$ = 278 kip-ft **controls**	W_a = 0.650 kip/ft + 0.200 kip/ft = 0.850 kip/ft $M_a = \dfrac{0.850\text{ kip/ft}\left(45.0\text{ ft}\right)^2}{8}$ = 215 kip-ft **controls**

Use Manual Table 3-2 to select a beam with $I_x \geq$ 1030 in.^{4}. Select W21×50, which has I_x = 984 in.^{4}, close to our target value, and has available flexural strengths of 413 kip-ft (LRFD) and 274 kip-ft (ASD). Manual Table 3-2

Check for possible live load reduction due to area

For interior beams K_{LL} = 2 IBC Table

1607.9.1

The beams are at 10.0 ft on center, therefore the area $A_T = (45.0\text{ ft})(10.0\text{ ft}) = 450\text{ ft}^2$.

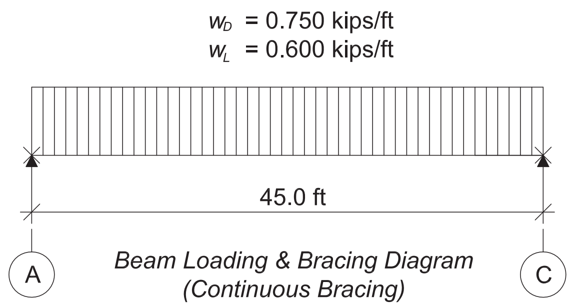
Since $K_{LL}A_T = 2(450\text{ ft}^2) = 900\text{ ft}^2 > 400\text{ ft}^2$, a reduced live load can be used. IBC 1607.9.1

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 80.0\text{ psf}\left(0.25 + \frac{15}{\sqrt{900\text{ ft}^2}}\right) = 60.0\text{ psf} \geq 0.50L_o = 40.0\text{ psf}$$

IBC Eqn 16-24

Therefore, use 60.0 psf.
The beam is continuously braced by the deck.

The beams are at 10 ft on center, therefore the loading diagram is as shown below.



Beam Loading & Bracing Diagram (Continuous Bracing)

Calculate the required flexural strength

LRFD	ASD
W_u = 1.2(0.750 kip/ft) + 1.6(0.600 kip/ft) = 1.86 kip/ft	W_a = 0.750 kip/ft + 0.600 kip/ft = 1.35 kip/ft
$M_u = \dfrac{1.86\text{ kip/ft}(45.0\text{ ft})^2}{8} = 471\text{ kip-ft}$	$M_a = \dfrac{1.35\text{ kip/ft}(45.0\text{ ft})^2}{8} = 342\text{ kip-ft}$

Assume initially a = 1.00 in. Manual Table 3-19
$Y2 = Y_{con} - a / 2$ = 6.00 in. – 1.00 in. / 2 = 5.50 in.

Use Manual Table 3-19 to check W21×50 selected above. Using values of 471 kip-ft (LRFD) or 342 kip-ft (ASD) and a $Y2$ value of 5.50 in.

Manual Table 3-19

LRFD	ASD
Select W21×50 beam, where	Select W21×50 beam, where
PNA = Location 7, $\sum Q_n$ = 184 kips	PNA = Location 7, $\sum Q_n$ = 184 kips
$\phi_b M_n$ = 599 kip-ft > 471 kip-ft **o.k.**	M_p / Ω_n = 399 kip-ft > 342 kip-ft **o.k.**

Determine b_{eff}

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline which shall not exceed: Specification Sec. I3.1a

(1) one-eighth of the span of the beam, center to center of supports

$$\frac{45.0\text{ ft}}{8}\left(2\text{ sides}\right) = 11.3\text{ ft}$$

(2) one-half the distance to the center line of the adjacent beam

$$\frac{10.0\text{ ft}}{2}\left(2\text{ sides}\right) = 10.0\text{ ft} \ \ \textbf{controls}$$

(3) the distance to the edge of the slab

Not applicable

Check a

LRFD	ASD
$a = \dfrac{\Sigma Q_n}{0.85 f'_c b}$ $= \dfrac{184\text{ kips}}{0.85\left(4\text{ ksi}\right)\left(10.0\text{ ft}\right)\left(12\text{ in./ft}\right)}$ $= 0.451$ in. < 1.00 in. **o.k.**	$a = \dfrac{\Sigma Q_n}{0.85 f'_c b}$ $= \dfrac{184\text{ kips}}{0.85\left(4\text{ ksi}\right)\left(10.0\text{ ft}\right)\left(12\text{ in./ft}\right)}$ $= 0.451$ in. < 1.00 in. **o.k.**

Check end shear strength

LRFD	ASD
$R_u = \dfrac{45.0\text{ ft}}{2}\left(1.86\text{ kip/ft}\right) = 41.9\text{ kips}$ $\phi_v V_n = 237$ kips > 41.9 kips **o.k.**	$R_a = \dfrac{45.0\text{ ft}}{2}\left(1.35\text{ kip/ft}\right) = 30.4\text{ kips}$ $V_n / \Omega_v = 158$ kips > 30.4 kips **o.k.**

Manual Table 3-2

Check live load deflection

$\Delta_{LL} = l/360 =$ (45.0 ft)(12 in./ft)/360 = 1.50 in.

IBC Table 1604.3

LRFD	ASD
W21×50: $Y2$ = 5.50 in., PNA = 7 I_{LB} = 1730 in.^{4} $\Delta_{LL} = \dfrac{w_{LL} l^4}{1290 I_{LB}} = \dfrac{0.600\text{ kip/ft}\left(45.0\text{ ft}\right)^4}{1290\left(1730\text{ in.}^4\right)}$ $= 1.10$ in. < 1.50 in. **o.k.**	W21×50: $Y2$ = 5.50 in., PNA = 7 I_{LB} = 1730 in.^{4} $\Delta_{LL} = \dfrac{w_{LL} l^4}{1290 I_{LB}} = \dfrac{0.600\text{ kip/ft}\left(45.0\text{ ft}\right)^4}{1290\left(1730\text{ in.}^4\right)}$ $= 1.10$ in. < 1.50 in. **o.k.**

Manual Table 3-20

Based on AISC Design Guide 3, 2nd Edition, limit the live load deflection, using 50 percent of the (unreduced) design live load, to L / 360 with a maximum absolute value of 1.0 in. across the bay.

$$\Delta_{LL} = \frac{0.400\text{ kip/ft}\left(45.0\text{ ft}\right)^4}{1290\left(1730\text{ in.}^4\right)}$$

$= 0.735$ in. < 1.00 in. **o.k.**

1.00 in. – 0.735 in. = 0.265 in.

Note: Limit the supporting girders to 0.265 in. deflection under the same load case at the connection point of the beam.

Determine the required shear stud connectors

Using perpendicular deck with one ¾-in. diameter stud per rib in normal weight, 4 ksi concrete, in weak position; Q_n = 17.2 kips/stud

Manual Table 3-21

LRFD	ASD
$\dfrac{\sum Q_n}{Q_n} = \dfrac{184 \text{ kips}}{17.2 \text{ kips/stud}} = 10.7 \text{ studs / side}$	$\dfrac{\sum Q_n}{Q_n} = \dfrac{184 \text{ kips}}{17.2 \text{ kips/stud}} = 10.7 \text{ studs / side}$

Therefore use 22 studs.

Based on AISC Design Guide 3, 2nd Ed., limit the wet concrete deflection in a bay to L / 360, not to exceed 1.0 in.

Camber the beam for 80% of the calculated wet deflection

$$\Delta_{DL(wet\ conc)} = \frac{0.650 \text{ kip/ft}\left(45.0 \text{ ft}\right)^4}{1290\left(984 \text{ in.}^4\right)} = 2.10 \text{ in.}$$

Camber = 0.800(2.10 in.) = 1.68 in.

Round the calculated value down to the nearest ¼ in; therefore, specify 1.5 in. of camber.

2.10 in. – 1.50 in. = 0.600 in.

1.00 in. – 0.600 in. = 0.400 in.

Note: Limit the supporting girders to 0.400 in. deflection under the same load combination at the connection point of the beam.

SELECT TYPICAL 30 FT INTERIOR COMPOSITE (OR NON-COMPOSITE) BEAM (10 FT ON CENTER)

Find a target moment of inertia for an unshored beam

Hold deflection to around 1.50 in. maximum to facilitate concrete placement.

$$I_{req} \approx \frac{0.650 \text{ kip/ft}\left(30.0 \text{ ft}\right)^4}{1290\left(1.50 \text{ in.}\right)} = 272 \text{ in.}^4$$

Determine the required strength to carry wet concrete and construction live load

w_{DL} = 0.065 kip/ft^2(10.0 ft) = 0.650 kip/ft

w_{LL} = 0.020 kip/ft^2(10.0 ft) = 0.200 kip/ft

Determine the required flexural strength due to wet concrete only

LRFD	ASD
W_u = 1.4(0.650 kip/ft) = 0.910 kip/ft $M_u = \dfrac{0.910\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 102\text{ kip-ft}$	W_a = 0.650 kip/ft $M_a = \dfrac{0.650\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 73.1\text{ kip-ft}$

Determine the required flexural strength due to wet concrete and construction live load

LRFD	ASD
W_u = 1.2(0.650 kip/ft) + 1.6(0.200 kip/ft) = 1.10 kip/ft $M_u = \dfrac{1.10\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 124\text{ kip-ft}$ **controls**	W_a = 0.650 kip/ft + 0.200 kip/ft = 0.850 kip/ft $M_a = \dfrac{0.850\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 95.6\text{ kip-ft}$ **controls**

Use Manual Table 3-2 to find a beam with an $I_x \geq 272$ in.^{4}. Select W16×26, which has an $I_x = 301$ in.^{4} which exceeds our target value, and has available flexural strengths of 166 kip-ft (LRFD) and 110 kip-ft (ASD). Manual Table 3-2

Check for possible live load reduction due to area

IBC Table 1607.9.1

For interior beams K_{LL} = 2

The beams are at 10 ft on center, therefore the area A_T = 30.0 ft × 10.0 ft = 300 ft^2. IBC 1607.9.1

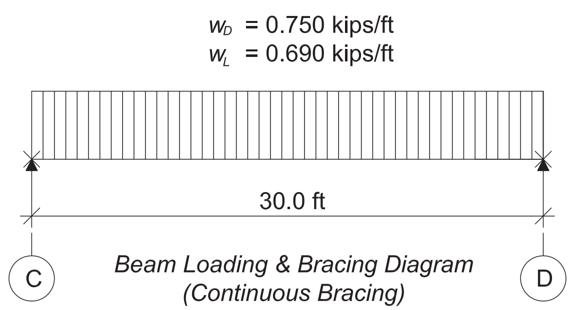
Since $K_{LL}A_T$ = 2(300 ft^2) = 600 ft^2 > 400 ft^2, a reduced live load can be used. The reduced live load is

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}A_T}}\right) = 80.0\text{ psf}\left(0.25+\frac{15}{\sqrt{600\text{ ft}^2}}\right) = 69.0\text{ psf} \geq 0.50\,L_o = 40.0\text{ psf}$$

IBC Eqn 16-24

Therefore, use 69.0 psf

The beams are at 10.0 ft on center, therefore the loading diagram is as shown below.



Beam Loading & Bracing Diagram
(Continuous Bracing)

Calculate the required strength

LRFD	ASD
W_u = 1.2(0.750 kip/ft) + 1.6 (0.690 kip/ft) = 2.00 kip/ft	W_a = 0.750 kip/ft + 0.690 kip/ft = 1.44 kip/ft

LRFD	ASD
$M_u = \dfrac{2.00\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 225\text{ kip-ft}$	$M_a = \dfrac{1.44\text{ kip/ft}\left(30.0\text{ ft}\right)^2}{8} = 162\text{ kip-ft}$

Assume initially $a = 1.00$ and $Y2 = 6.00\text{ in.} - \dfrac{1.00\text{ in.}}{2} = 5.50\text{ in.}$

Use Manual Table 3-19 to check the W16×26 selected above. Using required strengths of 225 kip-ft (LRFD) or 162 kip-ft (ASD) and a *Y2* value of 5.5 in

Manual Table 3-19

LRFD	ASD
Select W16×26 beam, where PNA = Location 7 and $\sum Q_n = 96.0$ kips $\phi_b M_n = 248\text{ kip-ft} > 225\text{ kip-ft}$ **o.k.**	Select W16×26 beam, where PNA = Location 7 and $\sum Q_n = 96.0$ kips $M_p / \Omega_n = 165\text{ kip-ft} > 162\text{ kip-ft}$ **o.k.**

Manual Table 3-19

Determine b_{eff}

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, which shall not exceed:

Specification Sec. I3.1a

(1) one-eighth of the span of the beam, center to center of supports

$$\frac{30.0\text{ ft}}{8}\left(2\text{ sides}\right) = 7.50\text{ ft} \quad \textbf{controls}$$

(2) one-half the distance to the center line of the adjacent beam

$$\frac{10.0\text{ ft}}{2}\left(2\text{ sides}\right) = 10.0\text{ ft}$$

(3) the distance to the edge of the slab

Not applicable

Check a

LRFD	ASD
$a = \dfrac{\sum Q_n}{0.85 f'_c b}$ $= \dfrac{96.0\text{ kips}}{0.85\left(4\text{ ksi}\right)\left(7.50\text{ ft}\right)\left(12\text{ in./ft}\right)}$ $= 0.314\text{ in.} < 1.0\text{ in.}$ **o.k.**	$a = \dfrac{\sum Q_n}{0.85 f'_c b}$ $= \dfrac{96.0\text{ kips}}{0.85\left(4\text{ ksi}\right)\left(7.50\text{ ft}\right)\left(12\text{ in./ft}\right)}$ $= 0.314\text{ in.} < 1.0\text{ in.}$ **o.k.**

Check end shear strength

LRFD	ASD
$R_u = \dfrac{30.0\text{ ft}}{2}\left(2.00\text{ kip/ft}\right) = 30.0\text{ kips}$ $\phi_v V_n = 106\text{ kips} > 30.0\text{ kips}$ **o.k.**	$R_a = \dfrac{30.0\text{ ft}}{2}\left(1.44\text{ kip/ft}\right) = 21.6\text{ kips}$ $V_n / \Omega_v = 70.5\text{ kips} > 21.6\text{ kips}$ **o.k.**

Manual Table 3-2

Check live load deflection

$\Delta_{LL} = l/360 = (30.0\text{ ft})(12\text{ in./ft})/360 = 1.00\text{ in.}$

IBC Table 1604.3

LRFD	ASD
W16×26 $Y2$ = 5.50 in. PNA = 7 I_{LB} = 575 in.^{4} $\Delta_{LL} = \dfrac{w_{LL}l^4}{1290I_{LB}} = \dfrac{0.690\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(575\text{ in.}^4\right)}$ = 0.753 in. < 1.00 in. **o.k.**	W16×26 $Y2$ = 5.50 in. PNA = 7 I_{LB} = 575 in.^{4} $\Delta_{LL} = \dfrac{w_{LL}l^4}{1290I_{LB}} = \dfrac{0.690\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(575\text{ in.}^4\right)}$ = 0.753 in. < 1.00 in. **o.k.**

Manual Table 3-20

Based on AISC Design Guide 3, 2nd Edition, limit the live load deflection, using 50 percent of the (unreduced) design live load, to L / 360 with a maximum absolute value of 1.0 in. across the bay.

$$\Delta_{LL} = \frac{0.400\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(575\text{ in.}^4\right)}$$

= 0.437 in. < 1.00 in. **o.k.**

1.00 in. – 0.437 in. = 0.563 in.

Note: Limit the supporting girders to 0.563 in. deflection under the same load combination at the connection point of the beam.

Determine the required shear stud connectors

Using perpendicular deck with one ¾-in. diameter stud per rib in normal weight, 4 ksi concrete, in weak position; Q_n = 17.2 kips/stud

Manual Table 3-21

LRFD	ASD
$\dfrac{\sum Q_n}{Q_n} = \dfrac{96.0\text{ kips}}{17.2\text{ kips/stud}} = 5.58\text{ studs / side}$ Use 12 studs	$\dfrac{\sum Q_n}{Q_n} = \dfrac{96.0\text{ kips}}{17.2\text{ kips/stud}} = 5.58\text{ studs / side}$ Use 12 studs

Note: There is a maximum spacing limit of 8(6 in.) = 4'- 0" not to exceed 3'-0" between studs.

Specification Sec. I3.2d(6)

Therefore use 12 studs, uniformly spaced at no more than 3'-0" on center.

Note: Although the studs may be placed up to 3'-0" o.c. the steel deck must still be anchored to the supporting member at a spacing not to exceed 18 in.

Specification Section I3.2c

Based on AISC Design Guide 3, 2nd Ed., limit the wet concrete deflection in a bay to L / 360, not to exceed 1.0 in.

Camber the beam for 80% of the calculated wet deflection

$$\Delta_{DL(wet\ conc)} = \frac{0.650\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(301\text{ in.}^4\right)} = 1.36\text{ in.}$$

Camber = 0.800(1.36 in.) = 1.09 in.

Round the calculated value down to the nearest ¼ in. Therefore, specify 1.0 in. of camber.

1.36 in. – 1.00 in. = 0.360 in.

1.00 in. – 0.360 in. = 0.640 in.

Note: Limit the supporting girders to 0.640 in. deflection under the same load combination at the connection point of the beam.

This beam could also be designed as a non-composite beam. Use Manual Table 3-2 with previous moments and shears:

LRFD	ASD
Select W18×35	Select W18×35
$\phi_b M_p$ = 249 kip-ft > 225 kip-ft **o.k.**	M_p / Ω_b = 166 kip-ft > 162 kip-ft **o.k.**
$\phi_v V_n$ = 159 > 30.0 kips **o.k.**	V_n / Ω_v = 106 kips > 21.6 kips **o.k.**

Manual Table 3-2

Check beam deflections

Check W18×35 with an I_x = 510 in.^{4}

Manual Table 3-2

$$\Delta_{LL} = \frac{0.690\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(510\text{ in.}\right)^4} = 0.850\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}$$

Based on AISC Design Guide 3, 2nd Edition, limit the live load deflection, using 50 percent of the (unreduced) design live load, to L / 360 with a maximum absolute value of 1.0 in. across the bay.

$$\Delta_{LL} = \frac{0.400\text{ kip/ft}\left(30.0\text{ ft}\right)^4}{1290\left(510\text{ in.}^4\right)}$$
$$= 0.492\text{ in.} < 1.00\text{ in.} \quad \textbf{o.k.}$$

1.00 in. – 0.492 in. = 0.508 in.

Note: Limit the supporting girders to 0.508 in. deflection under the same load combination at the connection point of the beam.

Note: Because this beam is stronger than the W16×26 composite beam, no wet concrete strength checks are required in this example.

Based on AISC Design Guide 3, 2nd Ed., limit the wet concrete deflection in a bay to L / 360, not to exceed 1.0 in.

Camber the beam for 80% of the calculated wet deflection

$$\Delta_{DL(wet\ conc)} = \frac{0.650 \text{ kip/ft}\left(30.0 \text{ ft}\right)^4}{1290\left(510 \text{ in.}^4\right)} = 0.800 \text{ in.} < 1.50 \text{ in.} \quad \textbf{o.k.}$$

Camber = 0.800(0.800 in.) = 0.640 in. < 0.750 in.

A good break point to eliminate camber is ¾ in.; therefore, do not specify a camber for this beam.

1.00 in. – 0.800 in. = 0.200 in.

Note: Limit the supporting girders to 0.200 in. deflection under the same load case at the connection point of the beam.

Therefore, selecting a W18×35 will eliminate both shear studs and cambering. The cost of the extra steel weight may be offset by the elimination of studs and cambering. Local labor and material costs should be checked to make this determination.

SELECT TYPICAL EDGE BEAM

The influence area ($K_{LL}A_T$) for these beams is less than 400 ft^2, therefore no live load reduction can be taken.

These beams carry 5.50 ft of dead load and live load as well as a wall load.

The floor dead load is:

w = 5.50 ft(0.075 kips/ft^2) = 0.413 kip/ft

Use 65 psf for the initial dead load.

$w_{D(initial)}$ = 5.50 ft(0.065 kips/ft^2) = 0.358 kips/ft

Use 10 psf for the superimposed dead load.

$w_{D(super)}$ = 5.50 ft(0.010 kips/ft^2) = 0.055 kips/ft

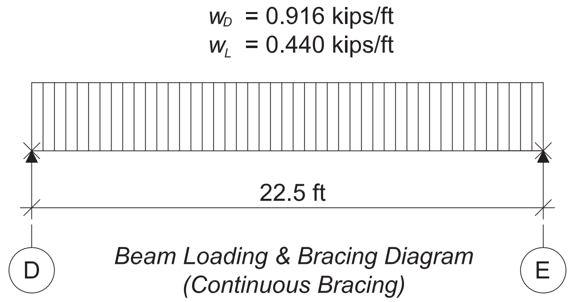
The dead load of the wall system at the floor is:

$$\begin{aligned} w &= 7.50 \text{ ft}\left(0.055 \text{ kip / ft}^2\right) + 6.00 \text{ ft}\left(0.015 \text{ kip / ft}^2\right) \\ &= 0.413 \text{ kip/ft} + 0.090 \text{ kip/ft} \\ &= 0.503 \text{ kip/ft} \end{aligned}$$

The total dead load is w_{DL} = 0.916 kip/ft

The live load is $w_{LL} = 5.5 \text{ ft}\left(0.080 \text{ kip / ft}^2\right) = 0.440 \text{ kip / ft}$

The loading diagram is as follows.



Beam Loading & Bracing Diagram (Continuous Bracing)

Calculate the required strengths

LRFD	ASD
$W_u = 1.2(0.916\text{ kip/ft}) + 1.6\ (0.440\text{ kip/ft}) = 1.80\text{ kip/ft}$	$W_a = 0.916\text{ kip/ft} + 0.440\text{ kip/ft} = 1.36\text{ kip/ft}$
$M_u = \dfrac{1.80\text{ kip/ft}\left(22.5\text{ ft}\right)^2}{8} = 114\text{ kip-ft}$	$M_a = \dfrac{1.36\text{ kip/ft}\left(22.5\text{ ft}\right)^2}{8} = 86.1\text{ kip-ft}$
$R_u = \dfrac{22.5\text{ ft}}{2}\left(1.80\text{ kip/ft}\right) = 20.3\text{ kips}$	$R_a = \dfrac{22.5\text{ ft}}{2}\left(1.36\text{ kip/ft}\right) = 15.3\text{ kips}$

Because these beams are less than 25 ft long, they will be most efficient as non-composite beams. The beams at the edges of the building carry a brick spandrel panel. For these beams, the cladding weight exceeds 25 percent of the total dead load on the beam. Therefore, per AISC Design Guide 3, Second Edition, limit the vertical deflection due to cladding and initial dead load to L/600 or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to L/600 or 0.3 in. max to accommodate the brick and L/360 or 0.25 in. max to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to L/600 or 0.25 in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. Note that it is typically not recommended to camber beams supporting spandrel panels.

Calculate minimum I_x to limit the superimposed dead and live load deflection to ¼ in.

$$I_{req} = \frac{0.495\text{ kip/ft}\left(22.5\text{ ft}\right)^4}{1290\left(0.25\text{ in.}\right)} = 393\text{ in.}^4 \quad \textbf{controls}$$

Calculate minimum I_x to limit the cladding and initial dead load deflection to ⅜ in.

$$I_{req} = \frac{0.861\text{ kip/ft}\left(22.5\text{ ft}\right)^4}{1290\left(0.375\text{ in.}\right)} = 456\text{ in.}^4$$

Select Beam from Manual Table 3-2

LRFD	ASD	
Select W18×35 with I_x = 510 in.^{4}	Select W18×35 with I_x = 510 in.^{4}	Manual Table 3-2
$\phi_b M_p$ = 249 kip-ft > 114 kip-ft **o.k.**	M_p / Ω_b = 166 kip-ft > 86.1 kip-ft **o.k.**	
$\phi_v V_n$ = 159 > 20.3 kips **o.k.**	V_n / Ω_v = 106 kips > 15.3 kips **o.k.**	

SELECT TYPICAL EAST - WEST SIDE GIRDER

The beams along the sides of the building carry the spandrel panel and glass, and dead load and live load from the intermediate floor beams. For these beams, the cladding weight exceeds 25 percent of the total dead load on the beam. Therefore, per AISC Design Guide 3, Second Edition, limit the vertical deflection due to cladding and initial dead load to L/600 or ⅜ in. maximum. In addition, because these beams are supporting brick above and there is continuous glass below, limit the superimposed dead and live load deflection to L/600 or 0.3 in. max to accommodate the brick and L/360 or 0.25 in. max to accommodate the glass. Therefore, combining the two limitations, limit the superimposed dead and live load deflection to L/600 or 0.25 in. The superimposed dead load includes all of the dead load that is applied after the cladding has been installed. These beams will be part of the moment frames on the North and South sides of the building and therefore will be designed as fixed at both ends.

Establish loads

The dead load reaction from the floor beams is:

P_D = 0.750 kip/ft(45.0 ft / 2) = 16.9 kips

$P_{D(initial)}$ = 0.650 kip/ft(45.0 ft / 2) = 14.6 kips

$P_{D(super)}$ = 0.100 kip/ft(45.0 ft / 2) = 2.25 kips

The uniform dead load along the beam is:

w_D = 0.500 ft(0.075 kip/ft^2) + 0.503 kip/ft = 0.541 kip/ft

$w_{D(initial)}$ = 0.500 ft(0.065 kip/ft^2) = 0.033 kip/ft

$w_{D(super)}$ = 0.500 ft(0.010 kip/ft^2) = 0.005 kip/ft

Select typical 30 foot composite(or non-composite) girders

Check for possible live load reduction

For edge beams with cantilevered slabs, K_{LL} = 1, per ASCE 7, Table 4-2. However, it is also permissible to calculate the value of K_{LL} based upon influence area. Because the cantilever dimension is small, K_{LL} will be closer to 2 than 1. The calculated value of K_{LL} based upon the influence area is

$$K_{LL} = \frac{(45.5\text{ ft})(30.0\text{ ft})}{\left(\dfrac{45.0\text{ ft}}{2}+0.500\text{ ft}\right)(30.0\text{ ft})} = 1.98$$

ASCE 7
Sec. C4.8 &
Fig. C4

The area A_T = (30.0 ft)(22.5 ft + 0.500 ft) = 690 ft^2

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}A_T}}\right) = 80.0\text{ psf}\left(0.25+\frac{15}{\sqrt{(1.98)(690\text{ ft}^2)}}\right) = 52.5\text{ psf} \geq 0.50\,L_o = 40.0\text{ psf}$$

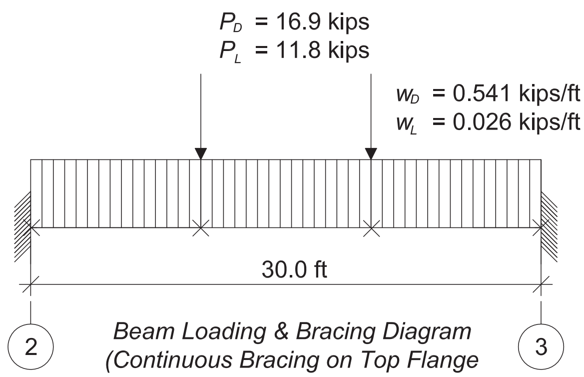
IBC Eqn
16-24

Therefore, use 52.5 psf.

