

# DESIGN EXAMPLES

## Version 13.0



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OF  
STEEL CONSTRUCTION  
INC.

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Printed in the United States of America

## PREFACE

The AISC *Design Examples CD* provides examples on the application of the 2005 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-05) and the AISC *Steel Construction Manual*, 13<sup>th</sup> Edition. The examples found herein 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 on this CD do not represent a stand-alone document. 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 and the Chapters are referred to by their corresponding letter reference from 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 difference between the approaches is 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 should be 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*, 13<sup>th</sup> 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,  $\phi$ , and the safety factor,  $\Omega$ , are applied.
4. The results of design equations are presented to 3 significant figures throughout these calculations.

### ACKNOWLEDGEMENTS

The AISC Committee on Manuals and Textbooks 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:

George Batcha  
 J. Brown  
 Jon Dean  
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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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Duane Becker  
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**APPENDIX A. CROSS-REFERENCE LIST FOR THE 2005 AISC SPECIFICATION**



## 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*, 13<sup>th</sup> 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](http://www.aisc.org), should be consulted.

#### A2. REFERENCED SPECIFICATIONS, 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 acceptable for use in accordance 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](http://www.aisc.org).

## CHAPTER B DESIGN REQUIREMENTS

### B1. GENERAL PROVISIONS

### B2. LOADS AND LOAD COMBINATIONS

In the absence of a building code to provide otherwise, the default load combinations to be used with this Specification are taken from ASCE7-02.

### 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 Guide 19, *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:

<sup>c</sup> Shape is slender in compression

<sup>f</sup> Shape exceeds compact limit for flexure

<sup>g</sup> The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility

<sup>h</sup> Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

<sup>v</sup> 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 has, for some time, required that 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 should include consideration of the combined effect of gravity and lateral loads, including the resulting second-order effects,  $P-\Delta$  and  $P-\delta$ . The effects of “leaning columns” should also be considered, as illustrated in the four-story building design example in Part III of AISC *Design Examples*.

$P-\Delta$  and  $P-\delta$  effects are illustrated in Commentary Figure C-C1.1. Several methods for addressing stability, including  $P-\Delta$  and  $P-\delta$  effects are discussed in 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**.

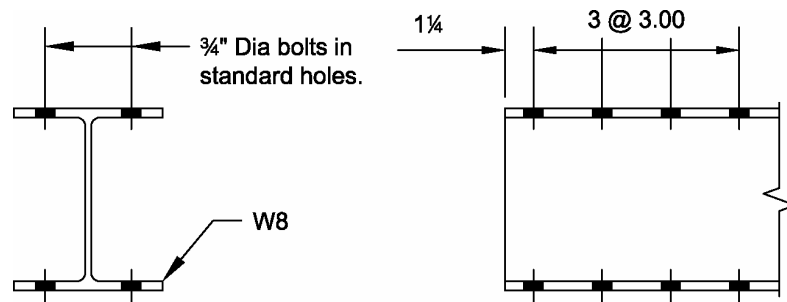
#### D6. EYEBARS

An example of an eyebar connection is given in **Example D.8**.

## 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.



### Solution:

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(30 \text{ kips}) + 1.6(90 \text{ kips})$ $= 180 \text{ kips}$	$P_a = 30 \text{ kips} + 90 \text{ kips}$ $= 120 \text{ kips}$

Try a W8×21

#### Material Properties:

W8×21    ASTM A992     $F_y = 50 \text{ ksi}$      $F_u = 65 \text{ ksi}$

Manual  
Table 2-3

#### Geometric Properties:

W8×21     $A_g = 6.16 \text{ in.}^2$      $b_f = 5.27 \text{ in.}$      $t_f = 0.400 \text{ in.}$      $d = 8.28 \text{ in.}$   
 $r_y = 1.26 \text{ in.}$   
 $\bar{y} = 0.831 \text{ in. (for WT4} \times 10.5)$

Manual  
Table 1-1  
Table 1-8

Check tensile yield limit state using tabulated values

LRFD	ASD
$277 \text{ kips} > 180 \text{ kips}$ <b>o.k.</b>	$184 \text{ kips} > 120 \text{ kips}$ <b>o.k.</b>

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

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{0.831 \text{ in.}}{9.00 \text{ in.}} = 0.908$$

Case 7

Table D3.1  
Case 2

$$b_f = 5.27 \text{ in.} \quad d = 8.28 \text{ in.} \quad b_f < 2/3d$$

$$U = 0.85$$

Table D3.1  
Case 7

Use  $U = 0.908$

Calculate  $A_n$

Section D3.2

$$A_n = A_g - 4(d_h + 1/16 \text{ in.})t_f \\ = 6.16 \text{ in.}^2 - 4(1\frac{3}{16} \text{ in.} + 1/16 \text{ in.})(0.400 \text{ in.}) = 4.76 \text{ in.}^2$$

Calculate  $A_e$

Section D3.3

$$A_e = A_n U \\ = 4.76 \text{ in.}^2 (0.908) = 4.32 \text{ in.}^2$$

Eqn. D3-1

$$A_e/A_g = 4.32 \text{ in.}^2 / 6.16 \text{ in.}^2 = 0.701 < 0.75 \quad \text{tabulated values for rupture n.a.}$$

$$P_n = F_u A_e = (65 \text{ ksi})(4.32 \text{ in.}^2) = 281 \text{ kips}$$

Eqn. D2-2

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

Section D2

Check the non-mandatory slenderness limit

Section D1

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

LRFD	ASD
225 kips > 180 kips <b>o.k.</b>	150 kips > 120 kips <b>o.k.</b>

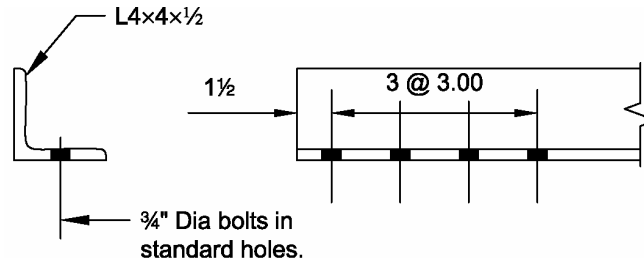
Manual  
Table 5-1

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

## 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.



### 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.<sup>2</sup>       $r_z = 0.776$  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

$$P_n = F_y A_g = (36 \text{ ksi})(3.75 \text{ in.}^2) = 135 \text{ kips}$$

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  $A_n$

$$A_n = A_g - (d_h + 1/16)t$$

$$= 3.75 \text{ in.}^2 - (13/16 \text{ in.} + 1/16 \text{ in.})(1/2 \text{ in.}) = 3.31 \text{ in.}^2$$

Section D3.2

Calculate  $A_e$

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

Section D3.3

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 L4×4×1/2 available tensile strength is governed by the tensile yielding limit state.

LRFD	ASD
$\phi_t P_n = 125 \text{ kips}$ $125 \text{ kips} > 120 \text{ kips}$	$P_n / \Omega_t = 83.5 \text{ kips}$ $83.5 \text{ kips} > 80.0 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Calculate recommended  $L_{max}$

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

Section D1

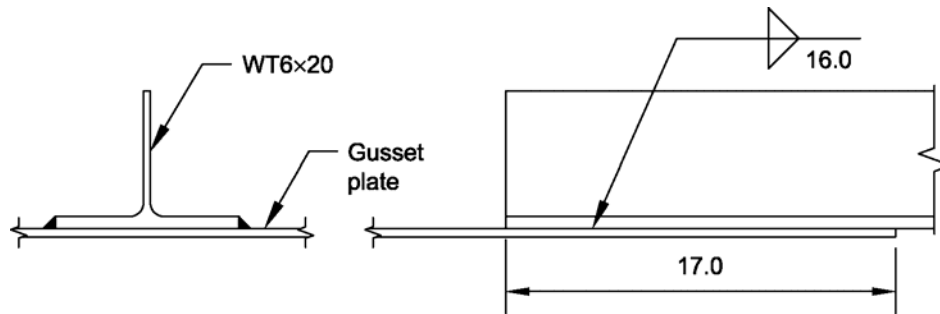
Note: The  $L/r$  limit is a recommendation, not a requirement.



### 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 and has a length of 16 in. Verify the member strength by both LRFD and ASD. Assume that the gusset plate and the weld have been checked and 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.<sup>2</sup>       $r_x = 1.57$  in.       $\bar{y} = 1.09$  in. =  $\bar{x}$  (in equation for  $U$ )

Manual  
Table 1-8

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(40.0 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240 \text{ kips}$	$P_u = 40.0 \text{ kips} + 120 \text{ kips}$ $= 160 \text{ kips}$

Check tensile yielding limit state using tabulated values

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

Manual  
Table 5-3

Check tensile rupture limit state using tabulated values

LRFD	ASD
$214 \text{ kips} < 240 \text{ kips}$ <b>n.g.</b>	$142 \text{ kips} < 160 \text{ kips}$ <b>n.g.</b>

Manual  
Table 5-3

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

Calculate  $U$

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

Table D3.1  
Case 2

Calculate  $A_n$

Section D3.2

$$A_n = A_g = 5.84 \text{ in.}^2 \quad (\text{because there are no holes})$$

Calculate  $A_e$

Section D3.3

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

Eqn. D3-1

Calculate  $P_n$

$$P_n = F_u A_e = 65 \text{ ksi} (5.44 \text{ in.}^2) = 354 \text{ kips}$$

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

Section D2

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:

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

Manual  
Table 5-3

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 <b>o.k.</b>	$P_n/\Omega_t = 175 \text{ kips}$ 175 kips > 160 kips <b>o.k.</b>

Check the non-mandatory slenderness limit

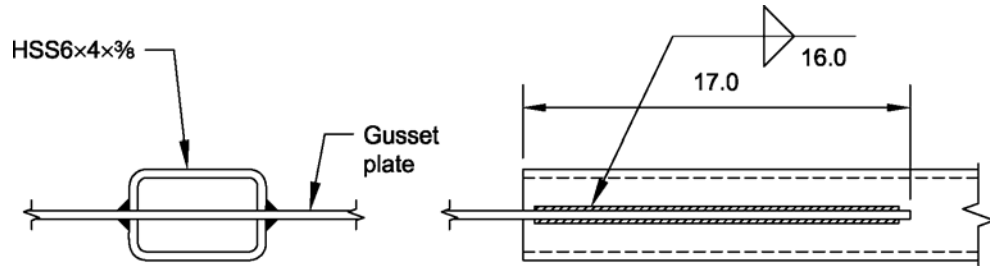
Section D1

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

### Example D.4 Rectangular HSS Tension Member

#### Given:

Verify, by LRFD and ASD, the strength of an HSS6×4× $\frac{3}{8}$ , 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 fillet welded to a  $\frac{1}{2}$  in. thick single concentric gusset plate and has a length of 16 in.



#### Solution:

##### Material Properties:

HSS6×4× $\frac{3}{8}$       ASTM A500 grade B       $F_y = 46$  ksi       $F_u = 58$  ksi

Manual  
Table 2-3

##### Member Geometric Properties:

HSS6×4× $\frac{3}{8}$        $A_g = 6.18$  in.<sup>2</sup>       $r_y = 1.55$  in.       $t = 0.349$  in.

Manual  
Table 1-11

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(35.0 \text{ kips}) + 1.6(105 \text{ kips})$ $= 210 \text{ kips}$	$P_a = 35.0 \text{ kips} + 105 \text{ kips}$ $= 140 \text{ kips}$

Check available tensile yield strength

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

Manual  
Table 5-4

Check available tensile rupture strength

LRFD	ASD
$201 \text{ kips} < 210 \text{ kips}$ <b>n.g.</b>	$134 \text{ kips} < 140 \text{ kips}$ <b>n.g.</b>

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

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

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

Calculate  $A_e$

Section D3.2

$$A_e = A_n U$$

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

Section D3.3

Calculate  $P_n$

Eqn. D3-1

$$P_n = F_u A_e$$

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

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

Section D2

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

Check the non-mandatory slenderness limit

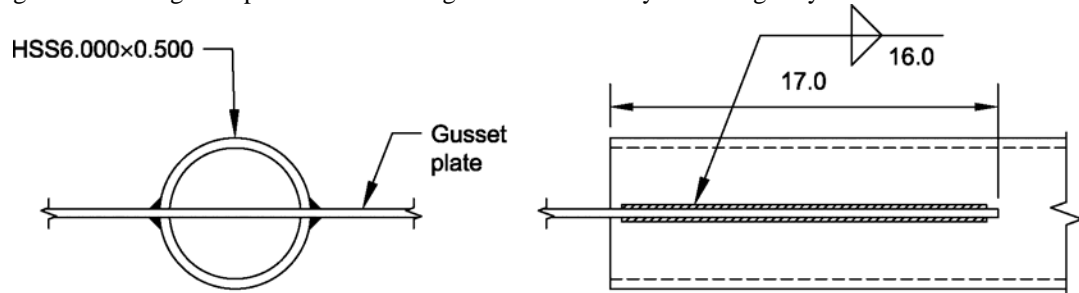
Section D1

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

## Example D.5 Round HSS Tension Member

### Given:

See Figure D-5 below. An HSS6.000×0.500, ASTM A500 grade B, has 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  $\frac{1}{2}$  in. thick single concentric gusset plate that has a length of 16 in. Verify the strength by both LRFD and ASD.



### 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.<sup>2</sup>       $r = 1.96$  in.       $t = 0.465$  in.      Manual Table 1-13

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(40.0 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240 \text{ kips}$	$P_a = 40.0 \text{ kips} + 120 \text{ kips}$ $= 160 \text{ kips}$

Check available tensile yield strength

LRFD	ASD
$\phi_t P_n = 306 \text{ kips} > 240$ <b>o.k.</b>	$P_n / \Omega_t = 203 \text{ kips} > 160 \text{ kips}$ <b>o.k.</b>

Manual Table 5-6

Check available tensile rupture strength

LRFD	ASD
$\phi_t P_n = 264 \text{ kips} > 240 \text{ kips}$ <b>o.k.</b>	$P_n / \Omega_t = 176 \text{ kips} > 160 \text{ kips}$ <b>o.k.</b>

Manual Table 5-6

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

$$L = 16.0 \text{ in.} \quad D = 6.00 \text{ in.} \quad L/D = 16.0 \text{ in.} / (6.00 \text{ in.}) = 2.67 > 1.3$$

$$U = 1.0$$

Manual Table D3.1  
Case 5

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

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

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

Section D3.2

Calculate  $A_e$

Section D3.3

$$\begin{aligned} A_e &= A_n U \\ &= 7.57 \text{ in.}^2 (1.0) = 7.57 \text{ in.}^2 \end{aligned}$$

Eqn. D3-1

$$A_e / A_g = 7.57 \text{ in.}^2 / (8.09 \text{ in.}^2) = 0.936 \text{ in.}^2 > 0.75 A_g \quad \mathbf{o.k.}$$

Check the non-mandatory slenderness limit

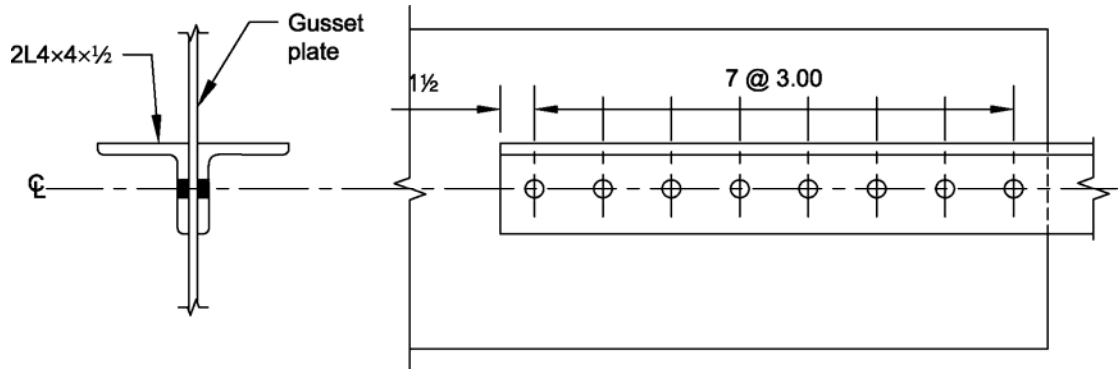
Section D1

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

## Example D.6 Double-Angle Tension Member

### Given:

A 2L4×4×½ (¾-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 strength by both LRFD and ASD.



### Solution:

#### Material Properties:

2L4×4×½ ASTM A36

$$F_y = 36 \text{ ksi}$$

$$F_u = 58 \text{ ksi}$$

Manual  
Table 2-3

#### Geometric Properties:

For a single L4×4×½

$$A_g = 3.75 \text{ in.}^2$$

$$r_x = 1.21 \text{ in.}$$

$$r_y = 1.83 \text{ in.}$$

$$\bar{x} = 1.18 \text{ in.}$$

Manual  
Table 1-7

Calculate the required tensile strength

LRFD	ASD
$P_n = 1.2(40.0 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240 \text{ kips}$	$P_n = 40.0 \text{ kips} + 120 \text{ kips}$ $= 160 \text{ kips}$

Calculate the available tensile yield strength

$$P_n = F_y A_g = (36 \text{ ksi})(2)(3.75 \text{ in.}^2) = 270 \text{ kips}$$

Eqn. D2-1

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(270 \text{ kips}) = 243 \text{ kips}$	$\Omega_t = 1.67$ $P_n / \Omega_t = (270 \text{ kips}) / 1.67 = 162 \text{ kips}$

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

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

$$= 2(3.75 \text{ in.}^2) - 2(13/16 \text{ in.} + 1/16 \text{ in.})(1/2 \text{ in.}) = 6.63 \text{ in.}^2$$

Calculate  $A_e$

$$A_e = A_n U = 6.63 \text{ in.}^2 (0.944) = 6.26 \text{ in.}^2$$

Eqn. D3-1

$$P_n = F_u A_e = (58 \text{ ksi})(6.26 \text{ in.}^2) = 363 \text{ kips}$$

Eqn. D2-2

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

Section D2

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

LRFD	ASD
243 kips > 240 kips <b>o.k.</b>	162 kips > 160 kips <b>o.k.</b>

Check the non-mandatory slenderness limit

Section D1

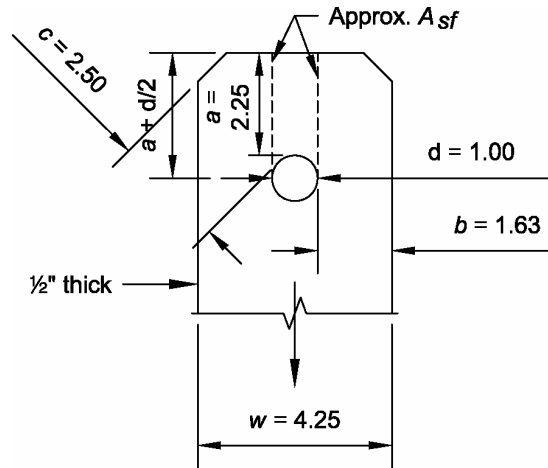
$$L/r = \left( \frac{25.0 \text{ ft}}{1.21 \text{ in.}} \right) \left( \frac{12.0 \text{ in.}}{\text{ft}} \right) = 248 < 300 \quad \text{o.k.}$$



### 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  $\frac{1}{32}$ -in. oversized hole. Assume that the pin itself is adequate. Verify the strength by both LRFD and ASD.



*Pin connected tension member*

#### 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.

Check dimensional requirements:

Section D5.2

- 1)  $b_{eff} = 2t + 0.63$  in. =  $2(0.500$  in.) + 0.63 in. = 1.63 in.
- 2)  $a \geq 1.33b_{eff}$              $2.25$  in.  $\geq (1.33)(1.63$  in.) = 2.17 in.            **o.k.**
- 3)  $w \geq 2b_{eff} + d$              $4.25$  in.  $\geq 2(1.63$  in.) + 1.00 in. = 4.26 in.  $\cong 4.25$  in.            **o.k.**
- 4)  $c \geq a$              $2.50$  in.  $> 2.25$  in.            **o.k.**

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(12.0$ kips) + $1.6(4.00$ kips) = 20.8 kips	$P_a = 12.0$ kips + $4.00$ kips = 16.0 kips

Calculate the available tensile rupture strength on the net effective area

$$P_n = 2t_{eff}F_u = (2)(0.500 \text{ in.})(1.63 \text{ in.})(58 \text{ ksi}) = 94.5 \text{ kips}$$

Eqn. D5-1

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(94.5 \text{ kips}) = 70.9 \text{ kips}$	$\Omega_t = 2.00$ $P_n/\Omega_t = (94.5 \text{ kips}) / 2.00 = 47.3 \text{ kips}$

Section D5.1

Calculate the available shear rupture strength

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

Section D5.1

$$P_n = 0.6F_u A_{sf} = (0.6)(58 \text{ ksi})(2.75 \text{ in.}^2) = 95.7 \text{ kips}$$

Eqn. D5-2

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

Section D5.1

Calculate the available bearing strength

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

$$R_n = 1.8F_y A_{pb} = 1.8(36 \text{ ksi})(0.500 \text{ in.}^2) = 32.4 \text{ kips}$$

Eqn. J7.1

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(32.4 \text{ kips}) = 24.3 \text{ kips}$	$\Omega_t = 2.00$ $P_n/\Omega_t = (32.4 \text{ kips}) / 2.00 = 16.2 \text{ kips}$

Section J7

Calculate the available tensile yielding strength

$$A_g = 4.25 \text{ in.}(0.500 \text{ in.}) = 2.13 \text{ in.}^2$$

Section D2

$$P_n = F_y A_g = 36 \text{ ksi}(2.13 \text{ in.}^2) = 76.5 \text{ kips}$$

Eqn. D2.1

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(76.5 \text{ kips}) = 68.9 \text{ kips}$	$\Omega_t = 1.67$ $P_n/\Omega_t = (76.5 \text{ kips}) / 1.67 = 45.8 \text{ kips}$

Section D2

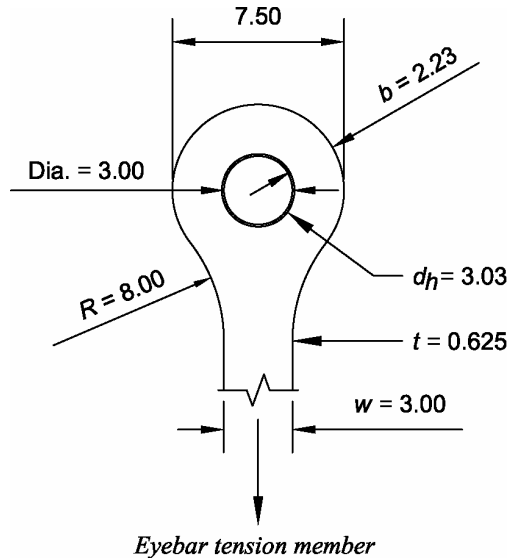
The available tensile strength is governed by the bearing strength limit state

LRFD	ASD
$\phi_t P_n = 24.3 \text{ kips}$ $24.3 \text{ kips} > 20.8 \text{ kips}$	$P_n/\Omega_t = 16.2 \text{ kips}$ $16.2 \text{ kips} > 16.0 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

## Example D.8 Eyebar Tension Member

### Given:

See Figure D-8 below. A  $\frac{5}{8}$  in. thick eyebar member, 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:

$w = 3$  in.       $b = 2.23$  in.       $t = 0.625$  in.  
 $d_b = 3$  in.       $d_h = 3.03$  in.       $R = 8.00$  in.

*Check dimensional requirements*

Section D4.2

- 1)  $t \geq \frac{1}{2}$  in.       $0.625$  in.  $\geq 0.500$  in.    **o.k.**
- 2)  $w \leq 8t$        $3.00$  in.  $\leq 8(0.625$  in.) =  $5$  in.    **o.k.**
- 3)  $d \geq \frac{7}{8}w$        $3.00$  in.  $\geq \frac{7}{8}(3.00$  in.) =  $2.63$  in.    **o.k.**
- 4)  $d_h \leq d + \frac{1}{32}$  in.       $3.03$  in.  $\leq 3.00$  in. +  $(\frac{1}{32}$  in.) =  $3.03$  in.    **o.k.**
- 5)  $R \geq d_h + 2b$        $8.00$  in.  $\geq 3.03$  in. +  $2(2.23$  in.) =  $7.50$  in.    **o.k.**
- 6)  $\frac{2}{3}w \leq b \leq \frac{3}{4}w$        $\frac{2}{3}(3.00$  in.)  $\leq 2.23$  in.  $\leq \frac{3}{4}(3.00$  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  $w$ )

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

$$P_n = F_y A_g = (36 \text{ ksi})(1.88 \text{ in.}^2) = 67.7 \text{ kips}$$

Eqn. D2-1

LRFD	ASD
$\phi_t = 0.90$ $\phi_t P_n = 0.90(67.7 \text{ kips}) = 60.9 \text{ kips}$  $60.9 \text{ kips} > 54.0 \text{ kips}$	$\Omega_t = 1.67$ $P_n/\Omega_t = (67.7 \text{ kips})/1.67 = 40.5 \text{ kips}$  $40.3 \text{ kips} > 40.0 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Section D2

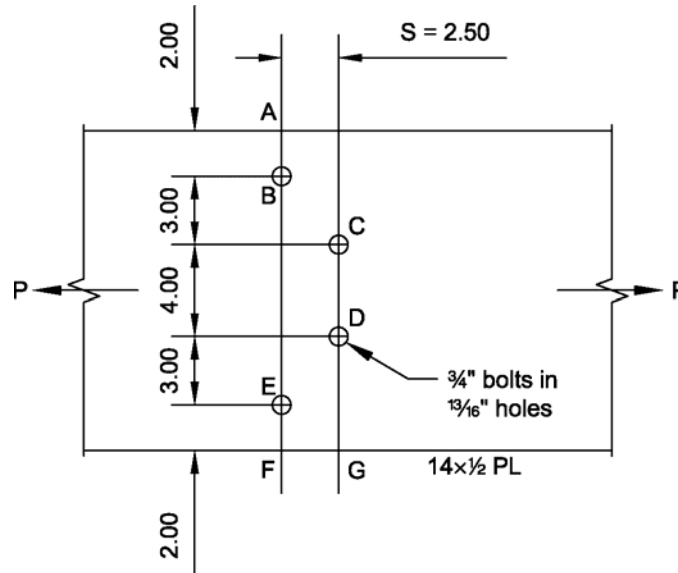
The eyebar tension member available strength is governed by the tension yield limit state.

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 lower than that of the eyebar, bearing.

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

#### Given:

See Fig. D-9 below. A 14 in. wide and  $\frac{1}{2}$  in. thick plate subject to tensile loading has staggered holes as shown. Compute  $A_n$  and  $A_e$



#### Solution:

Calculate net hole diameter

Section D3.2

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

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

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

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

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

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

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

Calculate  $U$

Since tension load is transmitted to all elements by the fasteners

$$U = 1.0$$

Table D3.1  
Case 1

$$A_e = A_n(1.0) = 5.75 \text{ in.}^2$$

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,  $P_n/\Omega_c$ , are determined as follows:

$P_n$  = nominal compressive strength based on the controlling buckling mode

$\phi_c = 0.90$  (LRFD)       $\Omega_c = 1.67$  (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  $3 \times 3 \times 1/4$  angle, with a  $3/8$ -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 $\times$ 22<sup>c</sup>.

#### 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.

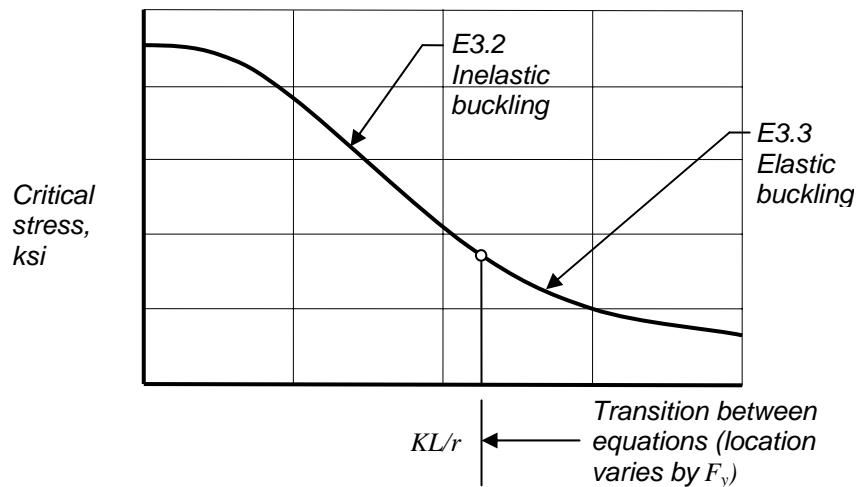


Figure E-1 Standard Column Curve

TRANSITION POINT LIMITING VALUES OF $KL/r$		
$F_y$ ksi (MPa)	Limiting $KL/r$	$0.44F_y$ 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 Tee 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 makes suggestions as to how select built-up members to avoid 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.

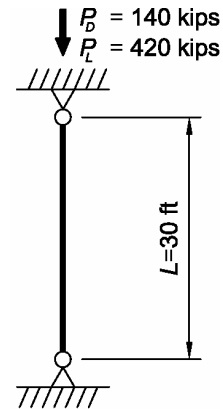
Design examples have been included in this Chapter for built-up I-shaped members with slender webs and slender flanges. Examples have also been included for a double angle, WT, and an HSS shape with slender elements.



### 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
$P_u = 1.2(140 \text{ kips}) + 1.6(420 \text{ kips}) = 840 \text{ kips}$	$P_a = 140 \text{ kips} + 420 \text{ kips} = 560 \text{ kips}$

Select a column using Manual Table 4-1

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

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 buckling 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.

Commentary  
Table  
C-C2.2

Shape		W14x											
		145		132		120		109		99		90	
		$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
		live length KL (ft) with respect to least radius of gyration $r_y$	0	1280	1920	1160	1740	1060	1590	959	1440	872	1310
	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:

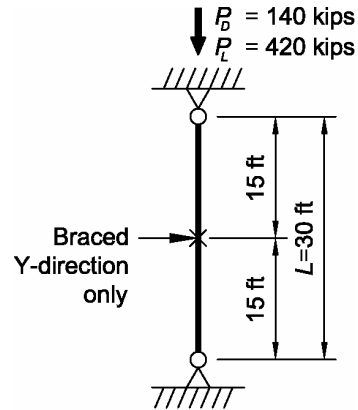
LRFD		ASD	
$\phi_c P_n = 892$ kips	<b>o.k.</b>	$P_n/\Omega_c = 594$ kips	<b>o.k.</b>

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
$P_u = 1.2(140 \text{ kips}) + 1.6(420 \text{ kips}) = 840 \text{ kips}$	$P_a = 140 \text{ kips} + 420 \text{ kips} = 560 \text{ kips}$

Select a column using Manual Table 4-1.

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

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
$\phi P_n = 1000 \text{ kips}$	$P_n/\Omega = 667 \text{ kips}$

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

$$KL = \frac{30.0 \text{ ft}}{1.66} = 18 \text{ ft}$$

Commentary  
Table  
C-C2.2

Shape		W14×											
		145		132		120		109		99		90	
		$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$	$P_n/\Omega_c$	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
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	970	1470	862	1300	783	1180	709	1070	642	966	583	876

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	<b>o.k.</b>	$P_n/\Omega_c = 618$ kips > 560 kips	<b>o.k.</b>

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.<sup>2</sup>  $r_x = 6.28$  in.  $r_y = 3.76$  in.

Manual  
Table 1-1

*Calculate the available strength*

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.

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

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

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

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 ft and weak axis and torsional unbraced lengths of 15 ft. The material properties and loads are as given in Example E.1b.

#### Geometric Properties:

$$W14 \times 90 \quad A_g = 26.5 \text{ in.}^2 \quad r_x = 6.14 \text{ in.} \quad r_y = 3.70 \text{ in.}$$

Manual  
Table 1-1

Check both slenderness ratios

$$K = 1.0$$

Commentary  
Table  
C-C2.2

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

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

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

$$4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 113 > 58.6$$

$$\text{Because } \frac{KL}{r} < 4.71 \sqrt{\frac{E}{F_y}}$$

$$F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y = \left[ 0.658 \frac{50.0 \text{ ksi}}{83.3 \text{ ksi}} \right] 50.0 \text{ ksi} = 38.9 \text{ ksi}$$

Eqn. E3-2

$$P_n = F_{cr} A_g = 38.9 \text{ ksi} (26.5 \text{ in.}^2) = 1030 \text{ kips}$$

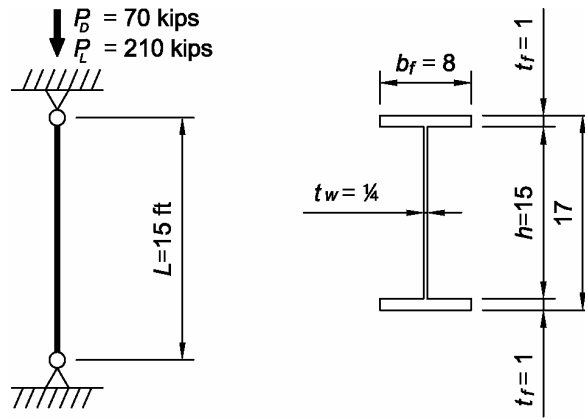
Eqn. E3-1

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(1030 \text{ kips})$ $= 928 \text{ kips} > 840 \text{ kips}$	$\Omega_c = 1.67$ $P_n / \Omega_c = (1030 \text{ kips}) / 1.67$ $= 618 \text{ kips} > 560 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

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

**Given:**

Verify that a built-up, ASTM A572 grade 50, column with PL1in.×8in. flanges and a PL¼in.×15in. 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 2-3

**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
$P_u = 1.2(70.0 \text{ kips}) + 1.6(210 \text{ kips}) = 420 \text{ kips}$	$P_a = 70.0 \text{ kips} + 210 \text{ kips} = 280 \text{ kips}$

Calculate built-up section properties (ignoring fillet welds)

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

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

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

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

$$= 2(8.00 \text{ in.}^2)(8.00 \text{ in.})^2 + \frac{(0.250 \text{ in.})(15.00 \text{ in.})^3}{12} + \frac{2(8.0 \text{ in.})(1.0 \text{ in.})^3}{12} = 1100 \text{ in.}^4$$

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.

$$\frac{KL_y}{r_y} = \frac{1.0(15.0 \text{ ft})}{2.08 \text{ in.}} \frac{12.0 \text{ in.}}{\text{ft}} = 86.6$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(86.6)^2} = 38.2 \text{ ksi}$$

Eqn. E3-4

Calculate the elastic critical torsional buckling stress

Note: Torsional buckling will not govern if  $KL_y > KL_z$ , 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

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

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

$$= 92.2 \text{ ksi} > 38.2 \text{ ksi}$$

Eqn. E4-4

Therefore, the flexural buckling limit state controls.

Use  $F_e = 38.2 \text{ ksi}$

Check for slender elements using Specification Section E7

Determine  $Q_s$ , the unstiffened element (flange) reduction factor

Calculate  $k_c$

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

Table B4.1  
Note [a]

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

Table B4.1  
Case 4



Therefore, the flange is not slender.

$$Q_s = 1.0$$

Determine  $Q_a$ , the stiffened element (web) reduction factor

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

$$Q_a = \frac{A_{eff}}{A} \text{ where } A_{eff} \text{ is effective area based on the reduced effective width of the web, } b_e. \quad \text{Section E7.2}$$

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

Eqn. E7-16

$KL/r = 86.6$  from above

$$4.71 \sqrt{\frac{E}{QF_y}} = 4.71 \sqrt{\frac{29,000 \text{ ksi}}{1.0(50 \text{ ksi})}} = 113 > 86.6$$

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

$$F_{cr} = Q \left[ 0.658 \frac{QF_y}{F_e} \right] F_y = 1.0 \left[ 0.658 \frac{1.0(50 \text{ ksi})}{38.2 \text{ ksi}} \right] (50 \text{ ksi}) = 28.9 \text{ ksi}$$

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

$$= 1.92(0.250 \text{ in.}) \sqrt{\frac{29,000 \text{ ksi}}{28.9 \text{ ksi}}} \left[ 1 - \frac{0.34}{(15.0 \text{ in.}/0.250 \text{ in.})} \sqrt{\frac{29,000 \text{ ksi}}{28.9 \text{ ksi}}} \right] \leq 15.0 \text{ in.}$$

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

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

$$Q_a = \frac{A_{eff}}{A} = \frac{19.1 \text{ in.}^2}{19.8 \text{ in.}^2} = 0.966$$

Eqn. E7-16

$$Q = Q_s Q_a = (1.00)(0.966) = 0.966$$

Section E7

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

$KL/r = 86.6$  from above

$$4.71 \sqrt{\frac{E}{QF_y}} = 4.71 \sqrt{\frac{29,000 \text{ ksi}}{0.966(50 \text{ ksi})}} = 115.4 > 86.6$$

Therefore, Specification Equation E7-2 applies.

$$F_{cr} = Q \left[ 0.658 \frac{QF_y}{F_e} \right] F_y = 0.966 \left[ 0.658 \frac{0.966(50 \text{ ksi})}{38.2 \text{ ksi}} \right] (50 \text{ ksi}) = 28.5 \text{ ksi}$$

Eqn. E7-2

Calculate the nominal compressive strength

$$P_n = F_{cr} A_g = 28.5 \text{ ksi} (19.8 \text{ in}^2) = 562 \text{ kips}$$

Eqn. E7-1

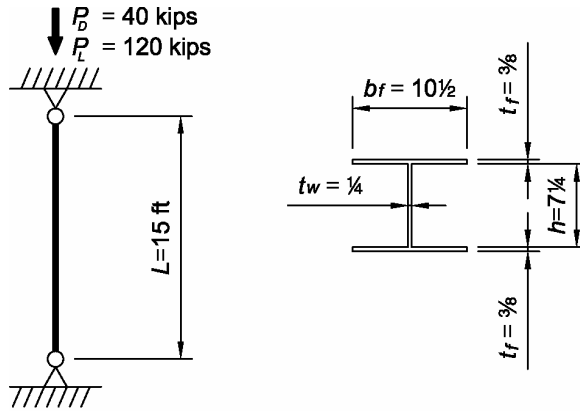
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(562 \text{ kips})$ $= 505 \text{ kips} > 420 \text{ kips}$	$\Omega_c = 1.67$ $P_n / \Omega_c = 562 \text{ kips} / 1.67$ $= 336 \text{ kips} > 280 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Section E1

### Example E.3 Built-up Column with Slender Flanges

#### Given:

Determine if a built-up, ASTM A572 grade 50 column with PL $\frac{3}{8}$  in. $\times$ 10 $\frac{1}{2}$  in. flanges and a PL $\frac{1}{4}$  in. $\times$ 7 $\frac{1}{4}$  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 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 2-3

##### 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
$P_u = 1.2(40.0 \text{ kips}) + 1.6(120 \text{ kips}) = 240 \text{ kips}$	$P_a = 40.0 \text{ kips} + 120 \text{ kips} = 160 \text{ kips}$

Calculate built-up section properties (ignoring fillet welds)

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

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,

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

Table B4.1  
Case 10

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

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

*Check flange slenderness*

Calculate  $k_c$

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

Table B4.1  
Note [a]

Use  $k_c = 0.743$

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

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

Table B4.1  
Case 4

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

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.

Section E7

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

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

Commentary  
Table  
C-C2.2

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

Calculate the elastic critical stress,  $F_e$ , for flexural buckling

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

Eqn. E3-4

Calculate the elastic critical stress,  $F_e$ , for torsional buckling

Not likely to govern, but check for completeness

$$C_w = \frac{I_y h_o^2}{4} = \frac{72.4 \text{ in.}^4 (7.63 \text{ in.})^2}{4} = 1050 \text{ in.}^6$$

Design  
Guide No. 3  
Eqn 3-5

$$J = \sum \frac{bt^3}{3} = \frac{2(10.5 \text{ in.})(0.375 \text{ in.})^3 + 7.25 \text{ in.}(0.25 \text{ in.})^3}{3} = 0.407 \text{ in.}^4$$

Design  
Guide No. 3  
Eqn 3-4

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

$$= \left[ \frac{\pi^2 (29,000 \text{ ksi})(1050 \text{ in.}^6)}{(180 \text{ in.})^2} + (11,200 \text{ ksi})(0.407 \text{ in.}^4) \right] \frac{1}{122 \text{ in.}^4 + 72.4 \text{ in.}^4}$$

$$= 71.2 \text{ ksi} > 65.9 \text{ ksi}$$

Eqn. E4-4

Therefore, use  $F_e = 65.9 \text{ ksi}$

Determine  $Q$ , the slenderness reduction factor

$Q = Q_s Q_a$ , where  $Q_a = 1.0$  because the web is not slender

Calculate  $Q_s$ , the unstiffened element (flange) reduction factor

Section E7-1

Determine the proper equation for  $Q_s$  by checking limits for Equations E7-7 to E7-9

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

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

Section  
E7-1(b)

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

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

$$Q_s = 1.415 - 0.65 \left( \frac{b}{t} \right) \sqrt{\frac{F_y}{Ek_c}}$$

$$= 1.415 - 0.65(14.0) \sqrt{\frac{50 \text{ ksi}}{(29,000 \text{ ksi})(0.743)}} = 0.977$$

Eqn. E7-8

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

Calculate nominal compressive strength

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

$$F_{cr} = Q \left[ 0.658 \frac{QF_y}{F_e} \right]_{F_y} = 0.977 \left[ 0.658 \frac{0.977(50 \text{ ksi})}{65.9 \text{ ksi}} \right] (50 \text{ ksi}) = 35.8 \text{ ksi} \quad \text{Eqn. E7-2}$$

$$P_n = F_{cr} A_g = (35.8 \text{ ksi})(9.69 \text{ in}^2) = 347 \text{ kips} \quad \text{Eqn. E7-1}$$

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

Section E1

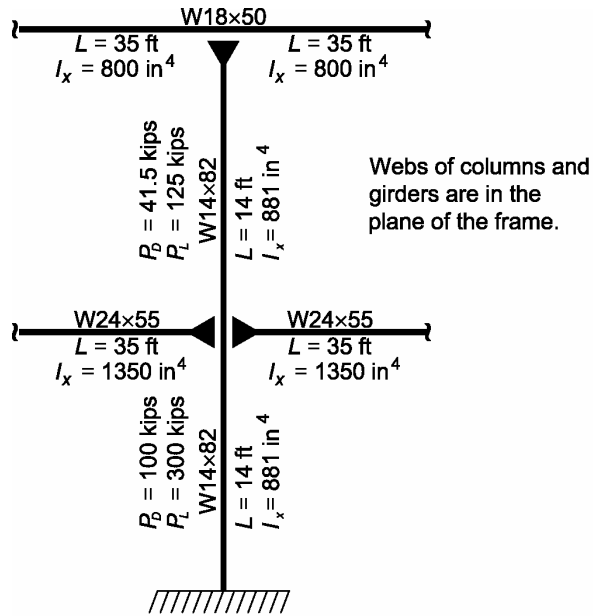
Note: Built-up sections are generally more expensive than standard rolled shapes; therefore, a standard compact shape, such as a W8×35 might be a better choice even if the weight is somewhat higher. This selection could be taken directly from Manual Table 4-1.

### 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 2-1

**Geometric Properties:**

W18x50  $I_x = 800$  in.<sup>4</sup>  
 W24x55  $I_x = 1350$  in.<sup>4</sup>  
 W14x82  $A_g = 24.0$  in.<sup>2</sup>  $I_x = 881$  in.<sup>4</sup>

Manual Table 1-1

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
$\frac{P_u}{A_g} = \frac{250 \text{ kips}}{24.0 \text{ in.}^2} = 10.4 \text{ ksi} < 18 \text{ ksi}$	$\frac{P_a}{A_g} = \frac{167 \text{ kips}}{24.0 \text{ in.}^2} = 6.96 \text{ ksi} < 12 \text{ ksi}$
$\tau = 1.00$	$\tau = 1.00$

Manual Table 4-21

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

Determine  $G_{top}$  and  $G_{bottom}$

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

Commentary  
C2.2

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

Commentary  
C2.2 &  
Fig. C-C2.4

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:

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

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	
$\phi_c P_n = 942 \text{ kips} > 250 \text{ kips}$	<b>o.k.</b>	$P_n / \Omega_c = 627 \text{ kips} > 167 \text{ kips}$	<b>o.k.</b>

Manual  
Table 4-1

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

LRFD	ASD
$P_u = 1.2(100 \text{ kips}) + 1.6(300 \text{ kips})$ $= 600 \text{ kips}$	$P_a = 100 \text{ kips} + 300 \text{ kips}$ $= 400 \text{ kips}$

Calculate the effective length factor,  $K$

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

Manual  
Table 4-21

Commentary  
Section  
C2.2b



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

From the alignment chart,  $K$  is approximately 1.42. 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:

$$KL = \frac{(KL)_x}{\left(\frac{r_x}{r_y}\right)} = \frac{1.42(14.0 \text{ ft})}{2.44} = 8.15 \text{ ft}$$

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

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

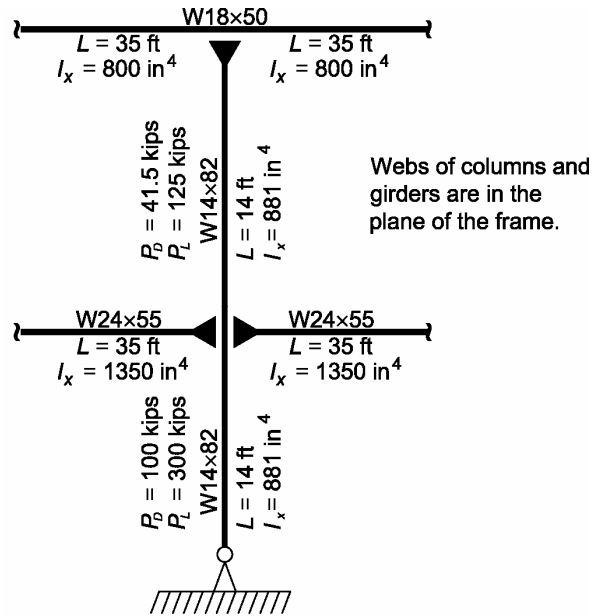
LRFD		ASD	
$\phi_c P_n = 942 \text{ kips} > 600 \text{ kips}$	<b>o.k.</b>	$P_n / \Omega_c = 627 \text{ kips} > 400 \text{ kips}$	<b>o.k.</b>

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 W14x82 from Manual Table 4-1

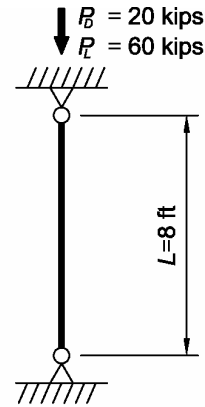
LRFD		ASD	
$\phi_c P_n = 863 \text{ kips} > 600 \text{ kips}$	<b>o.k.</b>	$P_n / \Omega_c = 574 \text{ kips} > 400 \text{ kips}$	<b>o.k.</b>

Manual  
 Table 4-1

### Example E.5 Double Angle Compression Member without Slender Elements

**Given:**

Verify the strength of a 2L4×3½×¾ 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:**

2L4×3½×¾ LLBB

$r_z = 0.719$  in. (single angle)

$r_x = 1.25$  in.

$r_y = 1.55$  in. for ¾ inch separation

Manual  
Table 1-15  
Table 1-7

$r_y = 1.69$  in. for ¾ inch separation

Calculate the required strength

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

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
$\phi_c P_n = 127 \text{ kips} > 120 \text{ kips}$ <b>o.k.</b>	$P_n / \Omega_c = 84.2 \text{ kips} > 80.0 \text{ kips}$ <b>o.k.</b>

Manual  
Table 4-9

For buckling about the y-y axis, the values are tabulated for a separation of ¾ in.

To adjust to a spacing of ¾ in.,  $(KL)_y$  is multiplied by the ratio of the  $r_y$  for a ¾-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	
$\phi_c P_n = 130 \text{ kips} > 120 \text{ kips}$	<b>o.k.</b>	$P_n / \Omega_c = 86.5 \text{ kips} > 80.0 \text{ kips}$	<b>o.k.</b>

Manual  
Table 4-9

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

*Determine the number of intermediate connectors required*

Per Table 4-12, at least two welded or pretensioned bolted intermediate connectors are required. This can be verified as follows:

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

Section E6.2

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.

$$\text{Therefore: } \frac{Ka}{r_i} \leq \frac{3}{4} \left( \frac{KL}{r} \right)_{\max}$$

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

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

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

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

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:

$$2L4 \times 3 \frac{1}{2} \times \frac{3}{8} \text{ LLBB} \quad A_g = 5.34 \text{ in.}^2 \quad r_y = 1.69 \text{ in.} \quad \bar{r}_0 = 2.33 \text{ in.} \quad H = 0.813$$

$$J = 0.132 \text{ in.}^4 \text{ (single angle)} \quad r_y = 1.05 \text{ in. (single angle)}$$

$$\bar{x} = 0.947 \text{ in. (single angle)}$$

Manual  
Table 1-15  
Table 1-7

*Check for slender elements*

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

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

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.

Check flexural buckling about the  $x$ - $x$  axis

Section E3

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

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

Eqn. E3-4

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

$$F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y = \left[ 0.658 \frac{36 \text{ ksi}}{48.5 \text{ ksi}} \right] (36 \text{ ksi}) = 26.4 \text{ ksi} \quad \text{controls}$$

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

$$a = 96.0 \text{ in.} / 3 = 32.0 \text{ in.}$$

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

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

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

Eqn. E6-2

$$= \sqrt{(56.8)^2 + 0.82 \frac{(1.26)^2}{1+(1.26)^2} \left(\frac{32.0 \text{ in.}}{1.05 \text{ in.}}\right)^2} = 60.7 < 134$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(60.7)^2} = 77.7 \text{ ksi}$$

Eqn. E3-4

$$F_{cry} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y = \left[ 0.658 \frac{36 \text{ ksi}}{77.7 \text{ ksi}} \right] (36 \text{ ksi}) = 29.7 \text{ ksi}$$

Eqn. E3-2

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} = \frac{(11,200 \text{ ksi})(2 \text{ angles})(0.132 \text{ in.}^4)}{(5.34 \text{ in.})(2.33 \text{ in.})^2} = 102 \text{ ksi}$$

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

$$= \left( \frac{29.7 \text{ ksi} + 102 \text{ ksi}}{2(0.813)} \right) \left[ 1 - \sqrt{1 - \frac{4(29.7 \text{ ksi})(102 \text{ ksi})(0.813)}{(29.7 \text{ ksi} + 102 \text{ ksi})^2}} \right] = 27.8 \text{ ksi}$$

**does not control**

$$P_n = F_{cr} A_g = (26.4 \text{ ksi}) 5.34 \text{ in}^2 = 141 \text{ kips}$$

Eqn. E4-1

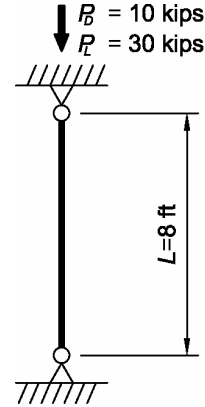
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(141 \text{ kips}) = 127 \text{ kips}$ $> 120 \text{ kips o.k.}$	$\Omega_c = 1.67$ $P_n / \Omega_c = \frac{141 \text{ kips}}{1.67} = 84.4 \text{ kips}$ $> 80.0 \text{ kips o.k.}$

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.  
 $r_y = 1.19$  in. for ⅜ inch separation  
 $r_y = 1.33$  in. for ¾ inch separation

Manual Table 1-15  
Table 1-7

Calculate the required strength

LRFD	ASD
$P_u = 1.2(10.0 \text{ kips}) + 1.6(30.0 \text{ kips}) = 60.0 \text{ kips}$	$P_a = 10.0 \text{ kips} + 30.0 \text{ kips} = 40.0 \text{ kips}$

**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

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

Manual Table 4-9

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
$\phi_c P_{ny} = 65.1 \text{ kips} > 60.0 \text{ kips}$ <b>o.k.</b>	$P_{ny} / \Omega_c = 43.3 \text{ kips} > 40.0 \text{ kips}$ <b>o.k.</b>

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

$$\text{Therefore, } \frac{Ka}{r_i} \leq \frac{3}{4} \left( \frac{KL}{r} \right)_{\max}$$

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

$$r_i = r_z = 0.652 \text{ in.}$$

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

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

**controls**

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

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:

$$2L5 \times 3 \times \frac{1}{4} \text{ LLBB} \quad A_g = 3.88 \text{ in.}^2 \quad r_y = 1.33 \text{ in.} \quad \bar{r}_o = 2.59 \text{ in.} \quad H = 0.657$$

$$J = 0.0438 \text{ in.}^4 \text{ (single angle)} \quad r_y = 0.853 \text{ in. (single angle)}$$

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

Manual  
Tables 1-15,  
1-7 and 4-9



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

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

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

$Q = Q_s(Q_a=1.0)$  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.

$$\begin{aligned} Q_s &= 1.34 - 0.76 \left( \frac{b}{t} \right) \sqrt{\frac{F_y}{E}} \\ &= 1.34 - 0.76(20.0) \sqrt{\frac{36 \text{ ksi}}{29,000 \text{ ksi}}} = 0.804 \end{aligned}$$

Eqn. E7-11

$Q_a = 1.0$  (no stiffened elements)

Therefore,  $Q = Q_s Q_a = 0.804(1.0) = 0.804$

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

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

$$F_e = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(59.3)^2} = 81.4 \text{ ksi}$$

**does not govern**

Eqn. E3-4

Check torsional and flexural-torsional buckling

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

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

Section E6

$$a = 96.0 \text{ in.} / 3 = 32.0 \text{ in.}$$

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

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

$$\begin{aligned} \left(\frac{KL}{r}\right)_m &= \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82 \frac{\alpha^2}{(1+\alpha)^2} \left(\frac{a}{r_{ib}}\right)^2} \\ &= \sqrt{(72.2)^2 + 0.82 \frac{(1.20)^2}{1+(1.20)^2} \left(\frac{32.0 \text{ in.}}{0.853 \text{ in.}}\right)^2} = 76.8 \end{aligned}$$

Eqn. E6-2

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)_m^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(76.8)^2} = 48.6 \text{ ksi}$$

Eqn. E4-10

$$F_{ez} = \left( \frac{\pi^2 EC_w}{(K_z L)^2} + GJ \right) \frac{1}{A_g \bar{r}_o^2}$$

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

$$= \left( \frac{48.6 \text{ ksi} + 37.7 \text{ ksi}}{2(0.657)} \right) \left[ 1 - \sqrt{1 - \frac{4(48.6 \text{ ksi})(37.7 \text{ ksi})(0.657)}{(48.6 \text{ ksi} + 37.7 \text{ ksi})^2}} \right]$$

$$= 26.6 \text{ ksi} \quad \textbf{governs}$$

$0.44QF_y = 0.44(0.804)(36 \text{ ksi}) = 12.7 \text{ ksi} < 26.6 \text{ ksi}$ , therefore Equation E7-2 applies.

$$F_{cr} = Q \left[ 0.658 \frac{QF_y}{F_c} \right] F_y = 0.804 \left[ 0.658 \frac{(0.804)(36 \text{ ksi})}{26.6 \text{ ksi}} \right] (36 \text{ ksi}) = 18.4 \text{ ksi} \quad \text{Eqn. E7-2}$$

$$P_n = F_{cr} A_g = (18.4 \text{ ksi}) 3.88 \text{ in.}^2 = 71.3 \text{ kips} \quad \text{Eqn. E7-1}$$

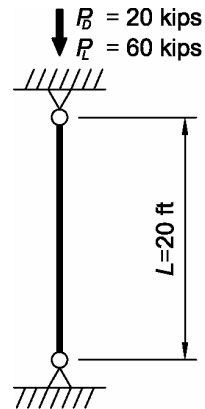
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(71.3 \text{ kips})$ $= 64.1 \text{ kips} > 60.0 \text{ kips} \quad \textbf{o.k.}$	$\Omega_c = 1.67$ $P_n / \Omega_c = \frac{71.3 \text{ kips}}{1.67}$ $= 42.7 \text{ kips} > 40.0 \text{ kips} \quad \textbf{o.k.}$

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
$P_u = 1.2(20.0 \text{ kips}) + 1.6(60.0 \text{ kips}) = 120 \text{ kips}$	$P_a = 20.0 \text{ kips} + 60.0 \text{ kips} = 80.0 \text{ kips}$

**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 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 WT7×34

The available strength in compression is:

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

Manual  
Table 4-7

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:**

WT7×34  $A_g = 9.99 \text{ in.}^2$   $r_x = 1.81 \text{ in.}$   $r_y = 2.46 \text{ in.}$   $\bar{r}_o = 3.19 \text{ in.}$   $J = 1.50 \text{ in.}^4$   
 $H = 0.916$   $d = 7.02 \text{ in.}$   $t_w = 0.415 \text{ in.}$   $b_f = 10.0 \text{ in.}$   $t_f = 0.720 \text{ in.}$

Manual  
Table 1-8

Check for slender elements

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

Table B4.1  
Case 8

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

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

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

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

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(133)^2} = 16.3 \text{ ksi}$$

Eqn. E3-4

$$F_{cr} = 0.877 F_e = 0.877(16.3 \text{ ksi}) = 14.3 \text{ ksi controls}$$

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  $\frac{KL}{r} = \frac{KL}{r_y}$

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

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(97.6)^2} = 30.1 \text{ ksi}$$

Eqn. E3-4

$$F_{cry} = F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y = \left[ 0.658 \frac{50.0 \text{ ksi}}{30.1 \text{ ksi}} \right] 50.0 \text{ ksi} = 24.9 \text{ ksi}$$

Eqn. E3-2

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

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

$$= \left( \frac{24.9 \text{ ksi} + 165 \text{ ksi}}{2(0.916)} \right) \left[ 1 - \sqrt{1 - \frac{4(24.9 \text{ ksi})(165 \text{ ksi})(0.916)}{(24.9 \text{ ksi} + 165 \text{ ksi})^2}} \right]$$

$$= 24.5 \text{ ksi}$$

**does not control**

x-x axis flexural buckling governs, therefore

$$P_n = F_{cr} A_g = (14.3 \text{ ksi}) 9.99 \text{ in.}^2 = 143 \text{ kips} \quad \text{Eqn. E3-1}$$

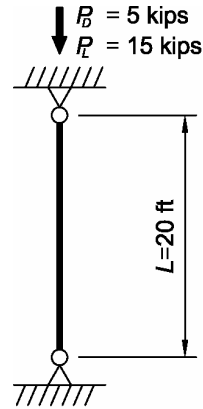
LRFD	ASD
$\phi_c P_n = 0.90(143 \text{ kips})$ $= 128 \text{ kips} > 120 \text{ kips} \quad \mathbf{o.k.}$	$P_n / \Omega_c = \frac{143 \text{ kips}}{1.67}$ $= 85.4 \text{ kips} > 80.0 \text{ kips} \quad \mathbf{o.k.}$

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
$P_u = 1.2(5.00 \text{ kips}) + 1.6(15.0 \text{ kips}) = 30.0 \text{ kips}$	$P_a = 5.00 \text{ kips} + 15.0 \text{ kips} = 20.0 \text{ kips}$

**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 $\times$ 15

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

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

Manual  
Table 4-7

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 $\times$ 15  $A_g = 4.42 \text{ in.}^2$   $r_x = 2.07 \text{ in.}$   $r_y = 1.49 \text{ in.}$   $\bar{r}_o = 2.90 \text{ in.}$   $J = 0.190 \text{ in.}^4$   
 $H = 0.772$   $Q_s = 0.609$   $d = 6.92 \text{ in.}$   $t_w = 0.270 \text{ in.}$   $b_f = 6.73 \text{ in.}$   
 $t_f = 0.385 \text{ in.}$

Manual  
Table 1-8

Check for slender elements

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

Table B4.1  
Case 8

$$\begin{aligned} \frac{b_f}{2t_f} &= \frac{6.73 \text{ in.}}{2(0.385 \text{ in.})} \\ &= 8.74 < 0.56 \sqrt{\frac{E}{F_y}} = 0.56 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 13.4 \end{aligned} \quad \text{therefore, the flange is not slender.}$$

Table B4.1  
Case 3

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

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

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(116)^2} = 21.3 \text{ ksi}$$

**does not control**

Eqn. E3-4

Calculate the critical elastic torsional and flexural-torsional buckling stress

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

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(161)^2} = 11.0 \text{ ksi}$$

Eqn. E4-10

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

Eqn. E4-11

$$F_{ez} = \frac{GJ}{A_g \bar{r}_o^2} = \frac{11,200 \text{ ksi}(0.190 \text{ in.}^4)}{4.42 \text{ in.}^2 (2.90 \text{ in.})^2} = 57.2 \text{ ksi}$$

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

Eqn. E4-5



$$F_e = \left( \frac{11.0 \text{ ksi} + 57.2 \text{ ksi}}{2(0.772)} \right) \left[ 1 - \sqrt{1 - \frac{4(11.0 \text{ ksi})(57.2 \text{ ksi})(0.772)}{(11.0 \text{ ksi} + 57.2 \text{ ksi})^2}} \right] = 10.5 \text{ ksi} \quad \text{controls}$$

The cross section is composed of only unstiffened compression elements.  
Therefore,  $Q_a = 1.0$

$$Q = Q_s (Q_a = 1.0) = 0.609$$

*Check limit for the applicable equation*

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

$$F_{cr} = 0.877F_e = 0.877(10.5 \text{ ksi}) = 9.21 \text{ ksi} \quad \text{Eqn. E7-3}$$

$$P_n = F_{cr}A_g = (9.21 \text{ ksi})4.42 \text{ in.}^2 = 40.7 \text{ kips} \quad \text{Eqn. E7-1}$$

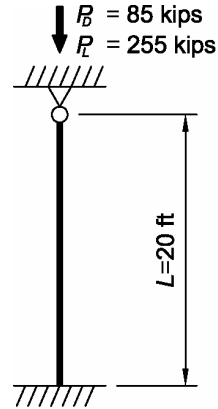
LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(40.7 \text{ kips})$ $= 36.5 \text{ kips} > 30.0 \text{ kips} \quad \text{o.k.}$	$\Omega_c = 1.67$ $P_n / \Omega_c = 40.7 \text{ kips} / 1.67$ $= 24.3 \text{ kips} > 20.0 \text{ kips} \quad \text{o.k.}$

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
$P_u = 1.2(85.0 \text{ kips}) + 1.6(255 \text{ kips}) = 510 \text{ kips}$	$P_a = 85.0 \text{ kips} + 255 \text{ kips} = 340 \text{ kips}$

**Table Solution:**

$$K = 0.8$$

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

Enter Manual Table 4-3 for rectangular sections or Table 4-4 for square sections.

Try an HSS12×10×½

Determine the available strength in axial compression

LRFD	ASD
$\phi_c P_n = 673 \text{ kips} > 510 \text{ kips}$ <b>o.k.</b>	$P_n / \Omega_c = 448 \text{ kips} > 340 \text{ kips}$ <b>o.k.</b>

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:**

$$\text{HSS12} \times 10 \times \frac{1}{2} \quad A_g = 19.0 \text{ in.}^2 \quad r_x = 4.56 \text{ in.} \quad r_y = 3.96 \text{ in.} \quad t_{des} = 0.465 \text{ in.}$$

Manual  
Table 1-11

*Check for slender elements*

Note: 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.

Section B4

*Calculate  $b/t$  of the most slender wall*

$$\frac{h}{t} = \frac{12.0 \text{ in.} - 3(0.465 \text{ in.})}{0.465 \text{ in.}} = 22.8 < 1.40 \sqrt{\frac{E}{F_y}} = 1.40 \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} = 35.2$$

Table B4.1  
Case 12

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*

$$\frac{K_y L}{r_y} = \frac{0.8(20.0 \text{ ft})(12 \text{ in./ft})}{3.96 \text{ in.}} = 48.5$$

$$4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} = 118 > 48.5, \text{ therefore, use Specification Equation E3-2.}$$

Section E3

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000)}{(48.5)^2} = 122 \text{ ksi}$$

Eqn. E3-4

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y = \left(0.658^{\frac{46 \text{ ksi}}{122 \text{ ksi}}}\right) (46 \text{ ksi}) = 39.3 \text{ ksi}$$

Eqn. E3-2

$$P_n = F_{cr} A_g = (39.3 \text{ ksi}) 19.0 \text{ in.}^2 = 746 \text{ kips}$$

Eqn. E3-1

LRFD	ASD
$\phi_c = 0.90$ $\phi_c P_n = 0.90(746 \text{ kips})$ $= 673 \text{ kips} > 510 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_c = 1.67$ $P_n / \Omega_c = \frac{746 \text{ kips}}{1.67}$ $= 448 \text{ kips} > 340 \text{ kips} \quad \mathbf{o.k.}$

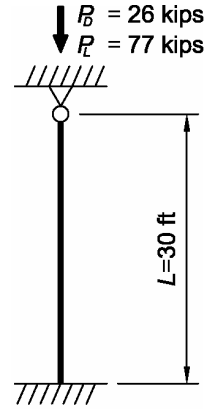
Specification  
Section E1

### 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 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
$P_u = 1.2(26.0 \text{ kips}) + 1.6(77.0 \text{ kips}) = 154 \text{ kips}$	$P_a = 26.0 \text{ kips} + 77.0 \text{ kips} = 103 \text{ kips}$

#### Table Solution:

$$K = 0.8 \quad \text{Therefore } (KL)_x = (KL)_y = 0.8(30.0 \text{ ft}) = 24.0 \text{ ft}$$

Commentary  
Table C-C2.2

Enter Manual Table 4-3, in the HSS12×8 section and proceed across the page until the lightest section is found with an available strength that is equal to or greater than the required strength, in this case a HSS 12×8× $\frac{3}{16}$ .

Determine the available strength in axial compression

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

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:

$$\text{HSS12} \times 8 \times \frac{3}{16} \quad A_g = 6.76 \text{ in.}^2 \quad r_x = 4.56 \text{ in.} \quad r_y = 3.35 \text{ in.} \quad \frac{b}{t} = 43.0$$

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

Manual  
Table 1-11

Determine for slender elements

Calculate the limiting width-thickness ratios

$$\lambda_r = 1.40 \sqrt{\frac{E}{F_y}} = 1.40 \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} = 35.2 < 43.0 \quad \text{therefore both the 10-in. and 12-in. walls are slender elements.}$$

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

$$\begin{aligned} b &= 8.00 \text{ in.} - 3(0.174 \text{ in.}) = 7.48 \text{ in.} \\ h &= 12.0 \text{ in.} - 3(0.174 \text{ in.}) = 11.5 \text{ in.} \end{aligned}$$

Specification  
Section  
B4.2d

For an HSS member with slender elements, the nominal compressive strength,  $P_n$ , shall be 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,  $A_{eff}$

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

Section E7

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 section of uniform thickness,

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

Eqn. E7-18

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

For the 8-in. walls,

$$\begin{aligned} b_e &= 1.92t \sqrt{\frac{E}{F_y}} \left[ 1 - \frac{0.38}{(b/t)} \sqrt{\frac{E}{F_y}} \right] \\ &= 1.92(0.174 \text{ in.}) \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} \left[ 1 - \frac{0.38}{(43.0)} \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} \right] = 6.53 \text{ in.} \leq 7.48 \text{ in.} \end{aligned}$$

Eqn. E7-18

Length that can not be used =  $b - b_e = 7.48 \text{ in.} - 6.53 \text{ in.} = 0.950 \text{ in.}$

For the 12 inch walls.

$$\begin{aligned}
 b_e &= 1.92t \sqrt{\frac{E}{F_y}} \left[ 1 - \frac{0.38}{(b/t)} \sqrt{\frac{E}{F_y}} \right] \\
 &= 1.92(0.174 \text{ in.}) \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} \left[ 1 - \frac{0.38}{(66.0)} \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} \right] = 7.18 \text{ in.} \leq 11.5 \text{ in.}
 \end{aligned}$$

Eqn. E-16

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

Therefore  $A_{eff} = 6.76 \text{ in.}^2 - 2(0.174 \text{ in.})(0.950 \text{ in.}) - 2(0.174 \text{ in.})(4.32 \text{ in.}) = 4.93 \text{ in.}^2$

$$\text{Therefore } Q = Q_a = \frac{A_{eff}}{A} = \frac{4.93 \text{ in.}^2}{6.76 \text{ in.}^2} = 0.729$$

Determine the appropriate equation for  $F_{cr}$

$$\frac{K_y L}{r_y} = \frac{0.8(30.0 \text{ ft})(12 \text{ in./ft})}{3.35 \text{ in.}} = 86.0 < 4.71 \sqrt{\frac{E}{QF_y}} = 4.71 \sqrt{\frac{29,000 \text{ ksi}}{0.729(46 \text{ ksi})}} = 139$$

$$\text{Therefore } F_{cr} = Q \left[ 0.658 \frac{QF_y}{F_e} \right] F_y \quad \text{Eqn. E7-2}$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{86.0^2} = 38.7 \text{ ksi} \quad \text{Eqn. E3-4}$$

$$F_{cr} = Q \left[ 0.658 \frac{QF_y}{F_e} \right] F_y = 0.729 \left[ 0.658 \frac{0.729(46 \text{ ksi})}{38.7 \text{ ksi}} \right] 46 \text{ ksi} = 23.3 \text{ ksi.} \quad \text{Eqn. E7-2}$$

$$P_n = F_{cr} A_g = 23.3 \text{ ksi}(6.76 \text{ in.}^2) = 158 \text{ kips} \quad \text{Eqn. 7-1}$$

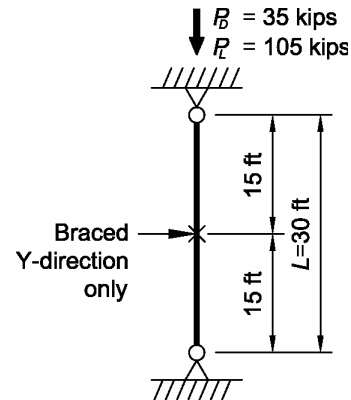
LRFD	ASD	
$\phi_c = 0.90$	$\Omega_c = 1.67$	
$\phi_c P_n = 0.90(158 \text{ kips}) = 142 \text{ kips} < 154 \text{ kips}$	$P_n / \Omega_c = \frac{158 \text{ kips}}{1.67} = 94.7 \text{ kips} < 103 \text{ kips}$	Section E1
<b>See note below</b>	<b>See note below</b>	

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
$P_u = 1.2(35.0 \text{ kips}) + 1.6(105 \text{ kips}) = 210 \text{ kips}$	$P_a = 35.0 \text{ kips} + 105 \text{ kips} = 140 \text{ kips}$

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

Manual  
Table 4-10

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.  $A = 11.1 \text{ in.}^2$   $r = 3.68 \text{ in.}$

Manual  
Table 4-10

All Steel Pipes shown in Manual Table 4-10 are compact at 35 ksi, so no local buckling check is required.

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

Section E3

$$4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{29,000 \text{ ksi}}{35 \text{ ksi}}} = 136 > 97.8$$

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

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
$\phi_c = 0.90$ $\phi_c P_n = 0.90(238 \text{ kips})$ $= 215 \text{ kips} > 210 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_c = 1.67$ $P_n / \Omega_c = \frac{238 \text{ kips}}{1.67}$ $= 143 \text{ kips} > 140 \text{ kips} \quad \mathbf{o.k.}$

Section E1

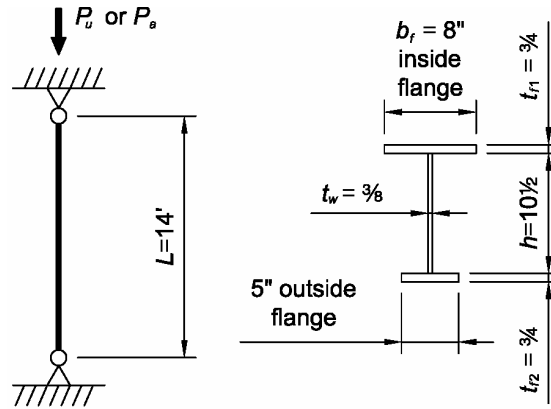
Note that the design procedure would be similar for a round HSS column, except that local buckling should also be checked.



### 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 PL $\frac{3}{4}$ ×5, the inside flange is PL $\frac{3}{4}$ ×8, and the web is PL $\frac{3}{8}$ ×10 $\frac{1}{2}$ . 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  $k_c$*

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

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

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

*Check inside flange slenderness*

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

Table B4.1  
Case 4

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

Check web slenderness

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

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

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

Table B4.1  
Case 14

Calculate section properties (ignoring welds)

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

$$\bar{y} = \frac{\sum A_i y_i}{\sum A_i} = \frac{(6.00 \text{ in.}^2)(11.6 \text{ in.}) + (3.94 \text{ in.}^2)(6.00 \text{ in.}) + (3.75 \text{ in.}^2)(0.375 \text{ in.})}{(6.00 \text{ in.}^2) + (3.94 \text{ in.}^2) + (3.75 \text{ in.}^2)} = 6.92 \text{ in.}$$

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

$$I_x = \left[ \frac{(8.0 \text{ in.})(0.75 \text{ in.})^3}{12} + (8.0 \text{ in.})(0.75 \text{ in.})(4.71 \text{ in.})^2 \right] + \\ \left[ \frac{(0.375 \text{ in.})(10.5 \text{ in.})^3}{12} + (0.375 \text{ in.})(10.5 \text{ in.})(0.92 \text{ in.})^2 \right] + \\ \left[ \frac{(5.0 \text{ in.})(0.750 \text{ in.})^3}{12} + (5.0 \text{ in.})(0.750 \text{ in.})(6.55 \text{ in.})^2 \right] = 334 \text{ in.}^4$$

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

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

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

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

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 (29,000 \text{ ksi})}{(34.0)^2} = 247 \text{ ksi} \quad \text{does not control} \quad \text{Eqn. E3-4}$$

Calculate the flexural-torsional critical elastic buckling stress

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

$$h_o = d - \frac{t_f}{2} - \frac{t_f}{2} = 12.0 \text{ in.} - \frac{0.750 \text{ in.}}{2} - \frac{0.750 \text{ in.}}{2} = 11.3 \text{ in.}$$

$$C_w = \frac{t_f h_o^2}{12} \left( \frac{b_1^3 b_2^3}{b_1^3 + b_2^3} \right) = \frac{(0.750 \text{ in.})(11.25 \text{ in.})^2}{12} \left( \frac{(8.00 \text{ in.})^3 (5.00 \text{ in.})^3}{(8.00 \text{ in.})^3 + (5.00 \text{ in.})^3} \right) = 795 \text{ in.}^6$$

Locate the shear center

Due to symmetry, both the centroid and the shear center lie on the y-axis. Therefore  $x_o = 0$

$$e = h_o \left( \frac{b_1^3}{b_1^3 + b_2^3} \right) = 11.3 \text{ in.} \left( \frac{(8.00 \text{ in.})^3}{(8.00 \text{ in.})^3 + (5.00 \text{ in.})^3} \right) = 9.04 \text{ in.}$$

Note that this shear center location is taken from the center of the outside flange. Therefore, add one-half the flange thickness to determine the shear center location measured from the bottom of the outside flange.

$$e + \frac{t_f}{2} = 9.04 \text{ in.} + \frac{0.75 \text{ in.}}{2} = 9.42 \text{ in.}$$

$$\text{Therefore } y_o = \left( e + \frac{t_f}{2} \right) - \bar{y} = 9.42 \text{ in.} - 6.92 \text{ in.} = 2.50 \text{ in.}$$

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} = 0 + (2.50 \text{ in.})^2 + \frac{334 \text{ in.}^4 + 39.9 \text{ in.}^4}{13.7 \text{ in.}^2} = 33.5 \text{ in.}^2$$

Eqn. E4-7

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} = 1 - \frac{0 + (2.50 \text{ in.})^2}{33.5 \text{ in.}^2} = 0.813$$

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

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

$$= \left[ \frac{\pi^2 (29,000 \text{ ksi})(795 \text{ in.}^6)}{[(1.0)(14.0 \text{ ft})(12 \text{ in./ft})]^2} + (11,200 \text{ ksi})(2.01 \text{ in.}^4) \right] \left( \frac{1}{(13.7 \text{ in.}^2)(33.5 \text{ in.}^2)} \right) = 66.6 \text{ ksi}$$

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

$$= \left( \frac{29.7 \text{ ksi} + 66.6 \text{ ksi}}{2(0.813)} \right) \left[ 1 - \sqrt{1 - \frac{4(29.7 \text{ ksi})(66.6 \text{ ksi})(0.813)}{(29.7 \text{ ksi} + 66.6 \text{ ksi})^2}} \right]$$

$$= 26.4 \text{ ksi} \quad \text{controls}$$

*Torsional and flexural-torsional buckling governs*

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

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

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

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

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 sections. LRFD and ASD information is presented side by side.

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

### F1. GENERAL PROVISIONS

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

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

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}.$$

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 many 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 moment capacity,  $M_n$ , 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 area 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 area 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  $M_n = 0.7M_y = 0.7F_y S_x$  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 not more than  $M_p$  as shown in the figure.

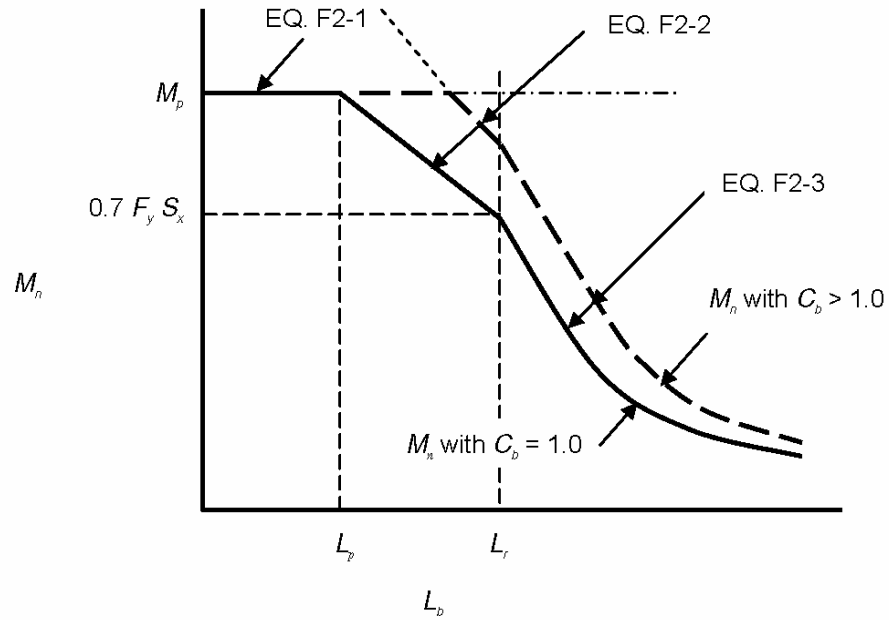


Figure F-1. Beam Strength versus Unbraced Length

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

$$M_n = C_b \left[ M_p - (M_p - 0.70 F_y S_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad \text{Eqn. F2-2}$$

$$M_n = F_{cr} S_x \leq M_p; \text{ where } F_{cr} \text{ is evaluated as shown below} \quad \text{Eqn. F2-3}$$

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

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 in with  $F_y = 50$  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 = 96%	W12×65 = 98%
W10×12 = 99%	W8×31 = 99%	W8×10 = 99%	W6×15 = 94%
W6×8.5 = 97%	M4×6 = 85%		

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  $\lambda_{pf}$  is the plastic yielding strength,  $M_p$ . The curved portion to the right of  $\lambda_{rf}$  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.

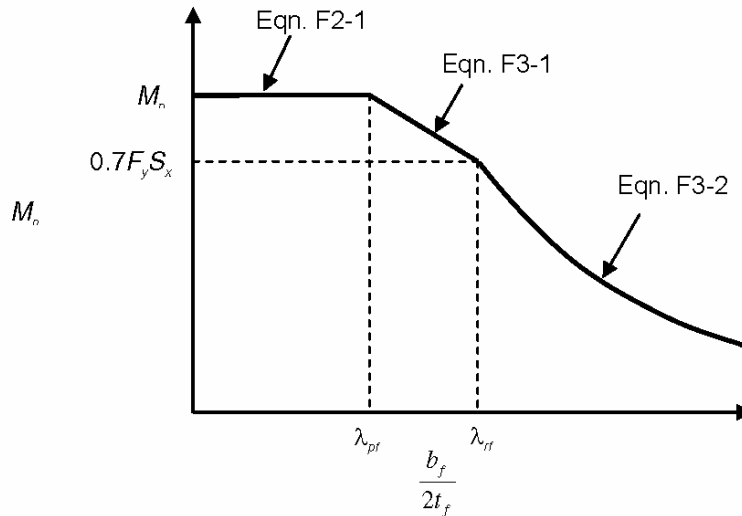


Figure F-2 Flange Local Buckling Strength

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

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

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \tag{Eqn. F3-2}$$

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 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**.

#### **F8. ROUND HSS AND PIPES**

The definition of HSS encompasses both tube and pipe products. The lateral-torsion buckling limit state does not apply, but round HSS are subject to strength reductions from local buckling. Available strengths of round HSS and Pipe are listed in Manual Table 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  $M_n = 0.424 \frac{EJ}{d}$ . 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**.

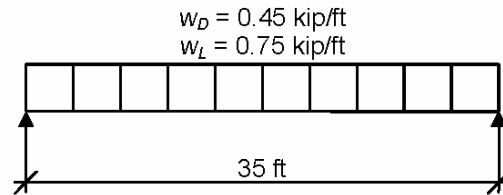
**F13. PROPORTIONS FOR BEAMS AND GIRDERS**

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     $F_y = 50 \text{ ksi}$      $F_u = 65 \text{ ksi}$

Manual  
Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.450 \text{ kip/ft}) + 1.6(0.750 \text{ kip/ft})$ $= 1.74 \text{ kip/ft}$	$w_a = 0.450 \text{ kip/ft} + 0.750 \text{ kip/ft}$ $= 1.20 \text{ kip/ft}$
$M_u = \frac{1.74 \text{ kip/ft}(35.0 \text{ ft})^2}{8} = 266 \text{ kip-ft}$	$M_a = \frac{1.20 \text{ kip/ft}(35.0 \text{ ft})^2}{8} = 184 \text{ kip-ft}$

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

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

$$I_{x(reqd)} = \frac{5wl^4}{384E\Delta_{max}} = \frac{5(0.750 \text{ kip/ft})(35.0 \text{ ft})^4(12 \text{ in./ft})^3}{384(29,000 \text{ ksi})(1.17 \text{ in.})} = 748 \text{ in.}^4$$

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
$\phi_b M_n = \phi_b M_{px} = 379 \text{ kip-ft} > 266 \text{ kip-ft}$ <b>o.k.</b>	$\frac{M_n}{\Omega_b} = \frac{M_{px}}{\Omega_b} = 252 \text{ kip-ft} > 184 \text{ kip-ft}$ <b>o.k.</b>
$I_x = 800 \text{ in.}^4 > 748 \text{ in.}^4$ <b>o.k.</b>	

Manual  
Table 3-2

Manual  
Table 3-2

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

#### Given:

Example F.1-1a 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:

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

Manual  
Table 2-3

Geometric Properties:

$$\text{W18} \times 50 \quad Z_x = 101 \text{ in.}^3$$

Manual  
Table 1-1

Required strength from Example F.1-1a

LRFD	ASD
$M_u = 266 \text{ kip-ft}$	$M_a = 184 \text{ kip-ft}$

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.

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

Eqn. F2-1

Calculate the available flexural strength

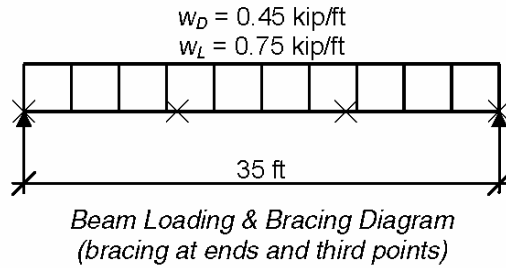
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(421 \text{ kip-ft})$ $= 379 \text{ kip-ft} > 266 \text{ kip-ft} \quad \mathbf{o.k.}$	$\Omega_b = 1.67$ $M_n / \Omega_b = (421 \text{ kip-ft}) / 1.67$ $= 252 \text{ kip-ft} > 184 \text{ kip-ft} \quad \mathbf{o.k.}$

Section F1

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

**Given:**

Verify the strength of the W18×50 beam selected in Example F.1-1a if the beam is braced at the ends and third points rather than continuously braced.



**Solution:**

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

LRFD	ASD
$M_u = 266 \text{ kip-ft}$	$M_a = 184 \text{ kip-ft}$

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

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
$\phi_b M_n \approx 302 \text{ kip-ft} > 266 \text{ kip-ft} \quad \mathbf{o.k.}$	$\frac{M_n}{\Omega_b} \approx 201 \text{ kip-ft} > 184 \text{ kip-ft} \quad \mathbf{o.k.}$

Manual  
Table 3-10

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

#### Given:

Example F.1-2a was 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:

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

Manual  
Table 2-3

##### Geometric Properties:

$$\text{W18} \times 50 \quad S_x = 88.9 \text{ in.}^3$$

Manual  
Table 1-1

Required strength from Example F.1-2a

LRFD	ASD
$M_u = 266 \text{ kip-ft}$	$M_a = 184 \text{ kip-ft}$

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

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

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.

$$L_p = 5.83 \text{ ft}$$

$$L_r = 17.0 \text{ ft}$$

Manual  
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  $L_p < L_b \leq L_r$ , the lesser of either the flexural yielding limit-state or the inelastic lateral-torsional buckling limit-state controls the nominal strength.

$$M_p = 5050 \text{ kip-in. (from Example F.1-2a)}$$

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

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

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

$$= 4070 \text{ kip-in. or } 339 \text{ kip-ft}$$

Calculate the available flexural strength

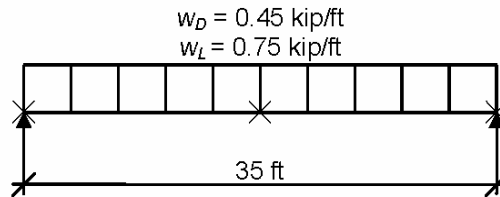
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(339 \text{ kip-ft})$ $= 305 \text{ kip-ft} > 266 \text{ kip-ft}$ <b>o.k.</b>	$\Omega_b = 1.67$ $M_n / \Omega_b = (339 \text{ kip-ft}) / 1.67$ $= 203 \text{ kip-ft} > 184 \text{ kip-ft}$ <b>o.k.</b>

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 beam selected in Example F.1-1a if the beam is braced at the ends and center point rather than continuously braced.



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

**Solution:**

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

LRFD	ASD
$M_u = 266 \text{ kip-ft}$	$M_a = 184 \text{ kip-ft}$

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

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 in from Manual Table 3-10 may be multiplied by  $C_b$ , but may not exceed the value of the horizontal portion ( $\phi M_n$  for LRFD,  $M_n/\Omega$  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 11.7 ft. Obtain the available strength from the appropriate vertical scale to the left.

LRFD	ASD
$\phi_b M_n \approx 222 \text{ kip-ft}$	$M_n / \Omega_b \approx 147 \text{ kip-ft}$
$\phi_b M_p \approx 379 \text{ kip-ft}$ (upper limit on $C_b M_n$ )	$M_p / \Omega_b \approx 252 \text{ kip-ft}$ (upper limit on $C_b M_n$ )
<i>Adjust for <math>C_b</math></i>	<i>Adjust for <math>C_b</math></i>
$(1.30)(222 \text{ kip-ft}) = 288 \text{ kip-ft}$	$(1.30)(147 \text{ kip-ft}) = 191 \text{ kip-ft}$

Manual  
Table 3-10

<p><i>Check Limit</i></p> <p><math>288 \text{ kip-ft} \leq \phi_b M_p = 379 \text{ kip-ft}</math> <b>o.k.</b></p> <p><i>Check available versus required strength</i></p> <p><math>288 \text{ kip-ft} &gt; 266 \text{ kip-ft}</math> <b>o.k.</b></p>	<p><i>Check Limit</i></p> <p><math>191 \text{ kip-ft} \leq M_p / \Omega_b = 252 \text{ kip-ft}</math> <b>o.k.</b></p> <p><i>Check available versus required strength</i></p> <p><math>191 \text{ kip-ft} &gt; 184 \text{ kip-ft}</math> <b>o.k.</b></p>
---	---

*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
$M_u' = 266 \text{ kip-ft} / 1.3 = 205 \text{ kip-ft}$	$M_a' = 184 \text{ kip-ft} / 1.3 = 142 \text{ kip-ft}$

*Obtain the available strength for a W18x50 with an unbraced length of 17.5 ft from Manual Table 3-10*

LRFD	ASD
$\phi_b M_n \approx 222 \text{ kip-ft} > 205 \text{ kip-ft}$ <b>o.k.</b>	$M_n / \Omega_b \approx 147 \text{ kip-ft} > 142 \text{ kip-ft}$ <b>o.k.</b>
$\phi_b M_p \approx 379 \text{ kip-ft} > 266 \text{ kips}$ <b>o.k.</b>	$M_p / \Omega_b \approx 252 \text{ kip-ft} > 184 \text{ kips}$ <b>o.k.</b>

Manual  
Table 3-10



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

#### Given:

Example F.1-3a was 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:

#### Geometric Properties:

$$W18 \times 50 \quad r_{ts} = 1.98 \text{ in.} \quad S_x = 88.9 \text{ in.}^3 \quad J = 1.24 \text{ in.}^4 \quad h_o = 17.4 \text{ in.}$$

Required strength from Example F.1-3a

LRFD	ASD
$M_u = 266 \text{ kip-ft}$	$M_a = 184 \text{ kip-ft}$

Calculate the nominal flexural strength,  $M_n$

Calculate  $C_b$

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

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

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

$$L_p = 5.83 \text{ ft}$$

$$L_r = 17.0 \text{ ft}$$

Manual  
Table 3-6

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  $L_b = 17.5 \text{ ft}$

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

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

$$M_n = F_{cr} S_x \leq M_p$$

Eqn. F2-3

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

$$M_n = 3840 \text{ kip-in or } 320 \text{ kip-ft}$$

Calculate the available flexural strength

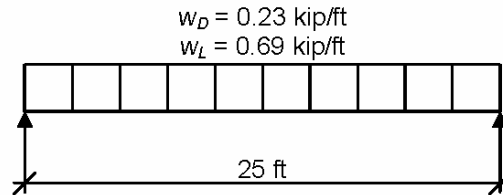
LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(320 \text{ kip-ft}) = 288 \text{ kip-ft}$	$\frac{M_n}{\Omega_b} = \frac{320 \text{ kip-ft}}{1.67} = 192 \text{ kip-ft}$
$288 \text{ kip-ft} > 266 \text{ kip-ft}$ <b>o.k.</b>	$192 \text{ kip-ft} > 184 \text{ kip-ft}$ <b>o.k.</b>

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     $F_y = 36 \text{ ksi}$      $F_u = 58 \text{ ksi}$

Manual  
Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.230 \text{ kip/ft}) + 1.6(0.690 \text{ kip/ft})$ $= 1.38 \text{ kip/ft}$	$w_a = 0.230 \text{ kip/ft} + 0.690 \text{ kip/ft}$ $= 0.920 \text{ kip/ft}$
$M_u = \frac{1.38 \text{ kip/ft}(25.0 \text{ ft})^2}{8} = 108 \text{ kip-ft}$	$M_a = \frac{0.920 \text{ kip/ft}(25.0 \text{ ft})^2}{8} = 71.9 \text{ kip-ft}$

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. Select C15×33.9 from Manual Table 3-8.

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

Check live load deflection

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

For C15×33.9,  $I_x = 315 \text{ in.}^4$

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

Manual  
Table 1-5

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:

$$\text{ASTM A36} \quad F_y = 36 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Manual  
Table 2-3

Geometric Properties:

$$\text{C15} \times 33.9 \quad Z_x = 50.8 \text{ in.}^3$$

Manual  
Table 1-5

Required strength from Example F.2-1a

LRFD	ASD
$M_u = 108 \text{ kip-ft}$	$M_a = 71.9 \text{ kip-ft}$

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.

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

Eqn. F2-1

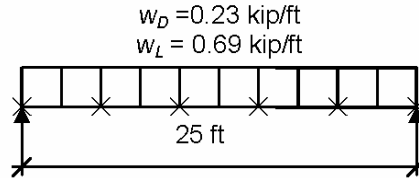
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(152 \text{ kip-ft})$ $= 137 \text{ kip-ft} > 108 \text{ kip-ft} \quad \mathbf{o.k.}$	$\Omega_b = 1.67$ $M_n/\Omega_b = 152 \text{ kip-ft} / 1.67$ $= 91.3 \text{ kip-ft} > 71.9 \text{ kip-ft} \quad \mathbf{o.k.}$

Section F1

### 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 & 1/5 points)*

**Solution:**

**Material Properties:**

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

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.

LRFD	ASD
$\phi_b M_n \approx 130$ kip-ft > 108 kip-ft <b>o.k.</b>	$M_n / \Omega_b \approx 87.0$ kip-ft > 71.9 kip-ft <b>o.k.</b>

Manual Table 3-11

### 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:

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

Manual  
Table 2-3

##### Geometric Properties:

C15×33.9  $S_x = 42.0$  in.<sup>3</sup>

Manual  
Table 3-8

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  $L_p < L_b \leq L_r$ , 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.2-2a,  $M_p = 1830$  kip-in.

$$M_n = C_b \left[ M_p - (M_p - 0.7F_y S_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p$$

Eqn. F2-2

$$M_n = 1.0 \left[ 1830 \text{ kip-in.} - (1830 \text{ kip-in.} - 0.7(36 \text{ ksi})(42.0 \text{ in.}^3)) \left( \frac{5.00 \text{ ft} - 3.75 \text{ ft}}{14.5 \text{ ft} - 3.75 \text{ ft}} \right) \right] \leq 1830 \text{ kip-in.}$$

$$= 1740 \text{ kip-in.} < 1830 \text{ kip-in.} \quad \mathbf{o.k.}$$

$M_n = 1740$  kip-in. or 145 kip-ft

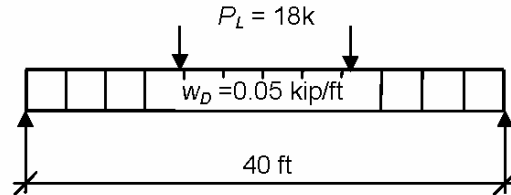
Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(145 \text{ kip-ft}) = 131 \text{ kip-ft}$	$\frac{M_n}{\Omega_b} = \frac{145 \text{ kip-ft}}{1.67} = 86.8 \text{ kip-ft}$
$131 \text{ kip-ft} > 108 \text{ kip-ft}$ <b>o.k.</b>	$86.8 \text{ kip-ft} > 71.9 \text{ kip-ft}$ <b>o.k.</b>

### 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:

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

Calculate the required flexural strength at midspan

LRFD	ASD
$w_u = 1.2(0.0500 \text{ kip/ft}) = 0.0600 \text{ kip/ft}$ $P_u = 1.6(18.0 \text{ kips}) = 28.8 \text{ kips}$ $M_u = \frac{(0.0600 \text{ kip/ft})(40.0 \text{ ft})^2}{8}$ $\quad + (28.8 \text{ kips})\frac{40.0 \text{ ft}}{3}$ $= 396 \text{ kip-ft}$	$w_a = 0.0500 \text{ kip/ft}$ $P_a = 18.0 \text{ kips}$ $M_a = \frac{(0.0500 \text{ kip/ft})(40.0 \text{ ft})^2}{8}$ $\quad + (18.0 \text{ kips})\frac{40.0 \text{ ft}}{3}$ $= 250 \text{ kip-ft}$

Select the lightest section with the required strength from the bold entries in Manual Table 3-2

Select W21×48.

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 3-2

Manual  
Table 1-1



Check the available strength

LRFD	ASD
$\phi_b M_n = \phi_b M_{px} = 398 \text{ kip-ft} > 396 \text{ kip-ft}$ <b>o.k.</b>	$\frac{M_n}{\Omega_b} = \frac{M_{px}}{\Omega_b} = 265 \text{ kip-ft} > 250 \text{ kip-ft}$ <b>o.k.</b>

Manual  
Table 3-2

Note: the value  $M_{px}$  in Table 3-2 include strength reductions due the noncompact nature of the shape

Calculate deflection

$$I_x = 959 \text{ in.}^4$$

Manual  
Table 1-1

$$\begin{aligned} \Delta_{\max} &= \frac{5wl^4}{384EI} + \frac{Pl^3}{28EI} \\ &= \frac{5(0.0500 \text{ kip/ft})(40.0 \text{ ft})^4 (12 \text{ in./ft})^3}{384 (29,000 \text{ ksi})(959 \text{ in.}^4)} + \frac{18.0 \text{ kips}(40.0 \text{ ft})^3 (12 \text{ in./ft})^3}{28(29,000 \text{ ksi})(959 \text{ in.}^4)} \\ &= 2.66 \text{ in.} \end{aligned}$$

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:

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

#### Geometric Properties:

$$\text{W21} \times 48 \quad S_x = 93.0 \text{ in.}^3 \quad Z_x = 107 \text{ in.}^3$$

Manual  
Table 2-3

Check flange slenderness

$$\lambda = \frac{b_f}{2t_f} = 9.47$$

Manual  
Table 1-1

The limiting width-thickness ratios for the compression flange are:

$$\lambda_{pf} = 0.38 \sqrt{\frac{E}{F_y}} = 0.38 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 9.15$$

Manual  
Table 1-1

$$\lambda_{rf} = 1.0 \sqrt{\frac{E}{F_y}} = 1.0 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 24.1$$

Table B4.1  
Case 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

$$M_p = F_y Z_x = 50 \text{ ksi}(107 \text{ in.}^3) = 5350 \text{ kip-in. or } 446 \text{ kip-ft.}$$

$$M_n = \left[ M_p - (M_p - 0.7F_y S_x) \left( \frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right) \right]$$

Eqn. F3-1

$$\begin{aligned} M_n &= \left[ 5350 \text{ kip-in.} - (5350 \text{ kip-in.} - 0.7(50 \text{ ksi})(93.0 \text{ in.}^3)) \left( \frac{9.47 - 9.15}{24.1 - 9.15} \right) \right] \\ &= 5310 \text{ kip-in. or } 442 \text{ kip-ft.} \end{aligned}$$

Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(442 \text{ kip-ft})$ $= 398 \text{ kip-ft} > 396 \text{ kip-ft}$ <b>o.k.</b>	$\Omega_b = 1.67$ $M_n/\Omega_b = 442 \text{ kip-ft} / 1.67$ $= 265 \text{ kip-ft} > 250 \text{ kip-ft}$ <b>o.k.</b>

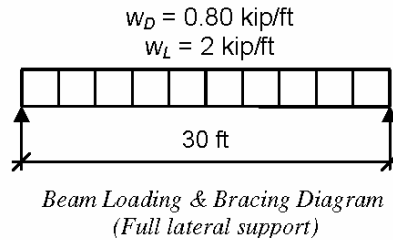
Section F1

Note that these available strength values are identical to the tabulated values in Manual Table 3-2, which account for the non-compact 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.



#### Solution:

#### Material Properties:

ASTM A992       $F_y = 50 \text{ ksi}$        $F_u = 65 \text{ ksi}$

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.800 \text{ kip/ft}) + 1.6(2.00 \text{ kip/ft})$ $= 4.16 \text{ kip/ft}$	$w_a = 0.800 \text{ kip/ft} + 2.00 \text{ kip/ft}$ $= 2.80 \text{ kip/ft}$
$M_u = \frac{4.16 \text{ kip/ft}(30.0 \text{ ft})^2}{8} = 468 \text{ kip-ft}$	$M_a = \frac{2.80 \text{ kip/ft}(30.0 \text{ ft})^2}{8} = 315 \text{ kip-ft}$

Calculate the minimum required moment of inertia

The maximum deflection,  $\Delta_{max}$ , occurs at mid-span and is calculated as

$$\Delta_{max} = \frac{5wl^4}{384EI}$$

Rearranging and substituting  $\Delta_{max} = 1.00 \text{ in.}$

$$I_{min} = \frac{5(2.00 \text{ kips/ft})(30.0 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29,000 \text{ ksi})(1.00 \text{ in.})} = 1,260 \text{ in.}^4$$

Select the lightest section with the required moment of inertia from the bold entries in Manual Table 3-3

Select a W24×55

$$I_x = 1,350 \text{ in.}^4 > 1,260 \text{ in.}^4 \quad \mathbf{o.k.}$$

Because the W24×55 is continuously braced and compact, its strength is governed by the yielding limit state and Section F2.1

Manual  
Table 3-23  
Diagram 1

Manual  
Table 1-1

Obtain the available strength from Manual Table 3-2

LRFD	ASD
$\phi_b M_n = \phi_b M_{px} = 503 \text{ kip-ft}$  $503 \text{ kip-ft} > 468 \text{ kip-ft}$ <b>o.k.</b>	$M_n/\Omega_b = M_{px}/\Omega_b = 334 \text{ kip-ft}$  $334 \text{ kip-ft} > 315 \text{ kip-ft}$ <b>o.k.</b>

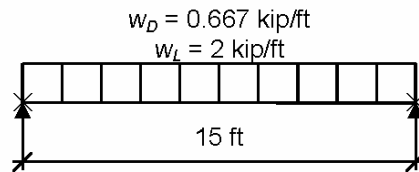
Manual  
Table 3-2

### 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     $F_y = 50 \text{ ksi}$      $F_u = 65 \text{ ksi}$

Manual  
Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.667 \text{ kip/ft}) + 1.6(2.00 \text{ kip/ft})$ $= 4.00 \text{ kip/ft}$	$w_a = 0.667 \text{ kip/ft} + 2.00 \text{ kip/ft}$ $= 2.67 \text{ kip/ft}$
$M_u = \frac{4.00 \text{ kip/ft}(15.0 \text{ ft})^2}{8} = 113 \text{ kip-ft}$	$M_a = \frac{2.67 \text{ kip/ft}(15.0 \text{ ft})^2}{8} = 75.0 \text{ kip-ft}$

Calculate the minimum required moment of inertia

$$\Delta_{max} = \frac{L}{240} = \frac{15.0 \text{ ft}(12 \text{ in./ft})}{240} = 0.750 \text{ in.}$$

$$I_{req} = \frac{5wl^4}{384E\Delta_{max}} = \frac{5(2.00 \text{ kip/ft})(15.0 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29,000 \text{ ksi})(0.750 \text{ in.})} = 105 \text{ in.}^4$$

Manual  
Table 3-23  
Diagram 1

Select the lightest section from the bold entries in Manual Table 3-3, due to the likelihood that deflection will govern this design.

Try a W12×58

**Geometric Properties:**

$$W12\times 58 \quad S_y = 21.4 \text{ in.}^3 \quad Z_y = 32.5 \text{ in.}^3 \quad I_y = 107 \text{ in.}^4$$

$$I_y = 107 \text{ in.}^4 > 105 \text{ in.}^4 \quad \mathbf{o.k.}$$

Manual  
Table 3-3  
Manual  
Table 1-1

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.

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y$$

Eqn. F6-1

$$= 50 \text{ ksi}(32.5 \text{ in.}^3) \leq 1.6(50 \text{ ksi})(21.4 \text{ in.}^3)$$

$$= 1630 \text{ kip-in.} \leq 1710 \text{ kip-in.} \quad \mathbf{o.k.}$$

$$= 1630 \text{ kip-in. or } 135 \text{ kip-ft}$$

Calculate the available flexural strength

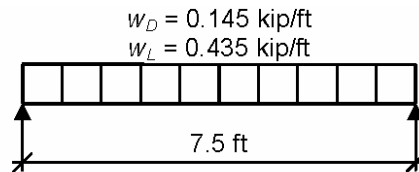
LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(135 \text{ kip-ft}) = 122 \text{ kip-ft}$	$M_n/\Omega_b = \frac{135 \text{ kip-ft}}{1.67} = 81.1 \text{ kip-ft}$
$122 \text{ kip-ft} > 113 \text{ kip-ft} \quad \mathbf{o.k.}$	$81.1 \text{ kip-ft} > 75.0 \text{ kip-ft} \quad \mathbf{o.k.}$

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

Calculate the required strength

LRFD	ASD
$w_u = 1.2(0.145 \text{ kip/ft}) + 1.6(0.435 \text{ kip/ft})$ $= 0.870 \text{ kip/ft}$	$w_a = 0.145 \text{ kip/ft} + 0.435 \text{ kip/ft}$ $= 0.580 \text{ kip/ft}$
$M_u = \frac{(0.870 \text{ kip/ft})(7.50 \text{ ft})^2}{8} = 6.12 \text{ kip-ft}$	$M_a = \frac{(0.580 \text{ kip/ft})(7.50 \text{ ft})^2}{8} = 4.08 \text{ kip-ft}$

Calculate the minimum required moment of inertia

$$\Delta_{max} = \frac{L}{240} = \frac{7.50 \text{ ft}(12 \text{ in./ft})}{240} = 0.375 \text{ in.}$$

$$I_{req} = \frac{5wl^4}{384E\Delta_{max}}$$

$$= \frac{5(0.435 \text{ kip/ft})(7.50 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29,000 \text{ ksi})(0.375 \text{ in.})} = 2.85 \text{ in.}^4$$

Manual  
Table 3-23  
Diagram 1

Select an HSS with a minimum  $I_x$  of  $2.85 \text{ in.}^4$ , using Manual Table 1-12, having adequate available strength, using Manual Table 3-13.

Select HSS3×3×¼

$$I_x = 3.02 \text{ in.}^4 > 2.85 \text{ in.}^4 \quad \text{o.k.}$$

Manual  
Table 1-12

Obtain the required strength from Table 3-13

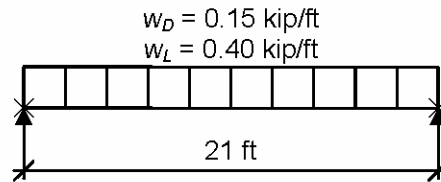
LRFD	ASD
$\phi_b M_n = 8.55 \text{ kip-ft} > 6.12 \text{ kip-ft} \quad \text{o.k.}$	$M_n / \Omega_b = 5.69 \text{ kip-ft} > 4.08 \text{ kip-ft} \quad \text{o.k.}$

Manual  
Table 3-13

### 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

Calculate the required strength

LRFD	ASD
$w_u = 1.2(0.150 \text{ kip/ft}) + 1.6(0.400 \text{ kip/ft})$ $= 0.820 \text{ kip/ft}$	$w_a = 0.150 \text{ kip/ft} + 0.400 \text{ kip/ft}$ $= 0.550 \text{ kip/ft}$
$M_u = \frac{0.820 \text{ kip/ft} (21.0 \text{ ft})^2}{8} = 45.2 \text{ kip-ft}$	$M_a = \frac{0.550 \text{ kip/ft} (21.0 \text{ ft})^2}{8} = 30.3 \text{ kip-ft}$

Calculate the minimum moment of inertia

$$\Delta_{max} = \frac{L}{240} = \frac{21.0 \text{ ft} (12 \text{ in./ft})}{240} = 1.05 \text{ in.}$$

$$\Delta_{max} = \frac{5wl^4}{384EI}$$

Rearranging and substituting  $\Delta_{max} = 1.05$  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 HSS with a minimum  $I_x$  of 57.5 in.<sup>4</sup>, using Manual Table 1-11, having adequate available strength, using Manual Table 3-12.

Select a HSS10×6× $\frac{3}{16}$  oriented in the strong direction. This rectangular HSS section was purposely selected for illustration purposes because it has a noncompact flange as noted by footnote "f" in Manual Table 3-12.

$$I_x = 74.6 \text{ in.}^4 > 57.5 \text{ in.}^4 \quad \text{o.k.}$$

Manual  
Table 3-18  
Diagram 1

Manual



Obtain the required strength from Table 3-12

Table 1-11

LRFD	ASD
$\phi_b M_n = 57.0 \text{ kip-ft} > 45.2 \text{ kip-ft}$ <b>o.k.</b>	$M_n / \Omega_b = 37.9 \text{ kip-ft} > 30.3 \text{ kip-ft}$ <b>o.k.</b>

Manual  
Table 3-12

## Example F.7b HSS Flexural Member with Noncompact Flanges

### Given:

Notice that in Example F.8b 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:

$$\text{ASTM A500 Gr. B} \quad F_y = 46 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Manual  
Table 2-3

#### Geometric Properties:

$$\text{HSS10} \times 6 \times \frac{3}{16} \quad Z_x = 18.0 \text{ in.}^3 \quad S_x = 14.9 \text{ in.}^3$$

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

$$\lambda_p = 2.42 \sqrt{\frac{E}{F_y}} = 2.42 \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} = 60.8 > 54.5, \text{ therefore the web is compact.}$$

Table B4.1  
Case 13

For HSS with non-compact flanges and compact webs, Specification Section F7.2(b) applies.

$$M_n = M_p - (M_p - F_y S) \left( 3.57 \frac{b}{t} \sqrt{\frac{F_y}{E}} - 4.0 \right) \leq M_p$$

Eqn. F7-2

$$M_p = F_y Z = 46 \text{ ksi}(18.0 \text{ in.}^3) = 828 \text{ kip-in.}$$

$$M_n = (828 \text{ kip-in.}) - \left[ (828 \text{ kip-in.}) - (46 \text{ ksi})(14.9 \text{ in.}^3) \right] \left( 3.57(31.5) \sqrt{\frac{46 \text{ ksi}}{29,000 \text{ ksi}}} - 4.0 \right)$$

$$= 760 \text{ kip-in. or } 63.3 \text{ kip-ft}$$

Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(63.3 \text{ kip-ft}) = 57.0 \text{ kip-ft}$	$M_n / \Omega_b = \frac{63.3 \text{ kip-ft}}{1.67} = 37.9 \text{ kip-ft}$

Section F1

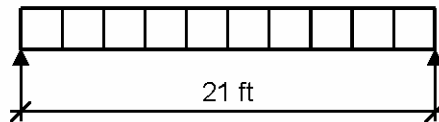
### Example F.8a HSS Flexural Member with Slender Flanges

#### 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  $L/240$ .

$$w_D = 0.125 \text{ kip/ft}$$

$$w_L = 0.375 \text{ kip/ft}$$



Beam Loading & Bracing Diagram  
(Full lateral support)

#### Solution:

#### Material Properties:

$$\text{ASTM A500 Gr. B} \quad F_y = 46 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.125 \text{ kip/ft}) + 1.6(0.375 \text{ kip/ft})$ $= 0.750 \text{ kip/ft}$	$w_a = 0.125 \text{ kip/ft} + 0.375 \text{ kip/ft}$ $= 0.500 \text{ kip/ft}$
$M_u = \frac{(0.750 \text{ kip/ft})(21.0 \text{ ft})^2}{8} = 41.3 \text{ kip-ft}$	$M_a = \frac{(0.500 \text{ kip/ft})(21.0 \text{ ft})^2}{8} = 27.6 \text{ kip-ft}$

Obtain the flexural strength of the HSS8×8×3/16 from Manual Table 3-13

LRFD	ASD
$\phi_b M_n = 45.3 \text{ kip-ft} > 41.3 \text{ kip-ft} \quad \mathbf{o.k.}$	$M_n / \Omega_b = 30.1 \text{ kip-ft} > 27.6 \text{ kip-ft} \quad \mathbf{o.k.}$

Manual  
Table 3-12

Check deflection

$$\Delta_{max} = \frac{l}{240} = \frac{(21.0 \text{ ft})(12 \text{ in./ft})}{240} = 1.05 \text{ in.}$$

$$I_x = 54.4 \text{ in.}^4$$

$$\Delta_{max} = \frac{5wl^4}{384EI} = \frac{5(0.375 \text{ kip/ft})^3 (21.0 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29,000 \text{ ksi})(54.4 \text{ in.}^4)} = 1.04 \text{ in.} < 1.05 \text{ in.} \quad \mathbf{o.k.}$$

Manual  
Table 1-12

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:

$$\text{ASTM A500 Gr. B} \quad F_y = 46 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Manual  
Table 2-3

#### Geometric Properties:

$$\begin{array}{llll} \text{HSS8} \times 8 \times \frac{3}{16} & I_x = 54.4 \text{ in.}^4 & Z_x = 15.7 \text{ in.}^3 & S_x = 13.6 \text{ in.}^3 \\ b_f = 8.0 \text{ in.} & t_f = 0.174 \text{ in.} & b/t = 43.0 & h/t = 43.0 \end{array}$$

Manual  
Table 1-12

#### Check flange slenderness

The assumed outside radius of the corners of HSS shapes is  $1.5t$ . 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 bending 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

$$\lambda = \frac{b}{t} = 43.0 > \lambda_r, \text{ therefore flange is slender.}$$

#### 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

$$\lambda = \frac{h}{t} = 43.0 > \lambda_p, \text{ therefore the web is compact.}$$

For HSS sections with slender flanges and compact webs, Specification F7-2(c) applies.

$$M_n = F_y S_{eff}$$

Eqn. F7-3

Where  $S_{eff}$  is the effective section modulus determined with the effective width of the compression flange taken as:

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

$$b = 8.00 \text{ in.} - 3(0.174 \text{ in.}) = 7.48 \text{ in.}$$

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

Eqn. F7-4

The ineffective width of the compression flange is:

$$7.48 \text{ in.} - 6.53 \text{ in.} = 0.950 \text{ in.}$$

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.

$$I_{eff} \approx 54.4 \text{ in.}^4 - 2 \left[ (0.950 \text{ in.})(0.174 \text{ in.})(3.91)^2 + \frac{(0.950 \text{ in.})(0.174 \text{ in.})^3}{12} \right] = 49.3 \text{ in.}^4$$

The effective section modulus can now be calculated as follows:

$$S_{eff} = \frac{I_{eff}}{d/2} = \frac{49.3 \text{ in.}^4}{(8.00 \text{ in.})/2} = 12.3 \text{ in.}^3$$

Calculate the nominal flexural strength,  $M_n$

$$M_n = F_y S_{eff} = 46 \text{ ksi}(12.3 \text{ in.}^3) = 567 \text{ kip-in. or } 47.3 \text{ kip-ft}$$

Eqn. F7-3

Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(47.3 \text{ kip-ft})$	$M_n / \Omega_b = \frac{47.3 \text{ kip-ft}}{1.67}$
$= 42.5 \text{ kip-ft} > 41.3 \text{ kip-ft}$ <b>o.k.</b>	$= 28.3 \text{ kip-ft} > 27.6 \text{ kip-ft}$ <b>o.k.</b>

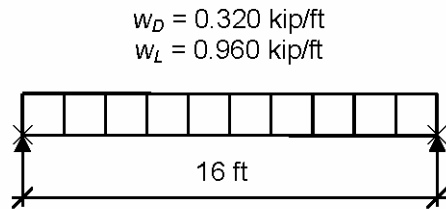
Section F1

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 grade 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:

$$\text{ASTM A500 Gr. B} \quad F_y = 35 \text{ ksi} \quad F_u = 60 \text{ ksi}$$

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.320 \text{ kip/ft}) + 1.6(0.960 \text{ kip/ft})$ $= 1.92 \text{ kip/ft}$	$w_a = 0.320 \text{ kip/ft} + 0.960 \text{ kip/ft}$ $= 1.28 \text{ kip/ft}$
$M_u = \frac{1.92 \text{ kip/ft}(16.0 \text{ ft})^2}{8} = 61.4 \text{ kip-ft}$	$M_a = \frac{1.28 \text{ kip/ft}(16.0 \text{ ft})^2}{8} = 41.0 \text{ kip-ft}$

Select a member from Manual Table 3-15 having the required strength

Select Pipe 8 X-Strong.

LRFD	ASD
$\phi_b M_n = 81.4 \text{ kip-ft} > 61.4 \text{ kip-ft} \quad \mathbf{o.k.}$	$M_n / \Omega_b = 54.1 \text{ kip-ft} > 41.0 \text{ kip-ft} \quad \mathbf{o.k.}$

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:

$$\text{ASTM A53 Gr. B} \quad F_y = 35 \text{ ksi} \quad F_u = 60 \text{ ksi}$$

Manual  
Table 2-3

#### Geometric Properties:

$$\text{Pipe 8 X-Strong} \quad Z = 31.0 \text{ in.}^3 \quad D = 8.63 \text{ in.} \quad t = 0.465 \text{ in.} \quad D/t = 18.5$$

Manual  
Table 1-14

Required flexural strength from Example F.9a

LRFD	ASD
$M_u = 61.4 \text{ kip-ft}$	$M_a = 41.0 \text{ kip-ft}$

#### 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

$\lambda = \frac{D}{t} = 18.5 < \lambda_p$ , therefore the section is compact and the limit state of flange local buckling does not apply.

By inspection,  $\frac{D}{t} < \frac{0.45E}{F_y}$ , therefore Specification Section F8 applies.