The live load from the floor beams is P_{LL} = 0.525 kip/ft(45.0 ft / 2) = 11.8 kips

The uniform live load along the beam is w_{LL} = 0.500 ft(0.0525 kip/ft^2) = 0.026 kip/ft

The loading diagram is shown below.



Beam Loading & Bracing Diagram
(Continuous Bracing on Top Flange
At Ends and ⅓ Points on Bottom Flange)

A summary of the moments, reactions and required moments of inertia, determined from a structural analysis of a fixed-end beam, is as follows:

Calculate the required strengths and select the beams for the floor side beams

LRFD	ASD
Typical side beam R_u = 49.5 kips M_u at ends =313 kip-ft M_u at ctr. =156 kip-ft	Typical side beam R_a = 37.2 kips M_a at ends = 234 kip-ft M_a at ctr. = 117 kip-ft

The maximum moment occurs at the support with compression in the bottom flange. The bottom laterally braced at 10 ft o.c. by the intermediate beams.

Note: During concrete placement, because the deck is parallel to the beam, the beam will not have continuous lateral support. It will be braced at 10 ft o.c. by the intermediate beams. By inspection, this condition will not control because the maximum moment under full loading causes compression in the bottom flange, which is braced at 10 ft o.c.

LRFD	ASD	
Calculate C_b = for compression in the bottom flange braced at 10′ - 0″ o.c.	Calculate C_b = for compression in the bottom flange braced at 10′ - 0″ o.c.	
C_b = 2.21 (from computer output)	C_b = 2.22 (from computer output)	
Select W21×44	Select W21×44	
With continuous bracing	With continuous bracing	
ϕM_n = 358 kip-ft > 156 kip-ft **o.k.**	M_n/Ω = 238 kip-ft > 117 kip-ft **o.k.**	Manual Table 3-2
For L_b = 10 ft and C_b = 2.21	For L_b = 10 ft and C_b = 2.22	
$\phi M_n = (265\text{ kip-ft})(2.21)$ $= 586\text{ kip-ft} \leq 358\text{ kip-ft}$	$M_n/\Omega = (176\text{ kip-ft})(2.22)$ $= 391\text{ kip-ft} \leq 238\text{ kip-ft}$	Manual Table 3-10

LRFD	ASD
358 kip-ft > 313 kip-ft **o.k.**	238 kip-ft > 234 kip-ft **o.k.**
A W21×44 has a design shear strength of 217 kips and an I_x of 843 in.^{4}	A W21×44 has an allowable shear strength of 145 kips and an I_x of 843 in.^{4}
Check deflection due to cladding and initial dead load.	Check deflection due to cladding and initial dead load.
$\Delta = 0.295 \text{ in.} < 0.375 \text{ in.}$ **o.k.**	$\Delta = 0.295 \text{ in.} < 0.375 \text{ in.}$ **o.k.**
Check deflection due to superimposed dead and live loads.	Check deflection due to superimposed dead and live loads.
$\Delta = 0.212 \text{ in.} < 0.250 \text{ in.}$ **o.k.**	$\Delta = 0.212 \text{ in.} < 0.250 \text{ in.}$ **o.k.**

Manual Table 3-2

Note that both of the deflection criteria stated previously for the girder and for the locations on the girder where the floor beams are supported have also been met.

Also noted previously, it is not typically recommended to camber beams supporting spandrel panels. The W21×44 is adequate for strength and deflection, but may be increased in size to help with moment frame strength or drift control.

SELECT TYPICAL EAST - WEST INTERIOR GIRDER

Establish loads

The dead load reaction from the floor beams is

P_{DL} = 0.750 kip/ft(45.0 ft + 30.0 ft)/2 = 28.1 kips

Check for live load reduction due to area

For interior beams, K_{LL} = 2

IBC Table 1607.9.1

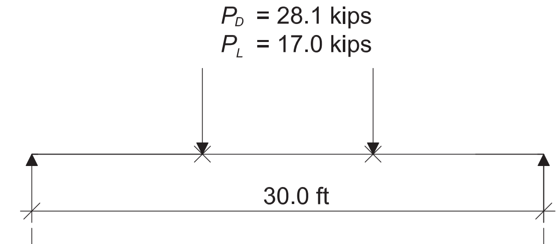
The area $A_T = (30.0 \text{ ft})(37.5 \text{ ft}) = 1130 \text{ ft}^2$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 80.0 \text{ psf}\left(0.25 + \frac{15}{\sqrt{(2)(1130 \text{ ft}^2)}}\right) = 45.2 \text{ psf} \geq 0.50\,L_o = 40.0 \text{ psf}$$

IBC Eqn 16-24

Therefore, use 45.2 psf

The live load from the floor beams is P_{LL} = 0.0452 kip/ft^2(10.0 ft)(37.5 ft) = 17.0 kips



Beam Loading & Bracing Diagram
(Continuous Bracing)

Note: The dead load for this beam is included in the assumed overall dead load.

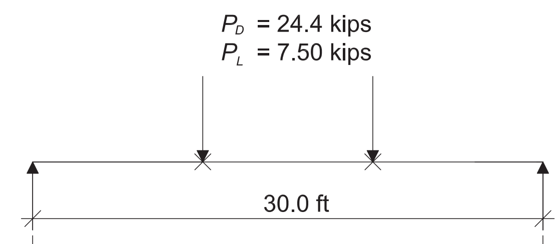
A summary of the simple moments and reactions is shown below:

Calculate the required strengths and select the size for the interior beams

LRFD	ASD
Typical interior beam R_u = 60.9 kips M_u = 609 kip-ft	Typical interior beam R_a = 45.1 kips M_a = 451 kip-ft

Check for beam requirements when carrying wet concrete

Note: During concrete placement, because the deck is parallel to the beam, the beam will not have continuous lateral support. It will be braced at 10 ft on center by the intermediate beams. Also, during concrete placement, a construction live load of 20 psf will be present. This load pattern and a summary of the moments, reactions, and deflection requirements is shown below. Limit wet concrete deflection to 1.5 in.



Beam Loading & Bracing Diagram
(Braced at ⅓ Points)

LRFD	ASD
Typical Interior Beam with wet concrete only R_u = 34.2 kips M_u = 342 kip-ft	Typical interior Beam with wet concrete only R_a = 24.4 kips M_a = 244 kip-ft

Target $I_x \geq$ 935 in.^{4}

LRFD	ASD
Typical Interior Beam with wet concrete and construction load R_u = 41.3 kips M_u (midspan) = 413 kip-ft	Typical Interior Beam with wet concrete and construction load R_a = 31.9 kips M_a (midspan) = 319 kip-ft

LRFD	ASD	
Select beam with an unbraced length of 10.0 ft and a conservative $C_b = 1.0$ Select W21×68, which has a design flexural strength of 532 kip-ft, a design shear strength of 273 kips, and an I_x of 1480 in.^{4} $\phi_b M_p = 532$ kip-ft > 413 kip-ft **o.k.**	Select beam with an unbraced length of 10.0 ft and a conservative $C_b = 1.0$ Select W21×68, which has an allowable flexural strength of 354 kip-ft, an allowable shear strength of 182 kips, and an I_x of 1480 in.^{4} $M_p / \Omega_b = 354$ kip-ft > 319 kip-ft **o.k.**	Manual Table 3-10

Check W21×68 *as a composite beam*

From previous calculations:

LRFD	ASD
Typical Interior Beam R_u = 60.9 kips M_u (midspan) = 609 kip-ft	Typical Interior beam R_a = 45.1 kips M_a (midspan) = 451 kip-ft

$Y2$ (from previous calculations, assuming an initial a = 1.00 in.) = 5.50 in.

Use Manual Table 3-19, check W21×68, using required flexural strengths of 609 kip-ft (LRFD) and 451 kip-ft (ASD) and $Y2$ value of 5.5 in.

LRFD	ASD
Select a W21×68 Where PNA = Location 7, $\sum Q_n$ = 251 kips $\phi_b M_n$ = 847 kip-ft > 609 kip-ft **o.k.**	Select a W21×68 Where PNA = Location 7, $\sum Q_n$ = 251 kips M_n / Ω_b = 564 kip-ft > 461 kip-ft **o.k.**

Manual Table 3-19

Based on AISC Design Guide 3, 2nd Ed., limit the wet concrete deflection in a bay to L / 360, not to exceed 1.0 in.

Camber the beam for 80% of the calculated wet deflection

$$\Delta_{DL(wet\ conc)} = \frac{24.4\text{ kips}\left(30.0\text{ ft}\right)^3\left(12\text{ in./ft}\right)^3}{28\left(29{,}000\text{ ksi}\right)\left(1480\text{ in.}^4\right)} = 0.947\text{ in.}$$

Camber = 0.800(0.947 in.) = 0.758 in.

Round the calculated value down to the nearest ¼ in. Therefore, specify ¾ in. of camber.

0.947 in. – 0.750 in. = 0.197 in. < 0.200 in. therefore, the total deflection limit of 1.0 in. for the bay has been met.

Determine b_{eff}

The effective width of the concrete slab is the sum of the effective widths for each side of the beam centerline, which shall not exceed:

Specification Sec. I3.1a

(1) one-eighth of the span of the beam, center to center of supports

$$\frac{30.0\text{ ft}}{8}\left(2\text{ sides}\right) = 7.50\text{ ft} \quad \textbf{controls}$$

(2) one-half the distance to the center line of the adjacent beam

$$\left(\frac{45.0\text{ ft}}{2} + \frac{30.0\text{ ft}}{2}\right) = 37.5\text{ ft}$$

(3) the distance to the edge of the slab

Not applicable.

LRFD	ASD
Check a $a = \dfrac{\sum Q_n}{0.85 f'_c b}$ $= \dfrac{251 \text{ kips}}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$ $= 0.820 \text{ in.} < 1.0 \text{ in.}$ **o.k.**	*Check a* $a = \dfrac{\sum Q_n}{0.85 f'_c b}$ $= \dfrac{251 \text{ kips}}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$ $= 0.820 \text{ in.} < 1.0 \text{ in.}$ **o.k.**

Check end shear strength

Manual Table 3-2

LRFD	ASD
R_u = 60.9 kips $\phi_v V_n$ = 273 kips > 60.9 kips **o.k.**	R_a = 45.1 kips V_n / Ω_v = 182 kips > 45.1 kips **o.k.**

Check live load deflection

$\Delta_{LL} = l/360$ = (30.0 ft)(12 in./ft)/360 = 1.00 in.

IBC Table 1604.3

Manual Table 3-20

LRFD	ASD
W21×68: $Y2$ = 5.50 in., PNA = Location 7 I_{LB} = 2520 in.^{4} $\Delta_{LL} = \dfrac{Pl^3}{28EI_{LB}}$ $= \dfrac{17.0 \text{ kips}(30.0 \text{ ft})^3(12 \text{ in./ft})^3}{28(29{,}000 \text{ ksi})(2520 \text{ in.}^4)}$ $= 0.388 \text{ in.} < 1.0 \text{ in.}$ **o.k.**	W21×68: $Y2$ = 5.50 in., PNA = Location 7 I_{LB} = 2520 in.^{4} $\Delta_{LL} = \dfrac{Pl^3}{28EI_{LB}}$ $= \dfrac{17.0 \text{ kips}(30.0 \text{ ft})^3(12 \text{ in./ft})^3}{28(29{,}000 \text{ ksi})(2520 \text{ in.}^4)}$ $= 0.388 \text{ in.} < 1.0 \text{ in.}$ **o.k.**

Based on AISC Design Guide 3, 2nd Edition, limit the live load deflection, using 50 percent of the (unreduced) design live load, to L / 360 with a maximum absolute value of 1.0 in. across the bay.

$$\Delta_{LL} = \frac{15.0 \text{ kips}(30.0 \text{ ft})^3(12 \text{ in./ft})^3}{28(29{,}000 \text{ ksi})(2520 \text{ in.}^4)}$$
$$= 0.342 \text{ in.} < 1.00 \text{ in.} \quad \text{o.k.}$$

Check the deflection at the location where the floor beams are supported

$$\Delta_{LL} = \frac{15.0 \text{ kips}(120 \text{ in.})}{6(29{,}000 \text{ ksi})(2520 \text{ in.}^4)}\left(3(360 \text{ in.})(120 \text{ in.}) - 4(120 \text{ in.})^2\right)$$
$$= 0.296 \text{ in.} > 0.265 \text{ in.} \quad \text{o.k.}$$

Therefore, the total deflection in the bay is 0.296 in. + 0.735 in. = 1.03 in. which is acceptably close to the limit of 1.0 in.

Determine the required shear stud connectors

Using parallel deck with, w_r / n_r > 1.5, one ¾ in. diameter stud in normal weight, 4 ksi concrete; Q_n = 21.5 kips/stud

Manual Table 3-21

LRFD	ASD
$\dfrac{\sum Q_n}{Q_n} = \dfrac{251 \text{ kips}}{21.5 \text{ kips/stud}}$ = 11.7 studs/side Therefore, use a minimum 24 studs for horizontal shear. The maximum stud spacing is 3'-0". Since the load is concentrated at ⅓ points, the studs are to be arranged as follows: Use 12 studs between supports and supported beams at ⅓ points. Between supported beams (middle ⅓ of span), use 4 studs to satisfy minimum spacing requirements. Thus, 28 studs are required in a 12:4:12 arrangement.	$\dfrac{\sum Q_n}{Q_n} = \dfrac{251 \text{ kips}}{21.5 \text{ kips/stud}}$ = 11.7 studs/side Therefore, use a minimum 24 studs for horizontal shear. The maximum stud spacing is 3'-0". Since the load is concentrated at ⅓ points, the studs are to be arranged as follows: Use 12 studs between supports and supported beams at ⅓ points. Between supported beams (middle ⅓ of span), use 4 studs to satisfy minimum spacing requirements. Thus, 28 studs are required in a 12:4:12 arrangement.

Section I3.2d(6)

Note: Although the studs may be placed up to 3'-0" o.c. the steel deck must still be anchored to be the supporting member at a spacing not to exceed 18 in.

Section I3.2c

Note: This W21×68 beam, with full lateral support, is very close to having sufficient available strength to support the imposed loads without composite action. A larger non-composite beam might be a better solution.

COLUMNS MEMBER DESIGN SELECTION FOR GRAVITY COLUMNS

Estimate column loads

Roof (from previous calculations)

Dead Load	20 psf
Live (Snow)	25 psf
Total	45 psf

Snow drift loads at the perimeter of the roof and at the mechanical screen wall from previous calculations

Reaction to column (side parapet):

w = (3.73 kips / 6.00 ft) - (0.025 ksf)(23.0 ft) = 0.0467 kips/ft

Reaction to column (end parapet):

w = (16.1 kips / 37.5 ft) - (0.025 ksf)(15.5 ft) = 0.0418 kips/ft

Reaction to column (screen wall along lines c & d):

w = (4.02 kips / 6.00 ft) - (0.025 ksf)(22.5 ft) = 0.108 kips/ft

Mechanical equipment and screen wall (average):

w = 40 psf

Col	**Loading Width ft**	**Loading Length ft**	**Area ft²**	**DL kips/ft²**	**P_D kips**	**LL kips/ft²**	**P_L kips**
2A, 2F, 3A, 3F, 4A, 4F 5A, 5F, 6A, 6F, 7A, 7F	23.0	30.0	690	0.020	13.8	0.025	17.3
snow drifting side		30.0				0.047 klf	1.41
exterior wall		30.0		0.413 klf	12.4		
					26.2		**18.7**
1B, 1E, 8B, 8E	3.50	22.5	78.8	0.020	1.58	0.025	1.97
snow drifting end		22.5				0.042 klf	0.945
exterior wall		22.5		0.413 klf	9.29		
					10.9		**2.92**
1A, 1F, 8A, 8F	23.0	15.5	357 $-\frac{(78.8\text{ ft}^2)}{2}$ = 318	0.020	6.36	0.025	7.95
snow drifting end		11.8				0.042 klf	0.496
snow drifting side		15.5				0.047 klf	0.729
exterior wall		27.3		0.413 klf	11.3		
					17.7		**9.18**
1C, 1D, 8C, 8D	37.5	15.5	581 $-\frac{(78.8\text{ ft}^2)}{2}$ = 542	0.020	10.8	0.025	13.6
snow-drifting end		26.3				0.042 klf	1.10
exterior wall		26.3		0.413 klf	10.9		
					21.7		**14.7**
2C, 2D, 7C, 7D	37.5	30.0	1,125	0.020	**22.5**	0.025	**28.1**
3C, 3D, 4C, 4D 5C, 5D, 6C, 6D	22.5	30.0	675	0.020	13.5	0.025	16.9
snow-drifting		30.0				0.108 klf	3.24
mechanical area	15.0	30.0	450	0.060	27.0	0.040	18.0
					40.5		**38.1**

Floor Loads (from previous calculations)

Dead load	75 psf
Live load	<u>80 psf</u>
Total load	155 psf

Calculate reduction in live loads, analyzed at the base of 3 floors

Note: The 6 in. cantilever of the floor slab has been ignored for the calculation of K_{LL} for columns in this building because it has a negligible effect.

Columns: 2A, 2F, 3A, 3F, 4A, 4F, 5A, 5F, 6A, 6F, 7A, 7F
Exterior column without cantilever slabs

$K_{LL} = 4$ $\quad L_o = 80.0$ psf $\quad n = 3$ — IBC Sect 1607.9

$A_T = (23.0 \text{ ft})(30.0 \text{ ft}) = 690 \text{ ft}^2$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} n A_T}}\right)$$ IBC Eqn 16-21

$$= 80.0 \text{ psf}\left(0.25 + \frac{15}{\sqrt{(4)(3)(690 \text{ ft}^2)}}\right)$$

$$= 33.2 \text{ psf} \geq 0.4L_o = 32.0 \text{ psf}$$

use $L = 33.2$ psf

Columns: 1B, 1E, 8B, 8E
Exterior column without cantilever slabs

$K_{LL} = 4$ $\quad L_o = 80.0$ psf $\quad n = 3$ — IBC Section 1607.9

$A_T = (15.5 \text{ ft})(22.5 \text{ ft}) = 349 \text{ ft}^2$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} n A_T}}\right)$$ IBC Eqn 16-21

$$= 80.0 \text{ psf}\left(0.25 + \frac{15}{\sqrt{(4)(3)(349 \text{ ft}^2)}}\right)$$

$$= 38.5 \text{ psf} \geq 0.4L_o = 32.0 \text{ psf}$$

use $L = 38.5$ psf

Columns: 1A, 1F, 8A, 8F
Corner column without cantilever slabs

$K_{LL} = 4$ $\quad L_o = 80.0$ psf $\quad n = 3$ — IBC Section 1607.9

$$A_T = (15.5\text{ ft})\left(\frac{22.5}{2}+0.5\right) = 182\text{ ft}^2$$

IBC Eqn 16-24

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}nA_T}}\right) \geq 0.4L_o$$

$$= 80.0\text{ psf}\left(0.25+\frac{15}{\sqrt{(4)(3)(182\text{ ft}^2)}}\right) \geq 0.4(80.0\text{ psf})$$

$$= 45.7\text{ psf} \geq 32.0\text{ psf}$$

use $L = 45.7$ psf

Columns: 1C, 1D, 8C, 8D
Exterior column without cantilever slabs

$K_{LL} = 4$ $L_o = 80.0$ psf $n = 3$ IBC Section 1607.9

$$A_T = (15.5\text{ ft})(37.5\text{ ft}) - (349\text{ ft}^2\ /\ 2) = 407\text{ ft}^2$$

IBC Eqn 16-24

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}nA_T}}\right) \geq 0.4L_o$$

$$= 80.0\text{ psf}\left(0.25+\frac{15}{\sqrt{(4)(3)(407\text{ ft}^2)}}\right) \geq 0.4(80.0\text{ psf})$$

$$= 37.2\text{ psf} \geq 32.0\text{ psf}$$

use $L = 37.2$ psf

Columns: 2C, 2D, 3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D, 7C, 7D
Interior column

$K_{LL} = 4$ $L_o = 80.0$ psf $n = 3$ IBC Section 1607.9

$A_T = (37.5\text{ ft})(30.0\text{ ft}) = 1125\text{ ft}^2$

IBC Eqn 16-24

$$L = L_o\left(0.25+\frac{15}{\sqrt{K_{LL}nA_T}}\right) \geq 0.4L_o$$

$$= 80.0\text{ psf}\left(0.25+\frac{15}{\sqrt{(4)(3)(1{,}125\text{ ft}^2)}}\right) \geq 0.4(80.0\text{ psf})$$

$$= 30.3\text{ psf} \leq 32.0\text{ psf}$$

use $L = 32.0$ psf

Column	**Loading Width ft**	**Loading Length ft**	**Tributary Area ft^2**	**DL kips/ft^2**	$\boldsymbol{P_D}$ **kips**	**LL kips/ft^2**	$\boldsymbol{P_L}$ **kips**
2A, 2F, 3A, 3F, 4A, 4F 5A, 5F, 6A, 6F, 7A, 7F	23.0	30.0	690	0.075	51.8	0.033	22.9
exterior wall		30.0		0.503 klf	15.1		
					66.9		**22.9**
1B, 1E, 8B, 8E	5.50	22.5	349	0.075	26.2	0.0385	13.4
exterior wall		22.5		0.503 klf	11.3		
					37.5		**13.4**
1A, 1F, 8A, 8F	23.0	15.5	182	0.075	13.6	0.0457	8.32
exterior wall		27.3		0.503 klf	13.7		
					27.3		**8.32**
1C, 1D, 8C, 8D	37.5	15.5	407	0.075	30.5	0.0372	15.1
exterior wall		26.3		0.503 klf	13.2		
					43.7		**15.1**
2C, 2D, 3C, 3D, 4C, 4D 5C, 5D, 6C, 6D, 7C, 7D	37.5	30.0	1,125	0.075	**84.4**	0.0320	**36.0**

Column load summary

Col	**Floor**	P_D	P_L
		kips	**kips**
2A, 2F, 3A, 3F, 4A, 4F	Roof	26.2	18.7
5A, 5F, 6A, 6F, 7A, 7F	4th	66.9	22.9
	3rd	66.9	22.9
	2nd	66.9	22.9
	Total	**227**	**87.4**
1B, 1E, 8B, 8E	Roof	10.9	2.92
	4th	37.5	13.4
	3rd	37.5	13.4
	2nd	37.5	13.4
	Total	**123**	**43.1**
1A, 1F, 8A, 8F	Roof	17.7	9.18
	4th	27.3	8.32
	3rd	27.3	8.32
	2nd	27.3	8.32
	Total	**99.6**	**34.1**
1C, 1D, 8C, 8D	Roof	21.7	14.7
	4th	43.7	15.1
	3rd	43.7	15.1
	2nd	43.7	15.1
	Total	**153**	**60.0**
2C, 2D, 7C, 7D	Roof	22.5	28.1
	4th	84.4	36.0
	3rd	84.4	36.0
	2nd	84.4	36.0
	Total	**276**	**136**
3C, 3D, 4C, 4D	Roof	40.5	38.1
5C, 5D, 6C, 6D	4th	84.4	36.0
	3rd	84.4	36.0
	2nd	84.4	36.0
	Total	**294**	**146**

Selection of interior column

Columns: 3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D

Elevation of second floor slab:	113.5 ft
Elevation of first floor slab:	100 ft
Column unbraced length:	13.5 ft

LRFD	ASD
P_u = 1.2(294 kips) + 1.6(108 kips) + 0.5(38.1 kips) = 545 kips	P_a = 294 kips + 0.75(108 kips) + 0.75(38.1 kips) = 404 kips

From the tables, enter with the effective length of 13.5 ft, and proceed across the table until reaching the lightest size that has sufficient available strength at the required unbraced length.

Manual Table 4-1

LRFD	ASD
W12×65	W12×65
$\phi_c P_n$ = 696 kips > 545 kips **o.k.**	P_n / Ω_c = 463 kips > 404 kips **o.k.**
W14×68	W14×68
$\phi_c P_n$ = 655 kips > 545 kips **o.k.**	P_n / Ω_c = 436 kips > 404 kips **o.k.**

Manual Table 4-1

Manual Table 4-1

Selection of interior column

Columns: 2C, 2D, 7C, 7D

Elevation of second floor slab:	113.5 ft
Elevation of first floor slab:	100.0 ft
Column unbraced length:	13.5 ft

LRFD	ASD
P_u = 1.2(276 kips) + 1.6(108 kips) + 0.5(28.1 kips) = 518 kips	P_a = 276 kips + 108 kips = 384 kips

From the tables, enter with the effective length of 13.5 ft, and proceed across the table until reaching the lightest size that has sufficient available strength at the required unbraced length.

Manual Table 4-1

LRFD	ASD
W12×65	W12×65
$\phi_c P_n$ = 696 kips > 518 kips **o.k.**	P_n / Ω_c = 463 kips > 384 kips **o.k.**
W14×61	W14×61
$\phi_c P_n$ = 586 kips > 518 kips **o.k.**	P_n / Ω_c = 390 kips > 384 kips **o.k.**

Manual Table 4-1

Selection of exterior column

Columns: 1B, 1E, 8B, 8E

Elevation of second floor slab: 113.5 ft
Elevation of first floor slab: 100.0 ft
Column unbraced length: 13.5 ft

LRFD	ASD
P_u = 1.2(123 kips) + 1.6(43.1 kips) + 0.5(2.92 kips) = 218 kips	P_a = 123 kips + 43.1 kips = 166 kips

From the tables, enter with the effective length of 13.5 ft, and proceed across the table until reaching the lightest size that has sufficient available strength at the required unbraced length.

Manual Table 4-1

LRFD	ASD
W12×40 $\phi_c P_n$ = 316 kips > 218 kips **o.k.**	W12×40 P_n / Ω_c = 210 kips > 166 kips **o.k.**

Manual Table 4-1

Note: A 12 in. column was selected above for ease of erection of framing beams.
(Bolted double-angle connections can be used without bolt staggering.)

WIND LOAD DETERMINATION

Simplified wind load method (mean roof height less than 60 ft) — IBC Sect 1609.6

To qualify for the simplified wind load method, the following must be true. (IBC Sect 1609.6.1.1)

1. simple diaphragm building per 1609.2 **o.k.**
2. not a flexible building per1609.2 **o.k.**
3. does not have response characteristics requiring special considerations **o.k.**
4. does not have expansion joints or separations **o.k.**
5. regular shape and symmetrical cross section **o.k.**

Define input parameters

1.	Occupancy category:	II	IBC Table 1604.5
2.	Basic wind speed V,	90 mph (3-second gust)	IBC Sect 1609.3
3.	Importance factor I_w	1.0	IBC Table 1604.5
4.	Exposure category:	C	IBC Sect 1609.4
5.	Mean roof height	55' - 0"	
6.	Height and exposure adjustment λ:	1.59	IBC Table 1609.6.2.1(4)
7.	Roof angle	0°	

$p_s = \lambda\, I_w\, p_{s30} =$ (IBC Table 1609.6.2.1(1) and Eqn 16-34)

(1.59)(1.0)(12.8 psf)	= 20.4 psf	Horizontal pressure zone A
(1.59)(1.0)(8.5 psf)	= 13.5 psf	Horizontal pressure zone C
(1.59)(1.0)(-15.4 psf)	= -24.5 psf	Vertical pressure zone E
(1.59)(1.0)(-8.8 psf)	= -14.0 psf	Vertical pressure zone F
(1.59)(1.0)(-10.7 psf)	= -17.0 psf	Vertical pressure zone G
(1.59)(1.0)(-6.8 psf)	= -10.8 psf	Vertical pressure zone H

a = 10% of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft

(IBC Fig 1609.6.2.1)

a = the lesser of:	10% of least horizontal dimension:	12.3 ft
	40% of eave height:	22.0 ft
but not less than	4% of least horizontal dimension or 3 ft	4.91 ft

a = 12.3 ft
2a = 24.6 ft

Zone A - End zone of wall (width = 2a)
Zone C – Interior zone of wall
Zone E – End zone of windward roof (width = 2a)
Zone F – End zone of leeward roof (width = 2a)
Zone G – Interior zone of windward roof
Zone H – Interior zone of leeward roof

Calculate load to roof diaphragm

Mechanical screen wall height:	6 ft
Wall height:	$\frac{1}{2}\left[55.0\text{ ft} - 3\left(13.5\text{ ft}\right)\right] = 7.25\text{ ft}$
Parapet wall height:	2 ft
Total wall height at roof at screen wall:	6 ft + 7.25 ft = 13.3 ft
Total wall height at roof at parapet:	2 ft + 7.25 ft = 9.25 ft

Calculate load to fourth floor diaphragm

Wall height:	½ (55.0 ft – 40.5 ft) = 7.25 ft
	½ (40.5 ft – 27.0 ft) = 6.75 ft
Total wall height at floor:	6.75 ft + 7.25 ft = 14.0 ft

Calculate load to third floor diaphragm

Wall height:	½ (40.5 ft – 27.0 ft) = 6.75 ft
	½ (27.0 ft – 13.5 ft) = 6.75 ft
Total wall height at floor:	6.75 ft + 6.75 ft = 13.5 ft

Calculate load to second floor diaphragm

Wall height:	½ (27.0 ft – 13.5 ft) = 6.75 ft
	½ (13.5 ft – 0.0 ft) = 6.75 ft
Total wall height at floor:	6.75 ft + 6.75 ft = 13.5 ft

Total load to diaphragm:

Load to diaphragm at roof:	$w_{s(A)}$ = (20.4 psf)(9.25 ft) = 189 plf
	$w_{s(C)}$ = (13.5 psf)(9.25 ft) = 125 plf at parapet
	$w_{s(C)}$ = (13.5 psf)(13.3 ft) = 180 plf at screenwall
Load to diaphragm at fourth floor:	$w_{s(A)}$ = (20.4 psf)(14.0 ft) = 286 plf
	$w_{s(C)}$ = (13.5 psf)(14.0 ft) = 189 plf
Load to diaphragm at second and third: floors	$w_{s(A)}$ = (20.4 psf)(13.5 ft) = 275 plf
	$w_{s(C)}$ = (13.5 psf)(13.5 ft) = 182 plf

l = length of structure, ft

b = width of structure, ft

h = height of wall at building element, ft

Determine the wind load to each frame at each level. Conservatively apply the end zone pressures on both ends of the building simultaneously.

Wind from a north – south direction

Total load to each frame: $P_{W(n\text{-}s)} = 2\,a\,w_{s(A)} + w_{s(C)}\,(l / 2 - 2\,a)$

Wind from a east – west direction

Total load to each frame: $P_{W(e\text{-}w)} = 2\,a\,w_{s(A)} + w_{s(C)}\,(b / 2 - 2\,a)$

	l **ft**	***b*** **ft**	**2*a*** **ft**	***h*** **ft**	$p_{s(A)}$ **psf**	$p_{s(C)}$ **psf**	$w_{s(A)}$ **plf**	$w_{s(C)}$ **plf**	$P_{W(n\text{-}s)}$ **kips**	$P_{W(e\text{-}w)}$ **kips**	$v_{(n\text{-}s)}$ **plf**	$v_{(e\text{-}w)}$ **plf**
Roof	213	123	24.6	9.25	20.4		189		4.65	4.65	145	48.4
	95.4	65.4		9.25		13.5		125	4.43	2.55		
	93.0	33.0		13.3		13.5		180	8.37	2.97		
Fourth	213	123	24.6	14.0	20.4	13.5	286	189	22.5	14.0	250	66.7
Third	213	123	24.6	13.5	20.4	13.5	275	182	21.7	13.5	241	64.3
Second	213	123	24.6	13.5	20.4	13.5	275	182	21.7	13.5	241	64.3
Base of frame									**83.4**	**51.2**		

Note: The chart above indicates the total wind load in each direction acting on a steel frame at each level. The wind load at the ground level has not been included in the chart because it does not affect the steel frame.

SEISMIC LOAD DETERMINATION

The floor plan area: 120 ft, column center line to column center line, by 210 ft, column center line to column center line, with the edge of floor slab or roof deck 6 in. beyond the column center line.

Area = (121 ft)(211 ft) = 25,500 ft^2

The perimeter cladding system length:

Length = (2)(123 ft) + (2)(213 ft) = 672 ft

The perimeter cladding weight at floors:

Brick spandrel panel with metal stud backup	(7.50 ft)(0.055 ksf) = 0.413 klf
Window wall system	(6.00 ft)(0.015 ksf) = 0.090 klf
Total	**0.503 klf**

Typical roof dead load (from previous calculations):

Although 40 psf was used to account for the mechanical units and screen wall for the beam and column design, the entire mechanical area will not be uniformly loaded. Use 30 percent of the uniform 40 psf mechanical area load to determine the total weight of all of the mechanical equipment and screen wall for the seismic load determination.

Roof Area = (25,500 ft^2)(0.020 ksf) =	510 kips
Wall perimeter = (672 ft)(0.413 klf) =	278 kips
Mechanical Area = (2,700 ft^2)(0.300)(0.040 ksf)	32.4 kips
Total	**820 kips**

Typical third and fourth floor dead load:

Note: An additional 10 psf has been added to the floor dead load to account for partitions per IBC 1617.5.

Floor Area = (25,500 ft^2)(0.085 ksf) =	2,170 kips
Wall perimeter = (672 ft)(0.503 klf) =	338 kips
Total	**2,510 kips**

Second floor dead load: the floor area is reduced because of the open atrium

Floor Area = (24,700 ft^2)(0.085 ksf) =	2,100 kips
Wall perimeter = (672 ft)(0.503 klf) =	338 kips
Total	**2,440 kips**

Total dead load of the building:

Roof	820 kips
Fourth floor	2,510 kips
Third floor	2,510 kips
Second floor	2,440 kips
Total	**8,280 kips**

Calculate the seismic forces

Office Building Category II	IBC Table 1604.5
Seismic Use Group I	IBC Sect. 1616.2
Importance Factor $I_E = 1.00$	IBC Table 1604.5
$S_S = 0.121g$	IBC Fig 1615(1)
$S_1 = 0.06g$	IBC Fig 1615(2)
Soil, site class D	IBC Table 1615.1.1
F_a @ $S_S \leq 0.25 = 1.6$	IBC Table 1615.1.2(1)
F_v @ $S_1 \leq 0.1 = 2.4$	IBC Table 1615.1.2(2)
$S_{MS} = F_a\, S_S = (1.6)(0.121g) = 0.194g$	IBC Eqn 16-38
$S_{M1} = F_v\, S_1 = (2.4)(0.060g) = 0.144g$	IBC Eqn. 16-39
$S_{DS} = ⅔\, S_{MS} = ⅔\, (0.194g) = 0.129g$	IBC Eqn. 16-40
$S_{D1} = ⅔\, S_{M1} = ⅔\, (0.144g) = 0.096g$	IBC Eqn. 16-41
$S_{DS} < 0.167$g, Seismic Use Group I: Seismic Design Category: A	IBC Table 1616.3(1)
$0.067\text{g} \leq S_{D1} < 0.133$g, Seismic Use Group I: Seismic Design Category: B	IBC Table 1616.3(2)
Seismic Design Category B may be used and it is therefore permissible to select a structural steel system not specifically detailed for seismic resistance, for which the Seismic Response Modification Coefficient $R = 3$	IBC Sect 1616.3
Building Height, $h_n = 55.0$ ft	ASCE 7 Table 12.8-2
$C_t = 0.02$: $x = 0.75$	
$T_a = C_t\,(h_n)^x = (0.02)(55.0\text{ ft})^{0.75} = 0.404$ sec	ASCE 7 Eqn. 12.8-7
$\rho = 1.0$	ASCE 7 Sec. 12.3.4

Determine the vertical seismic effect term

$$0.2S_{DS}D = 0.2(0.129_g)D$$
$$= 0.0258D$$

The following seismic load combinations indicated in ASCE 7 Sec. 2 have been adjusted for the combination of load effects indicated in ASCE 7 Sect. 12.4.2.3.

Seismic load combinations adjusted for the combination of load effects

LRFD	ASD
$1.2D + 1.0E + 0.5L + 0.2S$	$D + 0.7E$
$= 1.2D + 1.0(\rho Q_E + 0.2S_{DS}D) + 0.5L + 0.2S$	$= D + 0.7(\rho Q_E + 0.2S_{DS}D)$
$= 1.2D + 1.0(1.0Q_E + 0.0258D) + 0.5L + 0.2S$	$= D + 0.7(1.0Q_E + 0.0258D)$
$= 1.23D + 1.0Q_E + 0.5L + 0.2S$	$= 1.02D + 0.7Q_E$
$0.9D + 1.0E$	$D + 0.75(0.7E) + 0.75L + 0.75S$
$= 0.9D + 1.0(\rho Q_E - 0.2S_{DS}D)$	$= D + 0.75[0.7(\rho Q_E + 0.2S_{DS}D)] + 0.75L + 0.75S$
$= 0.9D + 1.0(1.0Q_E - 0.258D)$	$= D + 0.75[0.7(1.0Q_E + 0.0258D)] + 0.75L + 0.75S$
$= 0.874D + 1.0Q_E$	$= 1.01D + 0.75(0.7Q_E) + 0.75L + 0.75S$
	$0.6D + 0.7E$
	$= 0.6D + 0.7(\rho Q_E - 0.2S_{DS}D)$
	$= 0.6D + 0.7(1.0Q_E - 0.0258D)$
	$= 0.582D + 0.7Q_E$

Note: Q_E = the effect of horizontal seismic (earthquake induced) forces

Overstrength Factor: $\Omega_o = 3$ — ASCE 7 Table 12.2-1

Seismic Base Shear: $V = C_s W$ — ASCE 7 Eqn 12.8-1

$$C_s = \frac{S_{DS}}{R/I_E} = \frac{0.129}{3/1} = 0.043 \quad \textbf{\underline{controls}}$$

ASCE 7 Eqn 12.8-2

C_s need not be greater than: $C_s = \dfrac{S_{D1}}{\left(R/I_E\right)T} = \dfrac{0.096}{\left(3/1\right)\left(0.404\right)} = 0.079$

ASCE 7 Eqn 12.8-3

C_s shall not be taken less than: $C_s = 0.044\, S_{DS}I_E = 0.044(0.129)(1.0) = 0.006$

$V = 0.043(8{,}280 \text{ kips}) = 356 \text{ kips}$ — ASCE 7 Eqn 12.8-5

Calculate vertical distribution of seismic forces — ASCE 7 Sect 12.8-3

$F_x = C_{vx}V = C_{vx}(0.043)W$ — ASCE 7 Eqn 12.8-11

$$C_{vx} = \frac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$$

ASCE 7 Eqn 12.8-12

for structures having a period of 0.5 sec or less, $k = 1$

ASCE 7 Sect 12.8.3

Calculate horizontal shear distribution and torsion — ASCE 7 Sect 12.8.4

ASCE 7 Eqn 12.8-13

$$V_x = \sum_{i=x}^{n} F_i$$

Calculate Overturning Moment — ASCE 7 Sect 12.8.5

$$M_x = \sum_{i=x}^{n} F_i (h_i - h_x)$$

	w_x **kips**	$h_x{}^k$ **ft**	$w_x h_x{}^k$ **kip-ft**	C_{vx} **kips**	F_x **kips**	V_x **kips**	M_x **k-ft**
Roof	820	55.0	45,100	0.182	64.9	64.9	
Fourth	2,510	40.5	102,000	0.411	146	211	941
Third	2,510	27.0	67,800	0.273	97.5	308	3,790
Second	2,440	13.5	32,900	0.133	47.4	356	7,950
Base	**8,280**		**248,000**		**356**		**12,800**

Calculate strength and determine rigidity of diaphragms

Determine the diaphragm design forces:

F_p is the larger of:

1. The portion of the design seismic force at the level of the diaphragm that depends on the diaphragm for transmission to the vertical elements of the seismic force-resisting system, or
2. $0.2S_{DS}I_{WP} + V_{PX}$

	w_p **kips**	A **kips**	B **kips**	C **kips**	F_p **kips**	$v_{(n\text{-}s)}$ **plf**	$v_{(e\text{-}w)}$ **plf**
Roof	820	35.3	64.9	21.2	64.9	297	170
Fourth	2,510	108	146	64.8	146	892	382
Third	2,510	108	97.5	64.8	108	660	283
Second	2,440	105	47.4	63.0	105	642	279
	8,280	**356**					

$A = \frac{w_p}{W} V$, where W = 8,280 kips and V = 356 kips

B = force at a level based on the vertical distribution of seismic forces

$C = 0.2S_{DS} I w_p + V_{px}$, where V_{px} = 0 for this building

F_p = max(A, B, C)

Note: The diaphragm shear loads include the effects of openings in the diaphragm and the increased force due to real and accidental torsion.

Roof

Roof deck: 1½ in. deep, 22 gage, Type B,
support fasteners; ⅝ in. puddle welds and sidelap fasteners; #10 TEK screws
Joist spacing = 6.0 ft
Diaphragm length = 210 ft
Diaphragm width = 120 ft

Load to diaphragm = (64.9 kips) / (210 ft) = 0.309 klf

$$\text{Diaphragm shear load} = 0.550\frac{(64.9\text{ kips})}{(120\text{ ft})} = 0.297\,\text{klf}$$

Fastener layout = 36 / 5; sidelap = 3 #10 TEK

Steel Deck Institute *Diaphragm Design Manual*

D_{xx} = 758 ft $K1$ = 0.286 ft^{-1} $K2$ = 870 kip/in. $K4$ = 3.78

Shear strength

$\Omega = 3.00$ Eq
$\Omega = 2.35$ Wind

Nominal strength = 820 plf

$$\text{Available strength } Eq = \frac{820\text{ plf}}{3.00} = 273\text{ plf} > 0.7(297\text{ plf}) = 208\text{ plf} \quad \textbf{o.k.}$$

$$\text{Available strength } Wind = \frac{820\text{ plf}}{2.35} = 349\text{ plf} > 145\text{ plf} \quad \textbf{o.k.}$$

$$G' = \frac{K2}{K_A + \dfrac{0.3D_{xx}}{\text{span}} + 3K1\text{ span}} = \frac{870\text{ kips/in.}}{3.78 + \dfrac{0.3(758\text{ ft})}{6.00\text{ ft}} + 3\left(\dfrac{0.286}{\text{ft}}\right)(6.00\text{ ft})} = 18.6\text{ kips/in.}$$

$$\Delta = \frac{wL^2}{8BG'} = \frac{(0.309\text{ klf})(210\text{ ft})^2}{8(120\text{ ft})(18.6\text{ kips/in.})} = 0.763\text{ in.}$$

Story drift = 0.087 in. (from computer output)

IBC Sect 1602

Diaphragm, rigid. Lateral deformation ≤ 2 (story drift)

Δ = 0.763 in. > 2(0.141 in.) = 0.282 in.

Therefore, the roof diaphragm is flexible in the N-S direction, but using a rigid diaphragm distribution is more conservative for the analysis of this building. By similar reasoning, the roof diaphragm will also be treated as a rigid diaphragm in the E-W direction.

Third and Fourth floors

Floor deck: 3 in. deep, 22 gage, Composite deck with normal weight concrete,
support fasteners; ⅝ in. puddle welds and button punched sidelap fasteners

Beam spacing = 10.0 ft
Diaphragm length = 210 ft
Diaphragm width = 120 ft

Load to diaphragm = (146 kips) / (210 ft) = 0.695 klf

Diaphragm shear load $= \dfrac{0.550(146\text{ kips})}{(90.0\text{ ft})} = 0.892\text{ klf}$

Fastener layout = 36 / 4; 1 button punched sidelap fastener

Steel Deck Institute *Diaphragm Design Manual*

$K1 = 0.729\text{ ft}^{-1}$ $K2 = 870$ kips/in. $K3 = 2{,}380$ kips/in. $K4 = 3.54$

Shear strength

$\Omega = 3.25$ Filled Eq or Wind

Nominal strength = 5160 plf

Available strength $= \dfrac{5160\text{ plf}}{3.25} = 1590\text{ plf} > 0.7(892\text{ plf}) = 624\text{ plf}$ **o.k.**

$$G' = \left(\frac{K2}{K4 + 3K1\text{ span}}\right) + K3 = \left(\frac{870\text{ kips/in.}}{3.54 + 3\left(\dfrac{0.729}{\text{ft}}\right)(10.0\text{ ft})}\right) + 2{,}380\text{ kips/in.} = 2{,}410\text{ kips/in.}$$

$$\Delta = \frac{wL^2}{8BG'} = \frac{(0.695\text{ klf})(210\text{ ft})^2}{8(120\text{ ft})(2{,}410\text{ kips/in.})} = 0.0132\text{ in.}$$

Story drift; 0.245 in. (from computer output)

Diaphragm, rigid. Lateral deformation ≤ 2 (story drift)

IBC Sect 1602

$\Delta = 0.0132$ in. ≤ 0.245 in. = 0.490 in.

Therefore, the floor diaphragm is rigid in the N-S direction, and by inspection, is also rigid in the E-W direction.

Second floor

Floor deck: 3 in. deep, 22 gage, Composite deck with normal weight concrete,
support fasteners: ⅝ in. puddle welds and button punched sidelap fasteners
Beam spacing = 10.0 ft
Diaphragm length = 210 ft
Diaphragm width = 120 ft
Because of the open atrium in the floor diaphragm, an effective diaphragm depth of 75 ft, will be used for the deflection calculations

Load to diaphragm = (105 kips) / (210 ft) = 0.500 klf

Diaphragm shear load $= \dfrac{0.550(105\text{ kips})}{90.0\text{ ft}} = 0.642\text{ klf}$

Fastener layout = 36 / 4; 1 button punched sidelap fastener

Steel Deck Institute *Diaphragm Design Manual*

$K1 = 0.729\ \text{ft}^{-1}$ $\quad K2 = 870$ kips/in. $\quad K3 = 2{,}380$ kips/in. $\quad K4 = 3.54$

Shear strength

$\Omega = 3.25$ filled EQ or Wind

Nominal strength = 5160 plf

Available strength $Eq = \dfrac{5160\ \text{plf}}{3.25} = 1590\ \text{plf} > 0.7\left(642\ \text{plf}\right) = 449\ \text{plf}$ **o.k.**

Available strength $W = \dfrac{5160\ \text{plf}}{3.25} = 1590\ \text{plf} > 241\ \text{plf}$ **o.k.**

$$G' = \left(\frac{K2}{K4 + 3K1\ \text{span}}\right) + K3 = \left(\frac{870}{3.54 + 3\left(0.729\right)\left(10.0\ \text{ft}\right)}\right) + 2{,}380 = 2{,}410\ \text{k/in}$$

$$\Delta = \frac{wL^2}{8BG'} = \frac{\left(0.500\ \text{klf}\right)\left(210\ \text{ft}\right)^2}{8\left(75.0\ \text{ft}\right)\left(2{,}410\ \text{k/in.}\right)} = 0.0152\ \text{in.}$$

Story drift = 0.228 in. (from computer output)

Diaphragm, rigid. Lateral deformation ≤ 2 (story drift)

IBC Sect 1602

$\Delta = 0.0152$ in. ≤ 2(0.228 in.) = 0.456 in.

Therefore, the floor diaphragm is rigid in the N – S direction, and by inspection is also rigid in the E – W direction.

ASCE 7 Sect 12.8.4

Horizontal shear distribution and torsion:

Load to Grids 1 and 8

	F_y	Load to Frame		Accidental Torsion		Total
	kips	**%**	**kips**	**%**	**kips**	**kips**
Roof	64.9	50	32.5	5	3.25	35.8
Fourth	146	50	73.0	5	7.30	80.3
Third	97.5	50	48.8	5	4.88	53.7
Second	47.4	50	23.7	5	2.37	26.1
Base						**196**

Load to Grids A and F

	F_x	Load to Frame		Accidental Torsion		Total	
	kips	**%**	**kips**	**%**	**kips**	**kips**	
Roof	64.9	50	32.5	5	3.25	35.8	
Fourth	146	50	73.0	5	7.30	80.3	
Third	97.5	50	48.8	5	4.88	53.7	
Second	47.4	50.8(11)		24.1	5	2.37	26.5
Base						**196**	

Note: In this example, Grids A and F have both been conservatively designed for the slightly higher load on Grid A due to the atrium opening.

	Area ft^2	**Mass** **kips**	***y*-dist** **ft**	$\boldsymbol{M_y}$ **k-ft**
I	25,500	2,170	60.5	131,000
II	841	71.5	90.5	6,470
	24,700	**2,100**		**125,000**

y = 125,000 kip-ft/2,100 kips = 59.5 ft

(100%)(121 ft – 59.5 ft)/121 ft = 50.8%

MOMENT FRAME MODEL

Grids 1 and 8 were modeled in conventional structural analysis software as two-dimensional models. The second-order option in the structural analysis program was not used. Rather, for illustration purposes, second-order effects are calculated separately, using the "Second-Order Analysis by Amplified First-Order Elastic Analysis" method in Specification Chapter C2.1b.

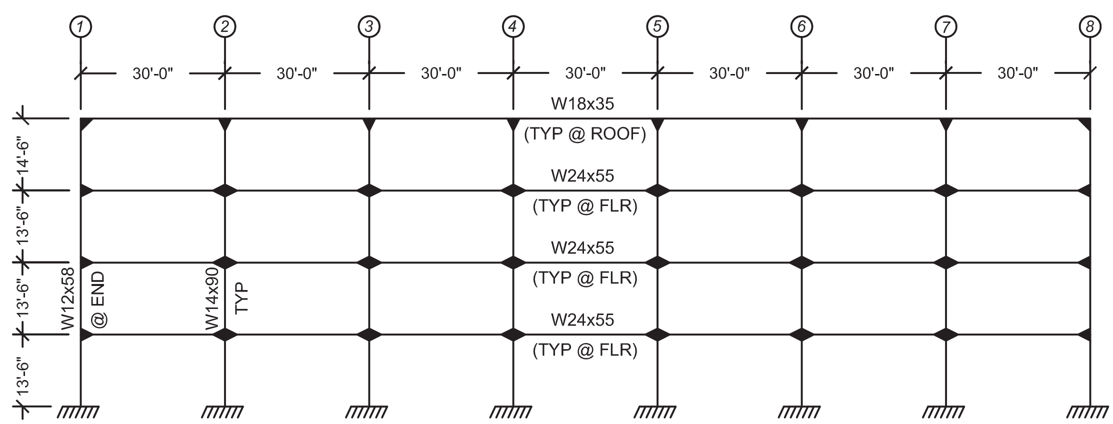
The column and beam layouts for the moment frames follow. Although the frames on Grids A and F are the same, slightly heavier seismic loads accumulate on grid F after accounting for the atrium area and accidental torsion. The models are half-building models. The frame was originally modeled with W14×82 interior columns and W21×44 non-composite beams, but was revised because the beams and columns did not meet the strength requirements. The W14×82 column size was incremented up to a W14×90 and the W21×44 beams were upsized to W24×55 beams. Minimum composite studs are specified for the beams (corresponding to $\sum Q_n = 0.25F_yA_s$), but the beams were modeled with a stiffness of $I_{eq} = I_s$. Alternatively, the beams could be modeled with a stiffness of $I_{eq} = 0.6I_{LB} + 0.4I_n$ (Formula 24, AISC Design Guide 8).

The frame was checked for both wind and seismic story drift limits. Based on the results on the computer analysis, the frame meets the L/400 drift criterion for a 10 year wind (0.7W) indicated in Section CB.1.2 of ASCE 7-02. In addition, the frame meets the $0.025h_{sx}$ allowable story drift limit per IBC Table 1617.3.1, Seismic Use Group I.

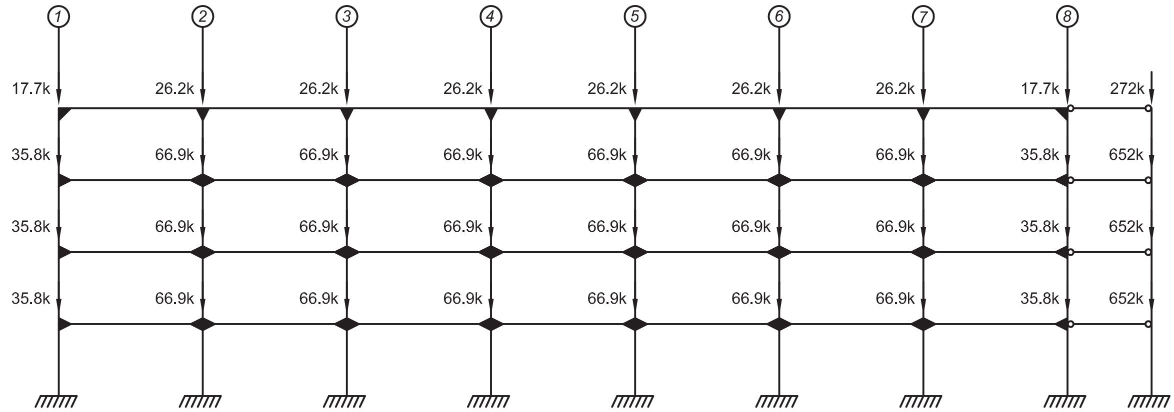
All of the vertical loads on the frame were modeled as point loads on the frame. The remainder of the half-building model gravity loads were accumulated in the leaning column, which was connected to the frame portion of the model with pinned ended links. See Geschwindner, AISC Engineering Journal, Fourth Quarter 1994, *A Practical Approach to the "Leaning" Column*. The dead load and live load are shown in the load cases that follow. The wind, seismic, and notional loads from leaning columns are modeled and distributed 1/14 to exterior columns and 1/7 to the interior columns. This approach minimizes the tendency to accumulate too much load in the lateral system nearest an externally applied load.