Section F8

Calculate the nominal flexural strength based on the flexural yielding limit state

$$M_n = M_p = F_y Z = (35 \text{ ksi})(31.0 \text{ in.}^3) = 1080 \text{ kip-in. or } 90.4 \text{ kip-ft}$$

Eqn. F8-1

Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$	$\Omega_b = 1.67$
$\phi_b M_n = 0.90(90.4 \text{ kip-ft})$	$M_n / \Omega_b = \frac{90.4 \text{ kip-ft}}{1.67}$
$= 81.4 \text{ kip-ft} > 61.4 \text{ kip-ft} \quad \mathbf{o.k.}$	$= 54.1 \text{ kip-ft} > 41.0 \text{ kip-ft} \quad \mathbf{o.k.}$

Section F1



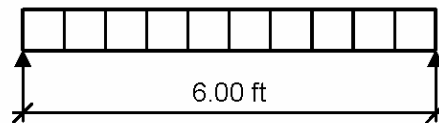
### Example F.10 WT Shape Flexural Member

#### 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.

$$w_D = 0.08 \text{ kip/ft}$$

$$w_L = 0.24 \text{ kip/ft}$$



*Beam Loading & Bracing Diagram  
(Full lateral support)*

#### Solution:

#### Material properties:

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

Manual  
Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.0800 \text{ kip/ft}) + 1.6(0.240 \text{ kip/ft})$ $= 0.480 \text{ kip/ft}$	$w_a = 0.0800 \text{ kip/ft} + 0.240 \text{ kip/ft}$ $= 0.320 \text{ kip/ft}$
$M_u = \frac{0.480 \text{ kip/ft} (6.00 \text{ ft})^2}{8} = 2.16 \text{ kip-ft}$	$M_a = \frac{0.320 \text{ kip/ft} (6.00 \text{ ft})^2}{8} = 1.44 \text{ kip-ft}$

Try WT 5×6

#### Geometric Properties:

$$\begin{array}{llll} \text{WT 5} \times 6 & I_x = 4.35 \text{ in.}^4 & Z_x = 2.20 \text{ in.}^3 & S_x = 1.22 \text{ in.}^3 \quad b_f = 3.96 \text{ in.} \\ & t_f = 0.210 \text{ in.} & \bar{y} = 1.36 \text{ in.} & S_{xc} = \frac{I_x}{y_c} = \frac{4.35 \text{ in.}^4}{1.36 \text{ in.}} = 3.20 \text{ in.}^3 \end{array}$$

Manual  
Table 1-8

Calculate the nominal flexural strength,  $M_n$

*Flexural yielding limit state*

$$M_p = F_y Z_x \leq 1.6 M_y \text{ for stems in tension}$$

Eqn. F9-2

$$1.6 M_y = 1.6 F_y S_x = 1.6 (50 \text{ ksi}) (1.22 \text{ in.}^3) = 97.6 \text{ kip-in.}$$

$$M_p = F_y Z_x = (50 \text{ ksi})(2.20 \text{ in.}^3) = 110 \text{ kip-in.} > 97.6 \text{ kip-in.}, \text{ therefore use}$$

$$M_p = 97.6 \text{ kip-in. or } 8.13 \text{ kip-ft}$$

$$M_n = M_p = 8.13 \text{ kip-ft} \quad \textbf{controls}$$

Eqn. F9-1

*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*

$$\lambda = \frac{b_f}{2t} = \frac{3.96 \text{ in.}}{2(0.210 \text{ in.})} = 9.43$$

$$\lambda_p = 0.38 \sqrt{\frac{E}{F_y}} = 0.38 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 9.15 < 9.43; \text{ therefore the flange is not compact.}$$

Table B4.1, Case 7

*Check flange slenderness*

$$\lambda_r = 1.0 \sqrt{\frac{E}{F_y}} = 1.0 \sqrt{\frac{29,000 \text{ ksi}}{50 \text{ ksi}}} = 24.1 > 9.43, \text{ therefore the flange is not slender.}$$

Table B4.1, Case 7

*Calculate critical flange local buckling stress*

For a Tee with a noncompact flange, the critical stress is:

$$F_{cr} = F_y \left( 1.19 - 0.50 \left( \frac{b_f}{2t_f} \right) \sqrt{\frac{F_y}{E}} \right) = (50 \text{ ksi}) \left( 1.19 - 0.50(9.43) \sqrt{\frac{50 \text{ ksi}}{29,000 \text{ ksi}}} \right) = 49.7 \text{ ksi}$$

Eqn. F9-7

*Calculate the nominal flexural strength*

$$M_n = F_{cr} S_{xc} = 49.7 \text{ ksi}(3.20 \text{ in.}^3) = 159 \text{ kip-in. or } 13.3 \text{ kip-ft} \quad \textbf{does not control}$$

Eqn. F9-6

*Calculate the available flexural strength*

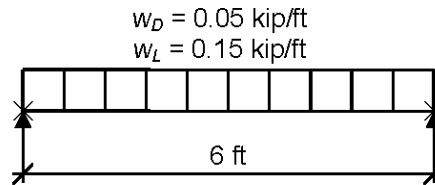
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(8.13 \text{ kip-ft})$ $= 7.32 \text{ kip-ft} > 2.16 \text{ kip-ft} \quad \textbf{o.k.}$	$\Omega_b = 1.67$ $M_n / \Omega_b = \frac{8.13 \text{ kip-ft}}{1.67}$ $= 4.87 \text{ kip-ft} > 1.44 \text{ kip-ft} \quad \textbf{o.k.}$

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     $F_y = 36 \text{ ksi}$      $F_u = 58 \text{ ksi}$

Manual  
Table 2-3

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.05 \text{ kip/ft}) + 1.6(0.15 \text{ kip/ft})$ $= 0.300 \text{ kip/ft}$	$w_a = 0.05 \text{ kip/ft} + 0.15 \text{ kip/ft}$ $= 0.200 \text{ kip/ft}$
$M_u = \frac{(0.300 \text{ kip/ft})(6 \text{ ft})^2}{8} = 1.35 \text{ kip-ft}$	$M_a = \frac{(0.200 \text{ kip/ft})(6 \text{ ft})^2}{8} = 0.900 \text{ kip-ft}$

Try  $L4 \times 4 \times \frac{1}{4}$

#### Geometric Properties:

$L4 \times 4 \times \frac{1}{4}$      $S_x = 1.03 \text{ in}^3$      $S_z = 0.419 \text{ in}^3$

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

$M_y = 0.80 S_x F_y = 0.80(1.03 \text{ in}^3)(36 \text{ ksi}) = 29.7 \text{ kip-in.}$

Section  
F10-2

*Flexural yielding limit state*

$M_n = 1.5 M_y = 1.5(29.7 \text{ kip-in.})$   
 $= 44.5 \text{ kip-in. or } 3.71 \text{ kip-ft} \quad \textbf{controls}$

Eqn. F10-1

*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.

$$M_e = \frac{0.66Eb^4tC_b}{L^2} \left( \sqrt{1 + 0.78 \left( \frac{Lt}{b^2} \right)^2} + 1 \right) \quad \text{Eqn. F10-4b}$$

$$= \frac{0.66(29,000 \text{ ksi})(4.00 \text{ in.})^4 (0.250 \text{ in.})(1.0)}{(72.0 \text{ in.})^2} \left( \sqrt{1 + 0.78 \left( \frac{(72.0 \text{ in.})(0.250 \text{ in.})}{(4.00 \text{ in.})^2} \right)^2} + 1 \right)$$

$$= 569 \text{ kip-in.} > 29.7 \text{ kip-in.} \text{ therefore, Equation F10-3 is applicable.}$$

$$M_n = \left( 1.92 - 1.17 \sqrt{\frac{M_y}{M_e}} \right) M_y \leq 1.5M_y \quad \text{Eqn. F10-3}$$

$$= \left( 1.92 - 1.17 \sqrt{\frac{29.7 \text{ kip-in.}}{569 \text{ kip-in.}}} \right) 29.7 \text{ kip-in.} \leq 1.5(29.7 \text{ kip-in.})$$

$$= 49.1 \text{ kip-in.} \leq 44.5 \text{ kip-in.} \text{ , therefore}$$

$M_n = 44.5 \text{ kip-in.}$  or  $3.71 \text{ kip-ft.}$

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(3.71 \text{ kip-ft})$ $= 3.34 \text{ kip-ft} > 1.35 \text{ kip-ft} \quad \mathbf{o.k.}$	$\Omega_b = 1.67$ $M_n / \Omega_b = \frac{3.71 \text{ kip-ft}}{1.67}$ $= 2.22 \text{ kip-ft} > 0.900 \text{ kip-ft} \quad \mathbf{o.k.}$

Section F1

**Note:** In this example it is assumed that the toe of the vertical leg of the single angle is in tension. If the toe of the outstanding leg is in compression, as in this example, the leg local buckling limit state must also be checked. The designer should also consider the possibility that restrained end conditions of a single angle member could unintentionally cause the outstanding leg to be in compression.

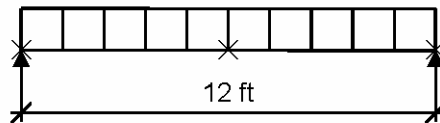
## Example F.12 Rectangular Bar in Strong-Axis Bending

### 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.

$$w_D = 0.44 \text{ kip/ft}$$

$$w_L = 1.32 \text{ kip/ft}$$



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

### Solution:

#### Material Properties:

$$\text{ASTM A36} \quad F_y = 36 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Manual  
Table 2-3

LRFD	ASD
$w_u = 1.2(0.440 \text{ kip/ft}) + 1.6(1.32 \text{ kip/ft})$ $= 2.64 \text{ kip/ft}$	$w_a = 0.440 \text{ kip/ft} + 1.32 \text{ kip/ft}$ $= 1.76 \text{ kip/ft}$
$M_u = \frac{2.64 \text{ kip/ft} (12.0 \text{ ft})^2}{8} = 47.5 \text{ kip-ft}$	$M_a = \frac{1.76 \text{ kip/ft} (12.0 \text{ ft})^2}{8} = 31.7 \text{ kip-ft}$

Try a 5 in.×3 in. bar.

#### Geometric Properties:

$$\text{Rectangular bar } 5 \times 3 \quad S_x = \frac{bd^2}{6} = \frac{(3.00 \text{ in.})(5.00 \text{ in.})^2}{6} = 12.5 \text{ in.}^3$$

$$Z_x = \frac{bd^2}{4} = \frac{(3.00 \text{ in.})(5.00 \text{ in.})^2}{4} = 18.8 \text{ in.}^3$$

### Solution:

Calculate nominal flexural strength,  $M_n$

Flexural yielding limit state

Check limit

$$\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$$

Section  
F11.1

$$\frac{(72.0 \text{ in.})(5.00 \text{ in.})}{(3.00 \text{ in.})^2} \leq \frac{0.08(29,000 \text{ ksi})}{(36 \text{ ksi})}$$

40.0 < 64.4, therefore the yielding limit state applies.

$$M_n = M_p = F_y Z \leq 1.6M_y$$

Eqn. F11-1

$$1.6M_y = 1.6F_y S_x = 1.6(36 \text{ ksi})(12.5 \text{ in.}^3) = 720 \text{ kip-in.}$$

$$M_p = F_y Z_x = (36 \text{ ksi})(18.8 \text{ in.}^3) = 675 \text{ kip-in.} < 720 \text{ kip-in.}$$

Use  $M_n = M_p = 675 \text{ kip-in.}$  or  $56.3 \text{ kip-ft}$

*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.

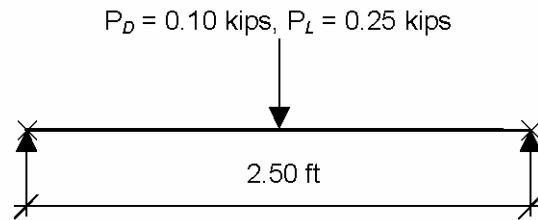
LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(56.3 \text{ kip-ft})$ $= 50.6 \text{ kip-ft} > 47.5 \text{ kip-ft} \quad \mathbf{o.k.}$	$\Omega_b = 1.67$ $M_n / \Omega_b = \frac{56.3 \text{ kip-ft}}{1.67}$ $= 33.7 \text{ kip-ft} > 31.7 \text{ kip-ft} \quad \mathbf{o.k.}$

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 2-3

Calculate the required flexural strength

LRFD	ASD
$P_u = 1.2(0.100 \text{ kip}) + 1.6(0.250 \text{ kip})$ $= 0.520 \text{ kip}$	$P_a = 0.100 \text{ kip} + 0.250 \text{ kip}$ $= 0.350 \text{ kip}$
$M_u = \frac{(0.520 \text{ kip})(2.50 \text{ ft})}{4} = 0.325 \text{ kip-ft}$	$M_a = \frac{(0.350 \text{ kip})(2.50 \text{ ft})}{4} = 0.219 \text{ kip-ft}$

Try 1 in. diameter rod.

#### Geometric Properties:

$$\text{Round bar} \quad S_x = \frac{\pi d^3}{32} = \frac{\pi(1.00 \text{ in.})^3}{32} = 0.0982 \text{ in.}^3$$

$$Z_x = \frac{d^3}{6} = \frac{(1.00 \text{ in.})^3}{6} = 0.167 \text{ in.}^3$$

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

$$1.6M_y = 1.6(36 \text{ ksi})(0.0982 \text{ in.}^3) = 5.66 \text{ kip-in}$$

$$F_y Z_x = 36 \text{ ksi}(0.167 \text{ in.}^3) = 6.00 \text{ kip-in} > 5.66 \text{ kip-in.}$$

Therefore,  $M_n = 5.66 \text{ kip-in.}$  or  $0.471 \text{ kip-ft}$

*Lateral-torsional buckling limit state*

Sec. F11.2

This limit state need not be considered for rounds.

*Calculate the available flexural strength*

$\phi_b = 0.90$ $\phi_b M_n = 0.90(0.471 \text{ kip-ft})$ $= 0.424 \text{ kip-ft} > 0.325 \text{ kip-ft} \quad \mathbf{o.k.}$	$\Omega_b = 1.67$ $M_n / \Omega_b = \frac{0.471 \text{ kip-ft}}{1.67}$ $= 0.282 \text{ kip-ft} > 0.219 \text{ kip-ft} \quad \mathbf{o.k.}$
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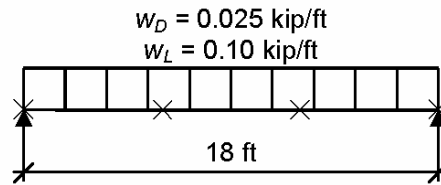
Section F1



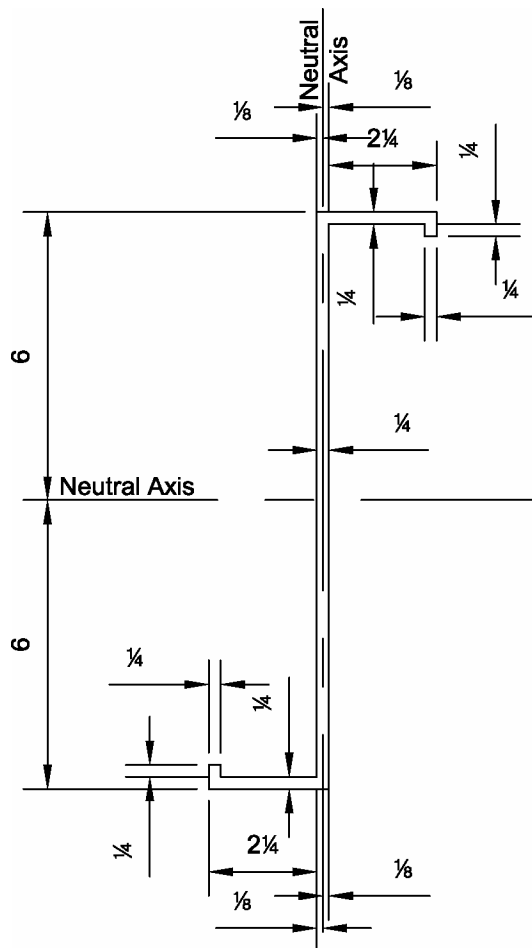
### Example F.14 Non-Symmetrical Z-shape in Strong-Axis Bending

**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:**

$$Z \text{ Purlin} \quad F_y = 36 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Manual  
Table 2-3**Geometric Properties:**

$$t_w = t_f = 0.250 \text{ in.}$$

$$A = (2.50 \text{ in.})(0.25 \text{ in.})(2) + (0.25 \text{ in.})(0.25 \text{ in.})(2) + (11.5 \text{ in.})(0.25 \text{ in.}) = 4.25 \text{ in.}^2$$

$$\begin{aligned} I_x &= \left[ \frac{(0.25 \text{ in.})(0.25 \text{ in.})^3}{12} + (0.25 \text{ in.})^2 (5.62 \text{ in.})^2 \right] (2) \\ &\quad + \left[ \frac{(2.50 \text{ in.})(0.25 \text{ in.})^3}{12} + (2.50 \text{ in.})(0.25 \text{ in.})(5.87 \text{ in.})^2 \right] (2) \\ &\quad + \frac{(0.25 \text{ in.})(11.5 \text{ in.})^3}{12} \\ &= 78.8 \text{ in.}^4 \end{aligned}$$

$$\bar{y} = 6.00 \text{ in.}$$

$$S_x = \frac{I_x}{\bar{y}} = \frac{78.8 \text{ in.}^4}{6.00 \text{ in.}} = 13.1 \text{ in.}^3$$

$$\begin{aligned} I_y &= \left[ \frac{(0.25 \text{ in.})(0.25 \text{ in.})^3}{12} + (0.25 \text{ in.})^2 (2.25 \text{ in.})^2 \right] (2) \\ &\quad + \left[ \frac{(0.25 \text{ in.})(2.50 \text{ in.})^3}{12} + (2.50 \text{ in.})(0.25 \text{ in.})(1.12 \text{ in.})^2 \right] (2) \\ &\quad + \frac{(11.5 \text{ in.})(0.25 \text{ in.})^3}{12} \\ &= 2.88 \text{ in.}^4 \end{aligned}$$

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2.87 \text{ in.}^4}{4.25 \text{ in.}^2}} = 0.823 \text{ in.}$$

$$r_{ts} \approx \frac{b_f}{\sqrt{12 \left( 1 + \frac{ht_w}{6b_f t_f} \right)}} = \frac{2.50 \text{ in.}}{\sqrt{12 \left( 1 + \frac{(11.5 \text{ in.})(0.250 \text{ in.})}{6(2.50 \text{ in.})(0.250 \text{ in.})} \right)}} = 0.543 \text{ in.}$$

Calculate the required flexural strength

LRFD	ASD
$w_u = 1.2(0.025 \text{ kip/ft}) + 1.6(0.10 \text{ kip/ft})$ $= 0.190 \text{ kip/ft}$	$w_a = 0.025 \text{ kip/ft} + 0.10 \text{ kip/ft}$ $= 0.125 \text{ kip/ft}$
$M_u = \frac{(0.190 \text{ kip/ft})(18.0 \text{ ft})^2}{8} = 7.70 \text{ kip-ft}$	$M_a = \frac{(0.125 \text{ kip/ft})(18.0 \text{ ft})^2}{8} = 5.06 \text{ kip-ft}$

*Flexural yielding limit state*

$$F_n = F_y = 36 \text{ ksi}$$

Eqn. F12-2

$$M_n = F_n S = 36 \text{ ksi}(13.1 \text{ in.}^3) = 473 \text{ kip-in. or } 39.4 \text{ kip-ft}$$

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,  $L_b = (6.00 \text{ ft})(12 \text{ in./ft}) = 72.0 \text{ in.}$

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76(0.823 \text{ in.}) \sqrt{\frac{29,000 \text{ ksi}}{36 \text{ ksi}}} = 41.1 \text{ in.} < 72.0 \text{ in.}$$

Eqn. F2-5

$$L_r = 1.95 r_{ts} \left( \frac{E}{0.7 F_y} \right) \sqrt{\frac{Jc}{S_x h_0}} \sqrt{1 + \sqrt{1 + 6.76 \left( \frac{0.7 F_y S_x h_0}{E Jc} \right)^2}}$$

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, therefore,

$$L_r = \pi r_{ts} \sqrt{\frac{E}{0.7F_y}} = \pi(0.543 \text{ in.}) \sqrt{\frac{29000 \text{ ksi}}{0.7(36 \text{ ksi})}} = 57.9 \text{ in.} < 72.0 \text{ in.}$$

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,

$$F_{cr} = (0.5) \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} = (0.5) \frac{1.0 \pi^2 (29,000 \text{ ksi})}{\left(\frac{72.0 \text{ in.}}{0.543 \text{ in.}}\right)^2} = 8.14 \text{ ksi}$$

$$F_n = F_{cr} \leq F_y \\ = 8.14 \text{ ksi} < 36 \text{ ksi} \quad \mathbf{o.k.}$$

Eqn. F12-3

$$M_n = F_n S \\ = (8.14 \text{ ksi})(13.1 \text{ in.}^3) = 107 \text{ kip-in. or } 8.89 \text{ k-ft} \quad \mathbf{controls}$$

Eqn. F12-1

Calculate the available strength

LRFD	ASD
$\phi_b = 0.90$ $\phi_b M_n = 0.90(8.89 \text{ kip-ft})$ $= 8.00 \text{ kip-ft} > 7.70 \text{ kip-ft} \quad \mathbf{o.k.}$	$\Omega_b = 1.67$ $M_n / \Omega_b = \frac{8.89 \text{ kip-ft}}{1.67}$ $= 5.32 \text{ kip-ft} > 5.06 \text{ kip-ft} \quad \mathbf{o.k.}$

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,  $V_n/\Omega_v$ , are determined as follows:

$V_n$  = nominal shear strength based on shear yielding or shear buckling

$\phi_v = 0.90$  (LRFD)                       $\Omega_v = 1.67$  (ASD).

Exception: For all current ASTM A6, *W*, *S*, and *HP* shapes except *W*44×230, *W*40×149, *W*36×135, *W*33×118, *W*30×90, *W*24×55, *W*16×26, and *W*12×14 for  $F_y = 50$  ksi:

$\phi_v = 1.00$  (LRFD)                       $\Omega_v = 1.50$  (ASD).

Section G2 does not utilize tension field action. Section G3 specifically addresses the use 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,  $\phi_v = 1.00$  (LRFD) and  $\Omega_v = 1.50$  (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 3 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 IS 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:**

Verify the shear strength of a W24×62 ASTM A992 beam 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
$V_u = 1.2(48.0 \text{ kips}) + 1.6(145 \text{ kips}) = 290 \text{ kips}$	$V_a = 48.0 \text{ kips} + 145 \text{ kips} = 193 \text{ kips}$

*Take the available shear strength from Manual Table 3-2*

LRFD	ASD
$\phi_v V_n = 306 \text{ kips}$ 306 kips > 290 kips <b>o.k.</b>	$V_n/\Omega_v = 204 \text{ kips}$ 204 kips > 193 kips <b>o.k.</b>

Manual  
Table 3-2

### Example G.1b W-Shape in Strong-Axis Shear.

The available shear strength, which can be easily determined by the tabulated values of the *Steel Construction Manual*, can be verified by directly applying the provisions of the Specification.

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  $A_w$

$$A_w = dt_w = 23.7 \text{ in.} (0.430 \text{ in.}) = 10.2 \text{ in.}^2$$

Calculate  $V_n$

$$V_n = 0.6F_y A_w C_v = 0.6(50 \text{ ksi})(10.2 \text{ in.}^2)(1.0) = 306 \text{ kips}$$

Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
$\phi_v = 1.00$ $\phi_v V_n = 1.00(306 \text{ kips}) = 306 \text{ kips}$	$\Omega_v = 1.50$ $V_n / \Omega_v = 306 \text{ kips} / 1.50 = 204 \text{ kips}$

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
$V_u = 1.2(17.5 \text{ kips}) + 1.6(52.5 \text{ kips})$ $= 105 \text{ kips}$	$V_a = 17.5 \text{ kips} + 52.5 \text{ kips}$ $= 70.0 \text{ kips}$

*Take the available shear strength from Manual Table 3-8*

LRFD	ASD
$\phi_v V_n = 117 \text{ kips}$ $117 \text{ kips} > 105 \text{ kips}$ <b>o.k.</b>	$V_n / \Omega_v = 77.6 \text{ kips}$ $77.6 \text{ kips} > 70.0 \text{ kips}$ <b>o.k.</b>

Manual Table  
3-8

### Example G.2b C-Shape in Strong-Axis Shear.

The available shear strength, which can be easily determined by the tabulated values of the Steel Construction Manual, can be verified by directly applying the provisions of the Specification.  $C_v$  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  $A_w$

$$A_w = 15.0 \text{ in.}(0.400 \text{ in.}) = 6.00 \text{ in.}^2$$

Calculate  $V_n$

$$V_n = 0.6F_y A_w C_v = 0.6(36 \text{ ksi})(6.00 \text{ in.}^2)(1.0) = 130 \text{ kips}$$

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
$\phi_v V_n = 0.90(130 \text{ kips}) = 117 \text{ kips}$	$V_n / \Omega_v = 130 \text{ kips} / 1.67 = 77.6 \text{ kips}$

### Example G.3      Angle in Shear.

**Given:**

Verify the shear strength of a  $5 \times 3 \times \frac{1}{4}$  (LLV) ASTM A36 angle with end shears of 3.5 kips from dead load and 10.5 kips from live load.

**Solution:**

**Material Properties:**

$L5 \times 3 \times \frac{1}{4}$  angle    ASTM A36       $F_y = 36$  ksi       $F_u = 58$  ksi

Manual  
Table 2-3

**Geometric Properties:**

$L5 \times 3 \times \frac{1}{4}$  angle     $d = 5.00$  in.       $t_w = 0.250$  in.

Manual  
Table 1-7

Calculate the required shear strength

LRFD	ASD
$V_u = 1.2(3.50 \text{ kips}) + 1.6(10.5 \text{ kips})$ $= 21.0 \text{ kips}$	$V_a = 3.50 \text{ kips} + 10.5 \text{ kips}$ $= 14.0 \text{ kips}$

Note: There are no tables for angles in shear, but the shear strength can be calculated as follows:

For angles in shear, use Specification Equation G2-1 with  $C_v = 1.0$ .

Section G4

Calculate  $A_w$

$$A_w = dt_w = (5.00 \text{ in.})(0.250 \text{ in.}) = 1.25 \text{ in.}^2$$

Calculate  $V_n$

$$V_n = 0.6F_yA_wC_v = 0.6(36 \text{ ksi})(1.25 \text{ in.}^2)(1.0) = 27.0 \text{ kips}$$

Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(27.0 \text{ kips}) = 24.3 \text{ kips}$ $24.3 \text{ kips} > 21.0 \text{ kips}$ <b>o.k.</b>	$\Omega_v = 1.67$ $V_n/\Omega_v = 27.0 \text{ kips} / 1.67 = 16.2 \text{ kips}$ $16.2 \text{ kips} > 14.0 \text{ kips}$ <b>o.k.</b>

Section G1

### Example G.4 Rectangular HSS in Shear.

#### Given:

Verify the shear strength of a HSS6×4× $\frac{3}{8}$  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× $\frac{3}{8}$  ASTM A500B  $F_y = 46$  ksi  $F_u = 58$  ksi

Manual  
Table 2-3

#### Geometric Properties:

HSS6×4× $\frac{3}{8}$   $h = 6.00$  in.  $w = 4.00$  in.  $t = 0.349$  in.

Manual  
Table 1-11

Calculate the required shear strength

LRFD	ASD
$V_u = 1.2(11.0 \text{ kips}) + 1.6(33.0 \text{ kips})$ $= 66.0 \text{ kips}$	$V_a = 11.0 \text{ kips} + 33.0 \text{ kips}$ $= 44.0 \text{ kips}$

User note: There are no Manual Tables for shear in HSS shapes, but the shear strength can be calculated as follows:

Calculate the nominal strength

Section G5

For rectangular HSS in shear, use Section G2.1 with  $A_w = 2ht$  and  $k_v = 5$ .

If the exact radius is unknown, the radius is taken as 3 times the design thickness.

$$h = d - 2(3t_w) = 6.00 \text{ in.} - 2(3)(0.349 \text{ in.}) = 3.91 \text{ in.}$$

$$h/t_w = 3.91 \text{ in.} / 0.349 \text{ in.} = 11.2$$

$$1.10\sqrt{k_v E/F_y} = 1.10\sqrt{5(29,000 \text{ ksi}/46 \text{ ksi})} = 61.8$$

$$11.2 \leq 61.8 \quad \text{Therefore } C_v = 1.0$$

Eqn. G2-3

Note: most standard HSS sections listed in the manual have  $C_v = 1.0$  at  $F_y \leq 46$  ksi. Calculate  $A_w$

$$A_w = 2ht = 2(3.91 \text{ in.})(0.349 \text{ in.}) = 2.73 \text{ in.}^2$$

Calculate  $V_n$

$$V_n = 0.6F_y A_w C_v = 0.6(46 \text{ ksi})(2.73 \text{ in.}^2)(1.0) = 75.2 \text{ kips}$$

Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(75.2 \text{ kips}) = 67.7 \text{ kips}$ $67.7 \text{ kips} > 66.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_v = 1.67$ $V_n/\Omega_v = 75.2 \text{ kips} / 1.67 = 45.0 \text{ kips}$ $45.0 \text{ kips} > 44.0 \text{ kips} \quad \mathbf{o.k.}$

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.<sup>2</sup>

Manual  
Table 1-13

Note: There are no Manual tables for Round HSS in shear, but the strength can be calculated as follows:

Calculate the required shear strength

LRFD	ASD
$V_u = 1.2(30.0 \text{ kips}) + 1.6(90.0 \text{ kips})$ $= 180 \text{ kips}$	$V_a = 30.0 \text{ kips} + 90.0 \text{ kips}$ $= 120 \text{ kips}$

Calculate  $F_{cr}$  as the smallest of:

Section G6

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D} \left(\frac{D}{t}\right)^{\frac{5}{4}}}} \quad \text{take } L_v \text{ as half the span} = 192 \text{ in.}$$

Eqn. G6-2a

$$= \frac{1.60(29,000 \text{ ksi})}{\sqrt{\frac{192 \text{ in.}}{16.0 \text{ in.}} \left(\frac{16.0 \text{ in.}}{0.349 \text{ in.}}\right)^{\frac{5}{4}}}} = 112 \text{ ksi}$$

Eqn. G6-2b

or

$$F_{cr} = \frac{0.78E}{(D/t)^{\frac{3}{2}}} = \frac{0.78(29,000 \text{ ksi})}{\left(\frac{16.0 \text{ in.}}{0.349 \text{ in.}}\right)^{\frac{3}{2}}} = 73 \text{ ksi}$$

or

$$F_{cr} = 0.6F_y = 0.6(42 \text{ ksi}) = 25.2 \text{ ksi} \quad \text{controls}$$

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

$$V_n = \frac{F_{cr} A_g}{2} = \frac{(25.2 \text{ ksi})(17.2 \text{ in.}^2)}{2} = 217 \text{ kips}$$

Calculate the available shear strength

Eqn. G6-1

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(217 \text{ kips}) = 195 \text{ kips}$ $195 \text{ kips} > 190 \text{ kips}$ <b>o.k.</b>	$\Omega_v = 1.67$ $V_n/\Omega_v = 217 \text{ kips} / 1.67 = 130 \text{ kips}$ $130 \text{ kips} > 120 \text{ kips}$ <b>o.k.</b>

Section G1

**Example G.6      Doubly-Symmetric Shape in Weak-Axis Shear.**

**Given:**

Verify the 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
$V_u = 1.2(20.0 \text{ kips}) + 1.6(60.0 \text{ kips})$ $= 120 \text{ kips}$	$V_a = 20.0 \text{ kips} + 60.0 \text{ kips} = 80.0 \text{ kips}$

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)

$$A_w = 2b_f t_f = 2(8.14 \text{ in.})(0.430 \text{ in.}) = 7.00 \text{ in}^2$$

*Calculate  $C_v$*

Section G2.1b

$$b_f/t_f = 8.14 \text{ in.} / 0.430 \text{ in.} = 18.9$$

$$1.10\sqrt{k_v E/F_y} = 1.10\sqrt{1.2(29,000 \text{ ksi}/50 \text{ ksi})} = 29.0 > 18.9 \text{ therefore, } C_v = 1.0$$

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  $V_n$*

$$V_n = 0.6F_y A_w C_v = 0.6(50 \text{ ksi})(7.00 \text{ in.}^2)(1.0) = 210 \text{ kips}$$

Eqn. G2-1

*Calculate the available shear strength*

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(210 \text{ kips}) = 189 \text{ kips}$ $189 \text{ kips} > 120 \text{ kips}$ <b>o.k.</b>	$\Omega_v = 1.67$ $V_n/\Omega_v = 210 \text{ kips} / 1.67 = 126 \text{ kips}$ $126 \text{ kips} > 80.0 \text{ kips}$ <b>o.k.</b>

Section G1

### Example G.7 Singly-Symmetric Shape in Weak-Axis Shear.

#### Given:

Verify the 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

Note: There are no Manual tables for this, but the strength can be calculated as follows:

*Calculate the required shear strength*

LRFD	ASD
$V_u = 1.2(5.00 \text{ kips}) + 1.6(15.0 \text{ kips})$ $= 30.0 \text{ kips}$	$V_u = 5.00 \text{ kips} + 15.0 \text{ kips}$ $= 20.0 \text{ kips}$

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)

$$A_w = 2b_f t_f = 2(2.65 \text{ in.})(0.413 \text{ in.}) = 2.19 \text{ in.}^2$$

Section G7

*Calculate  $C_v$*

$$b_f / t_f = 2.65 \text{ in.} / 0.413 \text{ in.} = 6.42$$

Section G2.1b

$$1.10 \sqrt{k_v E / F_y} = 1.10 \sqrt{1.2(29,000 \text{ ksi} / 36 \text{ ksi})} = 34.2 > 6.42 \quad \text{Therefore, } C_v = 1.0$$

Eqn. G2-3

*Calculate  $V_n$*

$$V_n = 0.6 F_y A_w C_v = 0.6(36 \text{ ksi})(2.19 \text{ in.}^2)(1.0) = 47.3 \text{ kips}$$

Eqn. G2-1

*Calculate the available shear strength*

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(47.3 \text{ kips}) = 42.6 \text{ kips}$ $42.5 \text{ kips} > 30.0 \text{ kips}$ <b>o.k.</b>	$\Omega_v = 1.67$ $V_n / \Omega_v = 47.3 \text{ kips} / 1.67 = 28.3 \text{ kips}$ $28.3 \text{ kips} > 20.0 \text{ kips}$ <b>o.k.</b>

Section G1

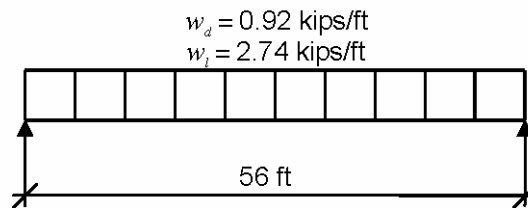


### Example G.8a Built-up Plate Girder with Transverse Stiffeners

#### 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. tall with 12 in.  $\times$  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.

User 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.       $b_{fl} = b_{fc} = 12.0$  in.  
 $t_f = 1.50$  in.       $h = 33.0$  in.

Calculate the required shear strength at the support

LRFD	ASD
$R_u = [1.2(0.92 \text{ klf}) + 1.6(2.74 \text{ klf})](28.0 \text{ ft})$ = 154 kips	$R_u = (0.92 \text{ klf} + 2.74 \text{ klf})(28.0 \text{ ft})$ = 102 kips

Determine if stiffeners are required

$$A_w = dt_w = (36.0 \text{ in.})(0.313 \text{ in.}) = 11.3 \text{ in.}^2$$

$$h/t_w = 33.0 \text{ in.} / 0.313 \text{ in.} = 105$$

$$105 < 260 \text{ Therefore, } k_v = 5$$

$$1.37 \sqrt{k_v E / F_y} = 1.37 \sqrt{5(29,000 \text{ ksi}) / (36 \text{ ksi})} = 86.9$$

105 > 86.9 therefore, use Specification Eqn. G2-5 to calculate  $C_v$

Section G2.1b

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} = \frac{1.51(29,000 \text{ ksi})(5)}{(105)^2(36 \text{ ksi})} = 0.547$$

Eqn. G2-5

Calculate  $V_n$

$$V_n = 0.6F_y A_w C_v = 0.6(36 \text{ ksi})(11.3 \text{ in.}^2)(0.547) = 134 \text{ kips}$$

Eqn. G2-1

Check the available shear strength without stiffeners

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(134 \text{ kips}) = 120 \text{ kips}$ $120 \text{ kips} < 154 \text{ kips}$ <b>not o.k.</b>  <b>Therefore, stiffeners are required.</b>	$\Omega_v = 1.67$ $V_n/\Omega_v = 134 \text{ kips} / 1.67 = 80.0 \text{ kips}$ $80.0 \text{ kips} < 102 \text{ kips}$ <b>not o.k.</b>  <b>Therefore, stiffeners are required.</b>

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 if any of the following are true:

- 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
<i>Use <math>V_u = \phi_v V_n</math> to determine the required stress in the web by dividing by the web area</i>  $\frac{\phi_v V_n}{A_w} = \frac{V_u}{A_w} = \frac{154 \text{ kips}}{11.3 \text{ in.}^2} = 13.6 \text{ ksi}$	<i>Use <math>V_a = V_n/\Omega_v</math> to determine the required stress in the web by dividing by the web area</i>  $\frac{V_n}{\Omega_v A_w} = \frac{V_a}{A_w} = \frac{102 \text{ kips}}{11.3 \text{ in.}^2} = 9.03 \text{ ksi}$

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,  $\frac{\phi_v V_n}{A_w} =$

Manual Table 3-16a

13.6 ksi for LRFD,  $\frac{V_n}{\Omega_v A_w} = 9.03 \text{ ksi}$  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 1.80 from the axis across the bottom. Since  $h = 33.0 \text{ in.}$ , stiffeners are required at  $(1.80)(33.0 \text{ in.}) = 59.4 \text{ in.}$  maximum. Therefore, use 59.0 in.

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, 59 in. from end

LRFD	ASD
$V_u = 154k - [1.2(0.92klf) + 1.6(2.74klf)] \left( \frac{59.0 \text{ in.}}{12 \text{ in./ft}} \right)$ $= 127 \text{ kips}$	$V_a = 102k - (0.92klf + 2.74klf) \left( \frac{59.0 \text{ in.}}{12 \text{ in./ft}} \right)$ $= 84.0 \text{ kips}$

Check the available shear strength without stiffeners

LRFD	ASD
$\phi_v = 0.90$  <i>From previous calculation</i> $\phi_v V_n = 0.90(134 \text{ kips}) = 120 \text{ kips}$ $120 \text{ kips} < 127 \text{ kips}$ <b>not o.k.</b>  <b>Therefore additional stiffeners are required.</b>  <i>Use <math>V_u = \phi_v V_n</math> to determine the required stress in the web by dividing by the web area</i>  $\frac{\phi_v V_n}{A_w} = \frac{V_u}{A_w} = \frac{127 \text{ kips}}{11.3 \text{ in.}^2} = 11.2 \text{ ksi}$	$\Omega_v = 1.67$  <i>From previous calculation</i> $V_n / \Omega_v = 134 \text{ kips} / 1.67 = 80.0 \text{ kips}$ $80.0 \text{ kips} < 84.0 \text{ kips}$ <b>not o.k.</b>  <b>Therefore additional stiffeners are required.</b>  <i>Use <math>V_a = V_n / \Omega_v</math> to determine the required stress in the web by dividing by the web area</i>  $\frac{V_n}{\Omega_v A_w} = \frac{V_a}{A_w} = \frac{84.0 \text{ kips}}{11.3 \text{ in.}^2} = 7.43 \text{ ksi}$

Section G1

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,  $\frac{\phi_v V_n}{A_w} = 11.2 \text{ ksi}$  for LRFD,  $\frac{V_n}{\Omega_v A_w} = 7.43 \text{ ksi}$  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  $h = 33.0 \text{ in.}$ , an additional stiffener is required at  $(3.0)(33.0 \text{ in.}) = 99.0 \text{ in.}$  maximum from the previous one.

Manual  
Table 3-16b

Select stiffener spacing for the second 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 pane, 158 in. from end

LRFD	ASD
$V_u = 154 \text{ kips}$ $- [1.2(0.920 \text{ klf}) + 1.6(2.74 \text{ klf})] \left( \frac{158.0 \text{ in.}}{12 \text{ in./ft}} \right)$ $= 81.7 \text{ kips}$	$V_a = 102 \text{ kips}$ $- (0.920 \text{ klf} + 2.74 \text{ klf}) \left( \frac{158.0 \text{ in.}}{12 \text{ in./ft}} \right)$ $= 53.8 \text{ kips}$

Check the available shear strength without stiffeners

LRFD	ASD
$\phi_v = 0.90$  <i>From previous calculation</i> $\phi_v V_n = 0.90(134 \text{ kips}) = 120 \text{ kips}$ $120 \text{ kips} > 81.7 \text{ kips} \quad \mathbf{o.k.}$  <b>Therefore additional stiffeners are not required.</b>	$\Omega_v = 1.67$  <i>From previous calculation</i> $V_n/\Omega_v = 134 \text{ kips} / 1.67 = 80.0 \text{ kips}$ $80.0 \text{ kips} > 53.8 \text{ kips} \quad \mathbf{o.k.}$  <b>Therefore additional stiffeners are not required.</b>

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 the proof below, the stiffener spacings that were selected from the charts in the example above are used.

Manual  
Table 3-16

### 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

$$k_v = 5 + \frac{5}{(a/h)^2} = 5 + \frac{5}{(1.80)^2} = 6.54$$

$$h/t_w = 33.0 \text{ in.} / 0.313 \text{ in.} = 105$$

Check  $a/h$  limits

$$a/h = \frac{59 \text{ in.}}{33 \text{ in.}} = 1.80 \leq 3.0$$

$$a/h = 1.80 < \left[ \frac{260}{(h/t_w)} \right]^2 = \left[ \frac{260}{105} \right]^2 = 6.13$$

Therefore, use  $k_v = 6.54$ .

Tension field action is not allowed since the panel is an end panel.

$$\text{Since } h/t_w > 1.37 \sqrt{k_v E / F_y} = 1.37 \sqrt{(6.54)(29,000 \text{ ksi}) / (36 \text{ ksi})} = 99.5,$$

$$C_v = \frac{1.51 E k_v}{(h/t_w)^2 F_y} = \frac{1.51(29,000 \text{ ksi})(6.54)}{(105)^2 36 \text{ ksi}} = 0.721$$

Eqn. G2-5

$$V_n = 0.6 F_y A_w C_v = 0.6(36 \text{ ksi})(11.3 \text{ in.}^2)(0.721) = 176 \text{ kips}$$

Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(176 \text{ kips}) = 159 \text{ kips}$ $159 \text{ kips} > 154 \text{ kips}$ <b>o.k.</b>	$\Omega_v = 1.67$ $V_n / \Omega_v = (176 \text{ kips}) / 1.67 = 106 \text{ kips}$ $106 \text{ kips} > 102 \text{ kips}$ <b>o.k.</b>

Section G1

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

$$a/h = \frac{99 \text{ in.}}{33 \text{ in.}} = 3.00 \leq 3.0$$

Section G2.1

$$a/h = 3.00 < \left[ \frac{260}{(h/t_w)} \right]^2 = \left[ \frac{260}{105} \right]^2 = 6.13$$

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

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

Section 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

$$= 0.6(36 \text{ ksi})(11.3 \text{ in.}^2) \left[ 0.613 + \frac{1 - 0.613}{1.15\sqrt{1 + (3.0)^2}} \right]$$

$$V_n = 176 \text{ kips}$$

Calculate the available shear strength

LRFD	ASD
$\phi_v = 0.90$ $\phi_v V_n = 0.90(176 \text{ kips}) = 158 \text{ kips}$ $156 \text{ kips} > 127 \text{ kips}$ <b>o.k.</b>	$\Omega_v = 1.67$ $V_n/\Omega_v = (176 \text{ kips}) / 1.67 = 105 \text{ kips}$ $105 \text{ kips} > 84.0 \text{ kips}$ <b>o.k.</b>

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 the results of a second-order analysis, as required by Section C of the Specification. This represents a significant change for ASD users, who are accustomed to using an interaction equation that includes a partial second-order amplification.

### Example H.1a W-shape Subjected to Combined Compression and Bending About Both Axes (braced frame).

#### Given:

Verify 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	$P_a = 267$ kips
$M_{ux} = 250$ kip-ft	$M_{ax} = 167$ kip-ft
$M_{uy} = 80.0$ kip-ft	$M_{ay} = 53.3$ kip-ft

#### Solution:

#### Material Properties:

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

LRFD	ASD
$p = \frac{0.866}{10^3 \text{ kips}}$ at 14.0 ft	$p = \frac{1.33}{10^3 \text{ kips}}$ at 14.0 ft
$b_x = \frac{1.38}{10^3 \text{ kip-ft}}$ at 14.0 ft	$b_x = \frac{2.08}{10^3 \text{ kip-ft}}$ at 14.0 ft
$b_y = \frac{2.85}{10^3 \text{ kip-ft}}$	$b_y = \frac{4.29}{10^3 \text{ kip-ft}}$
Check limit for Equation H1-1a $\frac{P_u}{\phi_c P_n} = \frac{400 \text{ kips}}{1,130 \text{ kips}} = 0.354$	Check limit for Equation H1-1a $\frac{P_a}{P_n / \Omega_c} = \frac{267 \text{ kips}}{751 \text{ kips}} = 0.356$
Since $\frac{P_u}{\phi_c P_n} > 0.2$ ,	Since $\frac{P_a}{P_n / \Omega_c} > 0.2$ ,
$pP_u + b_x M_{ux} + b_y M_{uy} \leq 1.0$	$pP_a + b_x M_{ax} + b_y M_{ay} \leq 1.0$
$= \left( \frac{0.866}{10^3 \text{ kips}} \right) (400 \text{ kips})$ $+ \left( \frac{1.38}{10^3 \text{ kip-ft}} \right) (250 \text{ kip-ft})$ $+ \left( \frac{2.85}{10^3 \text{ kip-ft}} \right) (80.0 \text{ kip-ft})$	$= \left( \frac{1.33}{10^3 \text{ kips}} \right) (267 \text{ kips})$ $+ \left( \frac{2.08}{10^3 \text{ kip-ft}} \right) (167 \text{ kip-ft})$ $+ \left( \frac{4.29}{10^3 \text{ kip-ft}} \right) (53.3 \text{ kip-ft})$
$= 0.346 + 0.345 + 0.228 = 0.927 \leq 1.0$ <b>o.k.</b>	$= 0.356 + 0.347 + 0.229 = 0.931 \leq 1.0$ <b>o.k.</b>

Manual  
Table 6-1

Manual  
Table 4-1

Section H1.1

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 Subjected to Combined Compression and Bending Moment About Both Axes (braced frame)

Verify 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 second-order 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	$P_a = 267$ kips
$M_{ux} = 250$ kip-ft	$M_{ax} = 167$ kip-ft
$M_{uy} = 80.0$ kip-ft	$M_{ay} = 53.3$ kip-ft

#### Solution:

#### Material Properties:

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	Manual Table 4-1
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	Manual Table 3-10
$M_{cy} = \phi M_{ny} = 311$ kip-ft	$M_{cy} = M_{ny} / \Omega = 207$ kip-ft	Manual Table 3-2
$\frac{P_u}{\phi_c P_n} = \frac{400 \text{ kips}}{1,130 \text{ kips}} = 0.354$	$\frac{P_a}{P_n / \Omega_c} = \frac{267 \text{ kips}}{751 \text{ kips}} = 0.356$	
Since $\frac{P_u}{\phi_c P_n} > 0.2$ , use Eqn. H1.1a	Since $\frac{P_a}{P_n / \Omega_c} > 0.2$ , use Eqn. H1.1a	
$\frac{P_r}{P_c} + \frac{8}{9} \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0$	$\frac{P_r}{P_c} + \frac{8}{9} \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0$	Eq. H1.1a
$\frac{400 \text{ kips}}{1130 \text{ kips}} + \frac{8}{9} \left( \frac{250 \text{ kip-ft}}{642 \text{ kip-ft}} + \frac{80.0 \text{ kip-ft}}{311 \text{ kip-ft}} \right)$ $= 0.354 + \frac{8}{9} (0.389 + 0.257) = 0.929 < 1.0$ <b>o.k.</b>	$\frac{267 \text{ kips}}{751 \text{ kips}} + \frac{8}{9} \left( \frac{167 \text{ kip-ft}}{428 \text{ kip-ft}} + \frac{53.3 \text{ kip-ft}}{207 \text{ kip-ft}} \right)$ $= 0.356 + \frac{8}{9} (0.390 + 0.257) = 0.931 < 1.0$ <b>o.k.</b>	

## Example H.2 W-Shape Column Subjected to Combined Compression and Bending Moment About Both Axes (by Specification Section H2)

### Given:

Verify if an ASTM A992 W14×99 shown in **Example H.1** has sufficient available strength, using Specification Section H2.1. This example is included primarily to illustrate the use of Specification Section H2.  $KL_x = KL_y = L_b = 14.0$  ft

LRFD	ASD
$P_u = 400$ kips	$P_a = 267$ kips
$M_{ux} = 250$ kip-ft	$M_{ax} = 167$ kip-ft
$M_{uy} = 80.0$ kip-ft	$M_{ay} = 53.3$ kip-ft

### Solution:

#### Material Properties:

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

Manual  
Table 2-3

#### Geometric Properties:

$$\text{W14} \times 99 \quad A = 29.1 \text{ in.}^2 \quad S_x = 157 \text{ in.}^3 \quad S_y = 55.2 \text{ in.}^3$$

Manual  
Table 1-1

Calculate the required flexural and axial stresses

LRFD	ASD
$f_a = \frac{P_u}{A} = \frac{400 \text{ kips}}{29.1 \text{ in.}^2} = 13.7 \text{ ksi}$	$f_a = \frac{P_a}{A} = \frac{267 \text{ kips}}{29.1 \text{ in.}^2} = 9.18 \text{ ksi}$
$f_{bw} = \frac{M_{ux}}{S_x} = \frac{250 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{157 \text{ in.}^3} = 19.1 \text{ ksi}$	$f_{bw} = \frac{M_{ax}}{S_x} = \frac{167 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{157 \text{ in.}^3} = 12.8 \text{ ksi}$
$f_{bz} = \frac{M_{uy}}{S_y} = \frac{80.0 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{55.2 \text{ in.}^3} = 17.4 \text{ ksi}$	$f_{bz} = \frac{M_{ay}}{S_y} = \frac{53.3 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{55.2 \text{ in.}^3} = 11.6 \text{ ksi}$

Calculate the available flexural and axial stresses from the available strengths in **Example H.1b**

LRFD	ASD
$F_a = \phi_c F_{cr} = \frac{\phi_c P_n}{A} = \frac{1,130 \text{ kips}}{29.1 \text{ in.}^2} = 38.8 \text{ ksi}$	$F_a = \frac{F_{cr}}{\Omega_c} = \frac{P_n}{\Omega_c A} = \frac{751 \text{ kips}}{29.1 \text{ in.}^2} = 25.8 \text{ ksi}$
$F_{bw} = \frac{\phi_b M_{nx}}{S_x} = \frac{645 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{157 \text{ in.}^3} = 49.3 \text{ ksi}$	$F_{bw} = \frac{M_{nx}}{\Omega_b S_x} = \frac{431 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{157 \text{ in.}^3} = 32.9 \text{ ksi}$
$F_{bz} = \frac{\phi_b M_{ny}}{S_y} = \frac{311 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{55.2 \text{ in.}^3} = 67.6 \text{ ksi}$	$F_{bz} = \frac{M_{ny}}{\Omega_b S_y} = \frac{207 \text{ kip-ft} \left( \frac{12 \text{ in.}}{\text{ft}} \right)}{55.2 \text{ in.}^3} = 45.0 \text{ ksi}$

As shown in the LRFD calculation of  $F_{bz}$  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
$\left  \frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \right  \leq 1.0$	$\left  \frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \right  \leq 1.0$
$\left  \frac{13.7 \text{ ksi}}{38.8 \text{ ksi}} + \frac{19.1 \text{ ksi}}{49.3 \text{ ksi}} + \frac{17.4 \text{ ksi}}{67.6 \text{ ksi}} \right  = 0.998 < 1.0$ <p style="text-align: right;"><b>o.k.</b></p>	$\left  \frac{9.18 \text{ ksi}}{25.8 \text{ ksi}} + \frac{12.8 \text{ ksi}}{32.9 \text{ ksi}} + \frac{11.6 \text{ ksi}}{45.0 \text{ ksi}} \right  = 1.00 \quad \mathbf{o.k.}$

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:

$$\begin{aligned} M_{xD} &= 32.0 \text{ kip-ft} & M_{xL} &= 96.0 \text{ kip-ft} \\ M_{yD} &= 11.3 \text{ kip-ft} & M_{yL} &= 33.8 \text{ kip-ft} \end{aligned}$$

The unbraced length is 30 ft and the ends are pinned. Assume the connections are made with no holes.

**Solution:**

**Material Properties:**

$$\text{ASTM A992} \quad F_y = 50 \text{ ksi} \quad F_u = 65 \text{ ksi}$$

Manual  
Table 2-3

Calculate the required strength

LRFD	ASD
$P_u = 1.2(29.0 \text{ kips}) + 1.6(87.0 \text{ kips})$ = 174 kips	$P_a = 29.0 \text{ kips} + 87.0 \text{ kips}$ = 116 kips
$M_{ux} = 1.2(32.0 \text{ kip-ft}) + 1.6(96.0 \text{ kip-ft})$ = 192 kip-ft	$M_{ay} = 32.0 \text{ kip-ft} + 96 \text{ kip-ft}$ = 128 kip-ft
$M_{uy} = 1.2(11.3 \text{ kip-ft}) + 1.6(33.8 \text{ kip-ft})$ = 67.6 kip-ft	$M_{ay} = 11.3 \text{ kip-ft} + 33.8 \text{ kip-ft}$ = 45.1 kip-ft

Verify that a W14×82 has sufficient available strength

**Geometric Properties:**

$$\begin{aligned} \text{W14} \times 82 \quad A &= 24.0 \text{ in.}^2 & S_x &= 123 \text{ in.}^3 & Z_x &= 139 \text{ in.}^3 & S_y &= 29.3 \text{ in.}^3 \\ & Z_y &= 44.8 \text{ in.}^3 & I_y &= 148 \text{ in.}^4 & L_p &= 8.76 \text{ ft} & L_r &= 33.1 \text{ ft} \end{aligned}$$

Manual  
Table 1-1  
Table 3-2

Calculate the nominal gross tensile strength

$$P_n = F_y A_g = (50 \text{ ksi})(24.0 \text{ in.}^2) = 1200 \text{ kips}$$

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

$$M_{nx} = M_p = F_y Z_x = 50 \text{ ksi}(139 \text{ in.}^3) = 6950 \text{ kip-in} = 579 \text{ kip-ft}$$

Eqn. F2-1

Lateral-torsional buckling limit state

$$L_b = 30.0 \text{ ft}$$

Since  $L_p < L_b \leq L_r$ , Equation F2-2 applies

Calculate lateral-torsion buckling modification factor

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
$\sqrt{1 + \frac{P_u}{P_{ey}}} = \sqrt{1 + \frac{174 \text{ kips}}{327 \text{ kips}}} = 1.24$	$\sqrt{1 + \frac{1.5P_a}{P_{ey}}} = \sqrt{1 + \frac{1.5(116 \text{ kips})}{327 \text{ kips}}} = 1.24$

$$C_b = 1.24(1.14) = 1.41$$

$$M_n = C_b \left[ M_p - (M_p - 0.7F_y S_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad \text{Eqn. F2-2}$$

$$M_n = 1.41 \left[ 6950 \text{ kip-in.} - (6950 \text{ kip-in.} - 0.7(50 \text{ ksi})(123 \text{ in.}^3)) \left( \frac{30.0 \text{ ft} - 8.76 \text{ ft}}{33.1 \text{ ft} - 8.76 \text{ ft}} \right) \right]$$

$$= 6550 \text{ kip} < M_p \text{ therefore use:}$$

$$M_n = 6550 \text{ kip-in. or } 545 \text{ kip-ft}$$

**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.

Manual  
Table 1-1

Calculate the nominal flexural strength for bending about the y-y axis

*Yielding limit state*

Since W14×82 has compact flanges, only the limit state of yielding applies.

$$M_{ny} = M_p = F_y Z_y \leq 1.6 F_y S_y \quad \text{Eqn. F6-1}$$

$$= 50 \text{ ksi}(44.8 \text{ in.}^3) \leq 1.6(50 \text{ ksi})(29.3 \text{ in.}^3)$$

$$= 2240 \text{ kip-in.} < 2344 \text{ kip-in.}, \text{ therefore use:}$$

$$M_{ny} = 2240 \text{ kip-in. or } 187 \text{ kip-ft}$$

LRFD	ASD
$\phi_b = \phi_t = 0.90$	$\Omega_b = \Omega_t = 1.67$
$P_c = \phi_t P_n = 0.90(1200 \text{ kips}) = 1080 \text{ kips}$	$P_c = P_n / \Omega_t = \frac{1200 \text{ kips}}{1.67} = 719 \text{ kips}$
$M_{cx} = \phi_b M_{nx} = 0.90(545 \text{ kip-ft}) = 491 \text{ kip-ft}$	$M_{cx} = M_{nx} / \Omega_b = \frac{545 \text{ kip-ft}}{1.67} = 327 \text{ kip-ft}$
$M_{cy} = \phi_b M_{ny} = 0.90(187 \text{ kip-ft}) = 168 \text{ kip-ft}$	$M_{cy} = M_{ny} / \Omega_b = \frac{187 \text{ kip-ft}}{1.67} = 112 \text{ kip-ft}$

Sections D2 and F1

Check limit for Equation H1-1a

LRFD	ASD
$\frac{P_r}{\phi_t P_n} = \frac{P_u}{\phi_t P_n} = \frac{174 \text{ kips}}{1080 \text{ kips}} = 0.161 < 0.2$	$\frac{P_r}{P_n / \Omega_t} = \frac{P_a}{P_n / \Omega_t} = \frac{116 \text{ kips}}{719 \text{ kips}} = 0.161 < 0.2$

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
$\frac{174 \text{ kips}}{2(1080 \text{ kips})} + \left( \frac{192 \text{ kip-ft}}{491 \text{ kip-ft}} + \frac{67.6 \text{ kip-ft}}{168 \text{ kip-ft}} \right) \leq 1.0$	$\frac{116 \text{ kips}}{2(719 \text{ kips})} + \left( \frac{128 \text{ kip-ft}}{327 \text{ kip-ft}} + \frac{45.1 \text{ kip-ft}}{112 \text{ kip-ft}} \right) \leq 1.0$
0.874 < 1.0 <b>o.k.</b>	0.875 < 1.0 <b>o.k.</b>

### 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, not including second-order effects:

$$\begin{aligned} M_{xD} &= 15 \text{ kip-ft} & M_{xL} &= 45 \text{ kip-ft} \\ M_{yD} &= 2 \text{ kip-ft} & M_{yL} &= 6 \text{ kip-ft} \end{aligned}$$

The member is not subject to sidesway.

#### Solution:

#### Material Properties:

ASTM A992       $F_y = 50 \text{ ksi}$        $F_u = 65 \text{ ksi}$

Calculate the required strength, not considering second-order effects

LRFD	ASD
$P_u = 1.2(5.00 \text{ kips}) + 1.6(15.0 \text{ kips})$ = 30.0 kips	$P_a = 5.00 \text{ kips} + 15.0 \text{ kips}$ = 20.0 kips
$M_{ux} = 1.2(15.0 \text{ kip-ft}) + 1.6(45.0 \text{ kip-ft})$ = 90.0 kip-ft	$M_{ax} = 15.0 \text{ kip-ft} + 45.0 \text{ kip-ft}$ = 60.0 kip-ft
$M_{uy} = 1.2(2.00 \text{ kip-ft}) + 1.6(6.00 \text{ kip-ft})$ = 12.0 kip-ft	$M_{ay} = 2.00 \text{ kip-ft} + 6.00 \text{ kip-ft}$ = 8.00 kip-ft

Try a W10×33

#### Geometric Properties:

W10×33       $A = 9.71 \text{ in.}^2$        $S_x = 35.0 \text{ in.}^3$        $Z_x = 38.8 \text{ in.}^3$        $I_x = 171 \text{ in.}^4$   
 $S_y = 9.20 \text{ in.}^3$        $Z_y = 14.0 \text{ in.}^3$        $I_y = 36.6 \text{ in.}^4$   
 $L_p = 6.85 \text{ ft}$        $L_r = 12.8 \text{ ft}$

Manual  
Table 1-1  
Table 3-1

Calculate the available axial strength

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

Commentary  
Table  
C-C2.2

Since  $KL_x = KL_y = 14.0 \text{ ft}$  and  $r_x > r_y$ , the y-y axis will govern.

LRFD	ASD
$P_c = \phi_c P_n = 253 \text{ kips}$	$P_c = P_n / \Omega_c = 168 \text{ kips}$

Manual  
Table 4-1

Calculate the required flexural strengths including second order amplification

Use “Amplified First-Order Elastic Analysis” procedure from Section C2.1b. Since the member is not subject to sidesway, only  $P$ - $\delta$  amplifiers need to be added.

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \tag{Eqn. C2-2}$$

$$C_m = 1.0$$

*X-X axis flexural magnifier*

$$P_{e1} = \frac{\pi^2 EI_x}{(K_1 L_x)^2} = \frac{\pi^2 (29,000 \text{ ksi})(171 \text{ in.}^4)}{((1.0)(14.0 \text{ ft})(12 \text{ in./ft}))^2} = 1730 \text{ kips} \tag{Eqn. C2-5}$$

LRFD	ASD
$\alpha = 1.0$ $B_1 = \frac{1.0}{1 - 1.0(30.0 \text{ kips} / 1730 \text{ kips})} = 1.02$ $M_{ux} = 1.02(90.0 \text{ kip-ft}) = 91.8 \text{ kip-ft}$	$\alpha = 1.6$ $B_1 = \frac{1.0}{1 - 1.6(20.0 \text{ kips} / 1730 \text{ kips})} = 1.02$ $M_{ux} = 1.02(60.0 \text{ kip-ft}) = 61.2 \text{ kip-ft}$

Eqn. C2-2

*Y-Y axis flexural magnifier*

$$P_{e1} = \frac{\pi^2 EI_y}{(K_1 L_y)^2} = \frac{\pi^2 (29,000 \text{ ksi})(36.6 \text{ in.}^4)}{((1.0)(14.0 \text{ ft})(12 \text{ in./ft}))^2} = 371 \text{ kips} \tag{Eqn. C2-5}$$

LRFD	ASD
$\alpha = 1.0$ $B_1 = \frac{1.0}{1 - 1.0(30.0 \text{ kips} / 371 \text{ kips})} = 1.09$ $M_{uy} = 1.09(12.0 \text{ kip-ft}) = 13.1 \text{ kip-ft}$	$\alpha = 1.6$ $B_1 = \frac{1.0}{1 - 1.6(20.0 \text{ kips} / 371 \text{ kips})} = 1.09$ $M_{uy} = 1.09 (8.00 \text{ kip-ft}) = 8.76 \text{ kip-ft}$

Eqn. C2-2

Calculate the nominal bending strength about the x-x axis

*Yielding limit state*

$$M_{nx} = M_p = F_y Z_x = 50 \text{ ksi}(38.8 \text{ in.}^3) = 1940 \text{ kip-in or } 162 \text{ kip-ft} \tag{Eqn. F2-1}$$

*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



$$M_{nx} = C_b \left[ M_p - (M_p - 0.7F_y S_x) \left( \frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \quad \text{Eqn. F2-2}$$

$$M_{nx} = 1.14 \left[ 1940 \text{ kip-in.} - (1940 \text{ kip-in.} - 0.7(50 \text{ ksi})(35.0 \text{ in.}^3)) \left( \frac{14.0 \text{ ft} - 6.85 \text{ ft}}{21.8 \text{ ft} - 6.85 \text{ ft}} \right) \right]$$

= 1820 kip-in. ≤ 1940 kip-in., therefore use:

$M_{nx} = 1820 \text{ kip-in.}$  or  $152 \text{ kip-ft}$                       **controls**

*Local buckling limit state*

Per Manual Table 1-1, the member is compact for  $F_y = 50 \text{ ksi}$ , so the local buckling limit state does not apply

Manual  
Table 1-1

*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.

$$M_{ny} = M_p = F_y Z_y \leq 1.6 F_y S_y$$

$$= 50 \text{ ksi}(14.0 \text{ in.}^3) \leq 1.6(50 \text{ ksi})(9.20 \text{ in.}^3)$$

$$= 700 \text{ kip-in.} < 736 \text{ kip-in.}, \text{ therefore}$$

Eqn. F6-1

Use  $M_{ny} = 700 \text{ kip-in.}$  or  $58.3 \text{ kip-ft}$

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 F2

*Check limit for Equation H1-1a*

LRFD	ASD
$\frac{P_r}{P_c} = \frac{P_u}{\phi_c P_n} = \frac{30.0 \text{ kips}}{253 \text{ kips}} = 0.119, \text{ therefore,}$ use Specification Equation H1.1b $\frac{P_r}{2P_c} + \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0$ $\frac{30.0 \text{ kips}}{2(253 \text{ kips})} + \left( \frac{91.8 \text{ kip-ft}}{137 \text{ kip-ft}} + \frac{13.1 \text{ kip-ft}}{52.5 \text{ kip-ft}} \right)$ $0.0593 + 0.920 = 0.979 \leq 1.0 \quad \mathbf{o.k.}$	$\frac{P_r}{P_c} = \frac{P_a}{P_n / \Omega_c} = \frac{20.0 \text{ kips}}{168 \text{ kips}} = 0.119, \text{ therefore}$ use Specification Equation H1.1b $\frac{P_r}{2P_c} + \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0$ $\frac{20.0 \text{ kips}}{2(168 \text{ kips})} + \left( \frac{61.2 \text{ kip-ft}}{91.0 \text{ kip-ft}} + \frac{8.76 \text{ kip-ft}}{34.9 \text{ kip-ft}} \right)$ $0.0595 + 0.924 = 0.983 \leq 1.0 \quad \mathbf{o.k.}$

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:**

$$\text{ASTM A500 Gr. B} \quad F_y = 46 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Manual  
Table 2-3

**Geometric properties:**

$$\text{HSS6} \times 4 \times \frac{1}{4} \quad \frac{h}{t} = 22.8 \quad \frac{b}{t} = 14.2 \quad t = 0.233 \text{ in.}$$

Manual  
Table 1-11

Evaluate wall slenderness to determine the appropriate critical stress equation

$$\frac{h}{t} > \frac{b}{t}, \text{ therefore, } \frac{h}{t} \text{ governs.}$$

$$\frac{h}{t} \leq 2.45 \sqrt{\frac{E}{F_y}}$$

$$22.8 \leq 2.45 \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} = 61.5 \text{ therefore, use Eqn. H3-3}$$

$$F_{cr} = 0.6 F_y = 0.6(46 \text{ ksi}) = 27.6 \text{ ksi}$$

Eqn. H3-3

Calculate the nominal torsional strength

$$T_n = F_{cr} C = 27.6 \text{ ksi} (10.1 \text{ in.}^3) = 278 \text{ kip-in.}$$

Eqn. H3-1

$$C = 10.1$$

Manual  
Table 1-11

Calculate the available torsional strength

LRFD	ASD
$\phi_T = 0.90$ $\phi_T T_n = 0.90(278 \text{ kip-in.}) = 250 \text{ kip-in.}$	$\Omega_T = 1.67$ $T_n / \Omega_T = 278 \text{ kip-in.} / 1.67 = 166 \text{ kip-in.}$

Section H3.1

Note: For more complete guidance on design 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:**

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

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D} \left(\frac{D}{t}\right)^4}} = \frac{1.23(29,000 \text{ ksi})}{\sqrt{\frac{14.0 \text{ ft} (12 \text{ in./ft})}{5.00 \text{ in.}} (21.5)^4}} = 133 \text{ ksi}$$

Eqn. H3-2a

and

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^2} = \frac{0.60(29,000 \text{ ksi})}{(21.5)^2} = 175 \text{ ksi}$$

Eqn. H3-2b

However,  $F_{cr}$  shall not exceed  $0.60 F_y = 0.60(42 \text{ ksi}) = 25.2$  ksi, therefore,

Section H3.1

$$F_{cr} = 25.2 \text{ ksi}$$

Calculate the torsional shear constant

$$C = 7.95 \text{ in.}^3$$

Manual  
Table 1-13

Calculate the nominal torsional strength

$$T_n = F_{cr}C = 25.2 \text{ ksi} (7.95 \text{ in.}^3) = 200 \text{ kip-in.}$$

Eqn. H3-1

Calculate the available torsional strength

LRFD	ASD
$\phi_T = 0.90$ $\phi_T T_n = 0.90(200 \text{ kip-in.}) = 180 \text{ kip-in.}$	$\Omega_T = 1.67$ $T_n/\Omega_T = 200 \text{ kip-in.}/1.67 = 120 \text{ kip-in.}$

Section H3.1

Note: For more complete guidance on design for torsion, see AISC Design Guide 9, *Torsional Analysis of Structural Steel Members*.

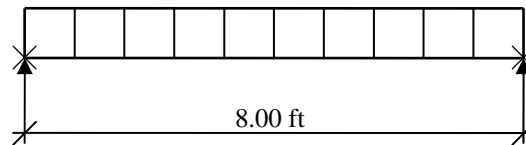
### Example H.5c HSS Combined Torsional and Flexural Strength

#### Given:

Verify the strength of an ASTM A500 Gr. B HSS6×4×1/4 loaded as shown. The beam is simply supported with torsionally fixed ends.

$$w_D = 0.460 \text{ kip/ft applied 6 in. off centerline}$$

$$w_L = 1.38 \text{ kip/ft applied 6 in. off centerline}$$



#### Solution:

##### Material Properties:

$$\text{ASTM A500 Gr. B} \quad F_y = 46 \text{ ksi} \quad F_u = 58 \text{ ksi}$$

Manual  
Table 2-3

##### Geometric properties:

$$\text{HSS } 6 \times 4 \times \frac{1}{4} \quad \frac{h}{t} = 22.8 \quad \frac{b}{t} = 14.2 \quad t = 0.233 \text{ in.} \quad Z_x = 8.53 \text{ in.}^3$$

Manual  
Table 1-11

Calculate the required strengths

LRFD	ASD
$w_u = 1.2(0.460 \text{ kip/ft}) + 1.6(1.38 \text{ kip/ft})$ $= 2.76 \text{ kip/ft}$	$w_a = 0.460 \text{ kip/ft} + 1.38 \text{ kip/ft}$ $= 1.84 \text{ kip/ft}$
$V_r = V_u = \frac{w_u l}{2}$ $= \frac{2.76 \text{ kip/ft}(8.00 \text{ ft})}{2} = 11.0 \text{ kips}$	$V_r = V_a = \frac{w_a l}{2}$ $= \frac{1.84 \text{ kip/ft}(8.00 \text{ ft})}{2} = 7.36 \text{ kips}$
$M_r = M_u = \frac{w_u l^2}{8}$ $= \frac{2.76 \text{ kip/ft}(8.00 \text{ ft})^2 (12 \text{ in./ft})}{8}$ $= 265 \text{ kip-in.}$	$M_r = M_a = \frac{w_a l^2}{8}$ $= \frac{1.84 \text{ kip/ft}(8.00 \text{ ft})^2 (12 \text{ in./ft})}{8}$ $= 177 \text{ kip-in.}$
$T_r = T_u = \frac{w_u l e}{2}$ $= \frac{2.76 \text{ kip/ft}(8.00 \text{ ft})(6.00 \text{ in.})}{2}$ $= 66.2 \text{ kip-in.}$	$T_r = T_a = \frac{w_a l e}{2}$ $= \frac{1.84 \text{ kip/ft}(8.00 \text{ ft})(6.00 \text{ in.})}{2}$ $= 44.2 \text{ kip-in.}$

Calculate the nominal shear strength

$$h = 6.00 \text{ in.} - 3(0.233 \text{ in.}) = 5.30 \text{ in.}$$

Section G5

$$A_w = 2ht = 2(5.30 \text{ in.})(0.233 \text{ in.}) = 2.47 \text{ in.}^2$$

$$k_v = 5$$

Calculate the web shear coefficient

$$\frac{h}{t_w} = 22.8 \leq 1.10 \sqrt{\frac{k_v E}{F_y}} = 1.10 \sqrt{\frac{5(29,000 \text{ ksi})}{46 \text{ ksi}}} = 61.7 \text{ therefore, } C_v = 1.0$$

Eqn. G2-3

$$V_n = 0.6F_y A_w C_v = 0.6(46 \text{ ksi})(2.47 \text{ in.}^2)(1.0) = 68.2 \text{ kips}$$

Eqn. G2-1

Calculate the available shear strength

LRFD	ASD
$\phi_v = 0.90$ $V_c = \phi_v V_n = 0.90(68.2 \text{ kips}) = 61.4 \text{ kips}$	$\Omega_v = 1.67$ $V_c = V_n / \Omega_v = 68.2 \text{ kips} / 1.67 = 40.8 \text{ kips}$

Section G1

Calculate the nominal flexural strength

Section F7

Flexural yielding limit state

$$M_n = M_p = F_y Z_x = 46 \text{ ksi}(8.53 \text{ in.}^3) = 392 \text{ kip-in.}$$

Eqn. F7-1

Flange local buckling limit state

$$\frac{b}{t} = 14.2 < 1.12 \sqrt{\frac{E}{F_y}} = 28.1 \text{ therefore the flange is compact and the flange local buckling}$$

Table B4.1  
Case 12

limit state does not apply.

Web local buckling limit state

$$\frac{h}{t} = 22.8 < 2.42 \sqrt{\frac{E}{F_y}} = 60.8 \text{ therefore the the web is compact and the web local buckling}$$

Table B4.1  
Case 13

limit state does not apply.

Therefore  $M_n = 392 \text{ kip-in.}$ , controlled by the flexural yielding limit state.

Calculate the available flexural strength

LRFD	ASD
$\phi_b = 0.90$ $M_c = \phi_b M_n = 0.90(392 \text{ kip-in.}) = 353 \text{ kip-in.}$	$\Omega_b = 1.67$ $M_c = M_n / \Omega_b = 392 \text{ kip-in.} / 1.67 = 235 \text{ kip-in.}$

Section F1

Take the available torsional strength from **Example H.5a**

$T_c = \phi_T T_n = 0.90(278 \text{ kip-in.}) = 250 \text{ kip-in.}$	$T_c = T_n / \Omega_T = 278 \text{ kip-in.} / 1.67 = 166 \text{ kip-in.}$
--	---

Ex. H.5a

Check combined strength

LRFD	ASD
$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0$	$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0$
$\left(0 + \frac{265 \text{ kip-in.}}{353 \text{ kip-in.}}\right)$	$\left(0 + \frac{177 \text{ kip-in.}}{235 \text{ kip-in.}}\right)$
$+ \left(\frac{11.0 \text{ kips}}{61.4 \text{ kips}} + \frac{66.2 \text{ kip-in.}}{250 \text{ kip-in.}}\right)^2$	$+ \left(\frac{7.36 \text{ kips}}{40.8 \text{ kips}} + \frac{44.2 \text{ kip-in.}}{166 \text{ kip-in.}}\right)^2$
<p>=0.948 &lt; 1.0 <b>o.k.</b></p>	<p>= 0.953 &lt; 1.0 <b>o.k.</b></p>

Eqn. H3-6

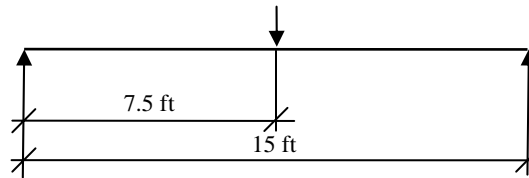
### Example H.6 W-Shape Torsional Strength.

This design example is taken from AISC *Design Guide 9 – Torsion 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.

**Given:**

$$P_D = 2.50 \text{ kips applied 6 in. off centerline}$$

$$P_L = 7.50 \text{ kips applied 6 in. off centerline}$$



Beam Loading Diagram

The end conditions are assumed to be flexurally and torsionally pinned. The eccentric load can be resolved into a torsional moment and a load applied through the shear center.

**Solution:**

**Material Properties:**

ASTM A992     $F_y = 50 \text{ ksi}$      $F_u = 65 \text{ ksi}$

Manual  
Table 2-3

**Geometric Properties:**

W10×49     $I_x = 272 \text{ in.}^4$      $S_x = 54.6 \text{ in.}^3$      $t_f = 0.560 \text{ in.}$      $t_w = 0.340 \text{ in.}$   
 $J = 1.39 \text{ in.}^4$      $C_w = 2070 \text{ in.}^6$

Manual  
Table 1-1

**Additional Torsional Properties:**

W10×49     $S_{wl} = 33.0 \text{ in.}^4$      $a = 62.1 \text{ in.}$      $W_{no} = 23.6 \text{ in.}^2$   
 $Q_f = 13.0 \text{ in.}^3$      $Q_w = 30.2 \text{ in.}^3$

Design  
Guide 9  
Appendix A

Calculate the required strength

LRFD	ASD
$P_u = 1.2(2.50 \text{ kips}) + 1.6(7.50 \text{ kips})$ $= 15.0 \text{ kips}$	$P_a = 2.50 \text{ kips} + 7.50 \text{ kips}$ $= 10.0 \text{ kips}$
$V_u = \frac{P_u}{2} = \frac{15.0 \text{ kips}}{2} = 7.50 \text{ kips}$	$V_a = \frac{P_a}{2} = \frac{10.0 \text{ kips}}{2} = 5.00 \text{ kips}$
$M_u = \frac{P_u l}{4} = \frac{15.0 \text{ kips}(15.0 \text{ ft})(12 \text{ in./ft})}{4}$ $= 675 \text{ kip-in.}$	$M_a = \frac{P_a l}{4} = \frac{10.0 \text{ kips}(15.0 \text{ ft})(12 \text{ in./ft})}{4}$ $= 450 \text{ kip-in.}$
$T_u = P_u e = 15.0 \text{ kips}(6.00 \text{ in.}) = 90.0 \text{ kip-in.}$	$T_a = P_a e = 10.0 \text{ kips}(6.00 \text{ in.}) = 60.0 \text{ kip-in.}$

Calculate the normal and shear stresses from flexure

LRFD	ASD	
$\sigma_{bx_u} = \frac{M_u}{S_x} = \frac{675 \text{ kip-in.}}{54.6 \text{ in.}^3} = 12.4 \text{ ksi}$ <p>(compression at top, tension at bottom)</p>	$\sigma_{bx_a} = \frac{M_a}{S_x} = \frac{450 \text{ kip-in.}}{54.6 \text{ in.}^3} = 8.24 \text{ ksi}$ <p>(compression at top, tension at bottom)</p>	Design Guide 9 Eqn.4.5
$\tau_{bw_u} = \frac{V_u Q_w}{I_x t_w} = \frac{7.5 \text{ kips} (30.2 \text{ in.}^3)}{272 \text{ in.}^4 (0.340 \text{ in.})} = 2.45 \text{ ksi}$	$\tau_{bw_a} = \frac{V_a Q_w}{I_x t_w} = \frac{5.00 \text{ kips} (30.2 \text{ in.}^3)}{272 \text{ in.}^4 (0.340 \text{ in.})} = 1.63 \text{ ksi}$	Design Guide 9 Eqn. 4.6
$\tau_{bf_u} = \frac{V_u Q_f}{I_x t_f} = \frac{7.5 \text{ kips} (13.0 \text{ in.}^3)}{272 \text{ in.}^4 (0.560 \text{ in.})} = 0.640 \text{ ksi}$	$\tau_{bf_a} = \frac{V_a Q_f}{I_x t_f} = \frac{5.00 \text{ kips} (13.0 \text{ in.}^3)}{272 \text{ in.}^4 (0.560 \text{ in.})} = 0.427 \text{ ksi}$	Design Guide 9 Eqn. 4.6

Calculate torsional stresses

The following functions are taken from AISC's Design Guide 9 – *Torsion Analysis of Structural Steel Members* Appendix B, Case 3, with  $\alpha = 0.5$ .

$$\frac{l}{a} = \frac{180 \text{ in.}}{62.1 \text{ in.}} = 2.90$$

Design Guide 9 Appendix B Case 3,  $\alpha = 0.5$

At midspan ( $z/l = 0.5$ )

Using the graphs for  $\theta$ ,  $\theta''$ ,  $\theta'$  and  $\theta'''$ , select values

For  $\theta$ ;  $\theta \times \left(\frac{GJ}{T_r}\right) \left(\frac{1}{l}\right) = +0.09$       Solve for  $\theta = +0.09 \frac{T_r l}{GJ}$

Design Guide 9 Appendix B Case 3,  $\alpha = 0.5$

For  $\theta''$ ;  $\theta'' \times \left(\frac{GJ}{T_r}\right) a = -0.44$       Solve for  $\theta'' = -0.44 \frac{T_r l}{GJa}$

For  $\theta'$ ;  $\theta' \times \left(\frac{GJ}{T_r}\right) = 0$       Therefore  $\theta' = 0$

For  $\theta'''$ ;  $\theta''' \times \left(\frac{GJ}{T_r}\right) a^2 = -0.50$       Solve for  $\theta''' = -0.50 \frac{T_r l}{GJa^2}$

At the support ( $z/l = 0$ )

For  $\theta$ ;  $\theta \times \left(\frac{GJ}{T_r}\right) \left(\frac{1}{l}\right) = 0$       Therefore  $\theta = 0$

Design Guide 9 Appendix B Case 3,  $\alpha = 0.5$

For  $\theta''$ ;  $\theta'' \times \left(\frac{GJ}{T_r}\right) a = 0$       Therefore  $\theta'' = 0$

For  $\theta'$ ;  $\theta' \times \left(\frac{GJ}{T_r}\right) = +0.28$       Solve for  $\theta' = +0.28 \frac{T_r}{GJ}$

For  $\theta'''$ ;  $\theta''' \times \left(\frac{GJ}{T_r}\right) a^2 = -0.22$       Solve for  $\theta''' = -0.22 \frac{T_r l}{GJa^2}$

In the above calculations note that the applied torque is negative with the sign convention used.



Calculate  $\frac{T_r}{GJ}$  for use below

LRFD	ASD
$\frac{T_u}{GJ} = \frac{-90.0 \text{ kip-in.}}{(11,200 \text{ ksi})(1.39 \text{ in.}^4)}$ $= -5.78 \times 10^{-3} \text{ rad/in.}$	$\frac{T_a}{GJ} = \frac{-60.0 \text{ kip-in.}}{(11,200 \text{ ksi})(1.39 \text{ in.}^4)}$ $= -3.85 \times 10^{-3} \text{ rad/in.}$

Calculate the shear stresses due to pure torsion

$$\tau_t = Gt\theta'$$

Design  
Guide 9  
Eqn. 4.1

LRFD	ASD
<p>At midspan <math>\theta' = 0; \tau_{ut} = 0</math></p> <p>At the support, for the web;  <math display="block">\tau_{ut} = 11,200 \text{ ksi}(0.340 \text{ in.})(0.28) \left( \frac{-5.78 \text{ rad}}{10^3 \text{ in.}} \right)</math> <math display="block">= -6.16 \text{ ksi}</math></p> <p>At the support, for the flange;  <math display="block">\tau_{ut} = 11,200 \text{ ksi}(0.560 \text{ in.})(0.28) \left( \frac{-5.78 \text{ rad}}{10^3 \text{ in.}} \right)</math> <math display="block">= -10.2 \text{ ksi}</math></p>	<p>At midspan <math>\theta' = 0; \tau_{at} = 0</math></p> <p>At the support, for the web;  <math display="block">\tau_{at} = 11,200 \text{ ksi}(0.340 \text{ in.})(0.28) \left( \frac{-3.85 \text{ rad}}{10^3 \text{ in.}} \right)</math> <math display="block">= -4.11 \text{ ksi}</math></p> <p>At the support, for the flange;  <math display="block">\tau_{at} = 11,200 \text{ ksi}(0.560 \text{ in.})(0.28) \left( \frac{-3.85 \text{ rad}}{10^3 \text{ in.}} \right)</math> <math display="block">= -6.76 \text{ ksi}</math></p>

Design  
Guide 9  
Eqn. 4.1

Design  
Guide 9  
Eqn. 4.1

Calculate the shear stresses due to warping

$$\tau_t = \frac{-ES_{wt}\theta''}{t_f}$$

Design  
Guide 9  
Eqn. 4.2a

LRFD	ASD
<p>At midspan  <math display="block">\tau_{tw} = \frac{-29,000 \text{ ksi}(33.0 \text{ in.}^4)}{0.560 \text{ in.}} \left[ \frac{-0.50(-5.78 \text{ rad})}{(62.1 \text{ in.})^2 10^3} \right]</math> <math display="block">= -1.28 \text{ ksi}</math></p> <p>At the support  <math display="block">\tau_{tw} = \frac{-29,000 \text{ ksi}(33.0 \text{ in.}^4)}{0.560 \text{ in.}} \left[ \frac{-0.22(-5.78 \text{ rad})}{(62.1 \text{ in.})^2 10^3} \right]</math> <math display="block">= -0.564 \text{ ksi}</math></p>	<p>At midspan  <math display="block">\tau_{tw} = \frac{-29,000 \text{ ksi}(33.0 \text{ in.}^4)}{0.560 \text{ in.}} \left[ \frac{-0.50(-3.85 \text{ rad})}{(62.1 \text{ in.})^2 10^3} \right]</math> <math display="block">= -0.853 \text{ ksi}</math></p> <p>At the support  <math display="block">\tau_{tw} = \frac{-29,000 \text{ ksi}(33.0 \text{ in.}^4)}{0.560 \text{ in.}} \left[ \frac{-0.22(-3.85 \text{ rad})}{(62.1 \text{ in.})^2 10^3} \right]</math> <math display="block">= -0.375 \text{ ksi}</math></p>

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 $\sigma_{uw} = 29,000 \text{ ksi} (23.6 \text{ in.}^2) \left[ \frac{-0.44(-5.78 \text{ rad})}{(62.1 \text{ in.})10^3} \right]$ $= 28.0 \text{ ksi}$	At midspan $\sigma_{aw} = 29,000 \text{ ksi} (23.6 \text{ in.}^2) \left[ \frac{-0.44(-3.85 \text{ rad})}{(62.1 \text{ in.})10^3} \right]$ $= 18.7 \text{ ksi}$
At the support Since $\theta'' = 0, \sigma_{uw} = 0$	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 (ksi)														
Location	LRFD							ASD						
	Normal Stresses			Shear Stresses				Normal Stresses			Shear Stresses			
	$\sigma_{uw}$	$\sigma_{ub}$	$f_{un}$	$\tau_{ut}$	$\tau_{uw}$	$\tau_{ub}$	$f_{uv}$	$\sigma_{aw}$	$\sigma_{ab}$	$f_{an}$	$\tau_{at}$	$\tau_{aw}$	$\tau_{ab}$	$f_{av}$
Midspan														
Flange	±28.1	±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	-0.564	±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
$\phi_T = 0.90$  <i>Normal Stresses</i>  <i>Yielding limit state</i>  $F_n = F_y = 50 \text{ ksi}$ <b>controls</b>  <i>Lateral-torsional buckling limit state</i>  $C_b = 1.32$	$\Omega_T = 1.67$  <i>Normal Stresses</i>  <i>Yielding limit state</i>  $F_n = F_y = 50 \text{ ksi}$ <b>controls</b>  <i>Lateral-torsional buckling limit state</i>  $C_b = 1.32$

Section H3.3

Eqn. H3-7

Manual  
Table 3-1  
Case 1

<p>Compute <math>F_n</math> using values from Table 3-10. with <math>L_b = 15.0</math> ft and <math>C_b = 1.0</math></p> <p><math>\phi_b M_n = 206</math> kip-ft</p> $F_n = F_{cr} = C_b \frac{\phi_b M_n}{\phi_b S_x} = 1.32 \frac{206 \text{ kip-ft}}{0.90(54.6 \text{ in.}^3)} \left( \frac{12 \text{ in.}}{\text{ft}} \right) = 66.4 \text{ ksi} > 50 \text{ ksi} \quad \text{does not control}$ <p><math>\phi_T F_n = 0.9(50 \text{ ksi}) = 45.0 \text{ ksi} &gt; 40.4 \text{ ksi} \quad \text{o.k.}</math></p> <p><i>Shear yielding limit state</i></p> <p><math>F_n = 0.6F_y</math></p> <p><i>Design shear strength</i></p> <p><math>\phi_T F_n = 0.90(0.6)(50 \text{ ksi}) = 27.0 \text{ ksi} &gt; 11.4 \text{ ksi} \quad \text{o.k.}</math></p>	<p>Compute <math>F_n</math> using values from Table 3-10. with <math>L_b = 15.0</math> ft and <math>C_b = 1.0</math></p> <p><math>M_n/\Omega_b = 138</math> kip-ft</p> $F_n = F_{cr} = C_b \Omega_b \frac{M_n/\Omega_b}{S_x} = 1.32(1.67) \frac{138 \text{ kip-ft}}{(54.6 \text{ in.}^3)} \left( \frac{12 \text{ in.}}{\text{ft}} \right) = 66.9 \text{ ksi} > 50 \text{ ksi} \quad \text{does not control}$ <p><math>F_n/\Omega_T = 50 \text{ ksi}/1.67 = 30.0 \text{ ksi} &gt; 26.9 \text{ ksi} \quad \text{o.k.}</math></p> <p><i>Shear yielding limit state</i></p> <p><math>F_n = 0.6F_y</math></p> <p><i>Allowable shear strength</i></p> <p><math>F_n/\Omega_T = (0.6)(50 \text{ ksi})/1.67 = 18.0 \text{ ksi} &gt; 7.56 \text{ ksi} \quad \text{o.k.}</math></p>	<p>Manual Table 3-10</p> <p>Eqn. H3-9</p> <p>Eqn. H3-8</p>
--	--	--

Calculate the maximum rotation at service load

The maximum rotation occurs at midspan. The service-load torque is:

$$T = Pe = -(2.50 \text{ kips} + 7.50 \text{ kips})(6.00 \text{ in.}) = -60.0 \text{ kip-in.}$$

The maximum rotation is:

$$\theta = +0.9 \frac{Tl}{GJ} = \frac{0.09(-60.0 \text{ kip-in.})(180 \text{ in.})}{11,200 \text{ ksi}(1.39 \text{ in.}^4)} = -0.624 \text{ rads} = -3.58 \text{ degrees}$$

Design Guide 9 Eqn. 4.1

# 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

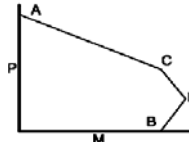
Because a plastic stress distribution is used for both LRFD and ASD designs, the flexural strength of composite beams is generally greater than that of former ASD designs. Shear connectors, in many cases, have lower horizontal shear strength than was permitted in past LRFD specifications. Designers are 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.

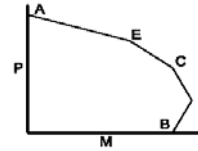
**PLASTIC CAPACITIES FOR RECTANGULAR, ENCASED W-SHAPES BENT ABOUT THE X-X AXIS**



Section	Stress Distribution	Point	Defining Equations
<p>(A)</p>		A	$P_A = A_s F_y + A_{sr} F_{yr} + 0.85 f'_c A_c$ $M_A = 0$ <p> <math>A_s</math> = area of steel shape  <math>A_{sr}</math> = area of all continuous reinforcing bars  <math>A_c = h_1 h_2 - A_s - A_{sr}</math> </p>
<p>(C)</p>		C	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
<p>(D)</p>		D	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = Z_s F_y + Z_r F_{yr} + \frac{1}{2} Z_c (0.85 f'_c)$ <p> <math>Z_s</math> = full x-axis plastic section modulus of steel shape  <math>A_{srs}</math> = area of continuous reinforcing bars at the centerline  <math>Z_r = (A_{sr} - A_{srs}) \left( \frac{h_2}{2} - c \right)</math>  <math>Z_c = \frac{h_1 h_2^2}{4} - Z_s - Z_r</math> </p>
<p>(B)</p>		B	$P_B = 0$ $M_B = M_D - Z_{sn} F_y - \frac{1}{2} Z_{cn} (0.85 f'_c)$ $Z_{cn} = h_1 h_n^2 - Z_{sn}$ <p>For <math>h_n</math> below the flange (<math>h_n \leq \frac{d}{2} - t_f</math>)</p> $h_n = \frac{0.85 f'_c (A_c + A_{srs}) - 2 F_{yr} A_{srs}}{2 [0.85 f'_c (h_1 - t_w) + 2 F_y t_w]}$ $Z_{sn} = t_w h_n^2$ <p>For <math>h_n</math> within the flange (<math>\frac{d}{2} - t_f &lt; h_n \leq \frac{d}{2}</math>)</p> $h_n = \frac{0.85 f'_c (A_c + A_s - db_f + A_{srs}) - 2 F_y (A_s - db_f) - 2 F_{yr} A_{srs}}{2 [0.85 f'_c (h_1 - b_f) + 2 F_y b_f]}$ $Z_{sn} = Z_s - b_f \left( \frac{d}{2} - h_n \right) \left( \frac{d}{2} + h_n \right)$ <p>For <math>h_n</math> above the flange (<math>h_n &gt; \frac{d}{2}</math>)</p> $h_n = \frac{0.85 f'_c (A_c + A_s + A_{srs}) - 2 F_y A_s - 2 F_{yr} A_{srs}}{2 (0.85 f'_c h_1)}$ $Z_{sn} = Z_{sx} = \text{full x-axis plastic section modulus of steel shape}$

Figure I-1a. W-Shapes, Strong-Axis Anchor Points.

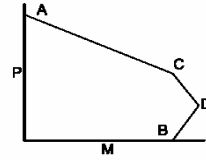
**PLASTIC CAPACITIES FOR RECTANGULAR, ENCASED W-SHAPES BENT ABOUT THE Y-Y AXIS**



Section	Stress Distribution	Point	Defining Equations
<p>(A)</p>		<p>A</p>	$P_A = A_s F_y + A_{sr} F_{yr} + 0.85 f'_c A_c$ $M_A = 0$ <p><math>A_s</math> = area of steel shape  <math>A_{sr}</math> = area of continuous reinforcing bars  <math>A_c = h_1 h_2 - A_s - A_{sr}</math></p>
<p>(E)</p>		<p>E</p>	$P_E = A_s F_y + (0.85 f'_c) \left[ A_c - \frac{h_1}{2} (h_2 - b_f) + \frac{A_{sr}}{2} \right]$ $M_E = M_D - Z_{sE} F_y - \frac{1}{2} Z_{cE} (0.85 f'_c)$ <p><math>Z_{sE} = Z_{sy}</math> = full y-axis plastic section modulus of steel shape  <math>Z_{cE} = \frac{h_1 b_f^2}{4} - Z_{sE}</math></p>
<p>(C)</p>		<p>C</p>	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
<p>(D)</p>		<p>D</p>	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = Z_s F_y + Z_r F_{sr} + \frac{1}{2} Z_c (0.85 f'_c)$ <p><math>Z_s</math> = full y-axis plastic section modulus of steel shape  <math>Z_r = A_{sr} \left( \frac{h_2}{2} - c \right)</math>  <math>Z_c = \frac{h_1 h_2^2}{4} - Z_s - Z_r</math></p>
<p>(C)</p>		<p>C</p>	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
<p>(E)</p>		<p>E</p>	$P_E = A_s F_y + (0.85 f'_c) \left[ A_c - \frac{h_1}{2} (h_2 - b_f) + \frac{A_{sr}}{2} \right]$ $M_E = M_D - Z_{sE} F_y - \frac{1}{2} Z_{cE} (0.85 f'_c)$ <p><math>Z_{sE} = Z_{sy}</math> = full y-axis plastic section modulus of steel shape  <math>Z_{cE} = \frac{h_1 b_f^2}{4} - Z_{sE}</math></p>
<p>(D)</p>		<p>D</p>	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = Z_s F_y + Z_r F_{sr} + \frac{1}{2} Z_c (0.85 f'_c)$ <p><math>Z_s</math> = full y-axis plastic section modulus of steel shape  <math>Z_r = A_{sr} \left( \frac{h_2}{2} - c \right)</math>  <math>Z_c = \frac{h_1 h_2^2}{4} - Z_s - Z_r</math></p>
<p>(B)</p>		<p>B</p>	$P_B = 0$ $M_B = M_D - Z_{sn} F_y - \frac{1}{2} Z_{cn} (0.85 f'_c)$ $Z_{cn} = h_1 h_n^2 - Z_{sn}$ <p>For <math>h_n</math> below the flange <math>\left( h_n \leq \frac{b_f}{2} \right)</math></p> $h_n = \frac{0.85 f'_c (A_c + A_s - 2t_f b_f) - 2F_y (A_s - 2t_f b_f)}{2[4t_f F_y + (h_1 - 2t_f) 0.85 f'_c]}$ $Z_{sn} = Z_s - 2t_f \left( \frac{b_f}{2} + h_n \right) \left( \frac{b_f}{2} - h_n \right)$
<p>(D)</p>		<p>D</p>	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = Z_s F_y + Z_r F_{sr} + \frac{1}{2} Z_c (0.85 f'_c)$ <p><math>Z_s</math> = full y-axis plastic section modulus of steel shape  <math>Z_r = A_{sr} \left( \frac{h_2}{2} - c \right)</math>  <math>Z_c = \frac{h_1 h_2^2}{4} - Z_s - Z_r</math></p>
<p>(B)</p>		<p>B</p>	<p>For <math>h_n</math> above the flange <math>\left( h_n &gt; \frac{b_f}{2} \right)</math></p> $h_n = \frac{0.85 f'_c (A_c + A_s) - 2F_y A_s}{2[0.85 f'_c h_1]}$ <p><math>Z_{sn} = Z_{sy}</math> = full y-axis plastic section modulus of steel shape</p>

Figure I-1b. W-Shapes, Weak-Axis Anchor Points

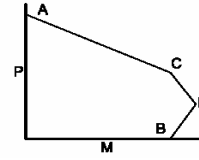
**PLASTIC CAPACITIES FOR COMPOSITE,  
FILLED HSS BENT ABOUT THE X-X OR Y-Y AXIS**



Section	Stress Distribution	Point	Defining Equations
<p>(A)</p>		A	$P_A = A_s F_y + A_c (0.85 f_c')$ $M_A = 0$ $A_s = \text{area of steel shape}$ $A_c = h_1 h_2 - 0.858 r_i^2$
<p>(C)</p>		C	$P_C = A_c (0.85 f_c')$ $M_C = M_B$
<p>(D)</p>		D	$P_D = \frac{0.85 f_c' A_c}{2}$ $M_D = Z_s F_y + \frac{1}{2} Z_c (0.85 f_c')$ $Z_s = \text{full y-axis plastic section modulus of steel shape}$ $Z_c = \frac{h_1 h_2^2}{4} - 0.192 r_i^3$
<p>(B)</p>		B	$P_B = 0$ $M_B = M_D - Z_{sn} F_y - \frac{1}{2} Z_{cn} (0.85 f_c')$ $Z_{sn} = 2 t_w h_n^2$ $Z_{cn} = h_1 h_n^2$ $h_n = \frac{0.85 f_c' A_c}{2 [0.85 f_c' h_1 + 4 t_w F_y]} \leq \frac{h_2}{2}$

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
<p>(A)</p>		A	$P_A = A_s F_y + 0.85 f'_c A_c^*$ $M_A = 0$ $A_s = \pi (d t - t^2)$ $A_c = \frac{\pi d^2}{4}$
<p>(C)</p>		C	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
<p>(D)</p>		D	$P_D = \frac{0.85 f'_c A_c}{2}$ $M_D = Z_s F_y + \frac{1}{2} Z_c (0.85 f'_c)$ $Z_s = \text{plastic section modulus of steel shape} = \frac{d^3}{6} - Z_c$ $Z_c = \frac{h^3}{6}$
<p>(B)</p>		B	$P_B = 0$ $M_B = Z_{sB} F_y - \frac{1}{2} Z_{cB} (0.85 f'_c)$ $Z_{sB} = \frac{d^3 \sin^3(\theta/2)}{6} - Z_{cB}$ $Z_{cB} = \frac{h^3 \sin^3(\theta/2)}{6}$ $\theta = \frac{0.0260 K_c - 2 K_s}{0.0848 K_c} + \sqrt{\frac{(0.0260 K_c + 2 K_s)^2 + 0.857 K_c K_s f'_c A_c}{0.0848 K_c}} \quad (\text{in rads})$ $K_c = f'_c h^2$ $K_s = F_y \left( \frac{d-t}{2} \right) t \quad (\text{"thin" HSS wall assumed})$ $h_n = \frac{h}{2} \sin\left(\frac{\pi-\theta}{2}\right) \leq \frac{h}{2} \quad (\text{not used, for reference only})$

\*  $P_A = A_s F_y + 0.95 f'_c A_c$  is permitted to be used when the composite column is loaded only in axial compression.

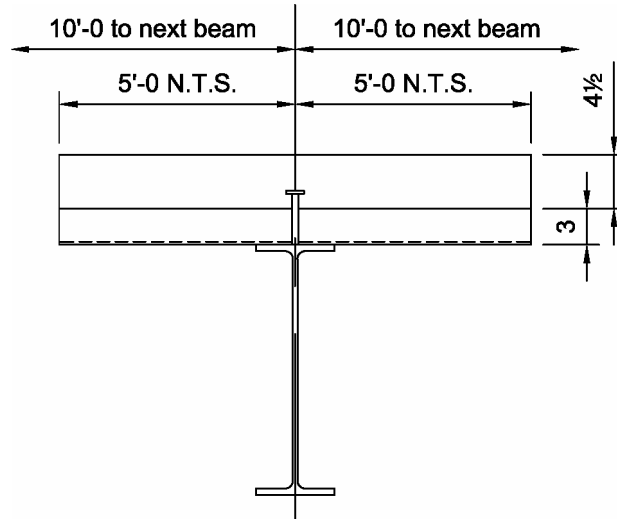
Figure I-1d. Filled Round HSS Anchor Points



## 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  $4\frac{1}{2}$  in. normal weight concrete above the deck, for fire protection and mass. Select an appropriate beam and determine the required number of shear studs.



### Solution:

#### Material Properties:

Concrete  $f'_c = 4$  ksi

Beam  $F_y = 50$  ksi  $F_u = 65$  ksi

Manual  
Table 2-3

#### Loads:

Dead load:

Slab = 0.075 kip/ft<sup>2</sup>  
 Beam weight = 0.008 kip/ft<sup>2</sup> (assumed)  
 Miscellaneous = 0.010 kip/ft<sup>2</sup> (ceiling etc.)

Live load:

Non-reduced = 0.10 kips/ft<sup>2</sup>

Since each beam is spaced at 10 ft. o.c.

Total dead load = 0.093 kip/ft<sup>2</sup>(10 ft.) = 0.93 kips/ft.

Total live load = 0.10 kip/ft<sup>2</sup>(10ft.) = 1.00 kips/ft.

Construction dead load (unshored) = 0.083 kip/ft<sup>2</sup>(10 ft) = 0.83 kips/ft

Construction live load (unshored) = 0.020 kip/ft<sup>2</sup>(10 ft) = 0.20 kips/ft

Determine the required flexural strength

LRFD	ASD
$w_u = 1.2(0.93 \text{ kip/ft}) + 1.6(1.0 \text{ kip/ft})$ $= 2.72 \text{ kip/ft}$	$w_a = 0.93 \text{ kip/ft} + 1.0 \text{ kip/ft}$ $= 1.93 \text{ kip/ft}$
$M_u = \frac{2.72 \text{ kip/ft}(45 \text{ ft})^2}{8} = 687 \text{ kip-ft.}$	$M_a = \frac{1.93 \text{ kip/ft}(45 \text{ ft})^2}{8} = 489 \text{ kip-ft.}$

Use Tables 3-19, 3-20 and 3-21 from the Manual to select an appropriate member

Determine  $b_{eff}$

Section  
I.3.1.1a

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:

(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

$$\frac{10 \text{ ft.}}{2}(2) = 10.0 \text{ ft.} \quad \mathbf{Controls}$$

(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,  $Y_2$ .

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.)

$$Y_2 = t_{slab} - a/2 = 7.5 - 1/2 = 7.0 \text{ in.}$$

Enter Manual Table 3-19 with the required strength and  $Y_2=7.0$  in. Select a beam and neutral axis location that indicates sufficient available strength.

Manual Table  
3-19

Select a W21×50 as a trial beam.

When PNA location 5 (BFL), this composite shape has an available strength of:

LRFD	ASD
$\phi_b M_n = 770 \text{ kip-ft} > 687 \text{ kip-ft} \quad \mathbf{o.k.}$	$M_n/\Omega_b = 512 \text{ kip-ft} > 489 \text{ kip-ft} \quad \mathbf{o.k.}$

Manual  
Table 3-19

Note that 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 the beam deflections and available strength*

Check the deflection of the beam under construction, considering only the weight of concrete as contributing to the construction dead load.

Limit deflection to a maximum of 2.5 in. to facilitate concrete placement.

$$I_{req} = \frac{5}{384} \frac{w_{DL} l^4}{E\Delta} = \frac{5(0.83 \text{ kip/ft})(45 \text{ ft})^4 (1728 \text{ in.}^3/\text{ft}^3)}{384(29,000 \text{ ksi})(2.5 \text{ in.})} = 1,060 \text{ in.}^4$$

From Manual Table 3-20, a W21×50 has  $I_x = 984 \text{ in.}^4$ , therefore this member does not satisfy the deflection criteria under construction.

Using Manual Table 3-20, revise the trial member selection to a W21×55, which has  $I_x = 1140 \text{ in.}^4$ , as noted in parenthesis below the shape designation.

*Check selected member strength as an un-shored beam under construction loads assuming adequate lateral bracing through the deck attachment to the beam flange.*

LRFD	ASD
<i>Calculate the required strength</i>	<i>Calculate the required strength</i>
1.4 DL = 1.4 (0.83 kips/ft) = 1.16 kips/ft	DL+LL = 0.83 + 0.20 = 1.03 kips.ft
1.2DL+1.6LL = 1.2 (0.83) + 1.6(0.20) = 1.32 klf	
$M_u (\text{unshored}) = \frac{1.31 \text{ kip/ft}(45 \text{ ft})^2}{8}$ =331 kip-ft	$M_a (\text{unshored}) = \frac{1.03 \text{ kips/ft}(45 \text{ ft})^2}{8}$ = 260 kip-ft
The design strength for a W21×55 is 473 kip-ft > 331 kip-ft <b>o.k.</b>	The allowable strength for a W21× 55 is 314 kip-ft > 260 kip-ft <b>o.k.</b>

For a W21×55 with  $Y_2=7.0 \text{ in.}$ , the member has sufficient available strength when the PNA is at location 6 and  $\sum Q_n = 292 \text{ kips.}$

Manual  
Table 3-19

LRFD	ASD
$\phi_b M_n = 767 \text{ kip-ft} > 687 \text{ kip-ft}$ <b>o.k.</b>	$M_n / \Omega_b = 510 \text{ kip-ft} > 489 \text{ kip-ft}$ <b>o.k.</b>

Manual  
Table 3-19

*Check a*

$$a = \frac{\sum Q_n}{0.85 f'_c b} = \frac{292 \text{ kips}}{0.85(4 \text{ ksi})(10 \text{ ft.})(12 \text{ in./ft.})} = 0.716 \text{ in.}$$

0.716 in. < 1.0 in. assumed **o.k.**

*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.

For a W21×55 with  $Y_2=7.0$  and the PNA at location 6,  $I_{LB} = 2440 \text{ in.}^4$

Manual Table  
3-20

$$\Delta_{LL} = \frac{5}{384} \frac{w_{LL} l^4}{EI_{LB}} = \frac{5(1.0 \text{ kip/ft})(45 \text{ ft})^4 (1728 \text{ in.}^3/\text{ft}^3)}{384(29,000 \text{ ksi})(2440 \text{ in.}^4)} = 1.30 \text{ in.}$$

1.30 in. < 1.5 in. **o.k.**

*Determine if the beam has sufficient available shear strength*

LRFD	ASD
$V_u = \frac{45\text{ft}}{2} (2.72 \text{ kip/ft}) = 61.2 \text{ kips}$	$V_a = \frac{45\text{ft}}{2} (1.93 \text{ kip/ft}) = 43.4 \text{ kips}$
$\phi V_n = 234 \text{ kips} > 61.2 \text{ kips}$ <b>o.k.</b>	$V_n/\Omega = 156 \text{ kips} > 43.4 \text{ kips}$ <b>o.k.</b>

Manual  
Table 3-3

*Determine the required number of shear stud connectors*

Using perpendicular deck with one  $\frac{3}{4}$ -in. diameter weak stud per rib in normal weight 4 ksi concrete.  $Q_n = 17.2 \text{ kips/stud}$

Manual Table  
3-21

$$\frac{\sum Q_n}{Q_n} = \frac{292 \text{ kips}}{17.2 \text{ kips}} = 17, \text{ on each side of the beam.}$$

Section 3.2d(5)

Total number of shear connectors; use  $2(17) = 34$  shear connectors.

Section 3.2d(6)

*Check the spacing of shear connectors*

Since each flute is 12 in., use one stud every flute, starting at each support, and proceed for 17 studs on each end of the span.

$6d_{stud} < 12 \text{ in.} < 8t_{slab}$ , therefore, the shear stud spacing requirements are met.

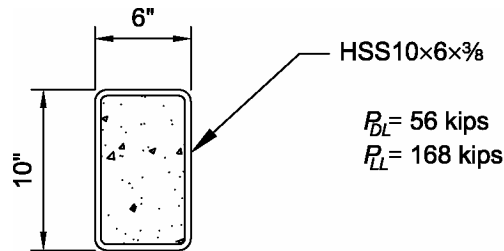
Section  
I3.2c (b)

The studs are to be 5 in. long, so that they will extend a minimum of  $1\frac{1}{2}$  in. into slab.

## Example I-2 Filled Composite Column in Axial Compression

### Given:

Determine if a 14-ft long HSS10×6× $\frac{3}{8}$  ASTM A500 grade 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 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
$P_u = 1.2(56 \text{ kips}) + 1.6(168 \text{ kips})$ $= 336 \text{ kips}$	$P_a = 56 \text{ kips} + 168 \text{ kips}$ $= 224 \text{ kips}$

The available strength in axial compression can be determined directly from the Manual at  $KL = 14$  ft as:

LRFD	ASD
$\phi_c P_n = 353 \text{ kips}$ $353 \text{ kips} > 336 \text{ kips}$ <b>o.k.</b>	$P_n / \Omega_c = 236 \text{ kips}$ $236 \text{ kips} > 224 \text{ kips}$ <b>o.k.</b>

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:

HSS10×6× $\frac{3}{8}$        $F_y = 46 \text{ ksi}$

$F_u = 58 \text{ ksi}$

Manual  
Table 2-3

Concrete

$f'_c = 5 \text{ ksi}$

$E_c = w^{1.5} \sqrt{f'_c} = (145)^{1.5} \sqrt{5} = 3,900 \text{ ksi}$

**Geometric Properties:**

HSS10×6×3/8       $t = 0.375$  in.     $b = 10.0$  in.     $h = 6.0$  in.

Manual Table  
1-11

Concrete:

The concrete area is calculated as follows

$$r = 2t = 2(0.375 \text{ in.}) = 0.75 \text{ in. (outside radius)}$$

$$b_f = b - 2r = 10 - 2(0.75) = 8.50 \text{ in.}$$

$$h_f = h - 2r = 6 - 2(0.75) = 4.50 \text{ in.}$$

$$\begin{aligned} A_c &= b_f h_f + \pi(r-t)^2 + 2b_f(r-t) + 2h_f(r-t) \\ &= (8.50 \text{ in.})(4.50 \text{ in.}) + \pi(0.375 \text{ in.})^2 + 2(8.50 \text{ in.})(0.375 \text{ in.}) + 2(4.50 \text{ in.})(0.375 \text{ in.}) \\ &= 48.4 \text{ in.}^2 \end{aligned}$$

$$I_c = \frac{b_1 h_1^3}{12} + \frac{2(b_2)(h_2)^3}{12} + 2(r-t) \left( \frac{\pi}{8} - \frac{8}{9\pi} \right) + 2 \left( \frac{\pi(r-t)^2}{2} \right) \left( \frac{h_2}{2} + \frac{4(r-t)}{3\pi} \right)^2$$

For this shape, buckling will take place about the weak axis, thus

$$h_1 = 6 - 2(0.375) = 5.25 \text{ in.}$$

$$b_1 = 10 - 4(0.375) = 8.5 \text{ in.}$$

$$h_2 = 6 - 4(0.375) = 4.5 \text{ in.}$$

$$b_2 = 0.375 \text{ in.}$$

$$(r-t) = 0.75 - 0.375 = 0.375 \text{ in.}$$

$$\begin{aligned} I_c &= \frac{(8.5)(5.25)^3}{12} + \frac{2(0.375)(4.5)^3}{12} + 2(0.375)^4 \left( \frac{\pi}{8} - \frac{8}{9\pi} \right) \\ &\quad + 2 \left( \frac{\pi(0.375)^2}{2} \right) \left( \frac{4.5}{2} + \frac{4(0.375)}{3\pi} \right)^2 \\ &= 111 \text{ in.}^4 \end{aligned}$$

Manual Table  
1-11

HSS10×6×3/8:

$$A_s = 10.4 \text{ in.}^2 \quad I_s = 61.8 \text{ in.}^4 \quad h/t = 25.7$$

Limitations:

Section I1.2

1) Normal weight concrete  $10 \text{ ksi} \geq f'_c \geq 3 \text{ ksi}$        $f'_c = 5 \text{ ksi}$       **o.k.**

2) Not Applicable.

3) The cross-sectional area of the steel HSS shall comprise at least one percent of the total composite cross section.

Section I2.2a

$$10.4 \text{ in.}^2 > (0.01)(48.6 \text{ in.}^2 + 10.4 \text{ in.}^2) = 0.590 \text{ in.}^2 \quad \mathbf{o.k.}$$

4) The maximum  $b/t$  ratio for a rectangular HSS used as a composite column shall be equal to  $2.26 \sqrt{E/F_y}$ .

$$b/t = 25.7 \leq 2.26 \sqrt{E/F_y} = 2.26 \sqrt{29,000 \text{ ksi}/46 \text{ ksi}} = 56.7 \quad \mathbf{o.k.}$$

User note: For all rectangular HSS sections found in the Manual the  $b/t$  ratios do not exceed  $2.26 \sqrt{E/F_y}$ .

5) Not Applicable.

Calculate the available compressive strength

$$C_2 = 0.85 \text{ for rectangular sections}$$

Sec. I2.2b

$$P_o = A_s F_y + A_{sr} F_{yr} + C_2 A_c f'_c$$

$$= (10.4 \text{ in.}^2)(46 \text{ ksi}) + 0.85(48.4 \text{ in.}^2)(5 \text{ ksi}) = 684 \text{ kips}$$

Eqn. I2-13

$$C_3 = 0.6 + 2 \left( \frac{A_s}{A_c + A_s} \right) = 0.6 + 2 \left( \frac{10.4 \text{ in.}^2}{48.4 \text{ in.}^2 + 10.4 \text{ in.}^2} \right) = 0.954 \geq 0.90$$

Eqn. I2-15

Therefore use 0.90

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c$$

$$= (29,000 \text{ ksi})(61.8 \text{ in.}^4) + (0.90)(3,900 \text{ ksi})(111 \text{ in.}^4)$$

$$= 2,180,000 \text{ kip-in.}^2$$

Eqn. I2-14

User note:  $K$  value is from Chapter C and for this case  $K = 1.0$ .

$$P_e = \pi^2 (EI_{eff}) / (KL)^2 = \pi^2 (2,180,000 \text{ kip-in.}^2) / ((1.0)(14\text{ft})(12\text{in./ft})^2) = 762 \text{ kips}$$

Eqn. I2-4

$$\frac{P_o}{P_e} = \frac{684 \text{ kips}}{762 \text{ kips}} = 0.898$$

$0.898 \leq 2.25$  Therefore use Eqn. I2-2 to solve  $P_n$

$$P_n = P_o \left[ 0.658 \left( \frac{P_o}{P_e} \right) \right] = (684 \text{ kips}) \left[ 0.658^{(0.898)} \right] = 470 \text{ kips}$$

Section  
I2.1b

Eqn. I2-6

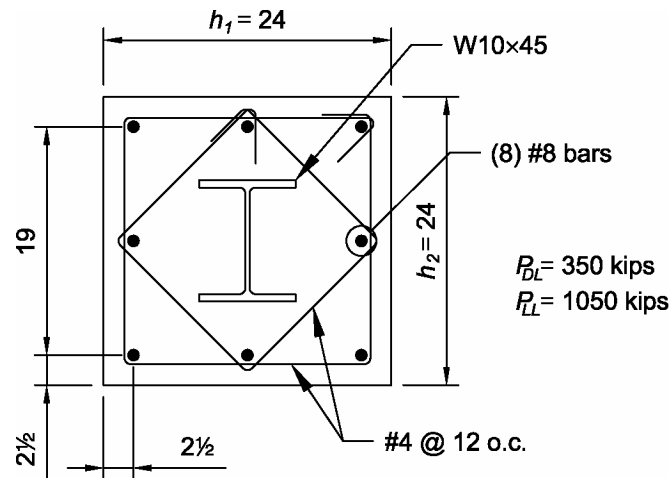
LRFD	ASD
$\phi_c = 0.75$	$\Omega_c = 2.00$
$\phi_c P_n = 0.75(470 \text{ kips}) = 353 \text{ kips}$	$P_n / \Omega_c = 470 \text{ kips} / 2.00 = 235 \text{ kips}$
$353 \text{ kips} > 336 \text{ kips}$ <b>o.k.</b>	$235 \text{ kips} > 224 \text{ kips}$ <b>o.k.</b>

Section I2.1b

### Example I-3 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 @ 12in. o/c., The column is pinned at both ends and the load is applied directly to the concrete encasement.



#### Solution:

##### Material Properties:

Column W10×45	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual Table 2-3
Concrete	$f'_c = 5$ ksi	$E_c = 3,900$ ksi (145 pcf concrete)	
Reinforcement	$F_{yst} = 60$ ksi		

##### Geometric Properties:

W10×45:

$$A_s = 13.3 \text{ in.}^2$$

$$I_y = 53.4 \text{ in.}^4$$

Reinforcing steel:

$$A_{sr} = 6.32 \text{ in.}^2 \quad (\text{the area of 1-#8 bar is } 0.79 \text{ in.}^2, \text{ per ACI})$$

$$I_{sr} = \frac{\pi r^4}{4} + Ad^2 = 8 \frac{\pi(0.50)^4}{4} + 6(0.79)(9.5)^2 = 428 \text{ in.}^4$$

Concrete:

$$A_c = A_{cg} - A_s - A_{sr} = 576 \text{ in.}^2 - 13.3 \text{ in.}^2 - 6.32 \text{ in.}^2 = 556 \text{ in.}^2$$

$$I_c = I_{cg} - I_s - I_{sr} = 27,200 \text{ in.}^4$$

Note: The weak axis moment of inertia is used as part of a slenderness check.



Limitations:

- 1) Normal weight concrete  $10 \text{ ksi} \geq f'_c \geq 3 \text{ ksi}$      $f'_c = 5 \text{ ksi}$     **o.k.**    Section I1.2
- 2)  $F_{yst} \leq 75 \text{ ksi}$      $F_{yst} = 60 \text{ ksi}$     **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

$$13.3 \text{ in.}^2 > (0.01)(576 \text{ in.}^2) = 5.76 \text{ in.}^2 \quad \mathbf{o.k.}$$

- 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  $0.009 \text{ in.}^2$  of tie spacing.

$$0.20 \text{ in.}^2/12 \text{ in.} = 0.0167 \text{ in.}^2/\text{in.} > 0.009 \text{ in.}^2/\text{in.} \quad \mathbf{o.k.}$$

- 5) The minimum reinforcement ratio for continuous longitudinal reinforcing,  $\rho_{sr}$ , shall be 0.004.

$$\rho_{sr} = \frac{A_{sr}}{A_g} = \frac{6.32 \text{ in.}^2}{576 \text{ in.}^2} = 0.011 > 0.004 \quad \mathbf{o.k.} \quad \text{Eqn. I2-1}$$

Calculate the total required compressive strength

LRFD	ASD
$P_u = 1.2(350 \text{ kips}) + 1.6(1050 \text{ kips})$ $= 2100 \text{ kips}$	$P_a = 350 \text{ kips} + 1050 \text{ kips}$ $= 1400 \text{ kips}$

Calculate the available compressive strength

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c$$

$$= (13.3 \text{ in.}^2)(50 \text{ ksi}) + (6.32 \text{ in.}^2)(60 \text{ ksi}) + 0.85(556 \text{ in.}^2)(5 \text{ ksi}) = 3410 \text{ kips} \quad \text{Eqn. I2-2}$$

$$C_I = 0.1 + 2 \left( \frac{A_s}{A_c + A_s} \right) = 0.1 + 2 \left( \frac{13.3 \text{ in.}^2}{556 \text{ in.}^2 + 13.3 \text{ in.}^2} \right) = 0.15 \quad \text{Eqn. I2-5}$$

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_I E_c I_c$$

$$= (29,000 \text{ ksi})(53.4 \text{ in.}^4) + 0.5(29,000 \text{ ksi})(428 \text{ in.}^4) + (0.15)(3,900 \text{ in.}^4)(27,200)$$

$$= 23,700,000 \text{ kip-in.}^2 \quad \text{Eqn. I2-4}$$

User note:  $K$  value is from Chapter C and for this case  $K = 1.0$ .

$$P_e = \pi^2 (EI_{eff}) / (KL)^2 = \pi^2 (23,700,000 \text{ kip-in.}^2) / ((1.0)(14 \text{ in.})(12 \text{ in./ft})^2) = 8,290 \text{ kips} \quad \text{Eqn. I2-3}$$

$$\frac{P_o}{P_e} = \frac{3,410 \text{ kips}}{8,290 \text{ kips}} = 0.411$$

$0.411 \leq 2.25$  Therefore use Eqn. I2-2 to solve  $P_n$     Section I2.1b

$$P_n = P_o \left[ 0.658 \left( \frac{P_o}{P_c} \right) \right] = (3,410 \text{ kips}) \left[ 0.658^{(0.411)} \right] = 2870 \text{ kips}$$

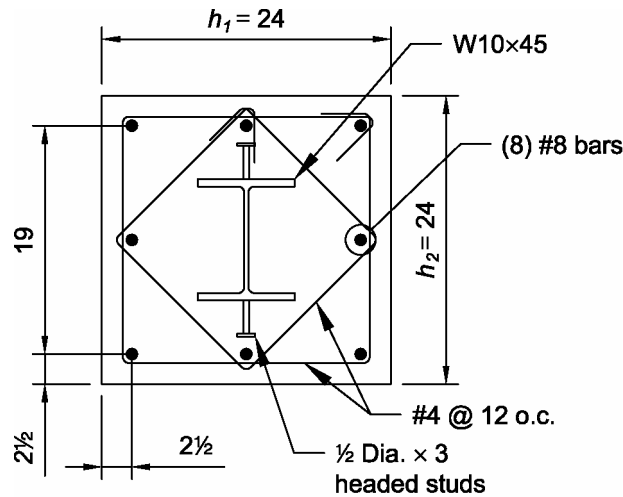
Eqn. I2-6

LRFD	ASD
$\phi_c = 0.75$	$\Omega_c = 2.00$
$\phi_c P_n = 0.75(2870 \text{ kips}) = 2150 \text{ kips}$	$P_n / \Omega_c = 2870 \text{ kips} / 2.00 = 1440 \text{ kips}$
2150 kips > 2100 kips <b>o.k.</b>	1440 kips > 1400 kips <b>o.k.</b>

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.

Determine the number and spacing of 1/2-in. diameter headed shear studs to transfer the axial force.



**Solution:**

**Material Properties:**

Conc.                       $f'_c = 5 \text{ ksi}$                        $E_c = 4070 \text{ ksi}$

Shear Studs               $F_u = 65 \text{ ksi}$

Manual Table 3-21

**Geometric Properties:**

W10x45               $A_{st} = 13.3 \text{ in.}^2$      $A_{flange} = 4.97 \text{ in.}^2$      $A_{web} = 3.36 \text{ in.}^2$                $d_{st} = 10.1 \text{ in.}$

Shear Studs               $A_{sc} = 0.196 \text{ in.}^2$

Conc.                       $d_c = 24 \text{ in.}$

Manual Table 1-1

Calculate the shear force to be transferred

LRFD	ASD
$V = \frac{P_u}{\phi_c} = \frac{2,100}{0.75} = 2,800 \text{ kips}$	$V = P_a \Omega_c = 1,400(2) = 2,800 \text{ kips}$

$$V' = V(A_s F_y / P_o) = 2800 \text{ kips} ((13.3 \text{ in.}^2)(50 \text{ ksi}) / (3410 \text{ kips})) = 546 \text{ kips}$$

Eqn. I2-10

Calculate the nominal strength of one 1/2 in. diameter shear stud connector

$$Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq A_{sc}F_u$$

$$0.5A_{sc}\sqrt{f'_c E_c} = 0.5(0.196 \text{ in.}^2)\sqrt{(5\text{ksi})(3900\text{ksi})} = 13.7 \text{ kips}$$

$$A_{sc}F_u = (0.196 \text{ in.}^2)(65 \text{ ksi}) = 12.7 \text{ kips}$$

Eqn. I2-12

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

Section  
I2.1f

The maximum stud spacing is 16 in.

The available column length is 14ft (12 in./ft) = 168 in. and the maximum spacing is = 168 in./((22+1)) = 7.3 in.

Therefore, on the flanges, use single studs @ 7 in.

Stud placement is to start 10.5 in. from one end.

Determine the length of the studs for the flanges;

$$\left(\frac{d_c - d_{st}}{2}\right) - 3 \text{ in.} = \left(\frac{24 \text{ in.} - 10.1 \text{ in.}}{2}\right) - 3 \text{ in.} = 3.95 \text{ in.}$$

Therefore use 3 ½ in. in length for the flanges.

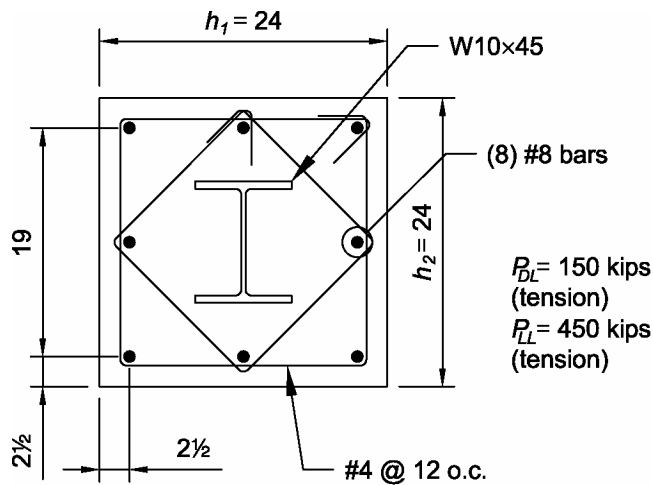
Note: The subtraction of 3 in. is to ensure sufficient cover.

Summary: use ½-in. diameter 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 attached at each end in order to transfer any tensile force.



#### Solution:

##### Material Properties:

Column W10x45	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual
Concrete	$f'_c = 5$ ksi	$E_c = 3,900$ ksi	Table 2-3
Reinforcement	$F_{yst} = 60$ ksi		ACI-318

##### Geometric Properties:

W10x45:  
 $A_s = 13.3$  in.<sup>2</sup>      $I_y = 53.4$  in.<sup>4</sup>

Reinforcing steel:  
 $A_{sr} = 6.32$  in.<sup>2</sup>      $I_{sr} = 428$  in.<sup>4</sup>

Concrete:  
 $A_c = 556$  in.<sup>2</sup>      $I_c = 27,200$  in.<sup>4</sup>

Manual  
Table 1-1

Limitations:

1) Normal weight concrete  $10 \text{ ksi} \geq f'_c \geq 3 \text{ ksi}$ ;  $f'_c = 5 \text{ ksi}$  **o.k.** Section I1.2

2)  $F_{yst} \leq 75 \text{ ksi}$ ;  $F_{yst} = 60 \text{ ksi}$  **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

$$13.3 \text{ in.}^2 > (0.01)(576 \text{ in.}^2) = 5.76 \text{ in.}^2 \text{ o.k.}$$

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  $0.009 \text{ in.}^2$  of tie spacing.

$$0.20 \text{ in.}^2 / 12 \text{ in.} = 0.0167 \text{ in.}^2 / \text{in.} > 0.009 \text{ in.}^2 / \text{in.} \text{ o.k.}$$

5) The minimum reinforcement ratio for continuous longitudinal reinforcing,  $\rho_{sr}$ , shall be 0.004.

$$\rho_{sr} = \frac{A_{sr}}{A_g} = \frac{6.32 \text{ in.}^2}{576 \text{ in.}^2} = 0.011 > 0.004 \text{ o.k.}$$

Eqn. I2-1

Calculate the required tensile strength

LRFD	ASD
$P_u = 0.9(-150 \text{ kips}) + 1.6(645 \text{ kips})$ $= 897 \text{ kips}$	$P_a = 0.6(-150 \text{ kips}) + 645 \text{ kips}$ $= 555 \text{ kips}$

Calculate the available tensile strength

$$P_n = P_t = A_s F_y + A_{sr} F_{yr}$$

$$= (13.3 \text{ in.}^2)(50 \text{ ksi}) + (6.32 \text{ in.}^2)(60 \text{ ksi}) = 1040 \text{ kips}$$

Eqn. I2-8

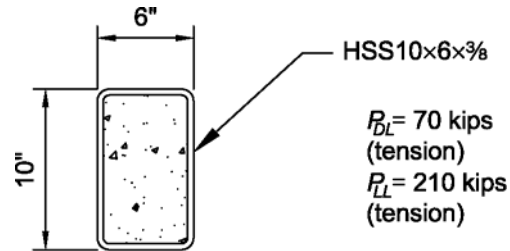
LRFD	ASD
$\phi_t = 0.90$	$\Omega_t = 1.67$
$\phi_t P_n = 0.90(1040 \text{ kips}) = 940 \text{ kips}$	$P_n / \Omega_t = 1040 \text{ kips} / 1.67 = 625 \text{ kips}$
$940 \text{ kips} > 897 \text{ kips}$ <b>o.k.</b>	$625 \text{ kips} > 555 \text{ kips}$ <b>o.k.</b>

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× $\frac{3}{8}$        $F_y = 46$  ksi

$F_u = 58$  ksi

Manual  
Table 2-3

Concrete       $f'_c = 5$  ksi

$E_c = 4,070$  ksi

##### Geometric Properties:

HSS10×6× $\frac{3}{8}$ :

$A = 10.4$  in.<sup>2</sup>

Manual  
Table 1-11

Calculate the required tensile strength

LRFD	ASD
$P_u = 0.9(-70 \text{ kips}) + 1.6(300 \text{ kips})$ $= 417$ kips	$P_a = 0.6(-70 \text{ kips}) + 300$ kips $= 258$ kips

Determine the available tensile strength

$$P_n = P_o = A_s F_y$$

$$= (10.4 \text{ in.}^2)(46 \text{ ksi}) = 478 \text{ kips}$$

Eqn. I2-16

LRFD	ASD
$\phi_t = 0.90$	$\Omega_t = 1.67$
$\phi_t P_n = 0.90(478 \text{ kips}) = 430$ kips	$P_n / \Omega_t = 478 \text{ kips} / 1.67 = 286$ kips
430 kips > 417 kips <b>o.k.</b>	286 kips > 258 kips <b>o.k.</b>

Section  
I2.1b

User 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.

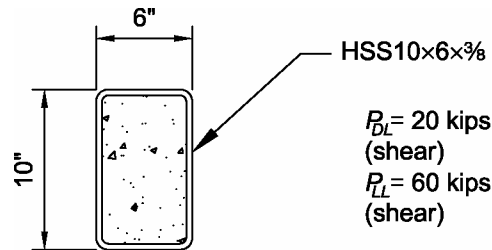
## Example I-6 Filled Composite Member Design for Shear.

### Given:

This composite member ASTM A500 grade B with 5ksi 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 by both LRFD and ASD analysis.

User note: Shear strength shall be 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:

ASTM A500 Gr. B  $F_y = 46 \text{ ksi}$   $F_u = 58 \text{ ksi}$

Manual  
Table 2.3

#### Geometric Properties:

HSS 10x6x3/8  $d = 10 \text{ in.}$   $t_w = 0.349 \text{ in.}$

Manual Table  
1-11

Calculate the required shear strength

LRFD	ASD
$V_u = 1.2(20 \text{ kips}) + 1.6(60 \text{ kips})$ $= 120 \text{ kips}$	$V_a = 20 \text{ kips} + 60 \text{ kips}$ $= 80 \text{ kips}$

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 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

User note:  $k_V = 5.0$  for all sizes of HSS rectangular or box members listed in the Manual. Also all HSS listed in the manual have a  $C_V = 1.0$ .

Section G5

Calculate  $h$

$$h = d - (3t_w) = 10 \text{ in.} - (3)(0.349 \text{ in.}) = 8.95 \text{ in.}$$

Section G5

Calculate  $A_w$

$$A_w = 2ht_w = 2(8.95 \text{ in.})(0.349 \text{ in.}) = 6.25 \text{ in}^2$$

Section G5

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
$\phi_v = 0.90$	$\Omega_v = 1.67$
$V_u = \phi_v V_n = 0.90(173 \text{ kips}) = 156 \text{ kips}$	$V_a = V_n / \Omega_v = 173 \text{ kips} / 1.67 = 104 \text{ kips}$
156 kips $\geq$ 120 kips <b>o.k.</b>	104 kips $\geq$ 80 kips <b>o.k.</b>

Section G1

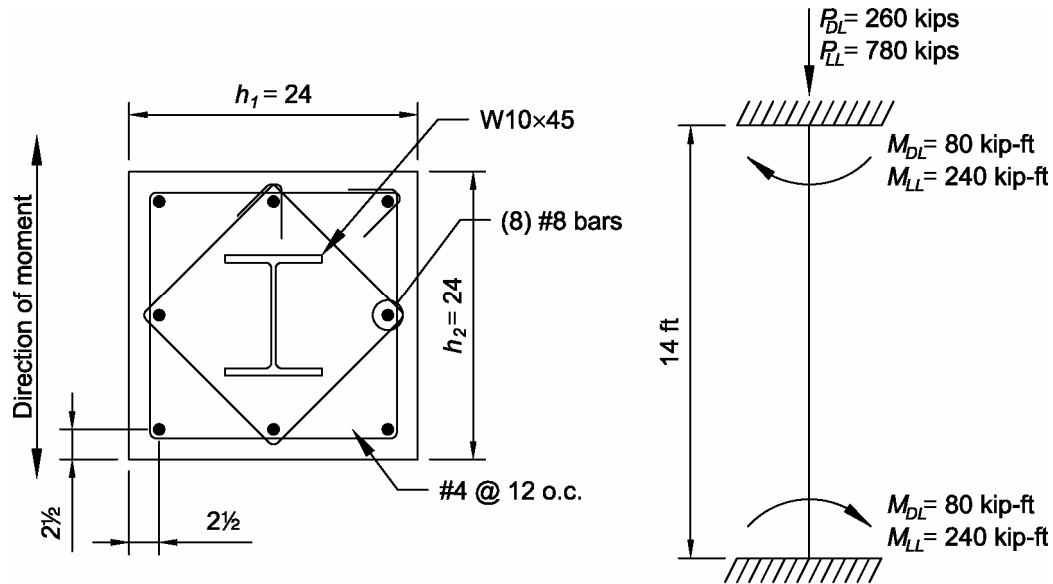
User note: For illustration of  $C_v = 1.0$  refer to Chapter G, Example G.4.



## 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.×24 in. 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 k-ft and a live load moment of 240 k-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

Concrete  
 $f'_c = 5$  ksi  $E_c = 3,900$  ksi

Reinforcement  
 $F_{yst} = 60$  ksi

#### Geometric Properties:

W10×45:  
 $A_s = 13.3$  in.<sup>2</sup>  $I_y = 53.4$  in.<sup>4</sup>  $Z = 54.9$  in.<sup>3</sup>  $d = 10.1$  in.  $b_f = 8.02$  in.  $t_w = 0.35$  in.  
 $t_f = 0.62$  in.

#### Reinforcing steel:

$A_{sr} = 6.32$  in.<sup>2</sup>  $A_{srs} = 1.59$  in.<sup>2</sup>  $I_{sr} = 428$  in.<sup>4</sup>

#### Concrete:

$A_c = 556$  in.<sup>2</sup>  $I_c = 27,200$  in.<sup>4</sup>

Manual  
 Table 2-3  
 ACI-318

Manual  
 Table 1-1

Limitations:

1) Normal weight concrete  $10 \text{ ksi} \geq f'_c \geq 3 \text{ ksi}$       $f'_c = 5 \text{ ksi}$      **OK**     Section I1.2

2)  $F_{yst} \leq 75 \text{ ksi}$       $F_{yst} = 60 \text{ ksi}$      **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

$13.3 \text{ in.}^2 > (0.01)(576 \text{ in.}^2) = 5.76 \text{ in.}^2$      **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  $0.009 \text{ in.}^2$  per inch of tie spacing.

$0.20 \text{ in.}^2/12 \text{ in.} = 0.0167 \text{ in.}^2/\text{in.} > 0.009 \text{ in.}^2/\text{in.}$      **o.k.**

5) The minimum reinforcement ratio for continuous longitudinal reinforcing,  $\rho_{sr}$ , shall be 0.004.

$\rho_{sr} = \frac{A_{sr}}{A_g} = \frac{6.32 \text{ in.}^2}{576 \text{ in.}^2} = 0.011 > 0.004$      **o.k.**

Eqn. I2-1

Calculate the required strengths

LRFD	ASD
$P_u = 1.2(260 \text{ kips}) + 1.6(780 \text{ kips})$ = 1560 kips	$P_a = 260 \text{ kips} + 780 \text{ kips}$ = 1040 kips
$M_u = 1.2(80 \text{ k-ft}) + 1.6(240 \text{ k-ft})$ = 480 k-ft	$M_a = 80 \text{ k-ft} + 240 \text{ k-ft}$ = 320 k-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 C4-3, and repeated here.

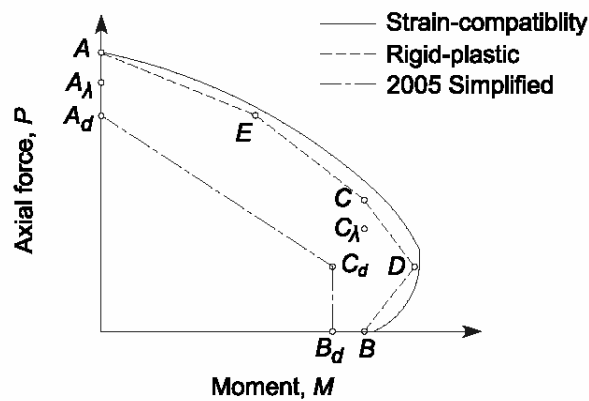


Fig. C-14.1 Interaction diagram for composite beam-column design

**Point A ( $M = 0$ )**

Determine the available compressive strength and moment strength

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c$$

$$= (13.3 \text{ in.}^2)(50 \text{ ksi}) + (6.32 \text{ in.}^2)(60 \text{ ksi}) + 0.85(556 \text{ in.}^2)(5 \text{ ksi}) = 3,410 \text{ kips}$$
Eqn. I2-4

$$C_l = 0.1 + 2 \left( \frac{A_s}{A_c + A_s} \right) = 0.1 + 2 \left( \frac{13.3 \text{ in.}^2}{556 \text{ in.}^2 + 13.3 \text{ in.}^2} \right) = 0.15$$
Eqn. I2-7

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_l E_c I_c$$

$$= (29,000 \text{ ksi})(53.4 \text{ in.}^4) + 0.5(29,000 \text{ ksi})(428 \text{ in.}^4) + (0.15)(3,900 \text{ in.}^4)(27,200)$$

$$= 23,700,000 \text{ kip-in.}^2$$
Eqn. I2-6

User note:  $K$  value is from Chapter C and for this case  $K = 1.0$ .

$$P_e = \pi^2 (EI_{eff}) / (KL)^2 = \pi^2 (23,700,000 \text{ kip-in.}^2) / ((1.0)(14 \text{ ft})(12 \text{ in./ft}))^2 = 8,290 \text{ kips}$$
Eqn. I2-5

$$\frac{P_o}{P_e} = \frac{3,410 \text{ kips}}{8,290 \text{ kips}} = 0.411$$

$0.411 \leq 2.25$  Therefore use Eqn. I2-2 to determine  $P_n$

$$P_{nA} = P_o \left[ 0.658 \left( \frac{P_o}{P_e} \right) \right] = (3410 \text{ kips}) \left[ 0.658^{(0.411)} \right] = 2870 \text{ kips}$$
Eqn. I2-2

$$\text{Point D} \left( P_D = \frac{A_c (0.85 f'_c)}{2} \right)$$

$$P_D = \frac{A_c (0.85 f'_c)}{2}$$

$$P_D = \frac{(556 \text{ in}^2)(0.85)(5.0 \text{ ksi})}{2}$$

$$P_D = 1180 \text{ kips}$$

$Z_s$  = full x-axis plastic modulus of steel shape

$$Z_s = 54.9 \text{ in}^3$$

$$Z_r = (A_{sr} - A_{srs}) \left( \frac{h_2}{2} - c \right) = (6.32 \text{ in}^2 - 1.59 \text{ in}^2) \left( \frac{24 \text{ in.}}{2} - 2 \frac{1}{2} \text{ in.} \right) = 44.9 \text{ in}^3$$

$$Z_c = \frac{h_1 h_2^2}{4} - Z_s - Z_r = \frac{(24 \text{ in})(24 \text{ in})^2}{4} - 54.9 \text{ in}^3 - 44.9 \text{ in}^3 = 3360 \text{ in}^3$$

$$M_D = Z_s F_y + Z_r F_{yr} + \frac{1}{2} Z_c (0.85 f'_c)$$

$$M_D = (54.9 \text{ in}^3)(50 \text{ ksi}) + (44.9 \text{ in}^3)(60.0 \text{ ksi}) + \frac{1}{2}(3360 \text{ in}^3)(0.85)(5.0 \text{ ksi})$$

$$= 12,600 \text{ k-in}$$

$$M_{nD} = \frac{12,600 \text{ k-in}}{12 \frac{\text{in}}{\text{ft}}} = 1050 \text{ k-ft}$$

$$P_{nD} = P_o \left[ 0.658 \left( \frac{P_o}{P_e} \right) \right] = (1,180 \text{ kips}) \left[ 0.658^{(0.411)} \right] = 994 \text{ kips}$$

Commentary  
Figure C-14.1

Eqn. I2-2  
and  
Commentary  
I4

**Point B ( $P_B = 0$ )**

For  $h_n$  in the flange  $\left(\left(\frac{d}{2} - t_f\right) < h_n \leq \frac{d}{2}\right)$

$$h_n = \frac{0.85 f_c' (A_c + A_s - db_f + A_{srs}) - 2F_y (A_s - db_f) - 2F_{yr} A_{srs}}{2 \left[ 0.85 f_c' (h_1 - b_f) + 2F_y b_f \right]}$$

$$h_n = \frac{0.85 (5.0 \text{ ksi}) (556 \text{ in.}^2 + 13.3 \text{ in.}^2 - 10.1(8.02) + 1.59) - 2(50 \text{ ksi}) (13.3 \text{ in.}^2 - 10.1(8.02)) - 2(60)(1.59)}{2 \left[ (0.85)(5.0 \text{ ksi}) (24 \text{ in.} - 8.02) + 2(50)(8.02) \right]}$$

$$h_n = 4.98 \text{ in.}$$

$$Z_{sn} = Z_s - b_f \left( \frac{d}{2} - h_n \right) \left( \frac{d}{2} + h_n \right)$$

$$Z_{sn} = 54.9 \text{ in.}^3 - 8.02 \left( \frac{10.1}{2} - 4.98 \right) \left( \frac{10.1}{2} + 4.98 \right) = 49.3 \text{ in.}^3$$

$$Z_{cn} = h_1 h_n^2 - Z_{sn} = (24 \text{ in.})(4.98 \text{ in.})^2 - 49.3 \text{ in.}^3$$

$$Z_{cn} = 546 \text{ in.}^3$$

$$M_B = M_D - Z_{sn} F_y - \frac{1}{2} Z_{cn} (0.85 f_c')$$

$$M_B = (12,600 \text{ k-in.}) - (49.3 \text{ in.}^3)(50.0 \text{ ksi})$$

$$- \frac{1}{2} (546 \text{ in.}^3)(0.85)(5.0 \text{ ksi}) = 8,970 \text{ k-in.}$$

$$M_{nB} = \frac{8,970 \text{ k-in.}}{12 \frac{\text{in.}}{\text{ft}}} = 748 \text{ k-ft}$$

**Point C ( $M_C = M_B$ ;  $P_C = 0.85 f_c' A_c$ )**

$$P_C = A_c (0.85 f_c')$$

$$P_C = (556 \text{ in.}^2)(0.85)(5.0 \text{ ksi}) = 2360 \text{ kips}$$

$$M_{nC} = M_{nB} = 748 \text{ k-ft}$$

$$P_{nC} = P_C \left[ 0.658 \left( \frac{P_o}{P_e} \right) \right] = 2360 \left[ 0.658^{0.411} \right] = 1990 \text{ kips}$$

Commentary  
Figure C-I4.1

Commentary  
Figure C-I4.1

Eqn. I2-2 and  
Commentary  
Section I4

### Summary of nominal strength including length effects

$$P_{nA} = 2870 \text{ kips}$$

$$M_{nA} = 0$$

$$P_{nB} = 0$$

$$M_{nB} = 748 \text{ k-ft}$$

$$P_{nC} = 1990 \text{ kips}$$

$$M_{nC} = 748 \text{ k-ft}$$

$$P_{nD} = 994 \text{ kips}$$

$$M_{nD} = 1050 \text{ k-ft}$$

LRFD	ASD
$\phi_c = 0.75 \quad \phi_b = 0.90$  $\phi_c P_{nA} = 2150 \text{ kips}$ $\phi_b M_{nA} = 0$ $\phi_c P_{nB} = 0$ $\phi_b M_{nB} = 673 \text{ k-ft}$ $\phi_c P_{nC} = 1490 \text{ kips}$ $\phi_b M_{nC} = 673 \text{ k-ft}$ $\phi_c P_{nD} = 746 \text{ kips}$ $\phi_b M_{nD} = 945 \text{ k-ft}$  <b>Interaction</b>  $P_u > P_c$ $1560 \text{ kips} > 1490 \text{ kips}$  thus  $\frac{P_u - P_c}{P_A - P_c} + \frac{M_{uy}}{M_{Cy}} \leq 1.0$  $\frac{1560 - 1490}{2150 - 1490} + \frac{480}{673} = 0.82 > 1.0$  Therefore, this column is adequate for the specified loading	$\Omega_c = 2.00 \quad \Omega_b = 1.67$  $P_{nA} / \Omega_c = 1440 \text{ kips}$ $M_{nA} / \Omega_b = 0$ $P_{nB} / \Omega_c = 0$ $M_{nB} / \Omega_b = 448 \text{ k-ft}$ $P_{nC} / \Omega_c = 995 \text{ kips}$ $M_{nC} / \Omega_b = 448 \text{ k-ft}$ $P_{nD} / \Omega_c = 497 \text{ kips}$ $M_{nD} / \Omega_b = 629 \text{ k-ft}$  <b>Interaction</b>  $P_u > P_c$ $1040 \text{ kips} > 995 \text{ kips}$  thus  $\frac{P_u - P_c}{P_A - P_c} + \frac{M_{uy}}{M_{Cy}} \leq 1.0$  $\frac{1040 - 995}{1440 - 995} + \frac{320}{448} = 0.82 > 1.0$  Therefore, this column is adequate for the specified loading

Section I4

Commentary  
Section I4

The shear connector requirements for this encased composite column are similar to that of Example I-2

## **CHAPTER J DESIGN OF CONNECTIONS**

### **INTRODUCTION**

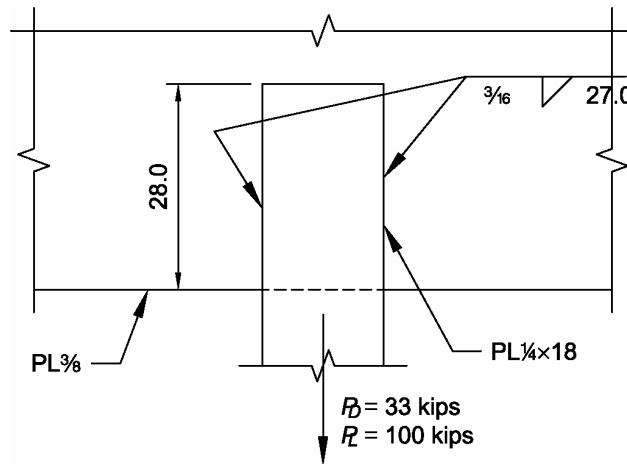
Chapter J of the Specification addresses the design and checking of connections. The chapter's primary focus is the design of welded and bolted connections. Design requirements for fillers, splices, column bases, concentrated forces, anchors rods, and other threaded parts are also covered. Special requirements for connections subject to fatigue are not covered in this chapter.

### Example J.1 Fillet Weld in Longitudinal Shear

**Given:**

An ¼ in.×18-in. wide plate is fillet welded to a ¾-in. plate. Assume that the plates are ASTM A572 grade 50 and have been properly sized. Assume  $F_{EXX} = 70$  ksi. Note that plates would normally be specified as ASTM A36, but  $F_y = 50$  ksi plate has been used here to demonstrate requirements for long welds.

Size the welds for the loads shown.



**Solution:**

*Determine the maximum weld size*

Because the overlapping plate is ¼ in., the maximum fillet weld size that can be used without special notation (built out to obtain full-throat thickness as required in AISC Specification Section J2.2b) is a ⅜-in. fillet weld. A ⅜-in. fillet weld can be deposited in the flat or horizontal position in a single pass (true up to ⅜-in.).

*Determine the required strength*

LRFD	ASD
$P_u = 1.2(33 \text{ kips}) + 1.6(100 \text{ kips}) = 200 \text{ kips}$	$P_a = 33 \text{ kips} + 100 \text{ kips} = 133 \text{ kips}$

*Determine the length of weld required*

LRFD	ASD
The design strength per inch of a ⅜-in. fillet weld is	The allowable strength per inch of a ⅜-in. fillet weld is
$\phi R_n = 1.392 (3) = 4.17 \text{ kips/in.}$	$R_n/\Omega = 0.928 (3) = 2.78 \text{ kips/in.}$
$\frac{P_u}{\phi R_n} = \frac{200 \text{ kips}}{4.17 \text{ kips/in.}} = 48 \text{ in.}$	$\frac{P_a \Omega}{R_n} = \frac{133 \text{ kips}}{2.78 \text{ kips/in.}} = 48 \text{ in.}$
or 24 in. of weld on each side	or 24 in. of weld on each side.



Check the weld for length to weld size ratio

$$\frac{l}{w} = \frac{24 \text{ in.}}{0.188 \text{ in.}} = 128 > 100,$$

Therefore Specification Equation J2-1 must be applied, and the length of weld increased, since the resulting  $\beta$  will reduce the available strength below the required strength.

Try a weld length of 27 in.

Specification  
Eqn. J2-1

The new length to weld size ratio is  $27 \text{ in.} / 0.188 \text{ in.} = 144$

For this ratio

$$\beta = 1.2 - 0.002(l/w) \leq 1.0; \quad 1.2 - 0.002(144) = 0.912$$

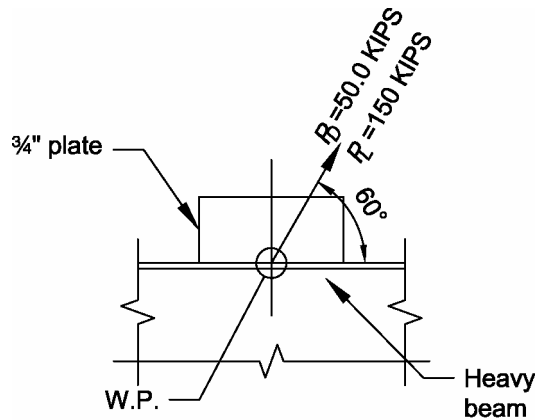
Recheck the weld at its reduced strength

LRFD	ASD
$\phi R_n = (0.912)(4.17 \text{ kips/in.})(54 \text{ in.})$ $= 205 \text{ kips} > P_u = 200 \text{ kips} \quad \mathbf{o.k.}$	$\frac{R_n}{\Omega} = (0.912)(2.78 \text{ kips/in.})(54 \text{ in.})$ $= 137 \text{ kips} > P_a = 133 \text{ kips} \quad \mathbf{o.k.}$
Therefore, use 27 in. of weld on each side	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
$P_u = 1.2(50 \text{ kips}) + 1.6(150 \text{ kips}) = 300 \text{ kips}$	$P_a = 50 \text{ kips} + 150 \text{ kips} = 200 \text{ kips}$

Assume a  $\frac{5}{16}$ -in. fillet weld is used on each side.

The shear strength of a  $\frac{5}{16}$ -in. fillet weld is

LRFD	ASD
$5(1.392) = 6.96 \text{ kip/in.}$	$5(0.928) = 4.64 \text{ kip/in.}$
And for two sides $2(6.96 \text{ kip/in.}) = 13.9 \text{ kip/in.}$	And for two sides $2(4.64 \text{ kip/in.}) = 9.28 \text{ kip/in.}$

Because the angle of the force relative to the axis of the weld is 60 degrees, the strength of the weld can be increased as follows:

$$\begin{aligned}
 k_w &= 0.60F_{EXX} (1.0 + 0.50 \sin^{1.5}\theta) \\
 &= 0.60(70)(1.0 + 0.50 (0.866)^{1.5}) \\
 &= 1.40
 \end{aligned}$$

*Find the increased strength and the required length of weld*

LRFD	ASD
<p><i>Calculate the increased strength.</i>  <math>13.9 \text{ kip/in.} \cdot (1.40) = 19.5 \text{ kip/in.}</math></p>	<p><i>Calculate the increased capacity.</i>  <math>9.280 \text{ kip/in.} \cdot (1.40) = 13.0 \text{ kip/in.}</math></p>
<p><i>Determine the required length of weld.</i>  <math>300 \text{ kips} / 19.5 \text{ kip/in.} = 15.4 \text{ in.}</math></p>	<p><i>Determine the required length of weld.</i>  <math>200 \text{ kips} / 13.0 \text{ kip/in.} = 15.4 \text{ in.}</math></p>
<p>Use 16 in. <b>o.k.</b></p>	<p>Use 16 in. <b>o.k.</b></p>

### Example J.3 Combined Tension and Shear in Bearing Type Connections

#### Given:

A 3/4-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 strength

LRFD	ASD	
Tension = $1.2(3.5) + 1.6(12.0) = 23.4$ kips Shear = $1.2(1.33) + 1.6(4.00) = 8.00$ kips	Tension = $3.5 + 12.0 = 15.5$ kips Shear = $1.33 + 4.00 = 5.33$ kips	
Calculate $f_v$	Calculate $f_v$	
$8.00/0.442 = 18.1$ ksi $\leq \phi F_{nv}$	$5.33/0.442 = 12.1$ ksi $\leq F_{nv}/\Omega$	
Check combined tension and shear.	Check combined tension and shear.	Eq. J3-3a and J3-3b
$F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}$	$F_{nt}' = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt}$	
$F_{nt} = 90$ ksi, $F_{nv} = 48$ ksi	$F_{nt} = 90$ ksi, $F_{nv} = 48$ ksi	
$F_{nt}' = 1.3(90) - \frac{90}{0.75(48)}(18.1)$ $= 71.7 < 90$	$F_{nt}' = 1.3(90) - \frac{2.00(90)}{48}(12.1)$ $= 71.6 < 90$	Table J3.2
$R_n = F_{nt}' A_b = 71.7(0.442) = 31.7$ kips	$R_n = F_{nt}' A_b = 71.6(0.442) = 31.6$ kips	Section J3.6
For combined tension and shear $\phi = 0.75$	For combined tension and shear $\Omega = 2.00$	
Design tensile strength	Allowable tensile strength	
$\phi R_n = 0.75(31.7) = 23.8$ kips $> 23.4$ kips <b>o.k.</b>	$R_n/\Omega = 31.6/2.00 = 15.8$ kips $> 15.5$ kips <b>o.k.</b>	

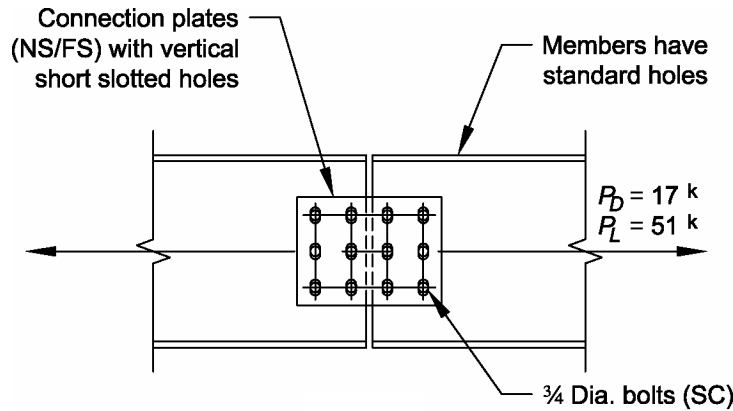
### Example J.4 Slip-Critical Connection with Short Slotted Holes

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 design for slip as a serviceability limit state. The design of 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 3/4-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
$P_u = 1.2(17 \text{ kips}) + 1.6(51 \text{ kips}) = 102 \text{ kips}$	$P_a = 17 \text{ kips} + 51 \text{ kips} = 68 \text{ kips}$

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:

$$\phi = 1.00 \quad \Omega = 1.50$$

Specification  
Section J3.8

Find  $R_n$ , where:

- $\mu = 0.35$  for Class A surface
- $D_u = 1.13$
- $h_{sc} = 0.85$  (short slotted holes)
- $T_b = 28$  kips
- $N_s = 2$ , number of slip planes

Table J3.1

$$R_n = \mu D_u h_{sc} T_b N$$

Eqn.J3-4

$$R_n = 0.35(1.13)(0.85)(28)(2) = 18.8 \text{ kips/bolt}$$

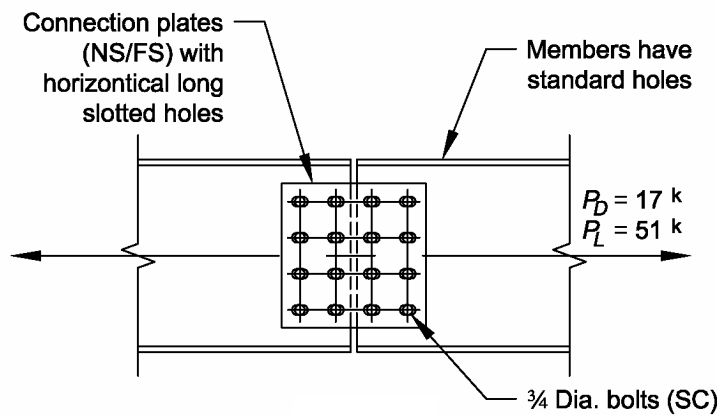
Determine the required number of bolts.

LRFD	ASD
102 kips/1.00(18.8 kips/bolt) = 5.42 bolts	68 kips / $\frac{18.8 \text{ kips/bolt}}{1.50} = 5.42 \text{ bolts}$
Use 6 bolts <b>o.k.</b>	Use 6 bolts <b>o.k.</b>

Manual Table 7-3

**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:**

$P_u = 102 \text{ kips}$  and  $P_a = 68 \text{ kips}$  per the first solution

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.

$$\phi = 0.85 \quad \Omega = 1.76$$

Specification 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)
- $T_b = 28 \text{ kips}$
- $N_s = 2$ , number of slip planes

Table J3.1

$$R_n = \mu D_u h_{sc} T_b N_s$$

$$R_n = 0.35(1.13)(0.70)(28)(2) = 15.5 \text{ kips/bolt}$$

Specification Eqn. J3-4

*Determine the required number of bolts*

LRFD	ASD
$\frac{102 \text{ kips}}{0.85(15.5 \text{ kips/bolt})} = 7.73 \text{ bolts}$	$\frac{68 \text{ kips (1.76)}}{15.5 \text{ kips/bolt}} = 7.63 \text{ bolts}$
Use 8 bolts <span style="float: right;"><b>o.k.</b></span>	Use 8 bolts <span style="float: right;"><b>o.k.</b></span>

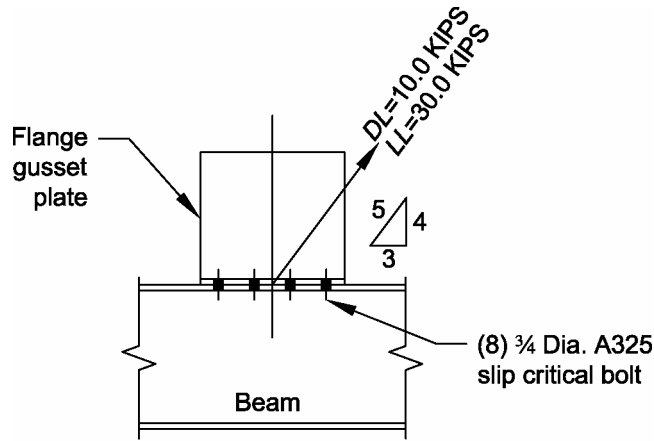
Manual  
Table 7-4

**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 at a serviceability limit state. Use 3/4-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 loads in a rigid fashion.



Solution:

The fastener pretension for a 3/4-in. diameter ASTM A325 bolt is 28 kips

Specification Table J3.1

$D_u = 1.13$  per Specification Section J3.8.

$N_b$  = number of bolts carrying the applied tension.

Determine the tension on bolts, check tension on the bolts and find  $k_s$

LRFD	ASD
$P_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips}) = 60 \text{ kips}$	$P_a = 10 \text{ kips} + 30 \text{ kips} = 40 \text{ kips}$
<i>By geometry,</i>	<i>By geometry,</i>
$T_u = \frac{4}{5} \left( \frac{60 \text{ kips}}{8 \text{ bolts}} \right) = 6 \text{ kips/bolt}$	$T_a = \frac{4}{5} \left( \frac{40 \text{ kips}}{8 \text{ bolts}} \right) = 4 \text{ kips/bolt}$
$V_u = \frac{3}{5} \left( \frac{60 \text{ kips}}{8 \text{ bolts}} \right) = 4.5 \text{ kips/bolt}$	$V_a = \frac{3}{5} \left( \frac{40 \text{ kips}}{8 \text{ bolts}} \right) = 3 \text{ kips/bolt}$
<i>Check bolt tension</i>	<i>Check bolt tension</i>
$\phi R_n = 29.8 \text{ kips/bolt} > 6 \text{ kips/bolt}$ <b>o.k.</b>	$R_n / \Omega = 19.9 \text{ kips/bolt} > 4 \text{ kips/bolt}$ <b>o.k.</b>

Manual Table 7-2



<p><i>Combined tension and shear factor</i></p> $k_s = 1 - \frac{T_u}{D_u T_b N_b} = 1 - \frac{6 \text{ kips}}{1.13(28 \text{ kips})(1)}$ $= 0.810$	<p><i>Combined tension and shear factor</i></p> $k_s = 1 - \frac{1.5T_u}{D_u T_b N_b} = 1 - \frac{1.5 \times 4 \text{ kips}}{1.13(28 \text{ kips})(1)}$ $= 0.810$
---	---

Eqn. J3-5a  
and J3-5b

Multiply the available shear strength of the bolts by the reduction factor  $k_s$

LRFD	ASD
$\phi R_n = 11.1 \text{ kips/bolt}$	$R_n / \Omega = 7.38 \text{ kips/bolt}$

Manual  
Table 7-3

Modify the slip resistance by  $k_s$  and check bolt shear

LRFD	ASD
$k_s \phi R_n = (0.810)(11.1 \text{ kips/bolt})$ $= 8.99 \text{ kips/bolt} > 4.50 \text{ kips/bolt} \quad \mathbf{o.k.}$	$\frac{k_s R_n}{\Omega} = (0.810)(7.38 \text{ kips/bolt})$ $= 5.98 \text{ kips/bolt} > 3.00 \text{ kips/bolt} \quad \mathbf{o.k.}$

### 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.

**Material Properties:**

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

**Solution:**

*Calculate the projected bearing area*

$$A_{pb} = dt_p = (1 \text{ in.})(1\frac{1}{2} \text{ in.}) = 1\frac{1}{2} \text{ in.}^2$$

*Calculate nominal bearing strength*

$$R_n = 1.8F_y A_{pb} = 1.8(36 \text{ ksi})(1\frac{1}{2} \text{ in.}) = 97.2 \text{ kips}$$

Specification  
Eqn. J7-1

*Calculate the available bearing strength*

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(97.2 \text{ kips}) = 72.9 \text{ kips}$	$\Omega = 2.00$ $R_n / \Omega = \frac{97.2 \text{ kips}}{2.00} = 48.6 \text{ kips}$

Specification  
Section J7

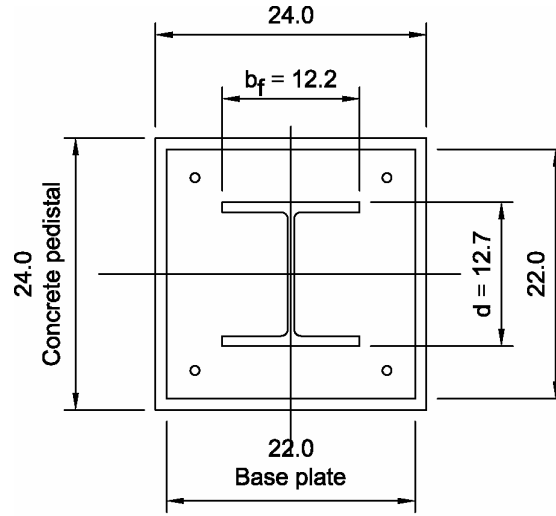
### Example J.7 Base Plate Bearing on Concrete

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:

$$P_D = 115 \text{ kips}$$

$$P_L = 345 \text{ kips}$$



**Material Properties:**

Column W12×96	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$
Base Plate	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$
Concrete Pedestal		$f'_c = 3 \text{ ksi}$	
Grout		$f'_c = 4 \text{ ksi}$	

**Geometric Properties:**

Column W12×96  $d = 12.7 \text{ in.}$   $b_f = 12.2 \text{ in.}$   $t_f = 0.900 \text{ in.}$   $t_w = 0.550 \text{ in.}$

Manual  
Table 1-1

**Solution:**

Calculate the required strength

LRFD	ASD
$P_u = 1.2(115 \text{ kips}) + 1.6(345 \text{ kips}) = 690 \text{ kips}$	$P_a = 115 \text{ kips} + 345 \text{ kips} = 460 \text{ kips}$

$$A_{ped} = (24 \text{ in.})(24 \text{ in.}) = 576 \text{ in.}^2$$

$$A_{col} = (12.7 \text{ in.})(12.2 \text{ in.}) = 155 \text{ in.}^2$$

Since the pedestal area is larger than the column footprint area, but less than 4 times column footprint area, the concrete bearing area will be the geometrically similar area of the pedestal to the base plate.

Calculate the base plate area

LRFD	ASD
$\phi_c = 0.60$ $A_{1(req)} = \frac{P_u}{\phi_c 0.85 f'_c}$ $= \frac{690 \text{ kips}}{0.6(0.85)(3 \text{ ksi})} = 451 \text{ in.}^2$	$\Omega_c = 2.50$ $A_{1(req)} = \frac{P_a \Omega_c}{0.85 f'_c}$ $= \frac{(460 \text{ kips})(2.50)}{(0.85)(3 \text{ ksi})} = 451 \text{ in.}^2$

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 in.×22 in. base plate

Check base plate dimensions

Verify  $N \geq d + 2(3 \text{ in.})$  and  $B \geq b_f + 2(3 \text{ in.})$

Table 14-2

$$d + 2(3 \text{ in.}) = 12.7 \text{ in.} + 2(3 \text{ in.}) = 18.7 \text{ in.} < 22 \text{ in.} \quad \mathbf{o.k.}$$

$$b_f + 2(3 \text{ in.}) = 12.2 \text{ in.} + 2(3 \text{ in.}) = 18.2 \text{ in.} < 22 \text{ in.} \quad \mathbf{o.k.}$$

$$\text{Base plate area, } A_1 = NB = (22 \text{ in.})(22 \text{ in.}) = 484 \text{ in.}^2 > 451 \text{ in.}^2 \quad \mathbf{o.k.}$$

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,

$$A_2 = (24 \text{ in.})(24 \text{ in.}) = 576 \text{ in.}^2$$

Verify the concrete bearing strength

LRFD	ASD
$\phi_c = 0.60$ $\phi_c P_p = \phi_c 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}}$ $= 0.6(0.85)(3 \text{ ksi})(484 \text{ in.}^2) \sqrt{\frac{576 \text{ in.}^2}{484 \text{ in.}^2}}$ $= 808 \text{ kips} > 690 \text{ kips} \quad \mathbf{o.k.}$	$\Omega_c = 2.50$ $P_p / \Omega_c = \frac{0.85 f'_c A_1}{\Omega_c} \sqrt{\frac{A_2}{A_1}}$ $= \frac{(0.85)(3 \text{ ksi})(484 \text{ in.}^2)}{2.5} \sqrt{\frac{576 \text{ in.}^2}{484 \text{ in.}^2}}$ $= 539 \text{ kips} > 460 \text{ kips} \quad \mathbf{o.k.}$

Specification  
Section J8  
Specification  
Eqn. J8-2

Note: It is permitted to take  $\phi_c = 0.65$  per ACI 318-02

Also note that as the area of the base plate approaches the area of concrete, the modifying

ratio,  $\sqrt{\frac{A_2}{A_1}}$ , approaches unity and Specification Eqn. J8-2 converges to Specification Eqn. J8-1.

Calculate required base plate thickness

$$m = \frac{N - 0.95d}{2} = \frac{22 \text{ in.} - 0.95(12.7 \text{ in.})}{2} = 4.97 \text{ in.}$$

$$n = \frac{B - 0.8b_f}{2} = \frac{22 \text{ in.} - 0.8(12.2 \text{ in.})}{2} = 6.12 \text{ in.}$$

$$n' = \frac{\sqrt{db_f}}{4} = \frac{\sqrt{(12.7 \text{ in.})(12.2 \text{ in.})}}{4} = 3.11 \text{ in.}$$

Manual  
Part 14

LRFD	ASD
<i>Calculate required base plate thickness</i>	<i>Calculate required base plate thickness</i>
$X = \frac{4db_f P_u}{(d + b_f)^2 \phi_c P_p}$	$X = \frac{4db_f P_a \Omega_c}{(d + b_f)^2 P_p}$
$= \frac{4(12.7 \text{ in.})(12.2 \text{ in.})(690 \text{ kips})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2 (808 \text{ kips})}$	$= \frac{4(12.7 \text{ in.})(12.2 \text{ in.})(460 \text{ kips})}{(12.7 \text{ in.} + 12.2 \text{ in.})^2 (539 \text{ kips})}$
$= 0.854$	$= 0.853$
$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$	$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$
$= \frac{2\sqrt{0.854}}{1 + \sqrt{1 - 0.854}}$	$= \frac{2\sqrt{0.853}}{1 + \sqrt{1 - 0.853}}$
$= 1.34 > 1, \text{ use } \lambda = 1.$	$= 1.34 > 1, \text{ use } \lambda = 1.$

Note:  $\lambda$  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
$f_{pu} = \frac{P_u}{BN} = \frac{690 \text{ kips}}{(22 \text{ in.})(22 \text{ in.})} = 1.43 \text{ ksi}$	$f_{pa} = \frac{P_a}{BN} = \frac{460 \text{ kips}}{(22 \text{ in.})(22 \text{ in.})} = 0.95 \text{ ksi}$
$t_{p(req)} = l \sqrt{\frac{2f_{pu}}{0.9F_y}}$	$t_{p(req)} = l \sqrt{\frac{3.33f_{pa}}{F_y}}$
$= (6.12 \text{ in.}) \sqrt{\frac{2(1.43 \text{ ksi})}{0.9(36 \text{ ksi})}}$	$= (6.12 \text{ in.}) \sqrt{\frac{3.33(0.950 \text{ ksi})}{(36 \text{ ksi})}}$
$= 1.82 \text{ in.}$	$= 1.82 \text{ in.}$

Use a 2 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**.

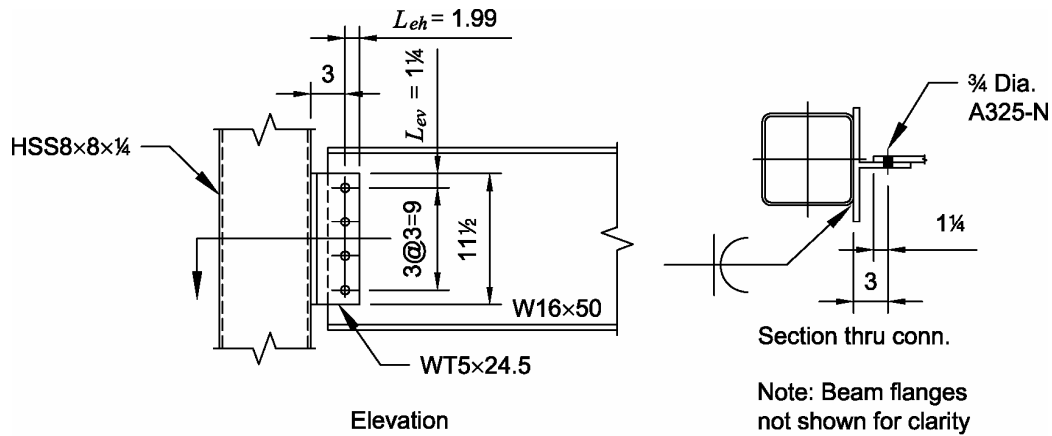
### Example K.1 Welded/bolted Wide Tee Connection to an HSS Column

**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<sub>ev</sub>* 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	
Tee	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Column	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi	

Manual  
Table 2-3

**Geometric Properties:**

Beam	W16×50	$t_w = 0.380$ in. $T = 13 \frac{5}{8}$ in.	$d = 16.3$ in.	$t_f = 0.630$ in.	
Tee	WT5×24.5	$t_s = t_w = 0.340$ in. $b_f = 10.0$ in.	$d = 4.99$ in. $k_l = \frac{13}{16}$ in.	$t_f = 0.560$ in.	
Column	HSS8×8×¼	$t = 0.233$ in.	$B = 8.00$ in.		

Manual  
Tables  
1-1, 1-8, and  
1-12

Calculate the required strength

LRFD	ASD
$P_u = 1.2(6.20 \text{ kips}) + 1.6(18.5 \text{ kips})$ $= 37.0$ kips	$P_a = 6.20 \text{ kips} + 18.5 \text{ kips}$ $= 24.7$ 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 $\phi r_n = 15.9$ kips	Determine the single bolt shear strength $r_n / \Omega = 10.6$ kips	Manual Table 7-1
Determine single bolt bearing strength based on edge distance	Determine single bolt bearing strength based on edge distance	
$L_{ev} = 1\frac{1}{4}$ -in. $\geq 1.25$ in. <b>o.k.</b>	$L_{ev} = 1\frac{1}{4}$ -in. $\geq 1.25$ in. <b>o.k.</b>	Table J3.4
$\phi r_n = 49.4$ kips/in.(0.340 in.) = 16.8 kips	$r_n / \Omega = 32.9$ 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. $> 3(\frac{3}{4}) = 2.25$ in.	$s = 3.00$ in. $> 3(\frac{3}{4}) = 2.25$ in.	
$\phi r_n = 87.8$ kips/in.(0.340 in.) = 29.8 kips	$r_n / \Omega = 58.5$ 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 $e = 3$ in. and $s = 3$ in., determine $C$ .	Using $e = 3$ in. and $s = 3$ in., determine $C$ .	
Try 4 bolts, $C = 2.81 > 2.33$ <b>o.k.</b>	Try 4 bolts, $C = 2.81 > 2.33$ <b>o.k.</b>	Manual Table 7-7

Check WT stem thickness limit

$$t_{max} = \frac{d_b}{2} + \frac{1}{16} \text{ in.} = \frac{(\frac{3}{4} \text{ in.})}{2} + \frac{1}{16} \text{ in.} = 0.438 \text{ in.} > 0.340 \text{ in.} \quad \mathbf{o.k.}$$

Manual  
Section 9

Note: beam web thickness is greater than 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  
Section 10

A W16x50 has a  $T$ -dimension of  $13\frac{5}{8}$  in.

$$L_{min} = T/2 = (13\frac{5}{8} \text{ in.})/2 = 6.81 \text{ in.}$$

Determine WT length required for bolt spacing and edge distances

$$L = 3(3.00 \text{ in.}) + 2(1.25 \text{ in.}) = 11.5 \text{ in.} < T = 13\frac{5}{8} \text{ in.} \quad \mathbf{o.k.}$$

Try  $L = 11.5$  in.

Calculate the stem shear yielding strength

$$R_n = 0.6F_y A_g = 0.6(50 \text{ ksi})(11.5 \text{ in.})(0.340 \text{ in.}) = 117 \text{ kips}$$

Eqn. J4-3

LRFD	ASD
$\phi R_n = 1.00(117 \text{ kips}) = 117 \text{ kips}$	$R_n / \Omega = \frac{117 \text{ kips}}{1.5} = 78.0 \text{ kips}$
117 kips > 37.0 kips <b>o.k.</b>	78.0 kips > 24.7 kips <b>o.k.</b>

Calculate the stem shear rupture strength

Section J4.2

$$\begin{aligned} R_n &= [L - n(d_h + 1/16)](t)(0.6F_u) \\ &= [11.5 \text{ in.} - 4(0.875 \text{ in.})](0.340 \text{ in.})(0.6)(65 \text{ ksi}) \\ &= 106 \text{ kips} \end{aligned}$$

LRFD	ASD
$\phi R_n = 0.75(106 \text{ kips}) = 79.6 \text{ kips}$	$R_n / \Omega = \frac{106 \text{ kips}}{2.00} = 53.0 \text{ kips}$
79.6 kips > 37.0 kips <b>o.k.</b>	53.0 kips > 24.7 kips <b>o.k.</b>

Calculate the stem available block shear rupture strength

Section J4.3

For this case  $U_{bs} = 1.0$

LRFD	ASD
$\frac{\phi F_u A_{nt}}{t} = 76.2 \text{ kips/in.}$	$\frac{F_u A_{nt}}{t\Omega} = 50.8 \text{ kips/in.}$
$\frac{\phi 0.6F_y A_{gv}}{t} = 231 \text{ kips/in.}$	$\frac{0.6F_y A_{gv}}{t\Omega} = 154 \text{ kips/in.}$
$\frac{\phi 0.6F_u A_{nv}}{t} = 210 \text{ kips/in.}$	$\frac{0.6F_u A_{nv}}{t\Omega} = 140 \text{ kips/in.}$
$\phi R_n = \phi 0.6F_u A_{nv} + \phi U_{bs} F_u A_{nt}$ $\leq \phi 0.6F_y A_{gv} + \phi U_{bs} F_u A_{nt}$	$\frac{R_n}{\Omega} = \frac{0.6F_u A_{nv}}{\Omega} + \frac{U_{bs} F_u A_{nt}}{\Omega}$ $\leq \frac{0.6F_y A_{gv}}{\Omega} + \frac{U_{bs} F_u A_{nt}}{\Omega}$
$\phi R_n = 0.340 \text{ in.}(210 \text{ kips/in.} + 76.2 \text{ kips/in.})$ $\leq 0.340 \text{ in.}(231 \text{ kips/in.} + 76.2 \text{ kips/in.})$ $= 97.3 \text{ kips} \leq 104 \text{ kips}$	$R_n / \Omega = 0.340 \text{ in.}(140 \text{ kips/in.} + 50.8 \text{ kips/in.})$ $\leq 0.340 \text{ in.}(154 \text{ kips/in.} + 50.8 \text{ kips/in.})$ $= 64.9 \text{ kips} \leq 69.6 \text{ kips}$
97.3 kips > 37.0 kips <b>o.k.</b>	64.9 kips > 24.7 kips <b>o.k.</b>

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
$M_u = P_u e = 37.0 \text{ kips}(3.00 \text{ in.}) = 111 \text{ kip-in.}$	$M_a = P_a e = 24.7 \text{ kips}(3.00 \text{ in.}) = 74.1 \text{ kip-in.}$

Calculate the stem nominal flexural yielding strength

$$Z = \frac{td^2}{4} = \frac{0.340 \text{ in.} (11.5 \text{ in.})^2}{4} = 11.2 \text{ in.}^3$$

$$S = \frac{td^2}{6} = \frac{0.340 \text{ in.} (11.5 \text{ in.})^2}{6} = 7.49 \text{ in.}^3$$

$$\begin{aligned} M_n &= M_p = F_y Z \leq 1.6 M_y \\ &= 50 \text{ ksi} (11.2 \text{ in.}^3) \leq 1.6 (50 \text{ ksi}) (7.49 \text{ in.}^3) \\ &= 560 \text{ kip-in.} < 599 \text{ kip-in.} \quad \mathbf{o.k.} \end{aligned}$$

Eqn F11-1

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

LRFD	ASD
$\phi M_n = 0.90 (560 \text{ kip-in.})$ $= 504 \text{ kip-in.} > 111 \text{ kip-in.} \quad \mathbf{o.k.}$	$M_n / \Omega = \frac{560 \text{ kip-in.}}{1.67}$ $= 335 \text{ kip-in.} > 74.1 \text{ kip-in.} \quad \mathbf{o.k.}$

Calculate the stem flexural rupture strength

$$\begin{aligned} Z_{net} &= \frac{td^2}{4} - 2t(d_h + 1/16 \text{ in.})(1.5 \text{ in.} + 4.5 \text{ in.}) \\ &= \frac{0.340 \text{ in.} (11.5 \text{ in.})^2}{4} - 2(0.340 \text{ in.})(13/16 \text{ in.} + 1/16 \text{ in.})(1.5 \text{ in.} + 4.5 \text{ in.}) = 7.67 \text{ in.}^3 \end{aligned}$$

$$M_n = F_u Z_{net} = 65 \text{ ksi} (7.67 \text{ in.}^3) = 499 \text{ kip-in.}$$

Manual  
Part 9

LRFD	ASD
$\phi M_n = 0.75 (499 \text{ kip-in.})$ $= 374 \text{ kip-in.} > 111 \text{ kip-in.} \quad \mathbf{o.k.}$	$M_n / \Omega = \frac{499 \text{ kip-in.}}{2.00}$ $= 250 \text{ kip-in.} > 74.1 \text{ kip-in.} \quad \mathbf{o.k.}$

Check beam web bearing

$$t_w > t_s \rightarrow 0.380 \text{ in.} > 0.340 \text{ in.}$$

Beam web is satisfactory for bearing by comparison with WT.

Calculate weld size

Table J2.2

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  $2(1/4 \text{ in.}) = 1/2 \text{ in.}$  and using AISC Specification Table J2.2, the effective throat thickness of the flare bevel weld is  $E = 5/16(1/2 \text{ in.}) = 0.156 \text{ in.}$

The equivalent fillet weld that provides the same throat dimension is

$$\left(\frac{D}{16}\right)\left(\frac{1}{\sqrt{2}}\right) = 0.156 \rightarrow D = 16\sqrt{2}(0.156) = 3.53 \text{ sixteenths of an inch}$$

The equivalent fillet weld size is used in the following calculations

*Check weld ductility*

Manual  
Section 9

$$b = \frac{b_f - 2k_1}{2} = \frac{8.00 \text{ in.} - 2\left(\frac{13}{16} \text{ in.}\right)}{2} = 3.19 \text{ in.}$$

$$w_{\min} = (0.0158) \frac{F_{yc} t_f^2}{b} \left(\frac{b^2}{L^2} + 2\right) \leq 16(0.625 t_s)$$

$$= (0.0158) \frac{(50 \text{ ksi})(0.560 \text{ in.})^2}{3.19 \text{ in.}} \left[\frac{(3.19 \text{ in.})^2}{(11.5 \text{ in.})^2} + 2\right] \leq (0.625)(0.340 \text{ in.})$$

$$= 0.161 \text{ in.} < 0.213 \text{ in.}$$

0161 in. = 2.58 sixteenths of an in.

$D_{\min} = 2.58 < 3.53$  sixteenths of an in.      **o.k.**

*Calculate the nominal weld shear strength*

The load is assumed to act concentrically with the weld group.

$a = 0$ , therefore,  $C = 3.71$

$$R_n = CC_1 D l = 3.71(1.00)(3.53 \text{ sixteenths of an in.})(11.5 \text{ in.}) = 151 \text{ kips}$$

Manual  
Table 8-4

*Check shear rupture in HSS at weld*

Manual  
Section 9

$$t_{\min} = \frac{3.09D}{F_u} = \frac{3.09(3.53 \text{ sixteenths})}{58 \text{ ksi}} = 0.188 \text{ in.}$$

Therefore the weld controls

*Calculate the available weld strength*

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(151 \text{ kips})$ $= 113 \text{ kips} > 37.0 \text{ kips}$ <b>o.k.</b>	$\Omega = 2.00$ $R_n / \Omega = 151 \text{ kips} / 2.00$ $= 75.3 \text{ kips} > 24.7 \text{ kips}$ <b>o.k.</b>

## Example K.2 Welded/bolted Narrow Tee Connection to an HSS Column

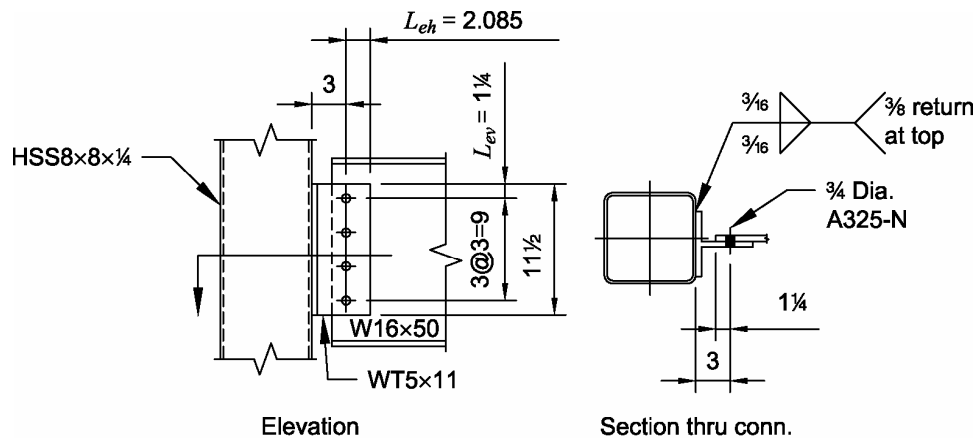
### Given:

Design a connection for a W16×50 beam to an HSS8×8× $\frac{1}{4}$  column using a tee with fillet welds against the flat width of the HSS. Use  $\frac{3}{4}$ -in. diameter A325-N bolts in standard holes with a bolt spacing,  $s$ , of 3.00 in., vertical edge distance  $L_{ev}$  of  $1\frac{1}{4}$ -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 6.5 in.

$$P_D = 6.2 \text{ kips}$$

$$P_L = 18.5 \text{ kips}$$

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.



Note: Beam flanges not shown for clarity

### Solution:

#### Material Properties:

Beam	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Tee	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Column	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi

Manual  
Table 2-3

#### Geometric Properties:

Beam	W16×50	$t_w = 0.380$ in.	$d = 16.3$ in.	$t_f = 0.630$ in.
Column	HSS8×8× $\frac{1}{4}$	$t = 0.233$ in.	$B = 8.00$ in.	
Tee	WT5×24.5	$t_s = t_w = 0.340$ in.	$d = 4.99$ in.	$t_f = 0.560$ in.
		$k_1 = \frac{13}{16}$ in.		

Manual  
Tables 1-1  
and 1-12

Calculate the required strength

LRFD	ASD
$P_u = 1.2(6.20 \text{ kips}) + 1.6(18.5 \text{ kips})$ $= 37.0 \text{ kips}$	$P_a = 6.20 \text{ kips} + 18.5 \text{ kips}$ $= 24.7 \text{ kips}$

Verify the strength of the WT

Maximum flange width assuming 1/4-in. welds and HSS corner radius equal to 1.5 times the nominal thickness  $(1.5)(1/4 \text{ in.}) = 3/8 \text{ in.}$

$$b_f \leq 8.00 \text{ in.} - 2(3/8 \text{ in.}) - 2(1/4 \text{ in.}) = 6.75 \text{ in.}$$

The strength of the stem thickness was verified in Example K.1.

Determine the number of bolts

LRFD	ASD	
Determine the bolt shear strength $\phi r_n = 15.9 \text{ kips}$	Determine the bolt shear strength $r_n / \Omega = 10.6 \text{ kips}$	Manual Table 7-1
Determine bolt bearing strength based on edge distance $L_{ev} = 1\ 1/4\text{-in.} \geq 1.25 \text{ in.}$	Determine bolt bearing strength based on edge distance $L_{ev} = 1\ 1/4\text{-in.} \geq 1.25 \text{ in.}$	Table J3.4
$\phi r_n = 49.4 \text{ kips/in.}(0.340 \text{ in.}) = 16.8 \text{ kips}$	$r_n / \Omega = 32.9 \text{ kips/in.}(0.340 \text{ in.}) = 11.2 \text{ kips}$	Manual Table 7-6
Bolt shear strength controls	Bolt shear strength controls	

Determine the coefficient for the eccentrically loaded bolt group

LRFD	ASD	
$\phi r_n = 15.9 \text{ kips}$	$r_n / \Omega = 10.6 \text{ kips}$	Manual Table 7-7
$C_{min} = \frac{P_u}{\phi r_n} = \frac{37.0 \text{ kips}}{15.9 \text{ kips}} = 2.33$	$C_{min} = \frac{P_a}{r_n / \Omega} = \frac{24.7 \text{ kips}}{10.6 \text{ kips}} = 2.33$	
Using $e = 3.00 \text{ in.}$ and $s = 3.00 \text{ in.}$ , enter Manual Table 7-7	Using $e = 3.00 \text{ in.}$ and $s = 3.00 \text{ in.}$ , enter Manual Table 7-7	
Try 4 bolts,	Try 4 bolts,	
$C = 2.81 > 2.33$ <b>o.k.</b>	$C = 2.81 > 2.33$ <b>o.k.</b>	

Determine the minimum fillet weld size based on the thinner part

Minimum size = 1/8 in. (D = 2) for welding to 1/4 in. material Table J2.4

Check weld ductility

$$b = \frac{(b_f - 2k_1)}{2} = \frac{[5\ 3/4 \text{ in.} - 2(5/8 \text{ in.})]}{2} = 2\ 1/4 \text{ in.}$$

$$w_{min} = (0.0158) \frac{F_{yc} t_f^2}{b} \left( \frac{b^2}{L^2} + 2 \right) \leq (0.625 t_s)$$

Manual  
Part 9

$$= (0.0158) \frac{(50 \text{ ksi})(0.360 \text{ in.})^2}{2 \frac{1}{4} \text{ in.}} \left[ \frac{(2 \frac{1}{4} \text{ in.})^2}{(11 \frac{1}{2} \text{ in.})^2} + 2 \right] \leq (0.625)(0.240 \text{ in.})$$

$$= 0.093 \text{ in.} \leq 0.150 \text{ in.}$$

$$D_{min} = 0.093(0.625) = 0.058 \text{ sixteenths of an in.} < 2 \quad \mathbf{o.k.}$$

Calculate minimum wall thickness in HSS to match weld strength

Manual  
Section 9

$$t_{min} = \frac{3.09D}{F_u} = \frac{3.09(2)}{58 \text{ ksi}} = 0.107 \text{ in.} < 0.233 \text{ in.}$$

Therefore weld controls

Calculate the weld nominal shear strength

The load is assumed to act concentrically with the weld group.

$$a = 0; \text{ therefore } C = 3.71$$

$$R_n = CC_1Dl = 3.71(1.00)(2 \text{ sixteenths of an in.})(11.5 \text{ in.}) = 85.3 \text{ kips}$$

Manual  
Table 8-4

Calculate the available weld shear strength

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(85.3 \text{ kips}) = 64.0 \text{ kips}$	$R_n / \Omega = \frac{85.3 \text{ kips}}{2.00} = 42.7 \text{ kips}$
$64.0 \text{ kips} > 37.0 \text{ kips} \quad \mathbf{o.k.}$	$42.7 \text{ kips} > 24.7 \text{ kips} \quad \mathbf{o.k.}$

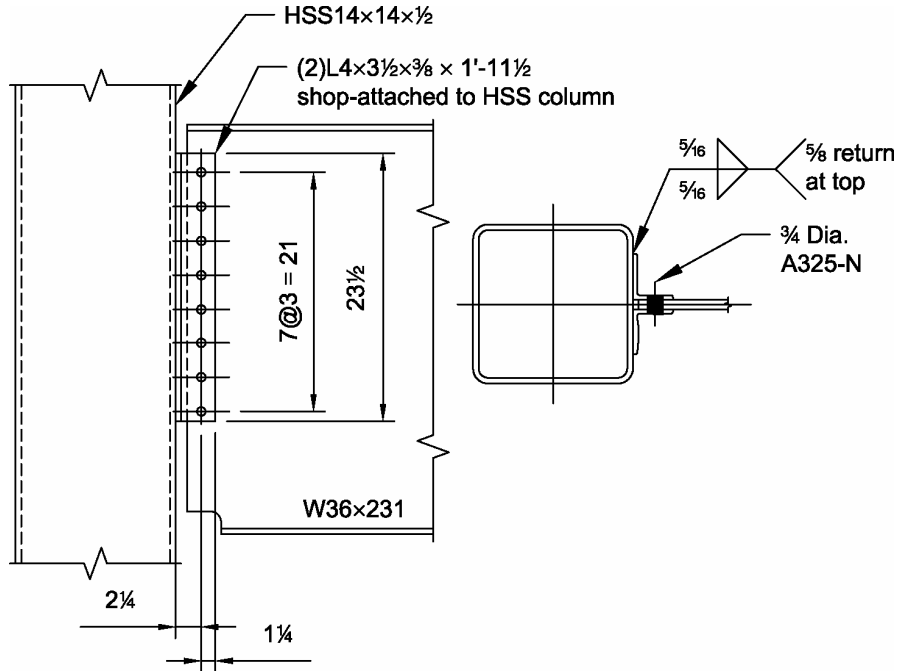
Note: Use a 3/16-in. fillet weld as a practical minimum.

### 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.

$P_D = 37.5$  kips  
 $P_L = 113$  kips



**Solution:**

**Material Properties:**

Beam	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Column	ASTM A500 Gr.	$F_y = 46$ ksi	$F_u = 58$ ksi	Manual 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 Tables 1-1 and 1-12
Column	HSS14×14×½	$t = 0.465$ in.	$B = 14.0$ in.	

Compute the required strength

LRFD	ASD
$R_u = 1.2(37.5 \text{ kips}) + 1.6(113 \text{ kips})$ $= 225 \text{ kips}$	$R_u = 37.5 \text{ kips} + 113 \text{ kips}$ $= 151 \text{ kips}$



*Design bolts and welds*

Try 8 rows of bolts and  $\frac{5}{16}$ -in. welds B

Obtain the bolt group and angle available strengths from Table 10-1

LRFD	ASD
$\phi R_n = 254 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 170 \text{ kips} > 151 \text{ kips}$ <b>o.k.</b>

Manual  
Table 10-1

Obtain the available weld strength from Table 10-2

LRFD	ASD
$\phi R_n = 279 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 186 \text{ kips} > 151 \text{ kips}$ <b>o.k.</b>

Manual  
Table 10-2

Check the minimum support thickness

Minimum support thickness = 0.238 in. < 0.465 in. **o.k.**

Manual  
Table 10-2

Calculate the minimum angle thickness

$$t_{min} = w + \frac{1}{16} = \frac{5}{16} + \frac{1}{16} = \frac{3}{8} \text{ in.}$$

Section  
J2.2b

Use  $\frac{3}{8}$ -in. angle thickness to accommodate the welded legs of the double angle connection.

Use 2L4×3½ × $\frac{3}{8}$ ×1'-11½".

L = 23.5 in. > T/2 **o.k.**

B of the HSS is greater than 12 in., the minimum acceptable width for 4-in. OSL angles.

Calculate the available beam web strength

LRFD	ASD
$\phi R_n = (702 \text{ kips/in.})(0.760 \text{ in.}) = 534 \text{ kips}$ 534 kips > 225 kips <b>o.k.</b>	$R_n / \Omega = (468 \text{ kips/in.})(0.760 \text{ in.}) = 356 \text{ kips}$ 356 kips > 151 kips <b>o.k.</b>

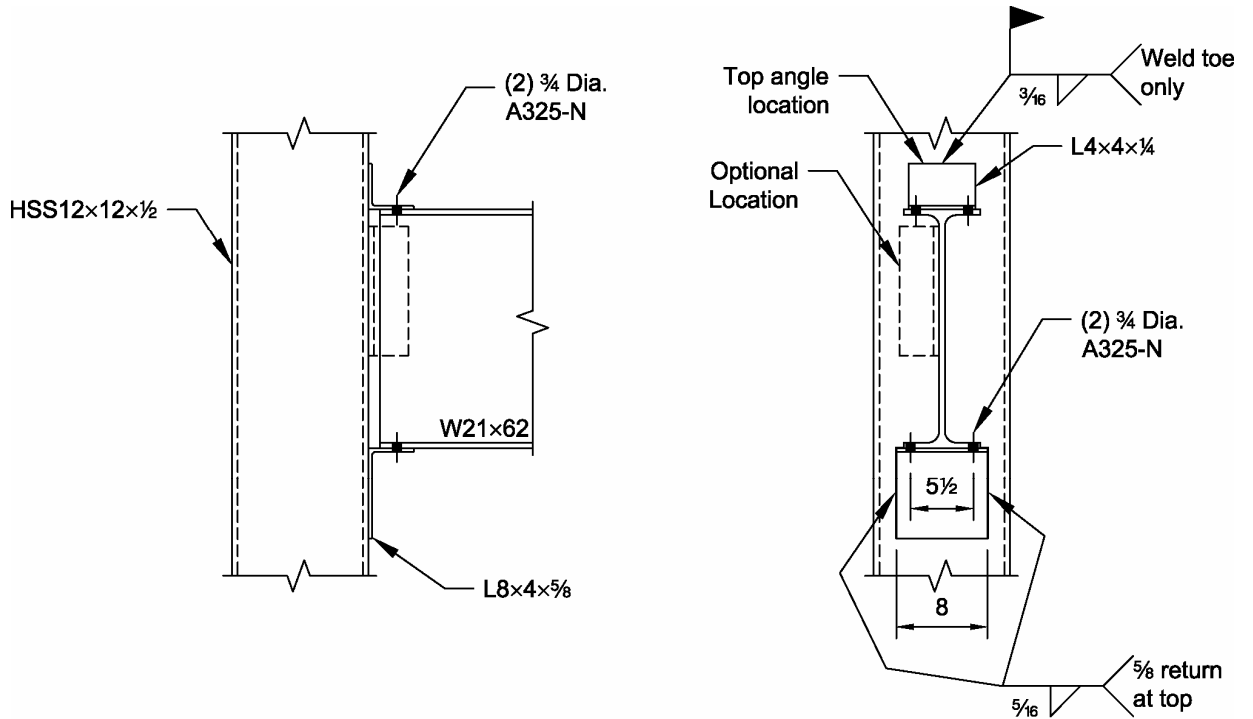
Manual  
Table 10-1

### 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	
Column	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi	
Angles	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	

Manual  
Table 2-3

**Geometric Properties:**

Beam	W21×62	$t_w = 0.400$ in.	$d = 21.0$ in.	$k = 1.12$ in.	
Column	HSS12×12×½	$t = 0.465$ in.	$B = 12.0$ in.		

Manual  
Tables 1-1  
and 1-12

Calculate the required strength

LRFD	ASD
$R_u = 1.2(9.00 \text{ kips}) + 1.6(27.0 \text{ kips})$ $= 54.0 \text{ kips}$	$R_a = 9.00 \text{ kips} + 27.0 \text{ kips}$ $= 36.0 \text{ kips}$

*Design the seat angle and weld*

LRFD	ASD
<p><i>Check local web yielding</i></p> $N_{min} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$ $= \frac{54.0 \text{ kips} - 55.8 \text{ kips}}{20.0 \text{ kips/in.}} \geq 1.12 \text{ in.}$ <p>Use <math>N_{min} = 1.12 \text{ in.}</math></p> <p><i>Check web crippling when <math>N/d \leq 0.2</math>,</i></p> $N_{min} = \frac{R_u - \phi R_3}{\phi R_4} = \frac{54.0 \text{ kips} - 71.7 \text{ kips}}{5.37 \text{ kips/in.}}$ <p>which results in a negative quantity.</p> <p><i>Check web crippling when <math>N/d &gt; 0.2</math>,</i></p> $N_{min} = \frac{R_u - \phi R_5}{\phi R_6} = \frac{54.0 \text{ kips} - 64.2 \text{ kips}}{7.16 \text{ kips/in.}}$ <p>which results in a negative quantity.</p>	<p><i>Check local web yielding</i></p> $N_{min} = \frac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k$ $= \frac{36.0 \text{ kips} - 37.2 \text{ kips}}{13.3 \text{ kips/in.}} \geq 1.12 \text{ in.}$ <p>Use <math>N_{min} = 1.12 \text{ in.}</math></p> <p><i>Check web crippling when <math>N/d \leq 0.2</math>,</i></p> $N_{min} = \frac{R_a - R_3 / \Omega}{R_4 / \Omega} = \frac{36.0 \text{ kips} - 47.8 \text{ kips}}{3.58 \text{ kips/in.}}$ <p>which results in a negative quantity.</p> <p><i>Check web crippling when <math>N/d &gt; 0.2</math>,</i></p> $N_{min} = \frac{R_a - R_5 / \Omega}{R_6 / \Omega} = \frac{36.0 \text{ kips} - 42.8 \text{ kips}}{4.78 \text{ kips/in.}}$ <p>which results in a negative quantity.</p>

Manual  
Table 9-4  
Manual  
Section 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 ( $N_{req} = k = 1.12 \text{ in.}$ ). Thus,  $N_{min} = 1.12 \text{ in.}$

Try a L8×4×<sup>5</sup>/<sub>8</sub> with <sup>5</sup>/<sub>16</sub> in. fillet welds.

*Determine outstanding angle leg available strength*

LRFD	ASD
$\phi R_n = 81.0 \text{ kips}$ $81.0 \text{ kips} > 54.0 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 53.9 \text{ kips}$ $53.9 \text{ kips} > 36.0 \text{ kips}$ <b>o.k.</b>

Manual  
Table 10-6

*Determine weld available strength*

LRFD	ASD
$\phi R_n = 66.7 \text{ kips} > 54.0 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 44.5 \text{ kips} > 36.0 \text{ kips}$ <b>o.k.</b>

Manual  
Table 10-6

Since the  $t$  of the HSS is greater than the  $t_{min}$  for the <sup>5</sup>/<sub>16</sub>-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×<sup>1</sup>/<sub>4</sub> in. top angle. Use <sup>3</sup>/<sub>16</sub>-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 <sup>3</sup>/<sub>4</sub>-in. diameter ASTM A325-N bolts.

Manual  
Part 10

### Example K.5 Stiffened Seated Connection to an HSS Column

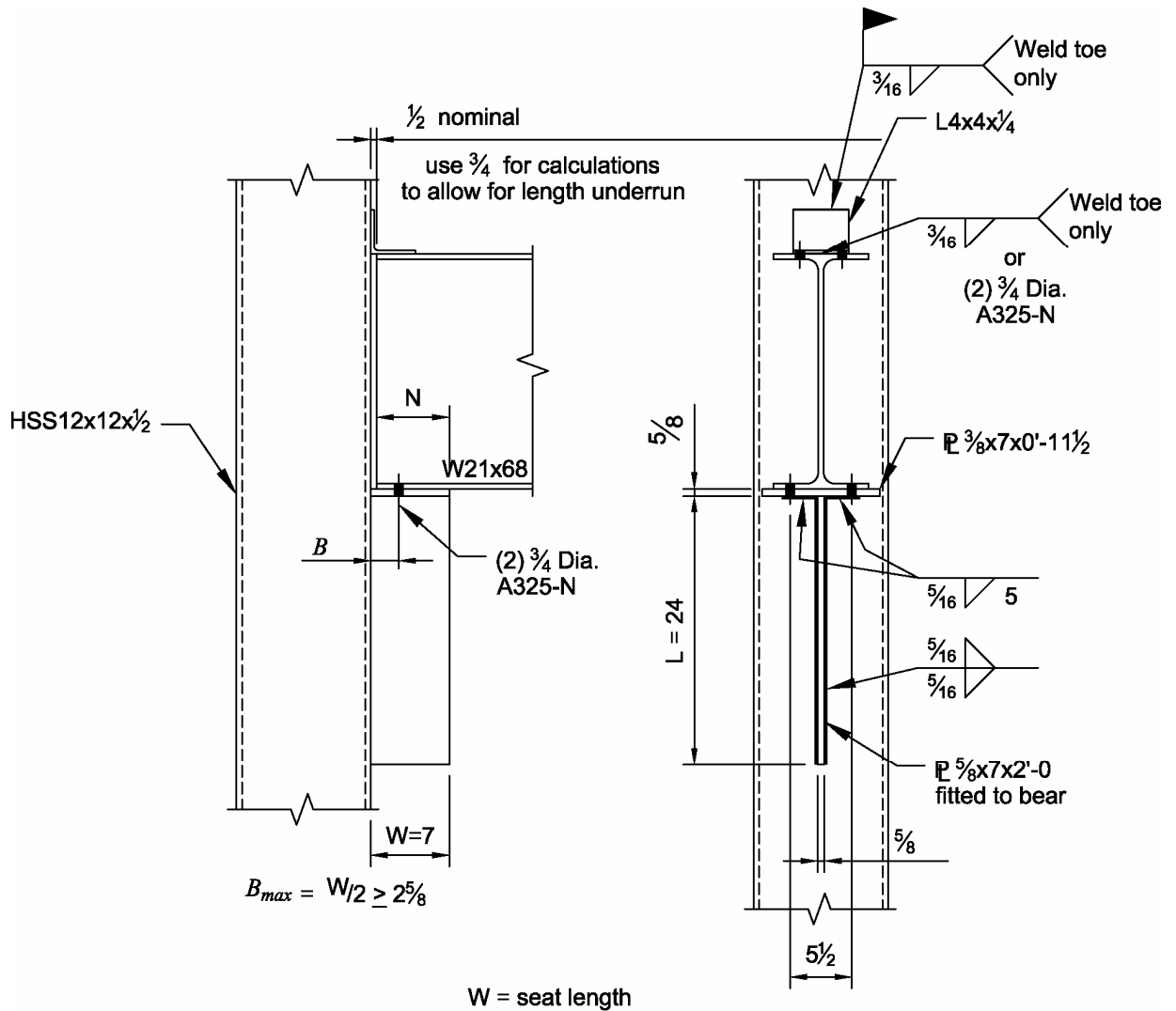
**Given:**

Use Manual Tables 10-8 and 10-14 to design a stiffened seated connection for a W21x68 beam to a HSS12x12x1/2 column.

$P_D = 20$  kips  
 $P_L = 60$  kips

Use 3/4-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	Table 2-3
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.	$k = 1.19$ in.
Column	HSS12×12×½	$t = 0.465$ in.	$B = 12.0$ in.	

Manual  
Tables 1-1  
and 1-12

Check limits of applicability for Specification Section K1

Strength:  $F_y = 46$  ksi < 52 ksi **o.k.**

Section K1.2

Ductility:  $\frac{F_y}{F_u} = \frac{46 \text{ ksi}}{58 \text{ ksi}} = 0.79 < 0.80$  **o.k.**

Calculate the required strength

LRFD	ASD
$P_u = 1.2(20.0 \text{ kips}) + 1.6(60.0 \text{ kips})$ = 120 kips	$P_a = 20.0 \text{ kips} + 60.0 \text{ kips}$ = 80.0 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  
Section 10

LRFD	ASD
$W_{\min} = \frac{R_u - \phi R_5}{\phi R_6} + \text{setback} \geq k + \text{setback}$ = $\frac{120 \text{ kips} - 75.9 \text{ kips}}{7.94 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ = 6.31 in.	$W_{\min} = \frac{R_a - R_5 / \Omega}{R_6 / \Omega} + \text{setback} \geq k + \text{setback}$ = $\frac{80.0 \text{ kips} - 50.6 \text{ kips}}{5.29 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ = 6.34 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
$W_{\min} = \frac{R_u - \phi R_1}{\phi R_2} + \text{setback}$ = $\frac{126 \text{ kips} - 63.7 \text{ kips}}{21.5 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ = 3.65 in.	$W_{\min} = \frac{R_a - R_1 / \Omega}{R_2 / \Omega} + \text{setback}$ = $\frac{84.0 \text{ kips} - 42.5 \text{ kips}}{14.3 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ = 3.65 in.

Manual  
Table 9-4

The minimum stiffener width,  $W$ , for web crippling controls. Use  $W = 7.00$  in.

Check bearing width assumption

$$N = 7.00 \text{ in.} - \frac{3}{4} \text{ in.} = 6 \frac{1}{4} \text{ in.}$$

$$N/d = \frac{6 \frac{1}{4} \text{ in.}}{21.1 \text{ in.}} = 0.30 > 0.2, \text{ as assumed.}$$

Determine the weld strength requirements for the seat plate

Try  $L = 24$  in. with 5/16 in. fillet welds. Enter Manual Table 10-8 using  $W = 7$  in. as determined above.

LRFD	ASD
$\phi R_n = 293 \text{ kips} > 126 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 195 \text{ kips} > 84.0 \text{ kips}$ <b>o.k.</b>

Manual  
Table 10-8

Since  $t$  of the HSS is greater than  $t_{min}$  for the  $\frac{5}{16}$ -in. weld, 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  $\frac{5}{16}$ -in. weld on each side of the stiffener.

Manual  
Section 10

*Determine the stiffener plate thickness*

To develop the stiffener-to-seat plate welds, the minimum stiffener thickness is

$$t_{p \min} = 2w = 2\left(\frac{5}{16} \text{ in.}\right) = \frac{5}{8} \text{ in.}$$

For a stiffener with  $F_y = 36$  ksi and a beam with  $F_y = 50$  ksi, the minimum stiffener thickness,

$$t_{p \min} = \left(\frac{F_{y \text{ beam}}}{F_{y \text{ stiffener}}}\right) t_w = \left(\frac{50 \text{ ksi}}{36 \text{ ksi}}\right) (0.430 \text{ in.}) = 0.597 \text{ in.}$$

For a stiffener with  $F_y = 36$  ksi and a column with  $F_u = 58$  ksi, the maximum stiffener thickness is

$$t_{p \max} = \frac{F_u t}{F_{yp}} = \frac{58 \text{ ksi} (0.465 \text{ in.})}{36 \text{ ksi}} = 0.749 \text{ in.}$$

Eqn. K1-10

Use stiffener thickness of  $\frac{5}{8}$  in.

*Determine the stiffener length using Manual Table 10-14*

LRFD	ASD
$\left(\frac{R_u W}{t^2}\right)_{req} = \frac{120 \text{ kips} (7.00 \text{ in.})}{(0.465 \text{ in.})^2}$ $= 4100 \text{ kips/in.}$ <p>To satisfy the minimum above, select a stiffener <math>L = 24</math> in. from Table 10-14</p> $\frac{R_u W}{t^2} = 4310 \text{ kips/in.} > 4100 \text{ kips/in.} \quad \mathbf{o.k.}$	$\left(\frac{R_a W}{t^2}\right)_{req} = \frac{80.0 \text{ kips} (7.00 \text{ in.})}{(0.465 \text{ in.})^2}$ $= 2720 \text{ kips/in.}$ <p>To satisfy the minimum above, select a stiffener <math>L = 24</math> in. from Table 10-14</p> $\frac{R_a W}{t^2} = 2870 \text{ kips/in.} > 2720 \text{ kips/in.} \quad \mathbf{o.k.}$

Manual  
Table 10-14

Use PL $\frac{5}{8} \times 7 \times 2'-0$  for the stiffener.

*Check the HSS width*

The minimum width is  $0.4L + t_p + 3t$

$$B = 12.0 \text{ in.} > 0.4(24.0 \text{ in.}) + \frac{5}{8} \text{ in.} + 3(0.465 \text{ in.}) = 11.6 \text{ in.}$$

*Determine the seat plate dimensions*

To accommodate two  $\frac{3}{4}$ -in. diameter ASTM A325-N bolts on a  $5\frac{1}{2}$  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:

$$0.4(24 \text{ in.}) + 0.625 \text{ in.} = 10.2 \text{ in.}$$

Manual  
Part 10

Note: To allow room to start and stop welds, an 11.5 in. width is used.

Use PL $\frac{3}{8} \times 7 \times 0' - 11\frac{1}{2}$  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  $\frac{3}{16}$  in. The angle thickness should be  $\frac{1}{16}$  in. larger.

Use L4 $\times$ 4 $\times$  $\frac{1}{4}$  with  $\frac{3}{16}$ -in. fillet welds along the toes of the angle to the beam flange and HSS. Alternatively two  $\frac{3}{4}$ -in. diameter ASTM A325-N bolts may be used to connect the beam leg of the angle to the beam flange.

### Example K.6 Single-Plate Connection to Rectangular HSS Column

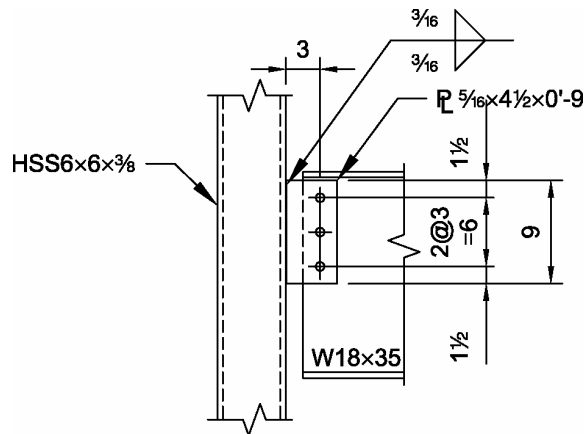
#### Given:

Use Manual Table 10-9 to design a single-plate connection for between a W18×35 beam and a HSS6×6× $\frac{3}{8}$  column.

$$P_D = 6.5 \text{ kips}$$

$$P_L = 19.5 \text{ kips}$$

Use  $\frac{3}{4}$ -in. diameter ASTM A325-N bolts in standard holes and 70 ksi weld electrode.



#### Solution:

##### Material Properties:

Beam	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Tables 2-3 and 2-4
Column	ASTM A500 Gr. B	$F_y = 46 \text{ ksi}$	$F_u = 58 \text{ ksi}$	
Plate	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

##### Geometric Properties:

Beam	W18×35	$d = 17.7 \text{ in.}$	$t_w = 0.300 \text{ in.}$	$T = 15\frac{1}{2} \text{ in.}$	Manual Tables 1-1 and 1-12
Column	HSS6×6× $\frac{3}{8}$	$B = H = 6.00 \text{ in.}$	$t = 0.349 \text{ in.}$	$b/t = 14.2$	

Determine applicability of Specification Section K1

Strength:  $F_y = 46 \text{ ksi} < 52 \text{ ksi}$      **o.k.**

Ductility:  $\frac{F_y}{F_u} = \frac{46 \text{ ksi}}{58 \text{ ksi}} = 0.79 < 0.80$      **o.k.**

Section K1.2

Determine if a single plate connection is suitable (the HSS wall is not slender)

Slenderness:  $\lambda = 1.40 \sqrt{\frac{E}{F_y}} = 1.40 \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} = 35.2 > 14.2$      **o.k.**

Table B4.1  
Case 12



Calculate the required strength

LRFD	ASD
$R_u = 1.2(6.50 \text{ kips}) + 1.6(19.5 \text{ kips})$ $= 39.0 \text{ kips}$	$R_a = 6.50 \text{ kips} + 19.5 \text{ kips}$ $= 26.0 \text{ kips}$

Calculate maximum single-plate thickness

$$t_{p \max} = \frac{F_u t}{F_{yp}} = \frac{58 \text{ ksi}(0.349 \text{ in.})}{36 \text{ ksi}} = 0.562 \text{ in.}$$

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  $\frac{5}{16}$ -in. plate thickness with  $\frac{1}{4}$ -in. fillet welds.

$$t_p = \frac{5}{16} \text{ in.} < 0.562 \text{ in.} \quad \mathbf{o.k.}$$

Obtain the available single plate connection strength from Manual Table 10-9

LRFD	ASD
$\phi R_n = 47.7 \text{ kips} > 39.0 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = 31.8 \text{ kips} > 26.0 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 10-9

Use a PL $\frac{5}{16} \times 4\frac{1}{2} \times 0' - 9''$

Check HSS shear rupture strength at welds

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(4)}{58 \text{ ksi}} = 0.427 \text{ in.} > t = 0.349 \text{ in.} \quad \mathbf{n.g.}$$

Manual  
Section 9

Since  $t < t_{min}$ , the shear rupture strength of the HSS is less than the weld strength. Consider reducing the weld size to  $\frac{3}{16}$  in.

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(3)}{58 \text{ ksi}} = 0.320 \text{ in.} < t = 0.349 \text{ in.} \quad \mathbf{o.k.}$$

Determine the weld shear strength

This is the special case where the load is not in the plane of the weld group. Use  $k = 0$ .

Manual  
Table 8-4

$$a = \frac{e}{l} = \frac{3.00 \text{ in.}}{9.00 \text{ in.}} = 0.333 \text{ Therefore } C = 2.95 \text{ (interpolated)}$$

$$R_n = CC_1Dl = 2.95(1.00)(3 \text{ sixteenths of an in.})(9.00 \text{ in.}) = 79.6 \text{ kips}$$

Calculate the available weld shear strength

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(79.6 \text{ kips}) = 59.7 \text{ kips}$ $59.7 \text{ kips} > 39.0 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $R_n / \Omega = \frac{79.6 \text{ kips}}{2.00} = 39.8 \text{ kips}$ $39.8 \text{ kips} > 26.0 \text{ kips} \quad \mathbf{o.k.}$

Calculate the available beam web bearing strength from Manual Table 10-1

For three 3/4-in. diameter bolts

LRFD	ASD
$\phi R_n = (263 \text{ kips/in.})(0.300 \text{ in.})$ $= 78.9 \text{ kips} > 39.0 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = (175 \text{ kips/in.})(0.300 \text{ in.})$ $= 52.5 \text{ kips} > 26.0 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 10-1

## Example K.7 Through-Plate Connection

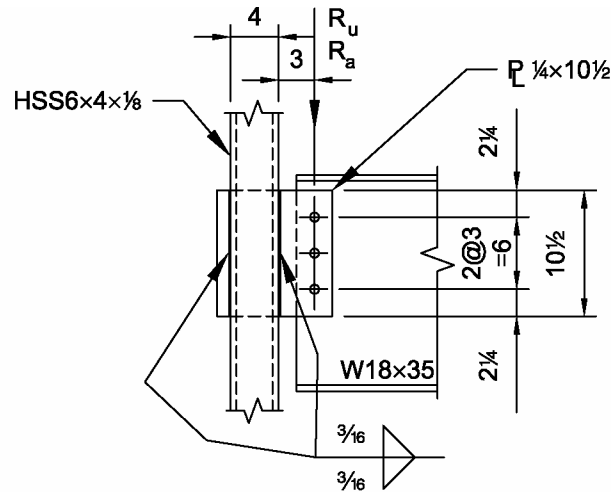
### Given:

Use Table 10-9 to design a through-plate connection between a W18×35 beam and a HSS6×4× $\frac{1}{8}$  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.

$$P_D = 6.5 \text{ kips}$$

$$P_L = 19.5 \text{ kips}$$

Use  $\frac{3}{4}$ -in. diameter ASTM A325-N bolts in standard holes and 70 ksi weld electrode.



### Solution:

#### Material Properties:

Beam	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Column	ASTM A500 Gr. B	$F_y = 46 \text{ ksi}$	$F_u = 58 \text{ ksi}$	
Plate	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

Manual  
Tables 2-3  
and 2-4

#### Geometric Properties:

Beam	W18×35	$d = 17.7 \text{ in.}$	$t_w = 0.300 \text{ in.}$	$T = 15\frac{1}{2} \text{ in.}$
Column	HSS6×4× $\frac{1}{8}$	$B = 4.00 \text{ in.}$	$H = 6.00 \text{ in.}$	$t = 0.116 \text{ in.}$
		$h/t = 48.7$		

Manual  
Tables 1-1  
and 1-11

Determine applicability of Specification Section K1

Strength:  $F_y = 46 \text{ ksi} < 52 \text{ ksi}$     **o.k.**

Section K1.2

Ductility:  $\frac{F_y}{F_u} = \frac{46 \text{ ksi}}{58 \text{ ksi}} = 0.79 < 0.80$     **o.k.**

Determine if a single-plate connection is suitable (HSS wall is not slender)

Slenderness:  $\lambda = 1.40 \sqrt{\frac{E}{F_y}} = 1.40 \sqrt{\frac{29,000 \text{ ksi}}{46 \text{ ksi}}} = 35.2 < 48.7$     **n.g.**

Table B4.1  
Case 12

Since the HSS6×4× $\frac{1}{8}$  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
$R_u = 1.2(3.30 \text{ kips}) + 1.6(9.90 \text{ kips})$ $= 19.8 \text{ kips}$	$R_a = 3.30 \text{ kips} + 9.90 \text{ kips}$ $= 13.2 \text{ kips}$

Design the portion of the through-plate connection that resembles a single-plate

Try 3 rows of bolts ( $L=8\frac{1}{2}$ ) and a  $\frac{1}{4}$  in. plate thickness with  $\frac{3}{16}$ -in. fillet welds.

Obtain the available single plate connection strength from Manual Table 10-9

LRFD	ASD
$\phi R_n = 38.3 \text{ kips} > 19.8 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 25.6 \text{ kips} > 13.2 \text{ kips}$ <b>o.k.</b>

Manual  
Table 10-9a

Check weld size for through-plate connection

$$e = 3.00 \text{ in.}$$

$$t_{\min} = \frac{6.19D}{F_u} = \frac{6.19(3)}{58 \text{ ksi}} = 0.320 \text{ in.} > 0.116 \text{ in.} \quad (\text{proration required})$$

Calculate the required weld strength

LRFD	ASD
$V_{fu} = \frac{R_u(B+e)}{B}$ $= \frac{(19.8 \text{ kips})(4.00 \text{ in.} + 3.00 \text{ in.})}{4.00 \text{ in.}}$ $= 34.7 \text{ kips}$	$V_{fa} = \frac{R_a(B+e)}{B}$ $= \frac{(13.2 \text{ kips})(4.00 \text{ in.} + 3.00 \text{ in.})}{4.00 \text{ in.}}$ $= 23.1 \text{ kips}$
$v_{fu} = \frac{V_{fu}}{L} = \frac{34.7 \text{ kips}}{8.50 \text{ in.}} = 4.08 \text{ kips/in.}$	$v_{fa} = \frac{V_{fa}}{L} = \frac{23.1 \text{ kips}}{8.50 \text{ in.}} = 2.72 \text{ kips/in.}$

Calculate the available weld strength

LRFD	ASD
$\phi r_n = 1.392 D n_w \left( \frac{t}{t_{\min}} \right)$ $= 1.392(3)(2 \text{ welds}) \left( \frac{0.116 \text{ in.}}{0.320 \text{ in.}} \right)$ $= 3.03 \text{ kips/in.} < 4.08 \text{ kips/in.}$ <b>n.g.</b>	$\phi r_n = 0.928 D n_w \left( \frac{t}{t_{\min}} \right)$ $= 0.928(3)(2 \text{ welds}) \left( \frac{0.116 \text{ in.}}{0.320 \text{ in.}} \right)$ $= 2.02 \text{ kips/in.} < 2.72 \text{ kips/in.}$ <b>n.g.</b>
A deeper plate is required.	A deeper plate is required.
$L_{req} = \frac{V_{fu}}{\phi r_n} = \frac{34.7 \text{ kips}}{3.03 \text{ kips/in.}} = 11.5 \text{ in.}$	$L_{req} = \frac{V_{fa}}{r_n / \Omega} = \frac{23.1 \text{ kips}}{2.02 \text{ kips/in.}} = 11.5 \text{ in.}$

Manual  
Part 9

Use an 11½ in. long plate and increase the vertical edge distance to 2¾ in.

*Recheck the plate length*

$L = 11\frac{1}{2} \text{ in.} < T = 15\frac{1}{2} \text{ in.} \quad \mathbf{o.k.}$

*Calculate the available beam web bearing strength*

For three ¾ in. diameter bolts

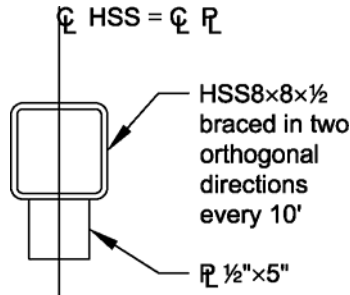
LRFD	ASD
$\phi R_n = (263 \text{ kips/in.})(0.300 \text{ in.})$ $= 78.9 \text{ kips} > 19.8 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = (175 \text{ kips/in.})(0.300 \text{ in.})$ $= 52.5 \text{ kips} > 13.2 \text{ kips} \quad \mathbf{o.k.}$

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 5 in. wide plate. The HSS  $8 \times 8 \times \frac{1}{2}$  is in compression with nominal axial loads of  $P_{D \text{ column}} = 54$  kips and  $P_{L \text{ column}} = 162$  kips. The HSS has negligible flexural required strength and the connecting member is 4 in. wide.



#### Solution:

#### Material Properties:

Column	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi	Manual Table 2-3 Table 2-4
Plate	ASTM A36	$F_{yp} = 36$ ksi	$F_u = 58$ ksi	

#### Geometric Properties:

Column	HSS $8 \times 8 \times \frac{1}{2}$	$B = 8.00$ in.	$t = 0.465$ in.	Manual Table 1-12
Plate		$B_p = 5.50$ in.	$t_p = 0.500$ in.	

Check the limits of applicability of Specification Section K1

1) Strength: $F_y$ less than or equal to 52 ksi for HSS	$F_y = 46$ ksi	<b>o.k.</b>	Section K1.2
2) Ductility: $F_y/F_u \leq 0.8$ for HSS	$F_y/F_u = 0.79$	<b>o.k.</b>	
3) $0.25 < B_p/B < 1.0$	$B_p/B = 0.69$	<b>o.k.</b>	Section
4) $B/t \leq 35$	$B/t = 17$	<b>o.k.</b>	K1.3b

Calculate the required strength

LRFD	ASD
<i>Transverse force from the plate</i> $P_u = 1.2(10.0 \text{ kips}) + 1.6(30.0 \text{ kips})$ $= 60.0 \text{ kips}$	<i>Transverse force from the plate</i> $P_a = 10.0 \text{ kips} + 30.0 \text{ kips}$ $= 40.0 \text{ kips}$
<i>Column axial force</i> $P_r = P_{u \text{ column}}$ $= 1.2(54.0 \text{ kips}) + 1.6(162 \text{ kips})$ $= 324 \text{ kips}$	<i>Column axial force</i> $P_r = P_{a \text{ column}}$ $= 54.0 \text{ kips} + 162 \text{ kips}$ $= 216 \text{ kips}$

Calculate available local yielding strength for uneven load distribution in the loaded plate

$$R_n = \left[ \frac{10F_y t}{\left(\frac{B}{t}\right)} \right] B_p \leq F_{yp} t_p B_p \quad \text{Eqn. K1-2}$$

$$R_n = \left[ \frac{10(46 \text{ ksi})(0.465 \text{ in.})}{\left(\frac{8.00 \text{ in.}}{0.465 \text{ in.}}\right)} \right] 5.50 \text{ in.} \leq 36 \text{ ksi}(0.500 \text{ in.})(5.50 \text{ in.})$$

$$= 68.4 \text{ kips} \leq 99.0 \text{ kips} \quad \mathbf{o.k.}$$

LRFD	ASD
$\phi = 0.95$	$\Omega = 1.58$
$\phi R_n = 0.95(68.4 \text{ kips}) = 65.0 \text{ kips}$	$R_n / \Omega = \frac{68.4 \text{ kips}}{1.58} = 43.3 \text{ kips}$
$65.0 \text{ kips} > 60.0 \text{ kips} \quad \mathbf{o.k.}$	$43.3 \text{ kips} > 40.0 \text{ kips} \quad \mathbf{o.k.}$

Section  
K1.3b

Check shear yielding (punching)

This limit state does not control when  $B_p > B-2t$ , nor when  $B_p < 0.85B$ .

Section  
K1.3b(b)

$$B-2t = 8.00 \text{ in.} - 2(0.500 \text{ in.}) = 7.00 \text{ in.}$$

$$0.85B = 0.85(8.00 \text{ in.}) = 6.80 \text{ in.}$$

Therefore, since  $B_p < 6.80 \text{ in.}$  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.

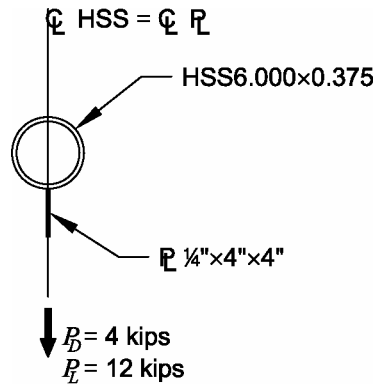
Section  
K1.3b(c)

Therefore, since  $B_p < 6.80 \text{ in.}$  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 tension member subject to the transverse loadings given below, applied through a 4 in. wide plate. The HSS6.000×0.375 is in tension with  $P_D = 4$  kips and  $P_L = 12$  kips.



**Solution:**

**Material Properties:**

Column	ASTM A500 Gr. B	$F_y = 42$ ksi	$F_u = 58$ ksi
Plate	ASTM A36	$F_{yp} = 36$ ksi	$F_u = 58$ ksi

Manual  
Table 2-3  
Table 2-4

**Geometric Properties:**

Column	HSS6.000×0.375	$D = 6.00$ in.	$t = 0.349$ in.
Plate		$N = 4.00$ in.	

Manual  
Table 1-13

Check the limits of applicability of Specification Section K1

- |   |                  |             |              |
|---|------------------|-------------|--------------|
| 1) Strength: $F_y$ less than or equal to 52 ksi for HSS | $F_y = 42$ ksi   | <b>o.k.</b> | Section K1.2 |
| 2) Ductility: $F_y/F_u \leq 0.8$ for HSS                | $F_y/F_u = 0.72$ | <b>o.k.</b> | Section      |
| 3) $D/t \leq 50$ for T-connections                      | $D/t = 17.2$     | <b>o.k.</b> | K1.4a        |

Calculate the required strength

LRFD	ASD
$P_u = 1.2(4.00 \text{ kips}) + 1.6(12.0 \text{ kips})$ $= 24.0 \text{ kips}$	$P_a = 4.00 \text{ kips} + 12.0 \text{ kips}$ $= 16.0 \text{ kips}$

Check the limit state of chord plastification

$$R_n = 5.5F_y t^2 \left( 1 + \frac{0.25N}{D} \right) Q_f \tag{Eqn. K1-8}$$

Since the column is in tension,  $Q_f = 1.0$

$$R_n = 5.5(42 \text{ ksi})(0.349 \text{ in.})^2 \left( 1 + \frac{0.25(4.00 \text{ in.})}{6.00 \text{ in.}} \right) 1.0 = 32.8 \text{ kips} \tag{Eqn. K1-8}$$



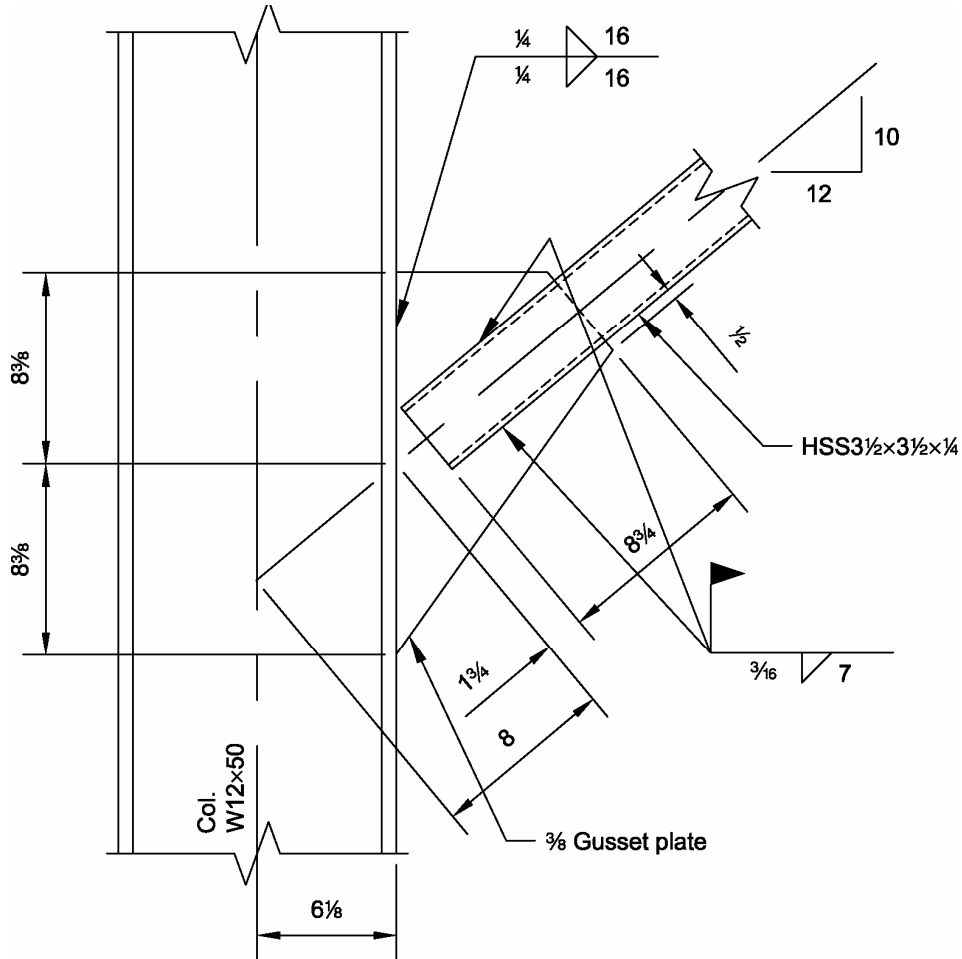
LRFD	ASD
$\phi R_n = 0.90(32.8 \text{ kips}) = 29.5 \text{ kips}$ $29.5 \text{ kips} > 24.0 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = \frac{32.8 \text{ kips}}{1.67} = 19.6 \text{ kips}$ $19.6 \text{ kips} > 16.0 \text{ kips}$ <b>o.k.</b>

Section  
K1.4a

**Example K.10 HSS Brace Connection to a W-shape Column.**

**Given:**

Design the connection shown below. The required axial strength in the brace is 80 kips (LRFD) and 52 kips (ASD). The axial force may be either tension or compression. The length of the brace is 6 ft.



**Solution:**

**Material Properties:**

Brace	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi	
Plate	ASTM A36	$F_{yp} = 36$ ksi	$F_u = 58$ ksi	

Manual  
Table 2-3

**Geometric Properties:**

Brace	HSS3 1/2 x 3 1/2 x 1/4	$A_g = 2.91$ in. <sup>2</sup>	$r = 1.32$ in.	$t = 0.233$ in.	
Plate		$t = 0.375$ in.			

Manual  
Table 1-12

Obtain the available axial compression strength of the brace from Manual Table 4-4

$K = 1.0$   
 $L_b = 6.00 \text{ ft}$

LRFD	ASD
$\phi_c P_n = 98.4 \text{ kips}$ $98.4 \text{ kips} > 80.0 \text{ kips}$ <b>o.k.</b>	$P_n/\Omega_c = 65.4 \text{ kips}$ $65.4 \text{ kips} > 52.0 \text{ kips}$ <b>o.k.</b>

Manual  
Table 4-4

Obtain the available tension yielding strength of the brace from Manual Table 5-5

LRFD	ASD
$\phi_t P_n = 120 \text{ kips}$ $120 \text{ kips} > 80.0 \text{ kips}$ <b>o.k.</b>	$P_n/\Omega_t = 80.2 \text{ kips}$ $80.2 \text{ kips} > 52.0 \text{ kips}$ <b>o.k.</b>

Manual  
Table 5-5

Calculate the tensile rupture strength of the brace

$$A_n = A_g - 2(t_w)(\text{slot}) = 2.91 - 2(0.233)(\frac{3}{8} + \frac{1}{16}) = 2.71 \text{ in.}^2$$

$$\bar{x} = \frac{B^2 + 2BH}{4(B + H)} = \frac{(3 \frac{1}{2} \text{ in.})^2 + 2(3 \frac{1}{2} \text{ in.})(3 \frac{1}{2} \text{ in.})}{4(3 \frac{1}{2} \text{ in.} + 3 \frac{1}{2} \text{ in.})} = 1.31 \text{ in.}$$

Table D3.1

8¾ in. of overlap occurs. Try four ¾-in. fillet welds, each 7-in. long

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.31 \text{ in.}}{7.00 \text{ in.}} = 0.813$$

$$A_e = UA_n = 0.813(2.71 \text{ in.}^2) = 2.20 \text{ in.}^2$$

$$P_n = F_u A_e = 58 \text{ ksi}(2.20 \text{ in.}^2) = 128 \text{ kips}$$

Eqn. D2-2

Calculate the available tension rupture strength

LRFD	ASD
$\phi_t = 0.75$ $\phi_t P_n = 0.75(128 \text{ kips}) = 95.8 \text{ kips}$ $95.8 \text{ kips} > 80.0 \text{ kips}$ <b>o.k.</b>	$\Omega_t = 2.00$ $P_n/\Omega_t = (128 \text{ kips})/2.00 = 63.9 \text{ kips}$ $63.9 \text{ kips} > 52.0 \text{ kips}$ <b>o.k.</b>

Calculate available strength of ¾-in. weld of HSS to plate

LRFD	ASD
$\phi R_n = 4(7 \text{ in.})(3 \text{ sixteenths})(1.392 \text{ kips/in.} / \text{sixteenth})$ $= 117 \text{ kips} > 80.0 \text{ kips}$ <b>o.k.</b>	$\frac{R_n}{\Omega} = 4(7 \text{ in.})(3 \text{ sixteenths})(0.928 \text{ kips/in.} / \text{sixteenth})$ $= 78.0 \text{ kips} > 52.0 \text{ kips}$ <b>o.k.</b>

Calculate the available strength of the weld of gusset plate to column

The slope of the member relative to the longitudinal weld along the column is 50 degrees. Therefore the standard values for longitudinal fillet welds can be increased by:

$$(1.0 + 0.50 \sin^{1.5}\theta) = 1.34$$

or 34-percent.

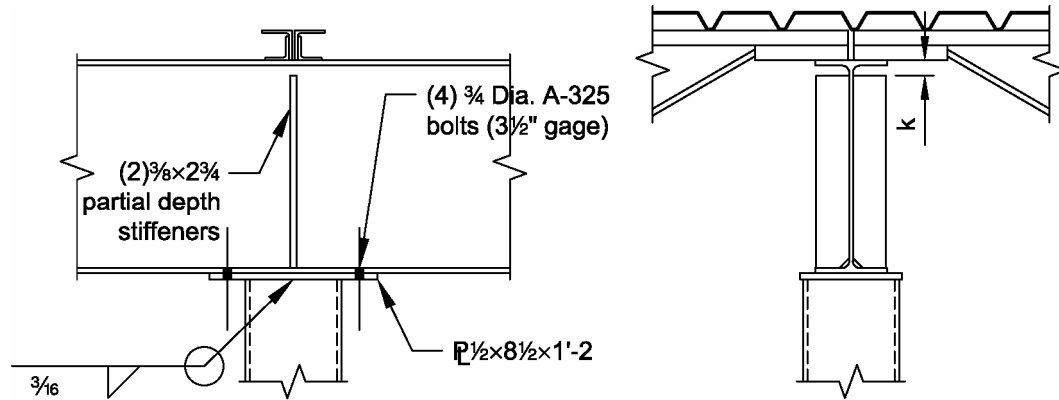
Minimum weld is  $\frac{3}{16}$ -in. fillet each side. Therefore:

LRFD	ASD
$\phi R_n = 2(16.0 \text{ in.})(3)(1.392)(1.34)$ $= 179 \text{ kips} > 80.0 \text{ kips} \quad \mathbf{o.k.}$	$R_n/\Omega = 2(16.0 \text{ in.})(3)(0.928)(1.34)$ $= 119 \text{ kips} > 52.0 \text{ kips} \quad \mathbf{o.k.}$

### Example K.11 Rectangular HSS Column with a Cap Plate, Supporting a Continuous Beam.

**Given:**

Verify the 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 stiffeners shown. The column axial forces are RDL = 24 kips and RLL = 30 kips.



**Solution:**

**Material Properties:**

Beam	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Column	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi	Manual Table 2-3
Cap Plate	ASTM A36	$F_{yp} = 36$ ksi	$F_u = 58$ ksi	

**Geometric Properties:**

Beam	W18x35	$d = 17.7$ in.	$b_f = 6.00$ in.	$t_w = 0.300$ in.	Manual Table 1-1
		$t_f = 0.425$ in.	$k_1 = 0.750$ in.		Manual Table 1-12
Column	HSS8x8x1/4	$t = 0.233$ in.			
Cap Plate		$t = 0.500$ in.			

Calculate the required strength

LRFD	ASD
$R_u = 1.2(24.0 \text{ kips}) + 1.6(30.0 \text{ kips})$ $= 76.8 \text{ kips}$	$R_u = 24.0 \text{ kips} + 30.0 \text{ kips}$ $= 54.0 \text{ kips}$

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 W18x35

Section K1.6

$$N = 2k_1 + 5t_f = 2(0.750 \text{ in.}) + 5(0.425 \text{ in.}) = 3.63 \text{ in.}$$

Check limit for number of HSS faces contributing

$$5t_p + N = 5(0.500 \text{ in.}) + 3.63 \text{ in.} = 6.13 \text{ in.} < 8.00 \text{ in.} \text{ therefore, only 2 walls contribute}$$

Calculate the nominal local wall yielding strength of the HSS

For each of the two walls:

$$R_n = F_y t (5t_p + N) \leq B F_y t$$

$$= 46 \text{ ksi} (0.233 \text{ in.}) (5(0.500 \text{ in.}) + 3.63 \text{ in.}) \leq 8.00 \text{ in.} (46 \text{ ksi}) (0.233 \text{ in.})$$

$$65.7 \text{ kips} \leq 85.7 \text{ kips}$$

Eqn. K1-11

Use  $R_n = 65.7$  kips per wall

Calculate the available local wall yielding strength of the HSS

LRFD	ASD	
$\phi = 1.00$ $\phi R_n = 1.00(65.7 \text{ kips})(2 \text{ walls})$ $= 131 \text{ kips} > 76.8 \text{ kips}$ <b>o.k.</b>	$\Omega = 1.50$ $R_n/\Omega = (65.7 \text{ kips}/1.50)(2 \text{ walls})$ $= 87.6 \text{ kips} > 54.0 \text{ kips}$ <b>o.k.</b>	Section K1.6

Calculate the nominal wall local crippling strength of the HSS

Section K1.6

For each of the two walls:

$$R_n = 0.8t^2 \left[ 1 + (6N/B) \left( t/t_p \right)^{1.5} \right] \left[ E F_y t_p / t \right]^{0.5}$$

$$R_n = 0.8(0.233 \text{ in.})^2 \left[ 1 + \left( \frac{6(3.63 \text{ in.})}{8.00 \text{ in.}} \right) \left( \frac{0.233 \text{ in.}}{0.500 \text{ in.}} \right)^{1.5} \right] \left[ \frac{29,000 \text{ ksi} (46 \text{ ksi}) (0.500 \text{ in.})}{0.233 \text{ in.}} \right]^{0.5}$$

$$= 137 \text{ kips per wall}$$

Eqn. K1-12

Calculate the available local wall local crippling strength of the HSS

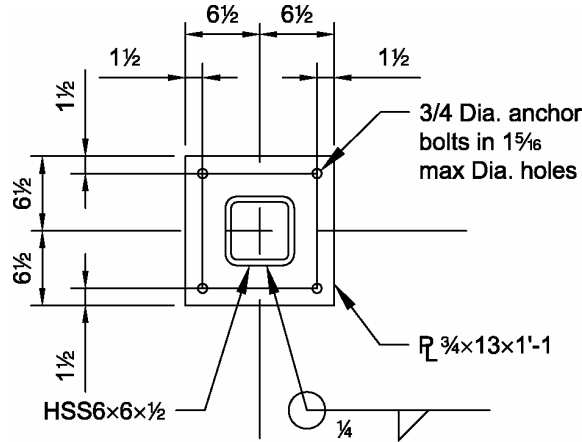
LRFD	ASD	
$\phi = 0.75$ $\phi R_n = 0.75(137 \text{ kips})(2 \text{ walls})$ $= 207 \text{ kips} > 76.8 \text{ kips}$ <b>o.k.</b>	$\Omega = 2.00$ $R_n/\Omega = (137 \text{ kips}/2.00)(2 \text{ walls})$ $= 137 \text{ kips} > 54.0 \text{ kips}$ <b>o.k.</b>	Section K1.6

The presence of the stiffeners eliminates the need to check the beam web for local web yielding and local web crippling.