There are five horizontal load cases. Two are the wind load and seismic load, per the previous discussion. In addition, notional loads of $N_i = 0.002Y_i$ were established. The model layout, nominal dead, live, and snow loads with associated notional loads, wind loads and seismic loads are shown in the figures below.

The same modeling procedures were used in the braced frame analysis. If column bases are not fixed in construction, they should not be fixed in the analysis.



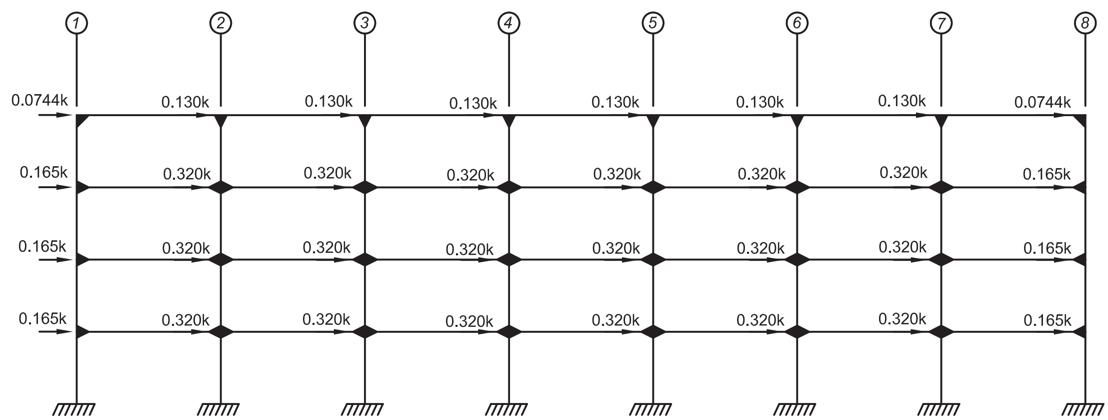
FRAME LAYOUT (GRID A & F)



FRAME (1/2 BLDG)

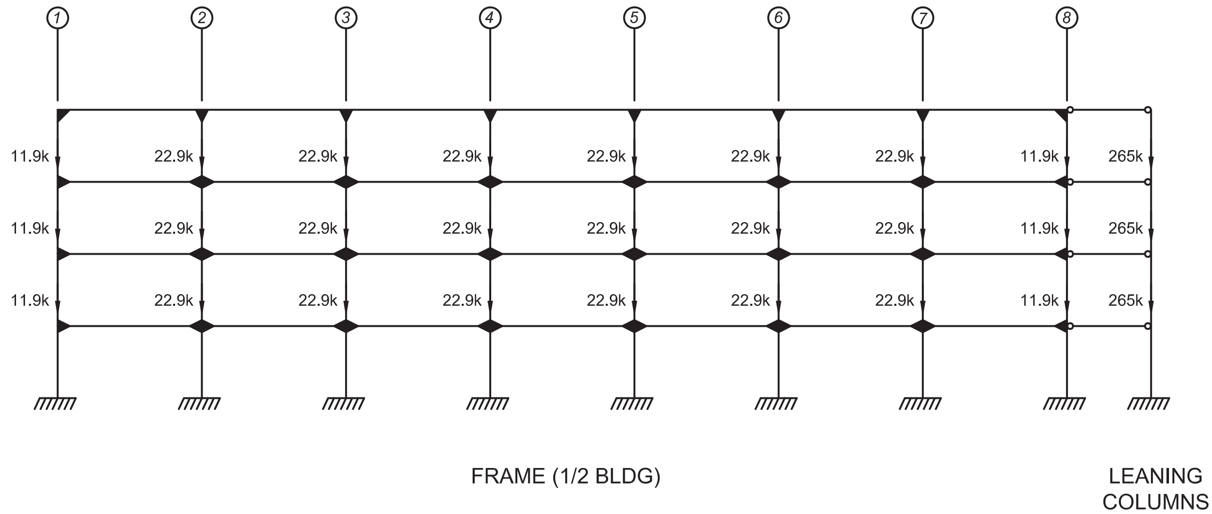
LEANING COLUMNS

<u>NOMINAL DEAD LOADS</u>



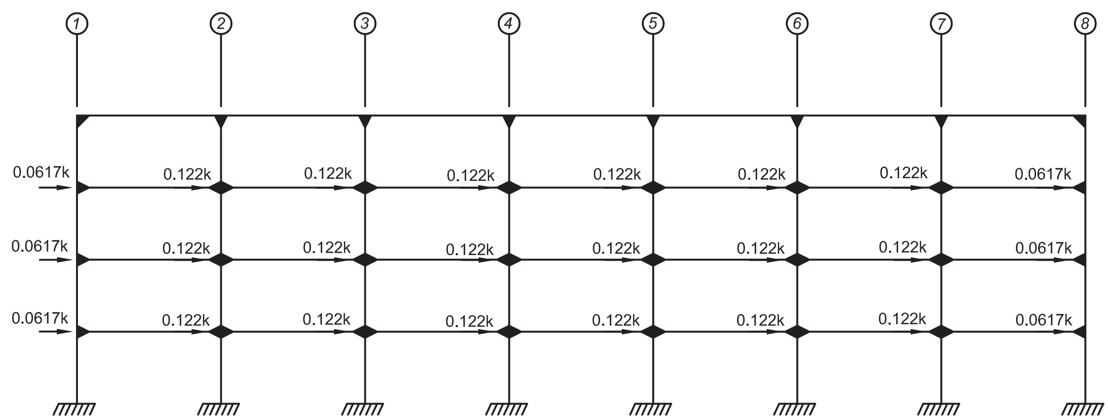
FRAME (1/2 BLDG)

<u>NOTIONAL DEAD LOADS</u>



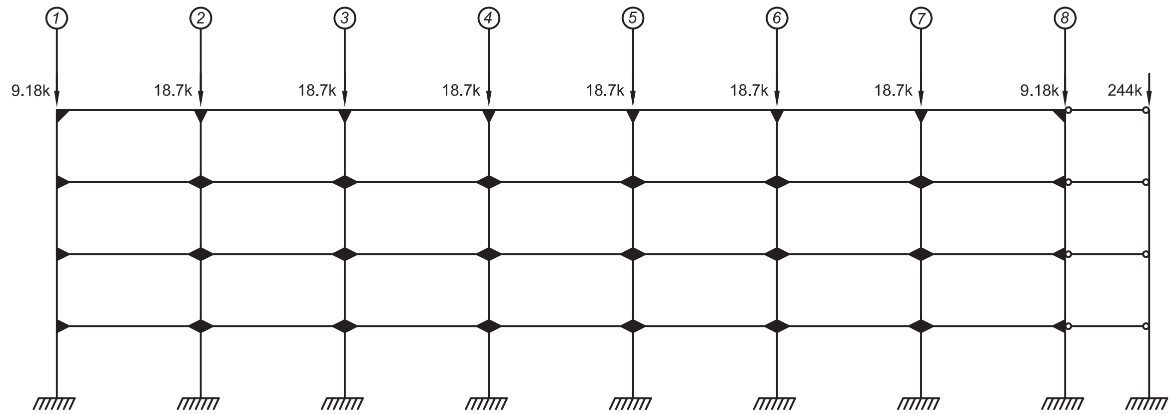
FRAME (1/2 BLDG)

NOMINAL LIVE LOADS



FRAME (1/2 BLDG)

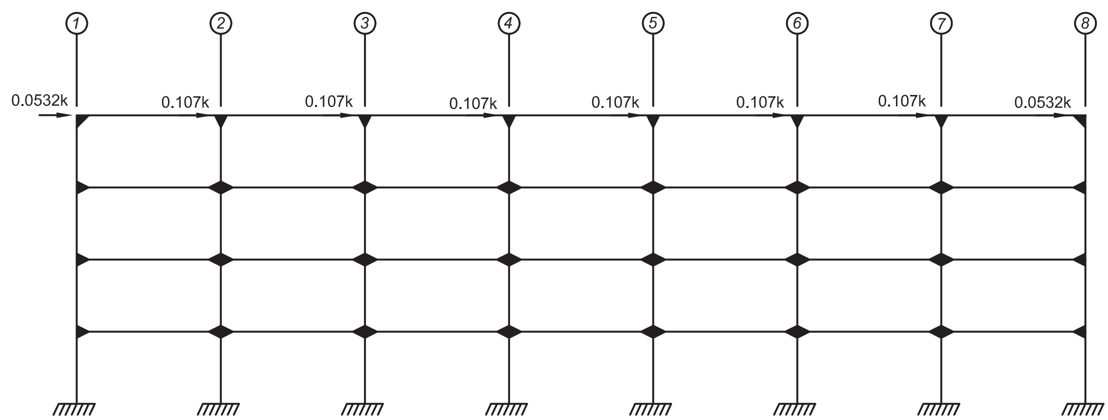
NOTIONAL LIVE LOADS



FRAME (1/2 BLDG)

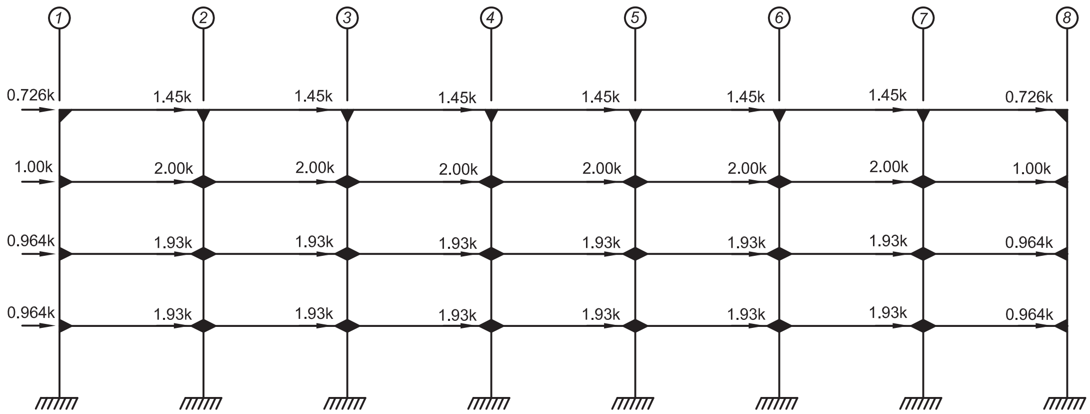
LEANING COLUMNS

NOMINAL SNOW LOADS



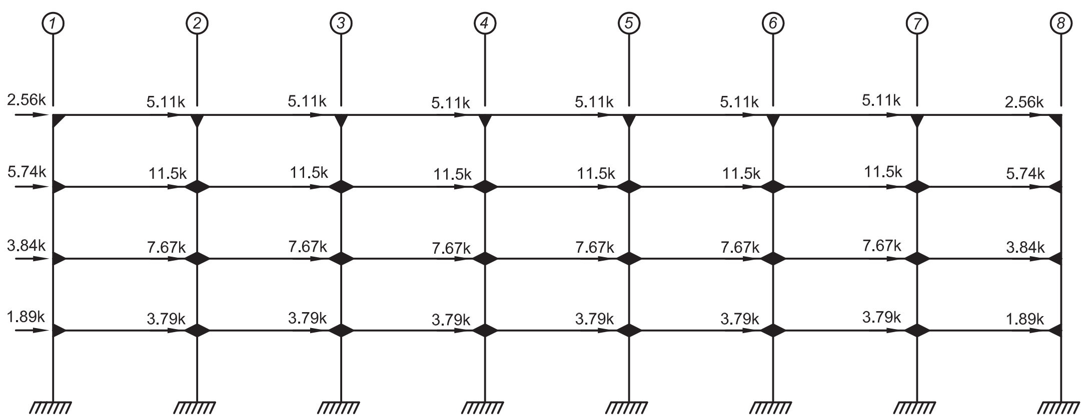
FRAME (1/2 BLDG)

NOTIONAL SNOW LOADS



FRAME (1/2 BLDG)

NOMINAL WIND LOADS



FRAME (1/2 BLDG)

SEISMIC LOADS (1.0Q_E)

CALCULATION OF REQUIRED STRENGTH - THREE METHODS

Three methods for checking one of the typical interior column designs at the base of the building are presented below. All three of presented methods require a second-order analysis (either direct via computer analysis techniques or by amplifying a first order analysis). A fourth method called the "First-Order Analysis Method" is also an option. This method does not require a second order analysis; however, this method is not presented below. For additional guidance on applying any of these methods, see the discussion in Manual Part 2 titled Required Strength, Stability, Effective Length, and Second-Order Effects.

GENERAL INFORMATION FOR ALL THREE METHODS

Seismic loads controlled over wind loads in the direction of the moment frames in the example building. The frame analysis was run for all LRFD and ASD load combinations; however, only the controlling combinations have been illustrated in the following examples. A minimum lateral load of 0.2% of gravity load was included for all load cases for which the lateral load was not greater.

The second order analysis for all the examples below was carried out by doing a first order analysis and then amplifying the results to achieve a set of second order design forces. A summary of the axial loads, moments and first floor drift from the first order analysis are summarized below. Second order member forces are determined from these forces in each of the examples below.

METHOD 1. EFFECTIVE LENGTH METHOD

Required strengths in the frame are determined from a second-order analysis. In this example the second-order analysis is performed by amplifying the axial forces and moments in members and connections from a first-order analysis. The available strengths of compression members are calculated using effective length factors computed from a sidesway stability analysis.

Specification C2.2a

A first-order frame analysis is run using the load combinations for LRFD or ASD. A minimum lateral load (notional load) equal to 0.2% of the gravity loads is included for any load case for which the lateral load is not already greater. The general load combinations are given in ASCE/SEI 7 and are summarized in Part 2 of the Manual.

A summary of the axial loads, moments and 1st floor drifts from the first-order computer analysis is shown below:

LRFD	ASD
$1.23D \pm 1.0Q_E + 0.5L + 0.2S$ (Controls for columns and beams) For Interior Column Design: P_u = 317 kips M_{1u} = 148 kip-ft (from first-order analysis) M_{2u} = 233 kip-ft (from first-order analysis) First-order first floor drift = 0.575 in.	$1.01D + 0.75(0.7Q_E) + 0.75L + 0.75S$ (Controls for columns and beams) For Interior Column Design: P_a = 295 kips M_{1a} = 77.9 kip-ft (from first-order analysis) M_{2a} = 122 kip-ft (from first-order analysis) First-order first floor drift = 0.302 in.

The required second-order flexural strength, M_r, and axial strength, P_r, are calculated as follows:

For typical interior columns, the gravity load moments are approximately balanced; therefore, M_{nt} = 0.0 kips.

LRFD	ASD	
$M_r = B_1M_{nt} + B_2M_{lt}$	$M_r = B_1M_{nt} + B_2M_{lt}$	Eqn. C2-1a
Determine B_1	*Determine B_1*	
P_r = required second-order axial strength using LRFD or ASD load combinations, kips.	P_r = required second-order axial strength using LRFD or ASD load combinations, kips.	Specification Section C2.1b
Note that for members subject to axial compression, β_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$.	Note that for members subject to axial compression, β_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$.	
Therefore, P_r = 317 kips (from previous calculations) and	Therefore, P_r = 295 kips (from previous calculations) and	
I = 999 in.^{4} (W14×90)	I = 999 in.^{4} (W14×90)	
$P_{e1} = \dfrac{\pi^2 EI}{(K_1L)^2} = \dfrac{\pi^2(29{,}000\text{ ksi})(999\text{ in}^4)}{\left[(1.0)(13.5\text{ ft})(12\text{ in/ft})\right]^2} = 10{,}900\text{ kips}$	$P_{e1} = \dfrac{\pi^2 EI}{(K_1L)^2} = \dfrac{\pi^2(29{,}000\text{ ksi})(999\text{ in}^4)}{\left[(1.0)(13.5\text{ ft})(12\text{ in/ft})\right]^2} = 10{,}900\text{ kips}$	Eqn. C2-5
$C_m = 0.6 - 0.4(M_1 / M_2) = 0.6 - 0.4\,(148\text{ kip-ft} / 233\text{ kip-ft}) = 0.346$	$C_m = 0.6 - 0.4(M_1 / M_2) = 0.6 - 0.4\,(77.9\text{ kip-ft} / 122\text{ kip-ft}) = 0.345$	Eqn. C2-4
$\alpha = 1.0$,	$\alpha = 1.6$,	
$B_1 = \dfrac{C_m}{1-\left(\dfrac{\alpha P_r}{P_{e1}}\right)} \geq 1 = \dfrac{0.346}{1-\left[\dfrac{(1.0)(317\text{ kips})}{10{,}900\text{ kips}}\right]} \geq 1 = 0.356 \geq 1;\ \text{Use } 1.00$	$B_1 = \dfrac{C_m}{1-\left(\dfrac{\alpha P_r}{P_{e1}}\right)} \geq 1 = \dfrac{0.345}{1-\left[\dfrac{(1.60)(295\text{ kips})}{10{,}900\text{ kips}}\right]} \geq 1 = 0.361 \geq 1;\ \text{Use } 1.00$	Eqn. C2-2
Calculate B_2	*Calculate B_2*	
$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha\sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$	$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha\sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$	Eqn. C2-3
where:	where:	
$\alpha = 1.0$,	α= 1.6,	
$\sum P_{nt}$ = 5,440 kips (from computer output)	$\sum P_{nt}$ = 5,120 kips (from computer output)	Specification

LRFD	ASD	
and $\sum P_{e2}$ may be taken as $= R_M \dfrac{\sum HL}{\Delta_H}$ where R_M is taken as 0.85 for moment frames	and $\sum P_{e2}$ may be taken as $= R_M \dfrac{\sum HL}{\Delta_H}$ where R_M is taken as 0.85 for moment frames	Section C2.1b
$\sum H = 1.23D + 1.0Q_E + 0.5L + 0.2S$ $= 196$ kips (Horizontal) (from previous seismic force distribution calculations)	$\sum H = 1.01D + 0.75(0.7Q_E) + 0.75L + 0.75S$ $= 103$ kips (Horizontal) (from previous seismic force distribution calculations)	
$\Delta_H = 0.575$ in. (from computer output)	$\Delta_H = 0.302$ in. (from computer output)	
$\sum P_{e2} = 0.85\dfrac{(196\text{ kips})(13.5\text{ ft})(12\text{ in/ft})}{0.575\text{ in.}}$ $= 46{,}900$ kips	$\sum P_{e2} = 0.85\dfrac{(103\text{ kips})(13.5\text{ ft})(12\text{ in/ft})}{0.302\text{ in.}}$ $= 47{,}000$ kips	Eqn. C2-6b
$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$ $= \dfrac{1}{1-\left[\dfrac{(1.0)(5{,}440\text{ kips})}{46{,}900\text{ kips}}\right]} \geq 1$ $= 1.13 \geq 1$	$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$ $= \dfrac{1}{1-\left[\dfrac{(1.6)(5{,}120\text{ kips})}{47{,}000\text{ kips}}\right]} \geq 1$ $= 1.21 \geq 1$	Eqn. C2-3
Note: $B_2 < 1.5$, therefore the effective length method is acceptable.	Note: $B_2 < 1.5$, therefore the effective length method is acceptable.	
Calculate amplified moment $M_r = (1.00)(0.0\text{ kip-ft}) + (1.13)(233\text{ kip-ft})$ $= 263$ kip-ft	*Calculate amplified moment* $M_r = (1.00)(0.0\text{ kip-ft}) + (1.21)(122\text{ kip-ft})$ $= 148$ kip-ft	Specification Section C2.2
Calculate amplified axial force $P_{nt} = 317$ kips (from computer analysis)	*Calculate amplified axial force* $P_{ult} = 295$ kips (from computer analysis)	
For a long frame, such as this one, the change in load to the interior columns associated with lateral load is negligible.	For a long frame, such as this one, the change in load to the interior columns associated with lateral load is negligible.	
Therefore, $P_{lt} = 0$ $P_r = P_{nt} + B_2P_{lt}$ $= 317$ kips + (1.13)(0.0 kips) $= 317$ kips	Therefore, $P_{lt} = 0$ $P_r = P_{nt} + B_2P_{lt}$ $= 295$ kips + (1.21)(0.0 kips) $= 295$ kips	

LRFD	ASD
Determine the controlling effective length	*Determine the controlling effective length*
For out-of-plane buckling in the braced frame	For out-of-plane buckling in the braced frame
K_y=1.41	K_y=1.41
For in-plane buckling in the moment frame, use the nomograph	For in-plane buckling in the moment frame, use the nomograph
$K_x = 1.43$	$K_x = 1.43$
To account for leaning columns in the controlling load case	To account for leaning columns in the controlling load case
For leaning columns, $\sum Q = 3{,}190$ kips; $\sum P = 2{,}250$ kips	For leaning columns, $\sum Q = 3{,}030$ kips; $\sum P = 2{,}090$ kips
$K = K_o\sqrt{1+\frac{\sum Q}{\sum P}}$ $= 1.41\sqrt{1+\frac{3{,}190\text{ kips}}{2{,}250\text{ kips}}} = 2.19$	$K = K_o\sqrt{1+\frac{\sum Q}{\sum P}}$ $= 1.41\sqrt{1+\frac{3{,}030\text{ kips}}{2{,}090\text{ kips}}} = 2.21$
$\frac{KL_x}{r_x / r_y} = \frac{2.19(13.5\text{ ft})}{1.66} = 17.8\text{ ft}$	$\frac{KL_x}{r_x / r_y} = \frac{2.21(13.5\text{ ft})}{1.66} = 18.0\text{ ft}$
$P_c = 933$ kips (W14×90 @ KL = 17.6 ft)	$P_c = 618$ kips (W14×90 @ KL = 18.0 ft)
$\frac{P_r}{P_c} = \frac{317\text{ kips}}{933\text{ kips}} = 0.340 \geq 0.2$	$\frac{P_r}{P_c} = \frac{295\text{ kips}}{618\text{ kips}} = 0.477 \geq 0.2$
$M_{cx} = 573$ kip-ft (W14×90 with $L_b = 13.5$ ft)	$M_{cx} = 382$ kip-ft (W14×90 with $L_b = 13.5$ ft)
$\frac{P_r}{P_c} + \left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$	$\frac{P_r}{P_c} + \left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$
$0.340 + \left(\frac{8}{9}\right)\left(\frac{263\text{ kip-ft}}{573\text{ kip-ft}}\right) \leq 1.0$	$0.477 + \left(\frac{8}{9}\right)\left(\frac{148\text{ kip-ft}}{382\text{ kip-ft}}\right) \leq 1.0$
$0.748 \leq 1.0$ **o.k.**	$0.821 \leq 1.0$ **o.k.**

Commentary Section C2.2b

Manual Table 4-1

Specification Sect H1.1

Manual Table 3-2

Eqn. H1-1a

METHOD 2. SIMPLIFIED EFFECTIVE LENGTH METHOD

A simplification of the effective length method using a method of second-order analysis based upon drift limits and other assumptions is described in Chapter 2 of the Manual. A first-order frame analysis is conducted using the load combinations for LRFD or ASD. A minimum lateral load (notional load) equal to 0.2% of the gravity loads is included for any load case for which the lateral load is not already greater.

LRFD	ASD
$1.23D \pm 1.0Q_E + 0.5L + 0.2S$ (Controls for columns and beams) For a first-order analysis For Interior Column Design: P_u = 317 kips M_{1u} = 148 kip-ft (from first-order analysis) M_{2u} = 233 kip-ft (from first-order analysis) First-floor first-order drift = 0.575 in.	$1.01D + 0.75(0.7Q_E) + 0.75L + 0.75S$ (Controls for columns and beams) For a first-order analysis For Interior Column Design: P_a = 295 kips M_{1a} = 77.9 kip-ft (from first-order analysis) M_{2a} = 122 kip-ft (from first-order analysis) First-floor first-order drift = 0.302 in.

Then the following steps are executed.

LRFD	ASD
Step 1: Lateral load = 196 kips Deflection due to first-order elastic analysis Δ = 0.575 in. between first and second floor Floor height = 13.5 ft Drift ratio = (13.5 ft)(12 in./ft) / 0.575 in. = 282 *Step 2:* Design story drift limit = H/400 Adjusted Lateral load = (282/ 400)(196 kips) = 138 kips *Step 3:* $\text{Load ratio} = (1.0)\dfrac{\text{total story load}}{\text{lateral load}}$ $= (1.0)\dfrac{5{,}440\text{ kips}}{138\text{ kips}}$ = 39.4 From the table: B_2 = 1.1	*Step 1:* Lateral load = 103 kips Deflection due to first-order elastic analysis Δ = 0.302 in. between first and second floor Floor height = 13.5 ft Drift ratio = (13.5 ft)(12 in./ft) / 0.302 in. = 536 *Step 2:* Design story drift limit = H/400 Adjusted Lateral load = (536 / 400)(103 kips) = 138 kips *Step 3:* (for an ASD design the ratio must be multiplied by 1.6) $\text{Load ratio} = (1.6)\dfrac{\text{total story load}}{\text{lateral load}}$ $= (1.6)\dfrac{5{,}120\text{ kips}}{138\text{ kips}}$ = 59.4 From the table: B_2 = 1.2

LRFD	ASD	
Which matches the value obtained in the first method to the 2 significant figures of the table	Which matches the value obtained in the first method to the 2 significant figures of the table	Manual Page 2-12

Note: Intermediate values are not interpolated from the table because the precision of the table is two significant digits. Additionally, the design story drift limit used in Step 2 does not necessarily need to be the same as other strength or serviceability drift limits used during the analysis and design of the structure.

Step 4. Multiply all the forces and moment from the first-order analysis by the value obtained from the table.

Step 5. Since the selection is in the shaded area of the chart, ($B_2 \leq 1.1$). For LRFD design, use *k* = 1.0. Furthermore, since the selection is in the unshaded area of the chart of ASD design ($B_2 >$ 1.1), the effective length factor, *K*, must be determined through analysis. From previous analysis, use an effective length of 18.0 ft.