**Example K.12 Rectangular HSS Column Base Plate.**

**Given:**

A HSS 6×6×½ column is supporting nominal loads of 40 kips from dead load and 120 kips from live load. The column is supported by a 7'-6" × 7'-6" concrete spread footing with a  $f'_c = 3000$  psi. Size a base plate for this column.



**Solution:**

**Material Properties:**

Column	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi
Base Plate	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi

Manual  
Table 2-3

**Geometric Properties:**

HSS 6×6×½	$B = H = 6.00$ in.
-----------	--------------------

Calculate the required strength

LRFD	ASD
$P_u = 1.2(40.0 \text{ kips}) + 1.6(120 \text{ kips})$ $= 240 \text{ kips}$	$P_a = 40.0 \text{ kips} + 120 \text{ kips}$ $= 160 \text{ kips}$

Note: The procedure illustrated here is similar to that presented in *AISC Design Guide 1-Column Base Plates*.

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

$$P_p = 0.85f'_c A_1 \sqrt{A_2/A_1} \text{ where } A_2/A_1 \leq 4$$

Eqn. J8-2

$$A_1 = (13.0 \text{ in.})(13.0 \text{ in.}) = 169 \text{ in.}^2$$

$$A_2 = (90.0 \text{ in.})(90.0 \text{ in.}) = 8100 \text{ in.}^2$$

$$P_p = 0.85(3 \text{ ksi})(169 \text{ in.}^2) \sqrt{\frac{8100 \text{ in.}^2}{169 \text{ in.}^2}} \leq 1.7(3 \text{ ksi})(169 \text{ in.}^2)$$

$$20,700 \text{ kips} \leq 862 \text{ kips}$$

Use  $P_p = 862 \text{ kips}$

Note: The limit on the right side of Equation J8-2 will control when  $A_2/A_1$  exceeds 4.0

LRFD	ASD
$\phi_c = 0.60$ $\phi_c P_p = 0.60(862 \text{ kips})$ $= 517 \text{ kips} > 240 \text{ kips}$	$\Omega_c = 2.50$ $P_p/\Omega_c = 862 \text{ kips}/2.50$ $= 345 \text{ kips} > 160 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

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.

Note: When the HSS is large compared to the edge extension, a quick and conservative approach is to include in the bearing area only the surface within the distance  $m$  or  $n$  of the HSS wall within the shape.

$$m = n = \frac{N - 0.95(\text{outside dimension})}{2}$$

$$= \frac{13.0 \text{ in.} - 0.95(6.00 \text{ in.})}{2} = 3.65 \text{ in.} > \frac{[6 - 2(\frac{1}{2})]}{2} = 2.50 \text{ in.}, \text{ therefore, the full area of the baseplate inside the HSS is effective in bearing.}$$

The critical bending moment is the cantilever moment outside the HSS perimeter.

LRFD	ASD
$f_{pu} = \frac{P_u}{A_{eff}} = \frac{240 \text{ kips}}{169 \text{ in.}^2} = 1.42 \text{ ksi}$	$f_{pa} = \frac{P_a}{A_{eff}} = \frac{160 \text{ kips}}{169 \text{ in.}^2} = 0.947 \text{ ksi}$
$M_u = \frac{f_{pu} l^2}{2}$	$M_a = \frac{f_{pa} l^2}{2}$
$Z = \frac{t_p^2}{4}$	$Z = \frac{t_p^2}{4}$
$\phi_b = 0.90$	$\Omega_b = 1.67$
$M_n = M_p = F_y Z$	$M_n = M_p = F_y Z$
Equating:	Equating:
$M_u = \phi_b M_n$ and solving for $t_p$ gives: $t_{p(req)} = \sqrt{\frac{2 f_{pu} l^2}{\phi_b F_y}}$ $= \sqrt{\frac{2(1.42 \text{ ksi})(3.65 \text{ in.})^2}{0.90(36 \text{ ksi})}} = 1.08 \text{ in.}$	$M_a = M_n/\Omega_b$ and solving for $t_p$ gives: $t_{p(req)} = \sqrt{\frac{2 f_{pa} l^2}{F_y / \Omega_b}}$ $= \sqrt{\frac{2(0.947 \text{ ksi})(3.65 \text{ in.})^2}{(36 \text{ ksi})/1.67}} = 1.08 \text{ in.}$

Eqn. F11-1

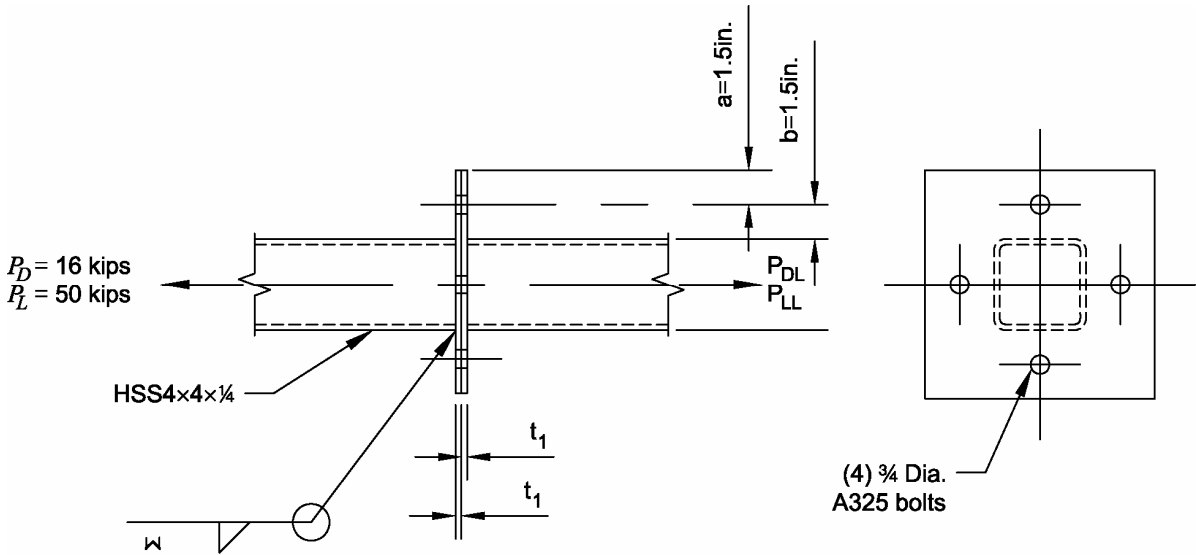
Therefore, use a 1¼ in. thick plate.



### Example K.13 Rectangular HSS Strut End Plate.

**Given:**

Determine 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.



**Solution:**

**Material Properties:**

Strut	ASTM A500 Gr. B	$F_y = 46$ ksi	$F_u = 58$ ksi	
End Plate	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	

Manual  
Table 2-3

**Geometric Properties:**

Strut	HSS 4×4×¼	$t = 0.233$ in.	$A = 3.37$ in. <sup>2</sup>	
End Plate		$b = a = 1.50$ in.		

Manual  
Table 1-12

Calculate the required tensile strength

LRFD	ASD
$P_u = 1.2(16.0 \text{ kips}) + 1.6(50.0 \text{ kips})$ $= 99.2 \text{ kips}$	$P_a = 16.0 \text{ kips} + 50.0 \text{ kips}$ $= 66.0 \text{ kips}$

Preliminary size of the (4) ASTM A325 bolts

LRFD	ASD
$r_{ut} = \frac{P_u}{n} = \frac{99.2 \text{ kips}}{4} = 24.8 \text{ kips}$ Try ¾ -in. diameter ASTM A325 bolts $\phi r_n = 29.8 \text{ kips}$	$r_{at} = \frac{P_a}{n} = \frac{66.0 \text{ kips}}{4} = 16.5 \text{ kips}$ Try ¾ -in. diameter ASTM A325 bolts $r_n / \Omega = 19.9 \text{ kips}$

Manual  
Table 7-2

Calculate required end-plate thickness with consideration of prying action

$$a' = a + \frac{d_b}{2} \leq 1.25b + \frac{d_b}{2}$$

$$= 1.50 \text{ in.} + \frac{3/4}{2} \leq 1.25(1.50 \text{ in.}) + \frac{3/4}{2}$$

$$= 1.88 \text{ in.} \leq 2.25 \text{ in.} \quad \mathbf{o.k.}$$

$$b' = b - \frac{d_b}{2} = 1.5 \text{ in.} - \frac{3/4}{2} = 1.12 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.12}{1.88} = 0.596$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{(\frac{3}{4} + \frac{1}{8})}{4.0} = 0.781$$

Manual  
Section 9

LRFD	ASD
$\beta = \frac{1}{\rho} \left( \frac{\phi r_n}{r_u} - 1 \right) = \frac{1}{0.596} \left( \frac{29.8}{24.8} - 1 \right) = 0.338$	$\beta = \frac{1}{\rho} \left( \frac{r_n / \Omega}{r_u} - 1 \right) = \frac{1}{0.596} \left( \frac{19.9}{16.5} - 1 \right) = 0.346$
$\alpha' = \frac{1}{\delta} \left( \frac{\beta}{1 - \beta} \right)$	$\alpha' = \frac{1}{\delta} \left( \frac{\beta}{1 - \beta} \right)$
$= \frac{1}{0.781} \left( \frac{0.338}{1 - 0.338} \right) = 0.654 \leq 1.0$	$= \frac{1}{0.781} \left( \frac{0.346}{1 - 0.346} \right) = 0.677 \leq 1.0$

Manual  
Section 9

LRFD	ASD
$t_{req} = \sqrt{\frac{4.44 r_u b'}{p F_u (1 + \delta \alpha')}} = \sqrt{\frac{4.44 (24.8 \text{ kips}) (1.12 \text{ in.})}{4.00 \text{ in.} (58 \text{ ksi}) (1 + 0.781 (0.654))}} = 0.593 \text{ in.}$	$t_{req} = \sqrt{\frac{6.66 r_u b'}{p F_u (1 + \delta \alpha')}} = \sqrt{\frac{6.66 (16.5 \text{ kips}) (1.12 \text{ in.})}{4.00 \text{ in.} (58 \text{ ksi}) (1 + 0.781 (0.677))}} = 0.589 \text{ in.}$
Use 5/8 in. end plate, $t_1 > 0.593$ , further bolt check for prying not required.	Use 5/8 in. end plate, $t_1 > 0.589$ , further bolt check for prying not required.
Use (4) 3/4 in. diameter A325 bolts	Use (4) 3/4 in. diameter A325 bolts

Manual  
Section 9

Calculate the weld size required

$$F_w = 0.60 F_{ext} (1.0 + 0.5 \sin^{1.5}(\theta)) = 0.60(70 \text{ ksi})(1.0 + 0.5 \sin^{1.5}(90)) = 63.0 \text{ ksi}$$

Eqn. J2-5

$$l = 4(4.00 \text{ in.}) = 16.0 \text{ in.}$$

LRFD	ASD
For shear and fillet welds $\phi = 0.75$	For shear and fillet welds $\Omega = 2.00$
$w \geq \frac{P_u}{\phi F_w (0.707) l}$	$w \geq \frac{\Omega P_u}{F_w (0.707) l}$
$\geq \frac{99.2 \text{ kips}}{0.75 (63.0 \text{ ksi}) (0.707) (16.0 \text{ in.})}$	$\geq \frac{2.00 (66.0 \text{ kips})}{(63.0 \text{ ksi}) (0.707) (16.0 \text{ in.})}$
$= 0.186 \text{ in.}$	$= 0.185 \text{ in.}$

Manual  
Section 8

*Check minimum weld size requirements*For  $t = \frac{5}{8}$  in. minimum weld =  $\frac{1}{4}$  in.

Table J2.4

LRFD	ASD
0.250 in. > 0.186 in. Use $\frac{1}{4}$ in. weld leg size	0.250 in. > 0.185 in. Use $\frac{1}{4}$ in. weld leg size

*Results*Use  $\frac{1}{4}$  in. weld with  $\frac{5}{8}$  in. end plate and (4)  $\frac{3}{4}$ -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

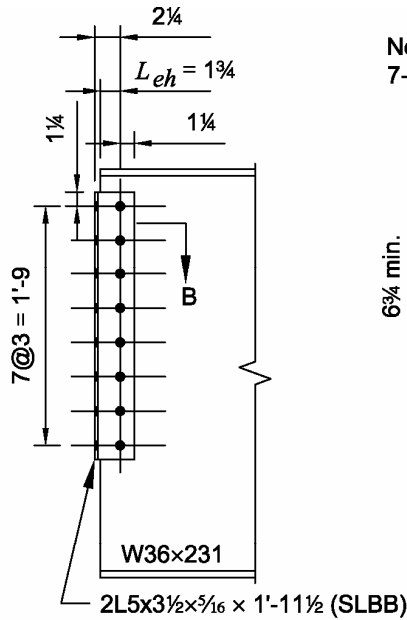
**Given:**

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:

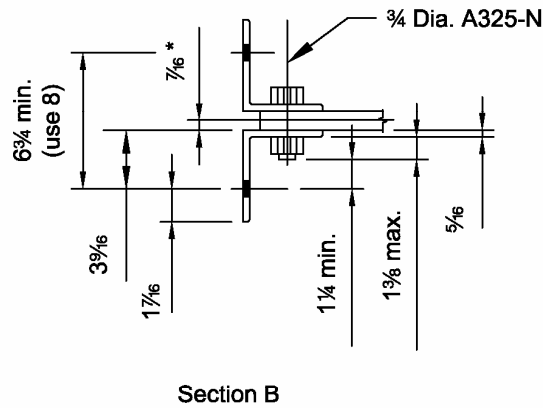
$$R_D = 37.5 \text{ kips}$$

$$R_L = 112.5 \text{ kips}$$

Use 3/4-in. diameter ASTM A325-N bolts in standard holes.



Note: The given dimensions are from Table 7-18, entering and tightening clearances.



\* 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 = 0.1775; 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.

**Material Properties:**

Beam	W36×231	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Table 2-3
Column	W14×90	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Angles	2L5×3 1/2	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

**Geometric Properties:**

Beam	W36×231	$t_w = 0.760 \text{ in.}$	Manual Table 1-1
Column	W14×90	$t_f = 0.710 \text{ in.}$	
Angles	2L5×3 1/2 SLBB		

**Solution:**

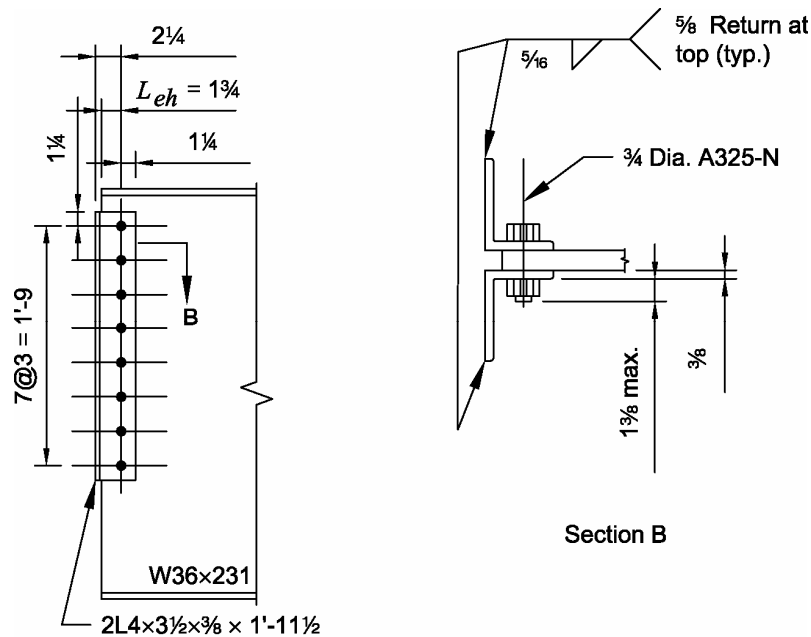
LRFD	ASD	
$R_u = 1.2(37.5 \text{ kips}) + 1.6(112.5 \text{ kips})$ $= 225 \text{ kips}$  <i>Manual Table 10-1 includes checks for the limit states of bearing, shear yielding, shear rupture, and block shear rupture on the angles, and shear on the bolts.</i>  Try 8 rows of bolts and $\frac{5}{16}$ -in. angle thickness.  $\phi R_n = 247 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	$R_a = 37.5 \text{ kips} + 112.5 \text{ kips}$ $= 150 \text{ kips}$  <i>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.</i>  Try 8 rows of bolts and $\frac{5}{16}$ -in. angle thickness.  $R_n / \Omega = 165 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1
<i>Check the beam web for bolt bearing. Block shear rupture, shear yielding and shear rupture will not control, since the beam is uncoped.</i>  Uncoped, $L_{eh} = 1\frac{3}{4} \text{ in.}$  $\phi R_n = (702 \text{ kips/in.})(0.760 \text{ in.})$ $= 534 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	<i>Check the beam web for bolt bearing. Block shear rupture, shear yielding and shear rupture will not control, since the beam is uncoped.</i>  Uncoped, $L_{eh} = 1\frac{3}{4} \text{ in.}$  $R_n / \Omega = (468 \text{ kips/in.})(0.760 \text{ in.})$ $= 356 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1
<i>Check supporting member flange for bolt bearing</i>  $\phi R_n = (1400 \text{ kips/in.})(0.710 \text{ in.})$ $= 994 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	<i>Check supporting member flange for bolt bearing</i>  $R_n / \Omega = (936 \text{ kips/in.})(0.710 \text{ in.})$ $= 665 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1

See **Example II.A-2** for a bolted/welded double angle connection.

## Example II.A-2 Bolted/Welded Double-Angle Connection

### Given:

Use Manual Table 10-2 to substitute welds for bolts in the support legs of the double-angle connection (welds B). Use 70 ksi electrodes.



### Material Properties:

Beam	W36x231	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual Table 2-3
Column	W14x90	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Angles	2L4x3 1/2	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	

### Geometric Properties:

Beam	W36x231	$t_w = 0.760$ in.	Manual Table 1-1
Column	W14x90	$t_f = 0.710$ in.	
Angles	2L4x3 1/2	SLBB	

**Solution:**

LRFD	ASD	
$R_u = 1.2(37.5 \text{ kips}) + 1.6(112.5 \text{ kips})$ $= 225 \text{ kips}$ <i>Design welds (welds B)</i> Try $\frac{5}{16}$ -in. weld size, $L = 23\frac{1}{2}$ in. $t_{f \min} = 0.238 \text{ in.} < 0.710 \text{ in.}$ <b>o.k.</b> $\phi R_n = 279 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	$R_a = 37.5 \text{ kips} + 112.5 \text{ kips}$ $= 150 \text{ kips}$ <i>Design welds (welds B)</i> Try $\frac{5}{16}$ -in. weld size, $L = 23\frac{1}{2}$ in. $t_{f \min} = 0.238 \text{ in.} < 0.710 \text{ in.}$ <b>o.k.</b> $R_n / \Omega = 186 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-2
<i>Check minimum angle thickness</i> $t_{\min} = w + \frac{1}{16}\text{-in.} \geq \frac{5}{16} \text{ in.}$ $= \frac{5}{16} \text{ in.} + \frac{1}{16} \text{ in.}$ $= \frac{3}{8} \text{ in.} > \frac{5}{16} \text{ in.}$ <b>o.k.</b> Use $2L4 \times 3\frac{1}{2} \times \frac{3}{8}$	<i>Check minimum angle thickness</i> $t_{\min} = w + \frac{1}{16}\text{-in.} \geq \frac{5}{16} \text{ in.}$ $= \frac{5}{16} \text{ in.} + \frac{1}{16} \text{ in.}$ $= \frac{3}{8} \text{ in.} > \frac{5}{16} \text{ in.}$ <b>o.k.</b> Use $2L4 \times 3\frac{1}{2} \times \frac{3}{8}$	Section J2.2b
<i>Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture</i> Check 8 rows of bolts and $\frac{3}{8}$ in. angle thickness. $\phi R_n = 254 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	<i>Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture</i> Check 8 rows of bolts and $\frac{3}{8}$ in. angle thickness. $R_n / \Omega = 170 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1
<i>Check beam web for bolt bearing.</i> Uncoped, $L_{eh} = 1\frac{3}{4}$ in. $\phi R_n = (702 \text{ kips/in.})(0.760 \text{ in.})$ $= 534 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	<i>Check beam web for bolt bearing.</i> Uncoped, $L_{eh} = 1\frac{3}{4}$ in. $R_n / \Omega = (468 \text{ kips/in.})(0.760 \text{ in.})$ $= 356 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1
<i>Check supporting member flange for bolt bearing</i> $\phi R_n = (1400 \text{ kips/in.})(0.710 \text{ in.})$ $= 994 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	<i>Check supporting member flange for bolt bearing</i> $R_n / \Omega = (936 \text{ kips/in.})(0.710 \text{ in.})$ $= 665 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	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 can not 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).

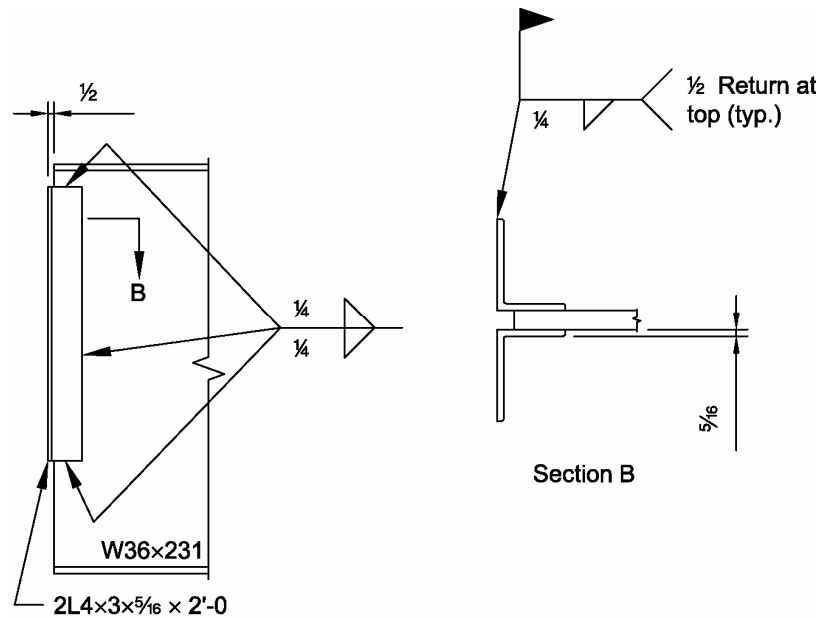


### Example II.A-3 All-Welded Double-Angle Connection

#### Given:

Use 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 2L4×3½	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	

#### Geometric Properties:

Beam W36×231	$t_w = 0.760$ in.	Manual Tables 1-1 and 1-15
Column W14×90	$t_f = 0.710$ in.	
Angles 2L4×3½ SLBB		

**Solution:**

LRFD	ASD	
$R_u = 1.2(37.5 \text{ kips}) + 1.6(112.5 \text{ kips})$ $= 225 \text{ kips}$ <i>Design the weld between the beam-web and the angle leg (welds A)</i> Try $\frac{1}{4}$ in. weld size, $L = 24$ in. $t_{w \min} = 0.381 \text{ in.} < 0.760 \text{ in.}$ <b>o.k.</b> $\phi R_n = 343 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	$R_a = 37.5 \text{ kips} + 112.5 \text{ kips}$ $= 150 \text{ kips}$ <i>Design the weld between the beam-web and the angle leg (welds A)</i> Try $\frac{1}{4}$ in. weld size, $L = 24$ in. $t_{w \min} = 0.381 \text{ in.} < 0.760 \text{ in.}$ <b>o.k.</b> $R_n / \Omega = 229 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-3
<i>Design the welds between support and the angle leg (welds B)</i> Try $\frac{1}{4}$ in. weld size $t_{f \min} = 0.190 \text{ in.} < 0.710 \text{ in.}$ <b>o.k.</b> $\phi R_n = 229 \text{ kips} > 225 \text{ kips}$ <b>o.k.</b>	<i>Design the welds between support and the angle leg (welds B)</i> Try $\frac{1}{4}$ in. weld size $t_{f \min} = 0.190 \text{ in.} < 0.710 \text{ in.}$ <b>o.k.</b> $R_n / \Omega = 153 \text{ kips} > 150 \text{ kips}$ <b>o.k.</b>	Manual Table 10-3
<i>Check the minimum angle thickness</i> $t_{\min} = w + \frac{1}{16} \text{ in.} \geq \frac{1}{8} \text{ in.}$ $= \frac{1}{4} \text{ in.} + \frac{1}{16} \text{ in.}$ $= \frac{5}{16} \text{ in.} \geq \frac{1}{8} \text{ in.}$ <b>o.k.</b> Use $2L4 \times 3 \times \frac{5}{16}$	<i>Check the minimum angle thickness</i> $t_{\min} = w + \frac{1}{16} \text{ in.} \geq \frac{1}{8} \text{ in.}$ $= \frac{1}{4} \text{ in.} + \frac{1}{16} \text{ in.}$ $= \frac{5}{16} \text{ in.} \geq \frac{1}{8} \text{ in.}$ <b>o.k.</b> Use $2L4 \times 3 \times \frac{5}{16}$	Section J2.2b

**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

Given:

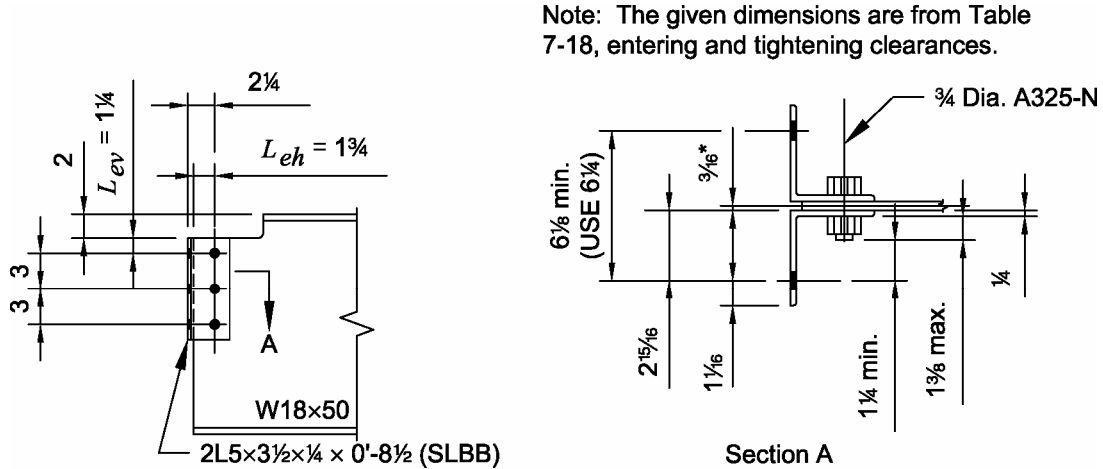
Use Manual Table 10-1 to select an all-bolted double-angle connection between a W18×50 beam and a W21×62 girder web to support the following beam end reactions:

$$R_D = 10 \text{ kips}$$

$$R_L = 30 \text{ kips}$$

The beam top flange is coped 2-in. deep by 4-in. long,  $L_{ev} = 1\frac{1}{4}$  in.,  $L_{eh} = 1\frac{3}{4}$  in. (assumed to be 1½ in. for calculation purposes to account for possible underrun in beam length)

Use ¾-in. diameter ASTM A325-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	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Table 2-3
Girder W21×62	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Angles 2L4×3½	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

#### Geometric Properties:

Beam W18×50	$d = 18.0 \text{ in.}$	$t_w = 0.355 \text{ in.}$	$S_{net} = 23.4 \text{ in.}^3$	Manual Tables 1-1 and 9-2
Cope	$c = 4 \text{ in.}$	$d_c = 2 \text{ in.}$	$e = 4 \text{ in.} + \frac{1}{2} \text{ in.} = 4\frac{1}{2} \text{ in.}$	
	$h_0 = 16.0 \text{ in.}$			
Girder W21×62	$t_w = 0.400 \text{ in.}$			
Angles 2L4×3 ½ SLBB				

**Solution:**

LRFD	ASD	
$R_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips}) = 60 \text{ kips}$  <i>Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture and block shear rupture</i>  Try 3 rows of bolts and ¼ in. angle thickness  $\phi R_n = 76.4 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	$R_a = 10 \text{ kips} + 30 \text{ kips} = 40 \text{ kips}$  <i>Check bolt shear. Check angles for bolt bearing, shear yielding, shear rupture and block shear rupture</i>  Try 3 rows of bolts and ¼ in. angle thickness  $R_n / \Omega = 50.9 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1
<i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i>  <i>Check beam web for bolt bearing</i>  Top flange coped, $L_{ev} = 1\frac{1}{4} \text{ in.}$ , $L_{eh} = 1\frac{3}{4} \text{ in.}$  $\phi R_n = (200 \text{ kips/in.})(0.355 \text{ in.})$  $= 71.0 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	<i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i>  <i>Check beam web for bolt bearing</i>  Top flange coped, $L_{ev} = 1\frac{1}{4} \text{ in.}$ , $L_{eh} = 1\frac{3}{4} \text{ in.}$  $R_n / \Omega = (133 \text{ kips/in.})(0.355 \text{ in.})$  $= 47.2 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1
<i>Check supporting member web for bolt bearing</i>  $\phi R_n = (526 \text{ kips/in.})(0.400)$  $= 210 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	<i>Check supporting member web for bolt bearing</i>  $R_n / \Omega = (351 \text{ kips/in.})(0.400 \text{ in.})$  $= 140 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1
<i>Check block shear rupture</i>  Since one vertical row of bolts is used, the tension stress can be taken as uniform, therefore  $U_{bs} = 1.0$  $\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$  Tension Component  $\phi F_u A_{nt} = 51.8 \text{ kips/in. (0.355 in.)}$  Shear Yielding Component  $\phi 0.6 F_y A_{gv} = 163 \text{ kips/in. (0.355 in.)}$  Shear Rupture Component  $\phi 0.6 F_u A_{nv} = 148 \text{ kips/in. (0.355 in.)}$	<i>Check block shear rupture</i>  Since one vertical row of bolts is used, the tension stress can be taken as uniform, therefore  $U_{bs} = 1.0$  $\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$  Tension Component  $F_u A_{nt} / \Omega = 34.5 \text{ kips/in. (0.355 in.)}$  Shear Yielding Component  $0.6 F_y A_{gv} / \Omega = 109 \text{ kips/in. (0.355 in.)}$  Shear Rupture Component  $0.6 F_u A_{nv} / \Omega = 98.7 \text{ kips/in. (0.355 in.)}$	Commentary Section J4.3   Eqn. J4-5   Manual Tables 9-3a   Manual Table 9-3b   Manual Table 9-3c

$\phi R_n = (148 \text{ kips/in.} + 51.8 \text{ kips/in.})(0.355 \text{ in.})$ $= 70.9 \text{ kips} < 76.3 \text{ kips}$ $70.9 \text{ kips} > 60 \text{ kips} \quad \mathbf{o.k.}$	$\frac{R_n}{\Omega} = (98.7 \text{ kips/in.} + 34.5 \text{ kips/in.})(0.355 \text{ in.})$ $= 47.3 \text{ kips} < 50.9 \text{ kips}$ $47.3 \text{ kips} > 40 \text{ kips} \quad \mathbf{o.k.}$
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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 are required for values of  $L_{ev}$  and  $L_{eh}$  beyond the limits of Table 10-1. For coped members, the limit states of flexural rupture and local buckling must be checked independently, per Part 9.

Manual  
Table 10-1

Check flexural rupture on the coped section

$$S_{net} = 23.4 \text{ in}^3$$

Manual  
Table 9-2

$$R_n = \frac{F_u S_{net}}{e} = \frac{(65 \text{ ksi})(23.4 \text{ in.}^3)}{4 \frac{1}{2} \text{ in.}} = 338 \text{ kips}$$

Manual  
Part 9

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(338 \text{ kips})$ $= 254 \text{ kips} > 60 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $R_n / \Omega = \frac{(338 \text{ kips})}{2.00}$ $= 169 \text{ kips} > 40 \text{ kips} \quad \mathbf{o.k.}$

Check local web buckling at the coped section

$$\text{Verify } c \leq 2d \text{ and } d_c \leq \frac{d}{2}$$

Manual  
Part 9

$$c = 4.0 \text{ in.} < 2(18.0 \text{ in.}) = 36.0 \text{ in.} \quad \mathbf{o.k.}$$

$$d_c = 2.0 \text{ in.} < \frac{18.0 \text{ in.}}{2} = 9.0 \text{ in.} \quad \mathbf{o.k.}$$

$$\frac{c}{d} = \frac{4 \text{ in.}}{18.0 \text{ in.}} = 0.222; \quad \frac{c}{h_0} = \frac{4 \text{ in.}}{16.0 \text{ in.}} = 0.250$$

$$\text{Since } \frac{c}{d} \leq 1.0,$$

$$f = 2 \left( \frac{c}{d} \right) = 2(0.222) = 0.444$$

Since  $\frac{c}{h_0} \leq 1.0$ ,

$$k = 2.2 \left( \frac{h_0}{c} \right)^{1.65} = 2.2 \left( \frac{16.0}{4.00} \right)^{1.65} = 21.7$$

$$F_{cr} = 26,210 \left( \frac{t_w}{h_0} \right)^2 f_k = 26,210 \left( \frac{0.355 \text{ in.}}{16.0 \text{ in.}} \right)^2 (0.444)(21.7) = 124 \text{ ksi} \leq F_y$$

Manual  
Part 9

$$R_n = \frac{F_{cr} S_{net}}{e} = \frac{(50 \text{ ksi})(23.4 \text{ in.}^3)}{4 \frac{1}{2} \text{ in.}} = 260 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = 0.90(260 \text{ kips})$	$R_n / \Omega = \frac{260 \text{ kips}}{1.67}$
$= 234 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	$= 156 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>

Section F1

*Check shear yielding on beam web*

$$R_n = 0.6 F_y A_g = 0.6(50 \text{ ksi})(0.355 \text{ in})(16.0 \text{ in}) = 170 \text{ kips}$$

Eqn J4-3

LRFD	ASD
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 1.00(170 \text{ kips})$	$R_n / \Omega = \frac{170 \text{ kips}}{1.50}$
$= 170 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	$= 113 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>

Section J4.2

*Check shear rupture on beam web*

$$A_{nv} = t_w [h_o - 3(0.875 \text{ in.})] = (0.355 \text{ in.})(16.0 \text{ in.} - 2.63 \text{ in.}) = 4.75 \text{ in.}^2$$

Equation  
J4-4

$$R_n = 0.6 F_u A_{nv} = 0.6(65 \text{ ksi})(4.75 \text{ in.}^2) = 185 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(185 \text{ kips})$	$R_n / \Omega = \frac{185 \text{ kips}}{2.00}$
$= 139 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	$= 92.6 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>

Section J4.2

Note: see **Example II.A-5** for a bolted/welded double-angle connection.

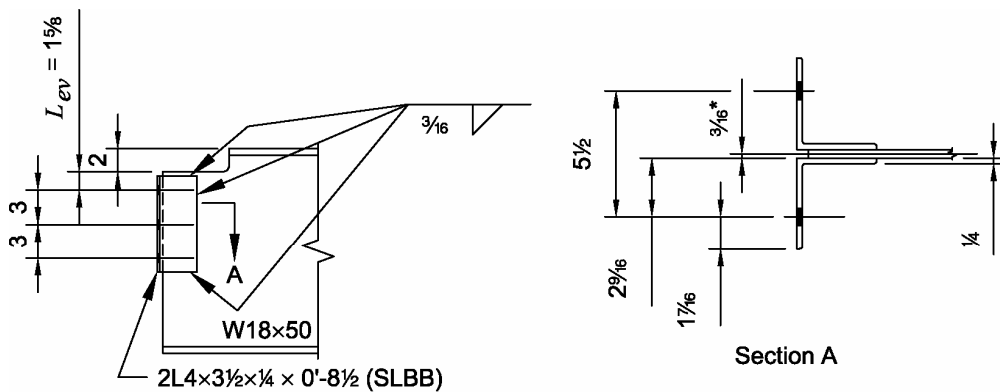
### Example II.A-5 Bolted/Welded Double-Angle Connection (beam-to-girder web).

#### Given:

Use 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 3/4-in. diameter ASTM A325-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 W18x50	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual Table 2-3
Girder W21x62	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Angles 2L4x3 1/2	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	

#### Geometric Properties:

Beam W18x50	$d = 18.0$ in.	$t_w = 0.355$ in.	$S_{net} = 23.4$ in. <sup>3</sup>	Manual Tables 1-1, 1-15, and 9-2
Cope	$c = 4$ in.	$d_c = 2$ in.	$e = 4$ in. + 1/2 in. = 4 1/2 in.	
	$h_0 = 16.0$ in.			
Girder W21x62	$t_w = 0.400$ in.			
Angles 2L4x3 1/2 SLBB				

**Solution:**

LRFD	ASD
$R_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips}) = 60.0 \text{ kips}$ <i>Design welds (welds A)</i> Try $\frac{3}{16}$ -in. weld size, $L = 8\frac{1}{2}$ in. $t_{w \text{ min}} = 0.286 \text{ in.} < 0.355 \text{ in.}$ <b>o.k.</b> $\phi R_n = 110 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	$R_a = 10 \text{ kips} + 30 \text{ kips} = 40.0 \text{ kips}$ <i>Design welds (welds A)</i> Try $\frac{3}{16}$ -in. weld size, $L = 8\frac{1}{2}$ in. $t_{w \text{ min}} = 0.286 \text{ in.} < 0.355 \text{ in.}$ <b>o.k.</b> $R_n / \Omega = 73.4 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>
<i>Check minimum angle thickness</i> $w = \text{weld size}$ $t_{\text{min}} = w + \frac{1}{16}\text{-in.} \geq \frac{1}{4} \text{ in.}$ $= \frac{3}{16} \text{ in.} + \frac{1}{16} \text{ in.}$ $= \frac{1}{4} \text{ in.} \geq \frac{1}{4} \text{ in.}$ <b>o.k.</b>	<i>Check minimum angle thickness</i> $w = \text{weld size}$ $t_{\text{min}} = w + \frac{1}{16}\text{-in.} \frac{1}{4} \text{ in.}$ $= \frac{3}{16} \text{ in.} + \frac{1}{16} \text{ in.}$ $= \frac{1}{4} \text{ in.} \geq \frac{1}{4} \text{ in.}$ <b>o.k.</b>

Manual  
Table 10-2

Section J2.2b

Manual  
Table 10-1

Note: see **Example II.A-4** for an all-bolted double-angle connection (beam-to-girder web).

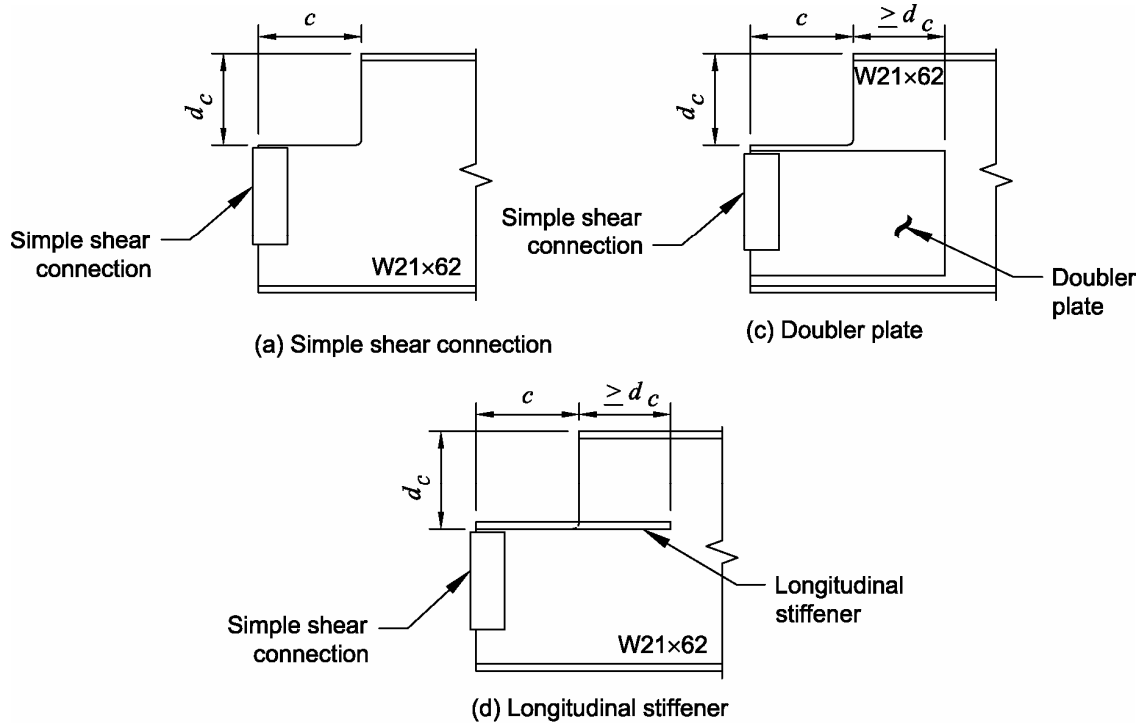


### Example II.A-6 Beam End Coped at the Top Flange Only

#### Given:

For a W21×62 coped 8-in. deep by 9-in. long at the top flange only:

- calculate the available strength of the beam end, considering the limit states of flexural rupture and local buckling. Assume  $e = 9\frac{1}{2}$  in.
- 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.
- determine the size of doubler plate needed to stiffen the W21×62 for the given end reaction.
- 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 Tables 2-3 and 2-4
Plate	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	

#### Geometric Properties:

Beam W21×62	$d = 21.0$ in.	$t_w = 0.400$ in.	$S_{net} = 17.8$ in. <sup>3</sup>	Manual Tables 1-1 and 9-2
Cope	$c = 9$ in.	$d_c = 8$ in.	$e = 9$ in. + $\frac{1}{2}$ in. = $9\frac{1}{2}$ in.	
	$h_0 = 13.0$ in.			

**Solution A:**

Check flexural rupture strength of the coped section

$$S_{net} = 17.8 \text{ in}^3$$

$$R_n = \frac{F_u S_{net}}{e} = \frac{(65 \text{ ksi})(17.8 \text{ in}^3)}{9\frac{1}{2} \text{ in.}} = 122 \text{ kips}$$

Manual  
Table 9-2

Manual  
Part 9

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(122 \text{ kips}) = 91.3 \text{ kips}$	$R_n / \Omega = \frac{122 \text{ kips}}{2.00} = 61 \text{ kips}$

Section F1

Check local buckling

Verify parameters

Manual  
Part 9

$$c \leq 2d$$

$$c = 9.0 \text{ in.} < 2(21.0 \text{ in.}) = 42.0 \text{ in.} \quad \text{o.k.}$$

$$d_c < d/2$$

$$d_c = 8.0 \text{ in.} < \frac{21.0 \text{ in.}}{2} = 10.5 \text{ in.} \quad \text{o.k.}$$

$$\frac{c}{d} = \frac{9 \text{ in.}}{21.0 \text{ in.}} = 0.429; \quad \frac{c}{h_0} = \frac{9 \text{ in.}}{13.0 \text{ in.}} = 0.693$$

$$\text{Since } \frac{c}{d} \leq 1.0,$$

$$f = 2\left(\frac{c}{d}\right) = 2(0.429) = 0.858$$

$$\text{Since } \frac{c}{h_0} \leq 1.0,$$

$$k = 2.2\left(\frac{h_0}{c}\right)^{1.65} = 2.2\left(\frac{13.0 \text{ in.}}{9.0 \text{ in.}}\right)^{1.65} = 4.03$$

For a top cope only, the critical buckling stress is

$$\begin{aligned} F_{cr} &= 26,210 \left(\frac{t_w}{h_0}\right)^2 f k \leq F_y \\ &= 26,210 \left(\frac{0.400 \text{ in.}}{13.0 \text{ in.}}\right)^2 (0.858)(4.03) \leq 50 \text{ ksi} \\ &= 85.8 \text{ ksi} \leq 50 \text{ ksi} \end{aligned}$$

Manual  
Part 9

Use  $F_{cr} = 50$  ksi

$$R_n = \frac{F_{cr} S_{net}}{e} = \frac{(50 \text{ ksi})(17.8 \text{ in.}^3)}{9 \frac{1}{2} \text{ in.}} = 93.7 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = 0.90(93.7 \text{ kips}) = 84.3 \text{ kips}$	$R_n / \Omega = \frac{93.7 \text{ kips}}{1.67} = 56.1 \text{ kips}$

Section F1

*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
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 1.00(156 \text{ kips}) = 156 \text{ kips}$	$R_n / \Omega = \frac{156 \text{ kips}}{1.50} = 104 \text{ kips}$

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
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(203 \text{ kips}) = 203 \text{ kips}$	$R_n / \Omega = \frac{203 \text{ kips}}{2.00} = 101 \text{ kips}$

Section J4.2

Thus, the available strength is controlled by local buckling, with

LRFD	ASD
$\phi R_n = 84.3 \text{ kips.}$	$R_n / \Omega = 56.1 \text{ kips.}$

**Solution B:**

LRFD	ASD
$R_u = 1.2(16.5 \text{ kips}) + 1.6(47 \text{ kips}) = 95 \text{ kips}$	$R_a = 16.5 \text{ kips} + 47 \text{ kips} = 63.5 \text{ kips}$

LRFD	ASD
Calculate required section modulus based on flexural rupture	Calculate required section modulus based on flexural rupture
$S_{req} = \frac{R_u e}{\phi F_y} = \frac{(95 \text{ kips})(9 \frac{1}{2} \text{ in.})}{0.75(65 \text{ ksi})}$ $= 18.6 \text{ in.}^3$	$S_{req} = \frac{R_a e \Omega}{F_u} = \frac{63.5 \text{ kips}(9 \frac{1}{2} \text{ in.})(2.00)}{65 \text{ ksi}}$ $= 18.6 \text{ in.}^3$

Try W21×73 with an 8-in. deep cope

$$S_{net} = 21.0 \text{ in.}^3 > 18.6 \text{ in.}^3 \quad \mathbf{o.k.}$$

Manual  
Table 9-2

*Check local buckling*

Similarly 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.

Therefore,

$$R_n = \frac{F_{cr} S_{net}}{e} = \frac{(50 \text{ ksi})(21.0 \text{ in.}^3)}{9 \frac{1}{2} \text{ in.}} = 111 \text{ kips}$$

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = 0.90(111 \text{ kips}) = 100 \text{ kips}$	$R_n / \Omega = \frac{111 \text{ kips}}{1.67} = 66.5 \text{ kips}$
$> 95.0 \text{ kips} \quad \mathbf{o.k.}$	$> 63.5 \text{ kips} \quad \mathbf{o.k.}$

Section F1

**Solution C:**

LRFD	ASD
<p><i>Design doubler plate</i></p> <p>Doubler plate must provide a required strength of</p> <p>95 kips – 84.3 kips = 10.7 kips.</p> $S_{req} = \frac{(R_u - \phi R_{n\ beam})e}{\phi F_y}$ $= \frac{(95\ \text{kips} - 84.3\ \text{kips})(9\ \frac{1}{2}\ \text{in.})}{0.90(50\ \text{ksi})}$ $= 2.26\ \text{in.}^3$ <p>For an 8-in. deep plate,</p> $t_{req} = \frac{6S_{req}}{d^2} = \frac{6(2.26\ \text{in.}^3)}{(8\ \text{in.})^2} = 0.212\ \text{in.}$	<p><i>Design doubler plate</i></p> <p>Doubler plate must provide a required strength of</p> <p>63.5 kips – 56.1 kips = 7.40 kips.</p> $S_{req} = \frac{(R_a - R_{n\ beam} / \Omega)e \Omega}{F_y}$ $= \frac{(63.5\ \text{kips} - 56.1\ \text{kips})(9\ \frac{1}{2}\ \text{in.})(1.67)}{50\ \text{ksi}}$ $= 2.35\ \text{in.}^3$ <p>For an 8-in. deep plate,</p> $t_{req} = \frac{6S_{req}}{d^2} = \frac{6(2.35\ \text{in.}^3)}{(8\ \text{in.})^2} = 0.220\ \text{in.}$

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  $\frac{5}{16}$  in.  $\times$  8 in.  $\times$  1'-5" with  $\frac{1}{4}$  in welds.

**Solution D:***Design longitudinal stiffeners*

Try PL  $\frac{1}{4}$  in.  $\times$  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, the neutral axis is located 4.40 in. from the bottom flange (8.84 in. from the top of the stiffener) and the elastic section modulus of the reinforced section is as follows:

	$I_o$ (in. <sup>4</sup> )	$Ad^2$ (in. <sup>4</sup> )	$I_o + Ad^2$ (in. <sup>4</sup> )
Stiffener	0.00521	76.0	76.0
W21 $\times$ 62 web	63.2	28.6	91.8
W21 $\times$ 62 bottom flange	0.160	84.9	85.1
			$\Sigma = I_x = 253$ in. <sup>4</sup>

$$S_{net} = \frac{I_x}{c} = \frac{253 \text{ in.}^4}{8.84 \text{ in.}} = 28.6 \text{ in.}^3$$

and the nominal strength of the reinforced section is

$$R_n = \frac{F_y S_{net}}{e} = \frac{(50 \text{ ksi})(28.6 \text{ in.}^3)}{9 \frac{1}{2} \text{ in.}} = 151 \text{ kips}$$

LRFD	ASD
<i>Calculate design strength</i>	<i>Calculate allowable strength</i>
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = 0.90(151 \text{ kips})$	$R_n / \Omega = \frac{151 \text{ kips}}{1.67}$
$= 136 \text{ kips} > 95 \text{ kips}$ <b>o.k.</b>	$= 90.4 \text{ kips} > 63.5 \text{ kips}$ <b>o.k.</b>

Section F1

*Plate dimensions*

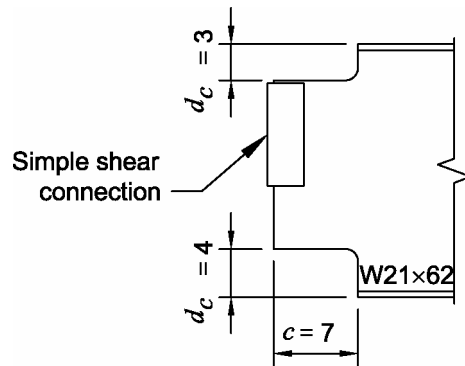
Since the longitudinal stiffening must extend at least  $d_c$  beyond cope,

Use PL  $\frac{1}{4}$  in.  $\times$  4 in.  $\times$  1'-5" with  $\frac{1}{4}$  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 rupture, local buckling, shear yielding, and shear rupture. Assume  $e = 7\frac{1}{2}$  in.



#### Material Properties:

Beam W21×62

ASTM A992

 $F_y = 50$  ksi $F_u = 65$  ksiManual  
Table 2-3

#### Geometric Properties:

Beam W21×62

 $d = 21.0$  in.  $t_w = 0.400$  in.

Cope

 $c = 7$  in.  $d_c = 4$  in.  $e = 7\frac{1}{2}$  in.  $h_o = 14.0$  in.Manual  
Tables 1-1  
and 9-2a

#### Solution:

Check flexural rupture on the coped section

$$S_{net} = \frac{t_w h_o^2}{6} = \frac{(0.400 \text{ in.})(14.0 \text{ in.})^2}{6} = 13.1 \text{ in.}^3$$

Manual  
Part 9

$$R_n = \frac{F_u S_{net}}{e} = \frac{(65 \text{ ksi})(13.1 \text{ in.}^3)}{7\frac{1}{2} \text{ in.}} = 114 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(114 \text{ kips}) = 85.1 \text{ kips}$	$R_n / \Omega = \frac{114 \text{ kips}}{2.00} = 57.0 \text{ kips}$

Section F1

*Check local buckling*

Verify parameters

Manual  
Part 9

$$c \leq 2d$$

$$c = 7.0 \text{ in.} < 2(21.0 \text{ in.}) = 42.0 \text{ in.} \quad \text{o.k.}$$

$$d_c \leq 0.2d$$

$$d_c = 3.0 \text{ in.} < 0.2(21.0) = 4.20 \text{ in.} \quad \text{o.k.}$$

$$f_d = 3.5 - 7.5 \left( \frac{d_c}{d} \right) = 3.5 - 7.5 \left( \frac{3.0 \text{ in.}}{21.0 \text{ in.}} \right) = 2.43$$

For the doubly-coped beam when  $c \leq 2d$  and  $d_c \leq 0.2d$ , the critical buckling stress isManual  
Part 9

$$\begin{aligned} F_{cr} &= 0.62\pi E \left( \frac{t_w^2}{ch_0} \right) f_d \leq F_y \\ &= 0.62\pi(29,000) \left[ \frac{(0.400 \text{ in.})^2}{(7 \text{ in.})(14.0 \text{ in.})} \right] (2.43) = 224 \text{ ksi} \leq 50 \text{ ksi} \end{aligned}$$

Use  $F_{cr} = 50 \text{ ksi}$ 

$$R_n = \frac{F_{cr} S_{net}}{e} = \frac{(50 \text{ ksi})(13.1 \text{ in.}^3)}{7 \frac{1}{2} \text{ in.}} = 87.3 \text{ kips (same as yielding)}$$

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = 0.90(87.3 \text{ kips}) = 78.6 \text{ kips}$	$R_n / \Omega = \frac{(87.3 \text{ kips})}{1.67} = 52.3 \text{ kips}$

*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
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 1.00(168 \text{ kips}) = 168 \text{ kips}$	$R_n / \Omega = \frac{168 \text{ kips}}{1.50} = 112 \text{ kips}$

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
$\phi = 0.75$ $\phi R_n = 0.75(218 \text{ kips}) = 164 \text{ kips}$	$\Omega = 2.00$ $R_n / \Omega = \frac{218 \text{ kips}}{2.00} = 109 \text{ kips}$

Specification  
Sect J4.2

Thus, the available strength is controlled by local bucking, with

LRFD	ASD
$\phi R_n = 78.6 \text{ kips}$	$R_n / \Omega = 52.3 \text{ kips}$

### Example II.A-8 All-Bolted Double-Angle Connections (beams-to-girder web).

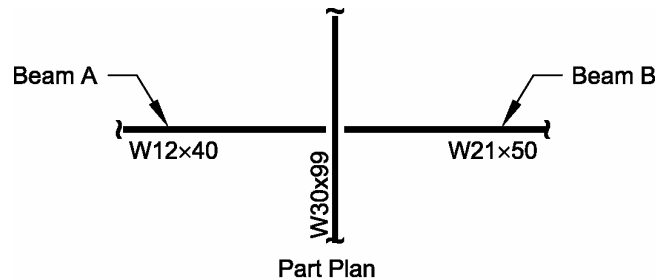
#### Given:

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  
 $R_{DA} = 4.17$  kips  
 $R_{LA} = 12.5$  kips

Beam B  
 $R_{DB} = 18.3$  kips  
 $R_{LB} = 55.0$  kips

Use  $\frac{3}{4}$ -in. diameter ASTM A325-N bolts in standard holes.



#### 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:

Beam W12×40 top flange cope	$t_w = 0.295$ in. $d_c = 2$ in.	$d = 11.9$ in. $c = 5$ in.	$h_o = 9.9$ in.	$S_{net} = 8.03$ in. <sup>3</sup>	Manual Table 1-1 and Manual Table 9-2
Beam W21×50 top flange cope	$t_w = 0.380$ in. $d_c = 2$ in.	$d = 20.8$ in. $c = 5$ in.	$h_o = 18.8$ in.	$S_{net} = 32.5$ in. <sup>3</sup>	
Girder W30×99	$t_w = 0.520$ in.	$d = 29.7$ in.			

**Solution:**

**Beam A:**

LRFD	ASD
$R_{Au} = (1.2)(4.17 \text{ kips}) + (1.6)(12.5 \text{ kips})$ $= 25 \text{ kips}$ <p><i>Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture for the W12×40 (beam A)</i></p> <p>Try two rows of bolts and ¼-in. angle thickness</p> $\phi R_n = 48.9 \text{ kips} > 25 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check beam web for bolt bearing, block shear rupture, shear yielding, and shear rupture (beam A)</i></p> <p>From Table 10-1, for two rows of bolts and <math>L_{ev} = 1\frac{1}{4}</math> in. and <math>L_{eh} = 1\frac{1}{2}</math> in.</p> $\phi R_n = (126 \text{ kips/in.})(0.295 \text{ in.})$ $= 37.2 \text{ kips} > 25 \text{ kips} \quad \mathbf{o.k.}$	$R_{Au} = 4.17 \text{ kips} + 12.5 \text{ kips}$ $= 16.7 \text{ kips}$ <p><i>Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture for the W12×40 (beam A)</i></p> <p>Try two rows of bolts and ¼-in. angle thickness</p> $R_n / \Omega = 32.6 \text{ kips} > 16.7 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check beam web for bolt bearing, block shear rupture, shear yielding, and shear rupture (beam A)</i></p> <p>From Table 10-1, for two rows of bolts and <math>L_{ev} = 1\frac{1}{4}</math> in. and <math>L_{eh} = 1\frac{1}{2}</math> in.</p> $R_n / \Omega = (83.7 \text{ kips/in.})(0.295 \text{ in.})$ $= 24.7 \text{ kips} > 16.7 \text{ kips} \quad \mathbf{o.k.}$
<p><i>Check flexural rupture of the coped section (beam A)</i></p> $S_{net} = 8.03 \text{ in}^3$ $\phi = 0.75$ $\phi R_n = \frac{\phi F_u S_{net}}{e} = \frac{0.75(65 \text{ ksi})(8.03 \text{ in.}^3)}{(5 \text{ in.} + \frac{1}{2} \text{ in.})}$ $= 71.2 \text{ kips} > 25 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check flexural rupture of the coped section (beam A)</i></p> $S_{net} = 8.03 \text{ in}^3$ $\Omega = 2.00$ $R_n / \Omega = \frac{F_u S_{net}}{e \Omega} = \frac{(65 \text{ ksi})(8.03 \text{ in.}^3)}{(5 \text{ in.} + \frac{1}{2} \text{ in.})(2.00)}$ $= 47.5 \text{ kips} > 16.7 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 10-1

Manual  
Table 10-1

Manual  
Table 9-2

Section F1

Manual  
Part 9

*Check coped section*

$$S_{net} = 8.03 \text{ in.}^3$$

Manual  
Table 9-2

*Verify parameters*

$$c \leq 2d$$

Manual  
Part 9

$$5 \text{ in.} \leq 2(11.9 \text{ in.}) = 23.8 \text{ in.} \quad \mathbf{o.k.}$$

$$d_c \leq d/2$$

$$2 \text{ in.} \leq 11.9 \text{ in.} / 2 = 5.95 \text{ in.} \quad \mathbf{o.k.}$$

$$\frac{c}{d} = \frac{5 \text{ in.}}{11.9 \text{ in.}} = 0.420 \leq 1.0; \quad \frac{c}{h_0} = \frac{5 \text{ in.}}{9.9 \text{ in.}} = 0.505 \leq 1.0$$

Calculate plate buckling model adjustment factor

$$f = 2\left(\frac{c}{d}\right) = 2(0.420) = 0.840$$

Calculate plate buckling coefficient

$$k = 2.2\left(\frac{h_0}{c}\right)^{1.65} = 2.2\left(\frac{9.90}{5.00}\right)^{1.65} = 6.79$$

$$F_{cr} = 26,210\left(\frac{t_w}{h_0}\right)^2 f k \leq F_y$$

$$= 26,210\left(\frac{0.250 \text{ in.}}{9.9 \text{ in.}}\right)^2 (0.840)(6.79) = 95.3 \text{ ksi} \leq 50 \text{ ksi}$$

Use  $F_{cr} = 50 \text{ ksi}$

LRFD	ASD
$\phi = 0.90$ $\phi R_n = \frac{\phi F_{cr} S_{net}}{e} = \frac{0.90(50 \text{ ksi})(8.03 \text{ in.}^3)}{(5 \frac{1}{2} \text{ in.})}$ $= 65.7 \text{ kips} > 25 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.67$ $R_n / \Omega = \frac{(F_{cr} / \Omega) S_{net}}{e} = \frac{(50 \text{ ksi})(8.03 \text{ in.}^3)}{1.67(5 \frac{1}{2} \text{ in.})}$ $= 43.7 \text{ kips} > 16.7 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Part 9

Check shear yielding on beam web

$$R_n = 0.6 F_y A_g = 0.6(50 \text{ ksi})(0.295 \text{ in.})(9.90 \text{ in.}) = 87.6 \text{ kips}$$

Eqn J4-3

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(87.6 \text{ kips})$ $= 87.6 \text{ kips} > 25 \text{ kips.} \quad \mathbf{o.k.}$	$\Omega = 1.50$ $R_n / \Omega = \frac{87.6 \text{ kips}}{1.50}$ $= 58.4 \text{ kips} > 16.7 \text{ kips} \quad \mathbf{o.k.}$

Section J4.2

Check shear rupture on beam web

$$A_{nv} = t_w [(h_o - 2(0.875 \text{ in.}))] = 0.295 \text{ in.} (9.9 \text{ in.} - 1.75 \text{ in.}) = 2.40 \text{ in}^2$$

$$R_n = 0.6 F_u A_{nv} = 0.6(65 \text{ ksi})(2.40 \text{ in}^2) = 93.8 \text{ kips}$$

Eqn J4-4

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(93.8 \text{ kips}) = 70.3 \text{ kips}$ $70.3 \text{ kips} > 25 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $R_n / \Omega = \frac{93.8 \text{ kips}}{2.00} = 46.9 \text{ kips}$ $46.9 \text{ kips} > 16.7 \text{ kips} \quad \mathbf{o.k.}$

Section J4.2

**Beam B:**

LRFD	ASD
$R_{Bu} = (1.2)(18.3\text{kips}) + (1.6)(55.0\text{ kips})$ $= 110\text{ kips}$  <i>Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture for beam B (W21×50)</i>  Try five rows of bolts and ¼ in. angle thickness.  $\phi R_n = 125\text{ kips} > 110\text{ kips}$ <b>o.k.</b>  <i>Check beam web for bolt bearing, block shear rupture, shear yielding, and shear rupture (beam B)</i>  From tables, for five rows of bolts and $L_{ev} = 1\frac{1}{4}\text{ in.}$ and $L_{eh} = 1\frac{1}{2}\text{ in.}$  $\phi R_n = (312\text{ kips/in.})(0.380\text{ in.})$ $= 119\text{ kips} > 110\text{ kips}$ <b>o.k.</b>	$R_{Bu} = 18.3\text{ kips} + 55.0\text{ kips}$ $= 73.3\text{ kips}$  <i>Check bolt shear, check angles for bolt bearing, shear yielding, shear rupture, and block shear rupture for beam B (W21×50)</i>  Try five rows of bolts and ¼ in. angle thickness.  $R_n/\Omega = 83.3\text{ kips} > 73.3\text{ kips}$ <b>o.k.</b>  <i>Check beam web for bolt bearing, block shear rupture, shear yielding, and shear rupture (beam B)</i>  From tables, for five rows of bolts and $L_{ev} = 1\frac{1}{4}\text{ in.}$ and $L_{eh} = 1\frac{1}{2}\text{ in.}$  $R_n/\Omega = (208\text{ kips/in.})(0.380\text{ in.})$ $= 79.0\text{ kips} > 73.3\text{ kips}$ <b>o.k.</b>
<i>Check flexural rupture of the coped section (beam B)</i>  $S_{net} = 32.5\text{ in}^3$  $\phi = 0.75$  $\phi R_n = \frac{\phi F_u S_{net}}{e} = \frac{0.75(65\text{ ksi})(32.5\text{ in}^3)}{(5\frac{1}{2}\text{ in.})}$  $= 288\text{ kips} > 110\text{ kips}$ <b>o.k.</b>	<i>Check flexural rupture of the coped section (beam B)</i>  $S_{net} = 32.5\text{ in}^3$  $\Omega = 2.00$  $R_n/\Omega = \frac{F_u S_{net}}{e \Omega} = \frac{(65\text{ ksi})(32.5\text{ in}^3)}{(5\frac{1}{2}\text{ in.})(2.00)}$  $= 192\text{ kips} > 73.3\text{ kips}$ <b>o.k.</b>

Manual  
Table 10-1Manual  
Table 10-1Manual  
Table 9-2Section F1  
Manual  
Part 9*Check coped section*

$$S_{net} = 32.5\text{ in.}^3$$

Manual Table  
9-2*Verify parameters*

$$c \leq 2d$$

Manual  
Part 9

$$5\text{ in.} \leq 2(20.8\text{ in.}) = 41.6\text{ in.} \quad \mathbf{o.k.}$$

$$d_c \leq d/2$$

$$2\text{ in.} \leq 20.8\text{ in.} / 2 = 10.4\text{ in.} \quad \mathbf{o.k.}$$

$$\frac{c}{d} = \frac{5\text{ in.}}{20.8\text{ in.}} = 0.240 \leq 1.0; \quad \frac{c}{h_0} = \frac{5\text{ in.}}{18.8\text{ in.}} = 0.266 \leq 1.0$$

Calculate plate buckling model adjustment factor

$$f = 2\left(\frac{c}{d}\right) = 2(0.240) = 0.480$$

Calculate plate buckling coefficient

$$k = 2.2\left(\frac{h_0}{c}\right)^{1.65} = 2.2\left(\frac{18.8}{5.00}\right)^{1.65} = 19.6$$

$$F_{cr} = 26,210\left(\frac{t_w}{h_0}\right)^2 \quad fk \leq F_y$$

$$= 26,210\left(\frac{0.380 \text{ in.}}{18.8 \text{ in.}}\right)^2 (0.480)(19.6) = 101 \text{ ksi} \leq 50 \text{ ksi}$$

Use  $F_{cr} = 50 \text{ ksi}$

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = \frac{\phi F_{cr} S_{net}}{e} = \frac{0.90(50 \text{ ksi})(32.5 \text{ in.}^3)}{(5 \frac{1}{2} \text{ in.})}$	$R_n / \Omega = \frac{(F_{cr} / \Omega) S_{net}}{e} = \frac{(50 \text{ ksi})(32.5 \text{ in.}^3)}{1.67(5 \frac{1}{2} \text{ in.})}$
$= 266 \text{ kips} > 110 \text{ kips} \quad \mathbf{o.k.}$	$= 117 \text{ kips} > 73.3 \text{ kips} \quad \mathbf{o.k.}$

Check shear yielding on beam web

$$R_n = 0.6 F_y A_g = 0.6(50 \text{ ksi})(0.380 \text{ in})(18.8 \text{ in}) = 214 \text{ kips}$$

Eqn J4-3

LRFD	ASD
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 1.00(214 \text{ kips})$	$R_n / \Omega = \frac{214 \text{ kips}}{1.50}$
$= 214 \text{ kips} > 110 \text{ kips} \quad \mathbf{o.k.}$	$= 143 \text{ kips} > 73.3 \text{ kips} \quad \mathbf{o.k.}$

Section J4.2

Check shear rupture on beam web

$$A_{nv} = t_w [h_o - (5)(0.875 \text{ in.})] = 0.380 \text{ in.} (18.8 \text{ in.} - 4.38 \text{ in.}) = 5.48 \text{ in.}^2$$

$$R_n = 0.6 F_u A_{nv} = 0.6(65 \text{ ksi})(5.48 \text{ in.}^2) = 213 \text{ kips}$$

Eqn J4-4

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(213 \text{ kips}) = 160 \text{ kips}$	$R_n / \Omega = \frac{213 \text{ kips}}{2.00} = 107 \text{ kips}$
$160 \text{ kips} > 110 \text{ kips} \quad \mathbf{o.k.}$	$107 \text{ kips} > 73.3 \text{ kips} \quad \mathbf{o.k.}$

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 kips or 6.25 kips/bolt.	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 110 kips or 11.0 kips/bolt.	From Beam B, each bolt must support one-tenth of 73.3 kips or 7.33 kips/bolt.
Thus,	Thus,
$R_u = 6.25 \text{ kips/bolt} + 11.0 \text{ kips/bolt}$	$R_a = 4.18 \text{ kips/bolt} + 7.33 \text{ kips/bolt}$
$= 17.3 \text{ kips/bolt}$	$= 11.5 \text{ kips/bolt}$
The design bearing strength per bolt is	The allowable bearing strength per bolt is
$\phi r_u = (87.8 \text{ kips/in.})(0.520 \text{ in.})$	$r_a / \Omega = (58.5 \text{ kips/in.})(0.520 \text{ in.})$
$= 45.7 \text{ kips/bolt} > 17.3 \text{ kips/bolt}$ <b>o.k.</b>	$= 30.4 \text{ kips/bolt} > 11.5 \text{ kips/bolt}$ <b>o.k.</b>

Manual  
Table 7-5

Although not required for design, the tabulated values may be verified by hand calculations, as follows

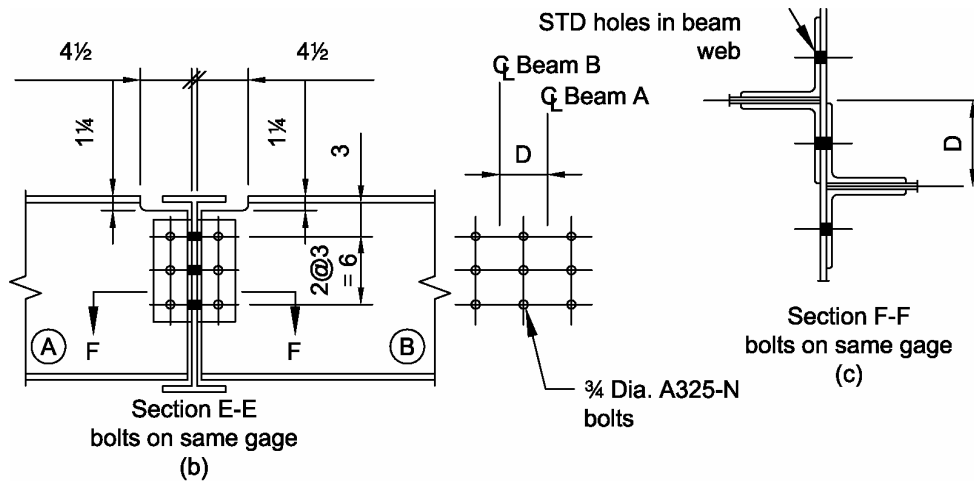
$\phi r_n = \phi(1.2L_c t F_u) \leq \phi(2.4dt F_u)$	$r_n / \Omega = (1.2L_c t F_u) / \Omega \leq (2.4dt F_u) / \Omega$
$\phi 1.2L_c t F_u =$	$1.2L_c t F_u / \Omega =$
$(0.75)(1.2)(3 \text{ in.} - \frac{7}{8} \text{ in.})(0.520 \text{ in.})(65 \text{ ksi})$	$(1.2)(3 \text{ in.} - \frac{7}{8} \text{ in.})(0.520 \text{ in.})(65 \text{ ksi}) / (2.00)$
$= 64.7 \text{ kips}$	$= 43.1 \text{ kips}$
$\phi(2.4dt F_u) =$	$(2.4dt F_u) / \Omega =$
$(0.75)(2.4)(\frac{3}{4} \text{ in.})(0.520 \text{ in.})(65 \text{ ksi})$	$(2.4)(\frac{3}{4} \text{ in.})(0.520 \text{ in.})(65 \text{ ksi}) / (2.00)$
$= 45.6 \text{ kips} < 64.6 \text{ kips}$	$= 30.4 \text{ kips} < 43.1 \text{ kips}$
$\phi r_n = 45.6 \text{ kips/bolt} > 17.3 \text{ kips/bolt}$ <b>o.k.</b>	$r_n / \Omega = 30.4 \text{ kips/bolt} > 11.5 \text{ kips/bolt}$ <b>o.k.</b>

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.



#### Material Properties:

W18×50	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual Table 2-3
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  $2L4 \times 3\frac{1}{2} \times \frac{1}{4}$  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 ( $6\frac{1}{4}$  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,

$$R_D = 10 \text{ kips}$$

$$R_L = 30 \text{ kips}$$



LRFD	ASD
$R_u = (1.2)(10 \text{ kips}) + (1.6)(30 \text{ kips}) = 60 \text{ kips}$	$R_a = 10 \text{ kips} + 30 \text{ kips} = 40 \text{ kips}$
The required bearing strength per bolt is	The required bearing strength per bolt is
$r_u = \frac{(2 \text{ connections})(60 \text{ kips/connection})}{6 \text{ bolts}}$	$r_a = \frac{(2 \text{ connections})(40 \text{ kips/connection})}{6 \text{ bolts}}$
= 20.0 kips/bolt	= 13.3 kips/bolt
<i>Check supporting girder web</i>	<i>Check supporting girder web</i>
The design strength per bolt is	The allowable strength per bolt is
$\phi r_u = (87.8 \text{ kips/in.})(0.355 \text{ in.})$	$r_a / \Omega = (58.5 \text{ kips/in.})(0.355 \text{ in.})$
= 31.2 kips/bolt > 20.0 kips/bolt <b>o.k.</b>	= 20.8 kips/bolt > 13.3 kips/bolt <b>o.k.</b>

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).**

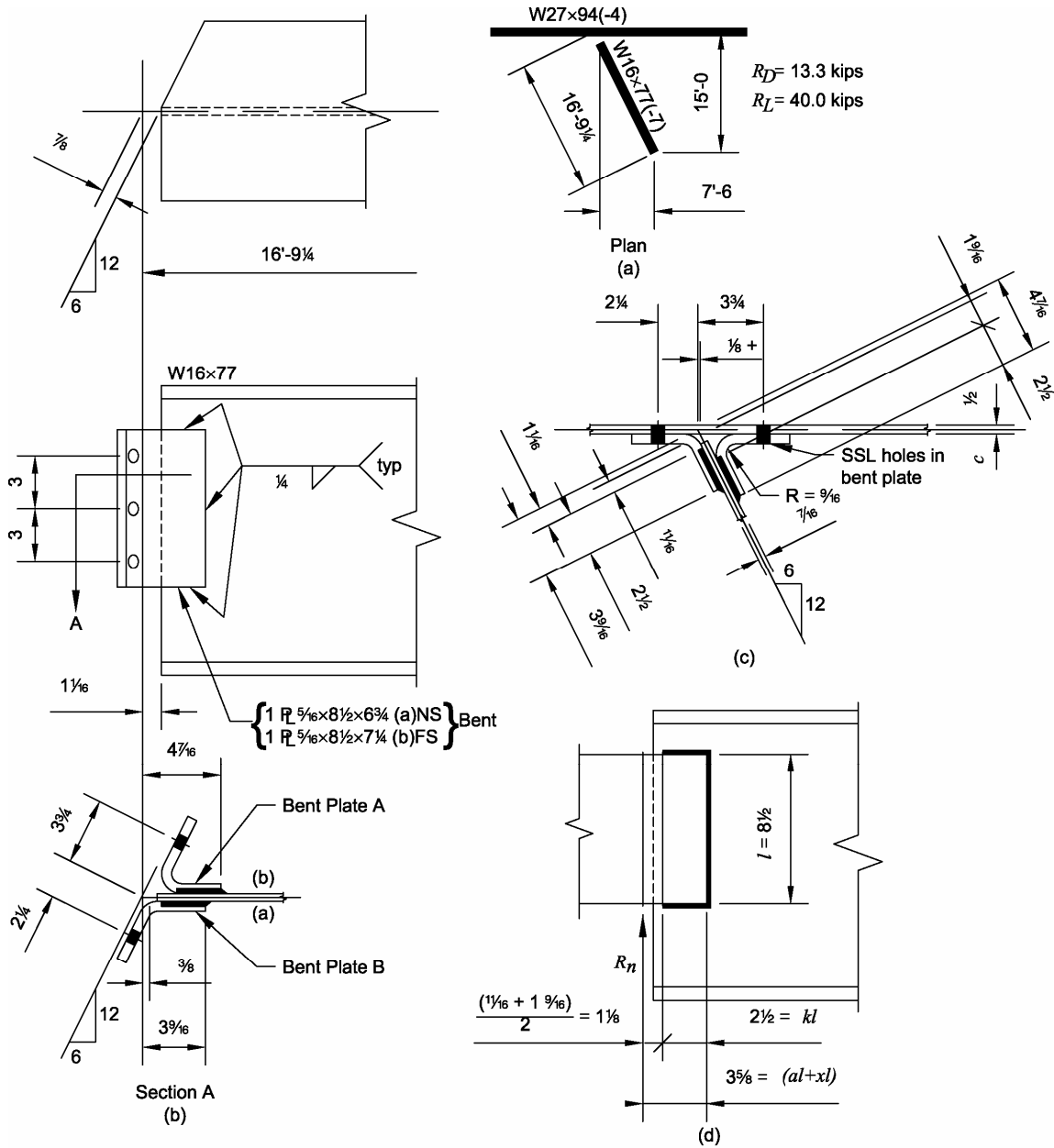
**Given:**

Design the skewed double bent-plate connection between the W16x77 beam to W27x94 girder-web to support the following beam end reactions:

$$R_D = 13.3 \text{ kips}$$

$$R_L = 40.0 \text{ kips}$$

Use 7/8-in. diameter ASTM A325-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
W27×94	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Plate Material	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi

Manual  
Tables 2-3  
and 2-4**Geometric Properties:**

W16×77	$t_w = 0.455$ in.	$d = 16.5$ in.
W27×94	$t_w = 0.490$ in.	

Manual  
Table 1-1**Solution:**

LRFD	ASD
$R_u = 1.2 (13.3 \text{ kips}) + 1.6 (40.0 \text{ kips})$	$R_u = 13.3 \text{ kips} + 40.0 \text{ kips}$
$= 80.0 \text{ kips}$	$= 53.3 \text{ kips}$

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
<i>Required strength of bent plate A</i>	<i>Required strength for bent plate A</i>
$R_u = \frac{80.0 \text{ kips} (2.25 \text{ in.})}{6.0 \text{ in.}} = 30.0 \text{ kips}$	$R_u = \frac{53.3 \text{ kips} (2.25 \text{ in.})}{6.0 \text{ in.}} = 20.0 \text{ kips}$
<i>Required strength for bent plate B</i>	<i>Required strength for bent plate B</i>
$R_u = 80.0 \text{ kips} - 30.0 \text{ kips} = 50.0 \text{ kips}$	$R_u = 53.3 \text{ kips} - 20.0 \text{ kips} = 33.3 \text{ kips}$

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.

$$k = \frac{kl}{l} = \frac{2\frac{1}{2} \text{ in.}}{8\frac{1}{2} \text{ in.}} = 0.294 \text{ in.}$$

From tables, with  $\theta = 0^\circ$  and  $k = 0.294$

$$xl = \frac{2.5(1.25)(2)}{2.5(2) + 8.5} = 0.463 \text{ in.}$$

$$x = \frac{0.463}{8.5} = 0.054$$

Manual  
Table 8-8

Thus,

$$a = \frac{(al + xl) - xl}{l} = \frac{3\frac{5}{8} \text{ in} - 0.463 \text{ in.}}{8\frac{1}{2} \text{ in.}} = 0.372$$

Interpolating from tables, with  $\theta = 0^\circ$ ,  $a = 0.372$ , and  $k = 0.294$ ,

$$C = 2.52$$

Manual  
Table 8-8

*The required weld size for two such welds is*

LRFD	ASD
$\phi = 0.75$ $D_{req} = \frac{R_u}{\phi CC_1 l}$ $= \frac{(50 \text{ kips})}{0.75(2.52)(1.0)(8\frac{1}{2} \text{ in.})}$ $= 3.11 \rightarrow 4 \text{ sixteenths}$	$\Omega = 2.00$ $D_{req} = \frac{\Omega R_a}{CC_1 l}$ $= \frac{2.0(33.3 \text{ kips})}{(2.52)(1.0)(8\frac{1}{2} \text{ in.})}$ $= 3.11 \rightarrow 4 \text{ sixteenths}$

Manual  
Table 8-9

Use 1/4-in. fillet 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

$$t_{min} = \frac{6.19D}{F_u} = \frac{(6.19)(4 \text{ sixteenths})}{65 \text{ ksi}} = 0.381 \text{ in.} < 0.455 \text{ in.} \quad \text{o.k.}$$

LRFD	ASD
<p><i>Design bolts</i></p> <p>Maximum shear to bent plate = 50 kips</p> <p>Use 3 rows of 7/8 in. diameter ASTM A325-N bolts.</p> <p><i>Check shear on bolts</i></p> $\phi R_n = n(\phi r_n)$ $= (3 \text{ bolts})(21.6 \text{ kips/bolt})$ $= 64.8 \text{ kips} > 50 \text{ kips} \quad \text{o.k.}$ <p><i>Check bearing on support</i></p> $\phi r_n = 102 \text{ kips/in.}$ $= (102 \text{ kips/in.})(0.490 \text{ in.})(3 \text{ bolts})$ $= 150 \text{ kips} > 50 \text{ kips} \quad \text{o.k.}$	<p><i>Design bolts</i></p> <p>Maximum shear to bent plate = 33.3 kips</p> <p>Use 3 rows of 7/8 in. diameter ASTM A325-N bolts.</p> <p><i>Check shear on bolts</i></p> $R_n / \Omega = n(r_n / \Omega)$ $= (3 \text{ bolts})(14.4 \text{ kips/bolt})$ $= 43.2 \text{ kips} > 33.3 \text{ kips} \quad \text{o.k.}$ <p><i>Check bearing on support</i></p> $R_n / \Omega = 68.3 \text{ kips/in.}$ $= (68.3 \text{ kips/in.})(0.490 \text{ in.})(3 \text{ bolts})$ $= 100 \text{ kips} > 33.3 \text{ kips} \quad \text{o.k.}$

Manual  
Table 7-1

Manual  
Table 7-6

<p><i>Design bent plates</i></p> <p>Try a PL <math>\frac{5}{16}</math> in.</p> <p><i>Check bearing on plates</i></p> $\phi_v r_{ni} = 91.4 \text{ kips/in.}$ $\phi_v r_{no} = 40.8 \text{ kips/in.}$ $\phi R_n = [(91.4 \text{ kips/in.}) (2 \text{ bolts}) + (40.8 \text{ kips/in.})(1 \text{ bolt})](\frac{5}{16} \text{ in.})$ $= 70 \text{ kips} > 50 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check shear yielding of plates</i></p> $\phi = 1.00$ $\phi R_n = \phi (0.6F_y) A_g$ $= (1.00)(0.6)(36 \text{ ksi})(8\frac{1}{2} \text{ in.})(\frac{5}{16} \text{ in.})$ $= 57.4 \text{ kips} > 50 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check shear rupture of the plates</i></p> $\phi = 0.75$ $A_n = [(8\frac{1}{2} \text{ in.}) - (3)(1.0 \text{ in.})](\frac{5}{16} \text{ in.})$ $= 1.72 \text{ in.}^2$ $\phi R_n = \phi (0.6F_u) A_n$ $= (0.75)(0.6)(58 \text{ ksi})(1.72 \text{ in.}^2)$ $= 44.9 \text{ kips} < 50 \text{ kips} \quad \mathbf{n.g.}$ <p><b>Increase the plate thickness to <math>\frac{3}{8}</math> in.</b></p> $A_n = [(8\frac{1}{2} \text{ in.}) - (3)(1.0 \text{ in.})](\frac{3}{8} \text{ in.})$ $= 2.06 \text{ in.}^2$ $\phi R_n = (0.75)(0.6)(58 \text{ ksi})(2.06 \text{ in.}^2)$ $= 53.8 \text{ kips} > 50 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check block shear rupture of the plate</i></p> <p>with <math>n = 3</math>, <math>L_{ev} = L_{eh} = 1\frac{1}{4}</math> in.,</p> $\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	<p><i>Design bent plates</i></p> <p>Try a PL <math>\frac{5}{16}</math> in.</p> <p><i>Check bearing on plates</i></p> $r_{ni}/\Omega_v = 60.9 \text{ kips/in.}$ $r_{no}/\Omega_v = 27.2 \text{ kips/in.}$ $= [(60.9 \text{ kips/in.}) (2 \text{ bolts}) + (27.2 \text{ kips/in.})(1 \text{ bolt})](\frac{5}{16} \text{ in.})$ $= 46 \text{ kips} > 33.3 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check gross shear yielding of plates</i></p> $\Omega = 1.50$ $R_n / \Omega = (0.6F_y) A_g / \Omega$ $= (0.6)(36 \text{ ksi})(8\frac{1}{2} \text{ in.})(\frac{5}{16} \text{ in.})/1.50$ $= 38.3 \text{ kips} > 33.3 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check shear rupture of the plates</i></p> $\Omega = 2.00$ $A_n = [(8\frac{1}{2} \text{ in.}) - (3)(1.0 \text{ in.})](\frac{5}{16} \text{ in.})$ $= 1.72 \text{ in.}^2$ $R_n / \Omega = (0.6F_u) A_n / \Omega$ $= (0.6)(58 \text{ ksi})(1.72 \text{ in.}^2)/2.00$ $= 29.9 \text{ kips} < 33.3 \text{ kips} \quad \mathbf{n.g.}$ <p><b>Increase the plate thickness to <math>\frac{3}{8}</math> in.</b></p> $A_n = [(8\frac{1}{2} \text{ in.}) - (3)(1.0 \text{ in.})](\frac{3}{8} \text{ in.})$ $= 2.06 \text{ in.}^2$ $R_n / \Omega = (0.6)(58 \text{ ksi})(2.06 \text{ in.}^2)/2.00$ $= 35.8 \text{ kips} > 33.3 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check block shear rupture of the plate</i></p> <p>with <math>n = 3</math>, <math>L_{ev} = L_{eh} = 1\frac{1}{4}</math> in.,</p> $\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$
--	---

Eqn. J4-5

<p>Tension rupture component</p> $\phi F_u A_{nt} = 21.8 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear yielding component</p> $\phi 0.6 F_y A_{gv} = 117 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $\phi 0.6 F_u A_{nv} = 124 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ $\phi R_n = (117 \text{ kips/in.} + 21.8 \text{ kips/in.}) (\frac{3}{8} \text{ in.})$ $= 52.1 \text{ kips} > 50 \text{ kips} \quad \mathbf{o.k.}$	<p>Tension rupture component</p> $F_u A_{nt} = 14.5 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear yielding component</p> $0.6 F_y A_{gv} = 78.3 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $0.6 F_u A_{nv} = 82.6 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ $R_n / \Omega = (78.3 \text{ kips/in.} + 14.5 \text{ kips/in.}) (\frac{3}{8} \text{ in.})$ $= 34.8 \text{ kips} > 33.3 \text{ kips} \quad \mathbf{o.k.}$	<p>Manual Table 9-3a</p> <p>Manual Table 9-3b</p> <p>Manual Table 9-3c</p>
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Thus, the configuration shown in Figure II.A-10 can be supported using  $\frac{3}{8}$ -in. bent plates, and  $\frac{1}{4}$ -in. fillet welds.

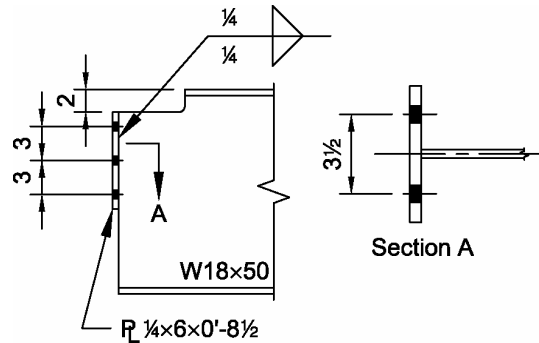
**Example II.A-11 Shear End-Plate Connection (beam to girder web).**

**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 3/4-in. diameter ASTM A325-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 Tables 2-3 and 2-4
Girder W21×62	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Plate	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. <sup>3</sup>	Manual Tables 1-1 and 9-2	
Cope	$c = 4 \frac{1}{4}$ in.	$d_c = 2$ in.	$e = 4 \frac{1}{2}$ in.		$h_0 = 16.0$ in.
Girder W21×62	$t_w = 0.400$ in.				

**Solution:**

LRFD	ASD
$R_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips}) = 60$ kips	$R_a = 10 \text{ kips} + 30 \text{ kips} = 40$ kips
<i>Check bolt shear. Check end plate for bolt bearing, shear yielding, shear rupture and block shear rupture.</i>	<i>Check bolt shear. Check end plate for bolt bearing, shear yielding, shear rupture and block shear rupture.</i>
Try 3 rows of bolts and 1/4 in. plate thickness	Try 3 rows of bolts and 1/4 in. plate thickness
$\phi R_n = 76.4$ kips > 60 kips <b style="float: right;">o.k.</b>	$R_n / \Omega = 50.9$ kips > 40 kips <b style="float: right;">o.k.</b>

Manual  
Table 10-4

<p><i>Check weld shear, check beam web shear rupture</i></p> <p>Try ¼-in. weld.</p> <p><math>t_{wmin} = 0.381 \text{ in.} &gt; 0.355 \text{ in.}</math> <b>Proration req'd</b></p> <p><math>\phi R_n = 89.1 \text{ kips} \frac{(0.355 \text{ in.})}{(0.381 \text{ in.})}</math></p> <p><math>= 83.0 \text{ kips} &gt; 60 \text{ kips}</math> <b>o.k.</b></p>	<p><i>Check weld shear, check beam web shear rupture</i></p> <p>Try ¼-in. weld.</p> <p><math>t_{wmin} = 0.381 \text{ in.} &gt; 0.355 \text{ in.}</math> <b>Proration req'd</b></p> <p><math>R_n / \Omega = 59.4 \text{ kips} \frac{(0.355 \text{ in.})}{(0.381 \text{ in.})}</math></p> <p><math>= 55.3 \text{ kips} &gt; 40 \text{ kips}</math> <b>o.k.</b></p>	Manual Table 10-4
<p><i>Check supporting member web or flange for bolt bearing</i></p> <p><math>\phi R_n = (526 \text{ kips/in.})(0.400 \text{ in.})</math></p> <p><math>= 210 \text{ kips} &gt; 60 \text{ kips}</math> <b>o.k.</b></p>	<p><i>Check supporting member web or flange for bolt bearing</i></p> <p><math>R_n / \Omega = (351 \text{ kips/in.})(0.400 \text{ in.})</math></p> <p><math>= 140 \text{ kips} &gt; 40 \text{ kips}</math> <b>o.k.</b></p>	

*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.



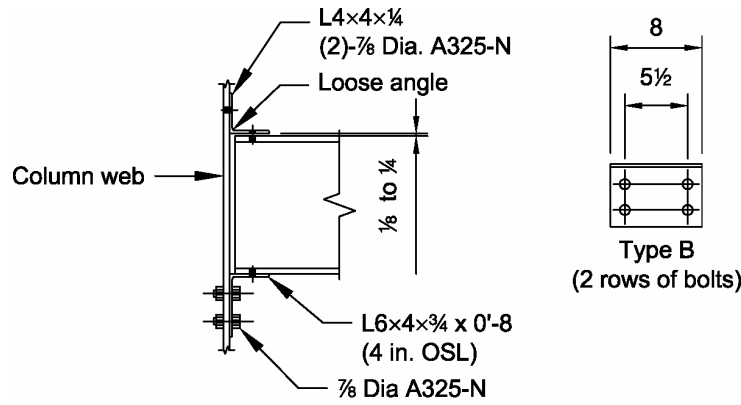
**Example II.A-12 All-Bolted Unstiffened Seated Connection (beam-to-column web).**

**Given:**

Design an all-bolted unstiffened seated connection between a W16×50 beam and W14×90 column web to support the following end reactions:

$R_D = 9$  kips  
 $R_L = 27.5$  kips

Use 7/8-in. diameter ASTM A325-N bolts in standard holes.



**Material Properties:**

Beam W16×50	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual Table 2-3, Table 2-4
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.	$t_f = 0.630$ in.	$k = 1.03$ in.	Manual Table 1-1
Column W14×90	$t_w = 0.440$ in.				

**Solution:**

LRFD	ASD
$R_u = 1.2(9 \text{ kips}) + 1.6(27.5 \text{ kips}) = 55 \text{ kips}$	$R_a = 9 \text{ kips} + 27.5 \text{ kips} = 36.5 \text{ kips}$
<i>Check beam web</i>	<i>Check beam web</i>
For local web yielding,	For local web yielding,
$N_{\min} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$	$N_{\min} = \frac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k$
$= \frac{55 \text{ kips} - 49.0 \text{ kips}}{19.0 \text{ kips}} \geq 1.03$	$= \frac{36.5 \text{ kips} - 32.7 \text{ kips}}{12.7 \text{ kips}} \geq 1.03$
$= 0.316 \text{ in.} < 1.03 \text{ in.}$	$= 0.299 \text{ in.} < 1.03 \text{ in.}$
$= 1.03 \text{ in.}$	$= 1.03 \text{ in.}$
For web crippling,	For web crippling,
When $\frac{N}{d} \leq 0.2$	When $\frac{N}{d} \leq 0.2$
$N_{\min} = \frac{R_u - \phi R_3}{\phi R_4}$	$N_{\min} = \frac{R_a - R_3 / \Omega}{R_4 / \Omega}$
$= \frac{55 \text{ kips} - 67.2 \text{ kips}}{5.81 \text{ kips}}$	$= \frac{36.5 \text{ kips} - 44.8 \text{ kips}}{3.87 \text{ kips}}$
which results in a negative quantity.	which results in a negative quantity.
When $\frac{N}{d} > 0.2$	When $\frac{N}{d} > 0.2$
$N_{\min} = \frac{R_u - \phi R_5}{\phi R_6}$	$N_{\min} = \frac{R_a - R_5 / \Omega}{R_6 / \Omega}$
$= \frac{55 \text{ kips} - 60.9 \text{ kips}}{7.74 \text{ kips/in.}}$	$= \frac{36.5 \text{ kips} - 40.6 \text{ kips}}{5.16 \text{ kips/in.}}$
which results in a negative quantity.	which results in a negative quantity.
Thus, $N_{\min} = 1.03 \text{ in.}$	Thus, $N_{\min} = 1.03 \text{ in.}$

Section J10

Manual  
Table 9-4

<p><i>Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web</i></p> <p>Try an 8 in. angle length with a <math>\frac{3}{4}</math> in. thickness and a <math>3\frac{1}{2}</math> in. minimum outstanding leg.</p> <p><math>\phi R_n = 117 \text{ kips} &gt; 55 \text{ kips}</math> <b>o.k.</b></p> <p>Try L6×4×<math>\frac{3}{4}</math> (4-in. OSL), 8-in. long with <math>5\frac{1}{2}</math>-in. bolt gage, connection type B (four bolts).</p> <p>For <math>\frac{7}{8}</math>-in. diameter ASTM A325-N bolts, <math>\phi R_n = \phi_v r_n n = 86.6 \text{ kips} &gt; 55 \text{ kips}</math> <b>o.k.</b></p>	<p><i>Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web</i></p> <p>Try an 8 in. angle length with a <math>\frac{3}{4}</math> in. thickness and a <math>3\frac{1}{2}</math> in. minimum outstanding leg.</p> <p><math>R_n / \Omega = 77.6 \text{ kips} &gt; 36.5 \text{ kips}</math> <b>o.k.</b></p> <p>Try L6×4×<math>\frac{3}{4}</math> (4-in. OSL), 8-in. long with <math>5\frac{1}{2}</math>-in. bolt gage, connection type B four bolts).</p> <p>For <math>\frac{7}{8}</math>-in. diameter ASTM A325-N bolts, <math>R_n / \Omega = (r_n / \Omega) n = 57.7 \text{ kips} &gt; 36.5 \text{ kips}</math> <b>o.k.</b></p>	Manual Table 10-5
<p><i>Check bolt bearing on the angle</i></p> <p>Bolt single shear strength = 21.6 kips</p> <p><math>\phi R_n = \phi(2.4dtF_u)</math></p> <p><math>= 0.75(2.4)(7/8 \text{ in.})(3/4 \text{ in.})(58 \text{ ksi})</math></p> <p><math>= 68.5 \text{ kips} &gt; 21.6 \text{ kips}</math> <b>o.k.</b></p>	<p><i>Check bolt bearing on the angle</i></p> <p>Bolt single shear strength = 14.4 kips</p> <p><math>R_n / \Omega = \frac{(2.4dtF_u)}{\Omega}</math></p> <p><math>= \frac{(2.4)(7/8 \text{ in.})(3/4 \text{ in.})(58 \text{ ksi})}{2.00}</math></p> <p><math>= 45.7 \text{ kips} &gt; 14.4 \text{ kips}</math> <b>o.k.</b></p>	Manual Table 7-1 and Section J3.10 Eqn (J3-6a)
<p><i>Check supporting column</i></p> <p>Bolt single shear strength = 21.6 kips</p> <p><math>\phi R_n = \phi(2.4dtF_u)</math></p> <p><math>= 0.75(2.4)(7/8 \text{ in.})(0.440 \text{ in.})(65 \text{ ksi})</math></p> <p><math>= 45.0 \text{ kips} &gt; 27.1 \text{ kips}</math> <b>o.k.</b></p>	<p><i>Check supporting column</i></p> <p>Bolt single shear strength = 14.4 kips</p> <p><math>R_n / \Omega = \frac{(2.4dtF_u)}{\Omega}</math></p> <p><math>= \frac{(2.4)(7/8 \text{ in.})(0.440 \text{ in.})(65 \text{ ksi})}{2.00}</math></p> <p><math>= 30.0 \text{ kips} &gt; 14.4 \text{ kips}</math> <b>o.k.</b></p>	Manual Table 7-1 and Section J3.10 Eqn (J3-6a)

*Select top angle and bolts*

Use an L4×4× $\frac{1}{4}$  with two  $\frac{7}{8}$ -in. diameter ASTM A325-N through each leg.

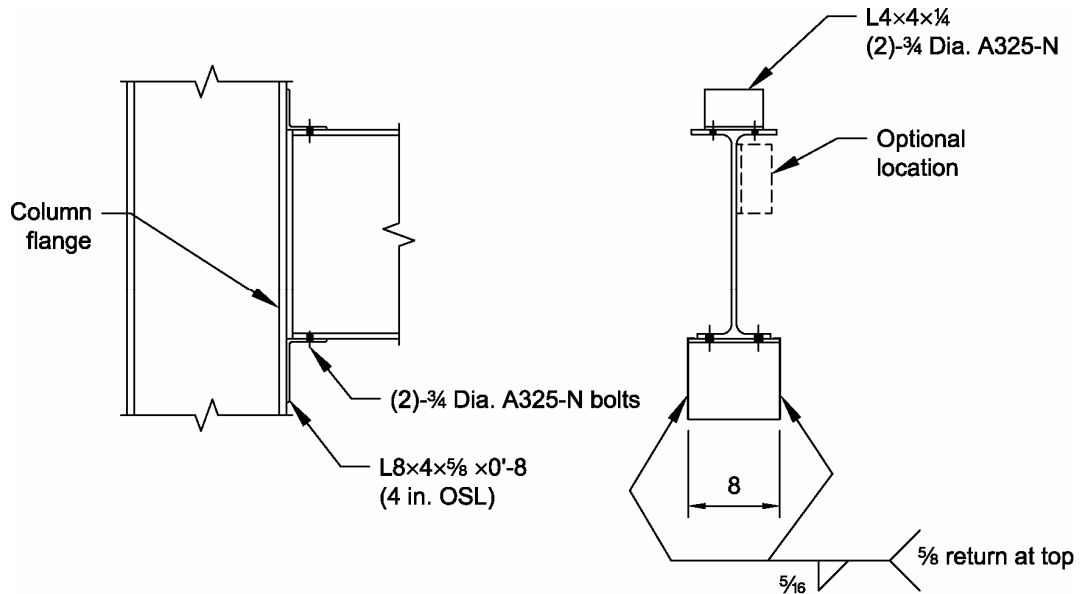
**Example II.A-13 Bolted/Welded Unstiffened Seated Connection (beam-to-column flange)**

**Given:**

Design an unstiffened seated connection between a W21×62 beam and a W14×61 column flange, to support the following beam end reactions:

$R_D = 9$  kips  
 $R_L = 27.5$  kips

Use 3/4-in. diameter ASTM A325-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 3/4 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.	$t_f = 0.615$ in.	$k = 1.12$ in.	Manual Table 1-1
Column	W14×61	$t_f = 0.645$ in.				

**Solution:**

LRFD	ASD
$R_u = 1.2(9 \text{ kips}) + 1.6(27.5 \text{ kips}) = 55 \text{ kips}$	$R_a = 9 \text{ kips} + 27.5 \text{ kips} = 36.5 \text{ kips}$
<i>Check the strength of the beam web</i>	<i>Check the strength of the beam web</i>
For local web yielding	For local web yielding
$N_{\min} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$ $= \frac{55 \text{ kips} - 55.8 \text{ kips}}{20 \text{ kips}} \geq 1.12 \text{ in.}$	$N_{\min} = \frac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k$ $= \frac{36.5 \text{ kips} - 37.2 \text{ kips}}{13.3 \text{ kips}} \geq 1.12 \text{ in.}$
$N_{\min} = 1.12 \text{ in.}$	$N_{\min} = 1.12 \text{ in.}$
For web crippling,	For web crippling,
$\left(\frac{N}{d}\right)_{\max} = \frac{3.25}{21}$ $= 0.16 < 0.2$	$\left(\frac{N}{d}\right)_{\max} = \frac{3.25}{21}$ $= 0.16 < 0.2$
When $\frac{N}{d} \leq 0.2$	When $\frac{N}{d} \leq 0.2$
$N_{\min} = \frac{R_u - \phi R_3}{\phi R_4}$ $= \frac{55 \text{ kips} - 71.7 \text{ kips}}{5.37 \text{ kips}}$	$N_{\min} = \frac{R_a - R_3 / \Omega}{R_4 / \Omega}$ $= \frac{36.5 \text{ kips} - 47.8 \text{ kips}}{3.58 \text{ kips}}$
which results in a negative quantity.	which results in a negative quantity.
Therefore, $N_{\min} = 1.12 \text{ in.}$	Therefore, $N_{\min} = 1.12 \text{ in.}$
<i>Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web</i>	<i>Check shear yielding and flexural yielding of angle. Check local yielding and crippling of beam web</i>
Try an 8 in. angle length with a $\frac{5}{8}$ -in. thickness and a $3\frac{1}{2}$ in. minimum outstanding leg.	Try an 8 in. angle length with a $\frac{5}{8}$ -in. thickness and a $3\frac{1}{2}$ in. minimum outstanding leg.
$N_{\text{req}} > 1\frac{1}{8} \text{ in.}$	$N_{\text{req}} > 1\frac{1}{8} \text{ in.}$
For $N = 1\frac{1}{8} \text{ in.}$	For $N = 1\frac{1}{8} \text{ in.}$
$\phi R_n = 81.0 \text{ kips} > 55 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 53.9 \text{ kips} > 36.5 \text{ kips}$ <b>o.k.</b>

Manual  
Table 9-4Manual  
Table 10-6

Try an L8×4× <sup>5</sup> / <sub>8</sub> (4 in. OSL), 8 in. long with <sup>5</sup> / <sub>16</sub> in. fillet welds.	Try an L8×4× <sup>5</sup> / <sub>8</sub> (4 in. OSL), 8 in. long with <sup>5</sup> / <sub>16</sub> in. fillet welds.
$\phi R_n = 66.7 \text{ kips} > 55 \text{ kips}$ <b>o.k.</b>	$R_n/\Omega = 44.5 \text{ kips} > 36.5 \text{ kips}$ <b>o.k.</b>
Use two <sup>3</sup> / <sub>4</sub> -in. diameter ASTM A325-N bolts to connect the beam to the seat angle.	Use two <sup>3</sup> / <sub>4</sub> -in. diameter ASTM A325-N bolts to connect the beam to the seat angle.

Manual  
Table 10-6

The strength of the bolts must be verified if horizontal forces are added to the connection.

*Select top angle, bolts, and welds*

Use L4×4×<sup>1</sup>/<sub>4</sub> with two <sup>3</sup>/<sub>4</sub> in. diameter ASTM A325-N bolts through the supported-beam leg of the angle. Use a <sup>3</sup>/<sub>16</sub>-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).**

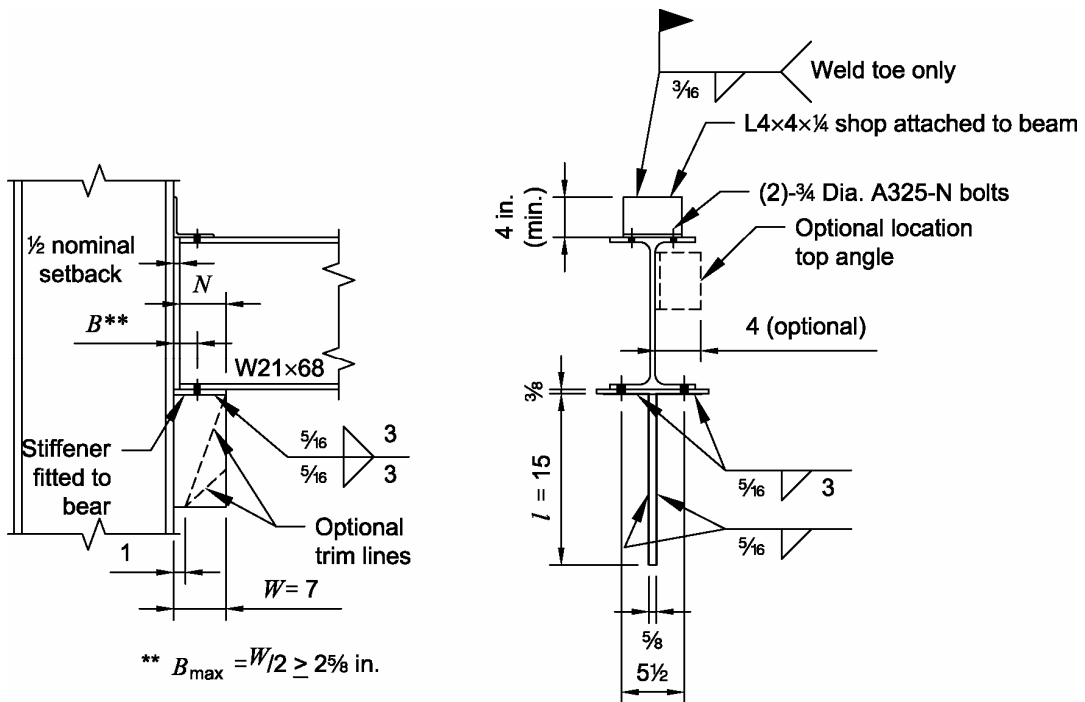
**Given:**

Design a stiffened seated connection between a W21×68 beam and a W14×90 column flange, to support the following end reactions:

$$R_D = 21 \text{ kips}$$

$$R_L = 62.5 \text{ kips}$$

Use 3/4 in. diameter ASTM A325-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 3/4 in. to account for possible beam underrun.

**Material Properties:**

Beam	W21×68	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Tables 2-3 and 2-4
Column	W14×90	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Angles and plates		ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

**Geometric Properties:**

Beam	W21×68	$t_w = 0.430 \text{ in.}$	$d = 21.1 \text{ in.}$	$t_f = 0.685 \text{ in.}$	Manual Table 1-1
Column	W14×90	$t_f = 0.710 \text{ in.}$			

**Solution:**

LRFD	ASD	
$R_u = 1.2(21 \text{ kips}) + 1.6(62.5 \text{ kips}) = 125 \text{ kips}$ <i>Determine stiffener width W required</i> For web crippling, assume $N/d > 0.2$ $W_{\min} = \frac{R_u - \phi R_5}{\phi R_6} + \text{setback}$ $= \frac{125 \text{ kips} - 75.9 \text{ kips}}{7.94 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ $= 6.93 \text{ in.}$ For local web yielding, $W_{\min} = \frac{R_u - \phi R_1}{\phi R_2} + \text{setback}$ $= \frac{125 \text{ kips} - 63.7 \text{ kips}}{21.5 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ $= 3.60 \text{ in.} < 6.93 \text{ in.}$ Use $W = 7 \text{ in.}$ <i>Check assumption</i> $\frac{N}{d} = \frac{6.93 \text{ in.} - 0.75 \text{ in.}}{21.1 \text{ in.}}$ $= 0.293 > 0.20 \quad \mathbf{o.k.}$	$R_a = 21 \text{ kips} + 62.5 \text{ kips} = 83.5 \text{ kips}$ <i>Determine stiffener width W required</i> For web crippling, assume $N/d > 0.2$ $W_{\min} = \frac{R_a - R_5 / \Omega}{R_6 / \Omega} + \text{setback}$ $= \frac{83.5 \text{ kips} - 50.6 \text{ kips}}{5.29 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ $= 6.97 \text{ in.}$ For local web yielding, $W_{\min} = \frac{R_a - R_1 / \Omega}{R_2 / \Omega} + \text{setback}$ $= \frac{83.5 \text{ kips} - 42.5 \text{ kips}}{14.3 \text{ kips/in.}} + \frac{3}{4} \text{ in.}$ $= 3.62 \text{ in.} < 6.97 \text{ in.}$ Use $W = 7 \text{ in.}$ <i>Check assumption</i> $\frac{N}{d} = \frac{6.97 \text{ in.} - 0.75 \text{ in.}}{21.1 \text{ in.}}$ $= 0.295 > 0.20 \quad \mathbf{o.k.}$	Manual Table 9-4
<i>Determine stiffener length L and stiffener to column flange weld size</i> Try a stiffener with $L = 15 \text{ in.}$ and $\frac{5}{16} \text{ in.}$ weld $\phi R_n = 139 \text{ kips} > 125 \text{ kips} \quad \mathbf{o.k.}$	<i>Determine stiffener length L and stiffener to column flange weld size</i> Try a stiffener with $L = 15 \text{ in.}$ and $\frac{5}{16} \text{ in.}$ weld $R_n / \Omega = 93.0 \text{ kips} > 83.5 \text{ kips} \quad \mathbf{o.k.}$	Manual Table 10-8

*Determine weld requirements for seat plate*

Use  $\frac{5}{16}$ -in. fillet welds on each side of the stiffener. Minimum length of seat-plate-to-column flange weld is  $0.2(L) = 3 \text{ in.}$  If the weld between the seat plate and stiffener plate is required to be stronger than the weld between the seat plate and the column flange, use  $\frac{5}{16}$ -in. fillet welds on each side of the stiffener to the seat plate, length of weld =  $7 \text{ in.} > 3 \text{ in.}$  **o.k.**

Manual  
Part 10



*Determine the seat plate dimensions*

A width of 8 in. is adequate to accommodate two  $\frac{3}{4}$ -in. diameter ASTM A325-N bolts on a  $5\frac{1}{2}$  in. gage connecting the beam flange to the seat plate.

Manual  
Part 10

Use a PL  $\frac{5}{8}$  in.  $\times$  7 in.  $\times$  8 in. for the seat plate.

LRFD	ASD
<i>Determine the stiffener plate thickness</i>	<i>Determine the stiffener plate thickness</i>
<i>Determine minimum plate thickness to develop the stiffener-to-seat-plate weld.</i>	<i>Determine minimum plate thickness to develop the stiffener-to-seat-plate weld.</i>
$t_{\min} = 2w = 2(\frac{5}{16} \text{ in.}) = \frac{5}{8} \text{ in.}$	$t_{\min} = 2w = 2(\frac{5}{16} \text{ in.}) = \frac{5}{8} \text{ in.}$
<i>Determine minimum plate thickness for a stiffener with <math>F_y = 36</math> ksi and beam with <math>F_y = 50</math> ksi.</i>	<i>Determine minimum plate thickness for a stiffener with <math>F_y = 36</math> ksi and beam with <math>F_y = 50</math> ksi.</i>
$t_{\min} = \frac{50}{36} t_w = \frac{50}{36} (0.430 \text{ in.})$	$t_{\min} = \frac{50}{36} t_w = \frac{50}{36} (0.430 \text{ in.})$
$= 0.597 \text{ in.} < \frac{5}{8} \text{ in.}$	$= 0.597 \text{ in.} < \frac{5}{8} \text{ in.}$
Use a PL $\frac{5}{8}$ in. $\times$ 7 in. $\times$ 15 in.	Use a PL $\frac{5}{8}$ in. $\times$ 7 in. $\times$ 15 in.

Manual  
Section 10

*Check column web thickness*

$$t_{w \min} = \frac{3.09D}{F_u} = \frac{3.09(5)(2)}{65} = 0.475 \text{ in.}$$

$t_w$  for W14 $\times$ 90 = 0.44 in.

LRFD	ASD
$\phi R_n = 139 \text{ kips} \left( \frac{0.440 \text{ in.}}{0.475 \text{ in.}} \right)$	$\frac{R_n}{\Omega} = 93.0 \text{ kips} \left( \frac{0.440 \text{ in.}}{0.475 \text{ in.}} \right)$
$= 129 \text{ kips} > 125 \text{ kips} \quad \mathbf{o.k.}$	$= 86.1 \text{ kips} > 83.5 \text{ kips} \quad \mathbf{o.k.}$

*Select top angle, bolts, and welds*

Use a L4 $\times$ 4 $\times$  $\frac{1}{4}$  with two  $\frac{3}{4}$  in. diameter ASTM A325-N bolts through the supported-beam leg of the angle. Use a  $\frac{3}{16}$  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).**

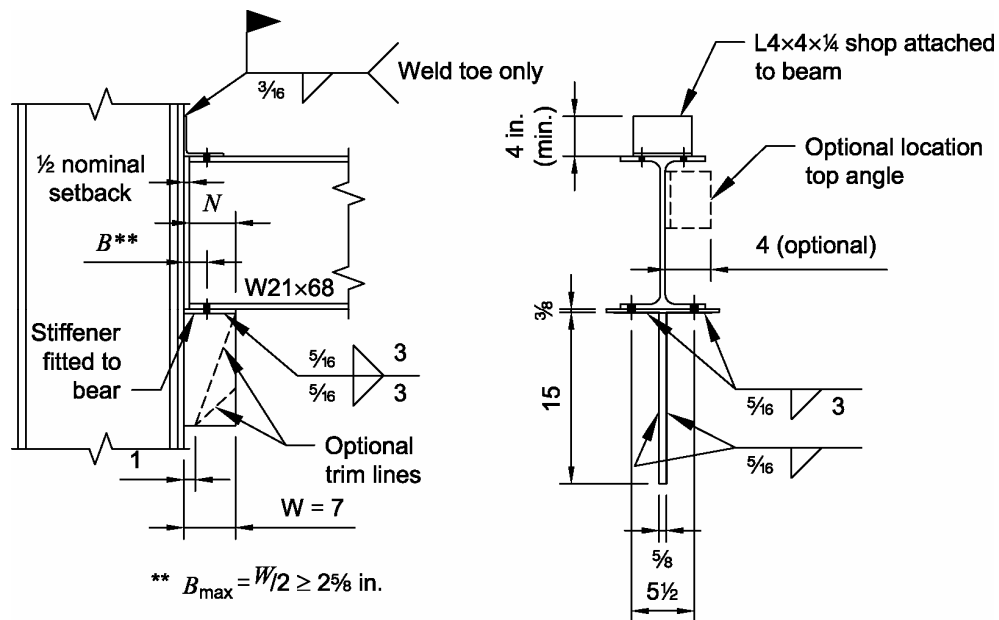
**Given:**

Design a stiffened seated connection between a W21×68 beam and a W14×90 column web to support the following beam end reactions:

$$R_D = 21 \text{ kips}$$

$$R_D = 62.5 \text{ kips}$$

Use 3/4 in. diameter ASTM A325-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
Column W14×90	$t_w = 0.440$ in.			

**Solution:**

LRFD	ASD
$R_u = 1.2(21 \text{ kips}) + 1.6(62.5 \text{ kips}) = 125 \text{ kips}$ <i>Determine stiffener width <math>W</math> required</i> As previously calculated in <b>Example II.A-14</b> , use $W = 7 \text{ in.}$	$R_a = 21 \text{ kips} + 62.5 \text{ kips} = 83.5 \text{ kips}$ <i>Determine stiffener width <math>W</math> required</i> As previously calculated in <b>Example II.A-14</b> , use $W = 7 \text{ in.}$
<i>Determine stiffener length <math>L</math> and stiffener to column web weld size</i> As previously calculated in <b>Example II.A-14</b> , use $L = 15 \text{ in.}$ and $\frac{5}{16} \text{ in.}$ weld size.	<i>Determine stiffener length <math>L</math> and stiffener to column web weld size</i> As previously calculated in <b>Example II.A-14</b> , use $L = 15 \text{ in.}$ and $\frac{5}{16} \text{ in.}$ weld size.

*Determine weld requirements for seat plate*

As previously calculated in **Example II.A-14**, use 3 in. of  $\frac{5}{16}$  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  $3\frac{1}{2}$  in. The PL  $\frac{3}{8}$  in.  $\times$  7 in.  $\times$  8 in. previously selected in **Example II.A-14** will accommodate these bolts.

Manual  
Table 10-8

*Determine stiffener plate thickness*

As previously calculated in **Example II.A-14**, use a PL  $\frac{5}{8}$  in.  $\times$  7 in.  $\times$  15 in.

*Select top angle, bolts, and welds*

Use L4  $\times$  4  $\times$   $\frac{1}{4}$  with two  $\frac{3}{4}$  in. diameter ASTM A325-N bolts through the supported-beam leg of the angle. Use a  $\frac{3}{16}$  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  
Table 10-8

$$t_{w \min} = \frac{3.09D}{F_u} = \frac{3.09(5)}{65} = 0.24 \text{ in.}$$

If both sides of the column web have a stiffened seated connection, then

$$t_{w \min} = \frac{6.19D}{F_u} = \frac{6.18(5)}{65} = 0.48 \text{ in.}$$

Column  $t_w = 0.44 \text{ in.}$ , 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).

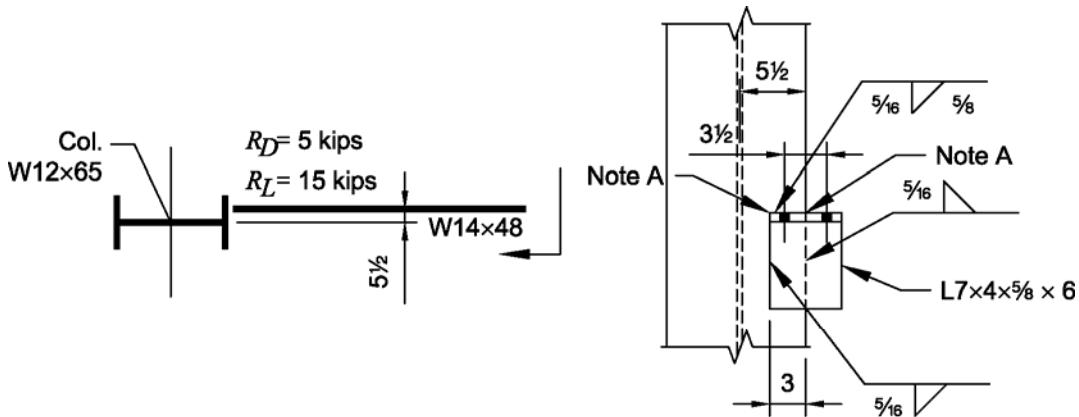
#### Given:

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:

$$R_D = 5 \text{ kips}$$

$$R_L = 15 \text{ kips}$$

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	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Tables 2-3 and 2-4
W12×65	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Angle	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

#### Geometric Properties:

W14×48	$t_w = 0.340 \text{ in.}$	$d = 13.8 \text{ in.}$	$t_f = 0.595 \text{ in.}$	$k = 1.19 \text{ in.}$	Manual Table 1-1
W12×65	$t_f = 0.605 \text{ in.}$				

**Solution:**

LRFD	ASD
$R_u = (1.2)(5 \text{ kips}) + (1.6)(15 \text{ kips}) = 30 \text{ kips}$	$R_a = 5 \text{ kips} + 15 \text{ kips} = 20 \text{ kips}$
<i>Design the seat angle and welds</i>	<i>Design the seat angle and welds</i>
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.
$R_{uR} = \frac{(30 \text{ kips})(3 \text{ in.})}{3 \frac{1}{2} \text{ in.}} = 25.7 \text{ kips}$	$R_{aR} = \frac{(20 \text{ kips})(3 \text{ in.})}{3 \frac{1}{2} \text{ in.}} = 17.1 \text{ kips}$
Selecting the welds on both sides of the seat to resist this force, the total required strength would be $R_u = 51.4 \text{ kips}$	Selecting the welds on both sides of the seat to resist this force, the total required strength would be $R_a = 34.2 \text{ kips}$
For local web yielding,	For local web yielding,
$N_{\min} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$	$N_{\min} = \frac{R_a - (R_1 / \Omega)}{(R_2 / \Omega)} \geq k$
$= \frac{(51.4 \text{ kips}) - (50.4 \text{ kips})}{17.0 \text{ kips/in.}} > 1.19 \text{ in.}$	$= \frac{(34.2 \text{ kips}) - (33.6 \text{ kips})}{11.3 \text{ kips/in.}} > 1.19 \text{ in.}$
$= 0.06 \text{ in.} > 1.19 \text{ in.}$	$= 0.05 \text{ in.} > 1.19 \text{ in.}$
$N_{\min} = 1.19 \text{ in.}$	$N_{\min} = 1.19 \text{ in.}$
For web crippling,	For web crippling,
When $\frac{N}{d} \leq 0.2$	When $\frac{N}{d} \leq 0.2$
$N_{\min} = \frac{R_u - \phi R_3}{\phi R_4}$	$N_{\min} = \frac{R_a - (R_3 / \Omega)}{(R_4 / \Omega)}$
$= \frac{(51.4 \text{ kips}) - (55.2 \text{ kips})}{5.19 \text{ kips/in.}}$	$= \frac{(34.2 \text{ kips}) - (36.8 \text{ kips})}{3.46 \text{ kips/in.}}$
which results in a negative quantity.	which results in a negative quantity
Thus, $N_{req} = 1 \frac{3}{16} \text{ in.}$	Thus, $N_{req} = 1 \frac{3}{16} \text{ in.}$
A 6 in. angle length with a $\frac{5}{8}$ -in. thickness provides	A 6 in. angle length with a $\frac{5}{8}$ -in. thickness provides
$\phi R_n = 55.2 \text{ kips} > 51.4 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = 36.7 \text{ kips} > 34.2 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 9-4Manual  
Table 9-4Manual  
Table 10-6

<p>With a L7×4 (OSL) angle and <math>\frac{5}{16}</math>-in. fillet welds, the weld strength from the tables is</p> <p><math>\phi R_n = 66.7 \text{ kips} &gt; 51.4 \text{ kips} \quad \mathbf{o.k.}</math></p> <p>Use L7×4×<math>\frac{5}{8}</math>×6 in. for the seat angle. Use two <math>\frac{3}{4}</math> in. diameter ASTM A325-N bolts to connect the beam to the seat angle and weld the angle to the column with <math>\frac{5}{16}</math>-in. fillet welds.</p> <p><i>Select top angle, bolts, and welds</i></p> <p>Use L4×4×<math>\frac{1}{4}</math> with two <math>\frac{3}{4}</math>-in. diameter ASTM A325-N bolts through the outstanding leg of the angle.</p> <p>Use a <math>\frac{3}{16}</math>-in. fillet weld along the toe of the angle to the column flange (maximum size permitted by the Specification).</p>	<p>With a L7×4 (OSL) angle and <math>\frac{5}{16}</math>-in. fillet welds, the weld strength from the tables is</p> <p><math>R_n / \Omega = 44.5 \text{ kips} &gt; 34.2 \text{ kips} \quad \mathbf{o.k.}</math></p> <p>Use L7×4×<math>\frac{5}{8}</math>×6 in. for the seat angle. Use two <math>\frac{3}{4}</math> in. diameter ASTM A325-N bolts to connect the beam to the seat angle and weld the angle to the column with <math>\frac{5}{16}</math>-in. fillet welds.</p> <p><i>Select top angle, bolts, and welds</i></p> <p>Use L4×4×<math>\frac{1}{4}</math> with two <math>\frac{3}{4}</math>-in. diameter ASTM A325-N bolts through the outstanding leg of the angle.</p> <p>Use a <math>\frac{3}{16}</math>-in. fillet weld along the toe of the angle to the column flange (maximum size permitted by the Specification).</p>
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Section J2.2b

### Example II.A-17 Single-Plate Connection (conventional – beam-to-column flange)

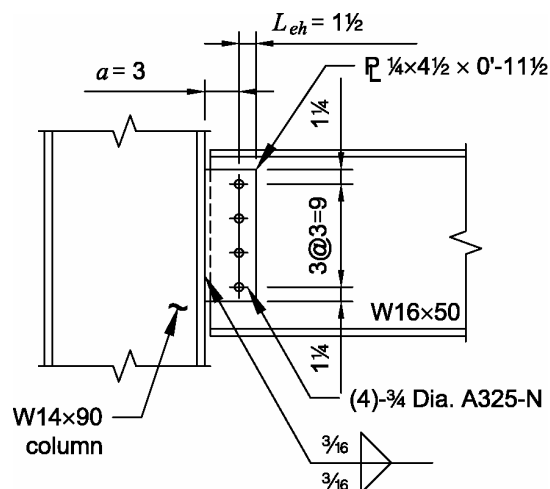
#### Given:

Design a single-plate connection between a W16×50 beam and a W14×90 column flange to support the following beam end reactions:

$$R_D = 8 \text{ kips}$$

$$R_L = 25 \text{ kips}$$

Use  $\frac{3}{4}$ -in. diameter ASTM A325-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	
$R_u = 1.2(8 \text{ kips}) + 1.6(25 \text{ kips}) = 50 \text{ kips}$  <i>Check bolt shear. Check plate for shear yielding, shear rupture, and block shear rupture</i>  Try four rows of bolts, 1/4 in. single plate thickness, and 3/16 in. fillet weld size.  $\phi R_n = 52.2 \text{ kips} > 50 \text{ kips}$ <b>o.k.</b>  <i>Check minimum fillet weld size</i>  Weld size = 3/16 in. <b>o.k.</b>	$R_a = 8 \text{ kips} + 25 \text{ kips} = 33 \text{ kips}$  <i>Check bolt shear. Check plate for shear yielding, shear rupture, and block shear rupture</i>  Try four rows of bolts, 1/4 in. single plate thickness, and 3/16 in. fillet weld size.  $R_n / \Omega = 34.8 \text{ kips} > 33 \text{ kips}$ <b>o.k.</b>  <i>Check minimum fillet weld size</i>  Weld size = 3/16 in. <b>o.k.</b>	Manual Table 10-9          Section J2.2b
<i>Check beam web for bolt bearing. Block shear rupture, shear yielding and shear rupture will not control for an uncoped section.</i>  For an uncoped section,  $\phi R_n = (351 \text{ kips/in.})(0.380 \text{ in.})$  $= 133 \text{ kips} > 50 \text{ kips}$ <b>o.k.</b>	<i>Check beam web for bolt bearing. Block shear rupture, shear yielding and shear rupture will not control for an uncoped section.</i>  For an uncoped section,  $R_n / \Omega = (234 \text{ kips/in.})(0.380 \text{ in.})$  $= 88.9 \text{ kips} > 33 \text{ kips}$ <b>o.k.</b>	Manual Table 10-1



### Example II.A-18 Single-Plate Connection (beam-to-girder web)

#### Given:

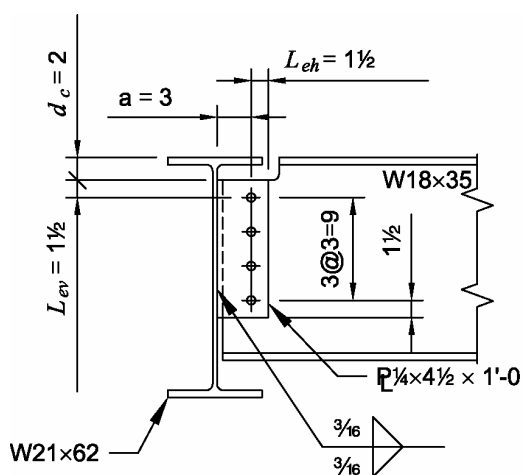
Design a single-plate connection between a W18×35 beam to a W21×62 girder web to support the following beam end reactions:

$$R_D = 6.5 \text{ kips}$$

$$R_L = 20 \text{ kips}$$

Top flange coped 2-in. deep by 4 -in. long,  $L_{ev} = 1\frac{1}{2}$  in.,  $L_{eh} = 1\frac{1}{2}$  in. (assumed to be  $1\frac{1}{4}$  in. for calculation purposes to account for possible underrun in beam length),

Use  $\frac{3}{4}$ -in. diameter ASTM A325-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 Tables 2-3 and 2-4
Girder W21×62	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Plate	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	$c = 4$ in.	$d_c = 2$ in.	$e = 4\frac{1}{2}$ in.	
Girder W21×62	$t_w = 0.400$ in.		$h_o = 15.7$ in.	

**Solution:**

Note: The connection plate dimensions for a  $\frac{3}{4}$  in. diameter bolt in Manual Table 10-9 are based on  $L_{eh} = 1\frac{1}{2}$  in. min. and  $L_{ev}$  (top & bottom) =  $1\frac{1}{4}$  in. min.

Manual  
Part 10

LRFD	ASD
$R_u = 1.2(6.5 \text{ kips}) + 1.6(20 \text{ kips}) = 40 \text{ kips}$  <i>Check bolt shear. Check plate for bolt bearing, shear yielding, shear rupture, and block shear rupture</i>  Try four rows of bolts, $\frac{1}{4}$ in. single plate thickness, and $\frac{3}{16}$ in. fillet weld size.  $\phi R_n = 52.2 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>  <i>Check minimum fillet weld size</i>  Weld size = $\frac{3}{16}$ in. <b>o.k.</b>	$R_a = 6.5 \text{ kips} + 20 \text{ kips} = 26.5 \text{ kips}$  <i>Check bolt shear. Check plate for bolt bearing, shear yielding, shear rupture, and block shear rupture</i>  Try four rows of bolts, $\frac{1}{4}$ in. single plate thickness, and $\frac{3}{16}$ in. fillet weld size.  $R_n / \Omega = 34.8 \text{ kips} > 26.5 \text{ kips}$ <b>o.k.</b>  <i>Check minimum fillet weld size</i>  Weld size = $\frac{3}{16}$ in. <b>o.k.</b>
<i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i>  For coped section, $n = 4$ , $L_{ev} = L_{eh} = 1\frac{1}{2}$ in.  $\phi R_n = (257 \text{ kips/in.})(0.300 \text{ in.})$  $= 77.1 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>	<i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i>  For coped section, $n = 4$ , $L_{ev} = L_{eh} = 1\frac{1}{2}$ in.  $R_n / \Omega = (171 \text{ kips/in.})(0.300 \text{ in.})$  $= 51.3 \text{ kips} > 26.5 \text{ kips}$ <b>o.k.</b>

Manual  
Table 10-9

Section J2.2b

Manual  
Table 10-1

Note: For coped beam sections, the limit states of flexural rupture and local buckling should be checked independently per Part 9. However, for the shallow cope in this example, flexural yielding and local buckling do not govern. For an illustration of these checks, see **Example II.A-4**.

### Example II.A-19 Extended Single-Plate Connection (beam-to-column web)

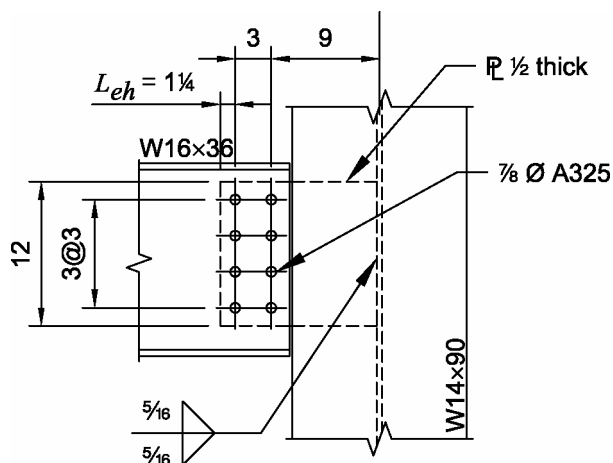
#### Given:

Design the connection between a W16×36 beam and the web of a W14×90 column, to support the following beam end reactions:

$$R_D = 6 \text{ kips}$$

$$R_L = 18 \text{ kips}$$

Use  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts in standard holes. The beam is braced by the floor diaphragm.



#### Material Properties:

W16×36	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Tables 2-3 and 2-4
W14×90	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Plate Material	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

#### Geometric Properties:

W16×36	$t_w = 0.295 \text{ in.}$	$d = 16.0 \text{ in.}$	Manual Table 1-1
W14×90	$t_w = 0.440 \text{ in.}$	$b_f = 14.5 \text{ in.}$	

#### Solution:

*Determine the required strength*

LRFD	ASD
$R_u = 1.2(6 \text{ kips}) + 1.6(18 \text{ kips}) = 36 \text{ kips}$	$R_a = 6 \text{ kips} + 18 \text{ kips} = 24 \text{ kips}$

Determine the distance from the support to the first line of bolts

$$a = 9 \text{ in.}$$

LRFD	ASD
<p>Determine the bearing strength of one bolt on the beam web</p> $\phi r_n = 2.4F_u t_w d_b$ $= 0.75(2.4)(65)(0.295)(0.875)(2.33)$ $= 30.2 \text{ kips}$	<p>Determine the bearing strength of one bolt on the beam web</p> $\frac{r_n}{\Omega} = \frac{2.4F_u t_w d_b}{\Omega}$ $= 2.4(65)(0.295)(0.875)(2.33)/2.00$ $= 20.1 \text{ kips}$
<p>Determine the shear strength of one bolt</p> $\phi r_n = 21.6 \text{ kips}$ <p>Therefore, shear controls over bearing</p> <p>Determine the strength of the bolt group</p> <p>For <math>e = 10.5 \text{ in.}</math>, <math>C = 2.33</math></p> $\phi R_n = C\phi r_n = 2.33(21.6 \text{ kips})$ $= 50.3 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	<p>Determine the shear strength of one bolt</p> $\frac{r_n}{\Omega} = 14.4 \text{ kips}$ <p>Therefore, shear controls over bearing</p> <p>Determine the strength of the bolt group</p> <p>For <math>e = 10.5 \text{ in.}</math>, <math>C = 2.33</math></p> $\frac{R_n}{\Omega} = \frac{Cr_n}{\Omega} = 2.33(14.4 \text{ kips})$ $= 33.6 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 7-5

Manual  
Table 7-8

Determine the maximum plate thickness such that the plate will yield before the bolts shear.

$$M_{max} = 1.25F_{nv}A_b C'$$

$$1.25F_{nv} = 1.25(48 \text{ ksi}) = 60 \text{ ksi}$$

$$A_b = 0.601 \text{ in.}^2$$

$$C' = 26 \text{ in.}$$

$$M_{max} = (60 \text{ ksi})(0.601 \text{ in.}^2)(26 \text{ in.})$$

$$= 938 \text{ kip-in.}$$

$$t_{max} = \frac{6M_{max}}{F_y d^2} = \frac{6(938 \text{ kip-in.})}{36 \text{ ksi}(12 \text{ in.})^2} = 1.09 \text{ in.}$$

Manual  
Table 7-8

Try a plate thickness of 1/2 in.

LRFD	ASD
<p><i>Check flexural strength of the plate</i></p> <p>The required strength is</p> $M_u = R_u e = (36 \text{ kips})(9 \text{ in.})$ $= 324 \text{ in.-kips}$ <p><i>Check shear yielding of the plate</i></p> $\phi R_n = \phi 0.6 F_y A_g$ $\phi = 1.00$ $= (1.00)(0.6)(36 \text{ ksi})(12 \text{ in.})(\frac{1}{2} \text{ in.})$ $= 130 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Determine critical flexural stress in presence of shear stress, <math>f_v</math></i></p> $f_v = \frac{36 \text{ kips}}{(\frac{1}{2} \text{ in.})(12 \text{ in.})} = 6 \text{ ksi}$ $\phi F_{cr} = \sqrt{(\phi F_y)^2 - 3 f_v^2}$ $= \sqrt{(0.9 \times 36 \text{ ksi})^2 - 3(6 \text{ ksi})^2}$ $= 30.7 \text{ ksi}$ $\phi M_n = \phi F_{cr} Z = (30.7 \text{ ksi}) \frac{(\frac{1}{2} \text{ in.})(12 \text{ in.})^2}{4}$ $= 503 \text{ kip-in.} > 324 \text{ kip-in.} \quad \mathbf{o.k.}$	<p><i>Check flexural strength of the plate</i></p> <p>The required strength is</p> $M_a = R_a e = (24 \text{ kips})(9 \text{ in.})$ $= 216 \text{ in.-kips}$ <p><i>Check shear yielding of the plate</i></p> $R_n / \Omega = 0.6 F_y A_g / \Omega$ $\Omega = 1.50$ $= (0.6)(36 \text{ ksi})(12 \text{ in.})(\frac{1}{2} \text{ in.}) / 1.50$ $= 86.4 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Determine critical flexural stress in presence of shear stress, <math>f_v</math></i></p> $f_v = \frac{24 \text{ kips}}{(\frac{1}{2} \text{ in.})(12 \text{ in.})} = 4 \text{ ksi}$ $\frac{F_{cr}}{\Omega} = \sqrt{\left(\frac{F_y}{\Omega}\right)^2 - 3 f_v^2}$ $= \sqrt{\left(\frac{36 \text{ ksi}}{1.67}\right)^2 - 3(4 \text{ ksi})^2}$ $= 20.4 \text{ ksi}$ $\frac{M_n}{\Omega} = \frac{F_{cr}}{\Omega} Z = (20.4 \text{ ksi}) \frac{(\frac{1}{2} \text{ in.})(12 \text{ in.})^2}{4}$ $= 367 \text{ kip-in.} > 216 \text{ kip-in.} \quad \mathbf{o.k.}$

Eqn. J4-3

$$Z_{net} = \frac{t_p d^2}{4} \left[ 1 - \frac{d_b + 0.125}{3} \right] = \frac{0.5(12)^2}{4} \left[ 1 - \frac{0.875 + 0.125}{3} \right] = 12.0 \text{ in.}^3$$

LRFD	ASD
For flexural rupture, $\phi M_n = \phi F_u Z_{net}$ $\phi = 0.75$ $\phi F_u Z_{net} = 0.75(58 \text{ ksi})(12.0 \text{ in.}^3)$ $= 522 \text{ in.-kips} > 324 \text{ in.-kips} \quad \mathbf{o.k.}$	For flexural rupture, $M_n / \Omega = F_u Z_{net} / \Omega$ $\Omega = 2.00$ $F_u Z_{net} / \Omega = \frac{(58 \text{ ksi})(12.0 \text{ in.}^3)}{2.00}$ $= 348 \text{ in.-kips} > 216 \text{ in.-kips} \quad \mathbf{o.k.}$

Check shear rupture of the plate

$$A_n = (t_p) [d - n(d_b + 0.125 \text{ in.})]$$

$$= (\frac{1}{2} \text{ in.}) [(12 \text{ in.}) - (4)(0.875 \text{ in.} + 0.125 \text{ in.})] = 4 \text{ in.}^2$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.6 F_u A_n$ $= 0.75(0.6)(58 \text{ ksi})(4 \text{ in.}^2)$ $= 104 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $R_n / \Omega = \frac{0.6 F_u A_n}{\Omega}$ $= \frac{0.6(58 \text{ ksi})(4 \text{ in.}^2)}{2.00}$ $= 69.6 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$
<p>Check block shear rupture of the plate</p> <p><math>n = 4, L_{ev} = 1\frac{1}{2} \text{ in.}, L_{eh} = 1\frac{1}{4} \text{ in.},</math></p> $\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	<p>Check block shear rupture of the plate</p> <p><math>n = 4, L_{ev} = 1\frac{1}{2} \text{ in.}, L_{eh} = 1\frac{1}{4} \text{ in.},</math></p> $\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$
<p>Tension rupture component</p> $\phi F_u A_{nt} = 35.4 \text{ kips/in.} (\frac{1}{2} \text{ in.})$	<p>Tension rupture component</p> $F_u A_{nt} / \Omega = 23.6 \text{ kips/in.} (\frac{1}{2} \text{ in.})$
<p>Shear yielding component</p> $\phi 0.6 F_y A_{gv} = 170 \text{ kips/in.} (\frac{1}{2} \text{ in.})$	<p>Shear yielding component</p> $0.6 F_y A_{gv} / \Omega = 113 \text{ kips/in.} (\frac{1}{2} \text{ in.})$
<p>Shear component</p> $\phi 0.6 F_u A_{nv} = 194 \text{ kips/in.} (\frac{1}{2} \text{ in.})$	<p>Shear component</p> $0.6 F_u A_{nv} / \Omega = 129 \text{ kips/in.} (\frac{1}{2} \text{ in.})$
$\phi R_n = (170 \text{ kips/in.} + 35.4 \text{ kips/in.}) (\frac{1}{2} \text{ in.})$ $= 103 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = (113 \text{ kips/in.} + 23.6 \text{ kips/in.}) (\frac{1}{2} \text{ in.})$ $= 68.3 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$

Eqn. J4-4

Eqn. J4-5

Manual  
Table 9-3aManual  
Table 9-3bManual  
Table 9-3c

*Check local buckling of the plate*

This check is analogous to the local buckling check for doubly coped beams as illustrated previously in the Manual Part 9 where  $c = 6$  in. and  $d_c = 1\frac{1}{2}$  in. at both the top and bottom flanges

$$F_{cr} = F_y Q$$

$$\lambda = \frac{h_o \sqrt{F_y}}{10t_w \sqrt{475 + 280 \left(\frac{h_o}{c}\right)^2}}$$

$$\lambda = \frac{(12 \text{ in.}) \sqrt{36 \text{ ksi}}}{10 \left(\frac{1}{2}\right) \sqrt{475 + 280 \left(\frac{12.0 \text{ in.}}{9 \text{ in.}}\right)^2}} = 0.462$$

$\lambda < 0.7$ , therefore,  $Q = 1$

$$F_{cr} = F_y$$

Therefore, plate buckling does not control.

### Example II.A-20 All-Bolted Single-Plate Shear Splice

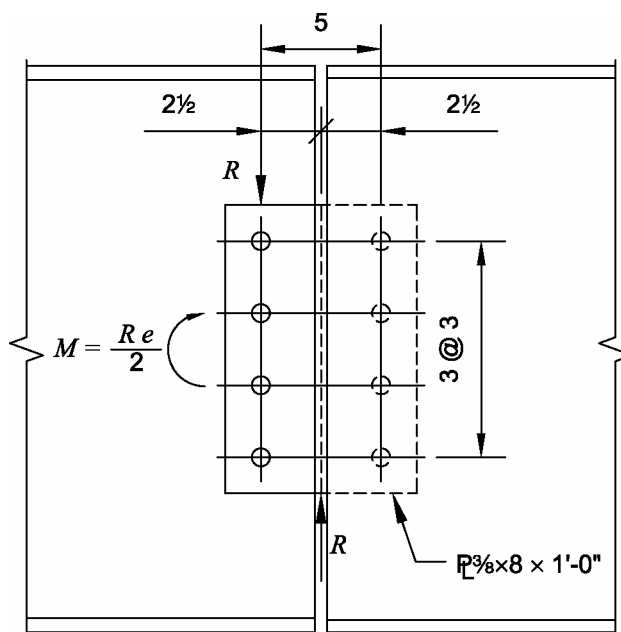
#### Given:

Design an all-bolted single-plate shear splice between a W24×55 beam and a W24×68 beam.

$$R_D = 10 \text{ kips}$$

$$R_L = 30 \text{ kips}$$

Use  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts with 5 in. between vertical bolt rows.



#### Material Properties:

W24×55	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Tables 2-3 and 2-4
W24×68	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Splice Plate	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

#### Geometric Properties:

Beam W24×55	$t_w = 0.395 \text{ in.}$	Manual Table 1-1
Beam W24×68	$t_w = 0.415 \text{ in.}$	



**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,  $R_u e$  or  $R_a e$ .

Using a symmetrical splice, each bolt group will carry one-half the eccentric moment. Thus, the eccentricity on each bolt group  $e/2 = 2\frac{1}{2}$  in.

LRFD	ASD	
$R_u = 1.2(10 \text{ kips}) + 1.6(30 \text{ kips})$ $= 60 \text{ kips}$	$R_a = 10 \text{ kips} + 30 \text{ kips}$ $= 40 \text{ kips}$	
For bolt shear,	For bolt shear,	Manual Table 7-1
$\phi r_n = 21.6 \text{ kips/bolt.}$	$r_n/\Omega = 14.4 \text{ kips/bolt.}$	
For bearing on the web of the W24×55,	For bearing on the web of the W24×55,	Manual Table 7-6
$\phi r_n = (102 \text{ kips/in.})(0.395 \text{ in.})$ $= 40.3 \text{ kips/bolt}$	$r_n/\Omega = (68.3 \text{ kips/in.})(0.395 \text{ in.})$ $= 27.0 \text{ kips/bolt}$	
Since bolt shear is more critical,	Since bolt shear is more critical,	
$C_{\min} = \frac{R_u}{\phi r_n} = \frac{60 \text{ kips}}{21.6 \text{ kips/bolt}} = 2.78$	$C_{\min} = \frac{R_a}{r_n/\Omega} = \frac{40 \text{ kips}}{14.4 \text{ kips/bolt}} = 2.78$	
with $\theta = 0^\circ$ and $e_x = 2\frac{1}{2}$ in., a four-bolt connection provides	with $\theta = 0^\circ$ and $e_x = 2\frac{1}{2}$ in., a four-bolt connection provides	Manual Table 7-7
$C = 3.07 > 2.78$ o.k.	$C = 3.07 > 2.78$ o.k.	
<i>Design splice plate</i>	<i>Design splice plate</i>	
Try PL $\frac{3}{8}$ in. × 8 in. × 1'-0".	Try PL $\frac{3}{8}$ in. × 8 in. × 1'-0".	
<i>Check flexure of the plate</i>	<i>Check flexure of the plate</i>	
$M_u = \frac{R_u e}{2} = \frac{(60 \text{ kips})(5 \text{ in.})}{2} = 150 \text{ in.-kips}$	$M_a = \frac{R_a e}{2} = \frac{(40 \text{ kips})(5 \text{ in.})}{2} = 100 \text{ in.-kips}$	
For flexural yielding,	For flexural yielding,	
$\phi M_n = \phi F_y Z_x$	$M_n/\Omega = F_y Z_x/\Omega$	Manual Part. 15
$\phi = 0.90$	$\Omega = 1.67$	
$= (0.90)(36 \text{ ksi}) \left[ \frac{(\frac{3}{8} \text{ in.})(12 \text{ in.})^2}{4} \right]$	$= (36 \text{ ksi}) \left[ \frac{(\frac{3}{8} \text{ in.})(12 \text{ in.})^2}{4} \right] / 1.67$	
$= 437 \text{ in.-kips} > 150 \text{ in.-kips}$	$= 291 \text{ in.-kips} > 100 \text{ in.-kips}$	

For flexure rupture,

$$Z_{net} \approx \frac{t}{4} \left[ d^2 - \frac{s^2 n (n^2 - 1) (d_n + \frac{1}{16} \text{ in.})}{d} \right]$$

$$\approx \frac{\frac{3}{8} \text{ in.}}{4} \left[ (12 \text{ in.})^2 - \frac{(3 \text{ in.})^2 (4) (4^2 - 1) (\frac{15}{16} \text{ in.} + \frac{1}{16} \text{ in.})}{12 \text{ in.}} \right] = 9.28 \text{ in.}^3$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi M_n = \phi F_u Z_{net}$	$M_n / \Omega = F_u Z_{net} / \Omega$
$= (0.75)(58 \text{ ksi})(9.28 \text{ in.}^3)$	$= (58 \text{ ksi})(9.28 \text{ in.}^3) / 2.00$
$= 404 \text{ in.-kips} > 150 \text{ in.-kips}$ <b>o.k.</b>	$= 269 \text{ in.-kips} > 100 \text{ in.-kips}$ <b>o.k.</b>
<i>Check shear yielding of the plate</i>	<i>Check shear yielding of the plate</i>
$\phi R_n = \phi (0.6 F_y A_g)$	$R_n / \Omega = (0.6 F_y A_g) / \Omega$
$\phi = 1.00$	$\Omega = 1.50$
$= 1.00 (0.6 (36 \text{ ksi}) (12 \text{ in.}) (\frac{3}{8} \text{ in.}))$	$= (0.6 (36 \text{ ksi}) (12 \text{ in.}) (\frac{3}{8} \text{ in.})) / 1.50$
$= 97.2 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	$= 64.8 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>
<i>Check shear rupture of the plate</i>	<i>Check shear rupture of the plate</i>
$\phi R_n = \phi (0.6 F_u A_n)$	$R_n / \Omega = (0.6 F_u A_n) / \Omega$
$\phi = 0.75$	$\Omega = 2.00$
$= 0.75 (0.6) (58 \text{ ksi}) [12 \text{ in.} - 4(1 \text{ in.})] (\frac{3}{8} \text{ in.})$	$= 0.6 (58 \text{ ksi}) [12 \text{ in.} - 4(1 \text{ in.})] (\frac{3}{8} \text{ in.}) / 2.00$
$= 78.3 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	$= 52.2 \text{ kips} > 40 \text{ kips}$ <b>o.k.</b>
<i>Check block shear rupture of the plate</i>	<i>Check block shear rupture of the plate</i>
$L_{eh} = L_{ev} = 1\frac{1}{2} \text{ in.},$	$L_{eh} = L_{ev} = 1\frac{1}{2} \text{ in.},$
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min \left( \phi 0.6 F_y A_{gv}, \phi F_u A_{nv} \right)$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min \left( \frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega} \right)$
Because maximum shear stress occurs at one row of bolts	Because maximum shear stress occurs at one row of bolts
$U_{bs} = 1.00$	$U_{bs} = 1.00$

Eqn. J4-3

Eqn. J4-4

Eqn. J4-5

Commentary  
Section J4.3

<p>Tension rupture component</p> $\phi F_u A_{nt} = 43.5 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear yielding component</p> $\phi 0.6 F_y A_{gv} = 170 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $\phi 0.6 F_u A_{nv} = 183 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ $\phi R_n = (170 \text{ kips/in.} + 43.5 \text{ kips/in.}) (\frac{3}{8} \text{ in.})$ $= 80.1 \text{ kips} > 60 \text{ kips} \quad \mathbf{o.k.}$ <p>Use PL <math>\frac{3}{8}</math> in. <math>\times</math> 8 in. <math>\times</math> 1'-0</p>	<p>Tension rupture component</p> $F_u A_{nt} / \Omega = 29.0 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear yielding component</p> $0.6 F_y A_{gv} / \Omega = 113 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $0.6 F_u A_{nv} / \Omega = 122 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ $R_n / \Omega = (113 \text{ kips/in.} + 29.0 \text{ kips/in.}) (\frac{3}{8} \text{ in.})$ $= 53.3 \text{ kips} > 40 \text{ kips} \quad \mathbf{o.k.}$ <p>Use PL <math>\frac{3}{8}</math> in. <math>\times</math> 8 in. <math>\times</math> 1'-0</p>	<p>Manual Table 9-3a</p>
		<p>Manual Table 9-3b</p>
		<p>Manual Table 9-3c</p>

## Example II.A-21 Bolted/Welded Single-Plate Shear Splice

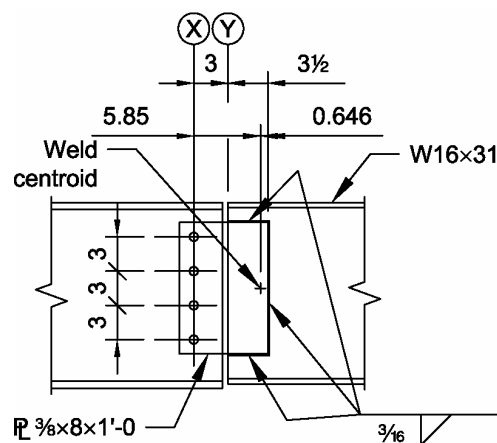
### Given:

Design a single-plate shear splice between a W16×31 beam and W16×50 beam (not illustrated) to support the following beam end reactions:

$$R_D = 8.0 \text{ kips}$$

$$R_L = 24.0 \text{ kips}$$

Use  $\frac{3}{4}$ -in. diameter ASTM A325-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	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Tables 2-3 and 2-4
W16×50	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Splice Plate	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

### Geometric Properties:

W16×31	$t_w = 0.275 \text{ in.}$	Manual Table 1-1
W16×50	$t_w = 0.380 \text{ in.}$	

### Solution:

LRFD	ASD
$R_u = 1.2(8 \text{ kips}) + 1.6(24 \text{ kips}) = 48 \text{ kips}$	$R_a = 8 \text{ kips} + 24 \text{ kips} = 32 \text{ kips}$

### 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  $\frac{3}{8}$  in.×8 in.×1'-0

$$k = \frac{kl}{l} = \frac{3\frac{1}{2} \text{ in.}}{12 \text{ in.}} = 0.292$$

By interpolation, with  $\theta = 0^\circ$ ,  $x = 0.0538$  and  $xl = 0.646$  in.

$$al = 6\frac{1}{2} \text{ in.} - 0.646 \text{ in.} = 5.85 \text{ in.}, \quad a = \frac{al}{l} = \frac{5.85 \text{ in.}}{12 \text{ in.}} = 0.488$$

By interpolation,  $C = 2.14$  and the required weld size is

Manual  
Table 8-8

LRFD	ASD
$D_{req} = \frac{P_u}{\phi C C_1 l} = \frac{(48 \text{ kips})}{0.75(2.14)(1.0)(12 \text{ in.})}$ $= 2.49 \rightarrow 3 \text{ sixteenths}$ <p>The minimum weld size is <math>\frac{3}{16}</math> in.</p> <p>Use a <math>\frac{3}{16}</math>-in. weld size.</p>	$D_{req} = \frac{P_a \Omega}{C C_1 l} = \frac{(32 \text{ kips})(2.0)}{(2.14)(1.0)(12 \text{ in.})}$ $= 2.49 \rightarrow 3 \text{ sixteenths}$ <p>The minimum weld size is <math>\frac{3}{16}</math> in.</p> <p>Use a <math>\frac{3}{16}</math>-in. weld size.</p>

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:

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(3 \text{ sixteenths})}{65 \text{ ksi}} = 0.143 < 0.275 \text{ in.}$$

**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
<p>For bolt shear</p> $\phi r_n = 15.9 \text{ kips/bolt}$ <p>For bearing on the <math>\frac{3}{8}</math>-in. thick single plate, conservatively use the design values provided for <math>L_e = 1\frac{1}{2}</math> in.</p> $\phi r_n = (44.0 \text{ kips/in./bolt})(\frac{3}{8} \text{ in.})$ $\phi r_n = 16.5 \text{ kips/bolt}$	<p>For bolt shear</p> $r_n / \Omega = 10.6 \text{ kips/bolt}$ <p>For bearing on the <math>\frac{3}{8}</math>-in. thick single plate, conservatively use the design values provided for <math>L_e = 1\frac{1}{2}</math> in.</p> $r_n / \Omega = (29.4 \text{ kips/in./bolt})(\frac{3}{8} \text{ in.})$ $r_n / \Omega = 11.0 \text{ kips/bolt}$

Manual  
Table 7-1

Manual  
Table 7-6

<p>Since bolt shear is more critical than bearing,</p> $n_{\min} = \frac{R_u}{\phi r_n}$ $= \frac{48 \text{ kips}}{15.9 \text{ kips/bolt}}$ $= 3.02 \rightarrow 4 \text{ bolts}$	<p>Since bolt shear is more critical than bearing,</p> $n_{\min} = \frac{R_u}{r_n / \Omega}$ $= \frac{32 \text{ kips}}{10.6 \text{ kips/bolt}}$ $= 3.02 \rightarrow 4 \text{ bolts}$
--	--

*Design the single plate*

As before, try a PL  $\frac{3}{8}$  in.  $\times$  8 in.  $\times$  1'-0

LRFD	ASD
<p><i>Check flexure of the plate</i></p> $M_u = R_u e = (48 \text{ kips})(5.85 \text{ in.}) = 281 \text{ in.-kips}$ <p>For flexural yielding</p> $\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= (0.9)(36 \text{ ksi}) \left[ \frac{(\frac{3}{8} \text{ in.})(12 \text{ in.})^2}{4} \right]$ $= 437 \text{ in.-kips} > 281 \text{ in.-kips} \quad \mathbf{o.k.}$	<p><i>Check flexure of the plate</i></p> $M_a = R_a e = (32 \text{ kips})(5.85 \text{ in.}) = 187 \text{ in.-kips}$ <p>For flexural yielding</p> $\Omega = 1.67$ $M_n / \Omega = F_y Z_x / \Omega$ $= (36 \text{ ksi}) \left[ \frac{(\frac{3}{8} \text{ in.})(12 \text{ in.})^2}{4} \right] / 1.67$ $= 291 \text{ in.-kips} > 187 \text{ in.-kips} \quad \mathbf{o.k.}$
<p><i>Check flexural rupture</i></p> $Z_{net} = Z_x = \frac{t_p d^2}{4}$ $= \frac{(\frac{3}{8} \text{ in.})(12 \text{ in.})^2}{4} = 13.5 \text{ in.}^3$ $\phi M_n = \phi F_u Z_{net}$ $\phi = 0.75$ $= 0.75(58 \text{ ksi})(13.5 \text{ in.}^3)$ $= 587 \text{ in.-kips} > 281 \text{ in.-kips} \quad \mathbf{o.k.}$	<p><i>Check flexural rupture</i></p> $Z_{net} = Z_x = \frac{t_p d^2}{4}$ $= \frac{(\frac{3}{8} \text{ in.})(12 \text{ in.})^2}{4} = 13.5 \text{ in.}^3$ $M_n / \Omega = \frac{F_u S_{net}}{\Omega}$ $\Omega = 2.00$ $= \frac{(58 \text{ ksi})(13.5 \text{ in.}^3)}{2.00}$ $= 392 \text{ in.-kips} > 187 \text{ in.-kips} \quad \mathbf{o.k.}$

Manual  
Part 15

Manual  
Part 9

<p><i>Check shear yielding of the plate</i></p> $\phi R_n = \phi(0.6F_y A_g)$ $\phi = 1.00$ $= 1.00[0.6(36 \text{ ksi})(12 \text{ in.})(\frac{3}{8} \text{ in.})]$ $= 97.2 \text{ kips} > 48 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear yielding of the plate</i></p> $R_n / \Omega = (0.6F_y A_g) / \Omega$ $\Omega = 1.50$ $= [0.6(36 \text{ ksi})(12 \text{ in.})(\frac{3}{8} \text{ in.})] / 1.50$ $= 64.8 \text{ kips} > 32 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-3
<p><i>Check shear rupture of the plate</i></p> $\phi R_n = \phi(0.6F_u A_n)$ $\phi = 0.75$ $= 0.75(0.6)(58 \text{ ksi})[12 \text{ in.} - 4(\frac{7}{8} \text{ in.})](\frac{3}{8} \text{ in.})$ $= 83.2 \text{ kips} > 48 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear rupture of the plate</i></p> $R_n / \Omega = (0.6F_u A_n) / \Omega$ $\Omega = 2.00$ $= 0.6(58 \text{ ksi})[12 \text{ in.} - 4(\frac{7}{8} \text{ in.})](\frac{3}{8} \text{ in.}) / 2.0$ $= 55.5 \text{ kips} > 32 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-4
<p><i>Check block shear rupture of the plate</i></p> $L_{eh} = L_{ev} = 1\frac{1}{2} \text{ in.},$ <p>Thus,</p> $\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6F_y A_{gv}, \phi F_u A_{nv})$ $U_{bs} = 1.0$ <p>Tension rupture component</p> $\phi F_u A_{nt} = 46.2 \text{ kips/in.}(\frac{3}{8} \text{ in.})$ <p>Shear yielding component</p> $\phi 0.6F_y A_{gv} = 170 \text{ kips/in.}(\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $\phi 0.6F_u A_{nv} = 194 \text{ kips/in.}(\frac{3}{8} \text{ in.})$ $\phi R_n = (170 \text{ kips/in.} + 46.2 \text{ kips/in.})(\frac{3}{8} \text{ in.})$ $= 81.1 \text{ kips} > 48 \text{ kips} \quad \mathbf{o.k.}$ <p>Use PL <math>\frac{3}{8} \text{ in.} \times 8 \text{ in.} \times 1'-0</math></p>	<p><i>Check block shear rupture of the plate</i></p> $L_{eh} = L_{ev} = 1\frac{1}{2} \text{ in.},$ <p>Thus,</p> $\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$ $U_{bs} = 1.0$ <p>Tension rupture component</p> $F_u A_{nt} / \Omega = 30.8 \text{ kips/in.}(\frac{3}{8} \text{ in.})$ <p>Shear yielding component</p> $0.6F_y A_{gv} / \Omega = 113 \text{ kips/in.}(\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $0.6F_u A_{nv} / \Omega = 129 \text{ kips/in.}(\frac{3}{8} \text{ in.})$ $R_n / \Omega = (113 \text{ kips/in.} + 30.8 \text{ kips/in.})(\frac{3}{8} \text{ in.})$ $= 53.9 \text{ kips} > 32 \text{ kips} \quad \mathbf{o.k.}$ <p>Use PL <math>\frac{3}{8} \text{ in.} \times 8 \text{ in.} \times 1'-0</math></p>	Eqn. J4-5
		Manual Table 9-3a
		Manual Table 9-3b
		Manual Table 9-3c

### Example II.A-22 Bolted Bracket Plate Design

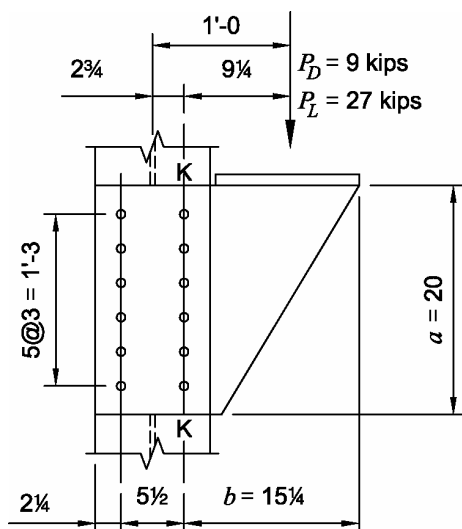
Given:

Design a bracket plate to support the following loads:

$$P_D = 9 \text{ kips}$$

$$P_L = 27 \text{ kips}$$

Use  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts in standard holes



Material Properties:

Plate Material

ASTM A36

$F_y = 36 \text{ ksi}$

$F_u = 58 \text{ ksi}$

Manual  
Table 2-4

**Solution:**

LRFD	ASD
$R_u = 1.2(9 \text{ kips}) + 1.6(27 \text{ kips}) = 54 \text{ kips}$	$R_a = 9 \text{ kips} + 27 \text{ kips} = 36 \text{ kips}$
<i>Design bolts</i>	<i>Design bolts</i>
$C_{\min} = \frac{R_u}{\phi r_n}$	$C_{\min} = \frac{R_a}{r_n / \Omega}$
$= \frac{54 \text{ kips}}{21.6 \text{ kips/bolt}}$	$= \frac{36 \text{ kips}}{14.4 \text{ kips/bolt}}$
$= 2.50$	$= 2.50$
For $\theta = 0^\circ$ , a $5\frac{1}{2}$ in. gage with $s = 3$ in., $e_x = 12$ in., and $n = 6$	For $\theta = 0^\circ$ , a $5\frac{1}{2}$ in. gage with $s = 3$ in., $e_x = 12$ in., and $n = 6$
$C = 4.53 > 2.50$ <b>o.k.</b>	$C = 4.53 > 2.50$ <b>o.k.</b>

Manual  
Table 7-9



<p><i>Check bolt bearing</i></p> <p>Try PL<math>\frac{3}{8}</math> in. <math>\times</math> 20 in., <math>L_e \geq 2 \frac{1}{4}</math> in.</p> $\phi r_n = (91.4 \text{ kips/bolt})(\frac{3}{8} \text{ in.})$ $= 34.3 \text{ kips/bolt}$ <p>Since this is greater than the single-shear strength of one bolt, bolt bearing is not critical.</p>	<p><i>Check bolt bearing</i></p> <p>Try PL<math>\frac{3}{8}</math> in. <math>\times</math> 20 in., <math>L_e \geq 2 \frac{1}{4}</math> in.</p> $r_n / \Omega = (60.9 \text{ kips/bolt})(\frac{3}{8} \text{ in.})$ $= 22.8 \text{ kips/bolt}$ <p>Since this is greater than the single-shear strength of one bolt, bolt bearing is not critical.</p>
<p><i>Check flexure in the bracket plate</i></p> <p>On line K, the required strength <math>M_u</math> is</p> $M_u = P_u e_b = (54 \text{ kips})(12 \text{ in.} - 2\frac{3}{4} \text{ in.})$ $= 500 \text{ in.-kips}$ <p>For flexural yielding on line K,</p> $\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(36 \text{ ksi}) \left( \frac{(\frac{3}{8} \text{ in.})(20 \text{ in.})^2}{4} \right)$ $= 1220 \text{ in.-kips} > 500 \text{ in.-kips} \quad \mathbf{o.k.}$	<p><i>Check flexure in the bracket plate</i></p> <p>On line K, the required strength <math>M_a</math> is</p> $M_a = P_a e_b = (36 \text{ kips})(12 \text{ in.} - 2\frac{3}{4} \text{ in.})$ $= 333 \text{ in.-kips}$ <p>For flexural yielding on line K,</p> $\Omega = 1.67$ $M_n / \Omega = F_y Z_x / \Omega$ $= \frac{(36 \text{ ksi}) \left( \frac{(\frac{3}{8} \text{ in.})(20 \text{ in.})^2}{4} \right)}{1.67}$ $= 808 \text{ in.-kips} > 333 \text{ in.-kips} \quad \mathbf{o.k.}$

For flexural rupture on line K,

$$Z_{net} = \frac{3}{8} \text{ in.} \left[ \frac{(20 \text{ in.})^2}{4} - 2(1.5 + 4.5 + 7.5)(1 \text{ in.}) \right]$$

$$= 27.4 \text{ in.}^3$$

LRFD	ASD
$\phi = 0.75$ $\phi M_n = \phi F_u Z_{net} = 0.75(58 \text{ ksi})(27.4 \text{ in.}^3)$ $= 1190 \text{ in.-kips} > 500 \text{ in.-kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $M_n / \Omega = \frac{F_u Z_{net}}{\Omega} = \frac{(58 \text{ ksi})(27.4 \text{ in.}^3)}{2.00}$ $= 795 \text{ in.-kips} > 333 \text{ in.-kips} \quad \mathbf{o.k.}$

For flexural yielding on the free edge of the triangular plate,

$$z = 1.39 - 2.2\left(\frac{b}{a}\right) + 1.27\left(\frac{b}{a}\right)^2 - 0.25\left(\frac{b}{a}\right)^3$$

$$= 1.39 - 2.2\left(\frac{15\frac{1}{4}\text{ in.}}{20\text{ in.}}\right) + 1.27\left(\frac{15\frac{1}{4}\text{ in.}}{20\text{ in.}}\right)^2 - 0.25\left(\frac{15\frac{1}{4}\text{ in.}}{20\text{ in.}}\right)^3 = 0.340$$

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi P_n = \phi F_y z b t$	$P_n / \Omega = \frac{F_y z b t}{\Omega}$
$= 0.90(36\text{ ksi})(0.340)(15.25\text{ in.})(0.375\text{ in.})$	$= \frac{(36\text{ ksi})(0.286)(15.25\text{ in.})(0.375\text{ in.})}{1.67}$
$= 63\text{ kips} > 54.0\text{ kips}$ <b>o.k.</b>	$= 41.9\text{ kips} > 36.0\text{ kips}$ <b>o.k.</b>

Check local buckling of the bracket plate

$$\frac{b}{a} = \frac{15\frac{1}{4}\text{ in.}}{20\text{ in.}} = 0.763 < 1.0$$

Since  $0.5 \leq \frac{b}{a} < 1.0$ ,

$$t_{\min} = b \left( \frac{\sqrt{F_y}}{250} \right) = (15\frac{1}{4}\text{ in.}) \left( \frac{\sqrt{36\text{ ksi}}}{250} \right) = 0.366\text{ in.} < \frac{3}{8}\text{ in.} \quad \text{o.k.}$$

LRFD	ASD
<i>Check shear yielding of the bracket plate</i>	<i>Check shear yielding of the bracket plate</i>
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = \phi(0.6F_y)A_g$	$R_n / \Omega = \frac{(0.6F_y)A_g}{\Omega}$
$= 1.0(0.6)(36\text{ ksi})(20\text{ in.})(\frac{3}{8}\text{ in.})$	$= \frac{0.6(36\text{ ksi})(20\text{ in.})(\frac{3}{8}\text{ in.})}{1.50}$
$= 162\text{ kips} > 54.0\text{ kips}$ <b>o.k.</b>	$= 108\text{ kips} > 36.0\text{ kips}$ <b>o.k.</b>
<i>Check shear rupture of the bracket plate</i>	<i>Check shear rupture of the bracket plate</i>
$\phi R_n = \phi(0.6F_u)A_{nv}$	$R_n / \Omega = \frac{0.6F_u A_{nv}}{\Omega}$
$\phi = 0.75$	$\Omega = 2.00$
$= 0.75(0.6)(58\text{ ksi})[20\text{ in.} - 6(1\text{ in.})](\frac{3}{8}\text{ in.})$	$= \frac{0.6(58\text{ ksi})[20\text{ in.} - 6(1\text{ in.})](\frac{3}{8}\text{ in.})}{2.00}$
$= 137\text{ kips} > 54.0\text{ kips}$ <b>o.k.</b>	$= 91.4\text{ kips} > 36.0\text{ kips}$ <b>o.k.</b>

Eqn. J4-3

Eqn. J4-4

Check block shear rupture of the bracket plate (shear plane on line K, tension plane across bottom two bolts)

Since the eccentricity reduces the 12 bolts to 4.53 effective, the equivalent block shear rupture force applied concentric to the bolt group is

Manual  
Table 7-9

LRFD	ASD
$R_u = (54.0 \text{ kips})(12/4.53) = 143 \text{ kips.}$	$R_a = (36.0 \text{ kips})(12/4.53) = 95.4 \text{ kips}$

$$A_{nt} = \left[ 5 \frac{1}{2} \text{ in.} + 2 \frac{1}{4} \text{ in.} - 1 \frac{1}{2} (1 \text{ in.}) \right] \left( \frac{3}{8} \text{ in.} \right) = 2.34 \text{ in.}^2$$

$$A_{gv} = (20 \text{ in.} - 2 \frac{1}{2} \text{ in.}) \left( \frac{3}{8} \text{ in.} \right) = 6.56 \text{ in.}^2$$

$$A_{nv} = 6.56 \text{ in.}^2 - 5.5 (1 \text{ in.}) \left( \frac{3}{8} \text{ in.} \right) = 4.50 \text{ in.}^2$$

Compare

$$0.6F_u A_{nt} = 0.6(58 \text{ ksi})(2.34 \text{ in.}^2) = 157 \text{ kips}$$

$$0.6F_y A_{gv} = 0.6(36 \text{ ksi})(6.56 \text{ in.}^2) = 142 \text{ kips}$$

Shear on the gross area controls, thus;

The connection has two vertical rows of bolts, therefore  $U_{bs} = 0.5$

$$R_n = 0.6F_y A_{gv} + U_{bs} F_u A_{nt} = 142 \text{ kips} + 0.5(58 \text{ ksi})(2.34 \text{ in.}^2) = 210 \text{ kips}$$

Commentary  
Section J4.3

Eqn. J4-5

LRFD	ASD
$\phi R_n = 0.75(210 \text{ kips}) = 157 \text{ kips}$	$R_n / \Omega = \frac{(210 \text{ kips})}{2.00} = 105 \text{ kips}$
157 kips > 143 kips <b>o.k.</b>	105 kips > 95.4 kips <b>o.k.</b>

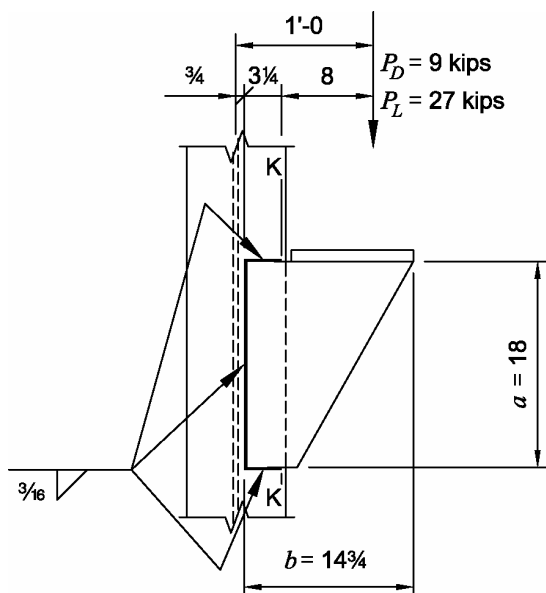
### Example II.A-23 Welded Bracket Plate Design.

#### Given:

Design a welded bracket plate, using 70 ksi electrodes, to support the following loads:

$$P_D = 9 \text{ kips}$$

$$P_L = 27 \text{ kips}$$



#### Material Properties:

Plate Material

ASTM A36

$F_y = 36 \text{ ksi}$

$F_u = 58 \text{ ksi}$

Manual  
Table 2-4

#### Solution:

LRFD	ASD
$R_u = 1.2(9 \text{ kips}) + 1.6(27 \text{ kips}) = 54 \text{ kips}$	$R_a = 9 \text{ kips} + 27 \text{ kips} = 36 \text{ kips}$

Assume PL  $\frac{1}{2}$  in.  $\times$  18 in.

Try a C-shaped weld with  $kl = 3$  in. and  $l = 18$  in.

$$k = kl/l = 3 \text{ in} / 18 \text{ in} = 0.167$$

$$x = 0.0221$$

and

$$al + xl = 11\frac{1}{4} \text{ in.}$$

$$a(18 \text{ in.}) + (0.0221)(18 \text{ in.}) = 11\frac{1}{4} \text{ in.}$$

Manual  
Table 8-8

$$a = 0.603$$

Interpolate using  $\theta = 0^\circ$ ,  $k = 0.167$ , and  $a = 0.603$

$$C = 1.47$$

$C_l = 1.0$  for E70XX electrode.

LRFD	ASD
$D_{req} = \frac{P_u}{\phi C C_1 l} = \frac{54 \text{ kips}}{0.75(1.47)(1.0)(18 \text{ in.})}$ $= 2.72 \rightarrow 3 \text{ sixteenths}$ $< \frac{1}{2} \text{ in.} - \frac{1}{16} \text{ in.} = \frac{7}{16} \quad \text{o.k.}$ <p>Use a <math>\frac{3}{16}</math>-in. fillet weld.</p> $D_{min} = 3 \quad \text{o.k.}$ <p><i>Check the flexural strength of the bracket plate</i></p> <p>Conservatively taking the required moment strength of the plate as equal to the moment strength of the weld group,</p> $M_u = P_u(a l) = (54 \text{ kips})(0.603)(18 \text{ in.})$ $= 586 \text{ in.-kips}$	$D_{req} = \frac{P_a \Omega}{C C_1 l} = \frac{(36 \text{ kips})(2.00)}{(1.47)(1.0)(18 \text{ in.})}$ $= 2.72 \rightarrow 3 \text{ sixteenths}$ $< \frac{1}{2} \text{ in.} - \frac{1}{16} \text{ in.} = \frac{7}{16} \quad \text{o.k.}$ <p>Use a <math>\frac{3}{16}</math>-in. fillet weld.</p> $D_{min} = 3 \quad \text{o.k.}$ <p><i>Check the flexural strength of the bracket plate</i></p> <p>Conservatively taking the required moment strength of the plate as equal to the moment strength of the weld group,</p> $M_a = P_a(a l) = (36 \text{ kips})(0.603)(18 \text{ in.})$ $= 391 \text{ in.-kips}$

Table J2.4

For flexural yielding of the plate,

$$M_n = F_y Z_x = (36 \text{ ksi}) \frac{(\frac{1}{2} \text{ in.})(18 \text{ in.})^2}{4} = 1,458 \text{ in.-kips}$$

LRFD	ASD
$\phi = 0.90$ $\phi M_n = 0.90(1,458 \text{ in.-kips}) = 1,312 \text{ in.-kips}$ $1,312 \text{ in.-kips} > 586 \text{ in.-kips} \quad \text{o.k.}$	$\Omega = 1.67$ $M_n / \Omega = \frac{(1,458 \text{ in.-kips})}{1.67} = 873 \text{ in.-kips}$ $873 \text{ in.-kips} > 391 \text{ in.-kips} \quad \text{o.k.}$

Manual Part 15

For yielding on the free edge of the triangular plate,

$$\frac{b}{a} = \frac{14\frac{3}{4} \text{ in.}}{18 \text{ in.}} = 0.820$$

$$z = 1.39 - 2.2\left(\frac{b}{a}\right) + 1.27\left(\frac{b}{a}\right)^2 - 0.25\left(\frac{b}{a}\right)^3 = 1.39 - 2.2(0.820) + 1.27(0.820)^2 - 0.25(0.820)^3$$

$$= 0.302$$

$$P_n = F_y z b t = (36 \text{ ksi})(0.302)(14\frac{3}{4} \text{ in.})(\frac{1}{2} \text{ in.}) = 80.2 \text{ kips}$$

Manual Part 15

LRFD	ASD
$\phi = 0.90$ $\phi P_n = 0.90(80.2 \text{ kips}) = 72.2 \text{ kips}$ $72.2 \text{ kips} > 54.0 \text{ kips}$ <b>o.k.</b>	$\Omega = 1.67$ $P_n / \Omega = \frac{(80.2 \text{ kips})}{1.67} = 48.0 \text{ kips}$ $48.0 \text{ kips} > 36.0 \text{ kips}$ <b>o.k.</b>

Check local buckling of the bracket plate

Manual  
Part 15

Since  $0.5 \leq \frac{b}{a} < 1.0$

$$t_{\min} = b \left( \frac{\sqrt{F_y}}{250} \right) = (14 \frac{3}{4} \text{ in.}) \left( \frac{\sqrt{36 \text{ ksi}}}{250} \right) = 0.354 \text{ in.} < \frac{1}{2} \text{ in.} \quad \text{o.k.}$$

Check shear yielding of the bracket plate

$$R_n = 0.6F_y A_g = 0.6(36 \text{ ksi})(18 \text{ in.})(\frac{1}{2} \text{ in.}) = 194 \text{ kips}$$

Eqn. J4-3

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(194 \text{ kips}) = 194 \text{ kips}$ $194 \text{ kips} > 54.0 \text{ kips}$ <b>o.k.</b>	$\Omega = 1.50$ $P_n / \Omega = \frac{(194 \text{ kips})}{1.50} = 130 \text{ kips} > 36.0 \text{ kips}$ $130 \text{ kips} > 36.0 \text{ kips}$ <b>o.k.</b>

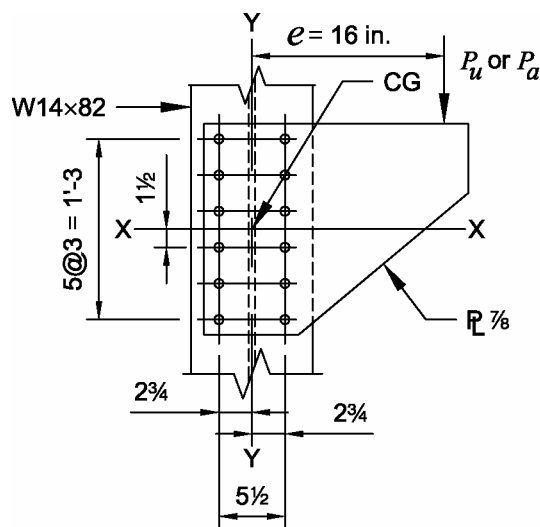
### Example II.A-24 Eccentrically-Loaded Bolt Group (IC method)

#### 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  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts in standard holes. Assume that bolt shear controls over bearing.

#### Solution A:

Assume the load is vertical ( $\theta = 0^\circ$ ) as illustrated below



With  $\theta = 0^\circ$ , with  $s = 3$  in.,  $e = 16$  in., and  $n = 6$ :

$$C = 3.55$$

Manual  
Table 7-9

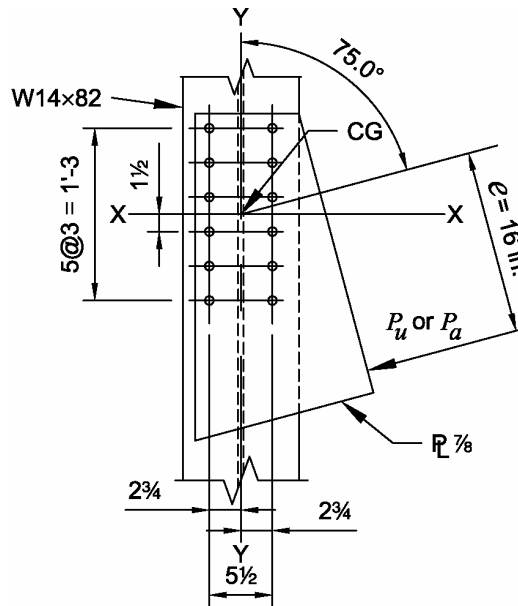
LRFD	ASD
$\phi r_n = 21.6$ kips	$r_n / \Omega = 14.4$ kips
$\phi R_n = C \phi r_n$	$R_n / \Omega = C (r_n / \Omega)$
$= 3.55(21.6)$ kips	$= 3.55(14.4)$ kips
$= 76.7$ kips	$= 51.1$ kips
Thus, $P_u$ must be less than or equal to 76.7 kips.	Thus, $P_a$ must be less than or equal to 51.1 kips.

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  $75^\circ$  with respect to vertical ( $\theta = 75^\circ$ ) as illustrated below



With  $\theta = 75^\circ$ ,  $s = 3$  in.,  $e = 16$  in., and  $n = 6$ :

$C = 7.90$

Manual  
Table 7-9

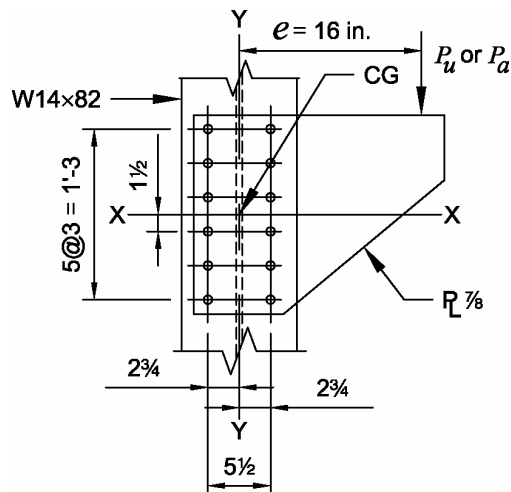
LRFD	ASD
$\phi R_n = C\phi r_n = 7.90(21.6 \text{ kips}) = 171 \text{ kips}$	$R_n / \Omega = C r_n / \Omega = 7.90(14.4 \text{ kips}) = 114 \text{ kips}$
Thus, $P_u$ must be less than or equal to 171 kips.	Thus, $P_a$ must be less than or equal to 114 kips.



### 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  $\theta = 0^\circ$ . Compare the result with that of the previous example. Use  $\frac{7}{8}$  in. diameter ASTM A325-N bolts in standard holes. Assume that bolt shear controls over bearing.  $I_p = 406 \text{ in.}^4 \text{ per in.}^2$



#### Solution:

LRFD	ASD
<p><i>Direct shear force per bolt</i></p> $r_{px} = 0, r_{py} = \frac{P_u}{n} = \frac{P_u}{12}$	<p><i>Direct shear force per bolt</i></p> $r_{px} = 0, r_{py} = \frac{P_a}{n} = \frac{P_a}{12}$
<p><i>Additional shear force due to eccentricity</i></p> $r_{mx} = \frac{P_u e c_y}{I_p} = \frac{P_u (16 \text{ in.}) (7 \frac{1}{2} \text{ in.})}{406 \text{ in.}^4 \text{ per in.}^2}$ $= 0.296 P_u$	<p><i>Additional shear force due to eccentricity</i></p> $r_{mx} = \frac{P_a e c_y}{I_p} = \frac{P_a (16 \text{ in.}) (7 \frac{1}{2} \text{ in.})}{406 \text{ in.}^4 \text{ per in.}^2}$ $= 0.296 P_a$
$r_{my} = \frac{P_u e c_x}{I_p} = \frac{P_u (16 \text{ in.}) \left( \frac{5 \frac{1}{2} \text{ in.}}{2} \right)}{406 \text{ in.}^4 \text{ per in.}^2}$ $= 0.108 P_u$	$r_{my} = \frac{P_a e c_x}{I_p} = \frac{P_a (16 \text{ in.}) \left( \frac{5 \frac{1}{2} \text{ in.}}{2} \right)}{406 \text{ in.}^4 \text{ per in.}^2}$ $= 0.108 P_a$

<p><i>Resultant shear force</i></p> $r_u = \sqrt{(r_{px} + r_{mx})^2 + (r_{py} + r_{my})^2}$ $= \sqrt{(0 + 0.296P_u)^2 + \left(\frac{P_u}{12} + 0.108P_u\right)^2}$ $= 0.352P_u$ <p>Since <math>r_u</math> must be less than or equal to the available strength,</p> $P_u \leq \frac{\phi r_n}{0.352} = \frac{21.6 \text{ kips}}{0.352} = 61.3 \text{ kips}$	<p><i>Resultant shear force</i></p> $r_a = \sqrt{(r_{px} + r_{mx})^2 + (r_{py} + r_{my})^2}$ $= \sqrt{(0 + 0.296P_a)^2 + \left(\frac{P_a}{12} + 0.108P_a\right)^2}$ $= 0.352P_a$ <p>Since <math>r_a</math> must be less than or equal to the available strength,</p> $P_a \leq \frac{r_n / \Omega}{0.352} = \frac{14.4 \text{ kips}}{0.352} = 40.9 \text{ kips}$
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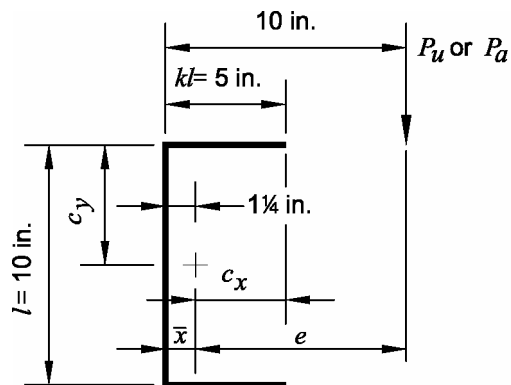
Note: the elastic method, shown here, is more conservative than the instantaneous center of rotation method, shown in **Example II.A-24a**.

### 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  $\frac{3}{8}$ -in. fillet weld and 70 ksi electrode.

**Solution A.** Assume that the load is vertical ( $\theta = 0^\circ$ ) as illustrated below



$$l = 10 \text{ in.}, kl = 5 \text{ in.}, \text{ therefore } k = 0.5$$

$$\text{With } \theta = 0^\circ, x = 0.125$$

$$xl + al = 10 \text{ in.}$$

$$0.125(10 \text{ in.}) + a(10 \text{ in.}) = 10 \text{ in.}$$

$$a = 0.875$$

By interpolation,

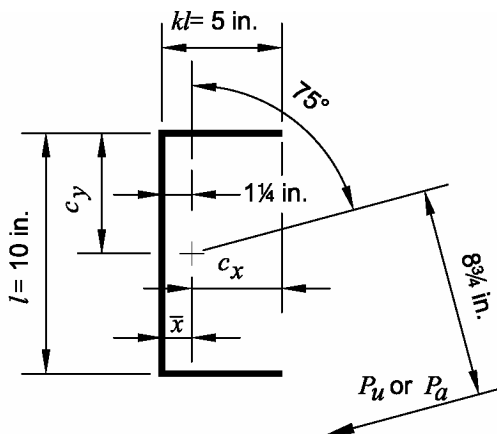
$$C = 1.88$$

Manual  
Table 8-8

LRFD	ASD
$\phi R_n = \phi C C_1 D l$ $= 0.75(1.88)(1.0)(6 \text{ sixteenths})(10 \text{ in.})$ $= 84.6 \text{ kips}$ <p>Thus, <math>P_u</math> must be less than or equal to 84.6 kips.</p>	$R_n / \Omega = \frac{C C_1 D l}{\Omega}$ $= \frac{1.88(1.0)(6 \text{ sixteenths})(10 \text{ in.})}{2.00}$ $= 56.4 \text{ kips}$ <p>Thus, <math>P_a</math> must be less than or equal to 56.4 kips.</p>

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  $75^\circ$  with respect to vertical ( $\theta = 75^\circ$ ) as illustrated below



As determined in solution a,

$k = 0.5$  and  $a = 0.875$ ,

By interpolation, with  $\theta = 75^\circ$

$C = 3.45$

Manual  
Table 8-8

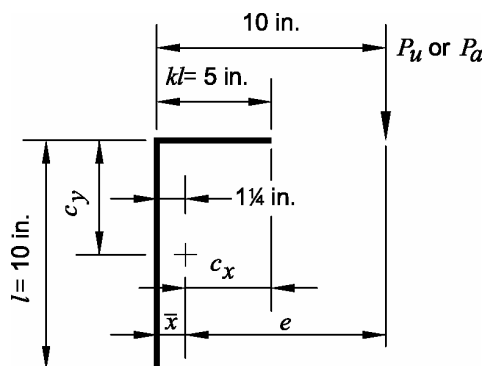
LRFD	ASD
$\phi R_n = \phi C C_1 D l$  $= 0.75(3.45)(1.0)(6 \text{ sixteenths})(10 \text{ in.})$  $= 155 \text{ kips}$  Thus, $P_u$ must be less than or equal to 155 kips.	$R_n / \Omega = \frac{C C_1 D l}{\Omega}$  $= \frac{3.45(1.0)(6 \text{ sixteenths})(10 \text{ in.})}{2.00}$  $= 103 \text{ kips}$  Thus, $P_a$ must be less than or equal to 103 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  $\frac{3}{8}$  in. fillet welds made with E70 electrodes.

$$I_p = 385 \text{ in.}^4 \text{ per in.}^2$$



#### Solution:

LRFD	ASD
<p><i>Direct shear force per inch of weld</i></p> $r_{px} = 0, r_{py} = \frac{P_u}{l} = \frac{P_u}{20}$	<p><i>Direct shear force per inch of weld</i></p> $r_{px} = 0, r_{py} = \frac{P_a}{l} = \frac{P_a}{20}$
<p><i>Additional shear force due to eccentricity</i></p> $r_{mx} = \frac{P_u e c_y}{I_p} = \frac{P_u (8.75 \text{ in.})(5 \text{ in.})}{385 \text{ in.}^4 \text{ per in.}^2} = 0.114 P_u$	<p><i>Additional shear force due to eccentricity</i></p> $r_{mx} = \frac{P_a e c_y}{I_p} = \frac{P_a (8.75 \text{ in.})(5 \text{ in.})}{385 \text{ in.}^4 \text{ per in.}^2} = 0.114 P_a$
$r_{my} = \frac{P_u e c_x}{I_p} = \frac{P_u (8.75 \text{ in.})(3.75 \text{ in.})}{385 \text{ in.}^4 \text{ per in.}^2}$ $= 0.0852 P_u$	$r_{my} = \frac{P_a e c_x}{I_p} = \frac{P_a (8.75 \text{ in.})(3.75 \text{ in.})}{385 \text{ in.}^4 \text{ per in.}^2}$ $= 0.0852 P_a$
<p><i>Resultant shear force</i></p> $r_u = \sqrt{(r_{px} + r_{mx})^2 + (r_{py} + r_{my})^2}$ $= \sqrt{(0 + 0.114 P_u)^2 + \left(\frac{P_u}{20} + 0.0852 P_u\right)^2}$ $= 0.177 P_u$	<p><i>Resultant shear force</i></p> $r_a = \sqrt{(r_{px} + r_{mx})^2 + (r_{py} + r_{my})^2}$ $= \sqrt{(0 + 0.114 P_a)^2 + \left(\frac{P_a}{20} + 0.0852 P_a\right)^2}$ $= 0.177 P_a$

<p>Since <math>r_u</math> must be less than or equal to the available strength,</p> $P_u \leq \frac{\phi r_n}{0.177} = \frac{1.392(6 \text{ sixteenths})}{0.177} = 47.2 \text{ kips}$	<p>Since <math>r_a</math> must be less than or equal to the available strength,</p> $P_a \leq \frac{r_n / \Omega}{0.177} = \frac{0.928(6 \text{ sixteenths})}{0.177} = 31.5 \text{ kips}$
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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:

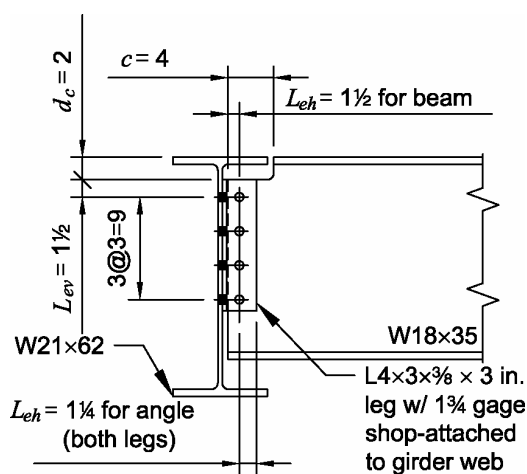
Design an all-bolted single-angle connection (case I in Table 10-10) between a W18×35 beam and a W21×62 girder-web, to support the following beam end reactions:

$$R_D = 6.5 \text{ kips}$$

$$R_L = 20 \text{ kips}$$

Top flange coped 2 in. deep by 4 in. long,  $L_{ev} = 1\frac{1}{2}$  in.,  $L_{eh} = 1\frac{1}{2}$  in. (assumed to be  $1\frac{1}{4}$  in. for calculation purposes to account for possible underrun in beam length),

Use  $\frac{3}{4}$ -in. diameter A325-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	$c = 4$ in.	$d_c = 2$ in.	$e = 4\frac{1}{2}$ -in.	
Girder W21×62	$t_w = 0.400$ in.		$h_0 = 15.7$ in.	

**Solution:**

LRFD	ASD	
$R_u = 1.2(6.5 \text{ kips}) + 1.6(20 \text{ kips}) = 40 \text{ kips}$  <i>Design the bolts and angle</i> Since half-web dimension of W18×35 is less than ¼ in., the tabular values may conservatively be used. Bolt shear is more critical than bolt bearing in this example; thus, $\phi_v r_n = 15.9 \text{ kips}$ .  $C_{\min} = \frac{R_u}{\phi r_n} = \frac{40 \text{ kips}}{15.9 \text{ kips/bolt}} = 2.52$  Try a four-bolt connection with a ⅜-in. thick angle.  $C = 3.07 > 2.52$ <b style="float: right;">o.k.</b>	$R_a = 6.5 \text{ kips} + 20 \text{ kips} = 26.5 \text{ kips}$  <i>Design the bolts and angle</i> Since half-web dimension of W18×35 is less than ¼ in., the tabular values may conservatively be used. Bolt shear is more critical than bolt bearing in this example; thus, $r_n/\Omega_v = 10.6 \text{ kips}$ .  $C_{\min} = \frac{R_a}{r_n / \Omega} = \frac{26.5 \text{ kips}}{10.6 \text{ kips/bolt}} = 2.52$  Try a four-bolt connection with a ⅜-in. thick angle.  $C = 3.07 > 2.52$ <b style="float: right;">o.k.</b>	Manual Table 7-1
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.	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 10-10
<i>Check shear yielding of the angle</i>  $\phi R_n = \phi(0.6F_y A_g)$ $= 1.0(0.6)(36 \text{ ksi})(11\frac{1}{2} \text{ in.})(\frac{3}{8}\text{-in.})$ $= 93.2 \text{ kips} > 40 \text{ kips}$ <b style="float: right;">o.k.</b>	<i>Check shear yielding of the angle</i>  $R_n / \Omega = 0.6F_y A_g / \Omega$ $= 0.6(36 \text{ ksi})(11\frac{1}{2} \text{ in.})(\frac{3}{8}\text{-in.}) / 1.50$ $= 62.1 \text{ kips} > 26.5 \text{ kips}$ <b style="float: right;">o.k.</b>	Section J4.2 Eqn J4-3
<i>Check shear rupture of the angle</i>  $\phi R_n = \phi(0.6F_u A_n)$ $= 0.75\{0.6(58 \text{ ksi})[(11\frac{1}{2} \text{ in.})(\frac{3}{8}\text{-in.}) - 4(\frac{7}{8}\text{-in.})(\frac{3}{8}\text{-in.})]\}$ $= 78.3 \text{ kips} > 40 \text{ kips}$ <b style="float: right;">o.k.</b>	<i>Check shear rupture of the angle</i>  $R_n / \Omega = (0.6F_u A_n) / \Omega$ $= 0.6(58 \text{ ksi})[(11\frac{1}{2} \text{ in.})(\frac{3}{8}\text{-in.}) - 4(\frac{7}{8}\text{-in.})(\frac{3}{8}\text{-in.})] / 2.00$ $= 52.2 \text{ kips} > 26.5 \text{ kips}$ <b style="float: right;">o.k.</b>	Section J4.2 Eqn J4-4
<i>Check block shear rupture of the angle</i> From the tables, with $n = 4$ , $L_{ev} = 1\frac{1}{2} \text{ in.}$ $L_{eh} = 1\frac{1}{4} \text{ in.}$ ,  $\phi R_n = \phi F_u A_{nt} U_{bs} + \min\left(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv}\right)$  Tension rupture component  $\phi F_u A_{nt} = 35.3 \text{ kips/in.}(\frac{3}{8} \text{ in.})$	<i>Check block shear rupture of the angle</i> From the tables, with $n = 4$ , $L_{ev} = 1\frac{1}{2} \text{ in.}$ $L_{eh} = 1\frac{1}{4} \text{ in.}$ ,  $\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$  Tension rupture component  $F_u A_{nt} / \Omega = 23.6 \text{ kips/in.}(\frac{3}{8} \text{ in.})$	Manual Table 9-3a



<p>Shear yielding component</p> $\phi 0.6F_y A_{gv} = 170 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $\phi 0.6F_u A_{nv} = 194 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Tension stress is uniform, therefore</p> $U_{bs} = 1.0$ $\phi R_n = (170 \text{ kips/in.} + 35.3 \text{ kips/in.}) (\frac{3}{8} \text{ in.})$ $= 77.0 \text{ kips} > 40 \text{ kips} \quad \mathbf{o.k.}$	<p>Shear yielding component</p> $0.6F_y A_{gv} / \Omega = 113 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Shear rupture component</p> $0.6F_u A_{nv} = 129 \text{ kips/in.} (\frac{3}{8} \text{ in.})$ <p>Tension stress is uniform, therefore</p> $U_{bs} = 1.0$ $R_n / \Omega = (113 \text{ kips/in.} + 23.6 \text{ kips/in.}) (\frac{3}{8} \text{ in.})$ $= 51.4 \text{ kips} > 26.5 \text{ kips} \quad \mathbf{o.k.}$	<p>Manual Table 9-3b</p> <p>Manual Table 9-3c</p> <p>Eqn. J4-5</p> <p>Commentary Section J4.3</p>
<p><i>Check flexure of the support-leg of the angle</i></p> <p>The required strength is</p> $M_u = R_u e = (40 \text{ kips})(1\frac{3}{4}\text{-in.} + 0.300 \text{ in./2})$ $= 76.0 \text{ in.-kips}$ <p>For flexural yielding</p> $\phi = 0.90$ $\phi M_n = \phi F_y Z_x$ $= 0.90(36 \text{ ksi}) \left[ \frac{(\frac{3}{8} \text{ in.})(11\frac{1}{2} \text{ in.})^2}{4} \right]$ $= 402 \text{ in.-kips} > 76.0 \text{ in.-kips} \quad \mathbf{o.k.}$	<p><i>Check flexure of the support-leg of the angle</i></p> <p>The required strength is</p> $M_a = R_a e = (26.5 \text{ kips})(1\frac{3}{4}\text{-in.} + 0.300 \text{ in./2})$ $= 50.4 \text{ in.-kips}$ <p>For flexural yielding</p> $\Omega = 1.67$ $M_n / \Omega = F_y Z_x / \Omega$ $= \frac{(36 \text{ ksi})}{1.67} \left[ \frac{(\frac{3}{8} \text{ in.})(11\frac{1}{2} \text{ in.})^2}{4} \right]$ $= 267 \text{ in.-kips} > 50.4 \text{ in.-kips} \quad \mathbf{o.k.}$	<p>Manual Part 15</p>

For flexural rupture,

$$Z_{net} \approx \frac{t}{4} \left[ d^2 - \frac{s^2 n (n^2 - 1) (d_h + \frac{1}{16} \text{ in.})}{d} \right]$$

$$\approx \frac{\frac{3}{8} \text{ in.}}{4} \left[ (11\frac{1}{2} \text{ in.})^2 - \frac{(3 \text{ in.})^2 (4) (4^2 - 1) (1\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})}{11\frac{1}{2} \text{ in.}} \right] = 8.55 \text{ in.}^3$$

LRFD	ASD
$\phi M_n = \phi F_u Z_{net} = 0.75(58 \text{ ksi})(8.55 \text{ in.}^3)$ $= 372 \text{ in.-kips} > 76.0 \text{ in.-kips} \quad \mathbf{o.k.}$	$M_n / \Omega = \frac{F_u Z_{net}}{\Omega} = \frac{(58 \text{ ksi})(8.55 \text{ in.}^3)}{2.00}$ $= 248 \text{ in.-kips} > 53.4 \text{ in.-kips} \quad \mathbf{o.k.}$

<p><i>Check beam web for bolt bearing and block shear rupture.</i></p> <p><math>n = 4</math>, <math>L_{ev} = 1\frac{1}{2}</math> in., and <math>L_{eh} = 1\frac{1}{2}</math> in. (Assumed to be <math>1\frac{1}{4}</math> in. for calculation purposes to provide for possible underrun in beam length),</p> <p><math>\phi R_n = (257 \text{ kips/in.})(0.300 \text{ in.})</math></p> <p><math>= 77.1 \text{ kips} &gt; 40 \text{ kips}</math>                      <b>o.k.</b></p>	<p><i>Check beam web for bolt bearing and block shear rupture.</i></p> <p><math>n = 4</math>, <math>L_{ev} = 1\frac{1}{2}</math> in., and <math>L_{eh} = 1\frac{1}{2}</math> in. (Assumed to be <math>1\frac{1}{4}</math> in. for calculation purposes to provide for possible underrun in beam length),</p> <p><math>R_n / \Omega = (171 \text{ kips/in.})(0.300 \text{ in.})</math></p> <p><math>= 51.3 \text{ kips} &gt; 26.5 \text{ kips}</math>                      <b>o.k.</b></p>
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Manual  
Table 10-1

### Example II.A-29 Bolted/Welded Single-Angle Connection (beam-to-column flange).

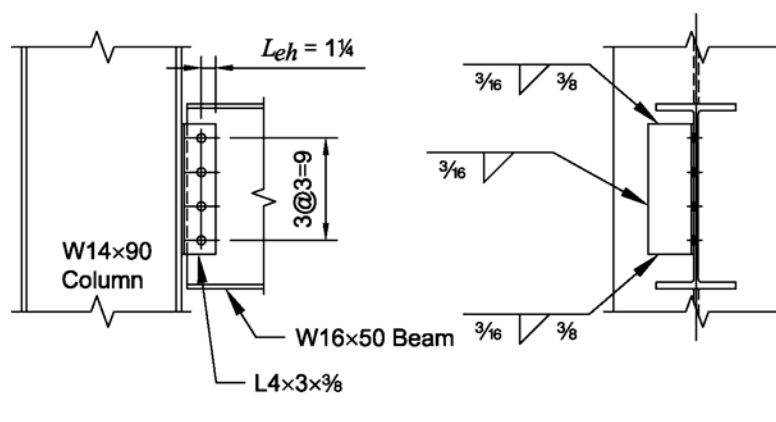
#### Given:

Design a single-angle connection between a W16×50 beam and a W14×90 column flange to support the following beam end reactions:

$$R_D = 9 \text{ kips}$$

$$R_L = 27 \text{ kips}$$

Use  $\frac{3}{4}$  in. diameter ASTM A325-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:

W16×50	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Tables 2-3 and 2-4
W14×90	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Angle	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

#### Geometric Properties:

Beam	W16×50	$t_w = 0.380 \text{ in.}$	$d = 16.3 \text{ in.}$	$t_f = 0.630 \text{ in.}$	Manual Table 1-1
Column	W14×90	$t_f = 0.710$			

#### Solution:

LRFD	ASD
$R_u = 1.2 (9 \text{ kips}) + 1.6 (27 \text{ kips}) = 54 \text{ kips}$	$R_u = 9 \text{ kips} + 27 \text{ kips} = 36 \text{ kips}$
<i>Design single angle, bolts, and welds</i>	<i>Design single angle, bolts, and welds</i>
Since the half-web dimension of a W16×50 is less than $\frac{1}{4}$ in., the tabulated values may conservatively be used.	Since the half-web dimension of a W16×50 is less than $\frac{1}{4}$ in., the tabulated values may conservatively be used.
Try a four bolt single angle (L4×3× $\frac{3}{8}$ ).	Try a four bolt single angle (L4×3× $\frac{3}{8}$ ).
$\phi R_n = 63.6 \text{ kips} > 54 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = 42.4 \text{ kips} > 36 \text{ kips}$ <b>o.k.</b>

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Table 10-11

<p>Also with a <math>\frac{3}{16}</math>-in fillet weld size</p> <p><math>\phi R_n = 56.6 \text{ kips} &gt; 54 \text{ kips}</math> <b>o.k.</b></p> <p>Use four-bolt single angle L4×3×<math>\frac{3}{8}</math>. 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.</p> <p><i>Check supported beam web</i></p> <p>The bearing strength of the beam web per bolt is</p> <p><math>s = 3 \text{ in.}, \frac{3}{4} \text{ in. diameter bolts, standard holes}</math></p> $\phi R_n = \phi r_n t_w n$ $= (87.8 \text{ kips/in.})(0.380 \text{ in.})(4 \text{ bolts})$ $= 133 \text{ kips} > 54 \text{ kips}$ <b>o.k.</b> <p><i>Check support</i></p> <p>Minimum support thickness for the <math>\frac{3}{16}</math>-in. welds is 0.286 in.</p> <p><math>t_w = 0.710 \text{ in} &gt; 0.286 \text{ in}</math> <b>o.k.</b></p>	<p>Also with a <math>\frac{3}{16}</math>-in. fillet weld size</p> <p><math>R_n / \Omega = 37.7 \text{ kips} &gt; 36 \text{ kips}</math> <b>o.k.</b></p> <p>Use four-bolt single angle L4×3×<math>\frac{3}{8}</math>. 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.</p> <p><i>Check supported beam web</i></p> <p>The bearing strength of the beam web per bolt is</p> <p><math>s = 3 \text{ in.}, \frac{3}{4} \text{ in. diameter bolts, standard holes}</math></p> $R_n / \Omega = \frac{r_n t_w n}{\Omega}$ $= (58.5 \text{ kips/in.})(0.380 \text{ in.})(4 \text{ bolts})$ $= 88.9 \text{ kips} > 36 \text{ kips}$ <b>o.k.</b> <p><i>Check support</i></p> <p>Minimum support thickness for the <math>\frac{3}{16}</math>-in. welds is 0.286 in.</p> <p><math>t_w = 0.710 \text{ in} &gt; 0.286 \text{ in}</math> <b>o.k.</b></p>	<p>Manual Table 10-11</p> <p>Manual Table 7-5</p> <p>Manual Table 10-11</p>
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### Example II.A-30 All-Bolted Tee Connection (beam-to-column flange)

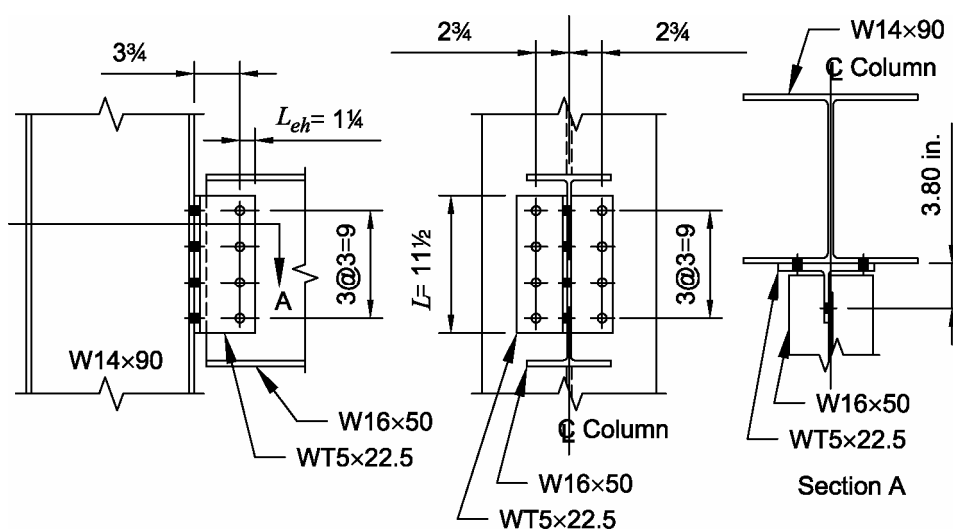
#### Given:

Design an all-bolted tee connection between a W16×50 beam and a W14×90 column flange to support the following beam end reactions:

$$R_D = 9 \text{ kips}$$

$$R_L = 27 \text{ kips}$$

Use  $\frac{3}{4}$  in. diameter ASTM A325-N bolts in standard holes. Try a WT5×22.5 with a four-bolt connection.



#### Material Properties:

W16×50	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Table 2-3
W14×90	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
WT5×22.5	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	

#### Geometric Properties:

Beam	W16×50	$t_w = 0.380 \text{ in.}$	$d = 16.3 \text{ in.}$	$t_f = 0.630 \text{ in.}$	Manual Tables 1-1 and 1-8
Column	W14×90	$t_f = 0.710 \text{ in.}$			
Tee	WT5×22.5	$d = 5.05 \text{ in.}$ $t_s = 0.350 \text{ in.}$	$b_f = 8.02 \text{ in.}$ $k_f = \frac{13}{16} \text{ in.}$ (see W10×45 Manual Table 1-1)	$t_f = 0.620 \text{ in.}$	

**Solution:**

$R_u = 1.2 (9 \text{ kips}) + 1.6 (27 \text{ kips}) = 54 \text{ kips}$	$R_a = 9 \text{ kips} + 27 \text{ kips} = 36 \text{ kips}$
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*Check limitation on tee stem thickness*

See Rotational Ductility discussion at the beginning of the Manual Part 9

$$t_{s \max} = \frac{d_b}{2} + \frac{1}{16} \text{ in.} = \frac{\frac{3}{4} \text{ in.}}{2} + \frac{1}{16} \text{ in.} = 0.438 \text{ in.} > 0.350 \text{ in.} \quad \text{o.k.}$$

*Check limitation on bolt diameter for bolts through tee flange*

Assuming a 5½ in. gage,

$b$  = flexible width in connection element

$$b = \frac{g - 2k_1}{2} = \frac{5 \frac{1}{2} \text{ in.} - 2(\frac{13}{16} \text{ in.})}{2} = 1.94 \text{ in.}$$

$$d_{b \min} = 0.163t_f \sqrt{\frac{F_y}{b} \left( \frac{b^2}{L^2} + 2 \right)} \leq 0.69\sqrt{t_s}$$

$$= 0.163(0.620 \text{ in.}) \sqrt{\frac{50 \text{ ksi}}{1.94 \text{ in.}} \left[ \frac{(1.94 \text{ in.})^2}{(11 \frac{1}{2} \text{ in.})^2} + 2 \right]} \leq 0.69\sqrt{0.350 \text{ in.}}$$

$$= 0.731 \text{ in.} \leq 0.408 \text{ in.}$$

$$= 0.408 \text{ in.}$$

Since  $d_b = \frac{3}{4} \text{ in.} > d_{b \min} = 0.408 \text{ in.}$  o.k.

*Check bolt group through beam web for shear and bearing*

$$a = d - L_{eh} = 5.05 \text{ in.} - 1 \frac{1}{4} \text{ in.} = 3.80 \text{ in.}$$

LRFD	ASD
<p>Since bolt shear is more critical than bolt bearing in this example, <math>\phi r_n = 15.9 \text{ kips}</math>,</p> <p>Thus,</p> <p><math>\phi R_n = n\phi r_n = (4 \text{ bolts})(15.9 \text{ kips})</math></p> <p><math>= 63.6 \text{ kips} &gt; 54 \text{ kips}</math> <span style="float: right;">o.k.</span></p>	<p>Since bolt shear is more critical than bolt bearing in this example, <math>r_n/\Omega = 10.6 \text{ kips}</math>,</p> <p>Thus,</p> <p><math>R_n/\Omega = nr_n/\Omega = (4 \text{ bolts})(10.6 \text{ kips})</math></p> <p><math>= 42.4 \text{ kips} &gt; 36 \text{ kips}</math> <span style="float: right;">o.k.</span></p>

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Table 7-1

<p><i>Check shear yielding of the tee stem</i></p> $\phi R_n = \phi 0.6 F_y A_g$ $\phi = 1.00$ $= 1.00 [0.6(50 \text{ ksi})(11\frac{1}{2} \text{ in.})(0.350 \text{ in.})]$ $= 121 \text{ kips} > 54 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear yielding of the tee stem</i></p> $R_n / \Omega = 0.6 F_y A_g / \Omega$ $\Omega = 1.50$ $= [0.6(50 \text{ ksi})(11\frac{1}{2} \text{ in.})(0.350 \text{ in.})] / 1.5$ $= 80.5 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-3
<p><i>Check shear rupture of the tee stem</i></p> $\phi R_n = \phi 0.6 F_u A_{nv}$ <p>Where <math>\phi = 0.75</math></p> $= 0.75(0.6)(65 \text{ ksi}) [11\frac{1}{2} \text{ in.} - 4(0.875 \text{ in.})]$ $(0.350 \text{ in.})$ $= 81.9 \text{ kips} > 54 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear rupture of the tee stem</i></p> $R_n / \Omega = 0.6 F_u A_{nv} / \Omega$ <p>Where <math>\Omega = 2.00</math></p> $= 0.6(65 \text{ ksi}) [11\frac{1}{2} - 4(0.875 \text{ in.})](0.350 \text{ in.})$ $/ 2.00$ $= 54.6 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	
<p><i>Check block shear rupture of the tee stem</i></p> $L_{eh} = L_{ev} = 1\frac{1}{4} \text{ in.},$ <p>Thus,</p> $\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$ <p>Tension rupture component</p> $\phi F_u A_{nt} = 39.6 \text{ kips/in. (0.350 in.)}$ <p>Shear yielding component</p> $\phi 0.6 F_y A_{gv} = 231 \text{ kips/in. (0.350 in.)}$ <p>Shear rupture component</p> $\phi 0.6 F_u A_{nv} = 210 \text{ kips/in. (0.350 in.)}$ $\phi R_n = (210 \text{ kips/in.} + 39.6 \text{ kips/in.})(0.350 \text{ in.})$ $= 87.4 \text{ kips} > 54 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check block shear rupture of the tee stem</i></p> $L_{eh} = L_{ev} = 1\frac{1}{4} \text{ in.},$ <p>Thus,</p> $\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$ <p>Tension rupture component</p> $F_u A_{nt} / \Omega = 26.4 \text{ kips/in. (0.350 in.)}$ <p>Shear yielding component</p> $0.6 F_y A_{gv} / \Omega = 154 \text{ kips/in. (0.350 in.)}$ <p>Shear rupture component</p> $0.6 F_u A_{nv} / \Omega = 140 \text{ kips/in. (0.350 in.)}$ $\frac{R_n}{\Omega} = (140 \text{ kips/in.} + 26.4 \text{ kips/in.})(0.350 \text{ in.})$ $= 58.2 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-5
Manual Table 9-3a	Manual Table 9-3b	

<p><i>Check bolt group through support for shear and bearing combined with tension due to eccentricity</i></p> <p>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</p> <p>Calculate tensile force per bolt <math>r_{ut}</math>.</p> $r_{ut} = \frac{R_u e}{n' d_m}$ $r_{ut} = \frac{54 \text{ kips (5.05 in. - 1/4 in.)}}{4 \text{ bolts (6 in.)}}$ $= 8.55 \text{ kips/bolts}$ <p><i>Check design strength of bolts for tension-shear interaction</i></p> <p>When threads are not excluded from the shear planes of ASTM A325 bolts</p> $r_{uv} = \frac{54 \text{ kips}}{8 \text{ bolts}}$ $= 6.75 \text{ kips/bolt} < 15.9 \text{ kips/bolt} \quad \mathbf{o.k.}$ <p><math>F_{nt} = 90 \text{ ksi}, F_{nv} = 48 \text{ ksi}</math></p> $F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}$ $\phi = 0.75$ $= 1.3(90 \text{ ksi}) - \left\{ \left[ \frac{90 \text{ ksi}}{(0.75)(48 \text{ ksi})} \right] \left( \frac{6.75 \text{ kips/bolt}}{0.442 \text{ in}^2} \right) \right\} \leq 90 \text{ ksi}$ $= 78.8 \text{ ksi} < 90 \text{ ksi}$ <p><math>\phi R_n = F_{nt}' A_b</math></p> $= 0.75(78.8 \text{ ksi})(0.442 \text{ in}^2)$ $= 26.1 \text{ kips/bolt} > 8.55 \text{ kips/bolt} \quad \mathbf{o.k.}$	<p><i>Check bolt group through support for shear and bearing combined with tension due to eccentricity</i></p> <p>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</p> <p>Calculate tensile force per bolt <math>r_{at}</math>.</p> $r_{at} = \frac{R_a e}{n' d_m}$ $r_{at} = \frac{36 \text{ kips (5.05 in. - 1/4 in.)}}{4 \text{ bolts (6 in.)}}$ $= 5.70 \text{ kips/bolts}$ <p><i>Check allowable strength of bolts for tension-shear interaction</i></p> <p>When threads are not excluded from the shear planes of ASTM A325 bolts</p> $r_{av} = \frac{36 \text{ kips}}{8 \text{ bolts}}$ $= 4.50 \text{ kips/bolt} < 10.6 \text{ kips/bolt} \quad \mathbf{o.k.}$ <p><math>F_{nt} = 90 \text{ ksi}, F_{nv} = 48 \text{ ksi}</math></p> $F_{nt}' = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt}$ $\Omega = 2.00$ $= 1.3(90 \text{ ksi}) - \left\{ \left[ \frac{(2.00)(90 \text{ ksi})}{(48 \text{ ksi})} \right] \left( \frac{4.50 \text{ kips/bolt}}{0.442 \text{ in}^2} \right) \right\} \leq 90 \text{ ksi}$ $= 78.8 \text{ ksi} < 90 \text{ ksi}$ <p><math>R_n/\Omega = F_{nt}' A_b/\Omega</math></p> $= (78.8 \text{ ksi})(0.442 \text{ in}^2)/2.00$ $= 17.4 \text{ kips/bolt} > 5.7 \text{ kips/bolt} \quad \mathbf{o.k.}$	<p>Manual Table 7-1</p> <p>Table J3.2</p> <p>Eqn. J3-3a and J3-3b</p>
		<p>Eqn. J3-2</p>



*Check bearing strength at bolt holes*

With  $L_e = 1\frac{1}{4}$  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  
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$$b = \frac{(g - t_s)}{2} = \frac{(5\frac{1}{2}\text{ in.} - 0.350\text{ in.})}{2} = 2.58\text{ in.}$$

$$a = \frac{(b_f - g)}{2} = \frac{(8.02\text{ in.} - 5\frac{1}{2}\text{ in.})}{2} = 1.26\text{ in.}$$

Since  $a = 1.26$  in. is less than  $1.25b = 3.23$  in., use  $a = 1.26$  in. for calculation purposes

$$b' = b - \frac{d}{2} = 2.58\text{ in.} - \left(\frac{\frac{3}{4}\text{ in.}}{2}\right) = 2.21\text{ in.}$$

$$a' = a + \left(\frac{d}{2}\right) = 1.26\text{ in.} + \left(\frac{\frac{3}{4}\text{ in.}}{2}\right) = 1.64\text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{2.21\text{ in.}}{1.64\text{ in.}} = 1.35$$

LRFD	ASD
$T = r_{ut} = 8.55$ kips/bolt	$T = r_{at} = 5.70$ kips/bolt
$B = \phi r_n = 26.1$ kips/bolt	$B = r_n / \Omega = 17.4$ kips/bolt
$\beta = \frac{1}{\rho} \left( \frac{B}{T} - 1 \right)$	$\beta = \frac{1}{\rho} \left( \frac{B}{T} - 1 \right)$
$= \frac{1}{1.35} \left( \frac{26.1\text{ kips/bolts}}{8.55\text{ kips/bolts}} - 1 \right) = 1.52$	$= \frac{1}{1.35} \left( \frac{17.4\text{ kips/bolts}}{5.70\text{ kips/bolts}} - 1 \right) = 1.52$

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Since  $\beta \geq 1$ , set  $\alpha' = 1.0$

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Part 9

$$p = \frac{11\frac{1}{2}\text{ in.}}{4\text{ bolts}} = 2.88\text{ in./bolt}$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{1\frac{3}{16}\text{ in.}}{2.88\text{ in.}} = 0.718$$

LRFD	ASD
$t_{req} = \sqrt{\frac{4.44r_{ut}b'}{pF_u(1+\delta\alpha')}} = \sqrt{\frac{4.44(8.55 \text{ kips/bolt})(2.21 \text{ in.})}{(2.88 \text{ in./bolt})(65 \text{ ksi})[1+(0.718)(1.0)]}}$	$t_{req} = \sqrt{\frac{6.66r_{ut}b'}{pF_u(1+\delta\alpha')}} = \sqrt{\frac{6.66(5.70 \text{ kips/bolt})(2.21 \text{ in.})}{(2.88 \text{ in./bolt})(65 \text{ ksi})[1+(0.718)(1.0)]}}$
= 0.511 in. < 0.620 in. <b>o.k.</b>	= 0.511 in. < 0.620 in. <b>o.k.</b>

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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
<p><i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i></p> <p>For single shear, four rows of 3/4 in. diameter bolts and an uncoped beam with <math>F_y = 50</math> ksi and <math>F_u = 65</math> ksi,</p> $\phi R_n = (702 \text{ kips/in.})(0.380 \text{ in.})/2$ $= 133 \text{ kips} > 54 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i></p> <p>For single shear, four rows of 3/4 in. diameter bolts and an uncoped beam with <math>F_y = 50</math> ksi and <math>F_u = 65</math> ksi,</p> $R_n/\Omega = (468 \text{ kips/in.})(0.380 \text{ in.})/2$ $= 88.9 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$
<p><i>Check supporting member web or flange for bolt bearing</i></p> <p>For, double shear, four rows of 3/4 in. diameter bolts with <math>F_y = 50</math> ksi and <math>F_u = 65</math> ksi,</p> $\phi R_n = (702 \text{ kips/in.})(0.710 \text{ in.})$ $= 498 \text{ kips} > 54 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check supporting member web or flange for bolt bearing</i></p> <p>For, double shear, for four rows of 3/4 in. diameter bolts with <math>F_y = 50</math> ksi and <math>F_u = 65</math> ksi,</p> $R_n/\Omega = (468 \text{ kips/in.})(0.710 \text{ in.})$ $= 332 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 10-1Manual  
Table 10-1

### Example II.A-31 Bolted/Welded Tee Connection (beam-to-column flange)

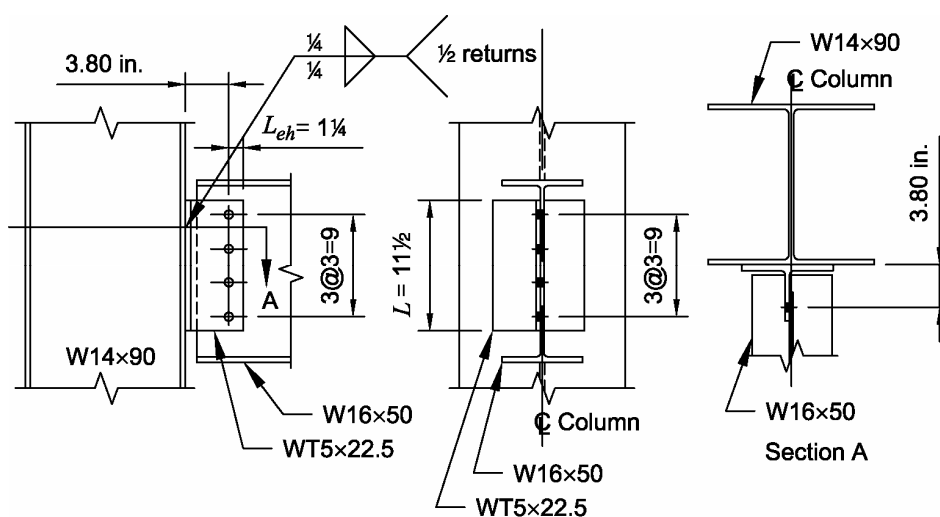
#### Given:

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:

$$R_D = 6 \text{ kips}$$

$$R_L = 18 \text{ kips}$$

Use  $\frac{3}{4}$  in. diameter ASTM A325-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.	$k_l = \frac{13}{16}$ in.		(see W10×45 Manual Table 1-1)

**Solution:**

LRFD	ASD
$R_u = 1.2 (6 \text{ kips}) + 1.6 (18 \text{ kips}) = 36 \text{ kips}$	$R_a = 6 \text{ kips} + 18 \text{ kips} = 24 \text{ kips}$

Check limitation on tee stem thickness

Manual  
Part 9

See Rotational Ductility discussion at the beginning of the Manual Part 9

$$t_{s,max} = \frac{d_b}{2} + \frac{1}{16} \text{ in.} = \frac{\frac{3}{4} \text{ in.}}{2} + \frac{1}{16} \text{ in.} = 0.438 \text{ in.} > 0.350 \text{ in.} \quad \text{o.k.}$$

Design the welds connecting the tee flange to the column flange

$b$  = flexible width in connection element

$$b = \frac{b_f - 2k_1}{2} = \frac{8.02 \text{ in.} - 2\left(\frac{13}{16} \text{ in.}\right)}{2} = 3.20 \text{ in.}$$

$$w_{\min} = 0.0158 \frac{F_y t_f^2}{b} \left( \frac{b^2}{L^2} + 2 \right) \leq \frac{5}{8} t_s$$

$$= 0.0158 \left[ \frac{(50 \text{ ksi})(0.620 \text{ in.})^2}{(3.20 \text{ in.})} \right] \left[ \left( \frac{(3.20 \text{ in.})^2}{(11 \frac{1}{2} \text{ in.})^2} \right) + 2 \right] \leq \left( \frac{5}{8} \right) (0.350 \text{ in.})$$

$$= 0.197 \text{ in.} \leq 0.212 \text{ in.}$$

Manual  
Part 9

Design welds for direct load only with no eccentricity

LRFD	ASD
Try $\frac{1}{4}$ -in. fillet welds.	Try $\frac{1}{4}$ -in. fillet welds.
$\phi R_n = 1.392Dl$	$R_n/\Omega = 0.928Dl$
$= 1.392 (4 \text{ sixteenths})(2 \text{ sides})(11 \frac{1}{2} \text{ in.})$	$= 0.928 (4 \text{ sixteenths})(2 \text{ sides})(11 \frac{1}{2} \text{ in.})$
$= 128 \text{ kips} > 36 \text{ kips} \quad \text{o.k.}$	$= 85.4 \text{ kips} > 24 \text{ kips} \quad \text{o.k.}$
Use $\frac{1}{4}$ -in. fillet welds.	Use $\frac{1}{4}$ -in. fillet welds.

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Part 8

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

$$a = d - L_{eh} = 5.05 \text{ in.} - 1 \frac{1}{4} \text{ in.} = 3.80 \text{ in.}$$

$$e_b = a = 3.80 \text{ in.}$$

LRFD	ASD
Thus the tee stem and bolts must be designed for $R_u = 36$ kips and $R_u e_b = 137$ in.-kips.	Thus the tee stem and bolts must be designed for $R_a = 24$ kips and $R_a e_b = 91.2$ in.-kips.
<i>Check bolt group through beam web for shear and bearing</i>	<i>Check bolt group through beam web for shear and bearing</i>
For $\theta = 0^\circ$ , with $s = 3$ in., $e_x = e_b = 3.80$ in., and $n = 4$ bolts,	For $\theta = 0^\circ$ , 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)	$R_n / \Omega = C r_n / \Omega = 2.45$ (10.6 kips/bolt)
$= 39.0$ kips $>$ 36.0 kips <b>o.k.</b>	$= 26.0$ kips $>$ 24.0 kips <b>o.k.</b>
<i>Check flexure on the tee stem</i>	<i>Check flexure on the tee stem</i>
For flexural yielding	For flexural yielding
$\phi = 0.90$	$\Omega = 1.67$
$\phi M_n = \phi F_y Z_x$	$M_n / \Omega = F_y Z_x / \Omega$
$= (0.9)(50 \text{ ksi}) \left[ \frac{(0.350 \text{ in.})(11\frac{1}{2} \text{ in.})^2}{4} \right]$	$= (50 \text{ ksi}) \left[ \frac{(0.350 \text{ in.})(11\frac{1}{2} \text{ in.})^2}{4} \right] / 1.67$
$= 520$ in.-kips $>$ 137 in.-kips <b>o.k.</b>	$= 346$ in.-kips $>$ 91.2 in.-kips <b>o.k.</b>

Manual  
Table 7-7Manual  
Table 7-1Manual  
Part 15

For flexural rupture,

$$Z_{net} \approx \frac{t}{4} \left[ d^2 - \frac{s^2 n (n^2 - 1) (d_h + \frac{1}{16} \text{ in.})}{d} \right]$$

$$\approx \frac{0.350 \text{ in.}}{4} \left[ (11\frac{1}{2} \text{ in.})^2 - \frac{(3 \text{ in.})^2 (4) (4^2 - 1) (\frac{13}{16} \text{ in.} + \frac{1}{16} \text{ in.})}{11\frac{1}{2} \text{ in.}} \right] = 7.98 \text{ in.}^3$$

LRFD	ASD
$\phi M_n = \phi F_u Z_{net}$	$M_n / \Omega = F_u Z_{net} / \Omega$
$\phi = 0.75$	$\Omega = 2.00$
$= (0.75)(65 \text{ ksi})(7.98 \text{ in.}^3)$	$= (65 \text{ ksi})(7.98 \text{ in.}^3) / 2.00$
$= 389$ in.-kips $>$ 137 in.-kips <b>o.k.</b>	$= 259$ in.-kips $>$ 91.2 in.-kips <b>o.k.</b>

<p><i>Check shear yielding of the tee stem</i></p> $\phi R_n = \phi 0.60 F_y A_g$ $\phi = 1.00$ $= 1.00(0.6)(50 \text{ ksi})(11\frac{1}{2} \text{ in.})(0.350 \text{ in.})$ $= 121 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear yielding of the tee stem</i></p> $R_n/\Omega = 0.60 F_y A_g / \Omega$ $\Omega = 1.50$ $= 0.6(50 \text{ ksi})(11\frac{1}{2} \text{ in.})(0.350 \text{ in.})/1.50$ $= 80.5 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-3
<p><i>Check shear rupture of the tee stem</i></p> $\phi = 0.75$ $\phi R_n = \phi 0.6 F_u A_{nv}$ $\phi R_n = 0.75(0.6)(65 \text{ ksi})$ $\times [11\frac{1}{2} - 4(0.875 \text{ in.})](0.350 \text{ in.})$ $= 81.9 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear rupture of the tee stem</i></p> $\Omega = 2.00$ $R_n/\Omega = 0.6 F_u A_{nv} / \Omega$ $= \frac{0.6(65 \text{ ksi})}{2.00}$ $\times [11\frac{1}{2} - 4(0.875 \text{ in.})](0.350 \text{ in.})$ $= 54.6 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-4
<p><i>Check block shear rupture of the tee stem</i></p> $L_{eh} = L_{ev} = 1\frac{1}{4} \text{ in.},$ <p>Thus.</p> $\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$ <p>Tension rupture component</p> $\phi F_u A_{nt} = 39.6 \text{ kips/in. (0.350 in.)}$ <p>Shear yielding component</p> $\phi 0.6 F_y A_{gv} = 231 \text{ kips/in. (0.350 in.)}$ <p>Shear rupture component</p> $\phi 0.6 F_u A_{nv} = 210 \text{ kips/in. (0.350 in.)}$ $\phi R_n = (210 \text{ kips/in.} + 39.6 \text{ kips/in.})(0.350 \text{ in.})$ $= 87.4 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check block shear rupture of the tee stem</i></p> $L_{eh} = L_{ev} = 1\frac{1}{4} \text{ in.},$ <p>Thus.</p> $\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$ <p>Tension rupture component</p> $F_u A_{nt} / \Omega = 26.4 \text{ kips/in. (0.350 in.)}$ <p>Shear yielding component</p> $0.6 F_y A_{gv} / \Omega = 154 \text{ kips/in. (0.350 in.)}$ <p>Shear rupture component</p> $0.6 F_u A_{nv} / \Omega = 140 \text{ kips/in. (0.350 in.)}$ $\frac{R_n}{\Omega} = (140 \text{ kips/in.} + 26.4 \text{ kips/in.})(0.350 \text{ in.})$ $= 58.2 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-5
		Manual Table 9-3a
		Manual Table 9-3b
		Manual Table 9-3c

<p><i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i></p> <p>From the table, single shear, four rows of 3/4-in. diameter bolts and an uncoped beam with <math>F_y = 50</math> ksi and <math>F_u = 65</math> ksi,</p> $\phi R_n = (702 \text{ kips/in.})(0.380 \text{ in.})/2$ $= 133 \text{ kips} > 36 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check the supporting column flange</i></p> <p>From the table, for column flange material with <math>F_y = 50</math> ksi, <math>n = 4</math>, <math>L = 11\frac{1}{2}</math>, Welds B, Weld size = 1/4-in. the minimum support thickness is 0.190 in.</p> $t_f = 0.710 \text{ in.} > 0.190 \text{ in.} \quad \mathbf{o.k.}$	<p><i>Check beam web for bolt bearing, block shear rupture, shear yielding and shear rupture</i></p> <p>From the table, single shear, four rows of 3/4-in. diameter bolts and an uncoped beam with <math>F_y = 50</math> ksi and <math>F_u = 65</math> ksi,</p> $R_n/\Omega = (468 \text{ kips/in.})(0.380 \text{ in.})/2$ $= 88.9 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Check the supporting column flange</i></p> <p>From the table, for column flange material with <math>F_y = 50</math> ksi, <math>n = 4</math>, <math>L = 11\frac{1}{2}</math>, Welds B, Weld size = 1/4-in. the minimum support thickness is 0.190 in.</p> $t_f = 0.710 \text{ in.} > 0.190 \text{ in.} \quad \mathbf{o.k.}$	<p>Manual Table 10-1</p>
		<p>Manual Table 10-2</p>

## **Chapter IIB**

# **Fully-Restrained (FR) Moment Connections**

The design of fully restrained (FR) moment connections is covered in Part 11 of the *AISC Steel Construction Manual*.



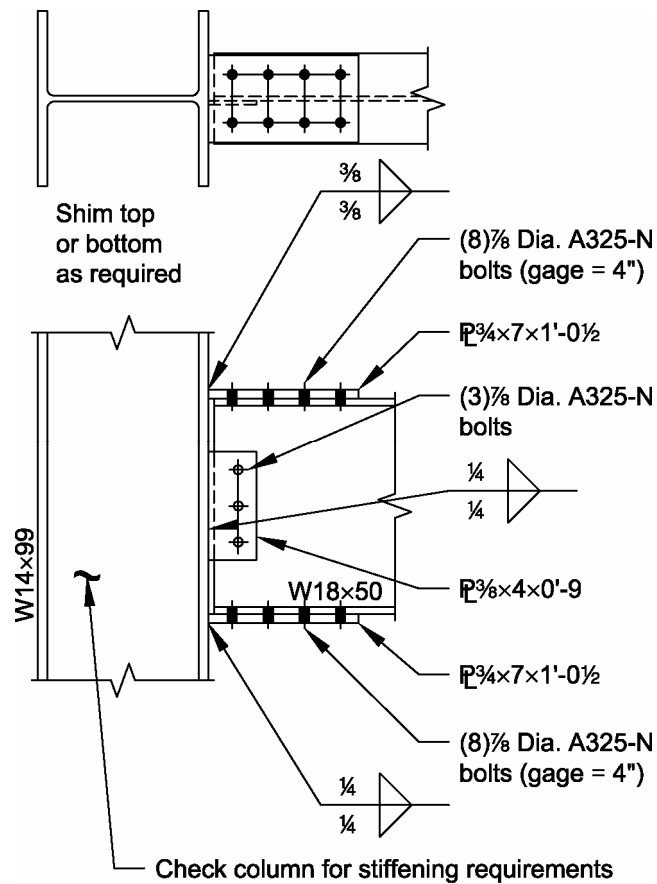
## Example II.B-1 Bolted Flange-Plate FR Moment Connection (beam-to-column flange)

### 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:

$$\begin{aligned} R_D &= 7 \text{ kips} & M_D &= 42 \text{ kip-ft} \\ R_L &= 21 \text{ kips} & M_L &= 126 \text{ kip-ft} \end{aligned}$$

Use  $\frac{7}{8}$ -in. diameter ASTM A325-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 2-2
Plate		ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	Table 2-3

### 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. <sup>3</sup>	Manual
Column	W14×99	$d = 14.2$ in.	$b_f = 14.6$ in.	$t_f = 0.780$ in.			Table 1-1

**Solution:**

LRFD	ASD
$R_u = 1.2(7 \text{ kips}) + 1.6(21 \text{ kips})$ $= 42 \text{ kips}$	$R_a = 7 \text{ kips} + 21 \text{ kips}$ $= 28 \text{ kips}$
$M_u = 1.2(42 \text{ kip-ft}) + 1.6(126 \text{ kip-ft})$ $= 252 \text{ kip-ft}$	$M_a = 42 \text{ kip-ft} + 126 \text{ kip-ft}$ $= 168 \text{ kip-ft}$

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

$$A_{fn} = A_{fg} - 2(d_b + \frac{1}{8} \text{ in.})t_f = 4.28 \text{ in.}^2 - 2(\frac{7}{8} \text{ in.} + \frac{1}{8} \text{ in.})(0.570 \text{ in.}) = 3.14 \text{ in.}^2$$

$$\frac{F_y}{F_u} = \frac{50 \text{ ksi}}{65 \text{ ksi}} = 0.769 < 0.8, \text{ therefore } Y_t = 1.0.$$

$$F_u A_{fn} = (65 \text{ ksi})(3.14 \text{ in.}^2) = 204 \text{ kips}$$

$$Y_t F_y A_{fg} = (1.0)(50 \text{ ksi})(4.28 \text{ in.}^2) = 214 \text{ kips} > 204 \text{ kips}$$

Therefore the nominal flexural strength,  $M_n$ , at the location of the holes in the tension flange is as follows:

$$M_n = \frac{F_u A_{fn} S_x}{A_{fg}} = \frac{(65 \text{ ksi})(3.14 \text{ in.}^2)(88.9 \text{ in.}^3)}{4.28 \text{ in.}^2} = 4240 \text{ kip-in. or } 353 \text{ kip-ft}$$

Specification  
Eqn. F13-1

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi_b M_n = 0.90(353 \text{ kip-ft}) = 318 \text{ kip-ft}$	$M_n / \Omega_b = \frac{353 \text{ kip-ft}}{1.67} = 211 \text{ kip-ft}$
$318 \text{ kip-ft} > 252 \text{ kip-ft}$ <b>o.k.</b>	$211 \text{ kip-ft} > 168 \text{ kip-ft}$ <b>o.k.</b>

Design single-plate web connection

Try a PL  $\frac{3}{8} \times 4 \times 0'-9$ , with three  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts and  $\frac{1}{4}$ -in. fillet welds.