LRFD	ASD	
$M_r = B_2(M_{nt} + M_{lt})$ $= 1.1(0\text{ kip-ft} + 233\text{ kip-ft}) = 256\text{ kip-ft}$	$M_r = B_2(M_{nt} + M_{lt})$ $= 1.2(0\text{ kip-ft} + 122\text{ kip-ft}) = 146\text{ kip-ft}$	User Note *Spec.* Sect C2-1b
$P_r = B_2(P_{nt} + P_{lt})$ $= 1.1(317\text{ kips} + 0.0\text{ kips}) = 349\text{ kips}$	$P_r = B_2(P_{nt} + P_{lt})$ $= 1.2(295\text{ kips} + 0.0\text{ kips}) = 354\text{ kips}$	
For $\dfrac{P_r}{P_c} = \dfrac{349\text{ kips}}{1{,}040\text{ kips}} = 0.336 \geq 0.2$ where	For $\dfrac{P_r}{P_c} = \dfrac{354\text{ kips}}{625\text{ kips}} = 0.566 \geq 0.2$	*Spec.* Sect H1.1
$P_c = 1{,}040\text{ kips}$ (W14×90 @ KL =13.5 ft)	$P_c = 625\text{ kips}$ (W14×90 @ KL = 18.0 ft)	Manual Table 4-1
$M_{cx} = 573\text{ kip-ft}$ (W14×90 with L_b = 13.5 ft)	$M_{cx} = 382\text{ kip-ft}$ (W14×90 with L_b = 13.5 ft)	Manual Table 3-2
$\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.336 + \left(\dfrac{8}{9}\right)\left(\dfrac{256\text{ kip-ft}}{573\text{ kip-ft}}\right) \leq 1.0$ $0.733 \leq 1.0$ **o.k.**	$\dfrac{P_r}{P_c} + \left(\dfrac{8}{9}\right)\left(\dfrac{M_{rx}}{M_{cx}} + \dfrac{M_{ry}}{M_{cy}}\right) \leq 1.0$ $0.566 + \left(\dfrac{8}{9}\right)\left(\dfrac{146\text{ kip-ft}}{382\text{ kip-ft}}\right) \leq 1.0$ $0.906 \leq 1.0$ **o.k.**	Eqn. H1-1a

METHOD 3. DIRECT ANALYSIS METHOD

Design for stability by the direct analysis method is found in Appendix 7 of the Specification. This method requires that both the flexural and axial stiffness are reduced and that 0.2% notional lateral loads are applied in the analysis to account for geometric imperfections and inelasticity. Any general second-order analysis method that considers both $P-\delta$ and $P-\Delta$ effects is permitted. The amplified first-order analysis method of section C2 is also permitted provided that the B1 and B2 factors are based on the reduced flexural and axial stiffnesses. A summary of the axial loads, moments and 1st floor drifts from first-order analysis is shown below:

Based on the previous analysis, B_2 (based on the unreduced stiffnesses) is less than 1.5. Therefore, the notional loads were applied as minimum lateral loads for the gravity-only load combinations and not in combination with other lateral loads. Appendix 7 7.3(2)

LRFD	ASD
$1.23D \pm 1.0Q_E + 0.5L + 0.2S$ (Controls columns and beams) For a 1st order analysis with notional loads and reduced stiffnesses: For Interior Column Design: P_u = 317 kips M_{1u} = 148 kip-ft (from first-order analysis) M_{2u} = 233 kip-ft (from first-order analysis) First-floor drift with reduced stiffnesses = 0.718 in.	$1.01D + 0.75(0.7Q_E) + 0.75L + 0.75(S)$ (Controls columns and beams) For a 1st order analysis with notional loads and reduced stiffnesses: For Interior Column Design: P_a = 295 kips M_{1a} = 77.9 kip-ft M_{2a} = 122 kip-ft First-floor drift with reduced stiffnesses = 0.377 in.

Note: For ASD, the second-order analysis should be carried out under 1.6 times the ASD load combinations and the results should be divided by 1.6 to obtain the required strengths. For this example, second order analysis by the amplified first order analysis method is used. The amplified first order analysis method incorporates the 1.6 multiplier directly in the $B1$ and $B2$ amplifiers, such that no other modification is needed.

The required second-order flexural strength, M_r, and axial strength, P_r, are as follows:
For typical interior columns the gravity-load moments are approximately balanced, therefore, M_{nt} = 0.0 kip-ft

LRFD	ASD	
$M_r = B_1M_{nt} + B_2M_{lt}$ *Determine B_1* P_r = required second-order axial strength using LRFD or ASD load combinations, kips. Note that for members subject to axial compression, B_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$. Therefore, P_r = 317 kips (from previous calculations) and	$M_r = B_1M_{nt} + B_2M_{lt}$ *Determine B_1* P_r = required second-order axial strength using LRFD or ASD load combinations, kips. Note that for members subject to axial compression, B_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$. Therefore, P_r = 295 kips (from previous calculations) and	Eqn. C2-1a Specification Section C2.16

LRFD	ASD	
$I = 999\text{ in.}^4$ (W14×90)	$I = 999\text{ in.}^4$ (W14×90)	
$P_{e1} = \dfrac{\pi^2 EI^*}{(K_1L)^2}$ $= \dfrac{\pi^2(0.8)(29{,}000\text{ ksi})(999\text{ in}^4)}{[(1.0)(13.5\text{ ft})(12\text{ in./ft})]^2}$ $= 8{,}720\text{ kips}$	$P_{e1} = \dfrac{\pi^2 EI^*}{(K_1L)^2}$ $= \dfrac{\pi^2(0.8)(29{,}000\text{ ksi})(999\text{ in}^4)}{[(1.0)(13.5\text{ ft})(12\text{ in./ft})]^2}$ $= 8{,}720\text{ kips}$	Eqn. C2-5
$C_m = 0.6 - 0.4(M_1 / M_2)$ $= 0.6 - 0.4$ (148 kip-ft / 233 kip-ft) $= 0.346$	$C_m = 0.6 - 0.4(M_1 / M_2)$ $= 0.6 - 0.4$ (77.9 kip-ft / 122 kip-ft) $= 0.345$	Eqn. C2-4
$\alpha = 1.0$,	$\alpha = 1.6$,	
$B_1 = \dfrac{C_m}{1-\left(\dfrac{\alpha P_r}{P_{e1}}\right)} \ge 1$ $= \dfrac{0.346}{1-\left[\dfrac{(1.00)(317\text{ kips})}{8{,}720\text{ kips}}\right]} \ge 1$ $= 0.359 \ge 1$; Use 1.0	$B_1 = \dfrac{C_m}{1-\left(\dfrac{\alpha P_r}{P_{e1}}\right)} \ge 1$ $= \dfrac{0.345}{1-\left[\dfrac{(1.60)(295\text{ kips})}{8{,}720\text{ kips}}\right]} \ge 1$ $= 0.365 \ge 1$; Use 1.0	Eqn. C2-2
Calculate B_2 $B_2 = \dfrac{1}{1-\left(\dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}\right)} \ge 1$ where:	*Calculate B_2* $B_2 = \dfrac{1}{1-\left(\dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}\right)} \ge 1$ where:	Eqn. C2-3
$\alpha = 1.0$,	$\alpha = 1.6$,	
$\sum P_{nt} = 5{,}440$ kips (from computer output) $\sum P_{e2}$ may be taken as $= R_M \dfrac{\sum HL}{\Delta_H}$ where R_M is taken as 0.85 for moment frames	$\sum P_{nt} = 5{,}120$ kips (from computer output) $\sum P_{e2}$ may be taken as $= R_M \dfrac{\sum HL}{\Delta_H}$ where R_M is taken as 0.85 for moment frames	Specification Section C2.6b
$\sum H = 1.23D + 1.0Q_E + 0.5L + 0.2S$ $= 196$ kips (Horizontal) (from previous seismic force distribution calculations)	$\sum H = 1.01D + 0.75(0.7Q_E) + 0.75L + 0.75S$ $= 103$ kips (Horizontal) (from previous seismic force distribution calculations)	
$\Delta_H = 0.718$ in. (from computer output)	$\Delta_H = 0.377$ in. (from computer output)	
$\sum P_{e2} = 0.85\dfrac{(196\text{ kips})(13.5\text{ ft})(12\text{ in./ft})}{0.718\text{ in.}}$ $= 37{,}600$ kips	$\sum P_{e2} = 0.85\dfrac{(103\text{ kips})(13.5\text{ ft})(12\text{ in./ft})}{0.377\text{ in.}}$ $= 37{,}600$ kips	Eqn. C2-6b

LRFD	ASD
$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$ $= \dfrac{1}{1-\left[\dfrac{(1.0)(5{,}440 \text{ kips})}{37{,}600 \text{ kips}}\right]} \geq 1$ $= 1.17 \geq 1$	$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha \sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$ $= \dfrac{1}{1-\left[\dfrac{(1.6)(5{,}120 \text{ kips})}{37{,}600 \text{ kips}}\right]} \geq 1$ $= 1.28 \geq 1$
Calculate amplified moment	*Calculate amplified moment*
$M_r = (1.0)(0.0 \text{ kip-ft}) + (1.17)(233 \text{ kip-ft})$ $= 273$ kip-ft	$M_r = (1.0)(0.0 \text{ kip-ft}) + (1.28)(122 \text{ kip-ft})$ $= 156$ kip-ft
Calculate amplified axial load	*Calculate amplified axial load*
$P_{nt} = 317$ kips (from computer analysis)	$P_{nt} = 295$ kips (from computer analysis)
For a long frame, such as this one, the change in load to the interior columns associated with lateral load is negligible.	For a long frame, such as this one, the change in load to the interior columns associated with lateral load is negligible.
$P_r = P_{nt} + B_2 P_{lt}$ $= 317 \text{ kips} + (1.17)(0.0 \text{ kips})$ $= 317$ kips	$P_r = P_{nt} + B_2 P_{lt}$ $= 295 \text{ kips} + (1.28)(0.0 \text{ kips})$ $= 295$ kips
The flexural and axial stiffness of all members in the moment frame was reduced using 0.8E in the computer analysis.	The flexural and axial stiffness of all members in the moment frame was reduced using 0.8E in the computer analysis.
Check that the flexural stiffness was adequately reduced for the analysis.	Check that the flexural stiffness was adequately reduced for the analysis.
$EI^* = 0.8\tau_b EI$	$EI^* = 0.8\tau_b EI$
$\alpha = 1.0$	$\alpha = 1.6$
$P_r = 317$ kips	$P_r = 295$ kips
$P_y = AF_y = 26.5 \text{ in.}^2 (50.0 \text{ ksi}) = 1{,}330$ kips (W14×90 column)	$P_y = AF_y = 26.5 \text{ in.}^2 (50.0 \text{ ksi}) = 1{,}330$ kips (W14×90 column)
$\dfrac{\alpha P_r}{P_y} = \dfrac{1.0(317 \text{ kips})}{1{,}330 \text{ kips}} = 0.238 \leq 0.5$	$\dfrac{\alpha P_r}{P_y} = \dfrac{1.6(29.5 \text{ kips})}{1{,}330 \text{ kips}} = 0.355 \leq 0.5$
Therefore, $\tau_b = 1.0$ **o.k.**	Therefore, $\tau_b = 1.0$ **o.k.**
Note: By inspection $\tau_b = 1.0$ for all of the	Note: By inspection $\tau_b = 1.0$ for all of the

Eqn. C2-3

Eqn. A-7-2

Appendix 7
Section 7.3(3)

LRFD	ASD
beams in the moment frame.	beams in the moment frame.
For this method, K=1.0.	For this method, K=1.0
$P_c = 1{,}040$ kips (W14×90 @ KL = 13.5 ft)	$P_c = 689$ kips (W14×90 @ KL = 13.5 ft)
$\frac{P_r}{P_c} = \frac{317\text{ kips}}{1{,}040\text{ kips}} = 0.305 \geq 0.2$	$\frac{P_r}{P_c} = \frac{295\text{ kips}}{689\text{ kips}} = 0.428 \geq 0.2$
$M_{cx} = 573$ kip-ft (W14×90 with L_b = 13.5 ft)	$M_{cx} = 382$ kip-ft (W14×90 with L_b = 13.5 ft)
$\frac{P_r}{P_c} + \left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$	$\frac{P_r}{P_c} + \left(\frac{8}{9}\right)\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$
$0.305 + \left(\frac{8}{9}\right)\left(\frac{273\text{ kip-ft}}{573\text{ kip-ft}}\right) \leq 1.0$	$0.428 + \left(\frac{8}{9}\right)\left(\frac{156\text{ kip-ft}}{382\text{ kip-ft}}\right) \leq 1.0$
$0.729 \leq 1.0$ **o.k.**	$0.791 \leq 1.0$ **o.k.**

Manual Table 4-1

Specification Section H1.1

Manual Table 3-2

Eqn. H1-1a

BEAM ANALYSIS IN THE MOMENT FRAME

The controlling load combinations for the beams in the frames are shown below and evaluated for the second floor beam. The dead load, live load and seismic moments were taken from a computer analysis.

1st – 2nd	**LRFD Combination**	**ASD Combination 1**	**ASD Combination 2**
	$1.23D + 1.0Q_E + 0.5L + 0.2S$	$1.02D + 0.7Q_E$	$1.01D + 0.75(0.7Q_E) + 0.75L + 0.75S$
$\sum H$	196 kips	137 kips	103 kips
L	13.5 ft	13.5 ft	13.5 ft
Δ_H	0.575 in.	0.402 in.	0.302 in.
$\sum P_{e2}$	46,900 kips	46,900 kips	47,000 kips
$\sum P_{nt}$	5,440 kips	3,920 kips	5,120 kips
B_2	1.13	1.15	1.21
2nd – 3rd	**LRFD Combination**	**ASD Combination 1**	**ASD Combination 2**
	$1.23D + 1.0Q_E + 0.5L +0.2S$	$1.02D + 0.7Q_E$	$1.01D + 0.75(0.7Q_E) + 0.75L + 0.75S$
$\sum H$	170 kips	119 kips	89.3 kips
L	13.5 ft	13.5 ft	13.5 ft
Δ_H	0.728 in.	0.509 in.	0.382 in.
$\sum P_{e2}$	32,200 kips	32,200 kips	32,200 kips
$\sum P_{nt}$	3,840 kips	2,770 kips	3.660 kips
B_2	1.14	1.16	1.22

For beam members, the larger of the B_2 values from the story above or below is used.

LRFD Combination

$$B_2M_{lt} = 1.14(154\text{ kip-ft}) = 176\text{ kip-ft}$$

$$M_u = \begin{bmatrix} 1.23(153\text{ kip-ft}) \\ +1.0(176\text{ kip-ft}) \\ +0.5(80.6\text{ kip-ft}) \end{bmatrix} = \begin{bmatrix} 188\text{ kip-ft} \\ +176\text{ kip-ft} \\ +40.3\text{ kip-ft} \end{bmatrix} = 404\text{ kip-ft}$$

ASD Combination 1

$$B_2M_{lt} = 1.16(154\text{ kip-ft}) = 179\text{ kip-ft}$$

$$M_a = \begin{bmatrix} 1.02(153\text{ kip-ft}) \\ +0.7(179\text{ kip-ft}) \end{bmatrix} = 156\text{ kip-ft} + 125\text{ kip-ft} = 281\text{ kip-ft}$$

ASD Combination 2

$$B_2M_{lt} = 1.22(154\text{ kip-ft}) = 188\text{ kip-ft}$$

$$M_a = \begin{bmatrix} 1.01(153\text{ kip-ft}) \\ +0.525(188\text{ kip-ft}) \\ +0.75(80.6\text{ kip-ft}) \end{bmatrix} = \begin{bmatrix} 155\text{ kip-ft} \\ +98.7\text{ kip-ft} \\ +60.5\text{ kip-ft} \end{bmatrix} = 314\text{ kip-ft}$$

LRFD	ASD
Calculate C_b for compression in the bottom flange braced at 10.0 ft o.c. C_b = 1.86 (from computer output)	Calculate C_b for compression in the bottom flange braced at 10.0 ft o.c. $D + 0.7Q_E$ C_b = 1.86 (from computer output) $D + 0.75(0.7Q_E) + 0.75L$

Check W24×55 With continuous bracing $\phi M_n = 503$ kip-ft **o.k.** For $L_b = 10.0$ ft and $C_b = 1.86$ $\phi M_n = (386 \text{ kip-ft})1.86 \leq 503$ kip-ft $= 718$ kip-ft ≤ 503 kip-ft 503 kip-ft > 404 kip-ft **o.k.** A W24×55 has an available end shear of 251 kips and an I_x of 1350 in.^{4}	$C_b = 2.01$ (from computer output) Check W24×55 *D + 0.7E* with continuous bracing $\dfrac{M_n}{\Omega} = 334 \text{ kip-ft} > 113 \text{ kip-ft}$ **o.k.** For $L_b = 10.0$ ft and $C_b = 1.89$ $\dfrac{M_n}{\Omega} = (256 \text{ kip-ft})1.85 \leq 334 \text{ kip-ft}$ $= 474$ kip-ft ≤ 334 kip-ft 334 kip-ft > 281 kip-ft **o.k.** $D + 0.75(0.7Q_E) + 0.75L$ With continuous bracing $\dfrac{M_n}{\Omega} = 334 \text{ kip-ft} > 133 \text{ kip-ft}$ **o.k.** For $L_b = 10$ ft and $C_b = 2.06$ $\dfrac{M_n}{\Omega} = (256 \text{ kip-ft})2.01$ $= 515$ kip-ft ≤ 334 kip-ft 334 kip-ft > 314 kip-ft **o.k.** A W24×55 has an available end shear of 167 kips and an I_x of 1350 in.^{4}	Manual Table 3-2 Manual Table 3-2

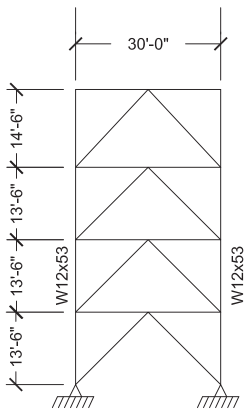
The moments and shears on the roof beams due to the lateral loads were also checked but do not control the design.

The connections of these beams can be designed by one of the techniques illustrated in the Chapter IIB of the design examples.

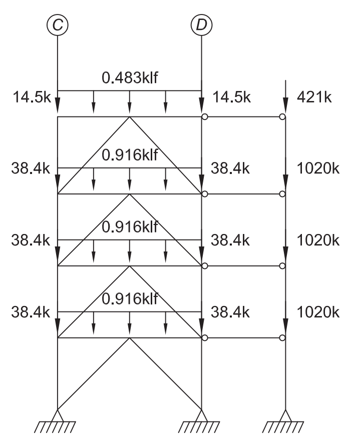
BRACED FRAME ANALYSIS

The braced frames at Grids 1 and 8 were analyzed for their lateral loads. The stability design requirements from Chapter C were applied to this system.

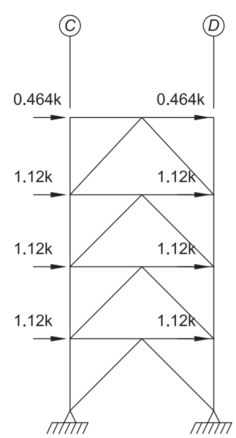
The model layout, nominal dead, live, and snow loads with associated notional loads, wind loads and seismic loads are shown in the figures below:



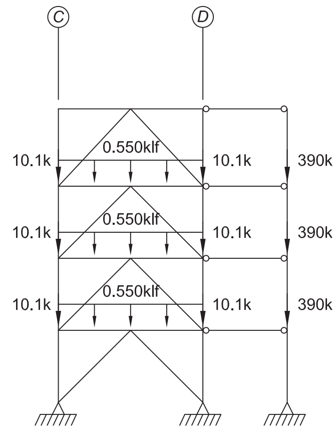
BRACED FRAME LAYOUT (GRID 1 & 8)



NOMINAL DEAD LOADS LEANING COLUMNS

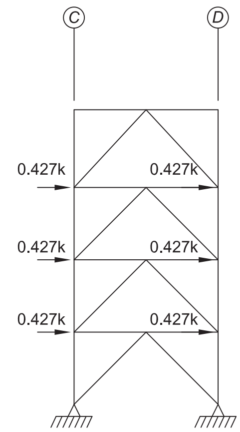


NOTIONAL DEAD LOADS

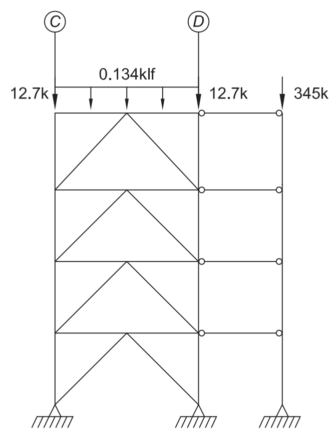


NOMINAL LIVE LOADS

LEANING COLUMNS

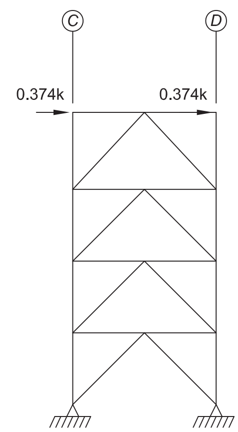


NOTIONAL LIVE LOADS

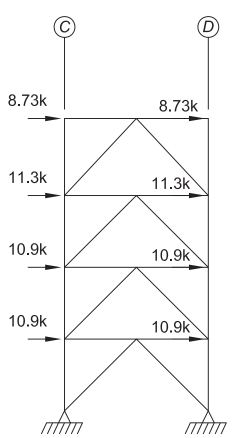


NOMINAL SNOW LOADS

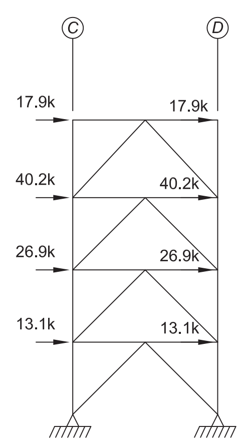
LEANING COLUMNS



NOTIONAL SNOW LOADS



NOMINAL WIND LOADS



SEISMIC LOADS (1.0 Q_E)

Second-order analysis by amplified first-order analysis Specification C2.1b

The following is a method to account for second-order effects in frames by amplifying the axial forces and moments in members and connections from a first-order analysis.

A first-order frame analysis is run using the load combinations for LRFD and ASD. From this analysis the critical axial loads, moments, and deflections are obtained.

The required second-order flexural strengths, M_r, and axial strengths, P_r, are as follows:

LRFD	ASD
$\sum H = 1.23D + 1.0Q_E + 0.5L + 0.2S$ $= 196$ kips (from previous calculations)	$\sum H = 1.01D + 0.75Q_E + 0.75L + 0.75S$ $= 103$ kips (from previous calculations)
$\Delta_H = 0.211$ in. (from computer output)	$\Delta_H = 0.111$ in. (from computer output)
$\sum P_{e2} = R_M \dfrac{\sum HL}{\Delta_H}$ $= 1.0\dfrac{(196 \text{ kips})(13.5 \text{ ft})(12 \text{ in./ft})}{0.211 \text{ in.}}$ $= 150{,}000$ kips	$\sum P_{e2} = R_M \dfrac{\sum HL}{\Delta_H}$ $= 1.0\dfrac{(103 \text{ kips})(13.5 \text{ ft})(12 \text{ in./ft})}{0.111 \text{ in.}}$ $= 150{,}000$ kips
$\sum P_{nt} = 5{,}440$ kips (from computer output)	$\sum P_{nt} = 5{,}120$ kips (from computer output)
$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha\sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$ $= \dfrac{1}{1-\left(\dfrac{(1.0)(5{,}440 \text{ kips})}{150{,}000 \text{ kips}}\right)} \geq 1$ $= 1.04 \geq 1$	$B_2 = \dfrac{1}{1-\left(\dfrac{\alpha\sum P_{nt}}{\sum P_{e2}}\right)} \geq 1$ $= \dfrac{1}{1-\left(\dfrac{(1.6)(5{,}120 \text{ kips})}{150{,}000 \text{ kips}}\right)} \geq 1$ $= 1.06 \geq 1$
$P_r = P_{nt} + B_2P_{lt}$ $= 249$ kips $+ (1.04)(146$ kips$)$ $= 401$ kips	$P_r = P_{nt} + B_2P_{lt}$ $= 232$ kips $+ (1.06)(76.7$ kips$)$ $= 313$ kips
$P_c = 513$ kips (W12×53)	$P_c = 341$ kips (W12×53)
For $\dfrac{P_r}{P_c} = \dfrac{401 \text{ kips}}{513 \text{ kips}} = 0.782 \leq 1.0$	For $\dfrac{P_r}{P_c} = \dfrac{313 \text{ kips}}{341 \text{ kips}} = 0.918 \leq 1.0$

Eqn. C2-6b

Eqn. C2-3

Eqn. C2-1b

Manual Table 4-1

Note: Notice that the lower sidesway displacements of the braced frame produce much lower values for B_2. Similar results could be expected for the other two methods of analysis.

Although not presented here, second order effects should be accounted for in the design of the beams and diagonal braces in the braced frames at Grids 1 and 8.

ANALYSIS OF DRAG STRUTS

The fourth floor has the highest force to the ends of the building at E = 80.3 kips (from previous calculations). The beams at the end of the building which span 22.5 ft are W18×35

The loads for these edge beams with a DL of 75.0 psf (5.50 ft) and exterior wall at 0.503 kip/ft and a LL of 80.0 psf (5.50 ft) are DL_{tot} = 0.916 kip/ft, LL_{tot} = 0.440 kip/ft.

The controlling load combinations are LRFD ($1.23D + 1.0Q_E + 0.50L$) and ASD ($1.01D + 0.75(0.7Q_E) + 0.75L$) or ($1.02D + 0.7Q_E$)

LRFD	ASD
M_u = 1.23(58.0 kip-ft) + 0.50(27.8 kip-ft) = 85.2 kip-ft	M_a = 1.01(58.0 kip-ft) + 0.75(27.8 kip-ft) = 79.4 kip-ft or M_a = 1.02(58.0 kip-ft) = 59.2 kip-ft
Load from the diaphragm shear F_p = 80.3 kips	Load from the diaphragm shear F_p = 0.75(0.70)(80.3 kips) = 42.2 kips or F_p = 0.70(80.3 kips) = 56.2 kips
Load to the drag struts	Load to the drag struts

Only the two 45 ft long segments on either side of the brace can transfer load into the brace, because the stair opening is in front of the brace.