LRFD	ASD	
<i>Design shear strength of the bolts</i>	<i>Allowable shear strength of bolts</i>	Manual Table 7-1
Single shear; $\phi r_n = 21.6$ kips/bolt $= 42$ kips/(21.6 kips/bolt) = 1.94 bolts	Single shear; $r_n/\Omega = 14.4$ kips/bolt $= 28$ kips/(14.4 kips/bolt) = 1.94 bolts	
<i>Bearing strength of bolts</i>	<i>Bearing strength of bolts</i>	Manual Table 7-5
Bolt spacing = 3 in. $\phi r_n = (91.4$ kips/in./bolt)( $\frac{3}{8}$ in.) $= 34.3$ kips/bolt $= 42$ kips/(34.3 kips/bolt) = 1.23 bolts	Bolt spacing = 3 in. $r_n/\Omega = (60.9$ kips/in./bolt)( $\frac{3}{8}$ in.) $= 22.8$ kips/bolt $= 28$ kips/(22.8 kips/bolt) = 1.23 bolts	
<i>Plate shear yielding</i>	<i>Plate shear yielding</i>	
$\phi = 1.00$ $\phi R_n = 0.60 \phi F_y A_g$ $= 0.60(1.00)(36$ ksi)(9 in.)( $\frac{3}{8}$ in.) $= 72.9$ kips > 42 kips <b>o.k.</b>	$\Omega = 1.50$ $r_n/\Omega = 0.60 F_y A_g / \Omega =$ $= 0.60(36$ ksi)(9 in.)( $\frac{3}{8}$ in.)/(1.50) $= 48.6$ kips > 28 kips <b>o.k.</b>	Eqn J4-3
<i>Plate shear rupture</i>	<i>Plate shear rupture</i>	
Where $\phi = 0.75$ $\phi R_n = 0.60 \phi F_u A_{nv}$ (3 bolts)( $\frac{7}{8}$ in. + $\frac{1}{16}$ in. + $\frac{1}{16}$ in.) = 3 in. $A_{nv} = (9$ in. – 3 in.)( $\frac{3}{8}$ in.) = 2.25 in <sup>2</sup> $= 0.60(0.75)(58$ ksi)(2.25 in <sup>2</sup> ) $= 58.7$ kips > 42 kips <b>o.k.</b>	$\Omega = 2.00$ $r_n/\Omega = 0.60 F_u A_{nv} / \Omega =$ (3 bolts)( $\frac{7}{8}$ in. + $\frac{1}{16}$ in. + $\frac{1}{16}$ in.) = 3 in. $A_{nv} = (9$ in. – 3 in.)( $\frac{3}{8}$ in.) = 2.25 in <sup>2</sup> $= 0.60(58$ ksi)(2.25 in <sup>2</sup> )/(2.00) $= 39.2$ kips > 28 kips <b>o.k.</b>	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	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$	
Tension rupture component $\phi F_u A_{nt} = 32.6$ kips/in( $\frac{3}{8}$ in)	Tension rupture component $F_u A_{nt} / \Omega = 21.8$ kips/in( $\frac{3}{8}$ in)	Manual Table 9-3a

<p>Shear yielding component</p> $\phi 0.60F_y A_{gv} = 121 \text{ kips/in}(\frac{3}{8} \text{ in})$ <p>Shear rupture component</p> $\phi 0.60F_u A_{nv} = 131 \text{ kips/in}(\frac{3}{8} \text{ in})$ $\phi R_n = (121 \text{ kips/in} + 32.6 \text{ kips/in})(\frac{3}{8} \text{ in})$ $= 57.6 \text{ kips} > 42 \text{ kips} \quad \mathbf{o.k.}$	<p>Shear yielding component</p> $0.60F_y A_{gv} / \Omega = 81.0 \text{ kips/in}(\frac{3}{8} \text{ in})$ <p>Shear rupture component</p> $0.60F_u A_{nv} / \Omega = 87.0 \text{ kips/in}(\frac{3}{8} \text{ in})$ $R_n / \Omega = (81.0 \text{ kips/in} + 21.6 \text{ kips/in})(\frac{3}{8} \text{ in})$ $= 38.5 \text{ kips} > 24 \text{ kips} \quad \mathbf{o.k.}$	Manual Table 9-3b
<p><i>Weld Strength</i></p> $\phi R_n = 1.392D l (2)$ $= 1.392(4 \text{ sixteenths})(9 \text{ in.})(2)$ $= 100 \text{ kips} > 42 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Weld Strength</i></p> $R_n / \Omega = 0.928Dl (2)$ $= 0.928(4 \text{ sixteenths})(9 \text{ in.})(2)$ $= 66.8 \text{ kips} > 28 \text{ kips} \quad \mathbf{o.k.}$	Manual Part 8

*Connecting Elements Rupture Strength at Welds*

*Shear rupture strength of base metal*

$$t_{\min} = \frac{0.6F_{EXX} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{0.6F_u} = \frac{3.09D}{F_u}$$

Section J4.2

Column flange;  $t_f = 0.780 \text{ in.}$

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(4 \text{ sixteenths})}{65 \text{ ksi}} = 0.190 \text{ in.}$$

Plate;  $t = \frac{3}{8} \text{ in.}$

$$t_{\min} = \frac{3.09(2)D}{F_u} = \frac{(6.19)(4 \text{ sixteenths})}{58 \text{ ksi}} = 0.427 \text{ in.} > \frac{3}{8} \text{ in.} \quad \mathbf{proration \text{ required.}}$$

LRFD	ASD
$\phi R_n = \frac{\frac{3}{8} \text{ in.}}{0.427 \text{ in.}} (50.1 \text{ kips}) = 44 \text{ kips}$ $44 \text{ kips} > 42 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{\frac{3}{8} \text{ in.}}{0.427 \text{ in.}} (33.4 \text{ kips}) = 29.3 \text{ kips}$ $29.3 \text{ kips} > 28 \text{ kips} \quad \mathbf{o.k.}$

## 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 $\frac{3}{4} \times 7$	Try a PL $\frac{3}{4} \times 7$	
Determine critical bolt strength	Determine critical bolt strength	
For shear, $\phi r_n = 21.6 \text{ kips/bolt}$	For shear, $r_n / \Omega = 14.4 \text{ kips/bolt}$	Manual Table 7-1
For bearing on flange;	For bearing on flange;	
$\phi r_n = (102 \text{ kips/bolt})t_f$ $= (102 \text{ kips/bolt})(0.570 \text{ in.})$ $= 58.1 \text{ kips/bolt}$	$r_n / \Omega = (68.3 \text{ kips/bolt})t_f$ $= (68.3 \text{ kips/bolt})(0.570 \text{ in.})$ $= 38.9 \text{ kips/bolt}$	Manual Table 7-6
For bearing in plate;	For bearing in plate;	
$\phi r_n = (91.4 \text{ kips/bolt})t_f$ $= (91.4 \text{ kips/bolt})(0.570 \text{ in.})$ $= 52.1 \text{ kips/bolt}$	$r_n / \Omega = (60.9 \text{ kips/bolt})t_f$ $= (60.9 \text{ kips/bolt})(0.570 \text{ in.})$ $= 34.7 \text{ kips/bolt}$	
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

$$P_n = F_y A_g = (36 \text{ ksi})(7 \text{ in.})(\frac{3}{4} \text{ in.}) = 189 \text{ kips}$$

Eqn. D2-1

LRFD	ASD
$P_{uf} = \frac{M_u}{d + t_p} = \frac{(252 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.} + \frac{3}{4} \text{ in.})}$ $= 161 \text{ kips}$	$P_{af} = \frac{M_u}{d + t_p} = \frac{(168 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.} + \frac{3}{4} \text{ in.})}$ $= 108 \text{ kips}$
$\phi = 0.90$	$\Omega = 1.67$

$\phi P_n = 0.90(189 \text{ kips}) = 170 \text{ kips}$  $170 \text{ kips} > 161 \text{ kips}$	$P_n / \Omega = \frac{189 \text{ kips}}{1.67} = 113 \text{ kips}$  $113 \text{ kips} > 108 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Check flange plate tension rupture

$$A_n \leq 0.85 A_g = 0.85(7 \text{ in.})(0.75 \text{ in.}) = 4.46 \text{ in.}^2$$

Eqn. J4-1

$$A_n = [B - 2(d_b + \frac{1}{8} \text{ in.})]t_p = [(7 \text{ in.}) - 2(\frac{7}{8} \text{ in.} + \frac{1}{8} \text{ in.})](\frac{3}{4} \text{ in.}) = 3.75 \text{ in.}^2$$

$$A_e = 3.75 \text{ in.}^2$$

$$P_n = F_u A_e = (58 \text{ ksi})(3.75 \text{ in.}^2) = 217 \text{ kips}$$

Eqn. D2-2

LRFD	ASD
$\phi = 0.75$  $\phi P_n = 0.75(217 \text{ kips}) = 163 \text{ kips}$  $163 \text{ kips} > 161 \text{ kips}$	$\Omega = 2.00$  $P_n / \Omega = \frac{217 \text{ kips}}{2.0} = 109 \text{ kips}$  $109 \text{ kips} > 108 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

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  $L_{eh} = 1\frac{1}{2} \text{ in.}$  and  $L_{ev} = 1\frac{1}{2} \text{ in.}$  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  $L_{eh} = 2 \text{ in.}$  and  $L_{ev} = 1\frac{1}{2} \text{ in.}$  Thus, the former case is the more critical.

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$
Tension component $\phi U_{bs} F_u A_{nt} = 43.5 \text{ kips } (\frac{3}{4} \text{ in.})(2)$	Tension component $F_u A_{nt} / \Omega = 29.0 \text{ kips } (\frac{3}{4} \text{ in.})(2)$
Shear yielding component $\phi 0.6 F_y A_{gv} = 170 \text{ kips } (\frac{3}{4} \text{ in.})(2)$	Shear yielding component $0.6 F_y A_{gv} / \Omega = 113 \text{ kips } (\frac{3}{4} \text{ in.})(2)$
Shear rupture component $\phi 0.6 F_u A_{nv} = 183 \text{ kips } (\frac{3}{4} \text{ in.})(2)$	Shear rupture component $0.6 F_u A_{nv} / \Omega = 122 \text{ kips } (\frac{3}{4} \text{ in.})(2)$
Shear yielding controls, thus  $\phi R_n = (170 \text{ kips} + 43.5 \text{ kips})(\frac{3}{4} \text{ in.})(2)$  $= 320 \text{ kips} > 161 \text{ kips}$	Shear yielding controls, thus  $R_n / \Omega = (113 \text{ kips} + 29.0 \text{ kips})(\frac{3}{4} \text{ in.})(2)$  $= 213 \text{ kips} > 108 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Manual Table 9-3a

Manual Table 9-3b

Manual Table 9-3c

Determine the required size of the fillet weld to supporting column flange

The applied tension load is perpendicular to the weld,  
therefore  $\theta = 90^\circ$  and  $1.0 + 0.5 \sin^{1.5} \theta = 1.5$ .

Eqns. J2-4,  
and J2-5

LRFD	ASD
$D_{\min} = \frac{P_{af}}{2(1.5)(1.392)l}$ $= \frac{161 \text{ kips}}{2(1.5)(1.392)(7 \text{ in.})}$ $= 5.51 \text{ sixteenths}$	$D_{\min} = \frac{P_{af}}{2(1.5)(0.928)l}$ $= \frac{108 \text{ kips}}{2(1.5)(0.928)(7 \text{ in.})}$ $= 5.54 \text{ sixteenths}$
Use $\frac{3}{8}$ -in. fillet welds, $6 > 5.51$ <b>o.k.</b>	Use $\frac{3}{8}$ -in. fillet welds, $6 > 5.54$ <b>o.k.</b>

Connecting Elements Rupture Strength at Welds

Tension rupture strength of base metal

$$t_{\min} = \frac{0.6F_{EXX} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{F_u} = \frac{1.86D}{F_u}$$

Column flange;  $t_f = 0.780$  in.

$$t_{\min} = \frac{1.86D}{F_u} = \frac{(1.86)(6 \text{ sixteenths})}{65 \text{ ksi}} = 0.171 \text{ in.}$$

Flange plate;  $t_f = 0.75$  in.

$$t_{\min} = \frac{3.71D}{F_u} = \frac{(1.86)(2)(6 \text{ sixteenths})}{58 \text{ ksi}} = 0.384 \text{ in.}$$

Design compression flange plate and connection

Try PL  $\frac{3}{4} \times 7$

Assume  $K = 0.65$  and  $l = 2.0$  in. ( $1\frac{1}{2}$  in. edge distance and  $\frac{1}{2}$  in. setback).

$$\frac{Kl}{r} = \frac{0.65(2.0 \text{ in.})}{\left( \frac{\frac{3}{4} \text{ in.}}{\sqrt{12}} \right)} = 6.00 < 25$$

Therefore,  $F_{cr} = F_y$

$$A = (7 \text{ in.}) \left( \frac{3}{4} \text{ in.} \right) = 5.25 \text{ in.}^2$$

Section J4.3

LRFD	ASD
$\phi = 0.90$ $\phi P_n = \phi F_y A_g = 0.90(36 \text{ ksi})(5.25 \text{ in.}^2)$ $= 170 \text{ kips} > 161 \text{ kips}$	$\Omega = 1.67$ $P_n / \Omega = \frac{F_y A_g}{\Omega} = \frac{(36 \text{ ksi})(5.25 \text{ in.}^2)}{1.67}$ $= 113 \text{ kips} > 108 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Eqn. J4-6

The compression flange plate will be identical to the tension flange plate; a  $\frac{3}{4}$  in.  $\times$  7 in. plate with eight bolts in two rows of four bolts on a 4 in. gage and  $\frac{3}{8}$  in. fillet welds to the supporting column flange.

Note: Tension due to load reversal must also be considered in the design of the fillet weld to the supporting column flange.

The column must be checked for stiffening requirements. For further information, see AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections – Wind and Seismic Applications* (Carter, 1999).



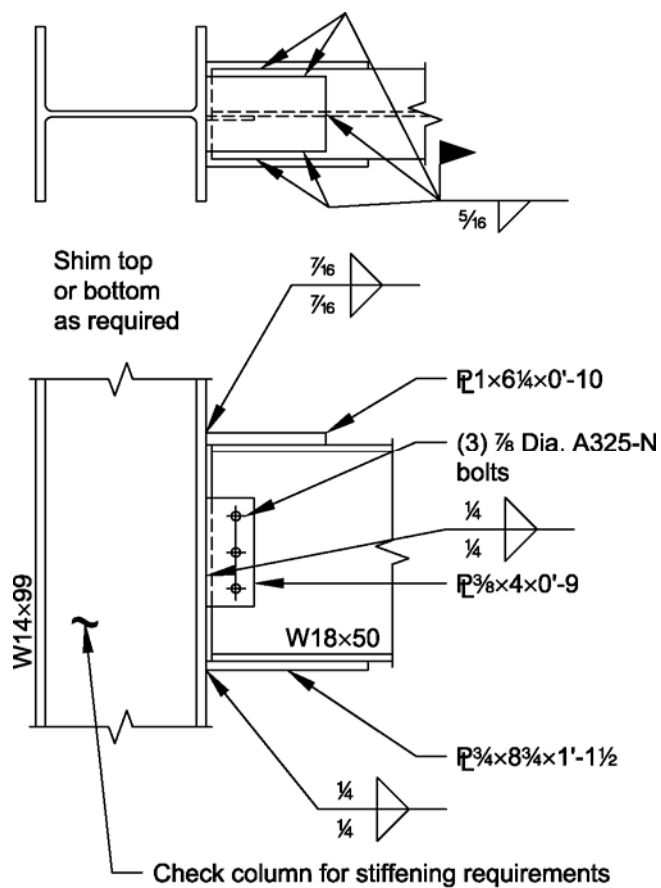
## Example II.B-2 Welded Flange-Plated FR Moment Connection (beam-to-column flange)

### 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:

$$\begin{aligned} R_D &= 7 \text{ kips} & M_D &= 42 \text{ kip-ft} \\ R_L &= 21 \text{ kips} & M_L &= 126 \text{ kip-ft} \end{aligned}$$

Use  $\frac{7}{8}$ -in. diameter ASTM A325-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. <sup>3</sup>	Manual
Column	W14×99	$d = 14.2$ in.	$b_f = 14.6$ in.	$t_f = 0.780$ in.			Table 1-1

**Solution:**

LRFD	ASD
$R_u = 1.2(7 \text{ kips}) + 1.6(21 \text{ kips}) = 42 \text{ kips}$	$R_a = 7 \text{ kips} + 21 \text{ kips} = 28 \text{ kips}$
$M_u = 1.2(42 \text{ kip-ft}) + 1.6(126 \text{ kip-ft})$ $= 252 \text{ kip-ft}$	$M_a = 42 \text{ kip-ft} + 126 \text{ kip-ft}$ $= 168 \text{ kip-ft}$

*Check the beam flexural strength*

LRFD	ASD
For a W18×50	For a W18×50
$\phi_b M_n = 379 \text{ kip-ft} > 252 \text{ kip-ft}$ <b>o.k.</b>	$M_n/\Omega_b = 252 \text{ kip-ft} > 168 \text{ kip-ft}$ <b>o.k.</b>

Manual  
Table 3-3*Design the single-plate web connection*Try a PL  $\frac{3}{8} \times 4 \times 0'-9$ , with three  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts and  $\frac{1}{4}$  in. fillet welds.

LRFD	ASD
<i>Shear strength of bolts</i>	<i>Shear strength of bolts</i>
Single shear;	Single shear;
$\phi r_n = 21.6 \text{ kips/bolt}$ $= 42 \text{ kips}/(21.6 \text{ kips/bolt}) = 1.94 \text{ bolts}$	$r_n/\Omega = 14.4 \text{ kips/bolt}$ $= 28 \text{ kips}/(21.6 \text{ kips/bolt}) = 1.94 \text{ bolts}$
<i>Bearing Strength of bolts</i>	<i>Bearing Strength of bolts</i>
Bolt spacing = 3 in.	Bolt spacing = 3 in.
$\phi r_n = (91.4 \text{ kips/in./bolt})(\frac{3}{8} \text{ in.})$ $= 34.3 \text{ kips/bolt}$ $= 42 \text{ kips}/(34.3 \text{ kips/bolt}) = 1.22 \text{ bolts}$	$r_n/\Omega = (60.9 \text{ kips/in./bolt})(\frac{3}{8} \text{ in.})$ $= 22.8 \text{ kips/bolt}$ $= 28 \text{ kips}/(22.8 \text{ kips/bolt}) = 1.22 \text{ bolts}$
<i>Plate shear yielding</i>	<i>Plate shear yielding</i>
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 0.60 \phi F_y A_g$ $= 0.60(1.00)(36 \text{ ksi})(9 \text{ in.})(\frac{3}{8} \text{ in.})$ $= 72.9 \text{ kips} > 42 \text{ kips}$ <b>o.k.</b>	$r_n/\Omega = 0.60 F_y A_g / \Omega$ $= 0.60(36 \text{ ksi})(9 \text{ in.})(\frac{3}{8} \text{ in.})/(1.50)$ $= 48.6 \text{ kips} > 28 \text{ kips}$ <b>o.k.</b>

Manual  
Table 7-1Manual  
Table 7-5

Eqn J4-3

<p><i>Plate shear rupture</i></p> <p><math>\phi = 0.75</math></p> <p><math>\phi R_n = 0.60 \phi F_u A_{nv}</math></p> <p>(3 bolts)(<math>\frac{7}{8}</math> in. + <math>\frac{1}{16}</math> in. + <math>\frac{1}{16}</math> in.) = 3 in.</p> <p><math>A_{nv} = (9 \text{ in.} - 3 \text{ in.})(\frac{3}{8} \text{ in.}) = 2.25 \text{ in}^2</math></p> <p><math>= 0.60(0.75)(58 \text{ ksi})(2.25 \text{ in}^2)</math></p> <p><math>= 58.7 \text{ kips} &gt; 42 \text{ kips} \quad \mathbf{o.k.}</math></p>	<p><i>Plate shear rupture</i></p> <p><math>\Omega = 2.00</math></p> <p><math>r_n/\Omega = 0.60 F_u A_{nv} / \Omega</math></p> <p>(3 bolts)(<math>\frac{7}{8}</math> in. + <math>\frac{1}{16}</math> in. + <math>\frac{1}{16}</math> in.) = 3 in.</p> <p><math>A_{nv} = (9 \text{ in.} - 3 \text{ in.})(\frac{3}{8} \text{ in.}) = 2.25 \text{ in}^2</math></p> <p><math>= 0.60(58 \text{ ksi})(2.25 \text{ in}^2)/(2.00)</math></p> <p><math>= 39.2 \text{ kips} &gt; 28 \text{ kips} \quad \mathbf{o.k.}</math></p>
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Eqn J4-4

*Block shear rupture strength for plate*

$L_{eh} = 1\frac{1}{4}$  in.;  $L_{ev} = 1\frac{1}{2}$  in.;  $U_{bs} = 1.0$ ;  $n = 3$

LRFD	ASD
<p><math>\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})</math></p> <p>Tension component</p> <p><math>\phi U_{bs} F_u A_{nt} = 32.6 \text{ kips/in}(\frac{3}{8} \text{ in})</math></p> <p>Shear yielding</p> <p><math>\phi 0.6 F_y A_{gv} = 121 \text{ kips/in}(\frac{3}{8} \text{ in})</math></p> <p>Shear rupture</p> <p><math>\phi 0.6 F_u A_{nv} = 131 \text{ kips/in}(\frac{3}{8} \text{ in})</math></p> <p><math>\phi R_n = (121 \text{ kips/in} + 32.6 \text{ kips/in})(\frac{3}{8} \text{ in})</math></p> <p><math>= 57.6 \text{ kips} &gt; 42 \text{ kips} \quad \mathbf{o.k.}</math></p>	<p><math>\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)</math></p> <p>Tension component</p> <p><math>U_{bs} F_u A_{nt} / \Omega = 21.8 \text{ kips/in}(\frac{3}{8} \text{ in})</math></p> <p>Shear yielding</p> <p><math>0.6 F_y A_{gv} / \Omega t = 81.0 \text{ kips/in}(\frac{3}{8} \text{ in})</math></p> <p>Shear rupture</p> <p><math>0.6 F_u A_{nv} / \Omega = 87.0 \text{ kips/in}(\frac{3}{8} \text{ in})</math></p> <p><math>R_n / \Omega = (81.0 \text{ kips/in} + 21.6 \text{ kips/in})(\frac{3}{8} \text{ in})</math></p> <p><math>= 38.5 \text{ kips} &gt; 24 \text{ kips} \quad \mathbf{o.k.}</math></p>

Manual  
Table 9-3a

Manual  
Table 9-3b

Manual  
Table 9-3c

*Weld Strength*

<p><math>\phi R_n = 1.392 D I(2)</math></p> <p><math>= 1.392(4 \text{ sixteenths})(9 \text{ in.})(2)</math></p> <p><math>= 100 \text{ kips} &gt; 42 \text{ kips} \quad \mathbf{o.k.}</math></p>	<p><math>R_n / \Omega = 0.928 D I(2)</math></p> <p><math>= 0.928(4 \text{ sixteenths})(9 \text{ in.})(2)</math></p> <p><math>= 66.8 \text{ kips} &gt; 28 \text{ kips} \quad \mathbf{o.k.}</math></p>
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Manual  
Part 8

*Connecting Elements Rupture Strength at Welds*

*Shear rupture strength of base metal*

$$t_{\min} = \frac{0.6 F_{EXX} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{0.6 F_u} = \frac{3.09 D}{F_u}$$

Column flange;  $t_f = 0.780$  in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(4 \text{ sixteenths})}{65 \text{ ksi}} = 0.190 \text{ in.}$$

Plate;  $t = \frac{3}{8}$  in.

$$t_{\min} = \frac{3.09(2)D}{F_u} = \frac{(6.19)(4 \text{ sixteenths})}{58 \text{ ksi}} = 0.427 \text{ in.} > \frac{3}{8} \text{ in.} \quad \text{proration required}$$

LRFD	ASD
$\phi R_n = \frac{\frac{3}{8} \text{ in.}}{0.427 \text{ in.}} (50.1 \text{ kips}) = 44 \text{ kips}$ <p style="text-align: center;">44 kips &gt; 42 kips      <b>o.k.</b></p>	$\frac{R_n}{\Omega} = \frac{\frac{3}{8} \text{ in.}}{0.427 \text{ in.}} (33.4 \text{ kips}) = 29.3 \text{ kips}$ <p style="text-align: center;">29.3 kips &gt; 28 kips      <b>o.k.</b></p>

*Design tension flange plate and connection*

Determine the flange force

LRFD	ASD
$P_{af} = \frac{M_u}{d + t_p}$ $= \frac{(252 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.} + \frac{3}{4} \text{ in.})} = 161 \text{ kips}$	$P_{af} = \frac{M_a}{d + t_p}$ $= \frac{(168 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.} + \frac{3}{4} \text{ in.})} = 108 \text{ kips}$

Manual  
Part 12

The top flange width  $b_f = 7\frac{1}{2}$  in. Assume a shelf dimension of  $\frac{5}{8}$  in. on both sides of the plate. The plate width, then, is  $7\frac{1}{2}$  in.  $- 2(\frac{5}{8} \text{ in.}) = 6\frac{1}{4}$  in. Try a 1 in.  $\times$   $6\frac{1}{4}$  in. flange plate.

*Check top flange plate tension yielding*

$$R_n = F_y A_g = (36 \text{ ksi})(6\frac{1}{4} \text{ in.})(1 \text{ in.}) = 225 \text{ kips}$$

Eqn. D2-1

LRFD	ASD
$\phi = 0.90$ $\phi R_n = 0.90(225 \text{ kips}) = 203 \text{ kips}$ <p style="text-align: center;">203 kips &gt; 161 kips      <b>o.k.</b></p>	$\Omega = 1.67$ $R_n / \Omega = \frac{225 \text{ kips}}{1.67} = 135 \text{ kips}$ <p style="text-align: center;">135 kips &gt; 108 kips      <b>o.k.</b></p>

Determine the force in the welds

LRFD	ASD
$P_{af} = \frac{M_u}{d}$ $= \frac{(252 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.})} = 168 \text{ kips}$	$P_{af} = \frac{M_a}{d}$ $= \frac{(168 \text{ kip-ft})(12 \text{ in./ft})}{(18.0 \text{ in.})} = 112 \text{ kips}$

Manual  
Part 12

Determine the required weld size and length for fillet welds to beam flange. Try a  $\frac{5}{16}$ -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
$l_{min} = \frac{P_{uf}}{1.392D} = \frac{168 \text{ kips}}{1.392(5)} = 24.1 \text{ in.}$	$l_{min} = \frac{P_{af}}{0.928D} = \frac{112 \text{ kips}}{0.928(5)} = 24.1 \text{ in.}$
Use 9 in. of weld along each side and $6\frac{1}{4}$ in. of weld along the end of the flange plate.	Use 9 in. of weld along each side and $6\frac{1}{4}$ in. of weld along the end of the flange plate.
$l = 2(9 \text{ in.}) + 6\frac{1}{4} \text{ in.}$	$l = 2(9 \text{ in.}) + 6\frac{1}{4} \text{ in.}$
$= 24.2 \text{ in.} > 24.1 \text{ in.}$ <b>o.k.</b>	$= 24.2 \text{ in.} > 24.1 \text{ in.}$ <b>o.k.</b>

Manual  
Part 8

### Connecting Elements Rupture Strength at Welds

#### Shear rupture strength of base metal

$$t_{min} = \frac{0.6F_{exx} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{0.6F_u} = \frac{3.09D}{F_u}$$

Beam flange;  $t_f = 0.570 \text{ in.}$

$$t_{min} = \frac{3.09D}{F_u} = \frac{(3.09)(5 \text{ sixteenths})}{65 \text{ ksi}} = 0.238 \text{ in.}$$

Flange plate;  $t_p = 1.00 \text{ in.}$

$$t_{min} = \frac{3.09D}{F_u} = \frac{(3.09)(5 \text{ sixteenths})}{58 \text{ ksi}} = 0.266 \text{ in.}$$

#### Tension rupture strength of base metal

$$t_{min} = \frac{0.6F_{exx} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{F_u} = \frac{1.86D}{F_u}$$

Beam flange;  $t_f = 0.570 \text{ in.}$

$$t_{min} = \frac{1.86D}{F_u} = \frac{(1.86)(5 \text{ sixteenths})}{65 \text{ ksi}} = 0.143 \text{ in.}$$

Flange plate;  $t_f = 1.00$  in.

$$t_{\min} = \frac{1.86D}{F_u} = \frac{(1.86)(5 \text{ sixteenths})}{58 \text{ ksi}} = 0.160 \text{ in.}$$

Determine the required fillet weld size to supporting column flange

Section J2.4  
Eqn. J2-5

The applied tension load is perpendicular to the weld, therefore  $\theta = 90^\circ$  and  $1.0 + \sin^{1.5} \theta = 1.5$ .

LRFD	ASD
$D_{\min} = \frac{P_{uf}}{2(1.5)(1.392)l}$ $= \frac{168 \text{ kips}}{2(1.5)(1.392)(6\frac{1}{4} \text{ in.})}$ $= 6.44 \text{ sixteenths}$	$D_{\min} = \frac{P_{af}}{2(1.5)(0.928)l}$ $= \frac{112 \text{ kips}}{2(1.5)(0.928)(6\frac{1}{4} \text{ in.})}$ $= 6.44 \text{ sixteenths}$
Use $\frac{7}{16}$ -in. fillet welds, $7 > 6.44$ <b>o.k.</b>	Use $\frac{7}{16}$ -in. fillet welds, $7 > 6.44$ <b>o.k.</b>

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*

*Tension rupture strength of base metal*

$$t_{\min} = \frac{0.6F_{exx} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{F_u} = \frac{1.86D}{F_u}$$

Column flange;  $t_f = 0.780$  in.

$$t_{\min} = \frac{1.86D}{F_u} = \frac{(1.86)(7 \text{ sixteenths})}{65 \text{ ksi}} = 0.200 \text{ in.}$$

flange plate;  $t_p = 1.00$  in.

$$t_{\min} = \frac{1.86(2)D}{F_u} = \frac{(3.71)(7 \text{ sixteenths})}{58 \text{ ksi}} = 0.448 \text{ in.}$$

*Design compression flange plate and connection*

The compression flange plate should have approximately the same area as the tension flange plate ( $A = 6\frac{1}{4} \text{ in.} \times 1 \text{ in.} = 6.25 \text{ in.}^2$ ). Assume a shelf dimension of  $\frac{5}{8}$  in. The plate width, then, is  $7\frac{1}{2} \text{ in.} + 2(\frac{5}{8} \text{ in.}) = 8\frac{3}{4} \text{ in.}$  To approximately balance the flange plate areas, try a  $\frac{3}{4} \text{ in.} \times 8\frac{3}{4} \text{ in.}$  compression flange plate.

Assume  $K = 0.65$  and  $l = 2.0$  in. ( $1\frac{1}{2}$  in. edge distance and  $\frac{1}{2}$ -in. setback).

$$\frac{Kl}{r} = \frac{0.65(2.0 \text{ in.})}{\left(\frac{\frac{3}{4} \text{ in.}}{\sqrt{12}}\right)} = 6.00 < 25$$

$$A = (8\frac{3}{4} \text{ in.})(\frac{3}{4} \text{ in.}) = 6.56 \text{ in.}^2$$

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$\phi R_n = \phi F_y A_g = 0.90(36 \text{ ksi})(6.56 \text{ in.}^2)$	$R_n / \Omega = \frac{F_y A_g}{\Omega} = \frac{(36 \text{ ksi})(6.56 \text{ in.}^2)}{1.67}$
$= 212 \text{ kips} > 161 \text{ kips}$ <b>o.k.</b>	$= 141 \text{ kips} > 108 \text{ kips}$ <b>o.k.</b>

Eqn. E3-1

*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  $\frac{5}{16}$  in. fillet weld and  $12\frac{1}{2}$  in. of weld along each side of the beam flange.

*Connecting Elements Rupture Strength at Welds*

*Shear rupture strength of base metal*

$$t_{\min} = \frac{0.6F_{\text{exx}} \left(\frac{\sqrt{2}}{2}\right) \left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u}$$

Beam flange;  $t_f = 0.570$  in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5 \text{ sixteenths})}{65 \text{ ksi}} = 0.238 \text{ in.}$$

Bottom flange plate;  $t_p = 0.750$  in.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(5 \text{ sixteenths})}{58 \text{ ksi}} = 0.266 \text{ in.}$$

The column must be checked for stiffening requirements. For further information, see AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections – Wind and Seismic Applications*. (Carter, 1999).

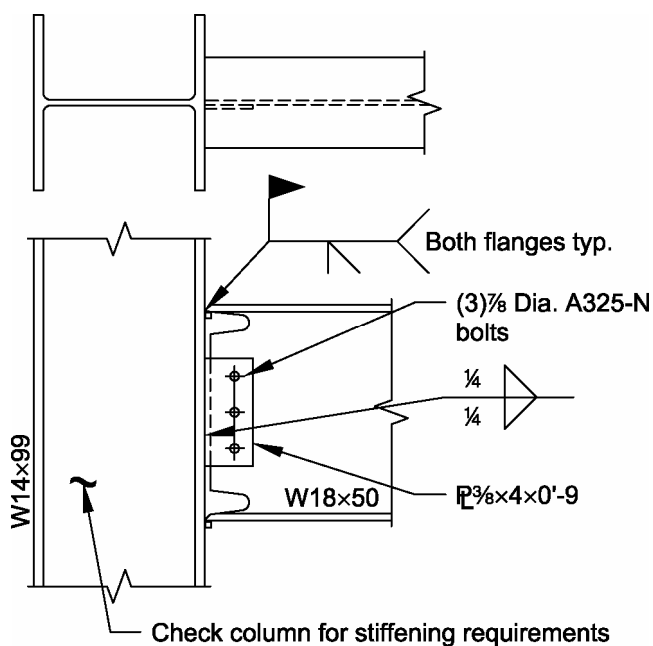
### Example II.B-3 Directly-Welded Flange FR Moment Connection (beam-to-column flange).

#### 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:

$$\begin{aligned} R_D &= 7 \text{ kips} & M_D &= 42 \text{ kip-ft} \\ R_L &= 21 \text{ kips} & M_L &= 126 \text{ kip-ft} \end{aligned}$$

Use  $\frac{7}{8}$ -in. diameter ASTM A325-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. <sup>3</sup>	Manual
Column	W14×99	$d = 14.2$ in.	$b_f = 14.6$ in.	$t_f = 0.780$ in.			Table 1-1



**Solution:**

LRFD	ASD
$R_u = 1.2(7 \text{ kips}) + 1.6(21 \text{ kips}) = 42 \text{ kips}$	$R_a = 7 \text{ kips} + 21 \text{ kips} = 28 \text{ kips}$
$M_u = 1.2(42 \text{ kip-ft}) + 1.6(126 \text{ kip-ft})$ $= 252 \text{ kip-ft}$	$M_a = 42 \text{ kip-ft} + 126 \text{ kip-ft}$ $= 168 \text{ kip-ft}$

*Check beam flexural strength*

LRFD	ASD
For a W18×50	For a W18×50
$\phi_b M_n = 379 \text{ kip-ft} > 252 \text{ kip-ft}$ <b>o.k.</b>	$M_n/\Omega_b = 252 \text{ kip-ft} > 168 \text{ kip-ft}$ <b>o.k.</b>

Manual  
Table 3-2*Design single-plate web connection*Try a PL  $\frac{3}{8} \times 4 \times 0'-9$ , with three  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts and  $\frac{1}{4}$  in. fillet welds.

LRFD	ASD
<i>Shear strength of bolts</i>	<i>Shear strength of bolts</i>
Single shear;	Single shear;
$\phi r_n = 21.6 \text{ kips/bolt}$ $= 42 \text{ kips}/(21.6 \text{ kips/bolt}) = 1.94 \text{ bolts}$	$r_n/\Omega = 14.4 \text{ kips/bolt}$ $= 28 \text{ kips}/(14.4 \text{ kips/bolt}) = 1.94 \text{ bolts}$
<i>Bearing Strength of bolts</i>	<i>Bearing Strength of bolts</i>
Bolt spacing = 3 in.	Bolt spacing = 3 in.
$\phi r_n = (91.4 \text{ kips/in./bolt})(\frac{3}{8} \text{ in.})$ $= 34.3 \text{ kips/bolt}$ $= 42 \text{ kips}/(34.3 \text{ kips/bolt}) = 1.47 \text{ bolts}$	$r_n/\Omega = (60.9 \text{ kips/in./bolt})(\frac{3}{8} \text{ in.})$ $= 22.8 \text{ kips/bolt}$ $= 28 \text{ kips}/(22.8 \text{ kips/bolt}) = 1.22 \text{ bolts}$
<i>Plate shear yielding</i>	<i>Plate shear yielding</i>
$\phi = 1.00$	$\Omega = 1.50$
$\phi R_n = 0.60\phi F_y A_g$ $= 0.60(1.00)(36 \text{ ksi})(9 \text{ in.})(\frac{3}{8} \text{ in.}) =$ $= 72.9 \text{ kips} > 42 \text{ kips}$ <b>o.k.</b>	$r_n/\Omega = 0.60F_y A_g/\Omega =$ $= 0.60(36 \text{ ksi})(9 \text{ in.})(\frac{3}{8} \text{ in.})/(1.50)$ $= 48.6 \text{ kips} > 28 \text{ kips}$ <b>o.k.</b>
<i>Plate shear rupture</i>	<i>Plate shear rupture</i>
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.60\phi F_u A_{nv}$ $(3 \text{ bolts})(\frac{7}{8} \text{ in.} + \frac{1}{16} \text{ in.} + \frac{1}{16} \text{ in.}) = 3 \text{ in.}$	$r_n/\Omega = 0.60F_u A_{nv}/\Omega$ $(3 \text{ bolts})(\frac{7}{8} \text{ in.} + \frac{1}{16} \text{ in.} + \frac{1}{16} \text{ in.}) = 3 \text{ in.}$

Manual  
Table 7-1Manual  
Table 7-5

Eqn. J4-3

Eqn. J4-4

$A_{nv} = (9 \text{ in.} - 3 \text{ in.})(\frac{3}{8} \text{ in.}) = 2.25 \text{ in}^2$ $= 0.60(0.75)(58 \text{ ksi})(2.25 \text{ in}^2)$ $= 58.7 \text{ kips} > 42 \text{ kips} \quad \mathbf{o.k.}$	$A_{nv} = (9 \text{ in.} - 3 \text{ in.})(\frac{3}{8} \text{ in.}) = 2.25 \text{ in}^2$ $= 0.60(58 \text{ ksi})(2.25 \text{ in}^2)/(2.00)$ $= 39.2 \text{ kips} > 28 \text{ kips} \quad \mathbf{o.k.}$
---	--

*Block shear rupture strength for plate*

$L_{eh} = 1\frac{1}{4} \text{ in.}; L_{ev} = 1\frac{1}{2} \text{ in.}; U_{bs} = 1.0; n = 3$

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$	Manual Table 9-3a
Tension component	Tension component	Manual Table 9-3b
$\phi F_u A_{nt} = 32.6 \text{ kips/in}(\frac{3}{8} \text{ in})$	$F_u A_{nt} / \Omega = 21.8 \text{ kips/in}(\frac{3}{8} \text{ in})$	
Shear yielding component	Shear yielding component	Manual Table 9-3c
$\phi 0.6 F_y A_{gv} = 121 \text{ kips/in}(\frac{3}{8} \text{ in})$	$0.6 F_y A_{gv} / \Omega = 81.0 \text{ kips/in}(\frac{3}{8} \text{ in})$	
Shear rupture component	Shear rupture component	
$\phi 0.6 F_u A_{nv} = 131 \text{ kips/in}(\frac{3}{8} \text{ in})$	$0.6 F_u A_{nv} / \Omega = 87.0 \text{ kips/in}(\frac{3}{8} \text{ in})$	
$\phi R_n = (121 \text{ kips/in} + 32.6 \text{ kips/in})(\frac{3}{8} \text{ in})$ $= 57.6 \text{ kips} > 42 \text{ kips}$	$R_n / \Omega = (81.0 \text{ kips/in} + 21.6 \text{ kips/in})(\frac{3}{8} \text{ in})$ $= 38.5 \text{ kips} > 24 \text{ kips}$	
Weld Strength	Weld Strength	Manual Part 8
$\phi R_n = 1.392 D I(2)$ $= 1.392(4 \text{ sixteenths})(9 \text{ in.})$ $= 50.1 \text{ kips} > 42 \text{ kips}$	$R_n / \Omega = 0.928 D I(2)$ $= 0.928(4 \text{ sixteenths})(9 \text{ in.})$ $= 33.4 \text{ kips} > 28 \text{ kips}$	

*Connecting Elements Rupture Strength at Welds*

*Shear rupture strength of base metal*

$$t_{\min} = \frac{0.6 F_{exx} \left(\frac{\sqrt{2}}{2}\right) \left(\frac{D}{16}\right)}{0.6 F_u} = \frac{3.09 D}{F_u}$$

Column flange  $t_f = 0.780 \text{ in.}$

$$t_{\min} = \frac{3.09 D}{F_u} = \frac{(3.09)(4 \text{ sixteenths})}{65 \text{ ksi}} = 0.190 \text{ in.}$$

Plate  $t_p = \frac{3}{8}$  in.

$$t_{\min} = \frac{3.09(2)D}{F_u} = \frac{(6.19)(4 \text{ sixteenths})}{58 \text{ ksi}} = 0.427 \text{ in.}$$

LRFD	ASD
$\phi R_n = \frac{\frac{3}{8} \text{ in.}}{0.427 \text{ in.}} (50.1 \text{ kips}) = 44 \text{ kips}$	$\frac{R_n}{\Omega} = \frac{\frac{3}{8} \text{ in.}}{0.427 \text{ in.}} (33.4 \text{ kips}) = 29.3 \text{ kips}$
44 kips > 42 kips <b>o.k.</b>	29.3 kips > 28 kips <b>o.k.</b>

A complete-joint penetration groove weld will transfer the entire flange force in tension and compression.

Table J2.5

Note: The column must be checked for stiffening requirements. 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).

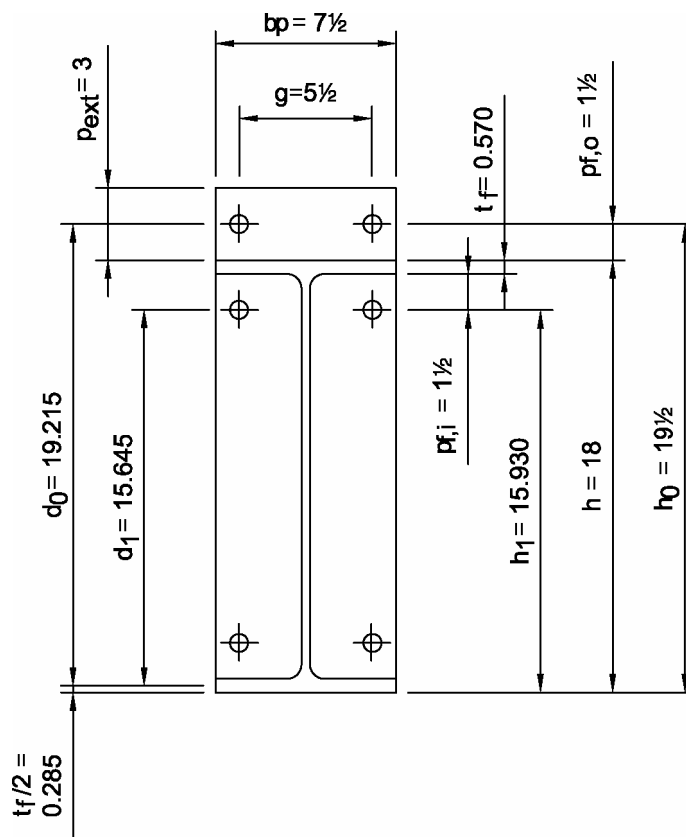
#### Given:

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:

$$\begin{aligned} R_D &= 7 \text{ kips} & M_D &= 42 \text{ kip-ft} \\ R_L &= 21 \text{ kips} & M_L &= 126 \text{ kip-ft} \end{aligned}$$

Use ASTM A325-N snug-tight bolts in standard holes and E70 electrodes.

- 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*.
- 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. <sup>3</sup>	Manual
Column	W14×99	$d = 14.2$ in.	$b_f = 14.6$ in.	$t_f = 0.780$ in.			Table 1-1

**Solution A:**

LRFD	ASD
$R_u = 1.2(7 \text{ kips}) + 1.6(21 \text{ kips}) = 42 \text{ kips}$	$R_a = 7 \text{ kips} + 21 \text{ kips} = 28 \text{ kips}$
$M_u = 1.2(42 \text{ kip-ft}) + 1.6(126 \text{ kip-ft})$ $= 252 \text{ kip-ft}$	$M_a = 42 \text{ kip-ft} + 126 \text{ kip-ft}$ $= 168 \text{ kip-ft}$

*Check beam flexural strength*

LRFD	ASD
For a W18x50	For a W18x50
$\phi_b M_n = 379 \text{ kip-ft} > 252 \text{ kip-ft}$ <b>o.k.</b>	$M_n / \Omega_b = 252 \text{ kip-ft} > 168 \text{ kip-ft}$ <b>o.k.</b>

Manual  
Table 3-3

**Extended end-plate geometric properties:**

$$b_p = 7\frac{1}{2} \text{ in.} \quad g = 5\frac{1}{2} \text{ in.} \quad p_{f,i} = 1\frac{1}{2} \text{ in.} \quad p_{f,o} = 1\frac{1}{2} \text{ in.} \quad p_{ext} = 3 \text{ in.}$$

*Calculate secondary dimensions*

$$h_0 = d + p_{f,o} = 18.0 \text{ in.} + 1\frac{1}{2} \text{ in.} = 19\frac{1}{2} \text{ in.}$$

$$d_o = h_0 - \frac{t_f}{2} = 19\frac{1}{2} \text{ in.} - \frac{0.570 \text{ in.}}{2} = 19.22 \text{ in.}$$

$$h_1 = d - p_{f,i} - t_f = 18.0 \text{ in.} - 1\frac{1}{2} \text{ in.} - 0.570 \text{ in.} = 15.9 \text{ in.}$$

$$d_1 = h_1 - \frac{t_f}{2} = 15.93 \text{ in.} - \frac{0.570 \text{ in.}}{2} = 15.6 \text{ in.}$$

$$\gamma_r = 1.0 \text{ for extended end-plates}$$

*Determine the required bolt diameter assuming no prying action*

For ASTM A325-N bolts,  $F_m = 90 \text{ ksi}$

Table J3.2

LRFD	ASD
$d_{breq} = \sqrt{\frac{2M_u}{\pi\phi F_m (\sum d_n)}}$ $= \sqrt{\frac{2(252 \text{ kip-ft})(12 \text{ in./ft})}{\pi(0.75)(90 \text{ ksi})(19.2 \text{ in.} + 15.6 \text{ in.})}}$ $= 0.905 \text{ in.}$ <p>Use 1-in. diameter ASTM A325-N snug-tightened bolts.</p>	$d_{breq} = \sqrt{\frac{2M_a\Omega}{\pi F_m (\sum d_n)}}$ $= \sqrt{\frac{2(168 \text{ kip-ft})(12 \text{ in./ft})(2.00)}{\pi(90 \text{ ksi})(19.2 \text{ in.} + 15.6 \text{ in.})}}$ $= 0.905 \text{ in.}$ <p>Use 1-in. diameter ASTM A325-N snug-tightened bolts.</p>

*Determine the required end-plate thickness*

$$s = \frac{\sqrt{b_p g}}{2} = \frac{\sqrt{(7\frac{1}{2} \text{ in.})(5\frac{1}{2} \text{ in.})}}{2} = 3.21 \text{ in.}$$

Verify interior bolt pitch,  $p_{f,i} = 1\frac{1}{2} \text{ in.} \leq s = 3.21 \text{ in.}$  **o.k.**

$$\begin{aligned} Y &= \frac{b_p}{2} \left[ h_1 \left( \frac{1}{p_{f,i}} + \frac{1}{s} \right) + h_0 \left( \frac{1}{p_{f,o}} \right) - \frac{1}{2} \right] + \frac{2}{g} [h_1 (p_{f,i} + s)] \\ &= \frac{7\frac{1}{2} \text{ in.}}{2} \left[ (15.93 \text{ in.}) \left( \frac{1}{1\frac{1}{2} \text{ in.}} + \frac{1}{3.21 \text{ in.}} \right) + (19\frac{1}{2} \text{ in.}) \left( \frac{1}{1\frac{1}{2} \text{ in.}} \right) - \frac{1}{2} \right] \\ &\quad + \frac{2}{5\frac{1}{2} \text{ in.}} [(15.93 \text{ in.})(1\frac{1}{2} \text{ in.} + 3.21 \text{ in.})] \\ &= 133 \end{aligned}$$

$$P_t = \frac{\pi d_b^2 F_{nt}}{4} = \frac{\pi (1 \text{ in.})^2 (90 \text{ ksi})}{4} = 70.7 \text{ kips}$$

$$M_n = 2P_t (\sum d_n) = 2(70.7 \text{ kips})(19.2 \text{ in.} + 15.6 \text{ in.}) = 4930 \text{ kip-in.}$$

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi M_n = 0.75(4930 \text{ kip-in.}) = 3700 \text{ kip-in.}$	$M_n / \Omega = \frac{4930 \text{ kip-in.}}{2.00} = 2460 \text{ kip-in.}$
$\phi = 0.90$	$\Omega = 1.67$
$t_{p \text{ req'd}} = \sqrt{\frac{1.11 \gamma_r \phi M_{np}}{\phi_b F_{py} Y}}$ $= \sqrt{\frac{1.11(1.0)(3700 \text{ kip-in.})}{(0.90)(36 \text{ ksi})(133 \text{ in.})}}$ $= 0.977 \text{ in.}$	$t_{p \text{ req'd}} = \sqrt{\frac{1.11 \gamma_r \left( \frac{M_{np}}{\Omega} \right)}{\left( \frac{F_{py}}{\Omega_b} \right) Y}}$ $= \sqrt{\frac{1.11(1.0)(2460 \text{ kip-in.})(1.67)}{(36 \text{ ksi})(133 \text{ in.})}}$ $= 0.978 \text{ in.}$
Use a 1-in. thick end-plate.	Use a 1-in. thick end-plate.

LRFD	ASD
<p><i>Calculate end-plate design strength</i></p> <p>From above, <math>\phi M_n = 3700</math> kip-in.</p> <p><math>\phi = 0.90</math></p> $\frac{\phi_b M_{pl}}{\gamma_r} = \frac{\phi_b F_{py} t_p^2 Y}{\gamma_r}$ $= \frac{0.90(36 \text{ ksi})(1 \text{ in.})^2(133 \text{ in.})}{1.0}$ <p>= 4310 kip-in.</p> $\phi M_n = \min\left(\phi M_{np}, \frac{\phi_b M_{pl}}{\gamma_r}\right)$ <p>= 3700 kip-in. or 308 kip-ft</p> <p>308 kip-ft &gt; 252 kip-ft      <b>o.k.</b></p>	<p><i>Calculate end-plate allowable strength</i></p> <p>From above, <math>M_n / \Omega = 2464</math> kip-in.</p> <p><math>\Omega = 1.67</math></p> $\frac{M_{pl}}{\Omega_b \gamma_r} = \frac{F_{py} t_p^2 Y}{\Omega_b \gamma_r}$ $= \frac{(36 \text{ ksi})(1 \text{ in.})^2(133 \text{ in.})}{1.67(1.0)}$ <p>= 2860 kip-in.</p> $M_n / \Omega_b = \min\left(M_{np} / \Omega_b, \frac{M_{pl}}{\Omega_b \gamma_r}\right)$ <p>= 2460 kip-in. or 205 kip-ft</p> <p>205 kip-ft &gt; 168 kip-ft      <b>o.k.</b></p>

*Check bolt shear*

Try the minimum of four bolts at tension flange and two bolts at compression flange.

LRFD	ASD
$\phi R_n = n\phi r_n = (2 \text{ bolts})(28.3 \text{ kips/bolt})$ <p>= 56.6 kips &gt; 42 kips      <b>o.k.</b></p>	$R_n / \Omega = n r_n / \Omega = (2 \text{ bolts})(18.8 \text{ kips/bolt})$ <p>= 37.6 kips &gt; 28 kips      <b>o.k.</b></p>
<p><i>Determine the required size of the beam web-to-end-plate fillet weld</i></p> $D_{\min} = \frac{\phi F_y t_w}{2(1.39)} = \frac{0.90(50 \text{ ksi})(0.355 \text{ in.})}{2(1.39)}$ <p>= 5.75 sixteenths</p>	<p><i>Determine the required size of the beam web-to-end-plate fillet weld</i></p> $D_{\min} = \frac{F_y t_w}{2\Omega(0.928)} = \frac{(50 \text{ ksi})(0.355 \text{ in.})}{2(1.67)(0.928)}$ <p>= 5.73 sixteenths</p>

Manual  
Table 7-1

Manual  
Part 8

Use  $\frac{3}{8}$  in. fillet welds on both sides of the beam web from the inside face of the beam 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, whichever 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
$D_{\min} = \frac{R_u}{2(1.39)l} = \frac{42 \text{ kips}}{2(1.39)(8.43 \text{ in.})}$ $= 1.79 \rightarrow 3 \text{ sixteenths (minimum size)}$ <p>Use <math>\frac{3}{16}</math>-in. fillet weld on both sides of the beam web below the tension-bolt region.</p>	$D_{\min} = \frac{R_a}{2(0.928)l} = \frac{28 \text{ kips}}{2(0.928)(8.43 \text{ in.})}$ $= 1.79 \rightarrow 3 \text{ sixteenths (minimum size)}$ <p>Use <math>\frac{3}{16}</math>-in. fillet weld on both sides of the beam web below the tension-bolt region.</p>

Table J2.4

*Connecting Elements Rupture Strength at Welds*

*Shear rupture strength of base metal*

$$t_{\min} = \frac{0.6F_{\text{exx}} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{0.6F_u} = \frac{3.09D}{F_u}$$

Beam web  $t_w = 0.355 \text{ in.}$

$$t_{\min} = \frac{3.09(2)D}{F_u} = \frac{(6.19)(3 \text{ sixteenths})}{65 \text{ ksi}} = 0.285 \text{ in.}$$

End plate  $t_p = 1 \text{ in.}$

$$t_{\min} = \frac{3.09D}{F_u} = \frac{(3.09)(3 \text{ sixteenths})}{58 \text{ ksi}} = 0.160 \text{ in.}$$

LRFD	ASD
$R_u = 1.392Dl \left( \frac{0.355 \text{ in.}}{0.285 \text{ in.}} \right)$ $= 1.392(3 \text{ sixteenths})(2)(8.43 \text{ in.}) \left( \frac{0.355 \text{ in.}}{0.285 \text{ in.}} \right)$ $= 87.7 \text{ kips} > 42 \text{ kips}$	$R_a = 0.928Dl \left( \frac{0.355 \text{ in.}}{0.476 \text{ in.}} \right)$ $= 0.928(3 \text{ sixteenths})(2)(8.43 \text{ in.}) \left( \frac{0.355 \text{ in.}}{0.285 \text{ in.}} \right)$ $= 58.5 \text{ kips} > 28 \text{ kips}$

*Determine required fillet weld size for the beam flange to end-plate connection*

$$l = 2(b_f + t_f) - t_w = 2(7.50 \text{ in.} + 0.570 \text{ in.}) - 0.355 \text{ in.} = 15.8 \text{ in.}$$



LRFD	ASD
$P_{uf} = \frac{2M_u}{\sum d_n} = \frac{2(252 \text{ kip-ft})(12 \text{ in./ft})}{19.2 \text{ in.} + 15.6 \text{ in.}}$	$P_{af} = \frac{2M_a}{\sum d_n} = \frac{2(168 \text{ kip-ft})(12 \text{ in./ft})}{19.2 \text{ in.} + 15.6 \text{ in.}}$
$= 173 \text{ kips}$	$= 116 \text{ kips}$
$D_{\min} = \frac{P_{uf}}{1.5(1.39)l} = \frac{173 \text{ kips}}{1.5(1.39)(15.8 \text{ in.})}$	$D_{\min} = \frac{P_{af}}{1.5(0.928)l} = \frac{116 \text{ kips}}{1.5(0.928)(15.8 \text{ in.})}$
$= 5.27 \rightarrow 6 \text{ sixteenths (minimum size)}$	$= 5.27 \rightarrow 6 \text{ sixteenths (minimum size)}$

Note that the 1.5 factor is from Specification J2.4.

Use  $\frac{3}{8}$ -in. fillet welds at beam tension flange. Welds at compression flange may be  $\frac{1}{4}$ -in. fillet welds (minimum size per Specification Table J2.4).

*Connecting Elements Rupture Strength at Welds*

*Tension rupture strength of base metal*

$$R_n = F_u A_e$$

$$t_{\min} = \frac{0.6F_{exx} \left( \frac{\sqrt{2}}{2} \right) \left( \frac{D}{16} \right)}{F_u} = \frac{1.86D}{F_u}$$

Section J4.1

Beam flange;  $t_f = 0.570$  in.

$$t_{\min} = \frac{1.86(2)D}{F_u} = \frac{(3.71)(6 \text{ sixteenths})}{65 \text{ ksi}} = 0.343 \text{ in.}$$

End plate;  $t_p = 1.00$  in.

$$t_{\min} = \frac{1.86D}{F_u} = \frac{(1.86)(6 \text{ sixteenths})}{58 \text{ ksi}} = 0.192 \text{ in.}$$

**Solution B:**

Only those portions of the design that vary from the solution “A” calculations are presented here.

LRFD	ASD
<i>Determine required end-plate thickness</i>	<i>Determine required end-plate thickness</i>
$\phi = 0.90$	$\Omega = 1.67$
$t_{preq} = \sqrt{\frac{\gamma_r M_u}{\phi_b F_y Y}}$	$t_{preq} = \sqrt{\frac{\gamma_r M_a \Omega_b}{F_y Y}}$
$= \sqrt{\frac{1.0(252 \text{ kip-ft})(12 \text{ in./ft})}{0.90(36 \text{ ksi})(133 \text{ in.})}}$	$= \sqrt{\frac{1.0(168 \text{ kip-ft})(12 \text{ in./ft})(1.67)}{(36 \text{ ksi})(133 \text{ in.})}}$
$= 0.84 \text{ in.}$	$= 0.84 \text{ in.}$
Use $t_p = 7/8 \text{ in.}$	Use $t_p = 7/8 \text{ in.}$

Select a trial bolt diameter and calculate the maximum prying forces

Try 1-in. diameter bolts.

$$w' = \frac{b_p}{2} - (d_b + 1/16 \text{ in.}) = \frac{7.50 \text{ in.}}{2} - (1 1/16 \text{ in.}) = 2.69 \text{ in.}$$

$$a_i = 3.682 \left( \frac{t_p}{d_b} \right)^3 - 0.085 = 3.682 \left( \frac{7/8 \text{ in.}}{1 \text{ in.}} \right)^3 - 0.085 = 2.38$$

$$F_i' = \frac{t_p^2 F_{yp} \left[ 0.85 \left( \frac{b_p}{2} \right) + 0.80 w' \right] + \frac{\pi d_b^3 F_{mi}}{8}}{4 p_{f,i}}$$

$$= \frac{(7/8)^2 (36) \left[ 0.85 \left( \frac{7 1/2}{2} \right) + 0.80 (2.69) \right]}{4 (1 1/2)}$$

$$+ \frac{\pi (1)^3 (90)}{8 (1 1/2)}$$

$$= 30.4 \text{ kips}$$

$$Q_{\max i} = \frac{w' t_p^2}{4 a_i} \sqrt{F_{yp}^2 - 3 \left( \frac{F_i'}{w' t_p} \right)^2}$$

$$= \frac{(2.69) \left( \frac{7}{8} \right)^2}{4 (2.38)} \sqrt{36^2 - 3 \left( \frac{30.4}{2.69 \left( \frac{7}{8} \right)} \right)^2}$$

$$= 6.10 \text{ kips}$$

$$a_o = \min [a_i, p_{ext} - p_{f,o}] = \min [2.38 \text{ in.}, 1 \frac{1}{2} \text{ in.}] = 1 \frac{1}{2} \text{ in.}$$

$$F_o' = F_i' \left( \frac{p_{fi}}{p_{fo}} \right) = (30.4 \text{ kips}) \left( \frac{1 \frac{1}{2} \text{ in.}}{1 \frac{1}{2} \text{ in.}} \right)$$

$$= 30.4 \text{ kips}$$

$$Q_{\max o} = \frac{w' t_p^2}{4 a_o} \sqrt{F_{yp}^2 - 3 \left( \frac{F_o'}{w' t_p} \right)^2}$$

$$= \frac{(2.69) \left( \frac{7}{8} \right)^2}{4 (1 \frac{1}{2})} \sqrt{36^2 - 3 \left( \frac{30.4}{2.69 \left( \frac{7}{8} \right)} \right)^2}$$

$$= 9.68 \text{ kips}$$

Calculate the connection available strength for the limit state of bolt rupture with prying action

$$P_t = \frac{\pi d_b^2 F_m}{4} = \frac{\pi (1 \text{ in.})^2 (90 \text{ ksi})}{4} = 70.7 \text{ kips}$$

Unmodified Bolt Pretension,  $T_{b0} = 51 \text{ kips}$

Modify bolt pretension for the snug-tight condition.

$$T_b = \frac{T_{b0}}{4} = \frac{51 \text{ kips}}{4} = 12.8 \text{ kips}$$

Table J3.1

AISC Design  
Guide 16  
Table 2-1,  
Table 4-2

LRFD	ASD
$\phi M_q =$ $\max \left\{ \begin{array}{l} \phi [2(P_t - Q_{\max o})d_0 + 2(P_t - Q_{\max i})d_1] \\ \phi [2(P_t - Q_{\max o})d_0 + 2(T_b)d_1] \\ \phi [2(P_t - Q_{\max i})d_1 + 2(T_b)d_0] \\ \phi [2(T_b)(d_0 + d_1)] \end{array} \right\}$ $= \max \left\{ \begin{array}{l} 0.75 \left[ \begin{array}{l} 2(70.7 - 9.68)(19.2) \\ +2(70.7 - 6.10)(15.6) \end{array} \right] \\ 0.75 \left[ \begin{array}{l} 2(70.7 - 9.68)(19.2) \\ +2(12.8)(15.6) \end{array} \right] \\ 0.75 \left[ \begin{array}{l} 2(70.7 - 6.10)(15.6) \\ +2(12.8)(19.2) \end{array} \right] \\ 0.75 [2(12.8)(19.215 + 15.6)] \end{array} \right\}$ $= \max \left\{ \begin{array}{l} 3275 \text{ kip-in.} \\ 2059 \text{ kip-in.} \\ 1885 \text{ kip-in.} \\ 669 \text{ kip-in.} \end{array} \right\} = 3275 \text{ kip-in.}$ $\phi M_q = 3275 \text{ kip-in.}$ $= 273 \text{ kip-ft} > 252 \text{ kip-ft} \quad \mathbf{o.k.}$	$\frac{M_q}{\Omega} =$ $\max \left\{ \begin{array}{l} \frac{1}{\Omega} [(P_t - Q_{\max o})d_0 + 2(P_t - Q_{\max i})d_1] \\ \frac{1}{\Omega} [2(P_t - Q_{\max o})d_0 + 2(T_b)d_1] \\ \frac{1}{\Omega} [2(P_t - Q_{\max i})d_1 + 2(T_b)d_0] \\ \frac{1}{\Omega} [2(T_b)(d_0 + d_1)] \end{array} \right\}$ $= \max \left\{ \begin{array}{l} \frac{1}{2.00} \left[ \begin{array}{l} 2(70.7 - 9.68)(19.2) \\ +2(70.7 - 6.10)(15.6) \end{array} \right] \\ \frac{1}{2.00} \left[ \begin{array}{l} 2(70.7 - 9.68)(19.2) \\ +2(12.8)(15.6) \end{array} \right] \\ \frac{1}{2.00} \left[ \begin{array}{l} 2(70.7 - 6.10)(15.6) \\ +2(12.8)(19.2) \end{array} \right] \\ \frac{1}{2.00} [2(12.8)(19.215 + 15.6)] \end{array} \right\}$ $= \max \left\{ \begin{array}{l} 2183 \text{ kip-in.} \\ 1373 \text{ kip-in.} \\ 1257 \text{ kip-in.} \\ 446 \text{ kip-in.} \end{array} \right\} = 2138 \text{ kip-in.}$ $M_q / \Omega = \frac{3275}{1.5} = 2183 \text{ kip-in.}$ $= 182 \text{ kip-ft} > 168 \text{ kip-ft} \quad \mathbf{o.k.}$

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 7/8-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:

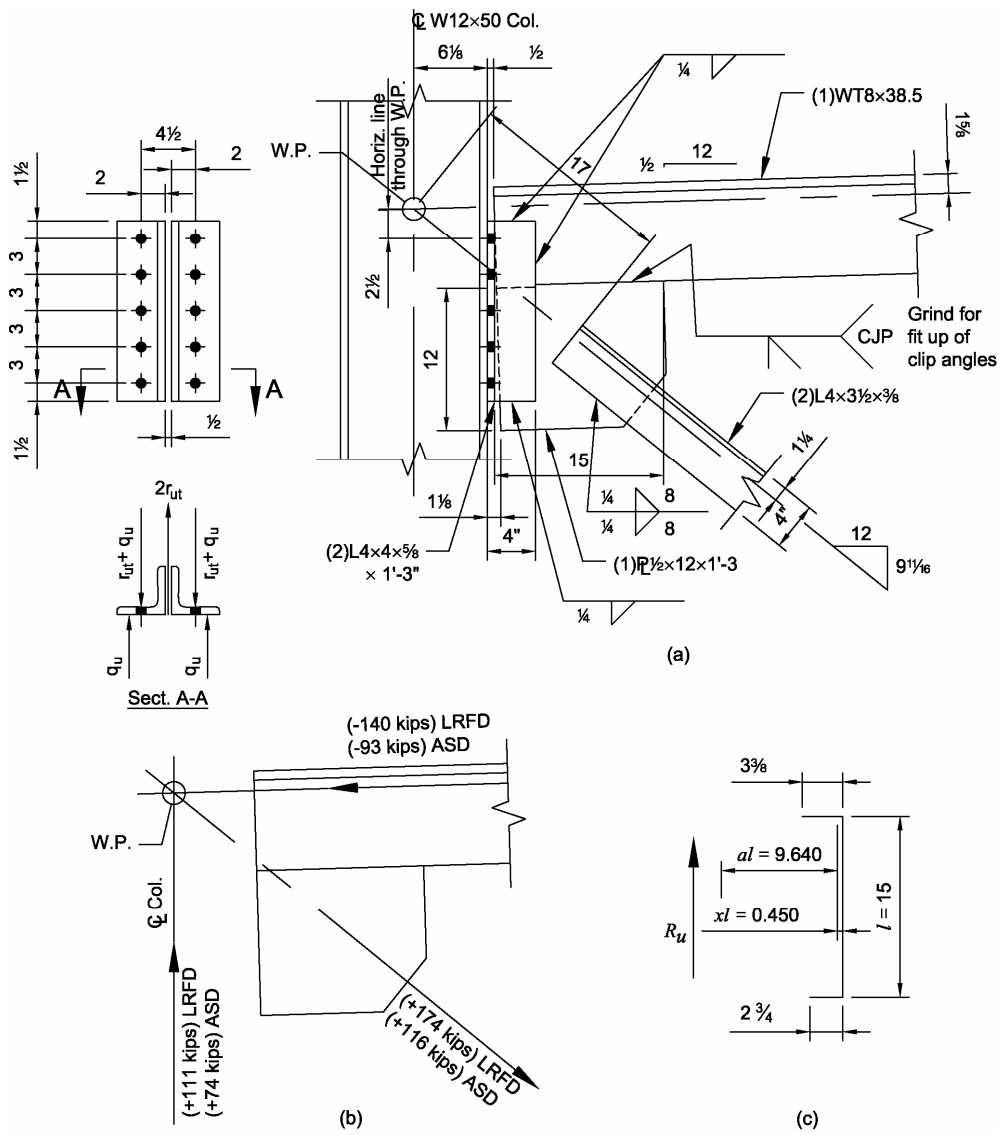
Determine:

- the connection requirements between gusset and column,
- the required gusset size and the weld requirements for member  $U_0L_1$  at the gusset.

$R_D = 18.5$  kips

$R_L = 55.5$  kips

Use  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts in standard holes and E70 electrodes.



**Material Properties:**

Top Chord	WT8×38.5	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Column	W12×50	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Brace	2L4×3½×¾	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

Manual  
Tables 2-3  
and 2-4**Geometric Properties:**

Top Chord	WT8×38.5	$t_w = 0.455$ in.
Brace	2L4×3½×¾	$t = 0.375$ in.

Manual  
Table 1-8**Solution:**

LRFD	ASD
Brace axial load	Brace axial load
$R_u = 174$ kips	$R_a = 116$ kips
Truss end reaction	Truss end reaction
$R_u = 1.2(18.5) + 1.6(55.5) = 111$ kips	$R_a = 18.5 + 55.5 = 74$ kips
Top chord axial load	Top chord axial load
$R_u = 140$ kips	$R_a = 93$ 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 ¾-in. angles,  $D_{\min} = 3$ , try ¼-in. fillet welds,  $D = 4$ .

Table J2.4

LRFD	ASD
$L_{req} = \frac{R_u}{4D(1.392)} = \frac{174 \text{ kips}}{4(4)(1.392)} = 7.81$ in.	$L_{req} = \frac{R_a}{4D(1.392)} = \frac{116 \text{ kips}}{4(4)(0.928)} = 7.81$ in.
Use 8 in. at the heel and 8 in. at the toe of each angle.	Use 8 in. at the heel and 8 in. at the toe of each angle.

*Determine minimum gusset thickness based on weld size*

Note: This check is required only when the stress in the plate is unknown. Since, in this case, the stress is known and gusset block shear rupture will be checked, this is not required. It is included here to illustrate the method.

For two ¼-in. fillet welds,

$$t_{\min} = \frac{6.19D}{F_u} = \frac{6.19(4)}{(58 \text{ ksi})} = 0.427 \text{ in.}$$

Use a 1/2 in. gusset plate. With the diagonal to gusset welds determined, a gusset plate layout as shown in Figure (c) can be made.

LRFD	ASD
<p><i>Design bolts connecting clip angles to column (shear and tension)</i></p> <p>The number of 7/8-in. diameter ASTM A325-N bolts required for shear only is as follows:</p> $n_{\min} = \frac{R_u}{\phi r_n} = \frac{111 \text{ kips}}{21.6 \text{ kips/bolt}} = 5.14$	<p><i>Design bolts connecting clip angles to column (shear and tension)</i></p> <p>The number of 7/8-in. diameter ASTM A325-N bolts required for shear only is as follows:</p> $n_{\min} = \frac{R_u}{\phi r_n} = \frac{74 \text{ kips}}{14.4 \text{ kips/bolt}} = 5.14$

Manual  
Table 7-1

Try a clip angle thickness of 5/8 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
<p>With 10 bolts,</p> $f_v = \frac{R_u}{nA_b} = \frac{111 \text{ kips}}{(10 \text{ bolts})(0.601 \text{ in.}^2)} = 18.5 \text{ ksi}$	<p>With 10 bolts,</p> $f_v = \frac{R_u}{nA_b} = \frac{74 \text{ kips}}{(10 \text{ bolts})(0.601 \text{ in.}^2)} = 12.3 \text{ ksi}$

LRFD	ASD
<p>The eccentric moment at the faying surface is as follows:</p> $M_u = R_u e = (111 \text{ kips})(6.1 \text{ in.}) = 677 \text{ kip-in.}$	<p>The eccentric moment at the faying surface is as follows:</p> $M_a = R_a e = (74 \text{ kips})(6.1 \text{ in.}) = 451 \text{ kip-in.}$

For the bolt group, the Case II approach of Manual Part 7 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 in.

LRFD	ASD
$T_u = \frac{M_u}{n' d_m} = \frac{677 \text{ kip-in.}}{(4 \text{ bolts})(9 \text{ in.})} = 18.8 \text{ kips/bolt}$	$T_a = \frac{M_a}{n' d_m} = \frac{451 \text{ kip-in.}}{(4 \text{ bolts})(9 \text{ in.})} = 12.5 \text{ kips/bolt}$

LRFD	ASD
<p><i>Check design tensile strength of bolts</i></p> $F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \leq F_{nt}$ $= 1.3(90 \text{ ksi}) - \frac{90 \text{ ksi}}{0.75(48 \text{ ksi})}(18.5 \text{ ksi})$ $= 70.8 \text{ ksi} < 90 \text{ ksi} \quad \mathbf{o.k.}$	<p><i>Check design tensile strength of bolts</i></p> $F_{nt}' = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \leq F_{nt}$ $= 1.3(90 \text{ ksi}) - \frac{(2.00)(90 \text{ ksi})}{48 \text{ ksi}}(12.3 \text{ ksi})$ $= 70.1 \text{ ksi} < 90 \text{ ksi} \quad \mathbf{o.k.}$

Eqn. J3-3a  
(LRFD) and  
J3-3b (ASD)

Table J3.2



$B = \phi F_{nt}' A_b = 0.75(70.8 \text{ ksi})(0.601 \text{ in.}^2)$ $= 31.9 \text{ kips} > 18.8 \text{ kips} \quad \mathbf{o.k.}$	$B = \frac{F_{nt}'}{\Omega} A_b = \frac{70.1 \text{ ksi}}{2.00}(0.601 \text{ in.}^2)$ $= 21.1 \text{ kips} > 12.5 \text{ kips} \quad \mathbf{o.k.}$
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Check the clip angles

Check prying action.

$$p = 3 \text{ in.}$$

$$b = 2 \text{ in.} - \frac{5/8 \text{ in.}}{2} = 1.69 \text{ in.}$$

Note: 1¼ in. entering and tightening clearance accommodated, **o.k.**

$$a = 4 \text{ in.} - 2 \text{ in.} \leq 1.25b \text{ (for calculation purposes)}$$

$$= 2 \text{ in.} < 1.25(1.69 \text{ in.}) = 2.11 \text{ in.} \quad \mathbf{o.k.}$$

$$b' = b - \frac{d}{2} = 1.69 \text{ in.} - \frac{(7/8 \text{ in.})}{2} = 1.25 \text{ in.}$$

$$a' = a + \frac{d}{2} = 2 \text{ in.} + \frac{(7/8 \text{ in.})}{2} = 2.44 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.25 \text{ in.}}{2.44 \text{ in.}} = 0.512$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{15/16 \text{ in.}}{3 \text{ in.}} = 0.688$$

LRFD	ASD
$t_c = \sqrt{\frac{4.44Bb'}{pF_u}} = \sqrt{\frac{4.44(31.9 \text{ kips})(1.25 \text{ in.})}{(3 \text{ in.})(58 \text{ ksi})}}$ $= 1.01 \text{ in.}$	$t_c = \sqrt{\frac{6.66Bb'}{pF_u}} = \sqrt{\frac{6.66(21.1 \text{ kips})(1.25 \text{ in.})}{(3 \text{ in.})(58 \text{ ksi})}}$ $= 1.01 \text{ in.}$

$$\alpha' = \frac{1}{\delta(1+\rho)} \left[ \left( \frac{t_c}{t} \right)^2 - 1 \right]$$

$$= \frac{1}{0.688(1+0.512)} \left[ \left( \frac{1.01 \text{ in.}}{0.625 \text{ in.}} \right)^2 - 1 \right]$$

$$= 1.55$$

Since  $\alpha' > 1$ , use  $\alpha' = 1.0$

LRFD	ASD
$T_a = B \left( \frac{t}{t_c} \right)^2 (1 + \delta \alpha')$ $= (31.9 \text{ kips}) \left( \frac{0.625 \text{ in.}}{1.01 \text{ in.}} \right)^2 [1 + 0.688(1)]$ $= 20.6 \text{ kips} > 18.8 \text{ kips} \quad \mathbf{o.k.}$	$T_a = B \left( \frac{t}{t_c} \right)^2 (1 + \delta \alpha')$ $= (21.1 \text{ kips}) \left( \frac{0.625 \text{ in.}}{1.01 \text{ in.}} \right)^2 [1 + 0.688(1)]$ $= 13.6 \text{ kips} > 12.5 \text{ kips} \quad \mathbf{o.k.}$
<p><i>Check shear yielding of the clip angles</i></p> $\phi R_n = \phi (0.6 F_y) A_g$ $= 1.0(0.6)(36 \text{ ksi}) [2(15.0 \text{ in.}) (\frac{5}{8} \text{ in.})]$ $= 405 \text{ kips} > 111 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear yielding of the clip angles</i></p> $R_n / \Omega = \frac{(0.6 F_y) A_g}{\Omega}$ $= \frac{(0.6)(36 \text{ ksi}) [2(15.0 \text{ in.}) (\frac{5}{8} \text{ in.})]}{1.50}$ $= 270 \text{ kips} > 74 \text{ kips} \quad \mathbf{o.k.}$
<p><i>Check shear rupture of the angles</i></p> $A_n = 2 [15.0 \text{ in.} - 5(1 \text{ in.})] (\frac{5}{8} \text{ in.}) = 12.5 \text{ in.}^2$ $\phi R_n = \phi (0.6 F_u) A_n$ $= 0.75(0.6)(58 \text{ ksi})(12.5 \text{ in.}^2)$ $= 326 \text{ kips} > 111 \text{ kips} \quad \mathbf{o.k.}$	<p><i>Check shear rupture of the angles</i></p> $A_n = 2 [15.0 \text{ in.} - 5(1 \text{ in.})] (\frac{5}{8} \text{ in.}) = 12.5 \text{ in.}^2$ $R_n / \Omega = \frac{(0.6 F_u) A_n}{\Omega}$ $= \frac{(0.6)(58 \text{ ksi})(12.5 \text{ in.}^2)}{2.00}$ $= 218 \text{ kips} > 74 \text{ kips} \quad \mathbf{o.k.}$

*Check block shear rupture of the clip angles*

Assume uniform tension stress, use  $U_{bs} = 1.0$ .

Gross area subject to shear,  $A_{gv} = 2(15.0 \text{ in.} - 1\frac{1}{2} \text{ in.}) (\frac{5}{8} \text{ in.}) = 16.9 \text{ in.}^2$

Net area subject to shear,  $A_{nv} = 16.9 \text{ in.}^2 - 2[4\frac{1}{2}(1 \text{ in.}) (\frac{5}{8} \text{ in.})] = 11.3 \text{ in.}^2$

Net area subject to tension,  $A_{nt} = 2[(2 \text{ in.}) (\frac{5}{8} \text{ in.}) - 0.5(1 \text{ in.}) (\frac{5}{8} \text{ in.})] = 1.88 \text{ in.}^2$

LRFD	ASD
$\phi R_n = \phi \left[ F_u A_{nt} + \min \left\{ 0.6 F_y A_{gv}, 0.6 F_u A_{nv} \right\} \right]$ $= 0.75 \left[ \begin{array}{l} (58 \text{ ksi})(1.88 \text{ in.}^2) + \\ \min \left\{ \begin{array}{l} 0.6(36 \text{ ksi})(16.9 \text{ in.}^2), \\ 0.6(58 \text{ ksi})(11.3 \text{ in.}^2) \end{array} \right\} \end{array} \right]$ $= 356 \text{ kips} > 111 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{\left[ F_u A_{nt} + \min \left\{ 0.6 F_y A_{gv}, 0.6 F_u A_{nv} \right\} \right]}{\Omega}$ $= \frac{\left[ \begin{array}{l} (58 \text{ ksi})(1.88 \text{ in.}^2) + \\ \min \left\{ \begin{array}{l} 0.6(36 \text{ ksi})(16.9 \text{ in.}^2), \\ 0.6(58 \text{ ksi})(11.3 \text{ in.}^2) \end{array} \right\} \end{array} \right]}{2.00}$ $= 237 \text{ kips} > 74 \text{ kips} \quad \mathbf{o.k.}$

Eqn. J4-5

Use 2L4×4×5/8.

Check bearing and tearout

The clear edge distance,  $L_c$ , for the top bolts is  $L_c = L_e - \frac{d'}{2}$ , where  $L_e$  is the distance to the center of the hole. Thus,

$$L_c = 1 \frac{1}{2} \text{ in.} - \frac{1}{2} \left( \frac{15}{16} \text{ in.} \right) = 1.03 \text{ in.}$$

The bearing/tearout capacity of the top bolt is as follows:

LRFD	ASD
$\phi r_n = \phi 1.2 L_c t F_u < 2.4 d t F_u$ $= 0.75(1.2)(1.03 \text{ in.}) \left( \frac{5}{8} \text{ in.} \right) (58 \text{ ksi})$ $\leq 0.75(2.4) \left( \frac{7}{8} \text{ in.} \right) \left( \frac{5}{8} \text{ in.} \right) (58 \text{ ksi})$ $= 33.6 \text{ kips} < 57.1 \text{ kips}$ $= 33.6 \text{ kips/bolt}$	$r_n / \Omega = \frac{1.2 L_c t F_u}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(1.03 \text{ in.}) \left( \frac{5}{8} \text{ in.} \right) (58 \text{ ksi})}{2.00}$ $\leq \frac{2.4 \left( \frac{7}{8} \text{ in.} \right) \left( \frac{5}{8} \text{ in.} \right) (58 \text{ ksi})}{2.00}$ $= 22.4 \text{ kips} < 38.1 \text{ kips}$ $= 22.4 \text{ kips/bolt}$

The bearing/ tearout capacity of each of the remaining bolts is  $L_c = 3 \text{ in.} - 1\left(\frac{15}{16} \text{ in.}\right) = 2.06 \text{ in.}$

LRFD	ASD
$\phi r_n = \phi 1.2 L_c t F_u < 2.4 d t F_u$ $= 0.75(1.2)(2.06 \text{ in.})\left(\frac{5}{8} \text{ in.}\right)(58 \text{ ksi})$ $\leq 0.75(2.4)\left(\frac{7}{8} \text{ in.}\right)\left(\frac{5}{8} \text{ in.}\right)(58 \text{ ksi})$ $= 67.2 \text{ kips} < 57.1 \text{ kips}$ $= 57.1 \text{ kips/bolt}$	$r_n / \Omega = \frac{1.2 L_c t F_u}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(2.06 \text{ in.})\left(\frac{5}{8} \text{ in.}\right)(58 \text{ ksi})}{2.00}$ $\leq \frac{2.4\left(\frac{7}{8} \text{ in.}\right)\left(\frac{5}{8} \text{ in.}\right)(58 \text{ ksi})}{2.00}$ $= 44.8 \text{ kips} < 38.0 \text{ kips}$ $= 38.1 \text{ kips/bolt}$
<p>Therefore the bearing/ tearout capacity of the bolt group is</p> $\phi R_n = (33.6 \text{ kips})(2 \text{ bolts}) +$ $(57.1 \text{ kips})(8 \text{ bolts})$ $= 524 \text{ kips} > 111 \text{ kips} \quad \mathbf{o.k.}$	<p>Therefore the bearing/ tearout capacity of the bolt group is</p> $R_n / \Omega = (22.4 \text{ kips})(2 \text{ bolts}) +$ $(38.1 \text{ kips})(8 \text{ bolts})$ $= 350 \text{ kips} > 74 \text{ kips} \quad \mathbf{o.k.}$

*Design clip angle-to-gusset connection*

The minimum weld size is  $\frac{3}{16}$  in. with top chord slope being  $\frac{1}{2}$  on 12, the horizontal welds are as shown due to the square cut end. Use the average length. Then,

$$l = 15.0 \text{ in.}$$

$$kl = \frac{3\frac{3}{8} \text{ in.} + 2\frac{3}{4} \text{ in.}}{2} = 3.06 \text{ in.}$$

$$k = \frac{kl}{l} = \frac{3.06 \text{ in.}}{15.0 \text{ in.}} = 0.204$$

With  $\theta = 0^\circ$ , by interpolation  $x = 0.030$  and

$$al + xl = 10.1 \text{ in.}$$

$$a = \frac{[10.1 \text{ in.} - 0.030(15.0 \text{ in.})]}{15.0 \text{ in.}} = 0.643$$

By interpolation,  $C = 1.50$

Table J2.4

Manual  
Table 8-9

LRFD	ASD
$D_{req} = \frac{R_u}{2(\phi CC_1 l)}$ $= \frac{111 \text{ kips}}{2(0.75)(1.50)(1.0)(15.0 \text{ in.})}$ $= 3.29 \rightarrow 4 \text{ sixteenths}$	$D_{req} = \frac{\Omega R_u}{2(CC_1 l)}$ $= \frac{(2.0)(74 \text{ kips})}{2(1.50)(1.0)(15.0 \text{ in.})}$ $= 3.29 \rightarrow 4 \text{ sixteenths}$
Use 1/4-in. fillet welds.	Use 1/4-in. fillet welds.

*Design chord extension plate*

The chord stem extension plate thickness should match or slightly exceed that of the tee stem; use 1/2-in. plate.

*Check tension yielding on the Whitmore section*

$$L_w = 4 \text{ in.} + 2(8 \text{ in.}) \tan 30^\circ = 13.2 \text{ in.}$$

LRFD	ASD
$\phi R_n = \phi F_y A_g = 0.9(36 \text{ ksi})(13.2 \text{ in.})(\frac{1}{2} \text{ in.})$ $= 214 \text{ kips} > 174 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{F_y A_g}{\Omega} = \frac{(36 \text{ ksi})(13.2 \text{ in.})(\frac{1}{2} \text{ in.})}{1.67}$ $= 142 \text{ kips} > 116 \text{ kips} \quad \mathbf{o.k.}$

*Check block shear rupture of the clip angles*

Assume uniform tension stress, use  $U_{bs} = 1.0$

$$\text{Gross area subject to shear, } A_{gv} = 2(8.0 \text{ in.})(\frac{1}{2} \text{ in.}) = 8.0 \text{ in.}^2$$

$$\text{Net area subject to shear, } A_{nv} = A_{gv} = 8.0 \text{ in.}^2$$

$$\text{Net area subject to tension, } A_{nt} = (4 \text{ in.})(\frac{1}{2} \text{ in.}) = 2.0 \text{ in.}^2$$

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$ $= 0.75(58 \text{ ksi})(2.0 \text{ in.}^2)$ $+ \min \left\{ \begin{array}{l} 0.75(0.6)(36 \text{ ksi})(8.0 \text{ in.}^2), \\ 0.75(0.6)(58 \text{ ksi})(8.0 \text{ in.}^2) \end{array} \right\}$ $= 217 \text{ kips} > 174 \text{ kips} \quad \mathbf{o.k.}$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min \left( \frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega} \right)$ $= \frac{(58 \text{ ksi})(2.0 \text{ in.}^2)}{2.00}$ $+ \min \left\{ \begin{array}{l} \frac{0.6(36 \text{ ksi})(8.0 \text{ in.}^2)}{2.00}, \\ \frac{0.6(58 \text{ ksi})(8.0 \text{ in.}^2)}{2.00} \end{array} \right\}$ $= 144 \text{ kips} > 116 \text{ kips} \quad \mathbf{o.k.}$

Section J4.3

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
$L_{req} = \frac{R_u}{\phi(0.6F_u)t}$ $= \frac{140 \text{ kips}}{0.75(0.6)(58 \text{ ksi})(0.455 \text{ in.})} = 11.8 \text{ in.}$	$L_{req} = \frac{\Omega R_a}{(0.6F_u)t}$ $= \frac{(2.00)(93 \text{ kips})}{(0.6)(58 \text{ ksi})(0.455 \text{ in.})} = 11.8 \text{ in.}$
Use $L = 15 \text{ in.}$	Use $L = 15 \text{ in.}$

The gusset depth depends upon the connection angles. From a scaled layout, the gusset must extend 1'-0" below the tee stem.

Use PL $\frac{1}{2}$ ×12×1'-3.

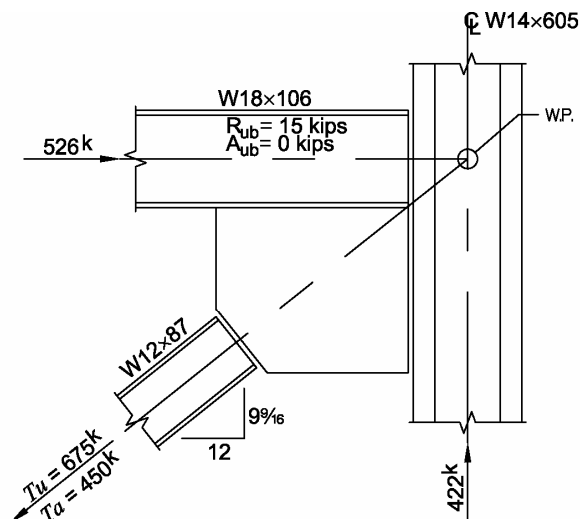
## Example II.C-2 Bracing Connection

### Given:

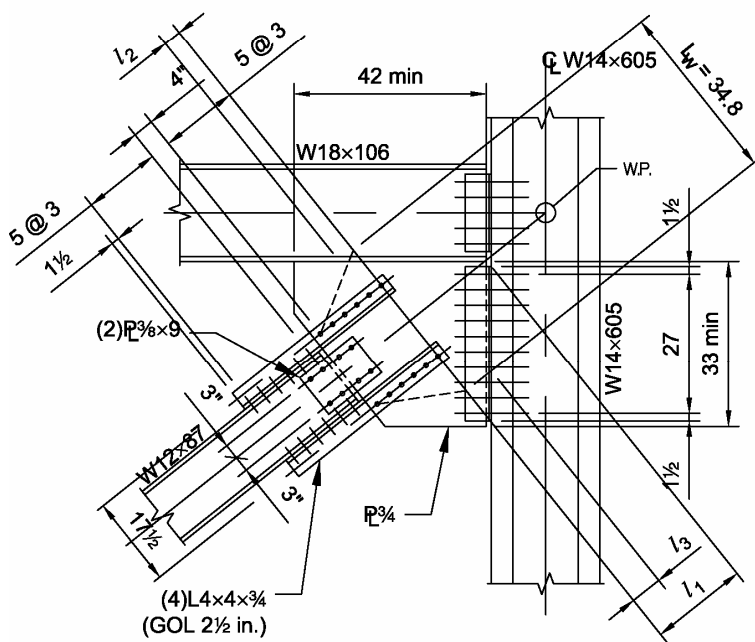
Design the diagonal bracing connection between the W12×87 brace and the W18×106 beam and the W14×605 column.

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	$P_u = 422$ kips	$P_a = 281$ kips

Use  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts in standard holes and E70 electrodes.



(a) members and forces



(b) connection

**Material Properties:**

Brace	W12x87	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual Tables 2-3 and 2-4
Beam	W18x106	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	
Column	W14x605	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	W12x87	$A=25.6$ in. <sup>2</sup>	$d=12.5$ in.	$t_w=0.515$ in.	$b_f=12.1$ in.	$t_f=0.810$ in.	Manual Table 1-1
Beam	W18x106	$d=18.7$ in.	$t_w=0.590$ in.	$b_f=11.2$ in.	$t_f=0.940$ in.	$k=1.34$	
Column	W14x605	$d=20.9$ 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 $P_{uf} = \frac{P_u b_f t_f}{A} = \frac{(675 \text{ kips})(12.1 \text{ in.})(0.810 \text{ in.})}{25.6 \text{ in.}^2}$ $= 259 \text{ kips}$	Force in flange $P_{af} = \frac{P_a b_f t_f}{A} = \frac{(450 \text{ kips})(12.1 \text{ in.})(0.810 \text{ in.})}{25.6 \text{ in.}^2}$ $= 172 \text{ kips}$
Force in web $P_{uw} = P_u - 2P_{uf} = 675 \text{ kips} - 2(259 \text{ kips})$ $= 157 \text{ kips}$	Force in web $P_{aw} = P_a - 2P_{af} = 450 \text{ kips} - 2(172 \text{ kips})$ $= 106 \text{ kips}$

*Design brace-flange-to-gusset connection.*

Determine number of 7/8-in. diameter ASTM A325-N bolts required on the brace side for single shear.

LRFD	ASD
$n_{\min} = \frac{P_{uf}}{\phi r_n} = \frac{259 \text{ kips}}{21.6 \text{ kips/bolt}} = 11.9 \rightarrow 12 \text{ bolts}$	$n_{\min} = \frac{P_{af}}{r_n / \Omega} = \frac{172 \text{ kips}}{14.4 \text{ kips/bolt}} = 11.9 \rightarrow 12 \text{ bolts}$

On the gusset side, since these 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 2L4x4x3/4 angles ( $A = 10.9 \text{ in.}^2$ ,  $\bar{x} = 1.27 \text{ in.}$ ).

LRFD	ASD
Check tension yielding of the angles $\phi R_n = \phi F_y A_g = 0.90(36 \text{ ksi})(10.9 \text{ in.}^2)$ $= 353 \text{ kips} > 259 \text{ kips} \quad \mathbf{o.k.}$	Check tension yielding of the angles $R_n / \Omega = \frac{F_y A_g}{1.67} = \frac{(36 \text{ ksi})(10.9 \text{ in.}^2)}{1.67}$ $= 235 \text{ kips} > 172 \text{ kips} \quad \mathbf{o.k.}$

*Check tension rupture of the angles*

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.27 \text{ in.}}{15 \text{ in.}} = 0.92$$

$$A_e = U A_n = 0.92 [10.9 \text{ in.}^2 - 2(\frac{3}{4} \text{ in.})(1 \text{ in.})] = 8.65 \text{ in.}^2$$

Table D3.1

LRFD	ASD
$\phi R_n = \phi F_u A_e = 0.75(58 \text{ ksi})(8.65 \text{ in.}^2)$ $= 376 \text{ kips} > 259 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{F_u A_e}{\Omega} = \frac{(58 \text{ ksi})(8.65 \text{ in.}^2)}{2.00}$ $= 251 \text{ kips} > 172 \text{ kips} \quad \mathbf{o.k.}$

Check block shear rupture of the angles.

Use  $n = 6$ ,  $L_{ev} = 1\frac{1}{2}$  in., and  $L_{eh} = 1\frac{1}{2}$  in.

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$	
Tension Rupture Component	Tension Rupture Component	Manual Table 9-3a
$\phi F_u A_{nt} = (43.5 \text{ kips/in.})(\frac{3}{4} \text{ in.})(2)$	$F_u A_{nt} / \Omega = (29.0 \text{ kips/in.})(\frac{3}{4} \text{ in.})(2)$	
Shear Yielding Component	Shear Yielding Component	Manual Table 9-3b
$\phi 0.6 F_y A_{gv} = (267 \text{ kips/in.})(\frac{3}{4} \text{ in.})(2)$	$0.6 F_y A_{gv} / \Omega = (178 \text{ kips/in.})(\frac{3}{4} \text{ in.})(2)$	
Shear Rupture Component	Shear Rupture Component	Manual Table 9-3c
$\phi 0.6 F_u A_{nv} = (287 \text{ kips/in.})(\frac{3}{4} \text{ in.})(2)$	$0.6 F_u A_{nv} / \Omega = (191 \text{ kips/in.})(\frac{3}{4} \text{ in.})(2)$	
$\phi R_n = (43.5 \text{ kips} + 267 \text{ kips})(\frac{3}{4} \text{ in.})(2)$ $= 465 \text{ kips} > 259 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = (29.0 \text{ kips} + 178 \text{ kips})(\frac{3}{4} \text{ in.})(2)$ $= 310 \text{ kips} > 172 \text{ kips}$ <b>o.k.</b>	Eqn. J4-5

Similarly, the block shear rupture strength of the brace flange is o.k.

*Design brace-web-to-gusset connection*

Determine number of  $\frac{7}{8}$ -in. diameter ASTM A325-N bolts required on the brace side (double shear) for shear.

LRFD	ASD	
$n_{\min} = \frac{P_{uw}}{\phi r_n} = \frac{157 \text{ kips}}{43.3 \text{ kips/bolt}} = 3.63 \rightarrow 4 \text{ bolts}$	$n_{\min} = \frac{P_{aw}}{r_n / \Omega} = \frac{106 \text{ kips}}{28.9 \text{ kips/bolt}} = 3.67 \rightarrow 4 \text{ bolts}$	Manual Table 7-1

On the gusset side, the same number of bolts are required. Try two rows of two bolts and two PL $\frac{3}{8}$ ×9.

LRFD	ASD	
<i>Check tension yielding of the plates</i>	<i>Check tension yielding of the plates</i>	
$\phi R_n = \phi F_y A_g = 0.90(36 \text{ ksi})(2)(\frac{3}{8} \text{ in.})(9 \text{ in.})$ $= 219 \text{ kips} > 157 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = F_y A_g / \Omega = \frac{(36 \text{ ksi})(2)(\frac{3}{8} \text{ in.})(9 \text{ in.})}{1.67}$ $= 146 \text{ kips} > 106 \text{ kips}$ <b>o.k.</b>	Eqn. D2-1

Check tension rupture of the plates

Take  $A_e$  as the lesser of  $A_n$  and  $0.85A_g$ ,

$$A_e = \min(A_n, 0.85A_g) = \min\left\{\left(\frac{3}{8} \text{ in.}\right)\left[2(9 \text{ in.}) - 4(1 \text{ in.})\right], 0.85(2)\left(\frac{3}{8} \text{ in.}\right)(9 \text{ in.})\right\} = 5.25 \text{ in.}^2$$

Section D3.2  
User Note

LRFD	ASD
$\phi R_n = \phi F_u A_e = 0.75(58 \text{ ksi})(5.25 \text{ in.}^2)$ $= 228 \text{ kips} > 157 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = F_u A_e / \Omega = \frac{(58 \text{ ksi})(5.25 \text{ in.}^2)}{2.00}$ $= 152 \text{ kips} > 106 \text{ kips} \quad \mathbf{o.k.}$

Eqn. D2-2

Check block shear rupture of the plates (outer blocks).

Use  $n = 2$ ,  $L_{ev} = 1\frac{1}{2}$  in., and  $L_{eh} = 1\frac{1}{2}$  in. The calculations here are done in the same manner as those for the angles. Thus,

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$
Tension Rupture Component	Tension Rupture Component
$\phi F_u A_{nt} = (43.5 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$	$F_u A_{nt} / \Omega = (29.0 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$
Shear Yielding Component	Shear Yielding Component
$\phi 0.6 F_y A_{gv} = (72.9 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$	$0.6 F_y A_{gv} / \Omega = (48.6 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$
Shear Rupture Component	Shear Rupture Component
$\phi 0.6 F_u A_{nv} = (78.3 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$	$0.6 F_u A_{nv} / \Omega = (52.2 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$
$\phi R_n = (43.5 \text{ kips} + 72.9 \text{ kips})\left(\frac{3}{4} \text{ in.}\right)(2)$ $= 175 \text{ kips} > 157 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = (29.0 \text{ kips} + 48.6 \text{ kips})\left(\frac{3}{4} \text{ in.}\right)(2)$ $= 116 \text{ kips} > 106 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 9-3a

Manual  
Table 9-3b

Manual  
Table 9-3c

Eqn. J4-5

Similarly, the block shear rupture strength of the interior blocks of the brace-web plates and the brace web are o.k.

Check block shear rupture of the brace web.

Use  $n = 2$ ,  $L_{ev} = 1\frac{1}{2}$  in., and  $L_{eh} = 3$  in.

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$	Manual Table 9-3a
Tension Rupture Component	Tension Rupture Component	
$\phi F_u A_{nt} = (122 \text{ kips/in.})(0.515 \text{ in.})(2)$	$F_u A_{nt} / \Omega = (81.3 \text{ kips/in.})(0.515 \text{ in.})(2)$	Manual Table 9-3b
Shear Yielding Component	Shear Yielding Component	
$\phi 0.6 F_y A_{gv} = (101 \text{ kips/in.})(0.515 \text{ in.})(2)$	$0.6 F_y A_{gv} / \Omega = (67.5 \text{ kips/in.})(0.515 \text{ in.})(2)$	Manual Table 9-3c
Shear Rupture Component	Shear Rupture Component	
$\phi 0.6 F_u A_{nv} = (87.8 \text{ kips/in.})(0.515 \text{ in.})(2)$	$0.6 F_u A_{nv} / \Omega = (58.5 \text{ kips/in.})(0.515 \text{ in.})(2)$	Eqn. J4-5
$\phi R_n = (122 \text{ kips} + 101 \text{ kips})(0.515 \text{ in.})(2)$ $= 216 \text{ kips} > 157 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = (81.3 \text{ kips} + 58.5 \text{ kips})(0.515 \text{ in.})(2)$ $= 144 \text{ kips} > 106 \text{ kips} \quad \mathbf{o.k.}$	

LRFD	ASD	
<i>Check tension yielding of the brace</i>		
$\phi R_n = \phi F_y A_g = 0.90(50 \text{ ksi})(25.6 \text{ in.}^2)$	$R_n / \Omega = F_y A_g / \Omega = \frac{(50 \text{ ksi})(25.6 \text{ in.}^2)}{1.67}$	Eqn. D2-1
$= 1150 \text{ kips} > 675 \text{ kips} \quad \mathbf{o.k.}$	$= 766 \text{ kips} > 450 \text{ kips} \quad \mathbf{o.k.}$	

*Check tension rupture of the brace*

Take  $A_e$  as  $A_n$ ,

$$A_e = A_n = 25.6 \text{ in.}^2 - [4(0.810 \text{ in.}) + 2(0.515 \text{ in.})](1 \text{ in.}) = 21.3 \text{ in.}^2$$

LRFD	ASD	
$\phi R_n = \phi F_u A_e = 0.75(65 \text{ ksi})(21.3 \text{ in.}^2)$	$R_n / \Omega = F_u A_e / \Omega = \frac{(65 \text{ ksi})(21.3 \text{ in.}^2)}{2.00}$	Eqn. D2-2
$= 1040 \text{ kips} > 675 \text{ kips} \quad \mathbf{o.k.}$	$= 692 \text{ kips} > 450 \text{ kips} \quad \mathbf{o.k.}$	

*Design the gusset*

From edge distance, spacing, and thickness requirements of the angles and web plates, try PL  $\frac{3}{4}$ .

Check block shear rupture for the force transmitted through web. Use  $n = 2$ ,  $L_{ev} = 1\frac{1}{2}$  in. and  $L_{eh} = 3$  in.

LRFD	ASD
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$

<p>Tension Rupture Component</p> $\phi F_u A_{nt} = (109 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$ <p>Shear Yielding Component</p> $\phi 0.6 F_y A_{gv} = (72.9 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$ <p>Shear Rupture Component</p> $\phi 0.6 F_u A_{nv} = (78.3 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$ $\phi R_n = (109 \text{ kips} + 72.9 \text{ kips})\left(\frac{3}{4} \text{ in.}\right)(2)$ $= 273 \text{ kips} > 157 \text{ kips} \quad \mathbf{o.k.}$	<p>Tension Rupture Component</p> $F_u A_{nt} / \Omega = (72.5 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$ <p>Shear Yielding Component</p> $0.6 F_y A_{gv} / \Omega = (48.6 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$ <p>Shear Rupture Component</p> $0.6 F_u A_{nv} / \Omega = (52.2 \text{ kips/in.})\left(\frac{3}{4} \text{ in.}\right)(2)$ $R_n / \Omega = (72.5 \text{ kips} + 48.6 \text{ kips})\left(\frac{3}{4} \text{ in.}\right)(2)$ $= 182 \text{ kips} > 106 \text{ kips} \quad \mathbf{o.k.}$	<p>Manual Table 9-3a</p> <p>Manual Table 9-3b</p> <p>Manual Table 9-4</p> <p>Eqn. J4-5</p>
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Check block shear rupture for total brace force

With  $A_{gv} = 24.8 \text{ in.}^2$ ,  $A_{nv} = 16.5 \text{ in.}^2$ , and  $A_{nt} = 12.4 \text{ in.}^2$ . Thus,

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6 F_y A_{gv}, \phi F_u A_{nv})$ <p>Tension Rupture Component</p> $\phi F_u A_{nt} = 0.75(58 \text{ ksi})(12.4 \text{ in.}^2)$ $= 539 \text{ kips}$ <p>Shear Yielding Component</p> $\phi 0.6 F_y A_{gv} = 0.75(0.6)(36 \text{ ksi})(24.8 \text{ in.}^2)$ $= 402 \text{ kips}$ <p>Shear Rupture Component</p> $\phi 0.6 F_u A_{nv} = 0.75(0.6)(58 \text{ ksi})(16.5 \text{ in.}^2)$ $= 431 \text{ kips}$ $\phi R_n = 539 \text{ kips} + \min(402 \text{ kips}, 431 \text{ kips})$ $= 941 \text{ kips} > 675 \text{ kips} \quad \mathbf{o.k.}$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6 F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$ <p>Tension Rupture Component</p> $\frac{F_u A_{nt}}{\Omega} = \frac{(58 \text{ ksi})(12.4 \text{ in.}^2)}{2.00}$ $= 360 \text{ kips}$ <p>Shear Yielding Component</p> $\frac{0.6 F_y A_{gv}}{\Omega} = \frac{(0.6)(36 \text{ ksi})(24.8 \text{ in.}^2)}{2.00}$ $= 268 \text{ kips}$ <p>Shear Rupture Component</p> $\frac{0.6 F_u A_{nv}}{\Omega} = \frac{(0.6)(58 \text{ ksi})(16.5 \text{ in.}^2)}{2.00}$ $= 287 \text{ kips}$ $R_n / \Omega = 360 \text{ kips} + \min(268 \text{ kips}, 287 \text{ kips})$ $= 628 \text{ kips} > 450 \text{ kips} \quad \mathbf{o.k.}$	<p>Section J4.3</p>

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,

LRFD	ASD
$\phi R_n = \phi F_y A_w$ $= 0.90 \left[ (36 \text{ ksi})(30.9 \text{ in.})\left(\frac{3}{4} \text{ in.}\right) + (50 \text{ ksi})(3.90 \text{ in.})(0.590 \text{ in.}) \right]$ $= 854 \text{ kips} > 675 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = F_y A_w / \Omega$ $= \frac{\left[ (36 \text{ ksi})(30.9 \text{ in.})\left(\frac{3}{4} \text{ in.}\right) + (50 \text{ ksi})(3.90 \text{ in.})(0.590 \text{ in.}) \right]}{1.67}$ $= 568 \text{ kips} > 450 \text{ kips} \quad \mathbf{o.k.}$

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:

$$r_n = 1.2L_c t F_u \leq 2.4dt F_u$$

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,

$$L_c = 1\frac{1}{2} \text{ in.} - \left(\frac{1}{2}\right)\left(\frac{15}{16} \text{ in.}\right) = 1.03 \text{ in.}$$

LRFD	ASD
$\phi r_n = \phi 1.2L_c t F_u < 2.4dt F_u$ $= 0.75(1.2)(1.03 \text{ in.})\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})$ $\leq 2.4(0.75)\left(\frac{7}{8} \text{ in.}\right)\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})$ $= 40.4 \text{ kips} < 68.6 \text{ kips}$ $= 40.4 \text{ kips/bolt}$	$r_n / \Omega = \frac{1.2L_c t F_u}{\Omega} \leq \frac{2.4dt F_u}{\Omega}$ $= \frac{1.2(1.03 \text{ in.})\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})}{2.00}$ $\leq \frac{2.4\left(\frac{7}{8} \text{ in.}\right)\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})}{2.00}$ $= 26.9 \text{ kips} < 45.7 \text{ kips}$ $= 26.9 \text{ kips/bolt}$

Eqn. J3-6a

For an interior bolt,

$$L_c = 3 \text{ in.} - (1)\left(\frac{15}{16} \text{ in.}\right) = 2.06 \text{ in.}$$

LRFD	ASD
$\phi r_n = \phi 1.2 L_c t F_u < 2.4 d t F_u$ $= 0.75(1.2)(2.06 \text{ in.})(\frac{3}{4} \text{ in.})(58 \text{ ksi})$ $\leq 0.75(2.4)(\frac{7}{8} \text{ in.})(\frac{3}{4} \text{ in.})(58 \text{ ksi})$ $= 81.0 \text{ kips} < 68.6 \text{ kips}$ $= 68.6 \text{ kips/bolt}$	$r_n / \Omega = \frac{1.2 L_c t F}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(2.06 \text{ in.})(\frac{3}{4} \text{ in.})(58 \text{ ksi})}{2.00}$ $\leq \frac{2.4(\frac{7}{8} \text{ in.})(\frac{3}{4} \text{ in.})(58 \text{ ksi})}{2.00}$ $= 54 \text{ kips} < 45.7 \text{ kips}$ $= 45.7 \text{ kips/bolt}$

Eqn. J3-6a

Note: The above strengths are per angle.

The gusset plate bolt strengths are the same as the angle bolt strengths since the gusset plate is the same thickness and material, and has the same edge distance. Thus,

LRFD	ASD
For an edge bolt, $\phi r_n = 40.4 \text{ kips/bolt}$	For an edge bolt, $r_n / \Omega = 26.9 \text{ kips/bolt}$
For an interior bolt, $\phi r_n = 68.6 \text{ kips/bolt}$	For an interior bolt, $r_n / \Omega = 45.7 \text{ kips/bolt}$

For the brace, edge bolt  $L_c = 1.03 \text{ in.}$  and interior bolt  $L_c = 2.06 \text{ in.}$

LRFD	ASD
<p>Brace edge bolt,</p> $\phi r_n = \phi 1.2 L_c t F_u < 2.4 d t F_u$ $= 0.75(1.2)(1.03 \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})$ $\leq 0.75(2.4)(\frac{7}{8} \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})$ $= 48.9 \text{ kips} < 83.3 \text{ kips}$ $= 48.9 \text{ kips/bolt}$	<p>Brace edge bolt,</p> $r_n / \Omega = \frac{1.2 L_c t F}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(1.03 \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})}{2.00}$ $\leq \frac{2.4(\frac{7}{8} \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})}{2.00}$ $= 32.6 \text{ kips} < 55.5 \text{ kips}$ $= 32.6 \text{ kips/bolt}$

Eqn. J3-6a

LRFD	ASD
<p>Brace interior bolt,</p> $\phi r_n = \phi 1.2 L_c t F_u < 2.4 d t F_u$ $= 0.75(1.2)(2.06 \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})$ $\leq 0.75(2.4)(\frac{7}{8} \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})$ $= 97.8 \text{ kips} < 83.2 \text{ kips}$ $= 83.2 \text{ kips/bolt}$	<p>Brace interior bolt,</p> $r_n / \Omega = \frac{1.2 L_c t F}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(2.06 \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})}{2.00}$ $\leq \frac{2.4(\frac{7}{8} \text{ in.})(0.810 \text{ in.})(65 \text{ ksi})}{2.00}$ $= 65.2 \text{ kips} < 55.5 \text{ kips}$ $= 55.5 \text{ kips/bolt}$

Eqn. J3-6a

The bearing strength of the flange connection can now be calculated. Summarizing the various strengths:

LRFD			ASD		
Member	Edge	Interior	Member	Edge	Interior
Angle (2)	80.8 kips	137 kips	Angle (2)	53.8 kips	91.4 kips
Gusset	40.4 kips	68.6 kips	Gusset	26.9 kips	45.7 kips
Brace	48.9 kips	83.2 kips	Brace	32.6 kips	55.5 kips
From the above table, $\phi R_n = (1 \text{ bolt})(40.4 \text{ kips/bolt})$ $+ (5 \text{ bolts})(68.6 \text{ kips/bolt})$ $= 383 \text{ kips} > 259 \text{ kips}$			From the above table, $R_n / \Omega = (1 \text{ bolt})(26.9 \text{ kips/bolt})$ $+ (5 \text{ bolts})(45.7 \text{ kips/bolt})$ $= 255 \text{ kips} > 172 \text{ kips}$		
<b>o.k.</b>			<b>o.k.</b>		

Note: The gusset edge bolt bearing strength is less than the bolt double shear strength; the bolt shear strength must be re-checked. Thus, the revised bolt shear strength is

LRFD	ASD
$\phi R_n = (1 \text{ bolt})(40.4 \text{ kips/bolt})$ $+ (5 \text{ bolts})(43.3 \text{ kips/bolt})$ $= 257 \text{ kips} < 259 \text{ kips}$	$R_n / \Omega = (1 \text{ bolt})(26.9 \text{ kips/bolt})$ $+ (5 \text{ bolts})(28.9 \text{ kips/bolt})$ $= 171 \text{ kips} < 172 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

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

$$e_b = \frac{d_{beam}}{2} = \frac{18.7 \text{ in.}}{2} = 9.35 \text{ in.}$$

$$e_c = \frac{d_{column}}{2} = \frac{20.9 \text{ in.}}{2} = 10.5 \text{ in.}$$

$$\tan \theta = \frac{12}{9 \frac{9}{16}} = 1.25$$

$$\text{and } e_b \tan \theta - e_c = (9.35 \text{ in.})(1.25) - (10.5 \text{ in.}) = 1.19 \text{ in.}$$

Try gusset PL 3/4 x 42 in. horizontally x 33 in. vertically (several intermediate gusset dimensions were inadequate). With connection centroids at the midpoint of the gusset edges

$$\alpha = \frac{42 \text{ in.}}{2} + \frac{1}{2} \text{ in.} = 21 \frac{1}{2} \text{ in.}$$

where 1/2 in. is allowed for the setback between the gusset and the column, and



$$\bar{\beta} = \frac{33 \text{ in.}}{2} = 16\frac{1}{2} \text{ in.}$$

Choosing  $\beta = \bar{\beta}$ , the  $\alpha$  required for the uniform forces is

$$\alpha = eb \tan \theta - ec + \beta \tan \theta = 1.19 + (16.5 \text{ in.})(1.25) = 21.8 \text{ in.}$$

The resulting eccentricity is  $\alpha - \bar{\alpha}$ , where

$$\alpha - \bar{\alpha} = 21.8 \text{ in.} - 21.5 \text{ in.} = 0.3 \text{ in.}$$

Since slight eccentricity is negligible. Use  $\alpha = 21.8 \text{ in.}$  and  $\beta = 16.5 \text{ in.}$

Calculate gusset interface forces

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(21.8 \text{ in.} + 10.5)^2 + (16.5 \text{ in.} + 9.35 \text{ in.})^2} = 41.4 \text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$H_{uc} = \frac{e_c P_u}{r} = \frac{(10\frac{1}{2} \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 171 \text{ kips}$	$H_{ac} = \frac{e_c P_a}{r} = \frac{(10\frac{1}{2} \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 114 \text{ kips}$
$V_{uc} = \frac{\beta P_u}{r} = \frac{(16\frac{1}{2} \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 269 \text{ kips}$	$V_{ac} = \frac{\beta P_a}{r} = \frac{(16\frac{1}{2} \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 179 \text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \frac{\alpha P_u}{r} = \frac{(21.8 \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 355 \text{ kips}$	$H_{ab} = \frac{\alpha P_a}{r} = \frac{(21.8 \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 237 \text{ kips}$
$V_{ub} = \frac{e_b P_u}{r} = \frac{(9.35 \text{ in.})(675 \text{ kips})}{41.4 \text{ in.}} = 153 \text{ kips}$	$V_{ab} = \frac{e_b P_a}{r} = \frac{(9.35 \text{ in.})(450 \text{ kips})}{41.4 \text{ in.}} = 102 \text{ kips}$

Design gusset-to-column connection

The forces involved are  $V_{uc} = 269 \text{ kips}$  ( $V_{ac} = 179 \text{ kips ASD}$ ) shear and  $H_{uc} = 171 \text{ kips}$  ( $H_{ac} = 114 \text{ kips ASD}$ ) tension.

Try 2L4×4×½×2'-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
$T_u = \frac{H_{uc}}{n} = \frac{171 \text{ kips}}{20 \text{ bolts}} = 8.55 \text{ kips/bolt}$	$T_a = \frac{H_{ac}}{n} = \frac{114 \text{ kips}}{20 \text{ bolts}} = 5.70 \text{ kips/bolt}$

<p>Check design strength of bolts for tension-shear interaction.</p> $r_{uv} = \frac{V_{uc}}{n} = \frac{269 \text{ kips}}{20 \text{ bolts}} = 13.5 \text{ kips/bolt}$ <p>13.5 kips/bolt &lt; 21.6 kips/bolt    <b>o.k.</b></p> $f_{uv} = \frac{r_{uv}}{A_b} = \frac{13.5 \text{ kips}}{0.6013 \text{ in.}^2} = 22.5 \text{ ksi}$ $F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{uv} \leq F_{nt}$ $= 1.3(90 \text{ ksi}) - \frac{90 \text{ ksi}}{0.75(48 \text{ ksi})}(22.5 \text{ ksi})$ $= 60.8 \text{ ksi} < 90 \text{ ksi}$ <p>Use <math>F_{nt}' = 60.8 \text{ ksi}</math></p> $B_u = \phi F_{nt}' A_b = 0.75(60.8 \text{ ksi})(0.6013 \text{ in.}^2)$ $= 27.4 \text{ kips/bolt} > 8.55 \text{ kips/bolt} \quad \mathbf{o.k.}$	<p>Check allowable strength of bolts for tension-shear interaction.</p> $r_{av} = \frac{V_{ac}}{n} = \frac{179 \text{ kips}}{20 \text{ bolts}} = 8.95 \text{ kips/bolt}$ <p>8.95 kips/bolt &lt; 14.4 kips/bolt    <b>o.k.</b></p> $f_{av} = \frac{r_{av}}{A_b} = \frac{8.95 \text{ kips}}{0.6013 \text{ in.}^2} = 14.9 \text{ ksi}$ $F_{nt}' = 1.3F_{nt} - \frac{F_{nt}\Omega}{F_{nv}} f_{av} \leq F_{nt}$ $= 1.3(90 \text{ ksi}) - \frac{90 \text{ ksi}(2.00)}{48 \text{ ksi}}(14.9 \text{ ksi})$ $= 61.1 \text{ ksi} < 90 \text{ ksi}$ <p>Use <math>F_{nt}' = 61.1 \text{ ksi}</math></p> $B_a = \frac{F_{nt}' A_b}{\Omega} = \frac{(61.1 \text{ ksi})(0.6013 \text{ in.}^2)}{2.00}$ $= 18.4 \text{ kips/bolt} > 5.70 \text{ kips/bolt} \quad \mathbf{o.k.}$
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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
$\phi r_n = \phi 1.2L_c t F_u < 2.4dt F_u$ $= 0.75(1.2)(1.03 \text{ in.})(\frac{1}{2} \text{ in.})(58 \text{ ksi})$ $\leq 0.75(2.4)(\frac{7}{8} \text{ in.})(\frac{1}{2} \text{ in.})(58 \text{ ksi})$ $= 26.9 \text{ kips} < 45.7 \text{ kips}$ $= 26.9 \text{ kips/bolt}$ <p>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 <b>o.k.</b></p>	$r_n / \Omega = \frac{1.2L_c t F}{\Omega} \leq \frac{2.4dt F_u}{\Omega}$ $= \frac{1.2(1.03 \text{ in.})(\frac{1}{2} \text{ in.})(58 \text{ ksi})}{2.00}$ $\leq \frac{2.4(\frac{7}{8} \text{ in.})(\frac{1}{2} \text{ in.})(58 \text{ ksi})}{2.00}$ $= 17.9 \text{ kips} < 30.5 \text{ kips}$ $= 17.9 \text{ kips/bolt}$ <p>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 <b>o.k.</b></p>

Eqn. J3-6a

Check prying action

$$b = g - \frac{t}{2} = 2\frac{1}{2} \text{ in.} - \frac{\frac{1}{2} \text{ in.}}{2} = 2\frac{1}{4} \text{ in.}$$

Note: 1/4 in. entering and tightening clearance is accommodated, **o.k.**

$$a = 4 \text{ in.} - g = 4 \text{ in.} - 2\frac{1}{2} \text{ in.} = 1\frac{1}{2} \text{ in.}$$

Since  $a = 1\frac{1}{2} \text{ in.}$  is less than  $1.25b = 2.81 \text{ in.}$ , use  $a = 1\frac{1}{2} \text{ in.}$

$$b' = b - \frac{d}{2} = 2\frac{1}{4} \text{ in.} - \frac{\frac{7}{8} \text{ in.}}{2} = 1.81 \text{ in.}$$

$$a' = a + \frac{d}{2} = 1\frac{1}{2} \text{ in.} + \frac{\frac{7}{8} \text{ in.}}{2} = 1.94 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.81 \text{ in.}}{1.94 \text{ in.}} = 0.935$$

LRFD	ASD
$\beta = \frac{1}{\rho} \left( \frac{B_u}{T_u} - 1 \right) = \frac{1}{0.935} \left( \frac{27.4 \text{ kips/bolt}}{8.55 \text{ kips/bolt}} - 1 \right)$ $= 2.36$	$\beta = \frac{1}{\rho} \left( \frac{B_a}{T_a} - 1 \right) = \frac{1}{0.935} \left( \frac{18.4 \text{ kips/bolt}}{5.70 \text{ kips/bolt}} - 1 \right)$ $= 2.38$

Since  $\beta > 1$ , set  $\alpha' = 1.0$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{\frac{15}{16} \text{ in.}}{3 \text{ in.}} = 0.688$$

LRFD	ASD
$t_{req} = \sqrt{\frac{4.44T_u b'}{pF_u (1 + \delta\alpha')}} = \sqrt{\frac{4.44(8.55 \text{ kips/bolt})(1.81 \text{ in.})}{(3 \text{ in.})(58 \text{ ksi})[1 + 0.688(1.0)]}}$ $= 0.484 \text{ in.}$ <p>Since <math>t = \frac{1}{2} \text{ in.} &gt; 0.484 \text{ in.}</math>, angles are <b>o.k.</b></p>	$t_{req} = \sqrt{\frac{6.66T_a b'}{pF_u (1 + \delta\alpha')}} = \sqrt{\frac{6.66(5.70 \text{ kips/bolt})(1.81 \text{ in.})}{(3 \text{ in.})(58 \text{ ksi})[1 + 0.688(1.0)]}}$ $= 0.484 \text{ in.}$ <p>Since <math>t = \frac{1}{2} \text{ in.} &gt; 0.484 \text{ in.}</math>, angles are <b>o.k.</b></p>

*Design welds*

Try fillet welds around the perimeter (three sides) of both angles.