LRFD	ASD
$V = \dfrac{80.3\text{ kips}}{2}(45.0\text{ ft}) = 0.892\text{ kip/ft}$	$V = \dfrac{42.2\text{ kips}}{2}(45.0\text{ ft}) = 0.469\text{ kip/ft}$ or $V = \dfrac{56.2\text{ kips}}{2}(45.0\text{ ft}) = 0.624\text{ kip/ft}$
The top flange stress due to bending f_b = 85.2 kip-ft(12 in./ft) / 57.6 in^3 = 17.8 ksi	The top flange stress due to bending f_b = 79.4 kip-ft(12 in./ft) / 57.6 in^3 = 16.5 ksi or f_b = 59.2 kip-ft(12 in./ft) / 57.6 in^3 = 12.3 ksi

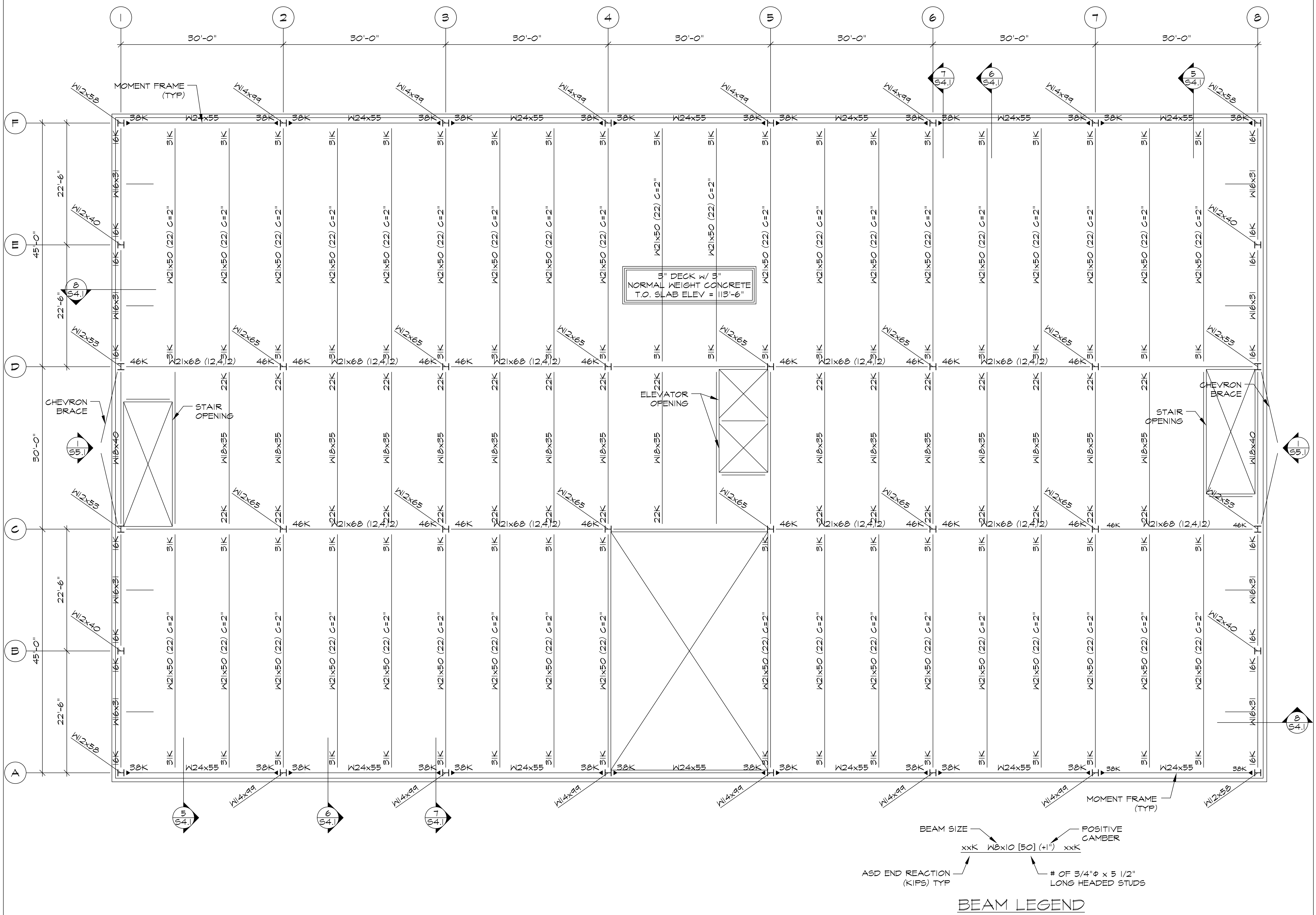
Note: It is often possible to resist the drag strut force using the slab directly. For illustration purposes, this solution will instead use the beam to resist the force independently of the slab. The full cross-section can be used to resist the force if the member is designed as a column braced at one flange only (plus any other intermediate bracing present, such as from filler beams). Alternatively, a reduced cross-section consisting of the top flange plus a portion of the web can be used. Arbitrarily use the top flange and 8 times the web thickness as an area to carry the drag strut component.

Area = 6.00 in.(0.425 in.) + 8(0.300 in.)^2 = 2.55 in.^{2} + 0.720 in.^{2} = 3.27 in.^{2}

Ignoring the small segment of the beam between Grid C and D, the stress due to the drag strut force is:

LRFD	ASD
$f_a = \left(\dfrac{90.0\text{ ft}}{2}\right)(0.892\text{ kip/ft})/3.27\text{ in.}^2$ $= 12.3\text{ ksi}$	$f_a = \left(\dfrac{90.0\text{ ft}}{2}\right)(0.469\text{ kip/ft})/3.27\text{ in.}^2$ $= 6.45\text{ ksi}$ or $f_a = 45.0\text{ ft}(0.624\text{ kip/ft})/3.27\text{ in.}^2 = 8.59\text{ ksi}$
Total top flange stress $f_a = 17.8\text{ ksi} + 12.3\text{ ksi}$ $= 30.1\text{ ksi}$	Total top flange stress $f_a = 16.5\text{ ksi} + 6.45\text{ ksi}$ $= 23.0\text{ ksi}$ **controls** or $f_a = 12.3\text{ ksi} + 8.59\text{ ksi}$ $= 20.9\text{ ksi}$
For bending or compression $\phi = 0.90$ $\phi F_y = 0.90(50\text{ ksi}) = 45.0\text{ ksi} > 30.1\text{ ksi}$ **o.k.**	For bending or compression $\Omega = 1.67$ $\dfrac{F_y}{\Omega} = \dfrac{(50\text{ ksi})}{1.67} = 29.9\text{ ksi} > 23.0\text{ ksi}$ **o.k.**

Note: Because the drag strut load is a horizontal load, notations indicating the method of transfer into the strut, and the extra horizontal load which must be accommodated by the beam end connections should be indicated on the drawings.



2ND FLOOR FRAMING PLAN

1/8" = 1'-0"

1 / S2.1

NORTH

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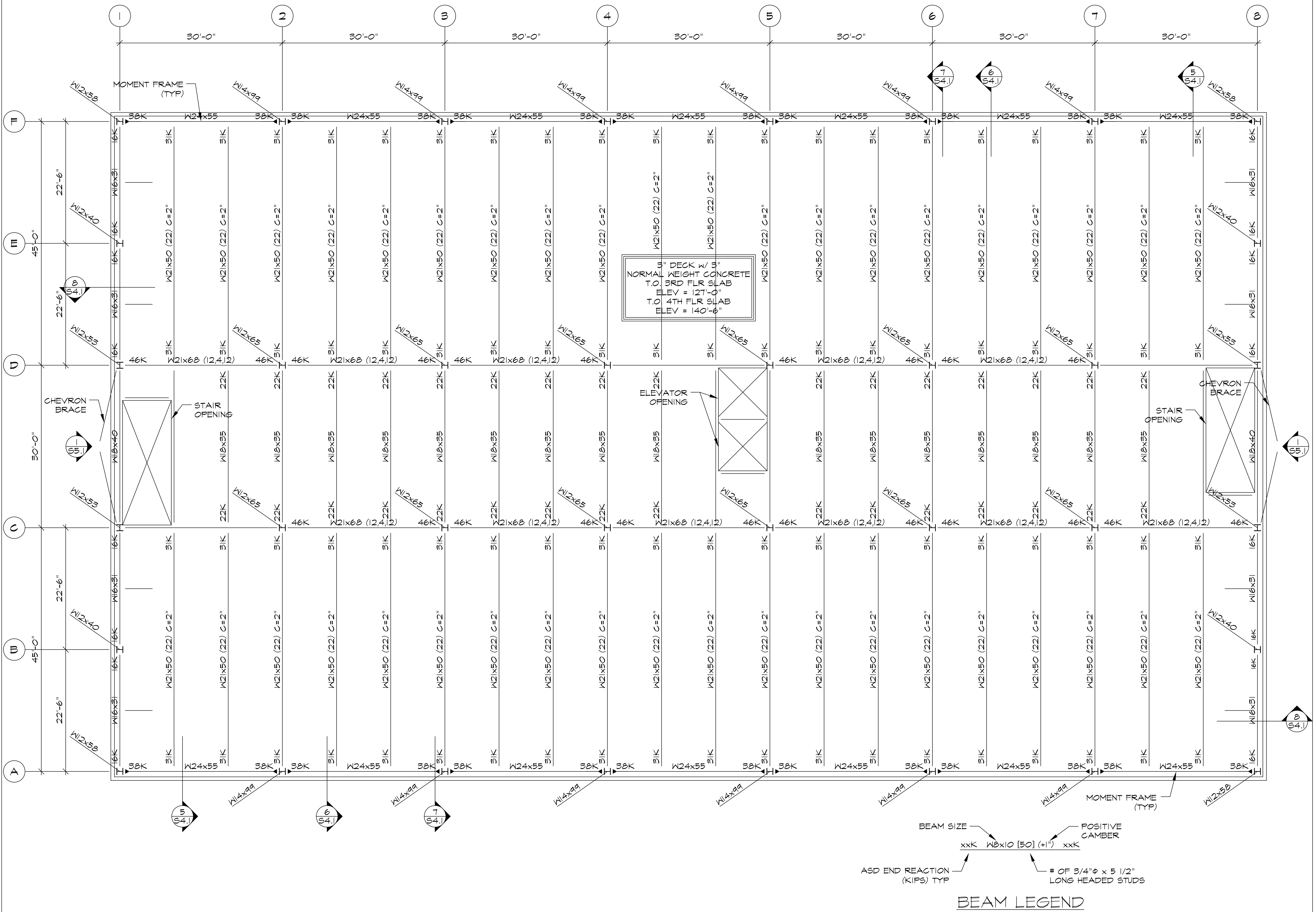
2ND FLOOR FRAMING PLAN

S2.1



3RD & 4TH FLOOR FRAMING PLAN 1/S2.2
1/8" = 1'-0"

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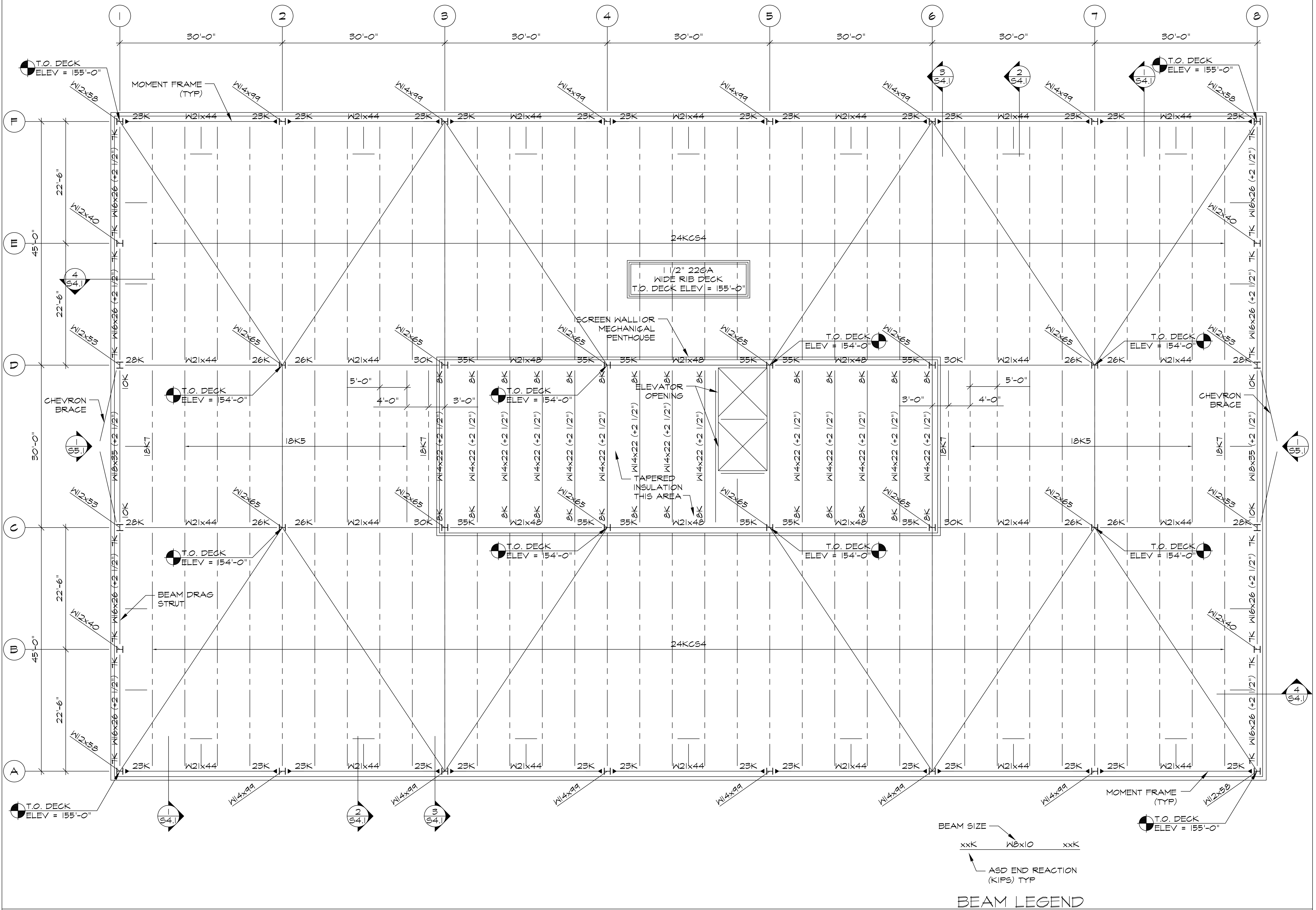
3RD FLOOR FRAMING PLAN

S2.2



ROOF FRAMING PLAN
1/8" = 1'-0"

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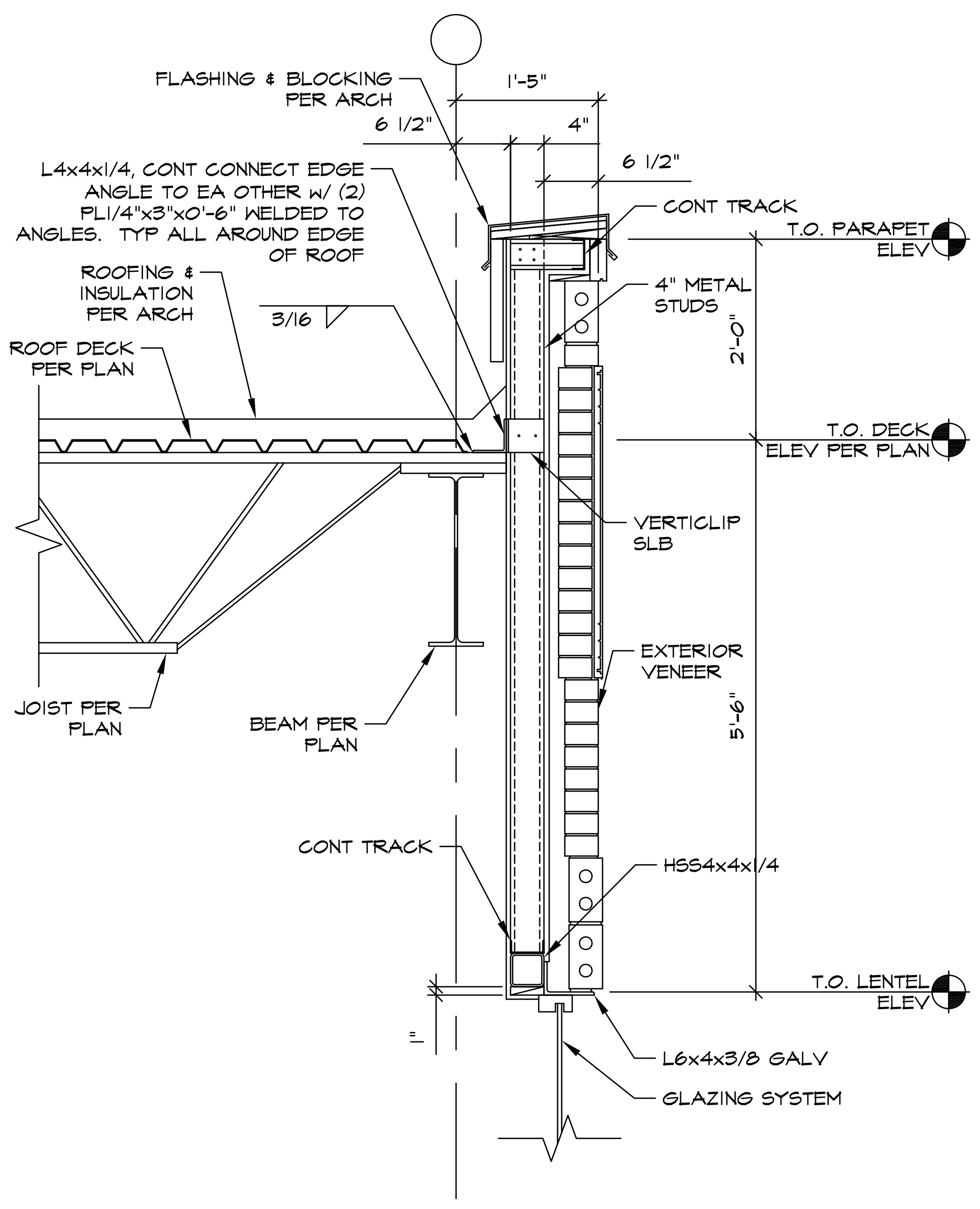
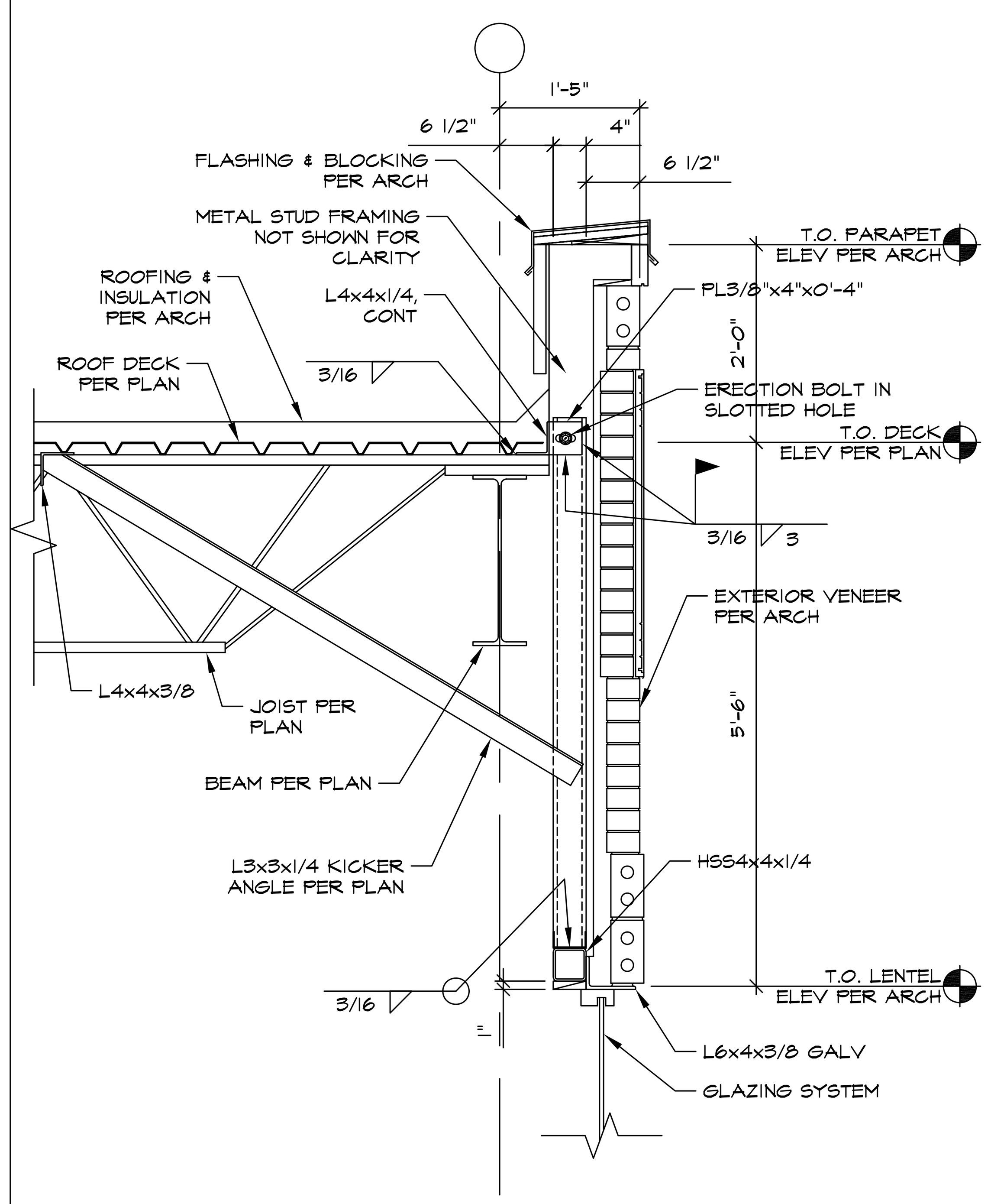
ROOF FRAMING PLAN

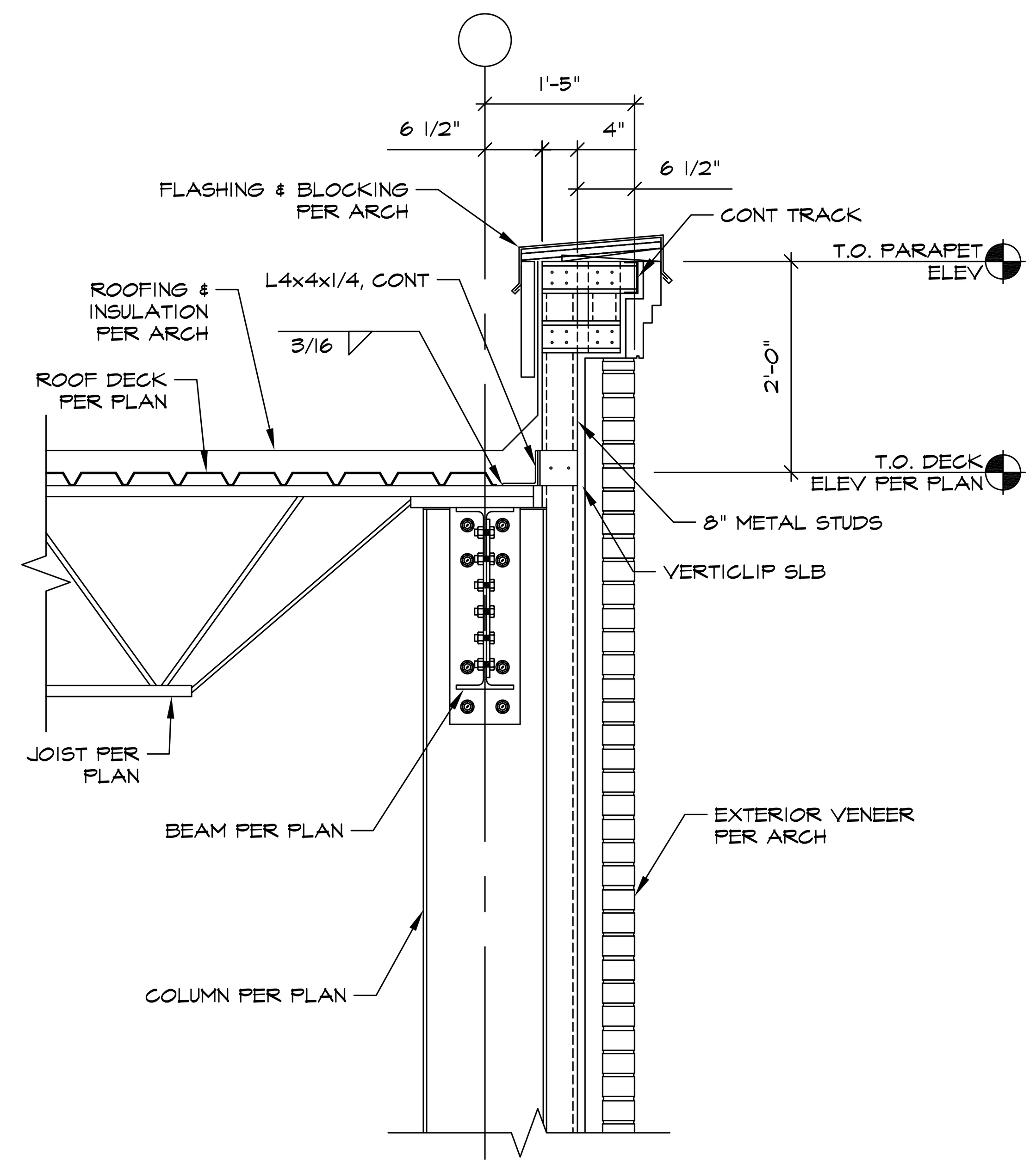
S2.3



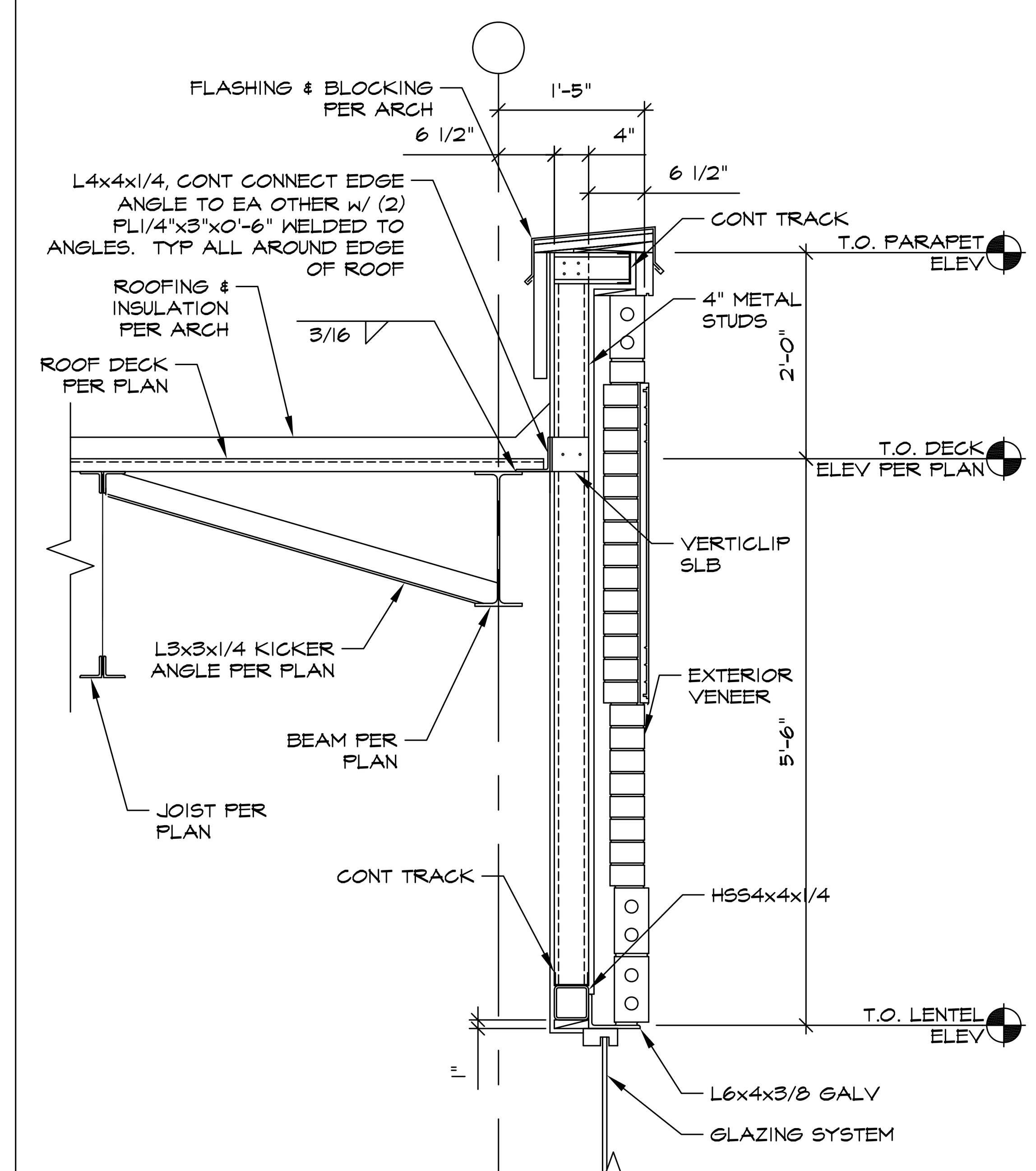
SECTION 1
3/4" = 1'-0"
S4.1



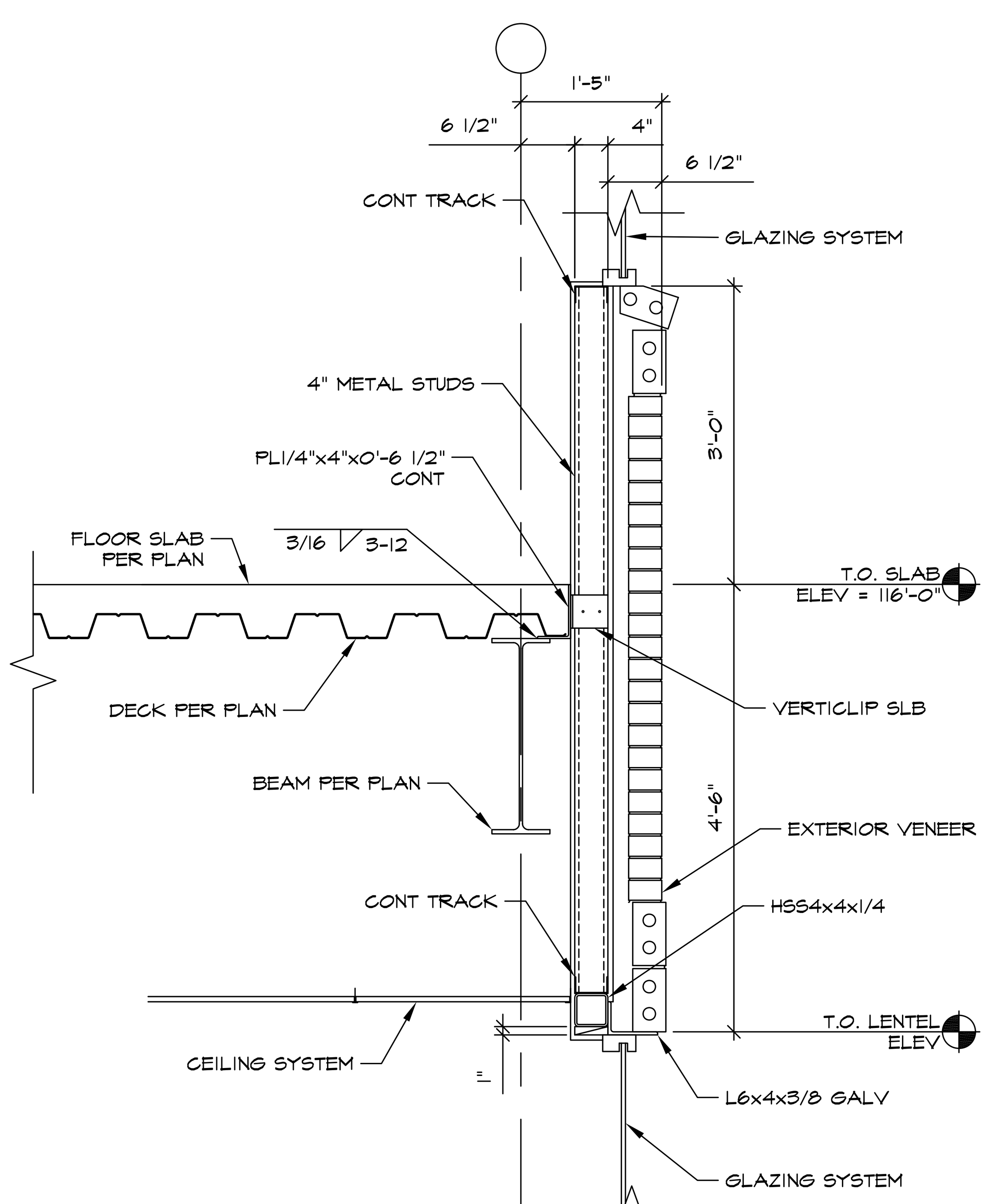
SECTION 2
3/4" = 1'-0"
S4.1



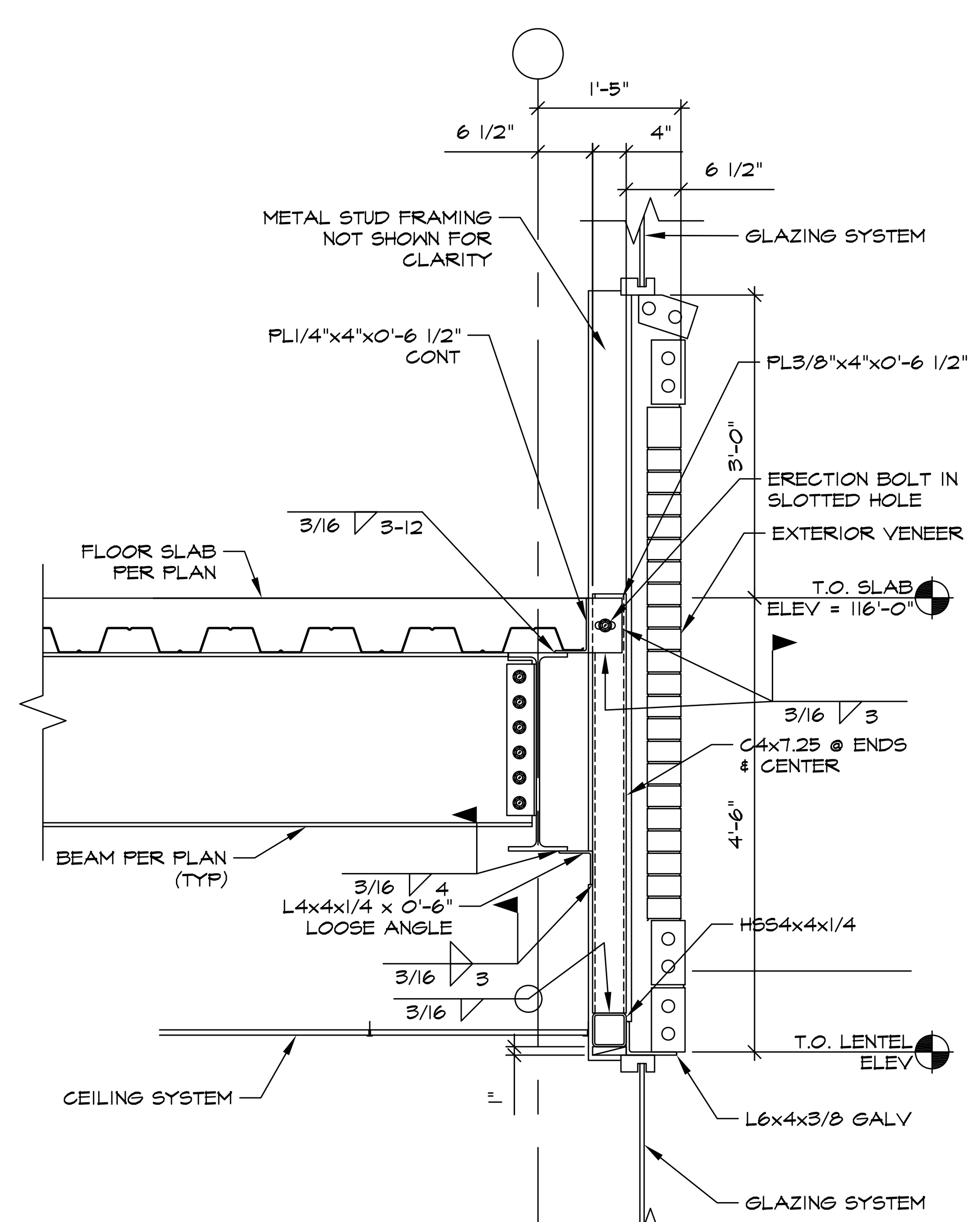
SECTION 3
3/4" = 1'-0"
S4.1



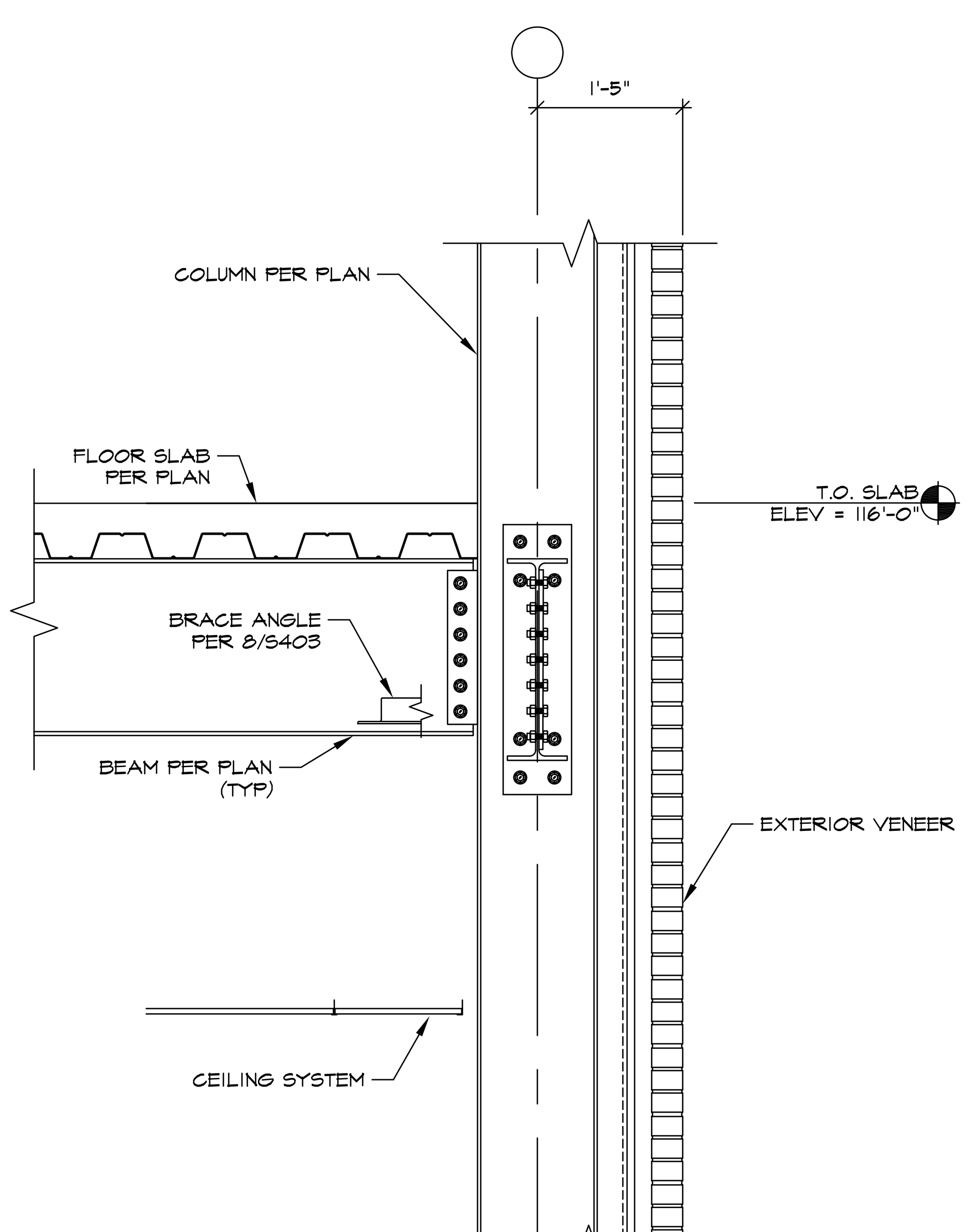
SECTION 4
3/4" = 1'-0"
S4.1



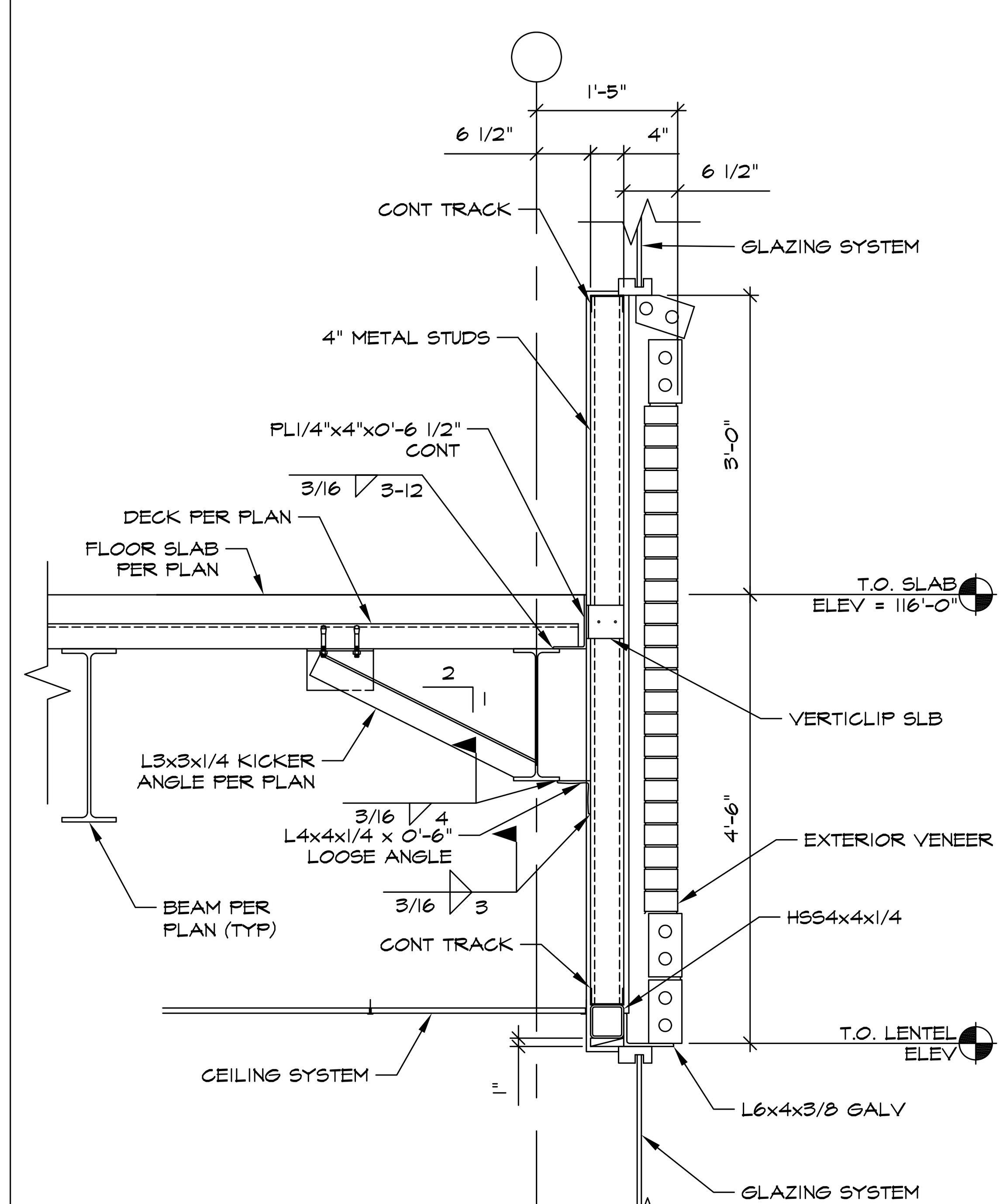
SECTION 5
3/4" = 1'-0"
S4.1



SECTION 6
3/4" = 1'-0"
S4.1



SECTION 7
3/4" = 1'-0"
S4.1



SECTION 8
3/4" = 1'-0"
S4.1

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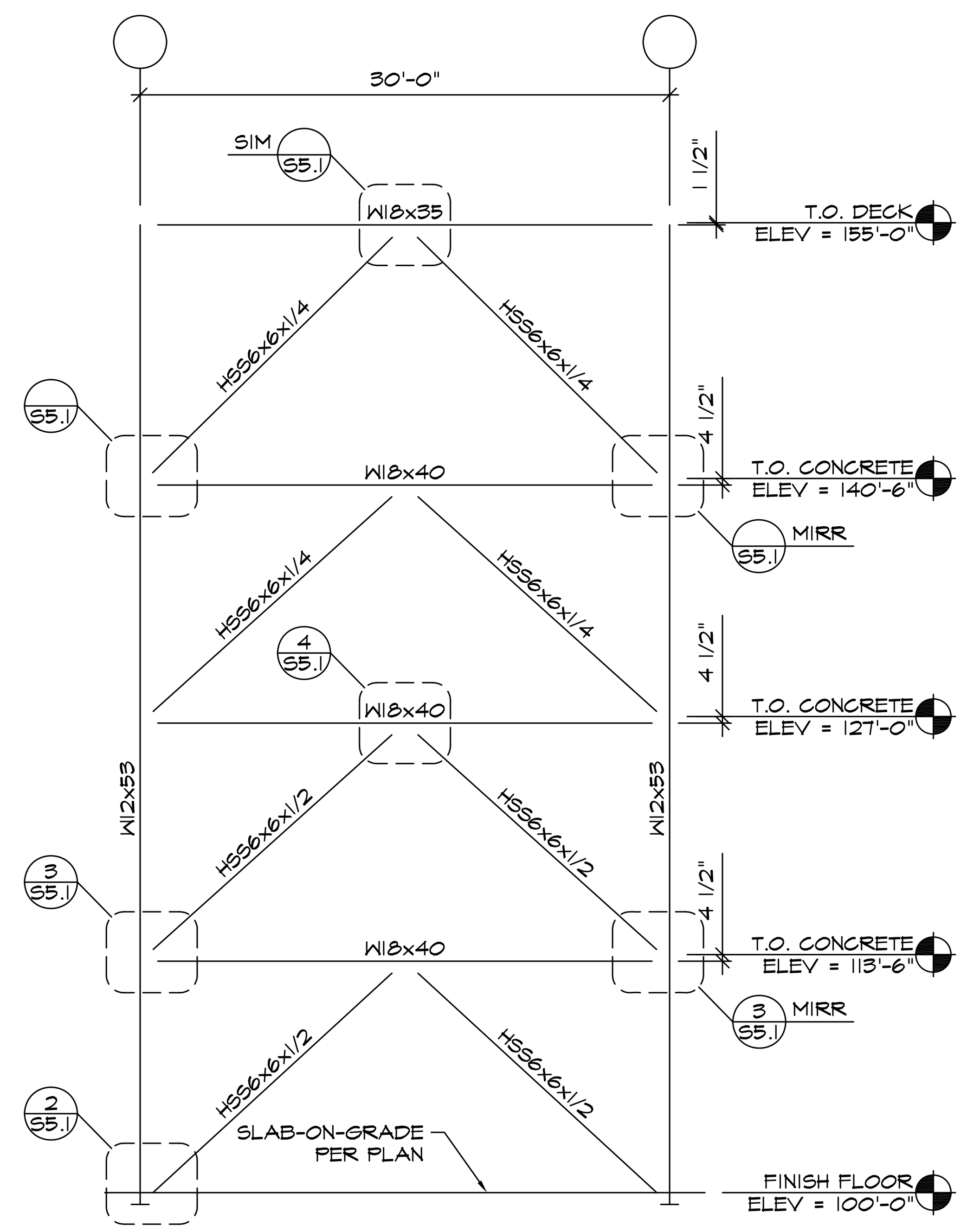
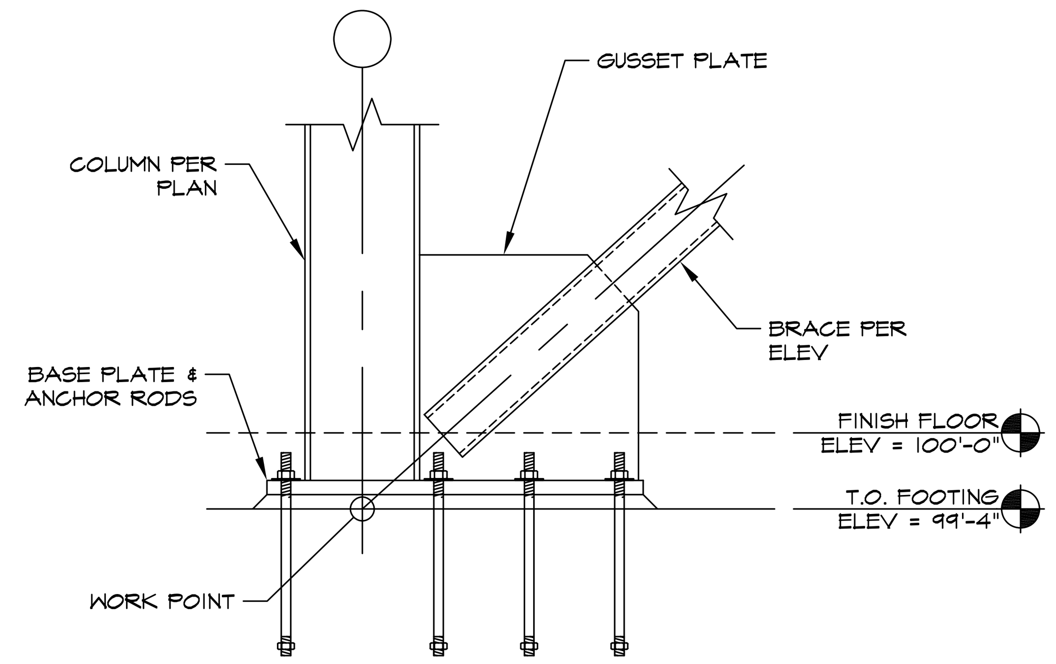
ROOF FRAMING SECTIONS

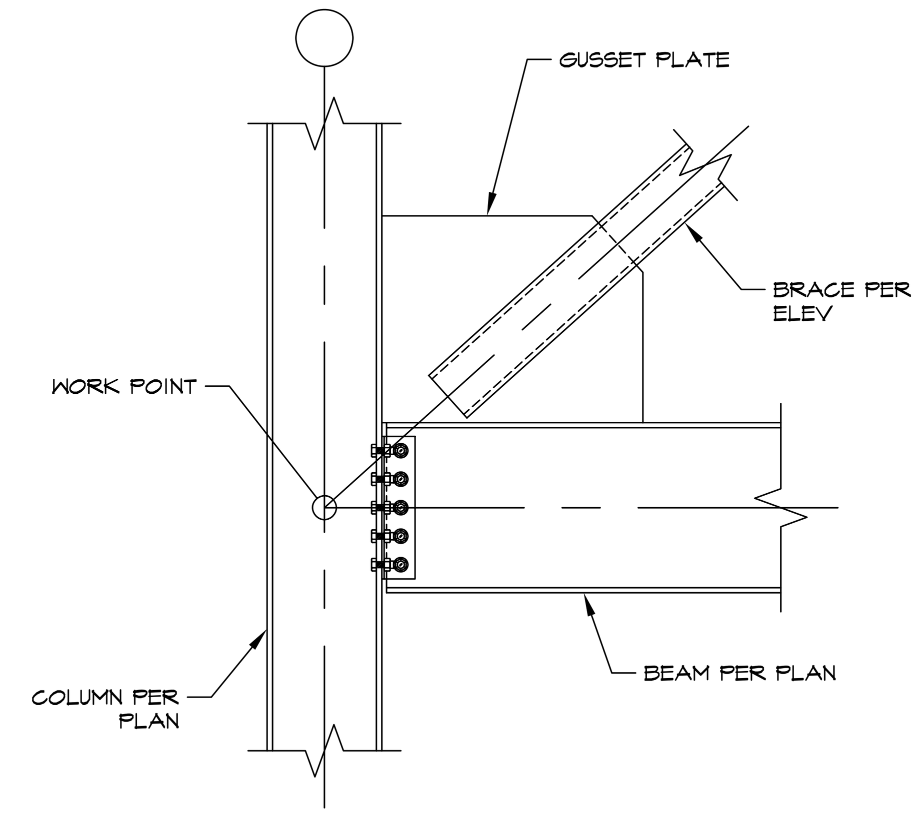
S4.1



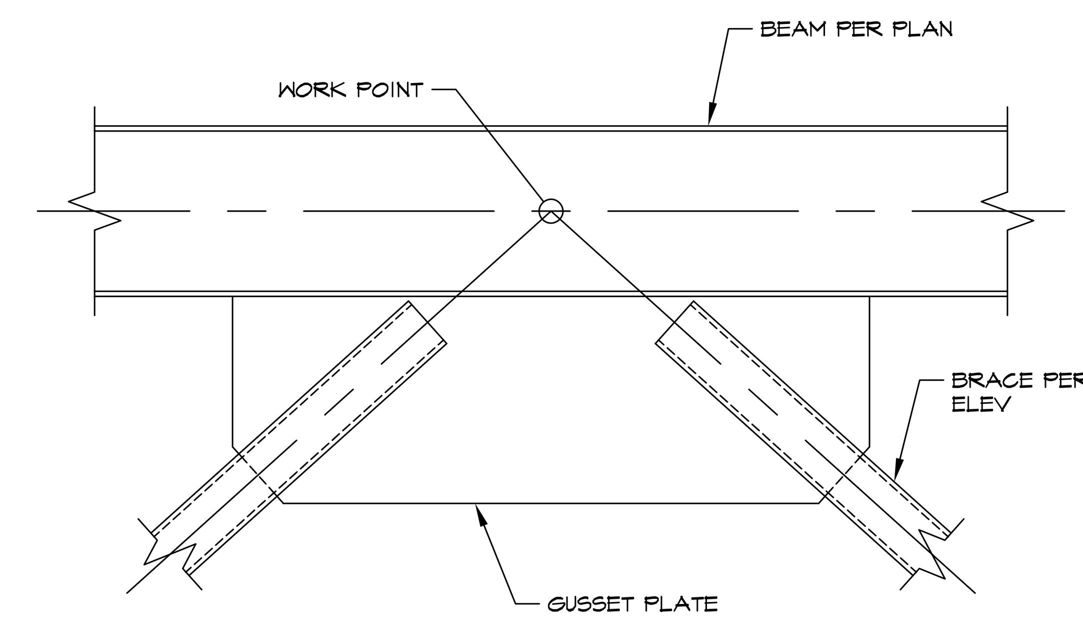
CHEVRON BRACE ELEVATION 1 S5.1
N.T.S.