LRFD	ASD
$P_{uc} = \sqrt{H_{uc}^2 + V_{uc}^2}$ $= \sqrt{(171 \text{ kips})^2 + (269 \text{ kips})^2} = 319 \text{ kips}$ $\theta = \tan^{-1}\left(\frac{H_{uc}}{V_{uc}}\right) = \tan^{-1}\left(\frac{171 \text{ kips}}{269 \text{ kips}}\right) = 32.4^\circ$	$P_{ac} = \sqrt{H_{ac}^2 + V_{ac}^2}$ $= \sqrt{(114 \text{ kips})^2 + (179 \text{ kips})^2} = 212 \text{ kips}$ $\theta = \tan^{-1}\left(\frac{H_{ac}}{V_{ac}}\right) = \tan^{-1}\left(\frac{114 \text{ kips}}{179 \text{ kips}}\right) = 32.5^\circ$

From Manual Table 8-8 with  $\theta = 30^\circ$ ,

$$l = 30 \text{ in.}, kl = 3\frac{1}{2} \text{ in.}, \text{ therefore } k = 0.117$$

By interpolation

$$x = 0.011, xl = 0.011(30 \text{ in.}) = 0.33 \text{ in.}$$

$$al = 4 \text{ in.} - xl = 4 \text{ in.} - 0.33 \text{ in.} = 3.67 \text{ in.}$$

$$a = 0.122$$

By interpolation

$$C = 2.60$$

LRFD	ASD
$D_{req} = \frac{P_{uc}}{\phi CC_1 l}$ $= \frac{319 \text{ kips}}{0.75(2.60)(1.0)(2 \text{ welds})(30 \text{ in.})}$ $= 2.73 \rightarrow 3 \text{ sixteenths}$	$D_{req} = \frac{P_{ac} \Omega}{CC_1 l}$ $= \frac{(212 \text{ kips})(2.00)}{(2.60)(1.0)(2 \text{ welds})(30 \text{ in.})}$ $= 2.72 \rightarrow 3 \text{ sixteenths}$

From Specification Table J2.4, minimum weld size is  $\frac{1}{4}$  in. Use  $\frac{1}{4}$ -in. fillet welds.

*Check gusset thickness against weld size required for strength*

$$\text{For two fillet welds, } t_{\min} = \frac{6.19D}{F_u} = \frac{6.19(2.73 \text{ sixteenths})}{58 \text{ ksi}} = 0.291 \text{ in.} < \frac{3}{4} \text{ in.} \quad \mathbf{o.k.}$$

Check strength of angles

Check shear yielding (due to  $V_{uc}$  or  $V_{ac}$ )

LRFD	ASD	
$\phi R_n = \phi(0.60F_y A_g)$ $= 1.00[0.60(36 \text{ ksi})(2)(30 \text{ in.})(\frac{1}{2} \text{ in.})]$ $= 648 \text{ kips} > 269 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{0.60F_y A_g}{\Omega}$ $= \frac{0.60(36 \text{ ksi})(2)(30 \text{ in.})(\frac{1}{2} \text{ in.})}{1.50}$ $= 432 \text{ kips} > 179 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-3

Similarly, shear yielding of the angles due to  $H_{uc}$  is not critical.

Check shear rupture

LRFD	ASD	
$\phi R_n = \phi(0.60F_u A_{nv})$ $= 0.75(0.60)(58 \text{ ksi})(\frac{1}{2} \text{ in.})$ $\times [(2)(30 \text{ in.}) - 20(1 \text{ in.})]$ $= 522 \text{ kips} > 269 \text{ kips}$ $\mathbf{o.k.}$	$R_n / \Omega = \frac{0.60F_u A_{nv}}{\Omega}$ $= \frac{0.60(58 \text{ ksi})(\frac{1}{2} \text{ in.})[(2)(30 \text{ in.}) - 20(1 \text{ in.})]}{2.00}$ $= 348 \text{ kips} > 179 \text{ kips} \quad \mathbf{o.k.}$	Eqn. J4-4

Block shear rupture

Use  $n = 10$ ,  $L_{ev} = 1\frac{1}{2}$  in. and  $L_{eh} = 1\frac{1}{2}$  in. Thus,

LRFD	ASD	
$\phi R_n = \phi F_u A_{nt} U_{bs} + \min(\phi 0.6F_y A_{gv}, \phi F_u A_{nv})$	$\frac{R_n}{\Omega} = \frac{F_u A_{nt} U_{bs}}{\Omega} + \min\left(\frac{0.6F_y A_{gv}}{\Omega}, \frac{F_u A_{nv}}{\Omega}\right)$	
Tension Rupture Component	Tension Rupture Component	Manual Table 9-3a
$\phi F_u A_{nt} = (43.5 \text{ kips/in.})(\frac{1}{2} \text{ in.})(2)$	$F_u A_{nt} / \Omega = (29.0 \text{ kips/in.})(\frac{1}{2} \text{ in.})(2)$	
Shear Yielding Component	Shear Yielding Component	Manual Table 9-3b
$\phi 0.6F_y A_{gv} = (462 \text{ kips/in.})(\frac{1}{2} \text{ in.})(2)$	$0.6F_y A_{gv} / \Omega = (308 \text{ kips/in.})(\frac{1}{2} \text{ in.})(2)$	
Shear Rupture Component	Shear Rupture Component	Manual Table 9-3c
$\phi 0.6F_u A_{nv} = (496 \text{ kips/in.})(\frac{1}{2} \text{ in.})(2)$	$0.6F_u A_{nv} / \Omega = (331 \text{ kips/in.})(\frac{1}{2} \text{ in.})(2)$	
$\phi R_n = (43.5 \text{ kips} + 462 \text{ kips})(\frac{1}{2} \text{ in.})(2)$ $= 506 \text{ kips} > 269 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = (29.0 \text{ kips} + 308 \text{ kips})(\frac{1}{2} \text{ in.})(2)$ $= 337 \text{ kips} > 179 \text{ kips} \quad \mathbf{o.k.}$	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
$V_{ub} = 153$ kips	$V_{ab} = 102$ kips

Because one edge of the gusset is welded and the other is bolted, the distribution of force on the welded edge is uneven. To account for this, the required strength of the gusset edge is amplified by a factor of 1.25 to allow for the redistribution of forces on the weld.

The stresses on the gusset plate at the welded edge are as follows:

LRFD	ASD	
$f_{ua} = \frac{V_{ub}}{tl} \leq \phi F_y$ $= \frac{153 \text{ kips}}{(\frac{3}{4} \text{ in.})(42 \text{ in.})} \leq 0.90(36 \text{ ksi})$ $= 4.86 \text{ ksi} < 32.4 \text{ ksi} \quad \mathbf{o.k.}$	$f_{aa} = \frac{V_{ab}}{tl} \leq \frac{F_y}{\Omega}$ $= \frac{102 \text{ kips}}{(\frac{3}{4} \text{ in.})(42 \text{ in.})} \leq \frac{36 \text{ ksi}}{1.67}$ $= 3.24 \text{ ksi} < 21.6 \text{ ksi} \quad \mathbf{o.k.}$	Eqn. J4-6
$f_{uv} = \frac{H_{ub}}{tl} \leq \phi(0.60F_y)$ $= \frac{355 \text{ kips}}{(\frac{3}{4} \text{ in.})(42 \text{ in.})} \leq 1.00(0.60)(36 \text{ ksi})$ $= 11.3 \text{ ksi} < 21.6 \text{ ksi} \quad \mathbf{o.k.}$	$f_{av} = \frac{H_{ab}}{tl} \leq \frac{0.60F_y}{\Omega}$ $= \frac{237 \text{ kips}}{(\frac{3}{4} \text{ in.})(42 \text{ in.})} \leq \frac{0.60(36 \text{ ksi})}{1.50}$ $= 7.52 \text{ ksi} < 14.4 \text{ ksi} \quad \mathbf{o.k.}$	Eqn. J4-3

LRFD	ASD	
$\theta = \tan^{-1}\left(\frac{V_{ub}}{H_{ub}}\right) = \tan^{-1}\left(\frac{153 \text{ kips}}{355 \text{ kips}}\right) = 23.3^\circ$	$\theta = \tan^{-1}\left(\frac{V_{ab}}{H_{ab}}\right) = \tan^{-1}\left(\frac{102 \text{ kips}}{237 \text{ kips}}\right) = 23.3^\circ$	Section J2.4

$\mu = 1.0 + 0.5 \sin^{1.5} \theta = 1.0 + 0.5 \sin^{1.5} (23.3^\circ) = 1.12$  Eqn. J2-5

The weld strength per 1/16 in. is as follows:

LRFD	ASD
$\phi r_w = (1.392 \text{ kips/in.})(1.12) = 1.56 \text{ kips/in.}$	$r_w / \Omega = (0.928 \text{ kips/in.})(1.12) = 1.04 \text{ kips/in.}$
The peak weld stress is given by	The peak weld stress is given by
$f_{u \text{ peak}} = \left(\frac{t}{2}\right) \sqrt{(f_{ua} + f_{ub})^2 + f_{uv}^2}$	$f_{a \text{ peak}} = \left(\frac{t}{2}\right) \sqrt{(f_{aa} + f_{ab})^2 + f_{av}^2}$

$= \left(\frac{3/4 \text{ in.}}{2}\right) \sqrt{(4.86 \text{ ksi} + 0 \text{ ksi})^2 + (11.3 \text{ ksi})^2}$ $= 4.61 \text{ kips/in.}$ <p>The average stress is</p> $f_{u \text{ ave}} = \frac{t \left[ \sqrt{(f_{ua} - f_{ub})^2 + f_{uv}^2} + \sqrt{(f_{ua} + f_{ub})^2 + f_{uv}^2} \right]}{4}$ <p>Since <math>f_{ub} = 0 \text{ ksi}</math> (there is no moment on the edge) <math>f_{u \text{ ave}} = f_{u \text{ peak}} = 4.61 \text{ kips/in.}</math>          The design weld stress is</p> $f_{u \text{ weld}} = \max \{ f_{u \text{ peak}}, 1.25 f_{u \text{ ave}} \}$ $= \max \{ 4.61 \text{ kips/in.}, 1.25(4.61 \text{ kips/in.}) \}$ $= 5.76 \text{ kips/in.}$ <p>The required weld size is</p> $D_{\text{req}} = \frac{f_{u \text{ weld}}}{\phi r_w}$ $= \frac{5.76 \text{ kips/in.}}{1.56 \text{ kips/in.}} = 3.69 \rightarrow 4 \text{ sixteenths}$ <p>Minimum weld size is <math>5/16 \text{ in.}</math> Use a <math>5/16 \text{-in.}</math> fillet weld.</p>	$= \left(\frac{3/4 \text{ in.}}{2}\right) \sqrt{(3.24 \text{ ksi} + 0 \text{ ksi})^2 + (7.52 \text{ ksi})^2}$ $= 3.07 \text{ kips/in.}$ <p>The average stress is</p> $f_{a \text{ ave}} = \frac{t \left[ \sqrt{(f_{aa} - f_{ab})^2 + f_{av}^2} + \sqrt{(f_{aa} + f_{ab})^2 + f_{av}^2} \right]}{4}$ <p>Since <math>f_{ab} = 0 \text{ ksi}</math> (there is no moment on the edge) <math>f_{a \text{ ave}} = f_{a \text{ peak}} = 4.61 \text{ kips/in.}</math>          The design weld stress is</p> $f_{a \text{ weld}} = \max \{ f_{a \text{ peak}}, 1.25 f_{a \text{ ave}} \}$ $= \max \{ 3.07 \text{ kips/in.}, 1.25(3.07 \text{ kips/in.}) \}$ $= 3.84 \text{ kips/in.}$ <p>The required weld size is</p> $D_{\text{req}} = \frac{\Omega f_{a \text{ weld}}}{r_w}$ $= \frac{3.84 \text{ kips/in.}}{1.04 \text{ kips/in.}} = 3.69 \rightarrow 4 \text{ sixteenths}$ <p>Minimum weld size is <math>5/16 \text{ in.}</math> Use a <math>5/16 \text{-in.}</math> fillet weld.</p>
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LRFD	ASD
<p>Check local web yielding of the beam</p> $\phi R_n = \phi (N + 2.5k) F_{yw} t_w$ $= 1.0 [42 \text{ in.} + 2.5(1.34 \text{ in.})] (50 \text{ ksi}) (0.590 \text{ in.})$ $= 1340 \text{ kips} > 153 \text{ kips} \quad \text{o.k.}$	<p>Check local web yielding of the beam</p> $R_n / \Omega = \frac{(N + 2.5k) F_{yw} t_w}{\Omega}$ $= \frac{[42 \text{ in.} + 2.5(1.34 \text{ in.})] (50 \text{ ksi}) (0.590 \text{ in.})}{1.50}$ $= 892 \text{ kips} > 102 \text{ kips} \quad \text{o.k.}$

Eqn. J10-3

*Design beam-to-column connection*

Since the brace may be in tension or compression, the required strength the beam-to-column connection is as follows. The required shear strength is

LRFD	ASD
$R_{ub} \pm V_{ub} = 15 \text{ kips} + 153 \text{ kips} = 168 \text{ kips}$	$R_{ab} \pm V_{ab} = 10 \text{ kips} + 102 \text{ kips} = 112 \text{ kips}$
and the required axial strength is	and the required axial strength is
$A_{ub} \pm (H_u - H_{ub}) = 0 \pm 171 \text{ kips} = 171 \text{ kips}$	$A_{ab} \pm (H_a - H_{ab}) = 0 \pm 114 \text{ kips} = 114 \text{ kips}$

Try 2L8×6×¾×1'-2½ (leg gage = 2¾ in.) welded to the beam web, bolted with five rows of 7/8 in. diameter A325-N bolts in standard holes to the column flange.

*Calculate tensile force per bolt*

LRFD	ASD
$T_u = \frac{A_{ub} \pm (H_u - H_{ub})}{n} = \frac{171 \text{ kips}}{10 \text{ bolts}}$	$T_a = \frac{A_{ab} \pm (H_a - H_{ab})}{n} = \frac{114 \text{ kips}}{10 \text{ bolts}}$
= 17.1 kips/bolt	= 11.4 kips/bolt

*Check available strength of bolts for tension-shear interaction*

LRFD	ASD
$V_u = \frac{R_{ub} \pm V_{ub}}{n} = \frac{168 \text{ kips}}{10 \text{ bolts}}$	$V_a = \frac{R_{ab} \pm V_{ab}}{n} = \frac{112 \text{ kips}}{10 \text{ bolts}}$
= 16.8 kips/bolt	= 11.2 kips/bolt
$f_{uv} = \frac{V_u}{A_b} = \frac{16.8 \text{ kips/bolt}}{0.6013 \text{ in.}^2} = 27.4 \text{ ksi}$	$f_{av} = \frac{V_a}{A_b} = \frac{11.2 \text{ kips/bolt}}{0.6013 \text{ in.}^2} = 18.6 \text{ ksi}$
$F_{nt}' = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{mv}} f_{uv} \leq F_{nt}$	$F_{nt}' = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{mv}} f_{av} \leq F_{nt}$
$= 1.3(90 \text{ ksi}) - \frac{90 \text{ ksi}}{0.75(48 \text{ ksi})}(27.4 \text{ ksi}) < 90 \text{ ksi}$	$= 1.3(90 \text{ ksi}) - \frac{2.00(90 \text{ ksi})}{48 \text{ ksi}}(18.6 \text{ ksi}) < 90 \text{ ksi}$
= 48.5 ksi < 90 ksi <b>o.k.</b>	= 47.2 ksi < 90 ksi <b>o.k.</b>
Use $F_{nt}' = 48.5 \text{ ksi}$ .	Use $F_{nt}' = 47.2 \text{ ksi}$ .
$B_u = \phi F_{nt}' A_b = 0.75(48.5 \text{ ksi})(0.6013 \text{ in.}^2)$	$B_a = \frac{F_{nt}' A_b}{\Omega} = \frac{(48.5 \text{ ksi})(0.6013 \text{ in.}^2)}{2.00}$
= 21.9 kips/bolt > 17.1 kips/bolt <b>o.k.</b>	= 14.2 kips/bolt > 11.4 kips/bolt <b>o.k.</b>

Table J3.2,  
Eqns. J3-3a  
and J3-3b



LRFD	ASD
<p><i>Check bearing strength</i></p> $\phi r_n = \phi 1.2 L_c t F_u < 2.4 d t F_u$ $= 0.75(1.2)(1.03 \text{ in.})\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})$ $\leq 0.75(2.4)\left(\frac{7}{8} \text{ in.}\right)\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})$ $= 40.4 \text{ kips} < 68.6 \text{ kips}$ $= 40.4 \text{ kips/bolt}$ <p>Since this edge bolt value exceeds the single-shear strength of the bolts 21.6 kips, bearing does not control. <b>o.k.</b></p>	<p><i>Check bearing strength</i></p> $r_n / \Omega = \frac{1.2 L_c t F}{\Omega} \leq \frac{2.4 d t F_u}{\Omega}$ $= \frac{1.2(1.03 \text{ in.})\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})}{2.00}$ $\leq \frac{2.4\left(\frac{7}{8} \text{ in.}\right)\left(\frac{3}{4} \text{ in.}\right)(58 \text{ ksi})}{2.00}$ $= 26.9 \text{ kips} < 45.7 \text{ kips}$ $= 26.9 \text{ kips/bolt}$ <p>Since this edge bolt value exceeds the single-shear strength of the bolts 14.4 kips, bearing does not control. <b>o.k.</b></p>

Eqn. J3-6a

*Check prying action*

$$b = g - \frac{t}{2} = 2\frac{3}{4} \text{ in.} - \frac{0.75 \text{ in.}}{2} = 2\frac{3}{8} \text{ in.}$$

Note: 1/4 in. entering and tightening clearance is accommodated, **o.k.**

$$a = 6 \text{ in.} - g = 6 \text{ in.} - 2\frac{3}{4} = 3\frac{1}{4} \text{ in.}$$

Since  $a = 3\frac{1}{4} \text{ in.}$  exceeds  $1.25b = 2.97 \text{ in.}$ , use  $a = 2.97 \text{ in.}$  for calculation purposes.

$$b' = b - \frac{d}{2} = 2\frac{3}{8} \text{ in.} - \frac{7}{8} \text{ in.}}{2} = 1.94 \text{ in.}$$

$$a' = a + \frac{d}{2} = 2.97 \text{ in.} + \frac{7}{8} \text{ in.}}{2} = 3.41 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.94 \text{ in.}}{3.41 \text{ in.}} = 0.569$$

$$p = \frac{14\frac{1}{2} \text{ in.}}{5 \text{ bolts}} = 2.90 \text{ in./bolt}$$

$$\delta = 1 - \frac{d'}{p} = 1 - \frac{15\frac{1}{16} \text{ in.}}{2.90 \text{ in.}} = 0.677$$

LRFD	ASD
$\beta = \frac{1}{\rho} \left( \frac{B_u}{T_u} - 1 \right) = \frac{1}{0.569} \left( \frac{21.9 \text{ kips}}{17.1 \text{ kips}} - 1 \right)$ $= 0.493$	$\beta = \frac{1}{\rho} \left( \frac{B_a}{T_a} - 1 \right) = \frac{1}{0.569} \left( \frac{14.2 \text{ kips}}{11.4 \text{ kips}} - 1 \right)$ $= 0.432$

$\alpha' = \frac{1}{\delta} \left( \frac{\beta}{1-\beta} \right) \leq 1.0$ $= \frac{1}{0.677} \left( \frac{0.493}{1-0.493} \right) \leq 1.0$ $= 1.44 > 1.0, \text{ use } \alpha' = 1.0.$ $t_{req} = \sqrt{\frac{4.44T_u b'}{pF_u (1+\delta\alpha')}}$ $= \sqrt{\frac{4.44(17.1 \text{ kips/bolt})(1.94 \text{ in.})}{(2.90 \text{ in.})(58 \text{ ksi})[1+(0.677)(1.0)]}}$ $= 0.723 \text{ in.} < \frac{3}{4} \text{ in.} \quad \mathbf{o.k.}$	$\alpha' = \frac{1}{\delta} \left( \frac{\beta}{1-\beta} \right) \leq 1.0$ $= \frac{1}{0.677} \left( \frac{0.432}{1-0.432} \right) \leq 1.0$ $= 1.12 > 1.0, \text{ use } \alpha' = 1.0.$ $t_{req} = \sqrt{\frac{6.66T_u b'}{pF_u (1+\delta\alpha')}}$ $= \sqrt{\frac{6.66(11.4 \text{ kips/bolt})(1.94 \text{ in.})}{(2.90 \text{ in.})(58 \text{ ksi})[1+(0.677)(1.0)]}}$ $= 0.723 \text{ in.} < \frac{3}{4} \text{ in.} \quad \mathbf{o.k.}$
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Design welds

Try fillet welds around perimeter (three sides) of both angles.

LRFD	ASD
$P_{uc} = \sqrt{(171 \text{ kips})^2 + (168 \text{ kips})^2} = 240 \text{ kips}$	$P_{ac} = \sqrt{(114 \text{ kips})^2 + (112 \text{ kips})^2} = 160 \text{ kips}$
$\theta = \tan^{-1} \left( \frac{171 \text{ kips}}{168 \text{ kips}} \right) = 45.5^\circ$	$\theta = \tan^{-1} \left( \frac{114 \text{ kips}}{112 \text{ kips}} \right) = 45.5^\circ$

For  $\theta = 45^\circ$ ,  $l = 14\frac{1}{2}$  in.,  $kl = 7\frac{1}{2}$  in., and  $k = 0.517$

By interpolation

$$x = 0.132,$$

$$xl = 0.132(14\frac{1}{2} \text{ in.}) = 1.91 \text{ in.}$$

$$al = 8 \text{ in.} - xl = 8 \text{ in.} - 1.91 \text{ in.} = 6.09 \text{ in.}$$

$$a = 0.420$$

By interpolation

$$C = 3.55$$

LRFD	ASD
$D_{req} = \frac{P_{uc}}{\phi CC_1 l}$	$D_{req} = \frac{\Omega P_{ac}}{CC_1 l}$
$= \frac{240 \text{ kips}}{0.75(3.55)(1.0)(2 \text{ welds})(14\frac{1}{2} \text{ in.})}$	$= \frac{2.00(160 \text{ kips})}{(3.55)(1.0)(2 \text{ welds})(14\frac{1}{2} \text{ in.})}$
$= 3.11 \rightarrow 4 \text{ sixteenths}$	$= 3.11 \rightarrow 4 \text{ sixteenths}$
Minimum weld size is $\frac{1}{4}$ in. Use $\frac{1}{4}$ -in. fillet welds.	Minimum weld size is $\frac{1}{4}$ in. Use $\frac{1}{4}$ -in. fillet welds.

Manual  
Table 8-8

Table J2.4

Check beam web thickness (against weld size required for strength)

For two fillet welds

$$t_{\min} = \frac{6.19D}{F_u} = \frac{6.19(3.11 \text{ sixteenths})}{65 \text{ ksi}} = 0.296 \text{ in.} < 0.590 \text{ in.} \quad \mathbf{o.k.}$$

Check the strength of the angles

Shear yielding

LRFD	ASD
$\phi R_n = \phi(0.60F_y A_g)$ $= 1.0(0.60)(36 \text{ ksi})(2)(14 \frac{1}{2} \text{ in.})(\frac{3}{4} \text{ in.})$ $= 470 \text{ kips} > 168 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{0.60F_y A_g}{\Omega}$ $= \frac{0.60(36 \text{ ksi})(2)(14 \frac{1}{2} \text{ in.})(\frac{3}{4} \text{ in.})}{1.50}$ $= 313 \text{ kips} > 112 \text{ kips} \quad \mathbf{o.k.}$

Eqn. J4-3

Similarly, shear yielding of the angles due to  $H_{uc}$  is not critical.

Shear rupture

$$A_{nv} = (\frac{3}{4} \text{ in.})[2(14 \frac{1}{2} \text{ in.}) - 10(1 \text{ in.})] = 14.3 \text{ in.}^2$$

LRFD	ASD
$\phi R_n = \phi(0.60F_u A_{nv})$ $= 0.75(0.60)(58 \text{ ksi})(14.3 \text{ in.}^2)$ $= 372 \text{ kips} > 168 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{0.60F_u A_{nv}}{\Omega}$ $= \frac{0.60(58 \text{ ksi})(14.3 \text{ in.}^2)}{2.00}$ $= 248 \text{ kips} > 11 \text{ kips} \quad \mathbf{o.k.}$

Eqn. J4-4

Block shear rupture

With  $n = 5$ ,  $L_{ev} = 1 \frac{1}{4} \text{ in.}$ ,  $L_{eh} = 3 \frac{1}{4} \text{ in.}$

Section J4.3

$$A_{nt} = [(3 \frac{1}{4} \text{ in.}) - (\frac{1}{2})(1.0 \text{ in.})](\frac{3}{4} \text{ in.})(2) = 4.12 \text{ in.}^2$$

$$A_{gv} = (13 \frac{1}{4} \text{ in.})(\frac{3}{4} \text{ in.})(2) = 19.8 \text{ in.}^2$$

$$A_{nv} = [(13 \frac{1}{4} \text{ in.}) - (4 \frac{1}{2})(1 \text{ in.})](\frac{3}{4} \text{ in.})(2) = 13.1 \text{ in.}^2$$

$$F_u A_{nt} = (58 \text{ ksi})(4.12 \text{ in.}^2) = 239 \text{ kips}$$

$$0.60F_y A_{gv} = 0.60(36 \text{ ksi})(19.8 \text{ in.}^2) = 430 \text{ kips} \quad \mathbf{controls}$$

$$0.60F_u A_{nv} = 0.6(58 \text{ ksi})(13.1 \text{ in.}^2) = 457 \text{ kips}$$

LRFD	ASD
$\phi R_n = 0.75(239 \text{ kips} + 430 \text{ kips})$ $= 502 \text{ kips} > 168 \text{ kips}$	$R_n / \Omega = \frac{(239 \text{ kips} + 430 \text{ kips})}{2.00}$ $= 334 \text{ kips} > 112 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

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  $F_{cr} / \Omega_c$  may be determined from  $\frac{Kl_1}{r}$  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  $l = \frac{l_1 + l_2 + l_3}{3}$  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 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

$$A_w = (30.9 \text{ in.})\left(\frac{3}{4} \text{ in.}\right) + (3.90 \text{ in.})(0.590 \text{ in.})\left(\frac{50 \text{ ksi}}{36 \text{ ksi}}\right) = 26.4 \text{ in.}^2$$

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  $\frac{Kl_1}{r} = \frac{0.5(17.0 \text{ in.})(\sqrt{12})}{\frac{3}{4} \text{ in.}} = 39.3$

From Specification Section E3

$$F_e = \frac{\pi^2 E}{\left(\frac{Kl}{r}\right)^2} = \frac{\pi^2 (29000 \text{ ksi})}{39.3^2} = 185$$

Check  $4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29000 \text{ ksi}}{36 \text{ ksi}}} = 134$

The buckling stress is given by  $F_{cr} = \left[ 0.658^{\frac{F_y}{F_e}} \right] F_y = \left[ 0.658^{\frac{36 \text{ ksi}}{185 \text{ ksi}}} \right] (36 \text{ ksi}) = 33.2 \text{ ksi}$

The required stress is

LRFD	ASD
$f_{ua} = \frac{675 \text{ kips}}{26.4 \text{ in.}^2} = 25.6 \text{ ksi}$	$f_{aa} = \frac{450 \text{ kips}}{26.4 \text{ in.}^2} = 17.0 \text{ ksi}$
The available stress is	The available stress is
$\phi F_{cr} = 0.90(33.2 \text{ ksi})$	$F_{cr} / \Omega = \frac{33.2 \text{ ksi}}{1.67}$
$= 29.9 \text{ ksi} > 25.6 \text{ ksi}$	$= 19.9 \text{ ksi} > 17.0 \text{ ksi}$
<b>o.k.</b>	<b>o.k.</b>

### Example II.C-3 Bracing Connection

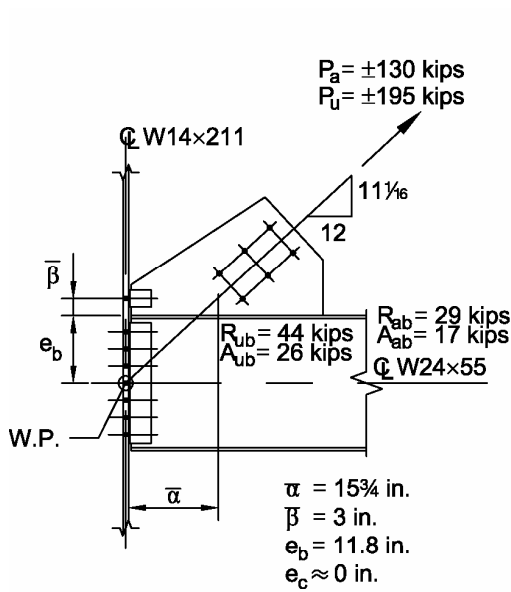
**Given:**

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.

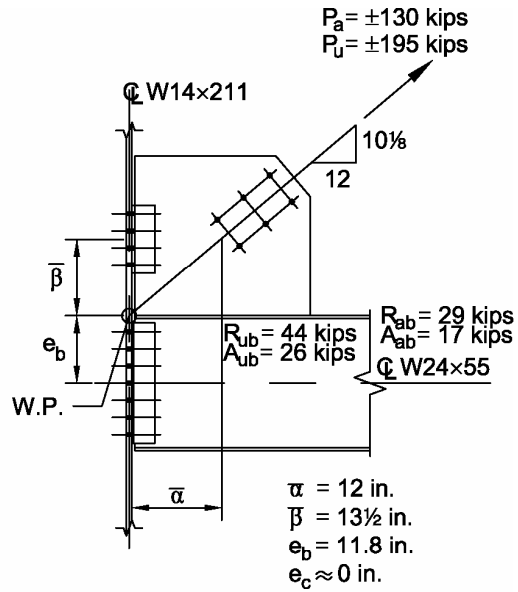
For the given values of  $\alpha$  and  $\beta$ , determine the interface forces on the gusset-to-column and gusset-to-beam connections for

- 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)

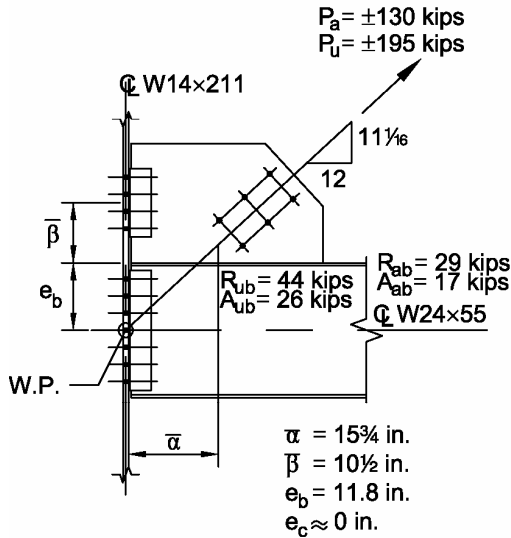
Brace Axial Load	$P_u = \pm 195$ kips	$P_a = \pm 130$ kips
Beam End Reaction	$R_u = 44$ kips	$R_a = 29$ kips
Beam Axial Load	$A_u = 26$ kips	$A_a = 17$ kips



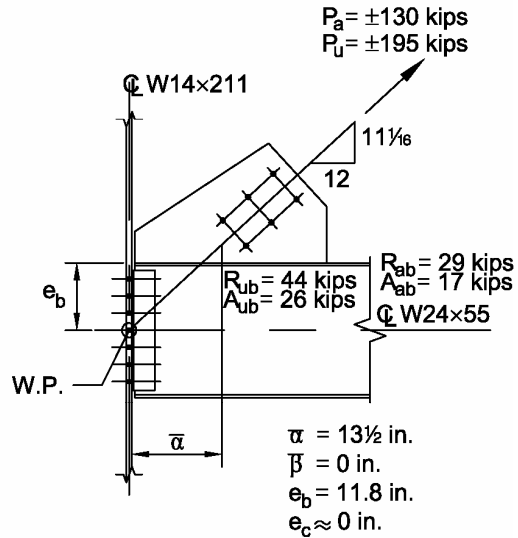
(a) General Case



(b) Special Case 1,  
Working Point at Gusset Corner



(c) Special Case 2,  $\Delta V_{ub} = V_{ubj.e.}$   
Shear in Beam-to-Column  
Connection Minimized



(d) Special Case 3,  
No Gusset-to-Column Web Connection

**Material Properties:**

Brace	W14x68	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Beam	W24x55	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Column	W14x211	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Gusset Plate		ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi

Manual  
Tables 2-3  
and 2-4

**Geometric Properties:**

Brace	W14×68	$A=20.0 \text{ in.}^2$	$d=14.0 \text{ in.}$	$t_w=0.415 \text{ in.}$	$b_f=10.0 \text{ in.}$	$t_f=0.720 \text{ in.}$
Beam	W24×55	$d=23.6 \text{ in.}$	$t_w=0.395 \text{ in.}$	$b_f=7.01 \text{ in.}$	$t_f=0.505 \text{ in.}$	$k=1.11$
Column	W14×211	$d=15.7 \text{ in.}$	$t_w=0.980 \text{ in.}$	$b_f=15.8 \text{ in.}$	$t_f=1.56 \text{ in.}$	

Manual  
Table 1-1

**Solution A (General Case):**

Assume  $\beta = \bar{\beta} = 3 \text{ in.}$

$$\alpha = e_b \tan \theta - e_c + \beta \tan \theta = (11.8 \text{ in.}) \left( \frac{12}{11\frac{1}{16}} \right) - 0 + (3 \text{ in.}) \left( \frac{12}{11\frac{1}{16}} \right) = 16.1 \text{ in.}$$

Since  $\alpha \neq \bar{\alpha}$ , an eccentricity exists on the gusset-to-beam connection.

Calculate the interface forces

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(16.1 \text{ in.} + 0 \text{ in.})^2 + (3 \text{ in.} + 11.8 \text{ in.})^2} = 21.9 \text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = \frac{\beta}{r} P_u = \frac{3 \text{ in.}}{21.9 \text{ in.}} (195 \text{ kips}) = 26.7 \text{ kips}$	$V_{ac} = \frac{\beta}{r} P_a = \frac{3 \text{ in.}}{21.9 \text{ in.}} (130 \text{ kips}) = 17.8 \text{ kips}$
$H_{uc} = \frac{e_c}{r} P_u = 0 \text{ kips}$	$H_{ac} = \frac{e_c}{r} P_a = 0 \text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \frac{\alpha}{r} P_u = \frac{16.1 \text{ in.}}{21.9 \text{ in.}} (195 \text{ kips}) = 143 \text{ kips}$	$H_{ab} = \frac{\alpha}{r} P_a = \frac{16.1 \text{ in.}}{21.9 \text{ in.}} (130 \text{ kips}) = 95.6 \text{ kips}$
$V_{ub} = \frac{e_b}{r} P_u = \frac{11.8 \text{ in.}}{21.9 \text{ in.}} (195 \text{ kips}) = 105 \text{ kips}$	$V_{ab} = \frac{e_b}{r} P_a = \frac{11.8 \text{ in.}}{21.9 \text{ in.}} (130 \text{ kips}) = 70.0 \text{ kips}$
$M_{ub} = V_{ub} (\alpha - \bar{\alpha})$	$M_{ab} = V_{ab} (\alpha - \bar{\alpha})$
$= \frac{(105 \text{ kips})(15\frac{3}{4} \text{ in.} - 16.1 \text{ in.})}{12 \text{ in./ft}}$	$= \frac{(70.0 \text{ kips})(15\frac{3}{4} \text{ in.} - 16.1 \text{ in.})}{12 \text{ in./ft}}$
$= -3.06 \text{ kip-ft}$	$= -2.04 \text{ kip-ft}$

In this case, this small moment is negligible.

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub} = 44 \text{ kips} + 105 \text{ kips} = 149 \text{ kips}$	$R_{ab} + V_{ab} = 29 \text{ kips} + 70 \text{ kips} = 99 \text{ kips}$



and the axial force is

LRFD	ASD
$A_{ub} + H_{uc} = 26 \text{ kips} \pm 0 \text{ kips} = 26 \text{ kips}$	$A_{ab} + H_{ac} = 17 \text{ kips} \pm 0 \text{ kips} = 17 \text{ kips}$

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.

The angle of the brace from the vertical is

$$\theta = \tan^{-1}\left(\frac{12}{10\frac{1}{8}}\right) = 49.8^\circ$$

The horizontal and vertical components of the brace force are

LRFD	ASD
$H_u = P_u \sin \theta = (195 \text{ kips}) \sin 49.8^\circ = 149 \text{ kips}$	$H_a = P_a \sin \theta = (130 \text{ kips}) \sin 49.8^\circ = 99.3 \text{ kips}$
$V_u = P_u \cos \theta = (195 \text{ kips}) \cos 49.8^\circ = 126 \text{ kips}$	$V_a = P_a \cos \theta = (130 \text{ kips}) \cos 49.8^\circ = 83.9 \text{ kips}$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = V_u = 126 \text{ kips}$	$V_{ac} = V_a = 83.9 \text{ kips}$
$H_{uc} = 0 \text{ kips}$	$H_{ac} = 0 \text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$V_{ub} = 0 \text{ kips}$	$V_{ab} = 0 \text{ kips}$
$H_{ub} = H_u = 149 \text{ kips}$	$H_{ab} = H_a = 99.3 \text{ kips}$

On the beam-to-column connection

LRFD	ASD
$R_{ub} = 44 \text{ kips (shear)}$	$R_{ab} = 29 \text{ kips (shear)}$
$A_{ub} = 26 \text{ kips (axial transfer force)}$	$A_{ab} = 17 \text{ kips (axial transfer force)}$

In addition to the forces on the connection interfaces, the beam is subjected to a moment  $M_{ub}$  or  $M_{ab}$ , where

LRFD	ASD
$M_{ub} = H_{ub}e_b = \frac{(149 \text{ kips})(11.8 \text{ in.})}{12 \text{ in./ft}}$	$M_{ab} = H_{ab}e_b = \frac{(99.3 \text{ kips})(11.8 \text{ in.})}{12 \text{ in./ft}}$
$= 147 \text{ kip-ft}$	$= 97.6 \text{ kip-ft}$

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  $\beta = \bar{\beta} = 10 \frac{1}{2}$  in.

$$\alpha = e_b \tan \theta - e_c + \beta \tan \theta = (11.8 \text{ in.}) \left( \frac{12}{11 \frac{1}{16}} \right) - 0 + (10 \frac{1}{2} \text{ in.}) \left( \frac{12}{11 \frac{1}{16}} \right) = 24.2 \text{ in.}$$

Calculate the interface forces for the general case before applying Special Case 2.

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(24.2 \text{ in.} + 0 \text{ in.})^2 + (10 \frac{1}{2} \text{ in.} + 11.8 \text{ in.})^2} = 32.9 \text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = \frac{\beta}{r} P_u = \frac{10 \frac{1}{2} \text{ in.}}{32.9 \text{ in.}} (195 \text{ kips}) = 62.2 \text{ kips}$	$V_{ac} = \frac{\beta}{r} P_a = \frac{10 \frac{1}{2} \text{ in.}}{32.9 \text{ in.}} (130 \text{ kips}) = 41.5 \text{ kips}$
$H_{uc} = \frac{e_c}{r} P_u = 0 \text{ kips}$	$H_{ac} = \frac{e_c}{r} P_a = 0 \text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \frac{\alpha}{r} P_u = \frac{24.2 \text{ in.}}{32.9 \text{ in.}} (195 \text{ kips}) = 143 \text{ kips}$	$H_{ab} = \frac{\alpha}{r} P_a = \frac{24.2 \text{ in.}}{32.9 \text{ in.}} (130 \text{ kips}) = 95.6 \text{ kips}$
$V_{ub} = \frac{e_b}{r} P_u = \frac{11.8 \text{ in.}}{32.9 \text{ in.}} (195 \text{ kips}) = 69.9 \text{ kips}$	$V_{ab} = \frac{e_b}{r} P_a = \frac{11.8 \text{ in.}}{32.9 \text{ in.}} (130 \text{ kips}) = 46.6 \text{ kips}$

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub} = 44 \text{ kips} + 69.9 \text{ kips} = 114 \text{ kips}$	$R_{ab} + V_{ab} = 29 \text{ kips} + 46.6 \text{ kips} = 75.6 \text{ kips}$

and the axial force is

LRFD	ASD
$A_{ub} + H_{uc} = 26 \text{ kips} \pm 0 \text{ kips} = 26 \text{ kips}$	$A_{ab} + H_{ac} = 17 \text{ kips} \pm 0 \text{ kips} = 17 \text{ kips}$

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
$V_{uc} = 62.2 \text{ kips} + 69.9 \text{ kips} = 132 \text{ kips}$	$V_{ac} = 41.5 \text{ kips} + 46.6 \text{ kips} = 88.1 \text{ kips}$
$H_{uc} = 0 \text{ kips}$ (unchanged)	$H_{ac} = 0 \text{ kips}$ (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)	$H_{ab} = 95.6 \text{ kips}$ (unchanged)
$V_{ub} = 69.9 \text{ kips} - 69.9 \text{ kips} = 0 \text{ kips}$	$V_{ab} = 46.6 \text{ kips} - 46.6 \text{ kips} = 0 \text{ kips}$
$M_{ub} = (\Delta V_{ub})\alpha = \frac{(69.9 \text{ kips})(24.2 \text{ in.})}{12\text{in./ft}}$	$M_{ab} = (\Delta V_{ab})\alpha = \frac{(46.6 \text{ kips})(24.2 \text{ in.})}{12\text{in./ft}}$
$= 141 \text{ kip-ft}$	$= 94.0 \text{ kip-ft}$

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + \Delta V_{ub} - \Delta V_{ub}$	$R_{ab} + \Delta V_{ab} - \Delta V_{ab}$
$= 44 \text{ kips} + 69.9 \text{ kips} - 69.9 \text{ kips}$	$= 29 \text{ kips} + 46.6 \text{ kips} - 46.6 \text{ kips}$
$= 44 \text{ kips}$	$= 29 \text{ kips}$

and the axial force is

LRFD	ASD
$A_{ub} + H_{uc} = 26 \text{ kips} \pm 0 \text{ kips} = 26 \text{ kips}$	$A_{ab} + H_{ac} = 17 \text{ kips} \pm 0 \text{ kips} = 17 \text{ kips}$

**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
$H_{ub} = \frac{\alpha}{r} P_u = \frac{12.8 \text{ in.}}{17.4 \text{ in.}} (195 \text{ kips}) = 143 \text{ kips}$	$H_{ab} = \frac{\alpha}{r} P_a = \frac{12.8 \text{ in.}}{17.4 \text{ in.}} (130 \text{ kips}) = 95.6 \text{ kips}$
$V_{ub} = \frac{e_b}{r} P_u = \frac{11.8 \text{ in.}}{17.4 \text{ in.}} (195 \text{ kips}) = 132 \text{ kips}$	$V_{ab} = \frac{e_b}{r} P_a = \frac{11.8 \text{ in.}}{17.4 \text{ in.}} (130 \text{ kips}) = 88.2 \text{ kips}$
$M_{ub} = V_{ub} (\alpha - \bar{\alpha})$	$M_{ab} = V_{ab} (\alpha - \bar{\alpha})$
$= \frac{(132 \text{ kips})(12.8 \text{ in.} - 13 \frac{1}{2} \text{ in.})}{12 \text{ in./ft}}$	$= \frac{(88.2 \text{ kips})(12.8 \text{ in.} - 13 \frac{1}{2} \text{ in.})}{12 \text{ in./ft}}$
$= -7.70 \text{ kip-ft}$	$= -5.15 \text{ kip-ft}$

In this case, this small moment is negligible.

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub} = 44 \text{ kips} + 132 \text{ kips} = 176 \text{ kips}$	$R_{ab} + V_{ab} = 29 \text{ kips} + 88.2 \text{ kips} = 117 \text{ kips}$

And the axial force is

LRFD	ASD
$A_{ub} + H_{uc} = 26 \text{ kips} \pm 0 \text{ kips} = 26 \text{ kips}$	$A_{ab} + H_{ac} = 17 \text{ kips} \pm 0 \text{ kips} = 17 \text{ kips}$

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:

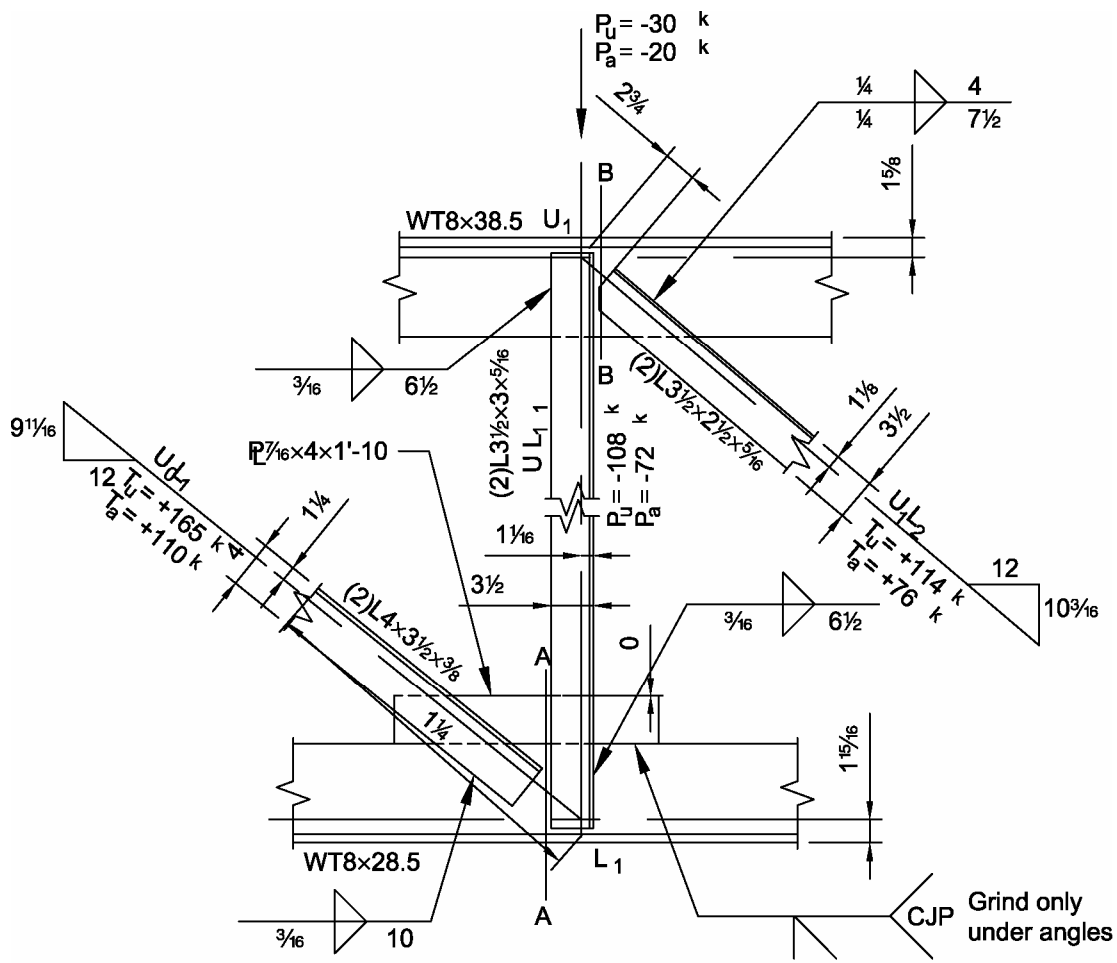
Determine the requirements for the following cases:

- a. Joint  $L_1$
- b. Joint  $U_1$

$R_D = 18.5$  kips

$R_L = 55.5$  kips

Use E70 electrodes.



**Material Properties:**

Top Chord	WT8x38.5	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Bottom Chord	WT8x28.5	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi
Diagonal $U_0L_1$	2L4x3½x¾	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi
Web $U_1L_1$	2L3½x3x⅝	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi
Diagonal $U_1L_2$	2L3½x2½x⅝	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.	
Bottom Chord	WT8×28.5	$t_w = 0.430$ in.	$d = 8.22$ in.
Diagonal $U_0L_1$	2L4×3½×¾	$A = 5.35$ in. <sup>2</sup>	$\bar{y} = 1.20$ in.
Web $U_1L_1$	2L3½×3×⅝	$A = 3.91$ in. <sup>2</sup>	
Diagonal $U_1L_2$	2L3½×2½×⅝	$A = 3.58$ in. <sup>2</sup>	

Manual  
Tables 1-8  
and 1-15

LRFD	ASD
Web $U_1L_1$ load	Web $U_1L_1$ load
$R_u = -108$ kips	$R_a = -72$ kips
Diagonal $U_0L_1$ load	Diagonal $U_0L_1$ load
$R_u = +165$ kips	$R_a = +110$ kips
Diagonal $U_1L_2$ load	Diagonal $U_1L_2$ load
$R_u = +114$ kips	$R_a = +76$ 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
$\phi R_n = 1.00(106 \text{ kips}) = 106 \text{ kips}$	$R_n / \Omega = \frac{106 \text{ kips}}{1.50} = 70.7 \text{ kips}$
106 kips < 108 kips <b>n.g.</b>	70.7 kips < 72 kips <b>n.g.</b>

Additional shear area must be provided.

Try PL¾ in.×4 in. complete-joint-penetration groove welded to the stem of the WT.

$$R_n = 0.6(36 \text{ ksi})(4 \text{ in.})(¾ \text{ in.}) = 37.8 \text{ kips}$$

LRFD	ASD
$\phi R_n = 106 \text{ kips} + 1.00(37.8 \text{ kips}) = 144 \text{ kips}$	$R_n / \Omega = 70.7 \text{ kips} + \frac{37.8 \text{ kips}}{1.50} = 95.9 \text{ kips}$
144 kips > 108 kips <b>o.k.</b>	95.9 kips > 72 kips <b>o.k.</b>

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  $w_{\max} = \text{thickness} - \frac{1}{16}$  in. =  $\frac{1}{4}$  in.

Calculate the minimum length of  $\frac{3}{16}$ -in. fillet weld.

LRFD	ASD
$L_{\min} = \frac{R_u}{1.392D} = \frac{108 \text{ kips}}{1.392(3 \text{ sixteenths})}$ $= 25.9 \text{ in.}$	$L_{\min} = \frac{R_a}{0.928D} = \frac{72 \text{ kips}}{0.928(3 \text{ sixteenths})}$ $= 25.9 \text{ in.}$

Use  $6\frac{1}{2}$  in. of  $\frac{3}{16}$ -in. weld at the heel and toe of both angles for a total of 26 in.

*Design welds for member  $U_0L_1$*

Table J2.4

The minimum weld size is  $w_{\min} = \frac{3}{16}$  in.

The maximum weld size is  $w_{\max} = \text{thickness} - \frac{1}{16}$  in. =  $\frac{5}{16}$  in.

Calculate the minimum length of  $\frac{3}{16}$  in. fillet weld.

LRFD	ASD
$L_{\min} = \frac{R_u}{1.392D} = \frac{165 \text{ kips}}{1.392(3 \text{ sixteenths})}$ $= 39.5 \text{ in.}$	$L_{\min} = \frac{R_a}{0.928D} = \frac{110 \text{ kips}}{0.928(3 \text{ sixteenths})}$ $= 39.5 \text{ in.}$

Use 10 in. of  $\frac{3}{16}$  in. weld at the heel and toe of both angles for a total of 40 in.

*Check tension yielding of diagonal  $U_0L_1$*

$$R_n = F_y A_g = (36 \text{ ksi})(5.35 \text{ in.}^2) = 193 \text{ kips}$$

Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.90(193 \text{ kips}) = 173 \text{ kips}$ $173 \text{ kips} > 165 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{193 \text{ kips}}{1.67} = 115 \text{ kips}$ $115 \text{ kips} > 110 \text{ kips} \quad \mathbf{o.k.}$

*Check tension rupture of diagonal  $U_0L_1$*

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.20 \text{ in.}}{10 \text{ in.}} = 0.880$$

Table D3.1  
Case 2

$$R_n = F_u A_e = (58 \text{ ksi})(0.880)(5.35 \text{ in.}^2) = 273 \text{ kips}$$

Eqn. J4-2

LRFD	ASD
$\phi R_n = 0.75(273 \text{ kips}) = 205 \text{ kips}$ $205 \text{ kips} > 165 \text{ kips} \quad \mathbf{o.k.}$	$R_n / \Omega = \frac{273 \text{ kips}}{2.00} = 137 \text{ kips}$ $137 \text{ kips} > 110 \text{ kips} \quad \mathbf{o.k.}$

**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
$\phi R_n = 1.00(113 \text{ kips}) = 113 \text{ kips}$ $113 \text{ kips} > 78 \text{ kips}$	$R_n / \Omega = \frac{113 \text{ kips}}{1.50} = 75.3 \text{ kips}$ $75.3 \text{ kips} > 52 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Design welds for member  $U_1L_1$

As calculated previously in Solution a, use  $6\frac{1}{2}$  in. of  $\frac{3}{16}$ -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.

The maximum weld size is  $w_{\max} = \frac{1}{4}$  in.

Table J2.4

Calculate the minimum length of  $\frac{1}{4}$  in. fillet weld.

LRFD	ASD
$L_{\min} = \frac{R_u}{1.392D} = \frac{114 \text{ kips}}{1.392(4 \text{ sixteenths})}$ $= 20.5 \text{ in.}$	$L_{\min} = \frac{R_a}{0.928D} = \frac{76 \text{ kips}}{0.928(4 \text{ sixteenths})}$ $= 20.5 \text{ in.}$

Use  $7\frac{1}{2}$  in. of  $\frac{1}{4}$ -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 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
$\phi R_n = 0.90(129 \text{ kips}) = 116 \text{ kips}$ $116 \text{ kips} > 114 \text{ kips}$	$R_n / \Omega = \frac{129 \text{ kips}}{1.67} = 77.2 \text{ kips}$ $77.2 \text{ kips} > 76 \text{ kips}$
<b>o.k.</b>	<b>o.k.</b>

Check tension rupture of diagonal  $U_1L_2$

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.13 \text{ in.}}{\left(\frac{7\frac{1}{2} \text{ in.} + 4 \text{ in.}}{2}\right)} = 0.803$$

Table D3.1  
Case 2

$$R_n = F_u A_e = (58 \text{ ksi})(0.803)(3.58 \text{ in.}^2) = 167 \text{ kips}$$

Eqn. J4-2



LRFD	ASD
$\phi R_n = 0.75(167 \text{ kips}) = 125 \text{ kips}$  125 kips > 114 kips <b>o.k.</b>	$R_n / \Omega = \frac{167 \text{ kips}}{2.00} = 83.4 \text{ kips}$  83.4 kips > 76 kips <b>o.k.</b>

*Check block shear rupture*

Because the cut end of the angle, the block shear rupture model presented in Part 9 does not directly apply. Conservatively, the block shear rupture strength will be based on the shear rupture strength of the WT stem along the length of the welds.

$$R_n = 0.6F_u A_w = 0.6(65 \text{ ksi})(7 \frac{1}{2} \text{ in.} + 4 \text{ in.})(0.455 \text{ in.}) = 204 \text{ kips}$$

Eqn. J4-4

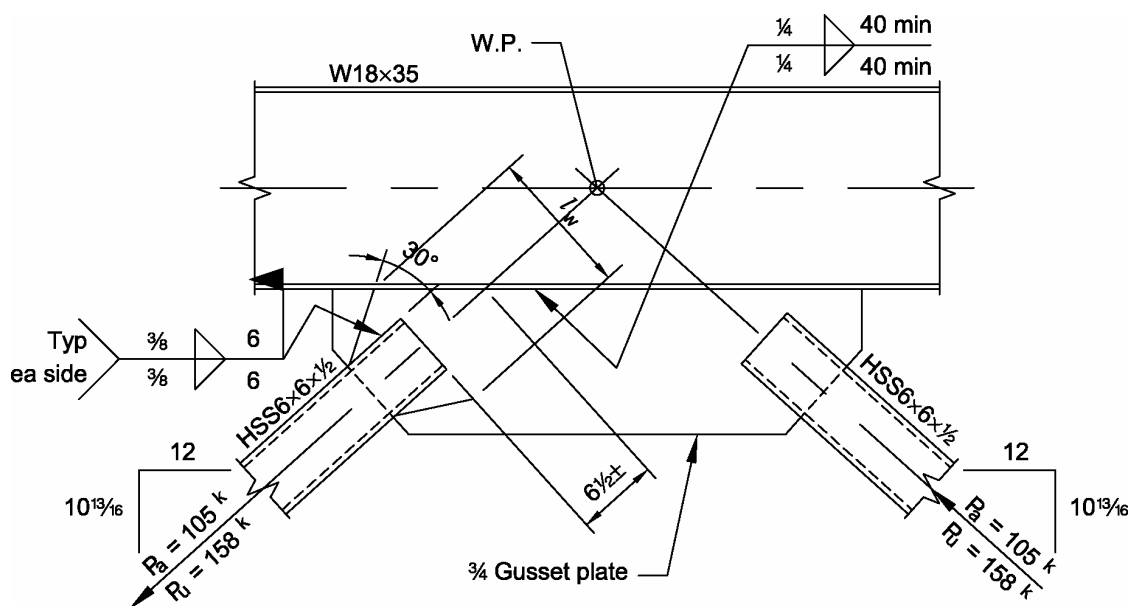
LRFD	ASD
$\phi R_n = 0.75(204 \text{ kips}) = 153 \text{ kips}$  153 kips > 114 kips <b>o.k.</b>	$R_n / \Omega = \frac{204 \text{ kips}}{2.00} = 102 \text{ kips}$  102 kips > 76 kips <b>o.k.</b>

### Example II.C-5 HSS Chevron Brace Connection

**Given:**

The loads shown are actual loads from the bottom chevron from the example problem at the back of the book. The beam above has been design to carry its load with out the chevron, and the end connections have been design to all vertical and horizontal loads required. 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 W18x35	$F_y = 50$ ksi	$F_u = 65$ ksi
Brace HSS6x6x1/2	$F_y = 46$ ksi	$F_u = 58$ ksi
Gusset Plate	$F_y = 36$ ksi	$F_u = 58$ ksi

Manual  
Tables 2-3  
and 2-4

**Geometric Properties:**

Beam W18x35	$d = 17.7$ in.	$t_w = 0.300$ in.	$k_{des.} = 0.827$ in.
Brace HSS6x6x1/2	$H = 6$ in.	$B = 6$ in.	$A = 9.74$ in. $t = 0.465$ in.

Manual  
Tables 1-1  
and 1-12

**Solution:**

Determine required brace-to-gusset weld size

Since the brace loads are axial, the angle between the longitudinal brace axis and line of force is  $\theta_w = 0^\circ$ .

$$F_w = 0.60F_{EXX} (1 + 0.5 \sin^{1.5} \theta_w) = 0.60(70 \text{ ksi})(1 + 0.5 \sin^{1.5} 0^\circ) = 42 \text{ ksi}$$

LRFD	ASD
$W_{wreq'd} = \frac{P_u}{\phi 4F_w (0.707) L_w} + \frac{1}{16} \text{ in.}$ $= \frac{158 \text{ kips}}{0.75(4)(42 \text{ ksi})(0.707)(6 \text{ in.})} + \frac{1}{16} \text{ in.}$ $= 0.296 \text{ in.} + \frac{1}{16} \text{ in.} = 0.358 \text{ in.}$ <p>Use <math>\frac{3}{8}</math>-in. fillet weld</p>	$W_{wreq'd} = \frac{P_d \Omega}{4F_w (0.707) L_w} + \frac{1}{16} \text{ in.}$ $= \frac{(105 \text{ kips})(2.00)}{(4)(42 \text{ ksi})(0.707)(6 \text{ in.})} + \frac{1}{16} \text{ in.}$ $= 0.295 \text{ in.}$ <p>Use <math>\frac{3}{8}</math>-in. fillet weld</p>

Note: the  $\frac{1}{16}$ -in. added to the weld size is to account for the slot in HSS

The minimum weld size for this connection is  $\frac{3}{16}$  in. The required weld size is larger therefore, use  $\frac{3}{8}$ -in. fillet welds.

Table J2.4

Determine required gusset plate thickness

$$W_e = W_w - \frac{3}{8} \text{ in.} - \frac{1}{16} \text{ in.} = \frac{5}{16} \text{ in.}$$

LRFD	ASD
$t_{1req'd} = \frac{\phi(0.60F_{EXX} W_e)(0.707)(2)}{\phi(0.60F_{y1})}$ $= \frac{0.75(0.60)(70 \text{ ksi})(\frac{1}{4} \text{ in.})(0.707)(2)}{1.00(0.60)(36 \text{ ksi})}$ $= 0.644 \text{ in.}$	$t_{1req'd} = \frac{\Omega(0.60F_{EXX} W_e)(0.707)(2)}{\Omega(0.60F_{y1})}$ $= \frac{(1.50)(0.60)(70 \text{ ksi})(\frac{1}{4} \text{ in.})(0.707)(2)}{(2.00)(0.60)(36 \text{ ksi})}$ $= 0.644 \text{ in.}$

Use a  $\frac{3}{4}$ -in. gusset plate.

Check gusset plate buckling (compression brace)

$$r = \frac{t_1}{\sqrt{12}} = \frac{\frac{3}{4} \text{ in.}}{\sqrt{12}} = 0.217 \text{ in.}$$

From the figure, the distance  $l_1 = 6\frac{1}{2}$  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  $K = 1.2$ .

$$\frac{Kl_1}{r} = \frac{1.2(6\frac{1}{2} \text{ in.})}{0.217 \text{ in.}} = 36.0$$

$$\text{Limiting slenderness ratio } 4.71 \sqrt{\frac{E}{F_y}} = 4.71 \sqrt{\frac{29000 \text{ ksi}}{36 \text{ ksi}}} = 134 > 36.0$$

$$F_e = \frac{\pi^2 E}{\left(\frac{Kl_1}{r}\right)^2} = \frac{\pi^2 (29000 \text{ ksi})}{(36.0)^2} = 221 \text{ ksi}$$

Eqn. E3-4

$$F_{cr} = \left[ 0.658 \frac{F_y}{F_e} \right] F_y = \left[ 0.658 \frac{36 \text{ ksi}}{221 \text{ ksi}} \right] (36 \text{ ksi}) = 33.6 \text{ ksi}$$

Eqn. E3-2

$$l_w = B + 2[(\text{connection length}) \tan 30^\circ] = 6 \text{ in.} + 2(6 \text{ in.}) \tan 30^\circ = 12.9 \text{ in.}$$

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.

Eqn. E3-1

$$A_w = l_w t_1 = (12.9 \text{ in.}) \left(\frac{3}{4} \text{ in.}\right) = 9.675 \text{ in.}^2$$

$$P_n = F_{cr} A_w = (33.6 \text{ ksi}) (9.675 \text{ in.}^2) = 325 \text{ kips}$$

LRFD	ASD
$\phi P_n = 0.90(325 \text{ kips}) = 293 \text{ kips}$ 293 kips > 158 kips <b>o.k.</b>	$P_n / \Omega = \frac{325 \text{ kips}}{1.67} = 195 \text{ kips}$ 204 kips > 105 kips <b>o.k.</b>

Check tension yielding of gusset plate (tension brace)

From above,  $A_w = 8.06 \text{ in.}^2$

$$R_n = F_{y1} A_w = (36 \text{ ksi}) (8.06 \text{ in.}^2) = 348 \text{ kips}$$

Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.90(348 \text{ kips}) = 313 \text{ kips}$ 313 kips > 158 kips <b>o.k.</b>	$R_n / \Omega = \frac{348 \text{ kips}}{1.67} = 208 \text{ kips}$ 208 kips > 105 kips <b>o.k.</b>

Check shear strength at brace-to-gusset welds

Try minimum weld length,  $L_w = 6 \text{ in.}$

Table D3.1  
Case 6

$$\text{Effective Area, } A_e = 4L_w t = 4(6 \text{ in.})(0.465 \text{ in.}) = 11.2 \text{ in.}^2$$

$$\text{Nominal Shear Strength, } V_n = 0.6F_y A_e = 0.6(46 \text{ ksi})(11.2 \text{ in.}^2) = 309 \text{ kips}$$

Eqn. J4-3

LRFD	ASD
$\phi R_n = 1.00(309 \text{ kips}) = 309 \text{ kips}$ 309 kips > 158 kips <b>o.k.</b>	$R_n / \Omega = \frac{309 \text{ kips}}{1.50} = 206 \text{ kips}$ 206 kips > 105 kips <b>o.k.</b>

Check shear lag fracture in HSS brace

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{(6 \text{ in.})^2 + 2(6 \text{ in.})(6 \text{ in.})}{4(6 \text{ in.} + 6 \text{ in.})} = 2.25 \text{ in.}$$

$$U = 1 - \frac{\bar{x}}{L_w} = 1 - \frac{2.25 \text{ in.}}{6 \text{ in.}} = 0.625 \text{ in.}$$

$$A_n = A_g - 2t_1 = 9.74 - 2(0.9)(0.465 \text{ in.})\left(\frac{3}{4} \text{ in.} + \frac{1}{8} \text{ in. gap}\right) = 9.01 \text{ in.}^2$$

$$A_e = UA_n = 0.625(9.16 \text{ in.}^2) = 5.72 \text{ in.}^2$$

$$R_n = F_u A_e = (58 \text{ ksi})(5.63 \text{ in.}) = 327 \text{ kips}$$

Table D3.1  
Case 6

Eqn. J4-4

LRFD	ASD
$\phi R_n = 0.75(327 \text{ kips}) = 245 \text{ kips}$	$R_n / \Omega = \frac{327 \text{ kips}}{2.00} = 164 \text{ kips}$
245 kips > 158 kips <b>o.k.</b>	164 kips > 105 kips <b>o.k.</b>

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.

$$e_b = \frac{d}{2} = \frac{17.7 \text{ in.}}{2} = 8.85 \text{ in.}$$

$$\theta = \tan^{-1}\left(\frac{12}{10^{13/16}}\right) = 48^\circ$$

Let  $\bar{\alpha} = \alpha = e_b \tan \theta = (8.85 \text{ in.}) \tan 48^\circ = 9.83 \text{ in.} \rightarrow$  Use 10 in.

$$\beta = e_c = 0$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(10 \text{ in.} + 0)^2 + (0 + 8.85 \text{ in.})^2} = 13.4 \text{ in.}$$

LRFD	ASD
$H_{ub} = \frac{\alpha P_u}{r} = \frac{(10 \text{ in.})(158 \text{ kips})}{13.4 \text{ in.}} = 118 \text{ kips}$	$H_{ab} = \frac{\alpha P_a}{r} = \frac{(10 \text{ in.})(105 \text{ kips})}{13.4 \text{ in.}} = 78.6 \text{ kips}$
$V_{ub} = \frac{e_b P_u}{r} = \frac{(8.85 \text{ in.})(158 \text{ kips})}{13.4 \text{ in.}} = 105 \text{ kips}$	$V_{ab} = \frac{e_b P_a}{r} = \frac{(8.85 \text{ in.})(105 \text{ kips})}{13.4 \text{ in.}} = 69.6 \text{ kips}$

*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,  $\alpha$ , for each brace. Therefore,  $l = 2\alpha = 2(10 \text{ in.}) = 20 \text{ in.}$

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.

LRFD	ASD
$D_{req'd} = \frac{1.25P_u}{1.392l} = \frac{1.25(158 \text{ kips})}{1.392(20 \text{ in.})(2)} = 3.55$	$D_{req'd} = \frac{1.25P_a}{0.928l} = \frac{1.25(105 \text{ kips})}{0.928(20 \text{ in.})(2)} = 3.54$

The minimum fillet weld size is  $\frac{1}{4}$  in. The required weld size is also  $\frac{1}{4}$  in., use a  $\frac{1}{4}$  in. fillet weld 40-in. long total.

Table J2.4

*Check gusset thickness (against weld size required for strength)*

$$t_{min} = \frac{6.19D}{F_u} = \frac{6.19(3.55)}{58 \text{ ksi}} = 0.379 \text{ in.} < \frac{5}{8} \text{ in.} \quad \mathbf{o.k.}$$

*Check local web yielding of the beam*

$$R_n = (N + 5k) F_y t_w = [20 \text{ in.} + 5(0.827 \text{ in.})](50 \text{ ksi})(0.300 \text{ in.}) = 362 \text{ kips}$$

Eqn. J10-2

LRFD	ASD
$\phi R_n = 1.00(662 \text{ kips}) = 662 \text{ kips}$	$R_n / \Omega = \frac{662 \text{ kips}}{1.50} = 441 \text{ kips}$
158 kips(cos 48°) = 105 kips	105 kips(cos 48°) = 70.2 kips
662 kips > 105 kips <b>o.k.</b>	441 kips > 70.2 kips <b>o.k.</b>

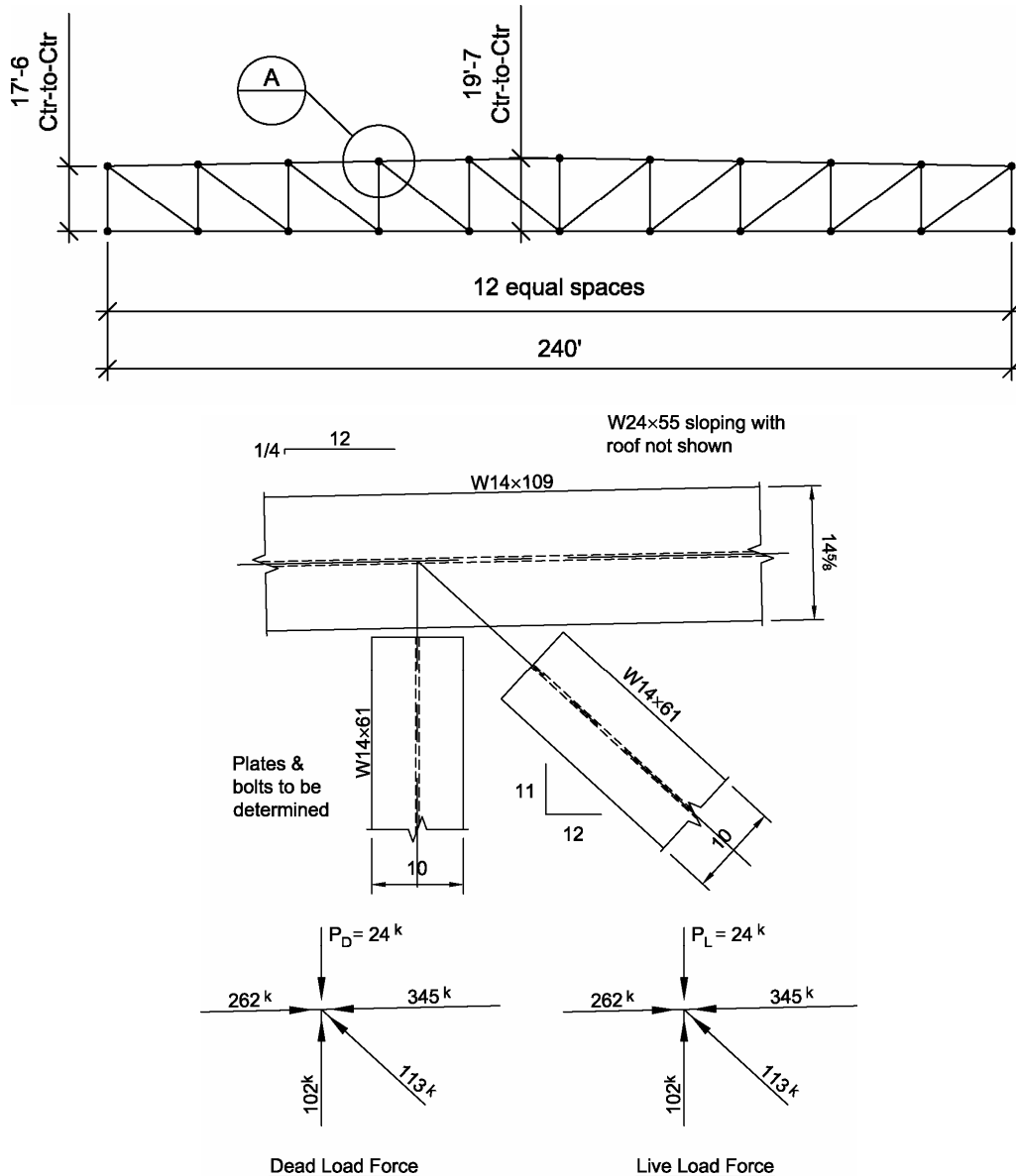
### Example II.C-6 Heavy Wide Flange Compression Connection (flanges on the outside)

**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
Gusset Plates	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi

Manual  
Tables 2-3  
and 2-4

**Geometric Properties:**

W14×109	$d = 14.3$ in.	$b_f = 14.6$ in.	$t_f = 0.860$ in.
W14×61	$d = 13.9$ in.	$b_f = 9.99$ in.	$t_f = 0.645$ in.

Manual  
Table 1-1

**Solution:**

*Determine the required strength of the member*

LRFD	ASD
Left top chord: $1.2(262 \text{ kips}) + 1.6(262 \text{ kips}) = 734 \text{ kips}$	Left top chord: $262 \text{ kips} + 262 \text{ kips} = 524 \text{ kips}$
Right top chord: $1.2(345 \text{ kips}) + 1.6(345 \text{ kips}) = 966 \text{ kips}$	Right top chord: $345 \text{ kips} + 345 \text{ kips} = 690 \text{ kips}$
Vertical: $1.2(102 \text{ kips}) + 1.6(102 \text{ kips}) = 286 \text{ kips}$	Vertical: $102 \text{ kips} + 102 \text{ kips} = 204 \text{ kips}$
Diagonal: $1.2(113 \text{ kips}) + 1.6(113 \text{ kips}) = 316 \text{ kips}$	Diagonal: $113 \text{ kips} + 113 \text{ kips} = 226 \text{ kips}$

*Determine the single shear strength*

$d_{\text{bolt}} = 1$  in.  
 ASTM A325-SC bolts  
 Class A faying surface  
 $\mu = 0.35$  (class A)  
 $d_{\text{hg}} = 1.25$  in. (diameter of holes at gusset plates)  
 $h_{\text{sc}} = 0.85$  (oversized and short-slotted holes)  
 $T_b = 51.0$  kips  
 $D_u = 1.13$

Section J3.8

Table J3.1

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.

Eqn. J3-4

$$R_n = \mu D_u h_{\text{sc}} T_b N_s = 0.35(1.13)(0.85)(51.0 \text{ kips})(1) = 17.1 \text{ kips}$$

LRFD	ASD
For connections designed to prevent slip at the required strength level;	For connections designed to prevent slip at the required strength level;
$\phi = 0.85$	$\Omega = 1.76$
$\phi R_n = 0.85(17.1 \text{ kips}) = 14.6 \text{ kips/bolt}$	$R_n / \Omega = 17.1 \text{ kips} / 1.76 = 9.74 \text{ kips/bolt}$

Section J3.8



*Diagonal connection*

LRFD	ASD
Axial force = 316 kips	Axial force = 226 kips
316 kips / 14.6 kips/bolt = 21.7 bolts	226 kips / 9.74 kips/bolt = 23.2 bolts
2 rows both sides = 21.7 bolts / 4 = 5.42	2 rows both sides = 23.2 bolts / 4 = 5.80
Therefore use 6 rows @ min. 3 in. spacing	Therefore use 6 rows @ min. 3 in. spacing

Check the Whitmore section in the gusset plate (tension only)

Whitmore section = gage of the bolts +  $\tan 30^\circ$  (length of bolt group)  $\times 2$

$$= (5 \frac{1}{2} \text{ in.}) + \tan 30^\circ [(5 \text{ spaces})(3 \text{ in.})] \times 2$$

$$= 22.8 \text{ in.}$$

Try  $\frac{3}{8}$  in. thick plate

$$A_g = (0.375 \text{ in.})(22.8 \text{ in.}) = 8.56 \text{ in}^2$$

Manual  
Part 9

LRFD	ASD
<p><i>Check tension yielding</i></p> <p><math>\phi = 0.90</math></p> <p><math>\phi R_n = \phi F_y A_g</math></p> <p><math>= 0.90(36 \text{ ksi})(8.56 \text{ in}^2)(2)</math></p> <p><math>= 555 \text{ kips} &gt; 316 \text{ kips}</math>      <b>o.k.</b></p>	<p><i>Check shear yielding</i></p> <p><math>\Omega = 1.67</math></p> <p><math>R_n / \Omega = F_y A_g / \Omega</math></p> <p><math>= (36 \text{ ksi})(8.56 \text{ in}^2)(2) / 1.67</math></p> <p><math>= 369 \text{ kips} &gt; 226 \text{ kips}</math>      <b>o.k.</b></p>
<p><i>Check block shear rupture in plate</i></p> <p><math>\phi = 0.75</math></p> <p>Tension stress is uniform,</p> <p>Therefore; <math>U_{bs} = 1.0</math></p> <p><math>t_p = 0.375 \text{ in.}</math></p> <p><math>A_{gv} / \text{in.} = (2 \text{ rows})[(6 \text{ bolts}-1)(3 \text{ in.}) + 2 \text{ in.}]</math> <math>= 34.0 \text{ in.}</math></p> <p><math>A_{nv} / \text{in.} = 34.0 \text{ in.} - (2 \text{ rows})(6 \text{ bolts} - 0.5)</math> <math>\times (1.25 \text{ in.} + 0.0625 \text{ in.}) = 19.6 \text{ in}</math></p> <p><math>A_{nt} / \text{in.} = 5.50 \text{ in.} - (1.25 \text{ in.} + 0.625 \text{ in.})</math> <math>= 4.19 \text{ in.}</math></p>	<p><i>Check block shear rupture in plate</i></p> <p><math>\Omega = 2.00</math></p> <p>Tension stress is uniform,</p> <p>Therefore; <math>U_{bs} = 1.0</math></p> <p>Try Plate with, <math>t_p = 0.375 \text{ in.}</math></p> <p><math>A_{gv} / \text{in.} = (2 \text{ rows})[(6 \text{ bolts}-1)(3 \text{ in.}) + 2 \text{ in.}]</math> <math>= 34.0 \text{ in.}</math></p> <p><math>A_{nv} / \text{in.} = 34.0 \text{ in.} - (2 \text{ rows})(6 \text{ bolts} - 0.5)</math> <math>\times (1.25 \text{ in.} + 0.0625 \text{ in.}) = 19.6 \text{ in}</math></p> <p><math>A_{nt} / \text{in.} = 5.50 \text{ in.} - (1.25 \text{ in.} + 0.625 \text{ in.})</math> <math>= 4.19 \text{ in.}</math></p>

Eqn. J4-1

Section J4.3

<p> <math display="block">\phi R_n = \phi(0.6F_u A_{nv} + U_{bs}F_u A_{nt}) \leq</math> <math display="block">\phi(0.6F_y A_{gv} + U_{bs}F_u A_{nt})</math> <p>Tension Rupture Component</p> <p> <math display="block">\phi U_{bs}(F_u A_{nt}) / t = 182 \text{ kips/in.}</math> </p> <p>Shear Yielding Component</p> <p> <math display="block">\phi(0.6F_y A_{gv}) / t = 551 \text{ kips/in.}</math> </p> <p>Shear Rupture Component</p> <p> <math display="block">\phi(0.6F_u A_{nv}) / t = 511 \text{ kips/in.}</math> </p> <p> <math display="block">\phi R_n = (511 \text{ kips/in.} + 182 \text{ kips/in.})(0.375 \text{ in.})</math> <math display="block">= 260 \text{ kips}</math> <math display="block">\leq (551 \text{ kips/in.} + 182 \text{ kips/in.})(0.355 \text{ in.})</math> <math display="block">= 275 \text{ kips}</math> <math display="block">= 260 \text{ kips} &lt; 275 \text{ kips}</math> <math display="block">260 \text{ kips} &gt; 316 \text{ kips} / 2 = 158 \text{ kips} \text{ o.k.}</math> <p><i>Check block shear rupture on beam flange</i> By inspection, block shear rupture on the beam flange will not control.</p> <p><i>Bolt bearing on plate</i></p> <p>Based on Bolt Spacing = 3 in.; oversized holes, <math>F_u = 58 \text{ ksi}</math></p> <p> <math display="block">\phi r_n = (91.4 \text{ kips/in.})(0.375 \text{ in.})</math> <math display="block">= 34.3 \text{ kips} &gt; 14.6 \text{ kips}</math> </p> <p>Based on Edge Distance = 2 in.; oversized holes, <math>F_u = 58 \text{ ksi}</math></p> <p> <math display="block">\phi r_n = (71.8 \text{ kips/in.})(0.375 \text{ in.})</math> <math display="block">= 26.9 \text{ kips} &gt; 14.6 \text{ kips}</math> </p> <p><i>Bolt bearing on flange</i></p> <p>Based on Bolt Spacing = 3 in.; standard holes, <math>F_u = 65 \text{ ksi}</math></p> <p> <math display="block">\phi r_n = (113 \text{ kips/in.})(0.645 \text{ in.})</math> <math display="block">= 72.9 \text{ kips} &gt; 14.6 \text{ kips}</math> </p> <p>Based on Edge Distance = 2 in.; standard holes, <math>F_u = 65 \text{ ksi}</math></p> </p></p>	<p> <math display="block">R_n / \Omega = (0.6F_u A_{nv} + U_{bs}F_u A_{nt}) / \Omega \leq</math> <math display="block">(0.6F_y A_{gv} + U_{bs}F_u A_{nt}) / \Omega</math> <p>Tension Rupture Component</p> <p> <math display="block">U_{bs}(F_u A_{nt}) / (t\Omega) = 121 \text{ kips/in.}</math> </p> <p>Shear Yielding Component</p> <p> <math display="block">(0.6F_y A_{gv}) / (t\Omega) = 367 \text{ kips/in.}</math> </p> <p>Shear Rupture Component</p> <p> <math display="block">(0.6F_u A_{nv}) / (t\Omega) = 340 \text{ kips/in.}</math> </p> <p> <math display="block">R_n / \Omega = (340 \text{ kips/in.} + 121 \text{ kips/in.})(0.355 \text{ in.})</math> <math display="block">= 173 \text{ kips}</math> <math display="block">\leq (367 \text{ kips/in.} + 121 \text{ kips/in.})(0.355 \text{ in.})</math> <math display="block">= 183 \text{ kips}</math> <math display="block">= 173 \text{ kips} &lt; 183 \text{ kips}</math> <math display="block">173 \text{ kips} &gt; 226 \text{ kips} / 2 = 113 \text{ kips} \text{ o.k.}</math> <p><i>Check block shear rupture on beam flange</i> By inspection, block shear rupture on the beam flange will not control.</p> <p><i>Bolt bearing on plate</i></p> <p>Based on Bolt Spacing = 3 in.; oversized holes, <math>F_u = 58 \text{ ksi}</math></p> <p> <math display="block">r_n / \Omega = (60.9 \text{ kips/in.})(0.375 \text{ in.})</math> <math display="block">= 22.8 \text{ kips} &gt; 9.74 \text{ kips}</math> </p> <p>Based on Edge Distance = 2 in.; oversized holes, <math>F_u = 58 \text{ ksi}</math></p> <p> <math display="block">r_n / \Omega = (47.9 \text{ kips/in.})(0.375 \text{ in.})</math> <math display="block">= 18.0 \text{ kips} &gt; 9.74 \text{ kips}</math> </p> <p><i>Bolt bearing on flange</i></p> <p>Based on Bolt Spacing = 3 in.; standard holes, <math>F_u = 65 \text{ ksi}</math></p> <p> <math display="block">r_n / \Omega = (75.6 \text{ kips/in.})(0.645)</math> <math display="block">= 48.8 \text{ kips} &gt; 9.74 \text{ kips}</math> </p> <p>Based on Edge Distance = 2 in.; standard holes, <math>F_u = 65 \text{ ksi}</math></p> </p></p>	<p>Eqn. J4-5</p>
		<p>Manual Table 7-5</p>
		<p>Manual Table 7-6</p>
		<p>Manual Table 7-5</p>

$\phi r_n = (85.9 \text{ kips/in.})(0.645 \text{ in.})$ $= 55.4 \text{ kips} > 14.6 \text{ kips}$	$r_n / \Omega = (57.3 \text{ kips/in.})(0.645 \text{ in.})$ $= 37.0 \text{ kips} > 9.74 \text{ kips}$	Manual Table 7-6
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*Horizontal connection*

LRFD	ASD	
Axial force = 966 kips – 734 kips = 232 kips	Axial force = 690 kips – 524 kips = 166 kips	Section J3.5
232 kips / 14.6 kips/bolt = 15.9 bolts	166 kips / 9.74 kips/bolt = 17.0 bolts	
2 rows both sides = 15.9 bolts / 4 = 3.98	2 rows both sides = 17.0 bolts / 4 = 4.26	
Use 6 bolts per row	Use 6 bolts per row	

<p><i>Check shear in plate</i></p> $\phi = 0.75$ <p>Try Plate with, <math>t_p = 0.375 \text{ in.}</math></p> $A_{gv} / t_p = (1 \text{ row})(6 \text{ bolts} - 1)(6 \text{ in.}) + 4 \text{ in.}$ $= 34.0 \text{ in.}$ $A_{nv} / t_p = 34.0 \text{ in.} - (1 \text{ row})(6 \text{ bolts})$ $\times (1.25 \text{ in.} + 0.0625 \text{ in.}) = 26.1 \text{ in.}$	<p><i>Check shear in plate</i></p> $\Omega = 2.00$ <p>Try Plate with, <math>t_p = 0.375 \text{ in.}</math></p> $A_{gv} / t_p = (1 \text{ row})(6 \text{ bolts} - 1)(6 \text{ in.}) + 4 \text{ in.}$ $= 34.0 \text{ in.}$ $A_{nv} / t_p = 34.0 \text{ in.} - (1 \text{ row})(6 \text{ bolts} - 0.5)$ $\times (1.25 \text{ in.} + 0.0625 \text{ in.}) = 26.8 \text{ in.}$	Section J4.3
<p><i>Check shear yielding</i></p> $\phi = 1.00$ $\phi R_n = \phi(0.6F_u A_g) = 275 \text{ kips}$	<p><i>Check shear yielding</i></p> $\Omega = 1.50$ $R_n / \Omega = (0.6F_y A_g) / \Omega = 184 \text{ kips}$	Eqn. J4-3
<p><i>Check shear rupture</i></p> $\phi = 0.75$ $\phi R_n = \phi(0.6F_u A_{nv}) = 256 \text{ kips}$ $= 256 \text{ kips} < 275 \text{ kips}$ $256 \text{ kips} > 232 \text{ kips} / 2 = 116 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Bolt bearing on plate</i></p>	<p><i>Check shear rupture</i></p> $\Omega = 2.00$ $R_n / \Omega = (0.6F_u A_{nv}) / \Omega = 170 \text{ kips}$ $= 170 \text{ kips} < 184 \text{ kips}$ $170 \text{ kips} > 166 \text{ kips} / 2 = 83.0 \text{ kips} \quad \mathbf{o.k.}$ <p><i>Bolt bearing on plate</i></p>	Eqn. J4-4

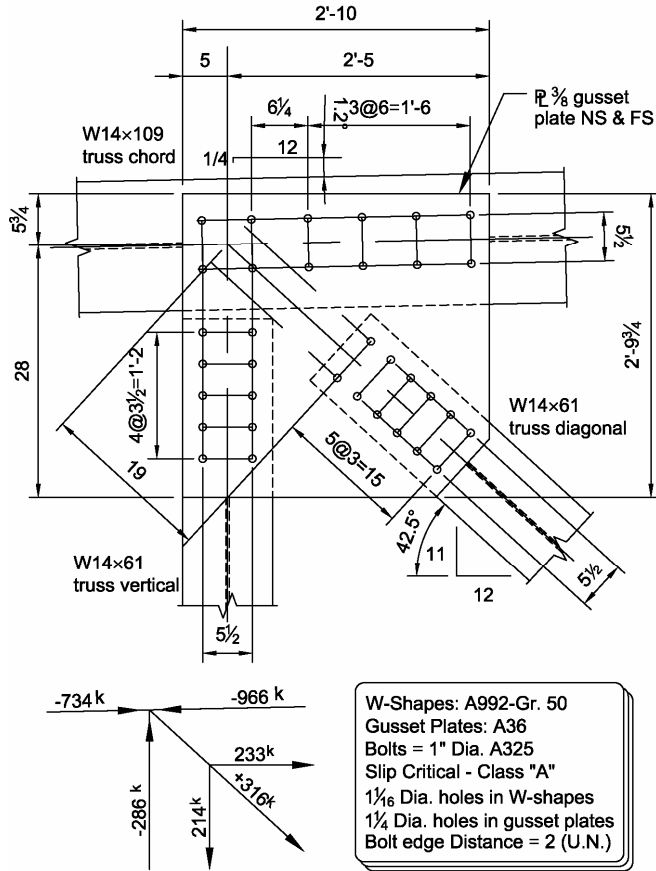
<p>Based on Bolt Spacing = 6 in.; oversized holes, <math>F_u = 58</math> ksi</p> $\phi r_n = (104 \text{ kips/in.})(0.375 \text{ in.})$ $= 39.0 \text{ kips} > 14.6 \text{ kips}$ <p>Based on Edge Distance = 2 in.; oversized holes, <math>F_u = 58</math> ksi</p> $\phi r_n = (71.8 \text{ kips/in.})(0.375 \text{ in.})$ $= 26.9 \text{ kips} > 14.6 \text{ kips}$ <p><i>Bolt bearing on flange</i></p> <p>Based on Bolt Spacing = 6 in.; standard holes, <math>F_u = 65</math> ksi</p> $\phi r_n = (117 \text{ kips/in.})(0.860 \text{ in.})$ $= 101 \text{ kips} > 14.6 \text{ kips}$ <p>Based on edge distance = 2 in.; standard holes, <math>F_u = 65</math> ksi</p> $\phi r_n = (85.9 \text{ kips/in.})(0.860 \text{ in.})$ $= 73.9 \text{ kips} > 14.6 \text{ kips}$	<p>Based on Bolt Spacing = 6 in.; oversized holes, <math>F_u = 58</math> ksi</p> $r_n / \Omega = (69.9 \text{ kips/in.})(0.375 \text{ in.})$ $= 26.1 \text{ kips} > 9.74 \text{ kips}$ <p>Based on Edge Distance = 2 in.; oversized holes, <math>F_u = 58</math> ksi</p> $r_n / \Omega = (47.9 \text{ kips/in.})(0.375 \text{ in.})$ $= 18.0 \text{ kips} > 9.74 \text{ kips}$ <p><i>Bolt bearing on flange</i></p> <p>Based on Bolt Spacing = 6 in.; standard holes, <math>F_u = 65</math> ksi</p> $r_n / \Omega = (78.0 \text{ kips/in.})(0.860)$ $= 67.0 \text{ kips} > 9.74 \text{ kips}$ <p>Based on edge distance = 2 in.; standard holes, <math>F_u = 65</math> ksi</p> $r_n / \Omega = (57.3 \text{ kips/in.})(0.860 \text{ in.})$ $= 49.3 \text{ kips} > 9.74 \text{ kips}$	<p>Manual Table 7-5</p> <p>Manual Table 7-6</p> <p>Manual Table 7-5</p> <p>Manual Table 7-6</p>
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<i>Vertical connection</i>		
LRFD	ASD	
<p>Axial force = 286 kips</p> <p>286 kips / 14.6 kips/bolt = 19.6 bolts</p> <p>2 rows both sides = 19.6 bolts / 4 = 4.91</p> <p>Therefore use 5 rows</p>	<p>Axial force = 204 kips</p> <p>204 kips / 9.74 kips/bolt = 20.9 bolts</p> <p>2 rows both sides = 20.9 bolts / 4 = 5.24</p> <p>Therefore use 6 rows</p>	
<p><i>Check shear in plate</i></p> <p>Try Plate with, <math>t_p = 0.375</math> in.</p> <p><math>A_{gv} / t_p = 33 \frac{3}{4}</math> in. (from sketch)</p> <p><math>A_{nv} / t_p = 33 \frac{3}{4}</math> in. – (1 row)(7 bolts)</p> <p style="padding-left: 40px;"><math>\times (1.25 \text{ in.} + 0.0625 \text{ in.}) = 24.6 \text{ in}</math></p> <p><i>Check shear yielding</i></p> <p><math>\phi = 1.00</math></p> <p><math>\phi R_n = \phi(0.6F_y A_g) = 273 \text{ kips}</math></p>	<p><i>Check shear in plate</i></p> <p>Try Plate with, <math>t_p = 0.375</math> in.</p> <p><math>A_{gv} / t_p = 33 \frac{3}{4}</math> in. (from sketch)</p> <p><math>A_{nv} / t_p = 33 \frac{3}{4}</math> in. – (1 row)(7 bolts)</p> <p style="padding-left: 40px;"><math>\times (1.25 \text{ in.} + 0.0625 \text{ in.}) = 24.6 \text{ in}</math></p> <p><i>Check shear yielding</i></p> <p><math>\Omega = 1.50</math></p> <p><math>R_n / \Omega = (0.6F_y A_g) / \Omega = 182 \text{ kips}</math></p>	<p>Specification Section J4.3</p> <p>Eqn. J4-3</p>

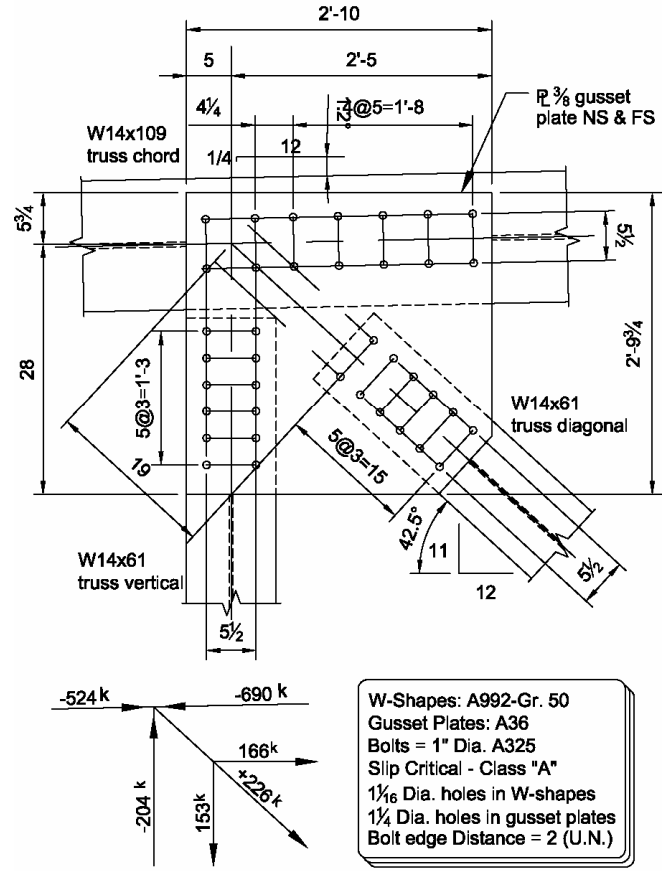
<p><i>Check shear rupture</i></p> <p>Where <math>\phi = 0.75</math></p> $\phi R_n = \phi(0.6F_u A_{nv}) = 240 \text{ kips}$ $= 240 \text{ kips} < 273 \text{ kips}$ <p>240 kips &gt; 286 kips / 2 = 143 kips <b>o.k.</b></p> <p><i>Bolt bearing on plate</i></p> <p>Conservatively based on Bolt Spacing = 3 in.; oversized holes,</p> $\phi r_n = (91.4 \text{ kips/in.})(0.375 \text{ in.})$ $= 34.3 \text{ kips} > 14.6 \text{ kips}$ <p>Based on Edge Distance = 2 in.; oversized holes,</p> $\phi r_n = (71.8 \text{ kips/in.})(0.375 \text{ in.})$ $= 26.9 \text{ kips} > 14.6 \text{ kips}$ <p><i>Bolt bearing on flange</i></p> <p>Conservatively based on Bolt Spacing = 3 in.; standard holes,</p> $\phi r_n = (104 \text{ kips/in.})(0.645 \text{ in.})$ $= 67.0 \text{ kips} > 14.6 \text{ kips}$ <p>Based on edge distance = 2 in.; standard holes,</p> $\phi r_n = (85.9 \text{ kips/in.})(0.645 \text{ in.})$ $= 55.4 \text{ kips} > 14.6 \text{ kips}$ <p><i>Bolt Shear Strength</i></p> <p>Based on ASTM A325-N; single shear</p> $\phi r_n = 28.3 \text{ kips} > 14.6 \text{ kips}$	<p><i>Check shear rupture</i></p> <p>Where <math>\Omega = 2.00</math></p> $R_n / \Omega = (0.6F_u A_{nv}) / \Omega = 160 \text{ kips}$ $= 160 \text{ kips} < 182 \text{ kips}$ <p>160 kips &gt; 204 kips / 2 = 102 kips <b>o.k.</b></p> <p><i>Bolt bearing on plate</i></p> <p>Conservatively based on Bolt Spacing = 3 in.; oversized holes,</p> $r_n / \Omega = (60.9 \text{ kips/in.})(0.375 \text{ in.})$ $= 22.8 \text{ kips} > 9.74 \text{ kips}$ <p>Based on Edge Distance = 2 in.; oversized holes,</p> $r_n / \Omega = (47.9 \text{ kips/in.})(0.375 \text{ in.})$ $= 18.0 \text{ kips} > 9.74 \text{ kips}$ <p><i>Bolt bearing on flange</i></p> <p>Conservatively based on Bolt Spacing = 3 in.; standard holes,</p> $r_n / \Omega = (75.6 \text{ kips/in.})(0.645)$ $= 48.8 \text{ kips} > 9.74 \text{ kips}$ <p>Based on edge distance = 2 in.; standard holes,</p> $r_n / \Omega = (57.3 \text{ kips/in.})(0.645 \text{ in.})$ $= 37.0 \text{ kips} > 9.74 \text{ kips}$ <p><i>Bolt Shear Strength</i></p> <p>Based on ASTM A325-N; single shear</p> $r_n / \Omega = 18.8 \text{ kips} > 9.74 \text{ kips}$	<p>Eqn. J4-4</p> <p>Manual Table 7-5</p> <p>Manual Table 7-6</p> <p>Manual Table 7-5</p> <p>Manual Table 7-6</p> <p>Manual Table 7-1</p>
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Note: The final layout for the connection is as follows:

Top Chord Panel Point U3 of Example Truss Design  
Case 1: LRFD Design Format



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**

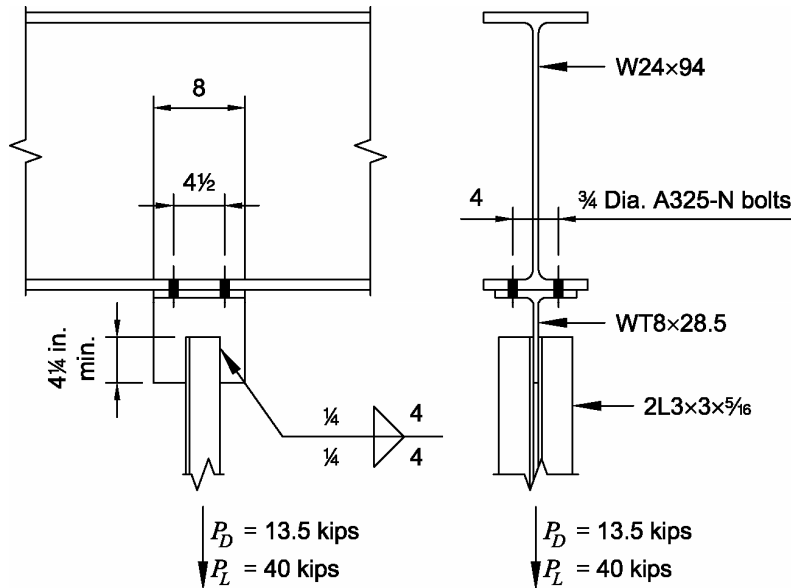
**Given:**

Design a WT tension-hanger connection between a 2L3×3×5/16 tension member and a W24×94 beam connection to support the following loads:

$$P_D = 13.5 \text{ kips}$$

$$P_L = 40 \text{ kips}$$

Use 3/4-in. diameter ASTM A325-N bolts and 70 ksi electrodes.



**Material Properties:**

Hanger	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	Manual Table 2-3
Beam W24×94	ASTM A992	$F_y = 50 \text{ ksi}$	$F_u = 65 \text{ ksi}$	
Angles 2L3×3×5/16	ASTM A36	$F_y = 36 \text{ ksi}$	$F_u = 58 \text{ ksi}$	

**Geometric Properties:**

Beam W24×94	$d = 24.3 \text{ in.}$	$t_w = 0.515 \text{ in.}$	$b_f = 9.07 \text{ in.}$	$t_f = 0.875 \text{ in.}$	Manual Tables 1-1, 1-7, and 1-15
Angles 2L3×3×5/16	$A = 3.55 \text{ in.}^2$	$\bar{x} = 0.860 \text{ in.}$			

**Solution:**

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

Eqn. D2-1

$$R_n = F_y A_g = (36 \text{ ksi})(3.55 \text{ in.}^2) = 128 \text{ kips}$$

LRFD	ASD
$\phi R_n = 0.90(128 \text{ kips})$ $= 115 \text{ kips} > 80.2 \text{ kips}$	$R_n / \Omega = \frac{128 \text{ kips}}{1.67}$ $= 76.6 \text{ kips} > 53.5 \text{ kips}$
Try 1/4-in. fillet welds $L_{\min} = \frac{P_u}{1.392D} = \frac{80.2 \text{ kips}}{1.392(4 \text{ sixteenths})}$ $= 14.4 \text{ in.}$ Use four 4-in. welds (16 in. total), one at each toe and heel of each angle.	Try 1/4-in. fillet welds $L_{\min} = \frac{P_a}{0.928D} = \frac{53.5 \text{ kips}}{0.928(4 \text{ sixteenths})}$ $= 14.4 \text{ in.}$ Use four 4-in. welds (16 in. total), one at each toe and heel of each angle.

Check tension rupture of angles

Calculate the effective net area

$$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{0.860 \text{ in.}}{4 \text{ in.}} = 0.785$$

Table D3.1  
Case 2

$$A_e = A_n U = (3.55 \text{ in.}^2)(0.785) = 2.80 \text{ in.}^2$$

Eqn. D2-2

LRFD	ASD
$\phi = 0.75$ $R_n = F_u A_e = (58 \text{ ksi})(2.80 \text{ in.}^2) = 163 \text{ kips}$ $\phi_t R_n = 0.75(163)$ $= 122 \text{ kips} > 80 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 2.00$ $R_n = F_u A_e = (58 \text{ ksi})(2.80 \text{ in.}^2) = 163 \text{ kips}$ $R_n / \Omega_t = \frac{163}{2.00}$ $= 81.5 \text{ kips} > 53.5 \text{ kips} \quad \mathbf{o.k.}$
Select a preliminary WT using beam gage $g = 4 \text{ in.}$ With four 3/4-in. diameter ASTM A325-N bolts, $T_u = r_{ut} = \frac{P_u}{n} = \frac{80 \text{ kips}}{4} = 20 \text{ kips/bolt}$ $B = \phi r_n = 29.8 \text{ kips} > 20 \text{ kips} \quad \mathbf{o.k.}$	Select a preliminary WT using beam gage $g = 4 \text{ in.}$ With four 3/4-in. diameter ASTM A325-N bolts, $T_a = r_{at} = \frac{P_a}{n} = \frac{53.5 \text{ kips}}{4} = 13.4 \text{ kips/bolt}$ $B = r_n / \Omega = 19.9 \text{ kips} > 13.4 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 7-2

With four bolts, the maximum effective length is  $2g = 8$  in. Thus, there are 4 in. of tee length tributary to each pair of bolts and

LRFD	ASD
$\frac{2 \text{ bolts}(20 \text{ kips/bolt})}{4 \text{ in.}} = 10.0 \text{ kips/in.}$	$\frac{2 \text{ bolts}(13.4 \text{ kips/bolt})}{4 \text{ in.}} = 6.7 \text{ kips/in.}$

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  $b = 4 \text{ in.}/2 = 2 \text{ in.}$ ,  $t_0 \approx 1/16 \text{ in.}$ , and  $d_{\min} = 4 \text{ in.} + 1/4 \text{ in.} + k \approx 6 \text{ in.}$ , appropriate selections include:

WT6×39.5    WT8×28.5  
 WT7×34    WT9×30

Try WT8×28.5;  $b_f = 7.12 \text{ in.}$ ,  $t_f = 0.715 \text{ in.}$ ,  $t_w = 0.430 \text{ in.}$

Manual  
 Table 1-8

Check prying action

Manual  
 Part 9

$$b = \frac{g - t_w}{2} = \frac{(4 \text{ in.} - 0.430 \text{ in.})}{2} = 1.79 \text{ in.} > 1\frac{1}{4} \text{ in. entering and tightening clearance, o.k.}$$

$$a = \frac{b_f - g}{2} = \frac{(7.12 \text{ in.} - 4 \text{ in.})}{2} = 1.56 \text{ in.}$$

Since  $a = 1.56 \text{ in.} < 1.25b = 2.24 \text{ in.}$ , use  $a = 1.56 \text{ in.}$

$$b' = b - \frac{d}{2} = 1.79 \text{ in.} - \left(\frac{3/4 \text{ in.}}{2}\right) = 1.42 \text{ in.}$$

$$a' = a + \frac{d}{2} = 1.56 \text{ in.} + \left(\frac{3/4 \text{ in.}}{2}\right) = 1.94 \text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.42 \text{ in.}}{1.94 \text{ in.}} = 0.732$$

LRFD	ASD
$\beta = \frac{1}{\rho} \left( \frac{B}{T} - 1 \right) = \frac{1}{0.732} \left( \frac{29.8 \text{ kips/bolt}}{20 \text{ kips}} - 1 \right)$	$\beta = \frac{1}{\rho} \left( \frac{B}{T} - 1 \right) = \frac{1}{0.732} \left( \frac{19.9 \text{ kips/bolt}}{13.4 \text{ kips}} - 1 \right)$
= 0.669	= 0.663

$$\delta = 1 - \frac{d'}{p} = 1 - \left( \frac{13/16 \text{ in.}}{4 \text{ in.}} \right) = 0.797$$

Since  $\beta < 1.0$ ,

LRFD	ASD
$\alpha' = \frac{1}{\delta} \left( \frac{\beta}{1-\beta} \right) \leq 1.0$ $= \frac{1}{0.797} \left( \frac{0.669}{1-0.669} \right)$ $= 2.54 \therefore \alpha' = 1.0$ $t_{\min} = \sqrt{\frac{4.44Tb'}{pF_u(1+\delta\alpha')}} = \sqrt{\frac{4.44(20 \text{ kips/bolt})(1.42 \text{ in.})}{(4 \text{ in.})(65 \text{ ksi})[1+(0.797)(1.0)]}}$ $= 0.521 \text{ in.} < t_f = 0.715 \text{ in.} \quad \mathbf{o.k.}$	$\alpha' = \frac{1}{\delta} \left( \frac{\beta}{1-\beta} \right) \leq 1.0$ $= \frac{1}{0.797} \left( \frac{0.663}{1-0.663} \right)$ $= 2.47 \therefore \alpha' = 1.0$ $t_{\min} = \sqrt{\frac{6.66Tb'}{pF_u(1+\delta\alpha')}} = \sqrt{\frac{6.66(13.4 \text{ kips/bolt})(1.42 \text{ in.})}{(4 \text{ in.})(65 \text{ ksi})[1+(0.797)(1.0)]}}$ $= 0.521 \text{ in.} < t_f = 0.715 \text{ in.} \quad \mathbf{o.k.}$

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

$$L_w = 3 \text{ in.} + 2(4 \text{ in.})(\tan 30^\circ) \leq 8 \text{ in.}$$

$$= 7.62 \text{ in.}$$

and the nominal strength is determined as

LRFD	ASD
$\phi = 0.90$ $R_n = F_y A_{g\text{eff}} = (50 \text{ ksi})(7.62 \text{ in.})(0.430 \text{ in.}) = 164 \text{ kips}$ $\phi R_n = 0.90(164) = 147 \text{ kips} > 80 \text{ kips} \quad \mathbf{o.k.}$	$\Omega = 1.67$ $R_n = F_y A_{g\text{eff}} = (50 \text{ ksi})(7.62 \text{ in.})(0.430 \text{ in.}) = 164 \text{ kips}$ $R_n / \Omega = \frac{164}{1.67} = 98.2 \text{ kips} > 53.5 \text{ kips} \quad \mathbf{o.k.}$

Eqn. D2-1

Manual  
Part 9

Check shear rupture of the base metal along the toe and heel of each weld line

$$t_{\min} = \frac{6.19D}{F_u} = 6.19 \left( \frac{4}{65 \text{ ksi}} \right) = 0.381 \text{ in.} < 0.430 \text{ in.} \quad \mathbf{o.k.}$$

Check block shear rupture of the tee stem

Since the angles are welded to the WT-hanger the gross area shear yielding will control.

$$A_{gv} = (2 \text{ welds})(4 \text{ in.})(0.430 \text{ in.}) = 3.44 \text{ in.}^2$$

Tension stress is uniform, therefore  $U_{bs} = 1.0$ .

Table D3.1

$$A_{nt} = A_g = 1.0(3 \text{ in.})(0.430 \text{ in.}) = 1.29 \text{ in.}^2$$

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

$$R_n = 0.6F_y A_{gv} + U_{bs} F_u A_{nt} = 0.6(50 \text{ ksi})(3.44 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.29 \text{ in.}^2) = 187 \text{ kips}$$

Eqn. J4-5

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(187) = 140 \text{ kips} > 80 \text{ kips}$ <b>o.k.</b>	$R_n / \Omega = \frac{187}{2.00} = 93.5 \text{ kips} > 53.5 \text{ kips}$ <b>o.k.</b>

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.,  $q_u \approx 0$ . Assuming  $b' = 1.42 \text{ in.}$

LRFD	ASD
$t_{\min} = \sqrt{\frac{4.44Tb'}{pF_u}}$ $= \sqrt{\frac{4.44(20 \text{ kips/bolt})(1.42 \text{ in.})}{(4 \text{ in./bolt})(65 \text{ ksi})}}$ $= 0.696 \text{ in.}$	$t_{\min} = \sqrt{\frac{6.66Tb'}{pF_u}}$ $= \sqrt{\frac{6.66(13.4 \text{ kips/bolt})(1.42 \text{ in.})}{(4 \text{ in./bolt})(65 \text{ ksi})}}$ $= 0.698 \text{ in.}$
Try WT9×35.5	Try WT9×35.5
$t_f = 0.810 \text{ in.} > 0.696 \text{ in.}$ <b>o.k.</b>	$t_f = 0.810 \text{ in.} > 0.698 \text{ in.}$ <b>o.k.</b>
$t_w = 0.495 \text{ in.} > 0.430 \text{ in.}$ <b>o.k.</b>	$t_w = 0.495 \text{ in.} > 0.430 \text{ in.}$ <b>o.k.</b>
$b_f = 7.64 \text{ in.} > 7.12 \text{ in.}$ <b>o.k.</b>	$b_f = 7.64 \text{ in.} > 7.12 \text{ in.}$ <b>o.k.</b>

Manual  
Table 1-8

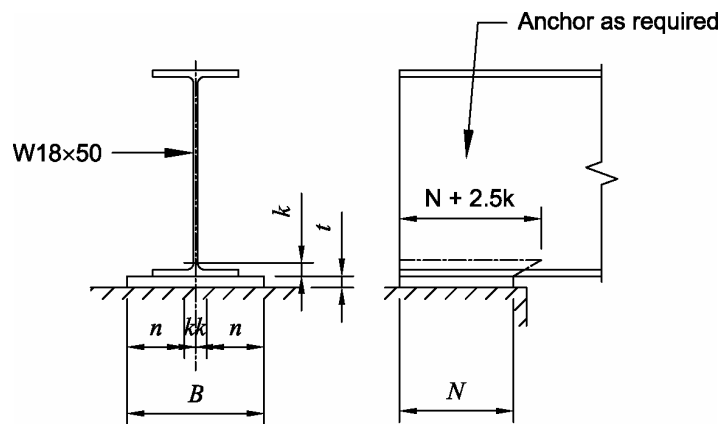
### Example II.D-2 Beam Bearing Plate

**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  $N = 6$  in. and the bearing plate is centered on the thickness of the wall.



**Material Properties:**

Beam W18×50	ASTM A992	$F_y = 50$ ksi	$F_u = 65$ ksi	Manual Tables 2-3 and 2-4
Bearing Plate (if required)	ASTM A36	$F_y = 36$ ksi	$F_u = 58$ ksi	
Concrete Wall	$f'_c = 3$ ksi			

**Geometric Properties:**

Beam W18×50	$d = 18.0$ in.	$t_w = 0.355$ in.	$b_f = 7.50$ in.	$t_f = 0.570$ in.	Manual Table 1-1
Concrete Wall	$k = 0.972$ in.	$k_t = 13/16$ in.		$h = 10$ in.	

**Solution A:**

LRFD	ASD
$R_u = 1.2(15 \text{ kips}) + 1.6(45 \text{ kips}) = 90 \text{ kips}$	$R_a = 15 \text{ kips} + 45 \text{ kips} = 60 \text{ kips}$
<i>Check local web yielding</i>	<i>Check local web yielding</i>
$N_{req} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$ $= \frac{90 \text{ kips} - 43.1 \text{ kips}}{17.8 \text{ kips/in.}} \geq 0.972 \text{ in.}$ $= 2.63 \text{ in.} < 10 \text{ in.} \quad \mathbf{o.k.}$	$N_{req} = \frac{R_a - R_1 / \Omega}{R_2 / \Omega} \geq k$ $= \frac{60 \text{ kips} - 28.8 \text{ kips}}{11.8 \text{ kips/in.}} \geq 0.972 \text{ in.}$ $= 2.64 \text{ in.} < 10 \text{ in.} \quad \mathbf{o.k.}$
<i>Check web crippling</i>	<i>Check web crippling</i>
$\frac{N}{d} = \frac{10 \text{ in.}}{18.0 \text{ in.}}$ $= 0.556$ <p>Since <math>\frac{N}{d} &gt; 0.2</math>,</p> $N_{req} = \frac{R_u - \phi R_5}{\phi R_6}$ $= \frac{90 \text{ kips} - 52.0 \text{ kips}}{6.30 \text{ kips/in.}}$ $= 6.03 \text{ in.} < 10 \text{ in.} \quad \mathbf{o.k.}$	$\frac{N}{d} = \frac{10 \text{ in.}}{18.0 \text{ in.}}$ $= 0.556$ <p>Since <math>\frac{N}{d} &gt; 0.2</math>,</p> $N_{req} = \frac{R_a - R_5 / \Omega}{R_6 / \Omega}$ $= \frac{60 \text{ kips} - 34.7 \text{ kips}}{4.20 \text{ kips/in.}}$ $= 6.03 \text{ in.} < 10 \text{ in.} \quad \mathbf{o.k.}$
<p>Verify <math>\frac{N}{d} &gt; 0.2</math>,</p> $\frac{N}{d} = 0.335 > 0.2 \quad \mathbf{o.k.}$	<p>Verify <math>\frac{N}{d} &gt; 0.2</math>,</p> $\frac{N}{d} = 0.335 > 0.2 \quad \mathbf{o.k.}$
<i>Check the bearing strength of concrete</i>	<i>Check the bearing strength of concrete</i>
$\phi_c = 0.60$	$\Omega_c = 2.50$
$\phi_c P_p = \phi_c (0.85 f'_c) A_1$ $= 0.60 (0.85)(3 \text{ ksi})(7.50 \text{ in.} \times 10 \text{ in.})$ $= 115 \text{ kips} > 90 \text{ kips} \quad \mathbf{o.k.}$	$P_p / \Omega_c = (0.85 f'_c) A_1 / \Omega_c$ $= (0.85)(3 \text{ ksi})(7.50 \text{ in.} \times 10 \text{ in.}) / 2.50$ $= 76.5 \text{ kips} > 60 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 9-4

Manual  
Table 9-4

Eqn. J8-1

Check beam flange thickness

Manual  
Part 14

LRFD	ASD
<p><i>Determine cantilever length</i></p> $n = \frac{b_f}{2} - k = \frac{7.50}{2} - 0.972 \text{ in.} = 2.78 \text{ in.},$ <p><i>Determine bearing pressure</i></p> $f_p = \frac{R_u}{A_1}$ <p><i>Determine cantilever moment</i></p> $M_u = \frac{R_u n^2}{2A_1}$ $Z = \frac{1}{4}t^2$ $M \leq F_y Z \leq F_y \left( \frac{t^2}{4} \right)$ $t_{req} = \sqrt{\frac{4M_u}{\phi F_y}} = \sqrt{\frac{2R_u n^2}{\phi A_1 F_y}}$ $\phi = 0.90$ $t_{min} = \sqrt{\frac{2.22R_u n^2}{A_1 F_y}}$ $= \sqrt{\frac{2.22(90 \text{ kips})(2.78 \text{ in.})^2}{(7.50 \text{ in.} \times 10 \text{ in.})(50 \text{ ksi})}}$ $= 0.643 \text{ in.} > 0.570 \text{ in.} \quad \mathbf{n.g.}$ <p><b>A bearing plate is required.</b></p>	<p><i>Determine cantilever length</i></p> $n = \frac{b_f}{2} - k = \frac{7.50}{2} - 0.972 \text{ in.} = 2.78 \text{ in.},$ <p><i>Determine bearing pressure</i></p> $f_p = \frac{R_a}{A_1}$ <p><i>Determine cantilever moment</i></p> $M_a = \frac{R_a n^2}{2A_1}$ $Z = \frac{1}{4}t^2$ $M \leq F_y Z \leq F_y \left( \frac{t^2}{4} \right)$ $t_{req} = \sqrt{\frac{\Omega 4M_a}{F_y}} = \sqrt{\frac{\Omega 2R_a n^2}{A_1 F_y}}$ $\Omega = 1.67$ $t_{min} = \sqrt{\frac{3.33R_a n^2}{A_1 F_y}}$ $= \sqrt{\frac{3.33(60 \text{ kips})(2.78 \text{ in.})^2}{(7.50 \text{ in.} \times 10 \text{ in.})(50 \text{ ksi})}}$ $= 0.643 \text{ in.} > 0.570 \text{ in.} \quad \mathbf{n.g.}$ <p><b>A bearing plate is required.</b></p>

**Solution B:**

$$N = 10 \text{ in.}$$

From Solution A, local web yielding and web crippling are not critical.