DETAIL 2 S5.1
1 1/2" = 1'-0"



DETAIL 3 S5.1
1 1/2" = 1'-0"



DETAIL 4 S5.1
1 1/2" = 1'-0"

NOT USED 5 S5.1

NOT USED 6 S5.1

NOT USED 7 S5.1

NOT USED 8 S5.1

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CHEVRON BRACE ELEVATION & DETAILS

S5.1

APPENDIX A

CROSS-REFERENCE LIST FOR THE 2005 AISC *SPECIFICATION FOR STRUCTURAL STEEL BUILDINGS* TO PAST AISC SPECIFICATIONS

The following cross-reference list relates the new 2005 *Specification for Structural Steel Buildings* table of contents to the corresponding sections, where applicable, of past AISC specifications. Cross references are given to the five standards that the new 2005 AISC Specification replaces:

- 1989 *Specification for Structural Steel Buildings-Allowable Stress Design and Plastic Design* (1989 ASD)
- 1989 *Specification for Allowable Stress Design of Single Angle Members* (1989 ASD Single Angle)
- 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (1999 LRFD)
- 2000 *Load and Resistance Factor Design Specification for Steel Hollow Structural Sections* (2000 LRFD HSS)
- 2000 *Load and Resistance Design Specification for Single-Angle Members* (2000 LRFD Single Angle)

CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION

2005 SPECIFICATION	AISC SPECIFICATION REFERENCES: 1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE 1989 ASD	SINGLE ANGLE 2000 LRFD
A. GENERAL PROVISIONS					
A1. Scope	A1	A1	1.1	1	1
1. Low-Seismic Applications	NEW	NEW	NEW	NEW	NEW
2. High-Seismic Applications		↓			
3. Nuclear Applications	↓	A1	↓	↓	↓
A2. Referenced Specifications, Codes and Standards	A6	A6	NEW	NEW	NEW
A3. Material					
1. Structural Steel Materials	A3.1	A3.1	1.2.1	NEW	NEW
1a. ASTM Designations	A3.1.a	A3.1a	1.2.1		
1b. Unidentified Steel	A3.1b	A3.1b	NEW		
1c. Rolled Heavy Shapes	A3.1c	A3.1c			
1d. Built-Up Heavy Shapes	A3.1c	A3.1c			
2. Steel Castings and Forgings	A3.2	A3.2			
3. Bolts, Washers and Nuts	A3.4	A3.3			
4. Anchor Rods and Threaded Rods	A3.5	A3.4			
5. Filler Metal and Flux for Welding	A3.6	A3.5			
6. Stud Shear Connectors	A3.7	A3.6	↓	↓	↓
A4. Structural Design Drawings and Specifications	A7	A7	NEW	NEW	NEW
B. DESIGN REQUIREMENTS					
B1. General Provisions	B	NEW	NEW	NEW	NEW
B2. Loads and Load Combinations	A4	A4	1.3	NEW	NEW
B3. Design Basis					
1. Required Strength	NEW	A5.1	NEW	NEW	NEW
2. Limit States	A5.1	A5.2			
3. Design for Strength Using Load and Resistance Factor Design (LRFD)	NEW	A5.3			
4. Design for Strength Using Allowable Strength Design (ASD)	A5.1	NEW			
5. Design for Stability	B4	B4	↓		
6. Design of Connections	J	J1.1	9		
6a. Simple Connections	J1.2	J1.2	NEW		
6b. Moment Connections	J1.3	A2			
7. Design for Serviceability	A5.4	A5.4			
8. Design for Ponding	K2	K2			
9. Design for Fatigue	K4	K3			
10. Design for Fire Conditions	NEW	NEW			
11. Design for Corrosion Effects	L5	L5	↓		
12. Design Wall Thickness for HSS	NEW	NEW	1.2.2		
13. Gross and Net Area Determination					
a. Gross Area	B1	B1	NEW		
b. Net Area	B2	B2	NEW	↓	↓
B4. Classification of Sections for Local Buckling	B5.1	B5.1, APP. B5.1	2.2	NEW	NEW
1. Unstiffened Elements		B5.1	NEW		
2. Stiffened Elements	↓	B5.1	NEW	↓	↓
B5. Fabrication, Erection and Quality Control	M	M	NEW	NEW	NEW
B6. Evaluation of Existing Structures	NEW	N	NEW	NEW	NEW
C. STABILITY ANALYSIS AND DESIGN					
C1. Stability Design Requirements					
1. General Requirements	B4, C1	B4	NEW	NEW	NEW
2. Members Stability Design Requirements		C3			
3. System Stability Design Requirements	↓	NEW			
3a. Braced-Frame and Shear-Wall Systems	C2.1	C2.1			
3b. Moment-Frame Systems	C2.2	C2.2			
3c. Gravity Framing Systems	NEW	NEW			
3d. Combined Systems	NEW	NEW	↓	↓	↓

CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION

2005 SPECIFICATION	AISC SPECIFICATION REFERENCES				
	1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE 1989 ASD	SINGLE ANGLE 2000 LRFD
C2. Calculation of Required Strengths					
1. Methods of Second-Order Analysis	NEW	NEW	NEW	NEW	NEW
1a. General Second-Order Elastic Analysis		NEW			
1b. Second-Order Analysis by Amplified First-Order Elastic Analysis		C2.2			
2. Design Requirements		NEW	NEW	NEW	NEW
2a. Design by Second-Order Analysis					
2b. Design by First-Order Analysis					
D. DESIGN OF MEMBERS FOR TENSION					
D1. Slenderness Limitations	B7	B7	2.3	NEW	NEW
D2. Tensile Strength	D1	D1	3.1	2	2
D3. Area Determination					
1. Gross Area	B1	B1	NEW	NEW	NEW
2. Net Area	B2	B2	NEW	2	2
3. Effective Net Area	B3	B3	2.1	2	2
D4. Built-Up Members	D2	D2	NEW	NEW	NEW
D5. Pin-Connected Members					
1. Tensile Strength	D3.1	D3.1, E5	NEW	NEW	NEW
2. Dimensional Requirements	NEW	D3.1, E5	NEW	NEW	NEW
D6. Eyebars	D3.3	D3.2	NEW	NEW	NEW
E. DESIGN OF MEMBERS FOR COMPRESSION					
E1. General Provisions	E	E	NEW	NEW	NEW
E2. Slenderness Limitations and Effective Length	E1	B7, E1.1	4.1.1	NEW	NEW
E3. Compressive Strength for Flexural Buckling of Members Without Slender Elements	E2	E2	4.2	4	4
E4. Compressive Strength for Torsional and Flexural-Torsional Buckling of Members Without Slender Elements	E3	E3	4.2	NEW	NEW
E5. Single Angle Compression Members	NEW	NEW	N.A.	4	4
E6. Built-Up Members					
1. Compressive Strength	E4	E4.1	NEW	NEW	NEW
2. Dimensional Requirements	E4	E4.2	NEW	NEW	NEW
E7. Members with Slender Elements	App. B5	App.B5, App. E3	4.2	4	4
1. Slender Unstiffened Elements, Qs	App. B5	App. E3	N.A.	4	4
2. Slender Stiffened Elements, Qa	App. B5	App. E3	4.2	N.A.	N.A.
F. DESIGN OF MEMBERS FOR FLEXURE					
F1. General Provisions	F	F	5.1	5	5
F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent About Their Major Axis	F1.1, F1.3	F1	N.A.	N.A.	N.A.
1. Yielding		F1.1			
2. Lateral-Torsional Buckling		F1.2			
F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent About Their Major Axis	F1.2, F1.3	App. F1	N.A.	N.A.	N.A.
1. Lateral-Torsional Buckling					
2. Compression Flange Local Buckling					
F4. Other I-Shaped Members with Compact or Noncompact Webs, Bent About Their Major Axis	F1	App. F1	N.A.	N.A.	N.A.
1. Compression Flange Yielding					
2. Lateral-Torsional Buckling					
3. Compression Flange Local Buckling					
4. Tension Flange Yielding					

CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION

2005 SPECIFICATION	**AISC SPECIFICATION REFERENCES**				
				SINGLE ANGLE	
	1989 ASD	**1999 LRFD**	**2000 LRFD HSS**	**1989 ASD**	**2000 LRFD**
F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent About Their Major Axis	G2	App. F1	N.A.	N.A.	N.A.
1. Compression Flange Yielding					
2. Lateral-Torsional Buckling					
3. Compression Flange Yielding					
4. Tension Flange Yielding	NEW	NEW			
F6. I-Shaped Members and Channels Bent About Their Minor Axis	F2	App. F1	N.A.	N.A.	N.A.
1. Yielding					
2. Flange Local Buckling					
F7. Square and Rectangular HSS and Box-Shaped Members	F3	App. F1	5.1	N.A.	N.A.
1. Yielding					
2. Flange Local Buckling					
3. Web Local Buckling					
F8. Round HSS	F3	App. F1	5.1	N.A.	N.A.
1. Yielding					
2. Local Buckling					
F9. Tees and Double Angles Loaded in the Plane of Symmetry	NEW	F1.2c	NEW	N.A.	N.A.
1. Yielding					
2. Lateral-Torsional Buckling					
3. Flange Local Buckling of Tees		NEW			
F10. Single Angles	NEW	NEW	N.A.	5	5
1. Yielding				5.1.2	5.1.2
2. Lateral-Torsional Buckling				5.1.3	5.1.3
3. Leg Local Buckling				5.2.2	5.2.2
F11. Rectangular Bars and Rounds	F2	App. F1	5	N.A.	N.A.
1. Yielding			5.1		
2. Lateral-Torsional Buckling			NEW		
F12. Unsymmetrical Shapes	NEW	NEW	N.A.	N.A.	N.A.
1. Yielding					
2. Lateral-Torsional Buckling					
3. Local Buckling					
F13. Proportions of Beams and Girders					
1. Hole Reductions	B10	B10	NEW	NEW	NEW
2. Proportioning Limits for I-Shaped Members	G1	App. G1			
3. Cover Plates	B10	B10			
4. Built-Up Beams	F6	NEW			
G. DESIGN OF MEMBERS FOR SHEAR					
G1. General Provisions	F4	F2	NEW	NEW	NEW
G2. Members with Unstiffened or Stiffened Webs					
1. Nominal Shear Strength	F4	F2, App. F2, App. G3	5.2	NEW	NEW
2. Transverse Stiffeners	F5	App. F2.3	N.A.	NEW	NEW
G3. Tension Field Action					
1. Limits on the Use of Tension Field Action	G3	App. G3	N.A.	NEW	NEW
2. Nominal Shear Strength with Tension Field Action	G3	App. G3	N.A.	NEW	NEW
3. Transverse Stiffeners	G4	App. G4	N.A.	NEW	NEW
G4. Single Angles	NEW	NEW	N.A.	3	3
G5. Rectangular HSS and Box Members	NEW	F2	5.2	N.A.	N.A.
G6. Round HSS	NEW	NEW	5.2	N.A.	N.A.
G7. Weak Axis Shear in Singly and Doubly Symmetric Shapes	NEW	H2	NEW	N.A.	N.A.
G8. Beams and Girders with Web Openings	NEW	F4	NEW	NEW	NEW

CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION

2005 SPECIFICATION	AISC SPECIFICATION REFERENCES				
	1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE 1989 ASD	SINGLE ANGLE 2000 LRFD
H. DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION					
H1. Doubly and Singly Symmetric Members Subject to Flexural and Axial Force					
1. Doubly and Singly Symmetric Members in Flexure and Compression	H1	H1.2	7.1	6.1	6.1
2. Doubly and Singly Symmetric Members in Flexure and Tension	H2	H1.1	7.1	6.2	6.2
3. Doubly Symmetric Members in Single Axis Flexure and Compression	NEW	NEW	7.2	N.A.	N.A.
H2. Unsymmetric and Other Members Subject to Flexural and Axial Force	H1	H2	N.A.	N.A.	N.A.
H3. Members Under Torsion and Combined Torsion, Flexure, Shear and/or Axial Force					
1. Torsional Strength of Round and Rectangular HSS	NEW	H2	6.1	N.A.	N.A.
2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force		H2	7.2	N.A.	N.A.
3. Strength of Non-HSS Members under Torsion and Combined Stress		H2	N.A.	NEW	NEW
I. DESIGN OF COMPOSITE MEMBERS					
I1. General Provisions	I	NEW	NEW	NEW	NEW
1. Nominal Strength of Composite Sections	NEW	NEW			
1a. Plastic Stress Distribution Method					
1b. Strain-Compatibilty Method					
2. Material Limitations		I2.1			
3. Shear Connectors	I4	I5.1			
I2. Axial Members					
1. Encased Composite Columns					
1a. Limitations	NEW	I2.1	NEW	NEW	NEW
1b. Compressive Strength	E2	I2.2			
1c. Tensile Strength	D1	NEW			
1d. Shear Strength	F4	NEW			
1e. Load Transfer	NEW	I2.4			
1f. Detailing Requirements	NEW	I2.1, I5.6			
1g. Strength of Stud Shear Connectors	A 3. 7	I5.3			
2. Filled Composite Columnns					
2a. Limitations	NEW	I2.1	NEW	N.A.	N.A.
2b. Compressive Strength		I2.2			
2c. Tensile Strength		NEW			
2d. Shear Strength		NEW			
2e. Load Transfer		I2.4			
2f. Detailing Requirements		I5.6			
I3. Flexural Members					
1. General					
1a. Effective Width	I1	I3.1	NEW	NEW	NEW
1b. Shear Strength	NEW	NEW			
1c. Strength During Construction	I2	I3.4			
2. Strength of Composite Beams with Shear Connectors					
2a. Positive Flexural Strength	I2	I3.2			
2b. Negative Flexural Strength	NEW	I3.2			
2c. Strength of Composite Beams with Formed Steel Deck	I5	I3.5			
2d. Shear Connectors	I4	I5			
3. Flexural Strength of Concrete-Encased and Filled Members	NEW, I2	I3.3			
I4. Combined Axial Force and Flexure	NEW	I4	7.1	NEW	NEW
I5. Special Cases	I6	I6	NEW	N.A.	N.A.

CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION

2005 SPECIFICATION	AISC SPECIFICATION REFERENCES				
	1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE 1989 ASD	SINGLE ANGLE 2000 LRFD
J. DESIGN OF CONNECTIONS					
J1. General Provisions					
1. Design Basis	J1.1	J1.1	NEW	NEW	NEW
2. Simple Connections	J1.2	J1.2, B9			
3. Moment Connections	J1.3	J1.3			
4. Compression Members with Bearing Joints	J1.4	J1.4			
5. Splices in Heavy Sections	J1.7	J1.5			
6. Beam Copes and Weld Access Holes	J1.8	J1.6			
7. Placement of Welds and Bolts	J1.9	J1.8			
8. Bolts in Combination with Welds	J1.10	J1.9			
9. High-Strength Bolts in Combination with Rivets	J1.11	J1.10			
10. Limitations on Bolted and Welded Connections	J1.12	J1.11			
J2. Welds	J2	J2	9.2	NEW	NEW
1. Groove Welds	J2.1	J2.1	9.2		
2. Fillet Welds	J2.2	J2.2	9.2		
3. Plug and Slot Welds	J2.3	J2.3	NEW		
4. Strength	J2.4	J2.4, App. J2.4			
5. Combination of Welds	J2.5	J2.5			
6. Filler Metal Requirements	NEW	J2.6			
7. Mixed Weld Metal	J2.6	J2.7			
J3. Bolts and Threaded Parts					
1. High-Strength Bolts	J3.1	J3.1	NEW	NEW	NEW
2. Size and Use of Holes	J3.2	J3.2			
3. Minimum Spacing	J3.8	J3.3			
4. Minimum Edge Distance	J3.9	J3.4			
5. Maximum Spacing and Edge Distance	J3.10	J3.5			
6. Tension and Shear Strength of Bolts and Threaded Parts	J3.4	J3.6			
7. Combined Tension and Shear in Bearing-Type Connections	J3.5	J3.7, App. J3.7			
8. High-Strength Bolts in Slip-Critical Connections	NEW	J3.8, App. J3.8			
9. Combined Tension and Shear in Slip-Critical Connections	J3.6	J3.9, App. J3.9			
10. Bearing Strength at Bolt Holes	J3.7	J3.10	9.1.1	NEW	NEW
11. Speical Fasteners	NEW	NEW	9.1.2		
12. Tension Fasteners	NEW	NEW	9.1.3		
J4. Affected Elements of Members and Connecting Elements					
1. Strength of Elements in Tension	NEW	J4.2, J5.2	9.3.2	NEW	NEW
2. Strength of Elements in Shear	NEW	J4.1, J5.3	9.3.1		
3. Block Shear Strength	J4, J5.2	J4.3	NEW		
4. Strength of Elements in Compression	NEW	NEW	NEW		
J5. Fillers	J6	J6	NEW	NEW	NEW
J6. Splices	J7	J7	NEW	NEW	NEW
J7. Bearing Strength	J8	J8	9.1.1	NEW	NEW
J8. Column Bases and Bearing on Concrete	J9	J9	NEW	NEW	NEW
J9. Anchor Rods and Embedments	NEW	J10	NEW	NEW	NEW

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2005 SPECIFICATION	AISC SPECIFICATION REFERENCES: 1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE: 1989 ASD	SINGLE ANGLE: 2000 LRFD
J10. Flanges and Webs with Concentrated Forces	K1	K1	NEW	NEW	NEW
1. Flange Local Bending	K1.2	K1.2			
2. Web Local Yielding	K1.3	K1.3			
3. Web Crippling	K1.4	K1.4			
4. Web Sidesway Buckling	K1.5	K1.5			
5. Web Compression Buckling	K1.6	K1.6			
6. Web Panel Zone Shear	K1.7	K1.7			
7. Unframed Ends of Beams and Girders	NEW	K1.8			
8. Additional Stiffener Requirements for Concentrated Forces	K1.8	K1.9			
9. Additional Doubler Plate Requirements for Concentrated Forces	NEW	K1.10			
K. DESIGN OF HSS AND BOX MEMBER CONNECTIONS					
K1. Concentrated Forces on HSS					
1. Definitions of Parameters	NEW	NEW	NEW	N.A.	N.A.
2. Limits of Applicability			NEW		
3. Concentrated Force Distribution Transversely					
3a. Criterion for Round HSS			8.1a		
3b. Criteria for Rectangular HSS			8.1b, 9.2		
4. Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis			8.2		
4a. Criterion for Round HSS			8.2a		
4b. Criterion for Rectangular HSS			8.2b		
5. Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis			NEW		
6. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate			8.3		
K2. HSS-to-HSS Truss Connections	NEW	NEW	9.4, 9.3.4	N.A.	N.A.
1. Definitions of Parameters			9.4.1		
2. Criteria for Round HSS			9.4.2		
2a. Limits of Applicability			9.4.2a		
2b. Branches with Axial Loads in T-, Y- and Cross-Connections			9.4.2b		
2c. Branches with Axial Loads in K-Connections			9.4.2b		
3. Criteria for Rectangular HSS			9.4.3		
3a. Limits of Applicability			9.4.3a		
3b. Branches with Axial Loads in T-,Y- and Cross-Connections	NEW	NEW	9.4.3b	NEW	NEW
3c. Branches with Axial Loads in Gapped K-Connections			9.4.3c		
3d. Branches with Axial Loads in Overlapped K-Connections			NEW		
3e. Welds to Branches			9.2		
K3. HSS-to-HSS Moment Connections	NEW	NEW	9.4	N.A.	N.A.
1. Definitions of Parameters			9.4.1		
2. Criteria for Round HSS			9.4.2		
2a. Limits of Applicability			9.4.2a		
2b. Branches with Axial Loads in T-, Y- and Cross-Connections			9.4.2b		
2c. Branches with Out-of-Plane Bending Moments in T-, Y- and Cross-Connections			NEW		
2d. Branches with in-Plane Bending Moments in T-,Y- and Cross-Connections			NEW		

CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION

2005 SPECIFICATION	AISC SPECIFICATION REFERENCES				
	1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE 1989 ASD	SINGLE ANGLE 2000 LRFD
3. Criteria for Rectangular HSS	NEW	NEW	9.4.3	N.A.	NEW
3a. Limits of Applicability			9.4.3a		
3b. Branches with In-Plane Bending Moments in T- and Cross-Connections			NEW		
3c. Branches with Out-of-Plane Bending Moments in T- and Cross-Connections					
3d. Branches with Combined Bending Moment and Axial Force in T- and Cross-Connections					
L. DESIGN FOR SERVICEABILITY					
L1. General Provisions	L	L	NEW	NEW	NEW
L2. Camber	L1	L1	NEW	NEW	NEW
L3. Deflections	L3.1	L3.1	NEW	NEW	NEW
L4. Drift	L3	L3.3	NEW	NEW	NEW
L5. Vibration	L3.2	L3.2	NEW	NEW	NEW
L6. Wind-Induced Motion	NEW	NEW	NEW	NEW	NEW
L7. Expansion and Contraction	L2	L2	NEW	NEW	NEW
L8. Connection Slip	L4	L4	NEW	NEW	NEW
M. FABRICATION, ERECTION AND QUALITY CONTROL					
M1. Shop and Erection Drawings	M1	M1	NEW	NEW	NEW
M2. Fabrication					
1. Cambering, Curving and Straightening	M2.1	M2.1	NEW	NEW	NEW
2. Thermal Cutting	M2.2	M2.2			
3. Planing of Edges	M2.3	M2.3			
4. Welded Construction	M2.4	M2.4			
5. Bolted Construction	M2.5	M2.5			
6. Compression Joints	M2.6	M2.6			
7. Dimensional Tolerances	M2.7	M2.7			
8. Finish of Column Bases	M2.8	M2.8			
9. Holes for Anchor Rods	NEW	NEW			
10. Drain Holes			10	N.A.	N.A.
11. Requirements for Galvanized Members			NEW	NEW	NEW
M3. Shop Painting					
1. General Requirements	M3.1	M3.1	NEW	NEW	NEW
2. Inaccessible Surfaces	M3.2	M3.2	NEW		
3. Contact Surfaces	M3.3	M3.3	NEW		
4. Finished Surfaces	M3.4	M3.4	NEW		
5. Surfaces Adjacent to Field Welds	M3.5	M3.5	10		
M4. Erection					
1. Alignment of Column Bases	M4.1	M4.1	NEW	NEW	NEW
2. Bracing	M4.2	M4.2			
3. Alignment	M4.3	M4.3			
4. Fit of Column Compression Joints and Base Plates	M4.4	M4.4			
5. Field Welding	M4.5	M4.5	10		
6. Field Painting	M4.6	M4.6	NEW		
7. Field Connections	M4.7	M4.7	NEW		
M5. Quality Control	M5	M5	NEW	NEW	NEW
1. Cooperation	M5.1	M5.1			
2. Rejections	M5.2	M5.2			
3. Inspection of Welding	M5.3	M5.3			
4. Inspection of Slip-Critical High-Strength Bolted Connections	M5.4	M5.4			
5. Identification of Steel	M5.5	M5.5			

CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION

2005 SPECIFICATION	AISC SPECIFICATION REFERENCES				
	1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE 1989 ASD	SINGLE ANGLE 2000 LRFD
Appendix 1. INELASTIC ANALYSIS AND DESIGN					
1.1 General Provisions	NEW	NEW	NEW	NEW	NEW
1.2 Materials	NEW	NEW	NEW	NEW	NEW
1.3 Moment Redistribution	NEW	NEW	NEW	NEW	NEW
1.4 Local Buckling	NEW	B5.2	2.2.2	NEW	NEW
1.5 Stability and Second-Order Effects	NEW	C1.1	NEW	NEW	NEW
1. Braced Frames	↓	NEW	↓	↓	↓
2. Moment Frames	↓	↓	↓	↓	↓
1.6 Columns and Other Compression Members	NEW	NEW	NEW	NEW	NEW
1.7 Beams and Others Flexural Members	NEW	F1.3	5.3	NEW	NEW
1.8 Members Under Combined Forces	NEW	NEW	NEW	NEW	NEW
1.9 Connections	NEW	NEW	NEW	NEW	NEW
Appendix 2. DESIGN FOR PONDING					
2.1 Simplified Design for Ponding	K2	K2	NEW	NEW	NEW
2.2 Improved Design for Ponding	NEW	App. K2	NEW	NEW	NEW
Appendix 3. DESIGN FOR FATIGUE					
3.1 General	K4, App. K4	K3, App. K3.1	NEW	NEW	NEW
3.2 Calculation of Maximum Stresses and Stress Ranges	App. K4.2	App. K3.2	NEW	NEW	NEW
3.3 Design Stress Range	App.K4.2	App. K3.3	NEW	NEW	NEW
3.4 Bolts and Threaded Parts	App. K4.3	App. K3.4	NEW	NEW	NEW
3.5 Special Fabrication and Erection Requirements	NEW	App. K3.5	NEW	NEW	NEW
Appendix 4. STRUCTURAL DESIGN FOR FIRE CONDITIONS	NEW	NEW	NEW	NEW	NEW
Appendix 5. EVALUATION OF EXISTING STRUCTURES					
5.1 General Provisions	NEW	N1	NEW	NEW	NEW
5.2 Material Properties					
1. Determination of Required Tests	NEW	N2.1	NEW	NEW	NEW
2. Tensile Properties	↓	N2.2	↓	↓	↓
3. Chemical Composition	↓	N2.3	↓	↓	↓
4. Base Metal Notch Toughness	↓	N2.4	↓	↓	↓
5. Weld Metal	↓	N2.5	↓	↓	↓
6. Bolts and Rivets	↓	N2.6	↓	↓	↓
5.3 Evaluation by Structural Analysis					
1. Dimensional Data	NEW	N3.1	NEW	NEW	NEW
2. Strength Evaluation	↓	N3.2	↓	↓	↓
3. Serviceabiltity Evaluation	↓	N3.3	↓	↓	↓
5.4 Evaluation by Load Tests					
1. Determination of Load Rating by Testing	NEW	N4.1	NEW	NEW	NEW
2. Serviceabiltity Evaluation	↓	N4.2	↓	↓	↓
5.5 Evaluation Report	↓	N5	↓	↓	↓
Appendix 6. STABILITY BRACING FOR COLUMNS AND BEAMS	NEW	C3	NEW	NEW	NEW
Appendix 7. DIRECT ANALYSIS METHOD	NEW	NEW	NEW	NEW	NEW