LRFD	ASD
<i>Calculate the required bearing-plate width.</i>	<i>Calculate the required bearing-plate width.</i>
$\phi_c = 0.60$	$\Omega_c = 2.50$
$A_{1 \text{ req}} = \frac{R_u}{\phi_c(0.85 f'_c)}$	$A_{1 \text{ req}} = \frac{R_a \Omega_c}{(0.85 f'_c)}$
$= \frac{90 \text{ kips}}{0.60(0.85 \times 3 \text{ ksi})}$	$= \frac{60 \text{ kips}(2.50)}{(0.85 \times 3 \text{ ksi})}$
$= 58.8 \text{ in}^2$	$= 58.8 \text{ in}^2$
$B_{\text{req}} = \frac{A_{1 \text{ req}}}{N}$	$B_{\text{req}} = \frac{A_{1 \text{ req}}}{N}$
$= \frac{58.8 \text{ in.}^2}{10 \text{ in.}}$	$= \frac{58.8 \text{ in.}^2}{10 \text{ in.}}$
$= 5.88 \text{ in.}$	$= 5.88 \text{ in.}$
Use $B = 8 \text{ in.}$ (least whole-inch dimension that exceeds $b_f$ )	Use $B = 8 \text{ in.}$ (least whole-inch dimension that exceeds $b_f$ )
<i>Calculate required bearing-plate thickness.</i>	<i>Calculate required bearing-plate thickness.</i>
$n = \frac{B}{2} - k$	$n = \frac{B}{2} - k$
$= \frac{8 \text{ in.}}{2} - 0.972 \text{ in.}$	$= \frac{8 \text{ in.}}{2} - 0.972 \text{ in.}$
$= 3.03 \text{ in.}$	$= 3.03 \text{ in.}$
$t_{\min} = \sqrt{\frac{2.22 R_u n^2}{A_1 F_y}}$	$t_{\min} = \sqrt{\frac{3.33 R_a n^2}{A_1 F_y}}$
$= \sqrt{\frac{2.22(90 \text{ kips})(3.03 \text{ in.})^2}{(10 \text{ in.} \times 8 \text{ in.})(36 \text{ ksi})}}$	$= \sqrt{\frac{3.33(60 \text{ kips})(3.03 \text{ in.})^2}{(10 \text{ in.} \times 8 \text{ in.})(36 \text{ ksi})}}$
$= 0.799 \text{ in.}$	$= 0.799 \text{ in.}$
Use PL1×10×0'-8	Use PL1×10×0'-8

Eqn. J8-1

Manual  
Part 14



**Solution C:**

$$N = 6 \text{ in.}$$

From Solution A, local web yielding and web crippling are not critical.

$$\text{Try } B = 8 \text{ in.}$$

$$A_1 = B \times N = (8 \text{ in.})(6 \text{ in.}) = 48 \text{ in.}^2$$

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  $A_1$ . The 6-in. dimension spreads 2 in. to each side to meet the concrete edge. Thus, the 8-in. dimension can also be spread 2 in. to each side. Therefore,

$$N_1 = 6 \text{ in.} + 2(2 \text{ in.}) = 10 \text{ in.}$$

$$B_1 = 8 \text{ in.} + 2(2 \text{ in.}) = 12 \text{ in.}$$

$$A_2 = B_1 \times N_1 = 12 \text{ in.} (10 \text{ in.}) = 120 \text{ in.}^2$$

$$\text{Check } \sqrt{A_2/A_1} = 1.58 \leq 2 \quad \mathbf{o.k.}$$

LRFD	ASD
$\phi_c = 0.60$ $A_{1 \text{ req}} = \frac{1}{A_2} \left( \frac{R_u}{\phi_c (0.85 f'_c)} \right)^2$ $= \frac{1}{120 \text{ in.}^2} \left( \frac{90 \text{ kips}}{0.60(0.85 \times 3 \text{ ksi})} \right)^2$ $= 28.8 \text{ in.}^2 < 48 \text{ in.}^2 \quad \mathbf{o.k.}$ <p><i>Calculate the required bearing-plate thickness.</i></p> $n = \frac{B}{2} - k$ $= \frac{8 \text{ in.}}{2} - 0.972 \text{ in.}$ $= 3.03 \text{ in.}$ $t_{\min} = \sqrt{\frac{2.22 R_u n^2}{A_1 F_y}}$ $= \sqrt{\frac{2.22(90 \text{ kips})(3.03 \text{ in.})^2}{(6 \text{ in.})(8 \text{ in.})(36 \text{ ksi})}}$ $= 1.03 \text{ in.}$ <p>Use PL1¼×6×0'-8</p>	$\Omega_c = 2.50$ $A_{1 \text{ req}} = \frac{1}{A_2} \left( \frac{R_u \Omega_c}{(0.85 f'_c)} \right)^2$ $= \frac{1}{120 \text{ in.}^2} \left( \frac{60 \text{ kips}(2.50)}{(0.85 \times 3 \text{ ksi})} \right)^2$ $= 28.8 \text{ in.}^2 < 48 \text{ in.}^2 \quad \mathbf{o.k.}$ <p><i>Calculate the required bearing-plate thickness.</i></p> $n = \frac{B}{2} - k$ $= \frac{8 \text{ in.}}{2} - 0.972 \text{ in.}$ $= 3.03 \text{ in.}$ $t_{\min} = \sqrt{\frac{3.33 R_u n^2}{A_1 F_y}}$ $= \sqrt{\frac{3.33(60 \text{ kips})(3.03 \text{ in.})^2}{(6 \text{ in.})(8 \text{ in.})(36 \text{ ksi})}}$ $= 1.03 \text{ in.}$ <p>Use PL1¼×6×0'-8</p>

Eqn. J8-2

Manual  
Part 14

**Example II.D-3 Slip-Critical Connection with Oversized Holes  
(Designed for Slip as a Serviceability Limit State)**

**Given:**

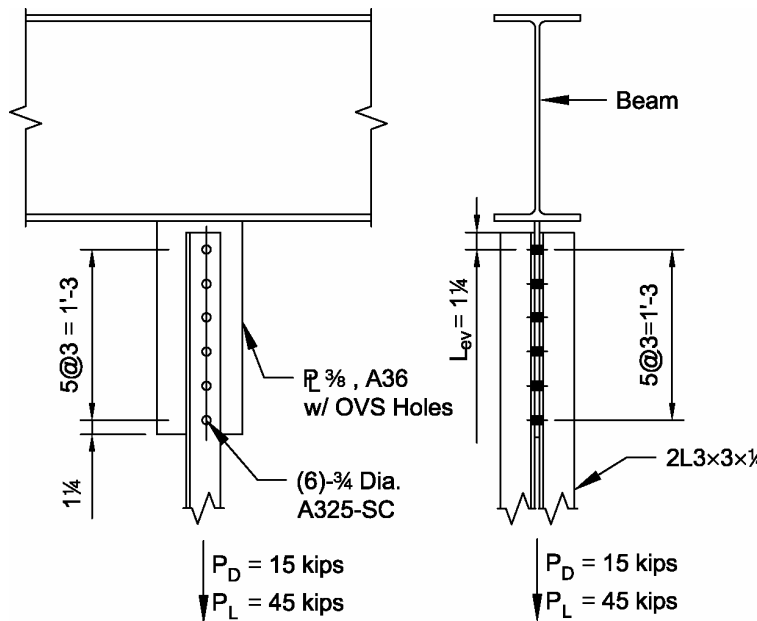
Determine the number of bolts required to connect, 2L3×3×¼ tension member to a plate under a beam as shown. The angles have standard holes and the plate has oversized holes per Manual Table J3.3.

$R_D = 15$  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:**

L3×3×¼  
Plate Material

ASTM A36  
ASTM A36

$F_y = 36$  ksi  
 $F_y = 36$  ksi

$F_u = 58$  ksi  
 $F_u = 58$  ksi

Manual  
Table 2-3  
and 2-4

**Solution:**

LRFD	ASD	
$R_u = (1.2)(15 \text{ kips}) + (1.6)(45 \text{ kips}) = 90 \text{ kips}$	$R_a = 15 \text{ kips} + 45 \text{ kips} = 60 \text{ kips}$	
<i>Design of bolts</i>	<i>Design of bolts</i>	
Because of oversized holes the slip is a serviceability limit state, Class A faying surface, $\frac{3}{4}$ in. diameter ASTM A325-SC bolts in double shear	Because of oversized holes the slip is a serviceability limit state, Class A faying surface, $\frac{3}{4}$ in. diameter ASTM A325-SC bolts in double shear	
$\phi r_n = 18.8 \text{ kips/bolt}$	$r_n / \Omega = 12.6 \text{ kips/bolt}$	Manual Table 7-4
$n = \frac{\phi R_n}{\phi r_n} = \frac{90 \text{ kips}}{18.8 \text{ kips/bolt}}$	$n = \frac{(R_n / \Omega)}{(r_n / \Omega)} = \frac{60 \text{ kips}}{12.6 \text{ kips/bolt}}$	
$= 4.78 \rightarrow 5 \text{ bolts}$	$= 4.76 \rightarrow 5 \text{ bolts}$	
Try (5) $\frac{3}{4}$ -in. dia. ASTM A325-SC bolts	Use (5) $\frac{3}{4}$ -in. dia. ASTM A325-SC bolts	
<i>Check bolt shear strength</i>	<i>Check bolt shear strength</i>	
$\phi r_n = \phi F_v A_b = 31.8 \text{ kips/bolt}$	$r_n / \Omega = \frac{F_v A_b}{\Omega} = 21.2 \text{ kips/bolt}$	Manual Table 7-1
$\phi R_n = \phi r_n n = (31.8 \text{ kips/bolt})(5 \text{ bolts})$	$R_n / \Omega = \frac{r_n}{\Omega} n = (21.2 \text{ kips/bolt})(5 \text{ bolts})$	
$= 159 \text{ kips} > 90 \text{ kips}$ <b>o.k.</b>	$= 106 \text{ kips} > 60 \text{ kips}$ <b>o.k.</b>	
Try, PL $\frac{3}{8}$ in.	Try, PL $\frac{3}{8}$ in.	
<i>Check bolt bearing strength on plate</i>	<i>Check bolt bearing strength on plate</i>	
$\phi r_n = \phi(1.2L_c t F_u) \leq \phi(2.4dt F_u)$	$r_n / \Omega = (1.2L_c t F_u) / \Omega \leq (2.4dt F_u) / \Omega$	Eqn J3-6a
$\phi = 0.75$	$\Omega = 2.00$	
Oversized holes, bolt spacing = 3 in.	Oversized holes, bolt spacing = 3 in.	
$\phi r_n = (78.3 \text{ kips/in.})(\frac{3}{8} \text{ in.})$	$r_n / \Omega = (52.2 \text{ kips/in.})(\frac{3}{8} \text{ in.})$	Manual Table 7-5
$= 29.4 \text{ kips/bolt}$	$= 19.6 \text{ kips/bolt}$	
Oversized holes, edge distance = $1\frac{1}{4}$ in.	Oversized holes, edge distance = $1\frac{1}{4}$ in.	
$\phi r_n = (40.8 \text{ kips/in.})(\frac{3}{8} \text{ in.})$	$r_n / \Omega = (27.2 \text{ kips/in.})(\frac{3}{8} \text{ in.})$	Manual Table 7-6
$= 15.3 \text{ kips/bolt}$	$= 10.2 \text{ kips/bolt}$	
$15.3 \text{ kips/bolt} < 29.4 \text{ kips/bolt}$	$10.2 \text{ kips/bolt} < 19.6 \text{ kips/bolt}$	

$\phi R_n = \phi r_n n$ $= (15.3 \text{ kips/bolt})(5 \text{ bolts})$ $= 76.5 \text{ kips} < 80 \text{ kips} \quad \mathbf{n.g.}$ <p>Try 6 bolts</p> $\phi R_n = \phi r_n n$ $= (15.3 \text{ kips/bolt})(6 \text{ bolts})$ $= 91.8 \text{ kips} > 80 \text{ kips} \quad \mathbf{o.k.}$ <p><b>Use 6 bolts</b></p>	$R_n / \Omega = r_n n / \Omega$ $= (10.2 \text{ kips/bolt})(5 \text{ bolts})$ $= 51.0 \text{ kips} < 80 \text{ kips} \quad \mathbf{n.g.}$ <p>Try 6 bolts</p> $R_n / \Omega = r_n n / \Omega$ $= (10.2 \text{ kips/bolt})(6 \text{ bolts})$ $= 61.2 \text{ kips} > 60 \text{ kips} \quad \mathbf{o.k.}$ <p><b>Use 6 bolts</b></p>
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### 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 members that are part of the gravity and lateral frame of a typical 4-story building. The design is completed in accordance with the 2005 AISC *Specification for Structural Steel Buildings* and the 13<sup>th</sup> Edition AISC *Steel Construction Manual*. Building code requirements are taken from International Building Code 2003 as the Design Code and loading criteria are based on SEI/ASCE 7-02.

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 each of the 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:

$$\text{Simple Beam:} \quad \Delta = \frac{5 w \text{ kip/in.}(l \text{ in.})^4}{384(29,000 \text{ ksi})(I \text{ in.})^4} = \frac{w \text{ kip/ft}(l \text{ ft})^4}{1290(I \text{ in.})^4}$$

$$\text{Beam Fixed at both Ends:} \quad \Delta = \frac{w \text{ kip/in.}(l \text{ in.})^4}{384(29,000 \text{ ksi})(I \text{ in.})^4} = \frac{w \text{ kip/ft}(l \text{ ft})^4}{6450(I \text{ in.})^4}$$

**DESIGN SEQUENCE**

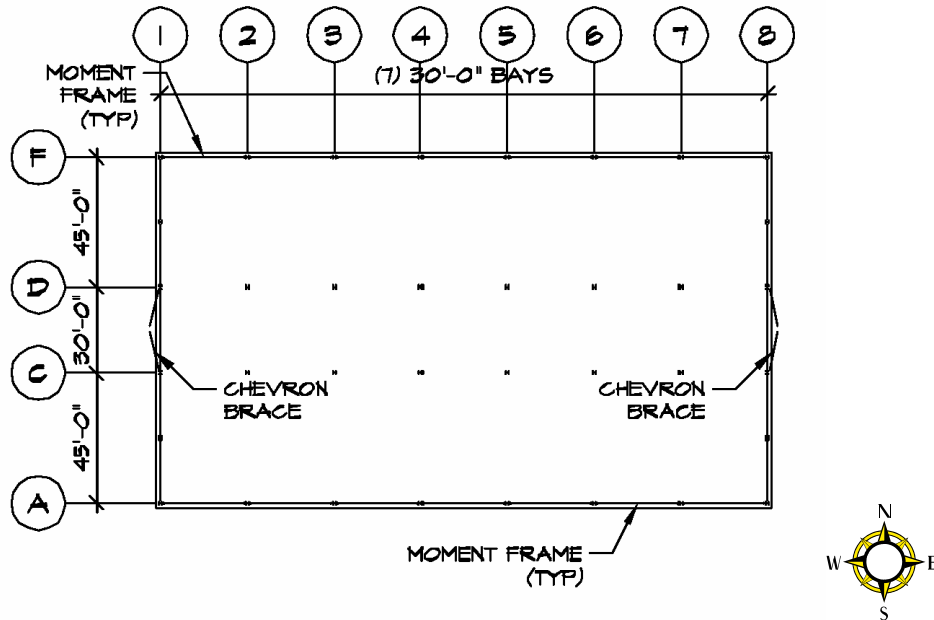
The design sequence is presented as follows:

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'-6" and the height from the fourth floor to the roof (at the edge of the building) is 14'-6". 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 help carry 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 located in an Office Park with substantial parking areas around it, 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

This building is located in an area where the sub-soil has been evaluated and the site class has been determined to be Category D. The area has a short period  $S_s = 0.121$  and a one-second period  $S_1 = 0.060$ . 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 ( $p_g$ ) for this building is 20 psf. The slope of the roof is  $\frac{1}{4}$  in./ft or more at all locations, but not exceeding  $\frac{1}{2}$  in./ft. Consequently, 5 psf rain-on-snow surcharge is to be considered, but no 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  $p_f = (I)p_g = 0.020$  kip/ft<sup>2</sup> snow live load. Snow drift will be applied at the edges of the roof and at the screen wall around the mechanical area.

Floor Loads:

The basic live load for the floor is 50 psf. An additional partition live load of 20 psf will be applied. However, 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.

### ROOF MEMBER DESIGN AND SELECTION

*Calculate dead load and live load*

Dead Load

Roofing	= 0.005 kip/ft <sup>2</sup>
Insulation	= 0.002 kip/ft <sup>2</sup>
Deck	= 0.002 kip/ft <sup>2</sup>
Beams	= 0.003 kip/ft <sup>2</sup>
Joists	= 0.003 kip/ft <sup>2</sup>
Misc.	= 0.005 kip/ft <sup>2</sup>
<u>Total</u>	<u>= 0.020 kip/ft<sup>2</sup></u>

Live Load

Snow	= 0.020 kip/ft <sup>2</sup>	ASCE 7
<u>Rain on Snow</u>	<u>= 0.005 kip/ft<sup>2</sup></u>	Section 7.3
Total	= 0.025 kip/ft <sup>2</sup>	and 7.10

Note: In this design, the Rain and Snow Load is greater than the Roof Live Load

The deck for this design example is  $1\frac{1}{2}$  in., wide rib, 22 gage, painted roof deck, placed in a pattern of 3 continuous spans minimum. The general joist layout is 6 ft on center. At 6 ft on center, this deck has an allowable total load capacity of 0.071 kip/ft<sup>2</sup>. This will require changing 2 spaces at 6 ft to a 5 ft, 4 ft, 3 ft pattern at the ends of the Mechanical area. The roof diaphragm and roof loads extend 6 in. past the centerline of grid as shown on Drawing S4.1



Flat roof snow load = 0.020 kip/ft<sup>2</sup>, Density  $\gamma = 0.0166$  kip/ft<sup>2</sup>,  $h_b = 1.20$  ft

ASCE 7  
Section 7.8

Summary of Drifts

	Upwind Roof Length ( $l_u$ )	Proj. Height	Max. Drift Load	Max Drift Width ( $W$ )	
Side Parapet	121 ft	2 ft	0.0132 kip/ft <sup>2</sup>	6.36 ft	Summary from Spread Sheet
End Parapet	211 ft	2 ft	0.0132 kip/ft <sup>2</sup>	6.36 ft	
Screen Wall	60.5 ft	6 ft	0.0305 kip/ft <sup>2</sup>	7.35 ft	

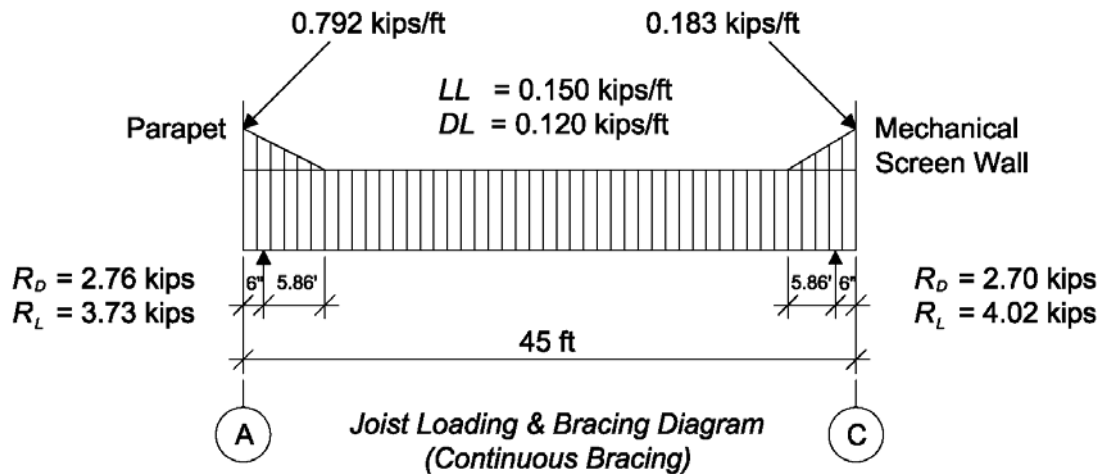
ASCE 7  
Figure 7-8

**SELECT ROOF JOISTS**

*Layout loads and size joists*

User Note: Joists are normally designed by ASD and are designed and selected in this manner here.

The 45-ft side joist with the heaviest loads is shown below along with its end reactions and maximum moment:



Because the load is not uniform, select a 24KCS4 JOIST AT 16.5 plf, which has an allowable moment of 92.3 kip-ft and an allowable shear of 8.40 kips.

The standard 30 ft joist in the middle bay will have a uniform load of

$$w = (0.020 \text{ kip/ft}^2 + 0.025 \text{ kip/ft}^2)(6 \text{ ft}) = 0.270 \text{ kip/ft.}$$

Per joist catalog, select an 18K5 joist at 7.7 plf.

Note: the first joist away from the screen wall and the first joist away from the end of the building must account for snow drift. An 18K7 joist will be used in these locations.

**SELECT ROOF BEAMS**

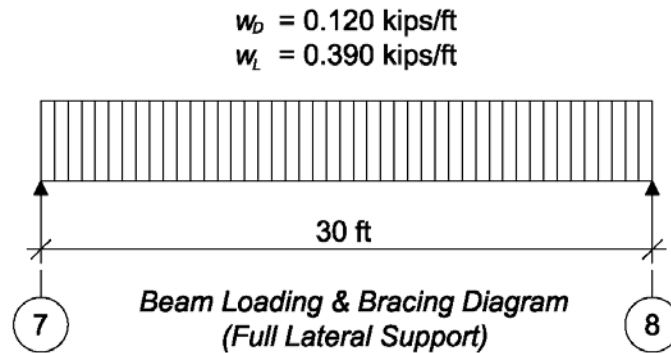
Calculate loads and select beams in the mechanical area

For the beams in the mechanical area, the mechanical units could weigh as much as 0.060 kip/ft<sup>2</sup>. Use 0.040 kip/ft<sup>2</sup> for additional live load, which will account for the mechanical units and any snow drift which could occur in the mechanical area. The beams in the mechanical area are at 6 ft on center.

Calculate minimum  $I_x$  to limit deflection to  $1/360 = 1$  in., because plaster ceiling will be used in the lobby area. Use 0.040 kip/ft<sup>2</sup> as an estimate of the snow load, including some drifting that could occur in this area, for deflection calculations.

IBC Table 1604.3

$$I_{req} \text{ (Live Load)} = \frac{0.240 \text{ kip/ft} (30 \text{ ft})^4}{1290 (1 \text{ in.})} = 151 \text{ in.}^4$$



Calculate the required strengths and select the beams in the mechanical area

LRFD	ASD
$W_u = 6 \text{ ft} [1.2 (0.020 \text{ kip/ft}^2) + 1.6 (0.025 \text{ kip/ft}^2 + 0.040 \text{ kip/ft}^2)] = 0.768 \text{ kip/ft}$	$W_a = 6 \text{ ft} (0.020 \text{ kip/ft}^2 + 0.025 \text{ kip/ft}^2 + 0.040 \text{ kip/ft}^2) = 0.510 \text{ kip/ft}$
$M_u = \frac{0.768 \text{ kip/ft} (30 \text{ ft})^2}{8} = 86.4 \text{ kip-ft}$	$M_a = \frac{0.510 \text{ kip/ft} (30 \text{ ft})^2}{8} = 57.4 \text{ kip-ft}$
Select a W14×22, which has an available moment strength of 125 kip-ft and an $I_x$ of 199 in. <sup>4</sup>	Select a W14×22, which has an available moment strength of 82.8 kip-ft and an $I_x$ of 199 in. <sup>4</sup>
$R_u = \frac{30 \text{ ft}}{2} (0.768 \text{ kip/ft}) = 11.5 \text{ kips}$	$R_a = \frac{30 \text{ ft}}{2} (0.510 \text{ kip/ft}) = 7.65 \text{ kips}$
$\phi_b V_n = 94.8 \text{ kips} > 11.5 \text{ kips}$ <b>o.k.</b>	$V_n / \Omega_b = 63.2 \text{ kips} > 7.65 \text{ kips}$ <b>o.k.</b>

Manual Table 3-2

Note: a W12×22 beam would also meet all criteria, but a 14 in. beam was selected so that consistent beam sizes could be 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. Because there is continuous glass underneath this window, limit live load deflection to 1/4-in. total movement. In addition, per AISC Design Guide 3, Second Edition, limit deflection due to spandrel weight to the L/480 or 3/8-in. maximum. In calculating the wall loads, the brick weight is taken as 55 lb/ft, and the glass weight is taken as 90 lb/ft. The spandrel panel weight is approximately

$$w = 7.50 \text{ ft} (0.055 \text{ kip/ft}^2) = 0.413 \text{ kip/ft.}$$

the dead load from the roof is equal to

$$w = 3.5 \text{ ft} (0.020 \text{ kip/ft}^2) = 0.070 \text{ kip/ft}$$

and the live load from the roof can be conservatively taken as

$$w = 3.5 \text{ ft} (0.025 \text{ kip/ft}^2 + 0.0132 \text{ kip/ft}^2) = 0.134 \text{ kip/ft.}$$

to account for the maximum snow drift as a uniform load

Assume the beams are simple spans of 22.5 ft.

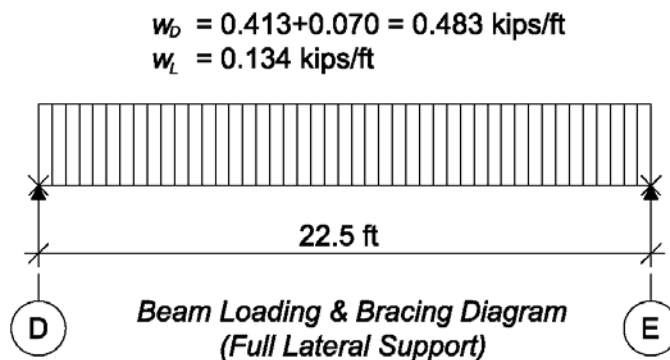
Calculate minimum  $I_x$  to limit live load deflection to 1/4 in.

$$I_{req} = \frac{0.134 \text{ kip/ft}(22.5 \text{ ft})^4}{1290(0.25 \text{ in.})} = 106 \text{ in.}^4$$

Calculate minimum  $I_x$  to limit spandrel weight deflection to 3/8 in.

$$I_{req} = \frac{0.413 \text{ kip/ft}(22.5 \text{ ft})^4}{1290(0.375 \text{ in.})} = 218 \text{ in.}^4$$

The loading diagram is as follows



Calculate the required strengths and select the beams for the roof ends

LRFD	ASD
$W_u = 1.2 (0.070 \text{ kip/ft}^2 + 0.413 \text{ kip/ft})$ $+ 1.6(0.134 \text{ kip/ft}^2)$ $= 0.794 \text{ kip/ft}$	$W_a = (0.070 \text{ kip/ft}^2 + 0.413 \text{ kip/ft})$ $+ 0.134 \text{ kip/ft}^2$ $= 0.617 \text{ kip/ft}$

$M_u = \frac{0.794 \text{ kip/ft}(22.5 \text{ ft})^2}{8} = 50.2 \text{ kip-ft}$ <p>Select W16×26, which has an available moment strength of 166 kip-ft and an <math>I_x</math> of 301 in.<sup>4</sup></p>	$M_a = \frac{0.617 \text{ kip/ft}(22.5 \text{ ft})^2}{8} = 39.0 \text{ kip-ft}$ <p>Select W16×26, which has an available moment strength of 110 kip-ft and an <math>I_x</math> of 301 in.<sup>4</sup></p>
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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. The spandrel beam is over glass, therefore, limit the live load deflection to ¼ in. In addition, per AISC Design Guide 3, Second Edition, limit deflection due to spandrel weight to the  $L/480$  or ⅜ in. maximum. 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 live load to this edge beam is equal to the joist end dead load and live load reaction. Treating this as a uniform load, divide this by the joist spacing.

$w_D = 2.76 \text{ kips}/6 \text{ ft} = 0.460 \text{ kip/ft.}$

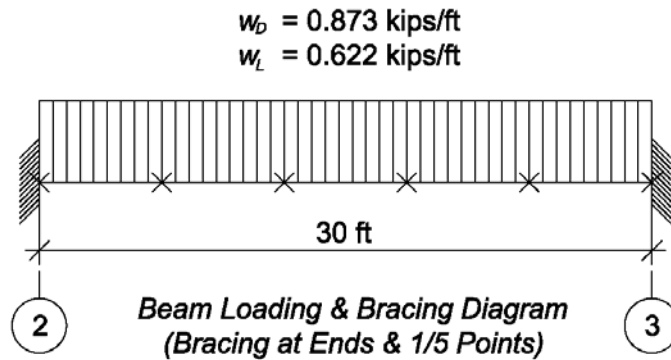
$w_L = 3.73 \text{ kips}/6 \text{ ft} = 0.622 \text{ kip/ft.}$

Calculate minimum  $I_x$  to limit the live load deflection to ¼ in.

$$I_{req} = \frac{(0.622 \text{ kip/ft})(30 \text{ ft})^4}{6450(0.25 \text{ in.})} = 312 \text{ in.}^4$$

Calculate minimum  $I_x$  to limit the spandrel weight deflection to ⅜ in.

$$I_{req} = \frac{(0.413 \text{ kip/ft})(30 \text{ ft})^4}{6450(0.375 \text{ in.})} = 138 \text{ in.}^4$$



Calculate the required strengths and select the beams for the roof sides

LRFD	ASD
$W_u = 1.2 (0.460 \text{ kip/ft}^2 + 0.413 \text{ kip/ft})$ $+ 1.6(0.622 \text{ kip/ft}^2)$ $= 2.04 \text{ kip/ft}$ $M_u = \frac{2.04 \text{ kip/ft}(30 \text{ ft})^2}{12} = 153 \text{ kip-ft}$ <p>For <math>L_b = 6 \text{ ft}</math> and <math>C_b = 1.0</math>, select W16×31, which has an available moment strength of 183 kip-ft and an <math>I_x</math> of 375 in.<sup>4</sup></p>	$W_a = (0.460 \text{ kip/ft}^2 + 0.413 \text{ kip/ft})$ $+ 0.622 \text{ kip/ft}^2$ $= 1.50 \text{ kip/ft}$ $M_a = \frac{1.50 \text{ kip/ft}(30 \text{ ft})^2}{12} = 113 \text{ kip-ft}$ <p>For <math>L_b = 6 \text{ ft}</math> and <math>C_b = 1.0</math>, select W16×31, which has an available moment strength of 122 kip-ft and an <math>I_x</math> of 375 in.<sup>4</sup></p>

Manual  
Table 3-10

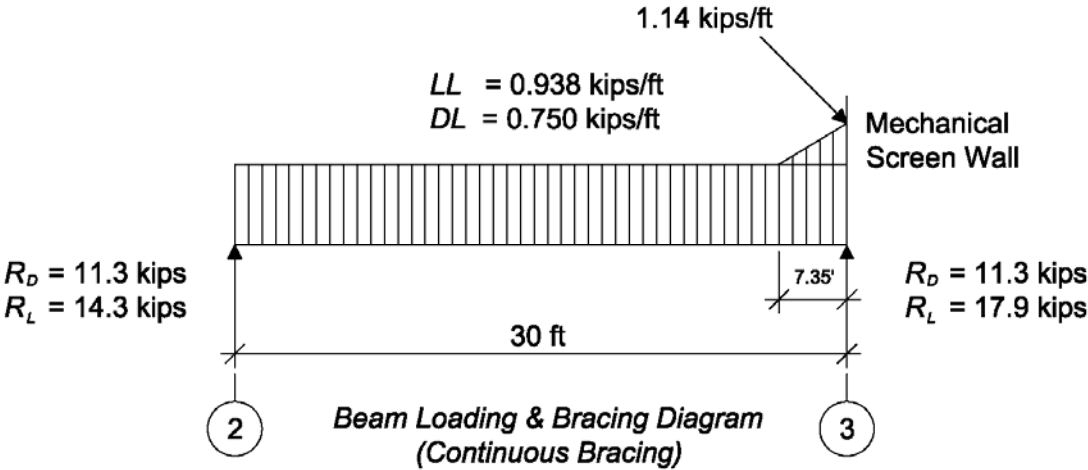
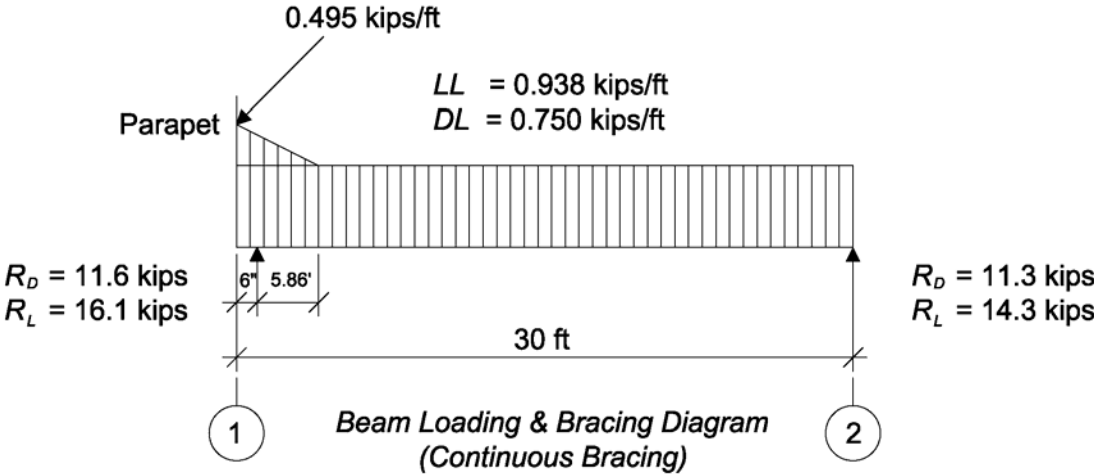
Note: This roof beam may need to be upsized during the lateral load analysis to increase the stiffness and strength of the member and improve lateral frame drift performance.

**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: by footnote G, for steel structural members, the dead load shall be taken as zero for D + L deflection calculation combination.



Summary of required strengths and required moment of inertia

LRFD	ASD	
Grids 1 to 2 and 7 to 8(opposite hand)	Grids 1 to 2 and 7 to 8(opposite hand)	
$R_u$ (left) = 1.2(11.6 kips) + 1.6(16.1 kips) = 38.8 kips	$R_a$ (left) = 11.6 kips + 15.6 kips = 27.2 kips	
$R_u$ (right) = 1.2(11.3 kips) + 1.6(14.2 kips) = 36.1 kips	$R_a$ (right) = 11.3 kips + 14.2 kips = 25.5 kips	
$M_u$ = 1.2(84.3 kip-ft) + 1.6(107 kip-ft) = 272 kip-ft	$M_a$ = 84.3 kip-ft + 107 kip-ft = 191 kip-ft	Manual Table 3-10
$I_{xreq'd} = \frac{(0.938 \text{ klf})(30 \text{ ft})^4}{1290 (1.5 \text{ in.})}$ = 393 in. <sup>4</sup>	$I_{xreq'd} = \frac{(0.938 \text{ klf})(30 \text{ ft})^4}{1290 (1.5 \text{ in.})}$ = 393 in. <sup>4</sup>	
For $L_b = 6$ ft and $C_b = 1.0$ , select W21×44 which has an available moment of 333 kip-ft and $I_x = 843$ in. <sup>4</sup>	For $L_b = 6$ ft and $C_b = 1.0$ , select W21×44 which has an available moment of 222 kip-ft and $I_x = 843$ in. <sup>4</sup>	Manual Table 3-10
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(14.3 kips) = 36.6 kips	$R_a$ (left) = 11.3 kips + 14.3 kips = 25.6 kips	
$R_u$ (right) = 1.2(11.3 kips) + 1.6(17.9 kips) = 42.1 kips	$R_a$ (right) = 11.3 kips + 17.9 kips = 29.2 kips	
$M_u$ = 1.2(84.4 kip-ft) + 1.6(109 kip-ft) = 278 kip-ft	$M_a$ = 84.4 kip-ft + 109 kip-ft = 195 kip-ft	
$I_{xreq'd} = \frac{(0.938 \text{ klf})(30 \text{ ft})^4}{1290 (1.5 \text{ in.})}$ = 393 in. <sup>4</sup>	$I_{xreq'd} = \frac{(0.938 \text{ klf})(30 \text{ ft})^4}{1290 (1.5 \text{ in.})}$ = 393 in. <sup>4</sup>	
For $L_b = 6$ ft and $C_b = 1.0$ , select W21×44 which has an available moment of 333 kip-ft and $I_x = 843$ in. <sup>4</sup>	For $L_b = 6$ ft and $C_b = 1.0$ , select W21×44 which has an available moment of 222 kip-ft and $I_x = 843$ in. <sup>4</sup>	

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.

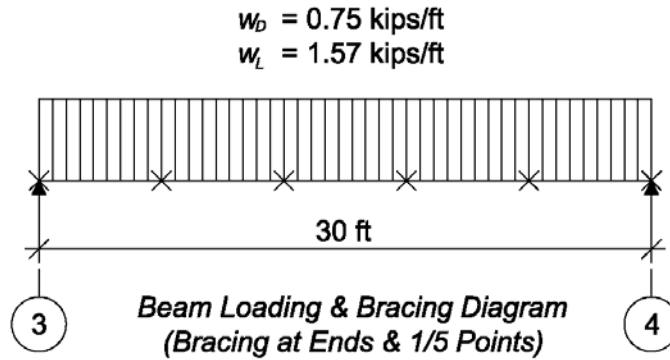
**SELECT THE BEAMS ALONG THE SIDES OF THE MECHANICAL AREA**

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: by footnote g, for steel structural members, the dead load shall be taken as zero for D + L deflection calculation load combination.

IBC Table 1604.3



LRFD	ASD
$W_u = 1.2 (0.750 \text{ kip/ft}^2) + 1.6(1.57 \text{ kip/ft}^2) = 3.41 \text{ kip/ft}$	$W_a = 0.750 \text{ kip/ft} + 1.57 \text{ kip/ft}^2 = 2.32 \text{ kip/ft}$
$M_u = \frac{3.41 \text{ kip/ft}(30 \text{ ft})^2}{8} = 384 \text{ kip-ft}$	$M_a = \frac{2.32 \text{ kip/ft}(30 \text{ ft})^2}{8} = 261 \text{ kip-ft}$
$R_u = \frac{30 \text{ ft}}{2}(3.41 \text{ kip/ft}) = 51.2 \text{ kips}$	$R_u = \frac{30 \text{ ft}}{2}(2.32 \text{ kip/ft}) = 34.8 \text{ kips}$
$I_{x \text{ req'd}} = \frac{1.57 \text{ kip/ft}(30.0 \text{ ft})^4}{1290(1.50 \text{ in.})} = 393 \text{ in.}^4$	$I_{x \text{ req'd}} = \frac{1.57 \text{ kip/ft}(30.0 \text{ ft})^4}{1290(1.50 \text{ in.})} = 393 \text{ in.}^4$
For $L_b = 6 \text{ ft}$ and $C_b = 1.0$ , select W21×48, which has an available moment strength of 398 kip-ft, an available end shear of 217 kips and an $I_x$ of 959 in. <sup>4</sup>	For $L_b = 6 \text{ ft}$ and $C_b = 1.0$ , select W21×48, which has an available moment strength of 265 kip-ft, an available end shear of 144 kips and an $I_x$ of 959 in. <sup>4</sup>

Manual Table 3-10



**FLOOR MEMBER DESIGN AND SELECTION**

*Calculate dead load and live load*

Dead Load		
Slab and Deck	= 0.057 kip/ft <sup>2</sup>	Steel Deck Institute <i>Diaphragm Design Manual</i>
Beams (est.)	= 0.008 kip/ft <sup>2</sup>	
Misc. ( ceiling, mechanical, etc.)	= 0.010 kip/ft <sup>2</sup>	
<u>Total</u>	<u>= 0.075 kip/ft<sup>2</sup></u>	
Live Load		
Total (can be reduced for area per IBC 2003)	= 0.080 kip/ft <sup>2</sup>	IBC Section 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 general layout for the floor beams is 10 ft on center. At 10 ft on center, this deck has an allowable superimposed live load capacity of 0.143 kip/ft<sup>2</sup>. 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 0.020 kip/ft<sup>2</sup> construction live load (per ASCE 37-02), when designed by the ASCE 7 load combinations and the provisions of Chapter F of the Specification.

**SELECT TYPICAL 45 FT 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.

$$I_{req} \approx \frac{0.65 \text{ kip/ft}(45 \text{ ft})^4}{1290(2.0 \text{ in.})} = 1030 \text{ in.}^4$$

*Determine minimum strength to carry wet concrete and construction live load*

$$w_{DL} = 0.065 \text{ kip/ft}^2(10 \text{ ft}) = 0.65 \text{ kip/ft}$$

$$w_{LL} = 0.020 \text{ kip/ft}^2(10 \text{ ft}) = 0.20 \text{ kip/ft}$$

*Determine the required moment strength due to wet concrete only*

LRFD	ASD
$W_u = 1.4(0.65 \text{ kip/ft}) = 0.91 \text{ kip/ft}$ $M_u = \frac{0.91 \text{ kip/ft}(45 \text{ ft})^2}{8} = 230 \text{ kip-ft}$	$W_a = 0.65 \text{ kip/ft}$ $M_a = \frac{0.65 \text{ kip/ft}(45 \text{ ft})^2}{8} = 165 \text{ kip-ft}$

*Determine the required moment strength due to wet concrete and construction live load*

LRFD	ASD
$W_u = 1.2(0.65 \text{ kip/ft}) + 1.6(0.20 \text{ kip/ft})$ $= 1.10 \text{ kip/ft}$ $M_u = \frac{1.10 \text{ kip/ft}(45 \text{ ft})^2}{8} = 278 \text{ kip-ft}$ <b>controls</b>	$W_a = 0.65 \text{ kip/ft} + 0.20 \text{ kip/ft}$ $= 0.85 \text{ kip/ft}$ $M_a = \frac{0.85 \text{ kip/ft}(45 \text{ ft})^2}{8} = 215 \text{ kip-ft}$ <b>Controls</b>

Use Manual Table 3-2 to select a beam with  $I_x \geq 1030 \text{ in.}^4$ . Select W21×50, which has  $I_x = 984 \text{ in.}^4$ , close to our target value, and has an available moment strength 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

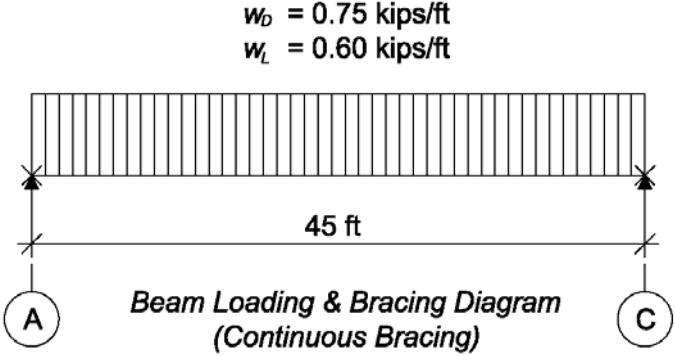
The beams are at 10 ft on center, therefore the area  $A_T = 45 \text{ ft} \times 10 \text{ ft} = 450 \text{ ft}^2$ .

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right) = 0.060 \text{ kip/ft}^2 \geq 0.50L_o = 0.040 \text{ kip/ft}^2$$

IBC Eqn  
16-21

Therefore use 0.060 kip/ft<sup>2</sup>.  
 The beam is continuously braced by the deck.

The beams are at 10 ft on center, therefore the loading diagram is as shown below.



Calculate the required moment strength

LRFD	ASD
$W_u = 1.2(0.75 \text{ kip/ft}) + 1.6(0.60 \text{ kip/ft})$ $= 1.86 \text{ kip/ft}$	$W_a = 0.75 \text{ kip/ft} + 0.60 \text{ kip/ft}$ $= 1.35 \text{ kip/ft}$
$M_u = \frac{1.86 \text{ kip/ft}(45 \text{ ft})^2}{8} = 471 \text{ kip-ft}$	$M_a = \frac{1.35 \text{ kip/ft}(45 \text{ ft})^2}{8} = 342 \text{ kip-ft}$

$Y_2 = 6 \text{ in.} - \frac{1}{2} \text{ in.} = 5.5 \text{ in.}$

Use Manual Table 3-19 to check W21×50 selected above. Using values of 471kip-ft (LRFD) or 342 kip-ft (ASD) and a Y2 value of 5.5 in

Manual Table 3-19

LRFD	ASD
Select W21×50 beam, where PNA = 7, $Q_n = 184 \text{ kips}$ $\phi_b M_p = 599 \text{ kip-ft} > 471 \text{ kip-ft}$ <b>o.k.</b>	Select W21×50 beam, where PNA = 7, $Q_n = 184 \text{ kips}$ $M_p/\Omega_b = 399 \text{ kip-ft} > 342 \text{ kip-ft}$ <b>o.k.</b>

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. I.3.1.1a

- (1) one-eighth of the span of the beam, center to center of supports

$\frac{45 \text{ ft}}{8} (2 \text{ sides}) = 11.3 \text{ ft.}$

- (2) one-half the distance to the center line of the adjacent beam

$\frac{10 \text{ ft}}{2} (2 \text{ sides}) = 10.0 \text{ ft.}$  **controls**

- (3) the distance to the edge of the slab  
 Not applicable

Check a

LRFD	ASD
$a = \frac{\sum Q_n}{0.85 f'_c b}$ $= \frac{184 \text{ kips}}{0.85(4 \text{ ksi})(10 \text{ ft})(12 \text{ in./ft})}$ $= 0.451 \text{ in.} < 1.0 \text{ in.} \quad \mathbf{o.k.}$	$a = \frac{\sum Q_n}{0.85 f'_c b}$ $= \frac{184 \text{ kips}}{0.85(4 \text{ ksi})(10 \text{ ft})(12 \text{ in./ft})}$ $= 0.451 \text{ in.} < 1.0 \text{ in.} \quad \mathbf{o.k.}$

Check end shear strength

LRFD	ASD
$R_u = \frac{45 \text{ ft}}{2} (1.86 \text{ kip/ft}) = 41.9 \text{ kips}$ $\phi_b V_n = 217 \text{ kips} > 41.9 \text{ kips} \quad \mathbf{o.k.}$	$R_a = \frac{45 \text{ ft}}{2} (1.35 \text{ kip/ft}) = 30.4 \text{ kips}$ $V_n / \Omega_b = 145 \text{ kips} > 30.4 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 3-2

Check live load deflection

$$\Delta_{LL} = l/360 = ((45 \text{ ft})(12 \text{ in./ft}))/360 = 1.5 \text{ in.}$$

IBC Table  
1604.3

LRFD	ASD
$W21 \times 50: Y2 = 5.5 \text{ in.}, \text{PNA} = 7$ $I_{LB} = 1640 \text{ in.}^4$ $\Delta_{LL} = \frac{w_{LL} l^4}{1290 I_{LB}} = \frac{0.60 \text{ kip/ft}(45 \text{ ft})^4}{1290(1730 \text{ in.}^4)}$ $= 1.10 \text{ in.} < 1.5 \text{ in.} \quad \mathbf{o.k.}$	$W21 \times 50: Y2 = 5.5 \text{ in.}, \text{PNA} = 7$ $I_{LB} = 1640 \text{ in.}^4$ $\Delta_{LL} = \frac{w_{LL} l^4}{1290 I_{LB}} = \frac{0.60 \text{ kip/ft}(45 \text{ ft})^4}{1290(1730 \text{ in.}^4)}$ $= 1.16 \text{ in.} < 1.5 \text{ in.} \quad \mathbf{o.k.}$

Manual  
Table 3-20

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
$\frac{\sum Q_n}{Q_n} = \frac{184 \text{ kips}}{17.2 \text{ kips/stud}} = 10.7 \text{ studs / side}$	$\frac{\sum Q_n}{Q_n} = \frac{184 \text{ kips}}{17.2 \text{ kips/stud}} = 10.7 \text{ studs / side}$

Therefore use 22 studs.

**SELECT TYPICAL 30 FT COMPOSITE (OR NON-COMPOSITE) BEAM  
(10 FT ON CENTER)**

Find a target moment of inertia for an un-shored beam

Hold deflection to around 1.5 in. maximum to facilitate concrete placement.

$$I_{req} \approx \frac{0.65 \text{ kip/ft}(30 \text{ ft})^4}{1290(1.50 \text{ in.})} = 272 \text{ in.}^4$$

Determine minimum strength to carry wet concrete and construction live load

$$w_{DL} = 0.065 \text{ kip/ft}^2(10 \text{ ft}) = 0.65 \text{ kip/ft}$$

$$w_{LL} = 0.020 \text{ kip/ft}^2(10 \text{ ft}) = 0.20 \text{ kip/ft}$$

Determine the required moment strength due to wet concrete only

LRFD	ASD
$W_u = 1.4(0.65 \text{ kip/ft}) = 0.91 \text{ kip/ft}$	$W_a = 0.65 \text{ kip/ft}$
$M_u = \frac{0.91 \text{ kip/ft}(30 \text{ ft})^2}{8} = 102 \text{ kip-ft}$	$M_a = \frac{0.65 \text{ kip/ft}(30 \text{ ft})^2}{8} = 73.1 \text{ kip-ft}$

Determine the required moment strength due to wet concrete and construction live load

LRFD	ASD
$W_u = 1.2(0.65 \text{ kip/ft}) + 1.6(0.20 \text{ kip/ft})$ $= 1.10 \text{ kip/ft}$	$W_a = 0.65 \text{ kip/ft} + 0.20 \text{ kip/ft}$ $= 0.85 \text{ kip/ft}$
$M_u = \frac{1.10 \text{ kip/ft}(30 \text{ ft})^2}{8} = 124 \text{ kip-ft}$	$M_a = \frac{0.85 \text{ kip/ft}(30 \text{ ft})^2}{8} = 95.6 \text{ kip-ft}$
<b>Controls</b>	<b>controls</b>

Use Manual Table 3-2 to find a beam with an  $I_x \geq 272 \text{ in.}^4$ . Select W16×26, which has an  $I_x = 301 \text{ in.}^4$  which exceeds our target value, and has an available moment 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  $K_{LL} = 2$

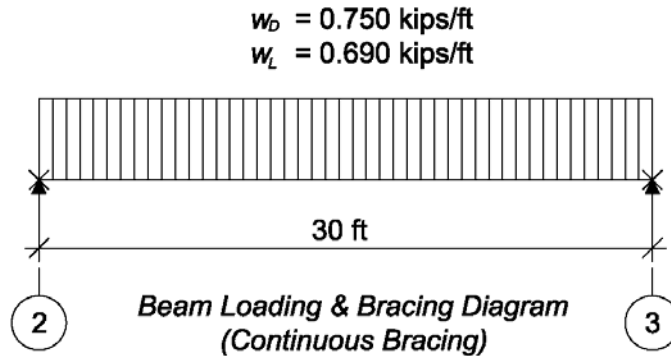
The beams are at 10 ft on center, therefore the area  $A_T = 30 \text{ ft} \times 10 \text{ ft} = 300 \text{ ft}^2$ .

IBC Eqn  
16-21

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right) = 0.069 \text{ kip/ft}^2 \geq 0.50 L_o = 0.040 \text{ kip/ft}^2$$

Therefore, use  $0.069 \text{ kip/ft}^2$

The beams are at 10 ft on center, therefore the loading diagram is as shown below.



Calculate the required strength

LRFD	ASD
$W_u = 1.2(0.75 \text{ kip/ft}) = 1.6 \text{ (0.69 kip/ft)}$ $= 2.00 \text{ kip/ft}$	$W_a = 0.750 \text{ kip/ft} + 0.690 \text{ kip/ft}$ $= 1.44 \text{ kip/ft}$
$M_u = \frac{2.00 \text{ kip/ft}(30 \text{ ft})^2}{8} = 225 \text{ kip-ft}$	$M_a = \frac{1.44 \text{ kip/ft}(30 \text{ ft})^2}{8} = 162 \text{ kip-ft}$

From previous calculation,  $Y2 = 5.5 \text{ in.}$

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 4.5 in

Manual  
Table 3-19

LRFD	ASD
Select W16×26 beam, where  PNA = 7 and $\Sigma Q_n = 96.0 \text{ kips}$  $\phi_b M_p = 248 \text{ kip-ft} > 225 \text{ kip-ft}$ <b>o.k.</b>	Select W16×26 beam, where  PNA = 7 and $\Sigma Q_n = 96.0 \text{ kips}$  $M_p/\Omega_b = 165 \text{ kip-ft} > 162 \text{ kip-ft}$ <b>o.k.</b>

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. I.3.1.1a

(1) one-eighth of the span of the beam, center to center of supports

$$\frac{30 \text{ ft}}{8} (2 \text{ sides}) = 7.50 \text{ ft.} \quad \text{Controls}$$

(2) one-half the distance to the center line of the adjacent beam

$$\frac{10 \text{ ft}}{2} (2 \text{ sides}) = 10.0 \text{ ft.}$$

(3) the distance to the edge of the slab

Not applicable

Check  $a$

LRFD	ASD
$a = \frac{\Sigma Q_n}{0.85 f'_c b}$ $= \frac{96.0 \text{ kips}}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$ $= 0.313 \text{ in.} < 1.0 \text{ in.} \quad \text{o.k.}$	$a = \frac{\Sigma Q_n}{0.85 f'_c b}$ $= \frac{145 \text{ kips}}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$ $= 0.473 \text{ in.} < 1.0 \text{ in.} \quad \text{o.k.}$

Check end shear strength

LRFD	ASD
$R_u = \frac{30 \text{ ft}}{2}(2.00 \text{ kip/ft}) = 30.0 \text{ kips}$ $\phi_b V_n = 106 \text{ kips} > 30.0 \text{ kips} \quad \mathbf{o.k.}$	$R_a = \frac{30 \text{ ft}}{2}(1.44 \text{ kip/ft}) = 21.6 \text{ kips}$ $V_n/\Omega_b = 70.5 \text{ kips} > 21.6 \text{ kips} \quad \mathbf{o.k.}$

Manual  
Table 3-2

Check live load deflection

$$\Delta_{LL} = l/360 = ((30 \text{ ft})(12 \text{ in./ft})/360 = 1.0 \text{ in.}$$

IBC Table  
1604.3

LRFD	ASD
W16×26  Y2 = 5.5 in. PNA = 7 $I_{LB} = 535 \text{ in.}^4$  $\Delta_{LL} = \frac{w_{LL} l^4}{1290 I_{LB}} = \frac{0.69 \text{ kip/ft}(30 \text{ ft})^4}{1290(535 \text{ in.}^4)}$ = 0.753 in. < 1.0 in. <b>o.k.</b>	W16×26  Y2 = 5.5 in. PNA = 6 $I_{LB} = 622 \text{ in.}^4$  $\Delta_{LL} = \frac{w_{LL} l^4}{1290 I_{LB}} = \frac{0.69 \text{ kip/ft}(30 \text{ ft})^4}{1290(622 \text{ in.}^4)}$ = 0.753 in. < 1.0 in. <b>o.k.</b>

Manual  
Table 3-20

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;  $Q_n = 17.2 \text{ kips/stud}$

Manual  
Table 3-21

LRFD	ASD
$\frac{\sum Q_n}{Q_n} = \frac{96.0 \text{ kips}}{17.2 \text{ kips/stud}} = 5.58 \text{ studs / side}$ Use 12 studs at about 3'-0" o.c.	$\frac{\sum Q_n}{Q_n} = \frac{96.0 \text{ kips}}{17.2 \text{ kips/stud}} = 5.58 \text{ studs / side}$ Use 17 studs

Note: There is a maximum spacing of  $8(6 \text{ in.}) = 4' - 0''$  or  $3' - 0''$  between studs.

Specification  
Sec. I3.2d(6)

Therefore use 12 studs, at no more than 3'-0" on center.

Note: 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 $\phi_b M_p = 249 \text{ kip-ft} > 225 \text{ kip-ft} \quad \mathbf{o.k.}$ $\phi_v V_n = 159 > 30.0 \text{ kips} \quad \mathbf{o.k.}$	Select W18×35 $M_p/\Omega_b = 166 \text{ kip-ft} > 162 \text{ kip-ft} \quad \mathbf{o.k.}$ $V_n/\Omega_v = 106 \text{ kips} > 21.6 \text{ kips} \quad \mathbf{o.k.}$

*Check beam deflections*

Check W18×35 with an  $I_x = 510 \text{ in.}^4$ , with only wet concrete on beams.

Manual  
Table 3-1

Note: Because this beam is stronger than the W16×26 composite beam, no wet concrete strength checks are required.

$$\Delta_{DL}(\text{wet concrete}) = \frac{0.65 \text{ kip/ft}(30 \text{ ft})^4}{1290(510 \text{ in.}^4)} = 0.80 \text{ in.} \quad \mathbf{o.k.}$$

Note: A good break point to eliminate camber is  $\frac{3}{4}$  in. If a W18×40 ( $I_x = 612 \text{ in.}^4$ ),  $\Delta_{DL}(\text{wet concrete}) = 0.615 \text{ in.}$  ) was selected, no camber is required.

Per previous smaller beam calculation, the live load deflection is **o.k.**

Therefore selecting a W18×40 would 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 in making this determination.

**SELECT TYPICAL END BEAM**

The influence area for these beams is less than  $400 \text{ ft}^2$ , therefore no live load reduction can be taken.

These beams carry 5.5 ft of dead load and live load as well as a wall load.

The dead load is

$$w = 5.5 \text{ ft}(0.075 \text{ kips/ft}) = 0.413 \text{ kip/ft.}$$

The dead load of the wall system at the floor is

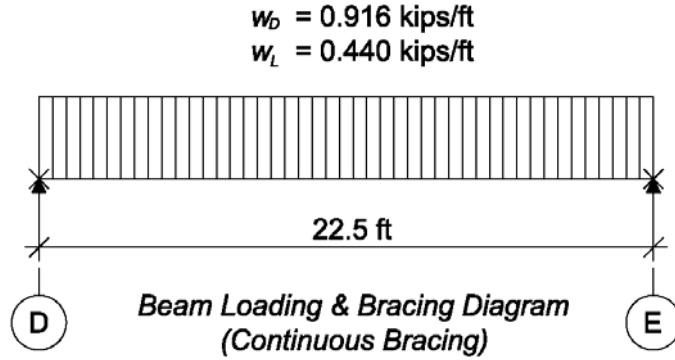
$$\begin{aligned} w &= 7.5 \text{ ft}(0.055 \text{ kip/ft} + 6.0 \text{ ft}( 0.015 \text{ kip/ft})) \\ &= 0.413 \text{ kip/ft} + 0.090 \text{ kip/ft} \\ &= 0.503 \text{ kip/ft.} \end{aligned}$$

The total dead load is  $w = 0.916 \text{ kip/ft.}$

The live load is  $w = 5.5 \text{ ft}(0.080 \text{ kip/ft}) = 0.440 \text{ kip/ft.}$



The loading diagram is as follows



Calculate the required strengths

LRFD	ASD
$W_u = 1.2(0.916 \text{ kip/ft}) + 1.6 (0.440 \text{ kip/ft})$ $= 1.80 \text{ kip/ft}$	$W_a = 0.916 \text{ kip/ft} + 0.440 \text{ kip/ft}$ $= 1.36 \text{ kip/ft}$
$M_u = \frac{1.80 \text{ kip/ft}(22.5 \text{ ft})^2}{8} = 114 \text{ kip-ft}$	$M_a = \frac{1.36 \text{ kip/ft}(22.5 \text{ ft})^2}{8} = 85.8 \text{ kip-ft}$
$R_u = \frac{22.5 \text{ ft}}{2} (1.80 \text{ kip/ft}) = 20.3 \text{ kips}$	$R_a = \frac{22.5 \text{ ft}}{2} (1.36 \text{ kip/ft}) = 15.3 \text{ kips}$

Because these beams are less than 25-ft long, they will be most efficient as non-composite beams. The beams at the ends of the building carry a brick spandrel panel. Because there is continuous glass underneath this spandrel, limit live load deflection to 1/4 in. total movement. In addition, per AISC Design Guide 3, Second Edition, limit deflection due to spandrel weight to L/480 or 3/8-in. maximum.

Calculate minimum  $I_x$  to limit live load deflection to 1/4 in.

$$I_{req} = \frac{0.440 \text{ kip/ft}(22.5 \text{ ft})^4}{1290(0.25 \text{ in.})} = 350 \text{ in.}^4 \text{ controls}$$

Calculate minimum  $I_x$  to limit spandrel weight deflection to 3/8 in.

$$I_{req} = \frac{0.413 \text{ kip/ft}(22.5 \text{ ft})^4}{1290(0.375 \text{ in.})} = 218 \text{ in.}^4$$

Select Beam from Manual Table 3-2

LRFD	ASD
Select W16×31 with an $I_x = 375 \text{ in.}^4$ $\phi_b M_p = 203 \text{ kip-ft} > 114 \text{ kip-ft}$ <b>o.k.</b> $\phi_v V_n = 131 > 20.3 \text{ kips}$ <b>o.k.</b>	Select W16×31 with an $I_x = 375 \text{ in.}^4$ $M_p/\Omega_b = 135 \text{ kip-ft} > 85.8 \text{ kip-ft}$ <b>o.k.</b> $V_n/\Omega_v = 87.3 \text{ kips} > 15.3 \text{ kips}$ <b>o.k.</b>

Manual  
Table 3-2

### SELECT TYPICAL SIDE BEAM

The beams along the side of the building carry the spandrel panel and glass, and dead load and live load from the intermediate floor beams. The spandrel beam is over glass below, therefore, limit the live load deflection to  $\frac{1}{4}$  in. In addition, per Steel Design Guide 3, Second Edition, limit deflection due to spandrel weight to the  $L/480$  or  $\frac{3}{8}$ -in. maximum. 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.

#### *Establish loads*

The dead load reaction from the floor beams is

$$P = 0.75 \text{ kip/ft} (45 \text{ ft} / 2) = 16.9 \text{ kips}$$

The uniform dead load along the beam is

$$w = 0.50 \text{ ft} (0.075 \text{ kip/ft}^2) + 0.503 \text{ kip/ft} = 0.541 \text{ kip/ft.}$$

#### *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 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

IBC Table  
1607.9.1

$$K_{LL} = 1.98.$$

IBC Eqn  
16-21

The  $A_T = 30 \text{ ft} \times (22.5 \text{ ft} + 0.5 \text{ ft}) = 690 \text{ ft}^2$ .

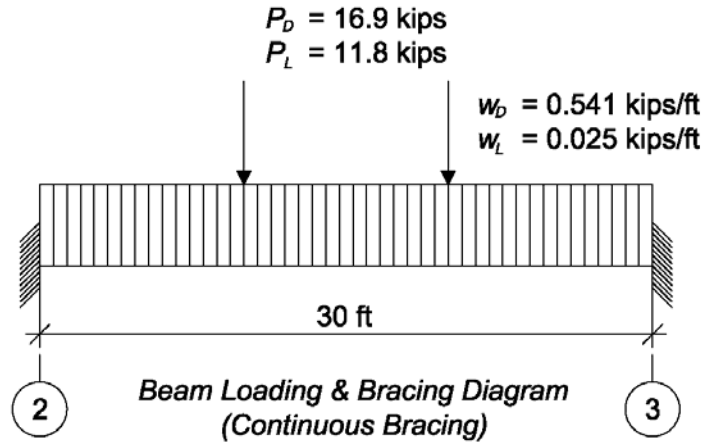
$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right) = 0.0523 \text{ kip/ft}^2 \geq 0.50 L_o = 0.040 \text{ kip/ft}^2$$

Therefore, use  $0.0523 \text{ kip/ft}^2$ .

The live load from the floor beams is  $P = 0.523 \text{ kip/ft}(45 \text{ ft} / 2) = 11.8 \text{ kips}$

The uniform live load along the beam is  $w = 0.50 \text{ ft} (0.0523 \text{ kip/ft}^2) = 0.025 \text{ kip/ft.}$

The loading diagram is shown below.



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 $\Delta_{LL} = 0.177$ in. < 1.0 in. <b>o.k.</b>	Typical side beam $R_a = 37.2$ kips $M_a$ at ends = 234 kip-ft $M_a$ at ctr. = 117 kip-ft $\Delta_{LL} = 0.177$ in. < 1.0 in. <b>o.k.</b>

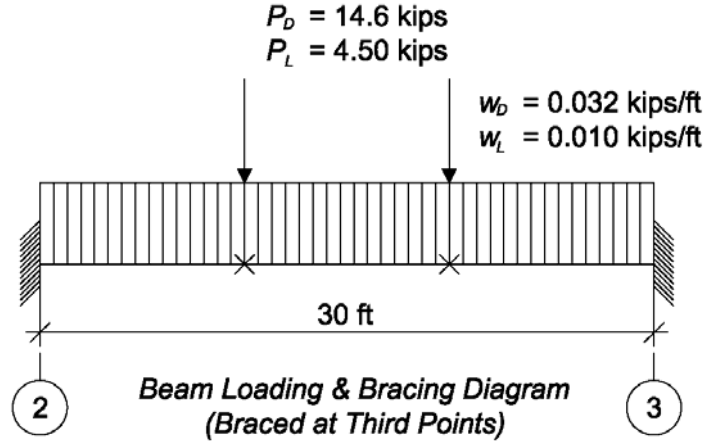
Calculate minimum  $I_x$  to limit the spandrel weight deflection to  $\frac{3}{8}$  in.

$$I_{req} = \frac{(0.503 \text{ kip/ft})(30 \text{ ft})^4}{6450(0.375 \text{ in.})} = 168 \text{ in.}^4$$

LRFD	ASD
Typical side beam Select W21×44 which has an available moment of 358 kip-ft and an $I_x$ of 843 in. <sup>4</sup>	Typical side beam Select W21×44 which has an available moment of 238 kip-ft and an $I_x$ of 843 in. <sup>4</sup>

Manual  
Table 3-2

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 0.020 kip/ft<sup>2</sup> will be present. This load pattern and a summary of the moments and reactions is shown below.



LRFD	ASD
Typical side beam with wet concrete only $R_u = 21.1$ kips $M_u$ at ends = 140 kip-ft $M_u$ at ctr. = 70.0 kip-ft	Typical side beam with wet concrete only $R_a = 15.1$ kips $M_a$ at ends = 99.9 kip-ft $M_a$ at ctr. = 50.0 kip-ft

$\Delta_{LL} = 0.220$  in. < 1.0 in. **o.k.**

LRFD	ASD
Typical side beam with wet concrete and construction load $R_u = 25.6$ kips $M_u$ at ends = 169 kip-ft <b>controls</b> $M_u$ at ctr. = 84.5 kip-ft	Typical side beam wet concrete and construction load $R_a = 19.8$ kips $M_a$ at ends = 131 kip-ft <b>controls</b> $M_a$ at ctr. = 65.3 kip-ft

$\Delta_{LL} = 0.288$  in. < 1.0 in. **o.k.**

LRFD	ASD
The available strength of a W21×44 with an unbraced length of 10 ft, and a conservative value of $C_b = 1.0$ is 262 kip-ft.  262 kip-ft > 169 kip-ft <b>o.k.</b>	The available strength of a W21×44 with an unbraced length of 10 ft, and a conservative value of $C_b = 1.0$ is 175 kip-ft.  175 kip-ft > 131 kip-ft <b>o.k.</b>

Manual  
Table 3-10

Note: 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 INTERIOR BEAM**

*Establish loads*

The dead load reaction from the floor beams is  $P = 0.75 \text{ kip/ft}(37.5 \text{ ft}) = 28.1 \text{ kips}$

*Check for live load reduction due to area*

For interior beams,  $K_{LL} = 2$

IBC Table  
1607.9.1

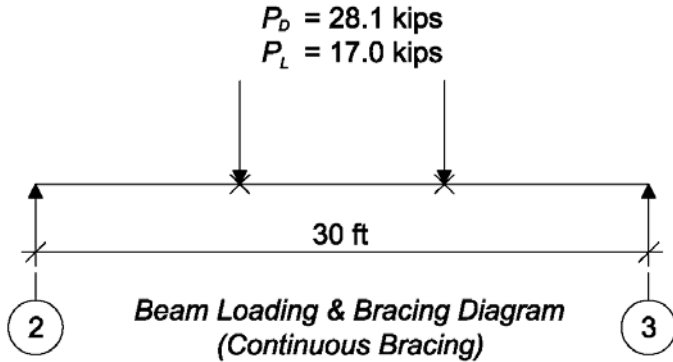
The  $A_T = 30 \text{ ft} \times 37.5 \text{ ft} = 1125 \text{ ft}^2$ .

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} A_T}} \right) = 0.0453 \text{ kip/ft}^2 \geq 0.50 L_o = 0.040 \text{ kip/ft}^2$$

IBC Eqn.  
16-21

Therefore, use  $0.0453 \text{ kip/ft}^2$

The live load from the floor beams is  $P = 0.0453 \text{ kip/ft}^2(10 \text{ ft})(37.5 \text{ ft}) = 17.0 \text{ kips}$



Note: The dead load for this beam is included in the assumed overall dead load.

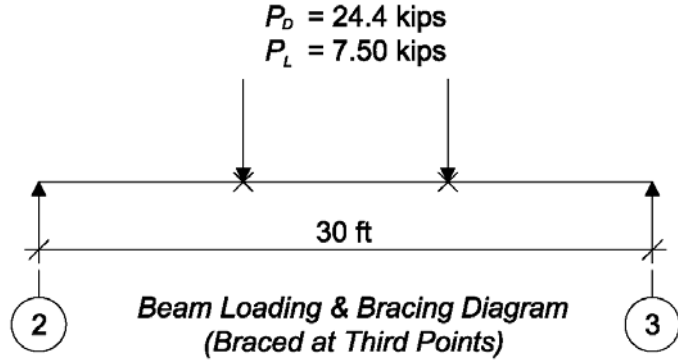
A summary of the simple moments and reactions is shown below:

*Calculate the total loads, moments and shears, and select the beams for the interior beams*

LRFD	ASD
Typical interior beam $R_u = 60.9 \text{ kips}$ $M_u = 609 \text{ kip-ft}$	Typical interior beam $R_a = 45.1 \text{ kips}$ $M_a = 451 \text{ kip-ft}$

*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  $0.020 \text{ kip/ft}^2$  will be present. This load pattern and a summary of the moments and reactions, and deflection requirements is shown below. Limit wet concrete deflection to 1.5 in.



LRFD	ASD
Typical Interior Beam with wet concrete only $R_u = 34.1$ kips $M_u = 341$ kip-ft	Typical interior Beam with wet concrete only $R_a = 24.4$ kips $M_a = 207$ kip-ft

Target  $I_x \geq 658$  in.<sup>4</sup>

LRFD	ASD
Typical Interior Beam with wet concrete and construction load  $R_u = 41.3$ kips $M_u$ (midspan) = 413 kip-ft  Select beam with an unbraced length of 10 ft and a conservative $C_b = 1.0$  Select W21×68, which has an available moment strength of 532 kip-ft  $\phi_b M_p = 532$ kip-ft > 413 kip-ft <b>o.k.</b>	Typical Interior Beam with wet concrete and construction load  $R_a = 31.9$ kips $M_a$ (midspan) = 319 kip-ft  Select beam with an unbraced length of 10 ft and a conservative $C_b = 1.0$  Select W21×68, which has an available moment strength of 354 kip-ft  $M_p/\Omega_b = 354$ kip-ft $\approx$ 319 kip-ft <b>o.k.</b>

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

$Y_2$  (from previous calculations) = 5.5 in.

Use Manual Table 3-19, check W21×68, using required strengths of 609 kip-ft (LRFD) and 451 kip-ft (ASD) and  $Y_2$  value of 5.5 in.

LRFD	ASD
Select a W21×68 Where PNA = 7, $\sum Q_n = 251$ kips $\phi_b M_n = 847$ kip-ft > 609 kip-ft <b>o.k.</b>	Select a W21×68 Where PNA = 7, $\sum Q_n = 251$ kips $M_n/\Omega_b = 564$ kip-ft > 451 kip-ft <b>o.k.</b>

Manual Table 3-19

This beam has an  $I_x = 1480 \text{ in.}^4 > 658 \text{ in.}^4$   
 Therefore, do not camber.

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. I.3.1.1a

(1) one-eighth of the span of the beam, center to center of supports

$$\frac{30 \text{ ft}}{8} (2 \text{ sides}) = 7.50 \text{ ft.} \quad \text{Controls}$$

(2) one-half the distance to the center line of the adjacent beam

$$\left( \frac{45 \text{ ft}}{2} + \frac{30 \text{ ft}}{2} \right) = 37.5 \text{ ft.}$$

(3) the distance to the edge of the slab  
 Not applicable.

LRFD	ASD
<p><i>Check a</i></p> $a = \frac{\sum Q_n}{0.85 f'_c b}$ $= \frac{251 \text{ kips}}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$ $= 0.820 \text{ in.} < 1.0 \text{ in.} \quad \text{o.k.}$	<p><i>Check a</i></p> $a = \frac{\sum Q_n}{0.85 f'_c b}$ $= \frac{251 \text{ kips}}{0.85(4 \text{ ksi})(7.50 \text{ ft})(12 \text{ in./ft})}$ $= 0.820 \text{ in.} < 1.0 \text{ in.} \quad \text{o.k.}$

Determine the required shear stud connectors

Using parallel deck with one 3/4-in. diameter stud in normal weight, 4 ksi concrete, in weak position;  $Q_n = 21.5 \text{ kips/stud}$

Manual  
 Table 3-21

LRFD	ASD
$\frac{\sum Q_n}{Q_n} = \frac{251 \text{ kips}}{21.5 \text{ kips/stud}} = 11.7 \text{ studs / side}$ <p>Therefore, use a minimum 24 studs for horizontal shear.</p> <p>The maximum stud spacing is 3'-0".</p> <p>Since the load is concentrated at 1/3 points, the studs are to be arranged as follows:</p> <p>Use 12 studs between supports and supported beams at 1/3 points. Between supported beams middle 1/3 of span, use 4 studs to satisfy minimum spacing requirements.</p> <p>Thus, 28 studs are required in a 12:4:12 arrangement.</p>	$\frac{\sum Q_n}{Q_n} = \frac{251 \text{ kips}}{21.5 \text{ kips/stud}} = 11.7 \text{ studs / side}$ <p>Therefore, use a minimum 24 studs for horizontal shear.</p> <p>The maximum stud spacing is 3'-0".</p> <p>Since the load is concentrated at 1/3 points, the studs are to be arranged as follows:</p> <p>Use 12 studs between supports and supported beams at 1/3 points. Between supported beams middle 1/3 of span, use 4 studs to satisfy minimum spacing requirements.</p> <p>Thus, 28 studs are required in a 12:4:12 arrangement.</p>

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. Since design was completed using the available Manual tables, a PNA location 7 was selected. A more economical solution would result if a PNA lower in the beam web could have been selected. In that case, very few studs would have been required and a larger non-composite beam might be a better solution. It is important to note the significant impact of properly designing for construction



**COLUMNS MEMBER DESIGN SELECTION FOR GRAVITY COLUMNS**

**Estimate column loads**

<b>Roof</b>	(from previous calculations)	
	Dead Load	0.020 kips/ft <sup>2</sup>
	Live (Snow)	<u>0.025 kips/ft<sup>2</sup></u>
	Total	0.045 kips/ft <sup>2</sup>

Snow drifting loads at the perimeter of the roof and at the mechanical screen wall from previous calculations

Reaction to column (side parapet)

$$w = (3.73 \text{ kips} / 6 \text{ ft}) - (0.025 \text{ ksf})(23.0 \text{ ft}) = 0.047 \text{ kips/ft}$$

Reaction to column (end parapet)

$$w = (0.43 \text{ klf}) - (0.025 \text{ ksf})(15.5 \text{ ft}) = 0.043 \text{ kips/ft}$$

Reaction to column (screen wall)

$$w = (0.48 \text{ klf}) - (0.025 \text{ ksf})(15.5 \text{ ft}) = 0.105 \text{ kips/ft}$$

Mechanical equipment (average)

$$w = 0.040 \text{ kips/ft}$$

Col	Loading		Area ft <sup>2</sup>	DL kips/ft <sup>2</sup>	P <sub>D</sub> kips	LL kips/ft <sup>2</sup>	P <sub>L</sub> kips
	Width ft	Length ft					
2A, 2F, 3A, 3F, 4A, 4F 5A, 5F, 6A, 6F, 7A, 7F snow drifting side exterior wall	23.0	30.0 30.0 30.0	690	0.020	13.8	0.025 0.047 klf	17.3 1.41
				0.413 klf	<u>12.4</u>		<u>18.7</u>
					<b>26.2</b>		<b>18.7</b>
1B, 1E, 8B, 8E snow drifting end exterior wall	3.50	22.5 22.5 22.5	79	0.020	1.58	0.025 0.043 klf	1.97 0.97
				0.413 klf	<u>9.29</u>		<u>2.94</u>
					<b>10.9</b>		<b>2.94</b>
1A, 1F, 8A, 8F  snow drifting end snow drifting side exterior wall	23.0	15.5  11.8 15.5 26.3	345  <u><math>\frac{-(79 \text{ ft}^2)}{2}</math></u> = 317	0.020	6.34	0.025 0.043 klf 0.047 klf	7.93 0.51 0.73
				0.413 klf	<u>10.8</u>		<u>9.16</u>
					<b>17.2</b>		<b>9.16</b>
1C, 1D, 8C, 8D  snow-drifting end exterior wall	37.5	15.5  26.3 26.3	581  <u><math>\frac{-(79 \text{ ft}^2)}{2}</math></u> = 542	0.020	10.8	0.025 0.043 klf	13.5 1.13
				0.413 klf	<u>10.8</u>		<u>14.7</u>
					<b>21.7</b>		<b>14.7</b>
2C, 2D, 7C, 7D	37.5	30.0	1,125	0.020	<b>22.5</b>	0.025	<b>28.1</b>
3C, 3D, 4C, 4D 5C, 5D, 6C, 6D snow-drifting mechanical area	22.5 15.0	30.0 30.0 30.0	675	0.020 0.060	13.5 <u>27.0</u> <b>40.5</b>	0.025 0.105 klf 0.025	16.9 3.15 <u>11.3</u> <b>31.3</b>

**Floor Loads** (from previous calculations)

Dead load	0.075 kips/ft <sup>2</sup>
Live load	<u>0.080 kips/ft<sup>2</sup></u>
Total load	0.155 kips/ft <sup>2</sup>

Reduction in live loads, analyzed at the base of 3 floors

Columns: 2A, 2F, 3A, 3F, 4A, 4F, 5A, 5F, 6A, 6F, 7A, 7F  
Exterior column without cantilever slabs

$$K_{LL} = 4 \quad L_o = 0.080 \text{ ksf} \quad n = 3$$

IBC Sect  
1607.9

$$A_T = (23 \text{ ft})(30 \text{ ft}) = 690 \text{ ft}^2$$

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} n A_T}} \right)$$

IBC Eqn  
16-21

$$= 0.080 \text{ ksf} \left( 0.25 + \frac{15}{\sqrt{(4)(3)(690 \text{ ft}^2)}} \right)$$

$$= 0.033 \text{ ksf} \geq 0.4L_o = 0.032 \text{ ksf}$$

use  $L = 0.033 \text{ ksf}$

Columns: 1B, 1E, 8B, 8E  
Exterior column without cantilever slabs

$$K_{LL} = 4 \quad L_o = 0.080 \text{ ksf} \quad n = 3$$

IBC Section  
1607.9

$$A_T = (5.5 \text{ ft})(22.5 \text{ ft}) = 124 \text{ ft}^2$$

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} n A_T}} \right)$$

IBC Eqn  
16-21

$$= 0.080 \text{ ksf} \left( 0.25 + \frac{15}{\sqrt{(4)(3)(124 \text{ ft}^2)}} \right)$$

$$= 0.051 \text{ ksf} \geq 0.4L_o = 0.032 \text{ ksf}$$

use  $L = 0.051 \text{ ksf}$

Columns: 1A, 1F, 8A, 8F

Corner column without cantilever slabs

$$\text{Conservatively assume } K_{LL} = 1 \quad L_o = 0.080 \text{ ksf} \quad n = 3$$

IBC Section  
1607.9

$$A_T = (15.5 \text{ ft})(23 \text{ ft}) - (124 \text{ ft}^2 / 2) = 295 \text{ ft}^2$$

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} n A_T}} \right) \geq 0.4 L_o$$

IBC Eqn  
16-21

$$= 0.080 \text{ ksf} \left( 0.25 + \frac{15}{\sqrt{(1)(3)(295 \text{ ft}^2)}} \right) \geq 0.4(0.08 \text{ ksf})$$

$$= 0.060 \text{ ksf} \geq 0.032 \text{ ksf}$$

use  $L = 0.060 \text{ ksf}$

Columns: 1C, 1D, 8C, 8D

Exterior column without cantilever slabs

$$K_{LL} = 4 \quad L_o = 0.080 \text{ ksf} \quad n = 3$$

IBC Section  
1607.9

$$A_T = (15.5 \text{ ft})(37.5 \text{ ft}) - (124 \text{ ft}^2 / 2) = 519 \text{ ft}^2$$

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} n A_T}} \right) \geq 0.4 L_o$$

IBC Eqn  
16-21

$$= 0.080 \text{ ksf} \left( 0.25 + \frac{15}{\sqrt{(4)(3)(519 \text{ ft}^2)}} \right) \geq 0.4(0.08 \text{ ksf})$$

$$= 0.035 \text{ ksf} \geq 0.032 \text{ ksf}$$

use  $L = 0.035 \text{ ksf}$

Columns: 2C, 2D, 3C, 3D, 4C, 4D, 5C, 5D, 6C, 6D, 7C, 7D  
Interior column

$$K_{LL} = 4 \quad L_o = 0.080 \text{ ksf} \quad n = 3$$

IBC Section  
1607.9

$$A_T = 1,125 \text{ ft}^2$$

$$L = L_o \left( 0.25 + \frac{15}{\sqrt{K_{LL} n A_T}} \right) \geq 0.4 L_o$$

IBC Eqn  
16-21

$$= 0.080 \text{ ksf} \left( 0.25 + \frac{15}{\sqrt{(4)(3)(1,125 \text{ ft}^2)}} \right) \geq 0.4(0.08 \text{ ksf})$$

$$= 0.030 \text{ ksf} \geq 0.032 \text{ ksf}$$

use  $L = 0.032 \text{ ksf}$

Column	Loading		Area ft <sup>2</sup>	DL	$P_D$	LL	$P_L$
	Width ft	Length ft		kips/ft <sup>2</sup>	kips	kips/ft <sup>2</sup>	kips
2A, 2F, 3A, 3F, 4A, 4F 5A, 5F, 6A, 6F, 7A, 7F exterior wall	23.0	30.0	690	0.075	51.8	0.033	22.9
		30.0		0.503 klf	<u>15.1</u>		<u>22.9</u>
					<b>66.8</b>		<b>22.9</b>
1B, 1E, 8B, 8E exterior wall	5.50	22.5	124	0.075	9.28	0.051	6.33
		22.5		0.503 klf	<u>11.3</u>		<u>6.33</u>
					<b>20.6</b>		<b>6.33</b>
1A, 1F, 8A, 8F exterior wall	23.9	15.5	370	0.075	22.1	0.060	17.8
			$\frac{-(124 \text{ ft}^2)}{2}$ = 295				
		27.3		0.503 klf	<u>13.7</u>		<u>17.8</u>
					<b>35.8</b>		<b>17.8</b>
1C, 1D, 8C, 8D exterior wall	37.5	15.5	581	0.075	39.0	0.035	18.3
			$\frac{-(124 \text{ ft}^2)}{2}$ = 519				
		22.5		0.503 klf	<u>11.3</u>		<u>18.3</u>
					<b>50.3</b>		<b>18.3</b>
2C, 2D, 3C, 3D, 4C, 4D 5C, 5D, 6C, 6D, 7C, 7D	37.5	30.0	1,125	0.075	<b>84.4</b>	0.032	<b>36.0</b>

**Column load summary**

<b>Col</b>	<b>Floor</b>	<b><math>P_D</math></b>	<b><math>P_L</math></b>
		<b>kips</b>	<b>kips</b>
2A, 2F, 3A, 3F, 4A, 4F 5A, 5F, 6A, 6F, 7A, 7F	Roof	26.2	18.7
	4 <sup>th</sup>	66.8	22.9
	3 <sup>rd</sup>	66.8	22.9
	2 <sup>nd</sup>	<u>66.8</u>	<u>22.9</u>
	<b>Total</b>	<b>227</b>	<b>87.4</b>
1B, 1E, 8B, 8E	Roof	10.9	2.94
	4 <sup>th</sup>	20.6	6.33
	3 <sup>rd</sup>	20.6	6.33
	2 <sup>nd</sup>	<u>20.6</u>	<u>6.33</u>
	<b>Total</b>	<b>72.7</b>	<b>21.9</b>
1A, 1F, 8A, 8F	Roof	17.2	9.16
	4 <sup>th</sup>	35.8	17.8
	3 <sup>rd</sup>	35.8	17.8
	2 <sup>nd</sup>	<u>35.8</u>	<u>17.8</u>
	<b>Total</b>	<b>125</b>	<b>62.5</b>
1C, 1D, 8C, 8D	Roof	21.7	14.7
	4 <sup>th</sup>	50.3	18.3
	3 <sup>rd</sup>	50.3	18.3
	2 <sup>nd</sup>	<u>50.3</u>	<u>18.3</u>
	<b>Total</b>	<b>173</b>	<b>69.5</b>
2C, 2D, 7C, 7D	Roof	22.5	28.1
	4 <sup>th</sup>	84.4	36.0
	3 <sup>rd</sup>	84.4	36.0
	2 <sup>nd</sup>	<u>84.4</u>	<u>36.0</u>
	<b>Total</b>	<b>276</b>	<b>136</b>
3C, 3D, 4C, 4D 5C, 5D, 6C, 6D	Roof	40.5	31.3
	4 <sup>th</sup>	84.4	36.0
	3 <sup>rd</sup>	84.4	36.0
	2 <sup>nd</sup>	<u>84.4</u>	<u>36.0</u>
	<b>Total</b>	<b>294</b>	<b>139</b>

**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
$P_u = 1.2(294 \text{ kips}) + 1.6(139 \text{ kips}) = 575 \text{ kips}$	$P_a = 294 \text{ kips} + 139 \text{ kips} = 433 \text{ kips}$

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 $\phi_c P_n = 696 \text{ kips} > 575 \text{ kips}$ <b>o.k.</b>	W12×65 $P_n/\Omega_c = 463 \text{ kips} > 433 \text{ kips}$ <b>o.k.</b>
W14×68 $\phi_c P_n = 655 \text{ kips} > 575 \text{ kips}$ <b>o.k.</b>	W14×68 $P_n/\Omega_c = 436 \text{ kips} > 433 \text{ kips}$ <b>o.k.</b>

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
$P_u = 1.2(276 \text{ kips}) + 1.6(136 \text{ kips}) = 549 \text{ kips}$	$P_a = 276 \text{ kips} + 136 \text{ kips} = 412 \text{ kips}$

From the tables, enter with the effective length of 13.5 ft, and proceed until reaching the lightest size that has sufficient available strength at the required unbraced length.

Manual  
Table 4-1

LRFD	ASD
W12×65 $\phi_c P_n = 696 \text{ kips} > 549 \text{ kips}$ <b>o.k.</b>	W12×65 $P_n/\Omega_c = 463 \text{ kips} > 412 \text{ kips}$ <b>o.k.</b>
W14×68 $\phi_c P_n = 655 \text{ kips} > 549 \text{ kips}$ <b>o.k.</b>	W14×68 $P_n/\Omega_c = 436 \text{ kips} > 412 \text{ kips}$ <b>o.k.</b>

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
$P_u = 1.2(72.7 \text{ kips}) + 1.6(21.9 \text{ kips})$	$P_a = 72.7 \text{ kips} + 21.9 \text{ kips}$
= 122 kips	= 95 kips

From the tables, enter with the effective length of 13.5 ft, and proceed down the sizes (across the page) until a size is found that satisfies the total load at the unbraced length.

Manual  
Table 4-1

LRFD	ASD
W12×40	W12×40
$\phi_c P_n = 316 \text{ kips} > 139 \text{ kips}$ <b>o.k.</b>	$P_n/\Omega_c = 210 \text{ kips} > 105 \text{ kips}$ <b>o.k.</b>

Manual  
Table 4-1

Note: A 12 in. column was selected above for ease of erection.

**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.

- 1. simple diaphragm building **o.k.**
- 2. not a flexible building **o.k.**
- 3. does not have response characteristics requiring special considerations **o.k.**
- 4. regular shape and symmetrical cross section **o.k.**

IBC Sect  
1609.6.1.1

*Define input parameters*

- 1. Occupancy category: II
- 2. Basic wind speed  $V$ , 90 mph
- 3. Importance factor  $I_w$  1.0
- 4. Exposure category: C
- 5. Height and exposure adjustment  $\lambda$ : 1.59

IBC Table  
1604.5

IBC Sect  
1609.3

IBC Table  
1604.5

IBC Sect  
1609.4

IBC Table  
1609.6.2.1(4)

$p_s = \lambda I_w p_{s30} = (1.59)(1.0)(12.8 \text{ psf}) = 20.4 \text{ psf}$	Horizontal pressure zone A	IBC Table 1609.6.2.1(1) and Eqn. 16-35
$(1.59)(1.0)(10.2 \text{ psf}) = 13.5 \text{ psf}$	Horizontal pressure zone C	
$(1.59)(1.0)(-15.4 \text{ psf}) = -24.5 \text{ psf}$	Vertical pressure zone E	
$(1.59)(1.0)(-8.8 \text{ psf}) = -14.0 \text{ psf}$	Vertical pressure zone F	
$(1.59)(1.0)(-10.7 \text{ psf}) = -17.0 \text{ psf}$	Vertical pressure zone G	
$(1.59)(1.0)(-6.8 \text{ psf}) = -10.8 \text{ psf}$	Vertical pressure zone H	

$a = 10\%$  of least horizontal dimension or  $0.4h$ , whichever is smaller, but not less than either  $4\%$  of least dimension or 3 ft.

IBC Fig  
1609.6.2.1

10% of least dimension:	12.0 ft
40% of eave height:	22.0 ft
4% of least dimension or 3 ft	4.8 ft
$a =$	12.0 ft
$2a =$	24.0 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*

Assume mechanical screen wall height: 6 ft  
 Wall height:  $\frac{1}{2} (55.0 \text{ ft} - 40.5 \text{ ft}) = 7.25 \text{ ft}$   
 Parapet wall height: 2 ft.  
 Total wall height at roof at screen wall:  $6 \text{ ft} + 7.25 \text{ ft} = 13.3 \text{ ft}$   
 Total wall height at roof at parapet:  $2 \text{ ft} + 7.25 \text{ ft} = 9.25 \text{ ft}$

*Calculate load to 4<sup>th</sup> floor diaphragm*

Wall height:  $\frac{1}{2} (55.0 \text{ ft} - 40.5 \text{ ft}) = 7.25 \text{ ft}$   
 $\frac{1}{2} (40.5 \text{ ft} - 27.0 \text{ ft}) = 6.75 \text{ ft}$   
 Total wall height at floor:  $6.75 \text{ ft} + 7.25 \text{ ft} = 14.0 \text{ ft}$

*Calculate load to third floor diaphragm*

Wall height:  $\frac{1}{2} (40.5 \text{ ft} - 27.0 \text{ ft}) = 6.75 \text{ ft}$   
 $\frac{1}{2} (27.0 \text{ ft} - 13.5 \text{ ft}) = 6.75 \text{ ft}$   
 Total wall height at floor:  $6.75 \text{ ft} + 6.75 \text{ ft} = 13.5 \text{ ft}$

*Calculate load to second floor diaphragm*

Wall height:  $\frac{1}{2} (27.0 \text{ ft} - 13.5 \text{ ft}) = 6.75 \text{ ft}$   
 $\frac{1}{2} (13.5 \text{ ft} - 0.0 \text{ ft}) = 6.75 \text{ ft}$   
 Total wall height at floor:  $6.75 \text{ ft} + 6.75 \text{ ft} = 13.5 \text{ ft}$

**Total load to diaphragm:**

Load to diaphragm:  $w_{s(A)} = (20.4 \text{ psf})(9.25 \text{ ft}) = 189 \text{ plf}$

$w_{s(C)} = (13.5 \text{ psf})(9.25 \text{ ft}) = 125 \text{ plf}$

$w_{s(C)} = (13.5 \text{ psf})(13.3 \text{ ft}) = 179 \text{ plf}$

Load to diaphragm:  $w_{s(A)} = (20.4 \text{ psf})(14.0 \text{ ft}) = 286 \text{ plf}$

$w_{s(C)} = (13.5 \text{ psf})(14.0 \text{ ft}) = 189 \text{ plf}$

Load to diaphragm:  $w_{s(A)} = (20.4 \text{ psf})(13.5 \text{ ft}) = 275 \text{ plf}$

$w_{s(C)} = (13.5 \text{ psf})(13.5 \text{ ft}) = 182 \text{ plf}$

$l$  = length of structure, ft

$b$  = width of structure, ft

$h$  = height of wall at building element, ft

Wind from a north – south direction

Total load to diaphragm:  $P_{W(n-s)} = 2 a w_{s(A)} + w_{s(C)} (l - 2 a)$

Wind from a east – west direction

Total load to diaphragm:  $P_{W(e-w)} = 2 a w_{s(A)} + w_{s(C)} (b - 2 a)$

	$l$	$b$	$2a$	$h$	$p_{s(A)}$	$p_{s(C)}$	$w_{s(A)}$	$w_{s(C)}$	$P_{W(n-s)}$	$P_{W(e-w)}$
	ft	ft	ft	ft	psf	psf	plf	plf	kips	kips
Roof	213	123	24	9.25	20.4		189		9.06	9.06
				9.25		13.5		125	9.37	5.62
				13.3		13.5		179	16.1	5.37
Fourth	213	123	24	14.0	20.4	13.5	286	189	42.6	25.6
Third	213	123	24	13.5	20.4	13.5	275	182	41.1	24.7
Second	213	123	24	13.5	20.4	13.5	275	182	<u>41.1</u>	<u>24.7</u>
Base									<b>159</b>	<b>95.0</b>

**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.

$$\text{Area} = (121 \text{ ft})(211 \text{ ft}) = 25,531 \text{ ft}^2$$

The perimeter cladding system length:

$$\text{Length} = (2)(122 \text{ ft}) + (2)(212 \text{ ft}) = 668 \text{ ft}$$

The perimeter cladding weight at floors:

Brick spandrel panel with metal stud backup	$(7.5 \text{ ft})(0.055 \text{ ksf}) = 0.413 \text{ klf}$
Window wall system	$(6.0 \text{ ft})(0.015 \text{ ksf}) = 0.090 \text{ klf}$
<b>Total</b>	<b>0.503 klf</b>

Typical roof dead load (from previous calculations):

Roof Area = $(25,531 \text{ ft}^2)(0.020 \text{ ksf}) =$	511 kips
Wall perimeter = $(668 \text{ ft})(0.413 \text{ klf}) =$	276 kips
Mechanical equipment =	<u>32 kips</u>
<b>Total</b>	<b>818 kips</b>

Typical third and fourth floor dead load:

Floor Area = $(25,531 \text{ ft}^2)(0.085 \text{ ksf}) =$	2,170 kips
Wall perimeter = $(668 \text{ ft})(0.503 \text{ klf}) =$	<u>336 kips</u>
<b>Total</b>	<b>2,506 kips</b>

Second floor dead load: the floor area is reduced because of the open atrium

Floor Area = $(24,226 \text{ ft}^2)(0.085 \text{ ksf}) =$	2,059 kips
Wall perimeter = $(668 \text{ ft})(0.503 \text{ klf}) =$	<u>336 kips</u>
<b>Total</b>	<b>2,395 kips</b>

Total dead load of the building:

Roof	818 kips
Fourth floor	2,506 kips
Third floor	2,506 kips
Second floor	<u>2,395 kips</u>
<b>Total</b>	<b>8,225 kips</b>

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
$S_S = 0.121$		IBC Fig 1615(1)
$S_I = 0.06$		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)
$F_v @ S_I \leq 0.1 = 2.4$		IBC Table 1615.1.2(2)
$S_{MS} = F_a S_S = (1.6)(0.121) = 0.194$		IBC Eqn 16-38
$S_{MI} = F_v S_I = (2.4)(0.06) = 0.144$		IBC Eqn 16-39
$S_{DS} = \frac{2}{3} S_{MS} = 0.129$		IBC Eqn 16-40
$S_{DI} = \frac{2}{3} S_{MI} = 0.096$		IBC Eqn 16-41
$S_{DS} \leq 0.167g$ , Seismic Use Group I:	Seismic Design Category: A	IBC Table 1616.3(1)
$0.067g \leq S_{DI} \leq 0.133g$ , Seismic Use Group I:	Seismic Design Category: B	IBC Table 1616.3(2)
Building Height, $h_n = 55$ ft		ASCE 7 Table 9.5.5.3.2
$C_t = 0.02$ : $x = 0.75$		
$T_a = C_t (h_n)^x = (0.02)(55 \text{ ft})^{0.75} = 0.404$		ASCE 7 Eqn 9.5.5.3.2-1
$0.8 T_s = 0.8 (S_{DI}/S_{DS}) = 0.8(0.129 / 0.096) = 0.595$		IBC Sec. 1615.1.4
Since $T_a < 0.8 T_s$ ; Seismic Design Category A 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
Overstrength Factor: $\Omega_o = 3$		ASCE 7 Table 9.5.2.2
Seismic Base Shear: $V = C_s W$		ASCE 7 Eqn 9.5.5.2-1
$C_s = \frac{S_{DS}}{R/I_E} = \frac{0.129}{3/1} = 0.043$ <u>Controls</u>		ASCE 7 Eqn 9.5.5.2.1-1

$$C_s \text{ need not exceed, } C_s = \frac{S_{D1}}{\left(\frac{R}{I_E}\right)T} = \frac{0.096}{\left(\frac{3}{1}\right)(0.404)} = 0.079$$

ASCE 7 Eqn  
9.5.5.2.1-2

$$\text{Min } C_s = 0.44 S_{Ds} I_E = 0.44(0.129)(1.0) = 0.006$$

$$V = 0.043(8225 \text{ kips}) = 354 \text{ kips}$$

ASCE 7 Eqn  
9.5.5.2.1-3

Calculate vertical distribution of seismic forces

$$F_x = C_{vx}V = C_{vx}(0.043)W$$

ASCE 7 Sect  
9.5.5.4

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^n w_i h_i^k}$$

ASCE 7 Eqn  
9.5.5.4-1

ASCE 7 Eqn  
9.5.5.4-2

for structures having a period of 0.5 sec or less,  $k = 1$

Calculate horizontal shear distribution and torsion

$$V_x = \sum_{i=x}^n F_i$$

ASCE 7 Sect  
9.5.5.4

Calculate Overturning Moment

ASCE 7 Eqn  
9.5.5.5

$$M_x = \sum_{i=x}^n F_i (h_i - h_x)$$

ASCE 7 Sect  
9.5.5.6

ASCE 7 Eqn  
9.5.5.6

	$w_x$	$h_x^k$	$w_x h_x^k$	$C_{vx}$	$F_x$	$V_x$	$M_x$
	kips	ft	kip-ft	kips	kips	kips	k-ft
Roof	818	55.0	45,000	0.18	64.6	64.6	
Fourth	2,506	40.5	101,500	0.41	146	210	936
Third	2,506	27.0	67,700	0.27	97.1	307	3,774
Second	<u>2,395</u>	13.5	<u>32,300</u>	0.13	<u>46.4</u>	354	<u>7,922</u>
<b>Base</b>	<b>8,225</b>		<b>246,500</b>		<b>354</b>		<b>12,696</b>

Determine rigidity of diaphragms

Roof

Roof deck: 1½ in. deep, 22 gage, Type B,  
support fasteners; 5/8 in. puddle welds and sidelap fasteners; #10 TEK screws  
Joist spacing = 6 ft  
Diaphragm length = 210 ft  
Diaphragm width = 120 ft

$$\text{Load to diaphragm} = (64.6 \text{ kips}) / (210 \text{ ft}) = 0.308 \text{ klf}$$

$$\text{Diaphragm shear load} = \frac{(308 \text{ plf})(210 \text{ ft})}{2(120 \text{ ft})} = 269 \text{ plf}$$

Fastener layout = 36 / 5; sidelap = 3 #10 TEK

$DB = 758$     $K1 = 0.287$     $K2 = 870$    Shear strength = 298 plf

$$G' = \frac{K2}{3.78 + \frac{0.3DB}{\text{span}} + 3K1 \text{ span}} = \frac{870}{3.78 + \frac{0.3(758)}{6} + 3(0.287)(6)} = 18.6 \text{ k/in}$$

$$\Delta = \frac{wL^2}{8BG'} = \frac{(0.308 \text{ klf})(210 \text{ ft})^2}{8(120 \text{ ft})(18.6 \text{ k/in})} = 0.761 \text{ in}$$

Allowable story drift:  $0.025 h_{sx} = 0.025(14.5 \text{ ft}) = 0.363 \text{ in.}$

Diaphragm, rigid. Lateral deformation  $\leq 2$  (story drift)

$$\Delta = 0.761 \text{ in.} > 2(0.363 \text{ in.}) = 0.725 \text{ in.}$$

Therefore, the roof diaphragm is flexible, but using at rigid diaphragm distribution is more conservative.

#### *Fourth floor*

Floor deck: 3 in. deep, 22 gage, Composite deck with normal weight concrete, support fasteners;  $\frac{5}{8}$  in. puddle welds and button punched sidelap fasteners  
 Beam spacing = 10 ft  
 Diaphragm length = 210 ft  
 Diaphragm width = 120 ft

Load to diaphragm = (146 kips) / (210 ft) = 0.695 klf

$$\text{Diaphragm shear load} = \frac{(695 \text{ plf})(210 \text{ ft})}{2(120 \text{ ft})} = 608 \text{ plf}$$

Fastener layout = 36 / 4; 1 button punched sidelap fastener

$K1 = 0.178$     $K2 = 870$     $K3 = 2,380$    Shear strength = 1,588 plf

$$G' = \left( \frac{K2}{3.590 + 3K1 \text{ span}} \right) + K3 = \left( \frac{870}{3.50 + 3(0.729)(6)} \right) + 2,380 = 2,533 \text{ k/in}$$

$$\Delta = \frac{wL^2}{8BG'} = \frac{(0.695 \text{ klf})(210 \text{ ft})^2}{8(120 \text{ ft})(2,533 \text{ k/in})} = 0.0126 \text{ in}$$

Steel Deck  
 Institute  
*Diaphragm  
 Design  
 Manual*

IBC Table  
 1617.3.1

IBC Sect  
 1602

Steel Deck  
 Institute  
*Diaphragm  
 Design  
 Manual*



Allowable story drift;  $0.025 h_{sx} = 0.025(13.5 \text{ ft}) = 0.338 \text{ in.}$

IBC Table  
1617.3.1

Diaphragm, rigid. Lateral deformation  $\leq 2$  (story drift)

IBC Sect  
1602

$$\Delta = 0.0126 \text{ in.} \leq 2(0.338 \text{ in.}) = 0.675 \text{ in.}$$

Therefore, the floor diaphragm is rigid

### *Second floor*

Floor deck: 3 in. deep, 22 gage, Composite deck with normal weight concrete,  
support fasteners:  $\frac{5}{8}$  in. puddle welds and button punched sidelap fasteners

Beam spacing = 10 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 =  $(46 \text{ kips}) / (210 \text{ ft}) = 0.221 \text{ klf}$

$$\text{Diaphragm shear load} = \frac{(221 \text{ plf})(210 \text{ ft})}{2(120 \text{ ft})} = 193 \text{ plf}$$

Fastener layout = 36 / 4; 1 button punched sidelap fastener

$K1 = 0.729$   $K2 = 870$   $K3 = 2,380$  Shear strength = 1,588 plf

Steel Deck  
Institute  
*Diaphragm  
Design  
Manual*

$$G' = \left( \frac{K2}{3.590 + 3K1 \text{ span}} \right) + K3 = \left( \frac{870}{3.50 + 3(0.729)(6)} \right) + 2,380 = 2,533 \text{ k/in}$$

$$\Delta = \frac{wL^2}{8BG'} = \frac{(0.221 \text{ klf})(210 \text{ ft})^2}{8(75 \text{ ft})(2,533 \text{ k/in})} = 0.00641 \text{ in}$$

Allowable story drift;  $0.025 h_{sx} = 0.025(13.5 \text{ ft}) = 0.338 \text{ in.}$

IBC Table  
1617.3.1

Diaphragm, rigid. Lateral deformation  $\leq 2$  (story drift)

IBC Sect  
1602

$$\Delta = 0.00641 \text{ in.} \leq 2(0.338 \text{ in.}) = 0.675 \text{ in.}$$

Therefore, the floor diaphragm is rigid

Horizontal shear distribution and torsion:

ASCE 7 Sect  
9.5.5.5

	Load to Grids 1 and 8					Total
	$F_y$	Load to Frame		Accidental Torsion		
	kip	%	kip	%	kip	
Roof	64.6	50	32.3	5	3.23	35.5
Fourth	146	50	73.0	5	7.30	80.3
Third	97.1	50	48.6	5	4.86	53.4
Second	46.4	50	23.2	5	2.32	<u>25.5</u>
Base						<b>195</b>

	Load to Grids A and F					Total
	$F_x$	Load to Frame		Accidental Torsion		
	kip	%	kip	%	kip	
Roof	64.6	50	32.3	5	3.23	35.5
Fourth	146	50	73.0	5	7.30	80.3
Third	97.1	50	48.6	5	4.86	53.4
Second	46.4	51.8 <sup>(1)</sup>	24.0	5	2.32	<u>26.4</u>
Base						<b>196</b>

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.

<sup>(1)</sup> At the Second Floor slab, because of the atrium opening, the center of mass is not at the center of rigidity.

	Area ft <sup>2</sup>	Mass kips	x-dist ft	$M_x$ k-ft	y-dist ft	$M_y$ k-ft
I	10,950	931	45.25	42,118	60.5	56,313
II	2,265	193	105.5	20,311	37.75	7,268
III	10,950	<u>931</u>	165.8	<u>154,279</u>	60.5	<u>56,313</u>
		<b>2,054</b>		<b>216,709</b>		<b>119,894</b>

$$x = 216,709 \text{ k-ft} / 2,054 \text{ kips} = 105.5 \text{ ft}$$

$$105.5 \text{ ft} / 211 \text{ ft} = 50\%$$

$$y = 119,894 \text{ k-ft} / 2,054 \text{ kips} = 58.37 \text{ ft}$$

$$(121 \text{ ft} - 58.37 \text{ ft}) / 121 \text{ ft} = 51.8\%$$

## 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  $B_1$ - $B_2$  approximation method given in Specification Section C2.1b.

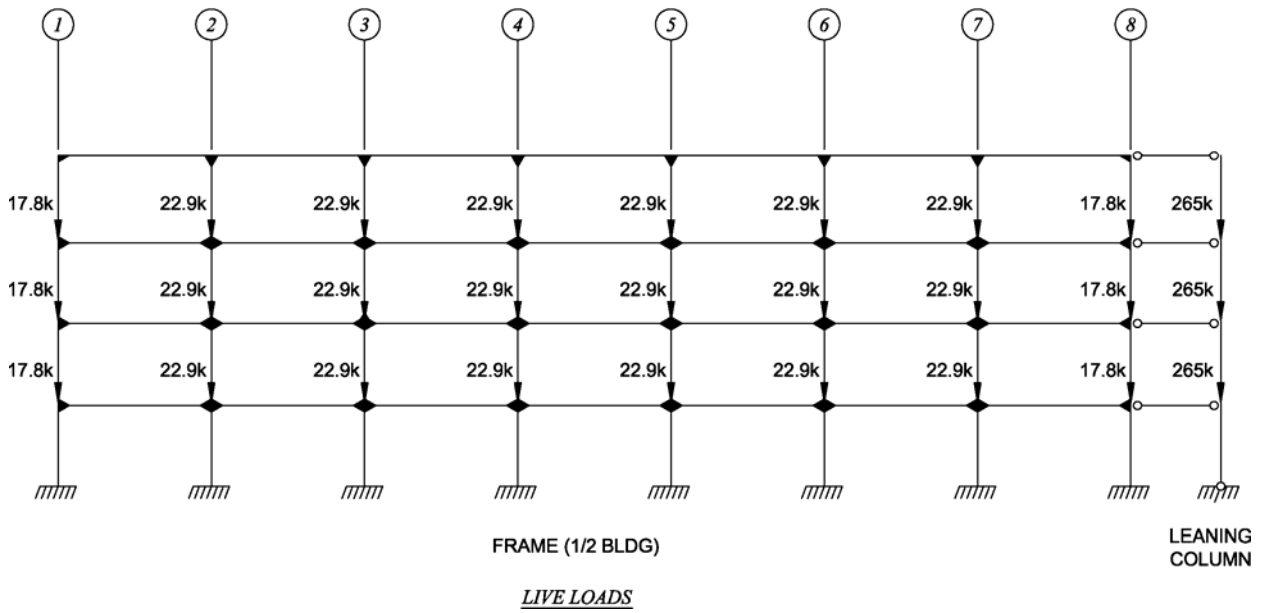
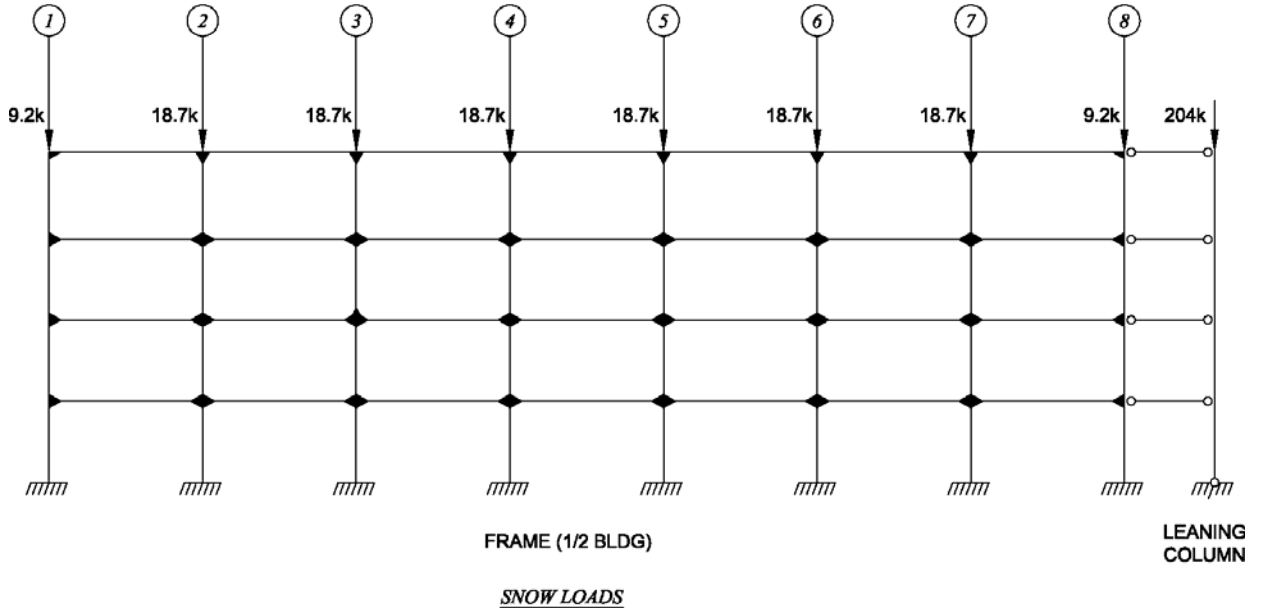
The column and beam layouts for the moment frames follow. Although the frames on Grids A and F are the same, slightly heavier seismic loads accumulate on grid F, after accounting for the atrium area on Grid A and accidental torsion. The models are half-building models. The frame was originally modeled with W14×82 interior columns and W21×44 non-composite beams. This model had a drift that substantially exceeded the  $0.025h_{sx}$  story drift allowed per IBC Table 1617.3.1, Seismic Use Group I. The column size was incremented up to a W14×99 and the W21×44 beams were upsized to W24×55 (with minimum composite studs) and the beams were modeled with a stiffness of  $I_{eq} = I_s$ . Alternatively, the beams could be modeled as  $I_{eq} = 0.6I_{LB} + 0.4I_n$  (Formula 24),  $s$ . This equation is given in AISC Design Guide 8. These changes resulted in a drift that satisfied the  $L / 400$  limit. This layout is shown in the Grid A and F, Frame ( $\frac{1}{2}$  building) layout that follows.

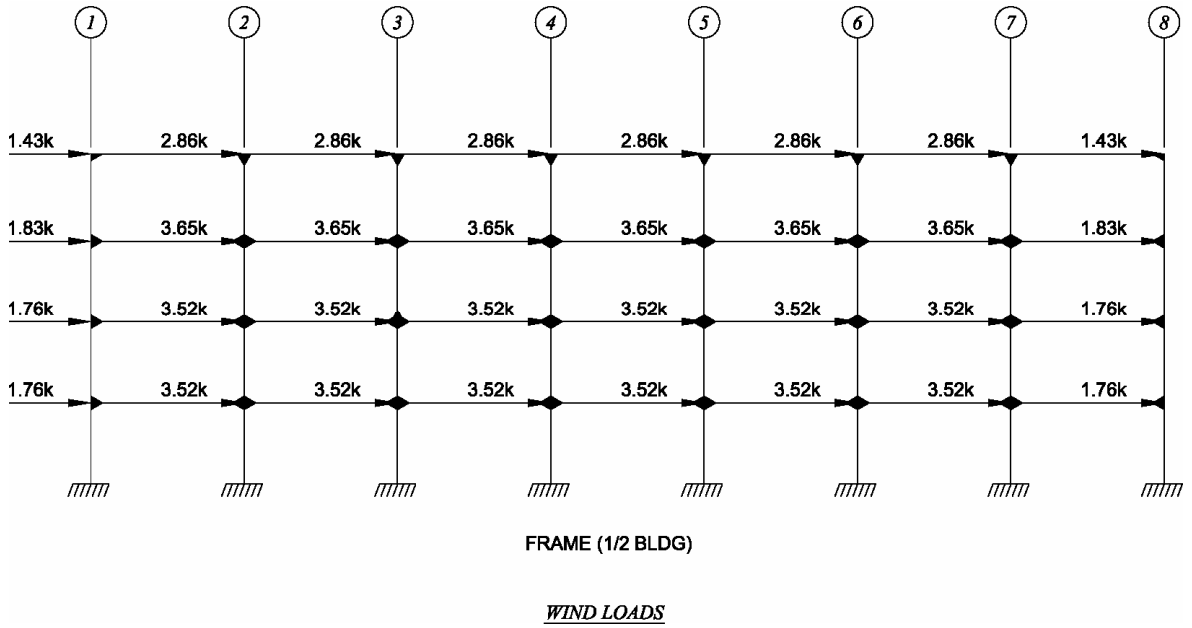
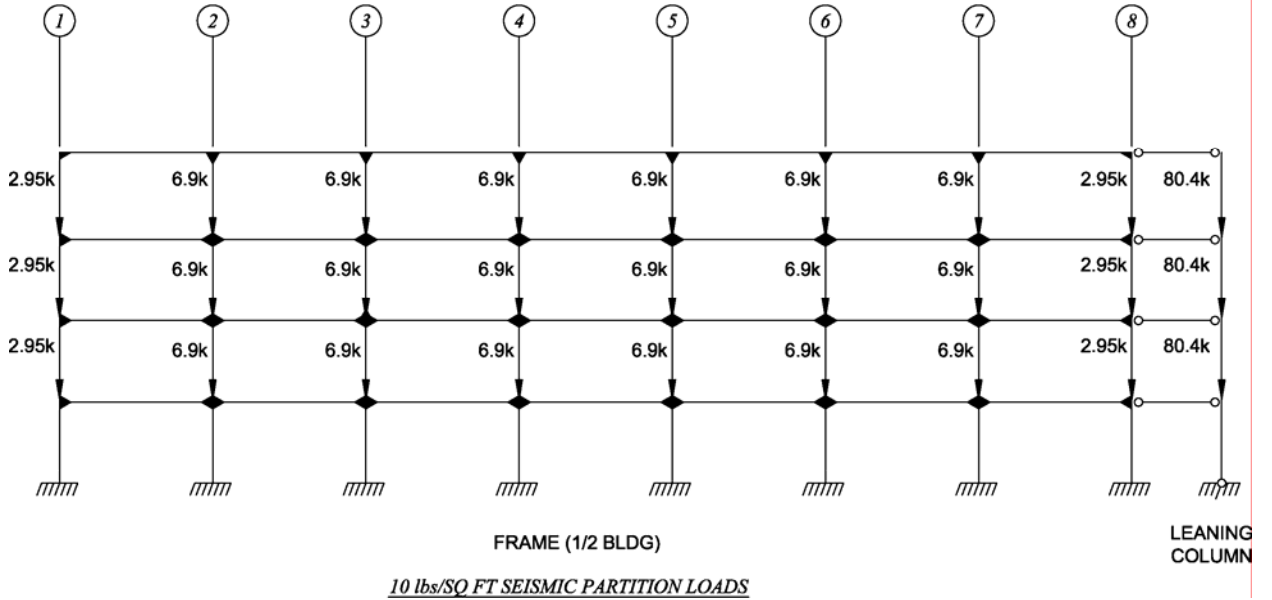
All of the vertical loads on the frame were modeled as point loads on the frame. As noted in the description of the dead load,  $W$ , in IBC Section 1617.5,  $0.010 \text{ kip/ft}^2$  is included in the dead load combinations. The remainder of the half-building model gravity loads were accumulated in 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 and the seismic loads 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.

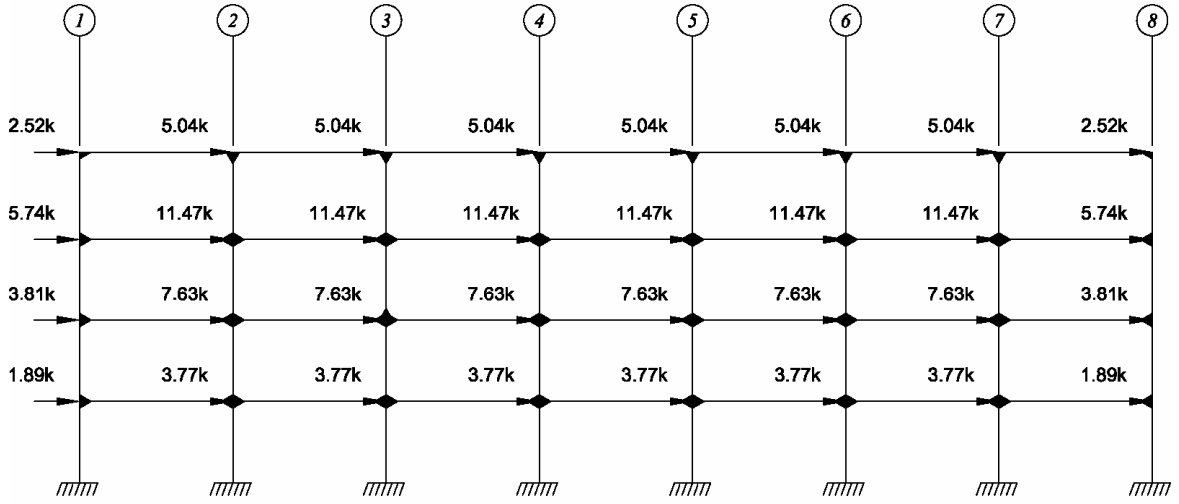
There are four horizontal load cases. Two are the wind load and seismic load, per the previous discussion. In addition, notional loads of  $N_i = 0.002Y_i$  were established. These load cases are shown in the load cases that follow.

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.



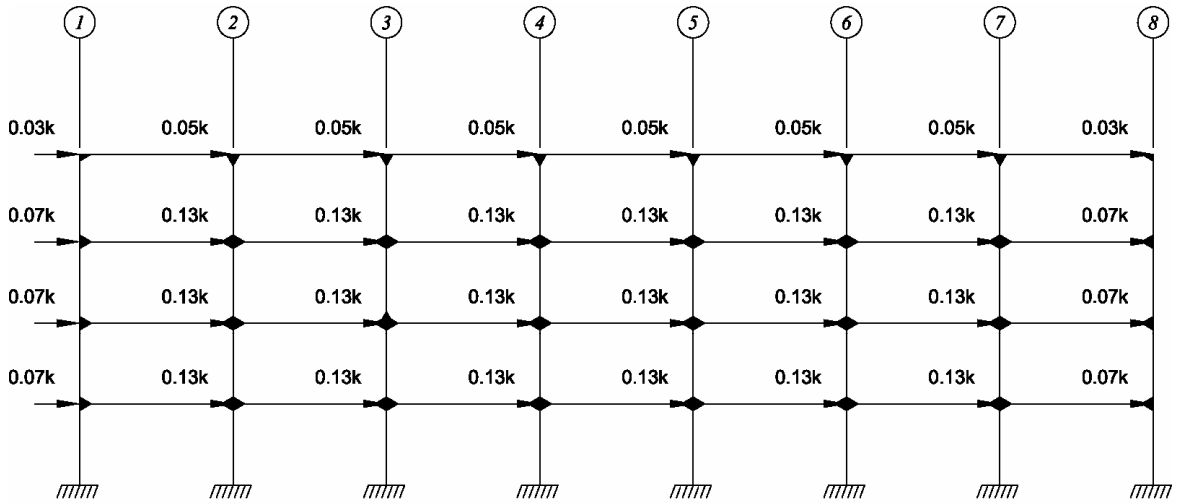






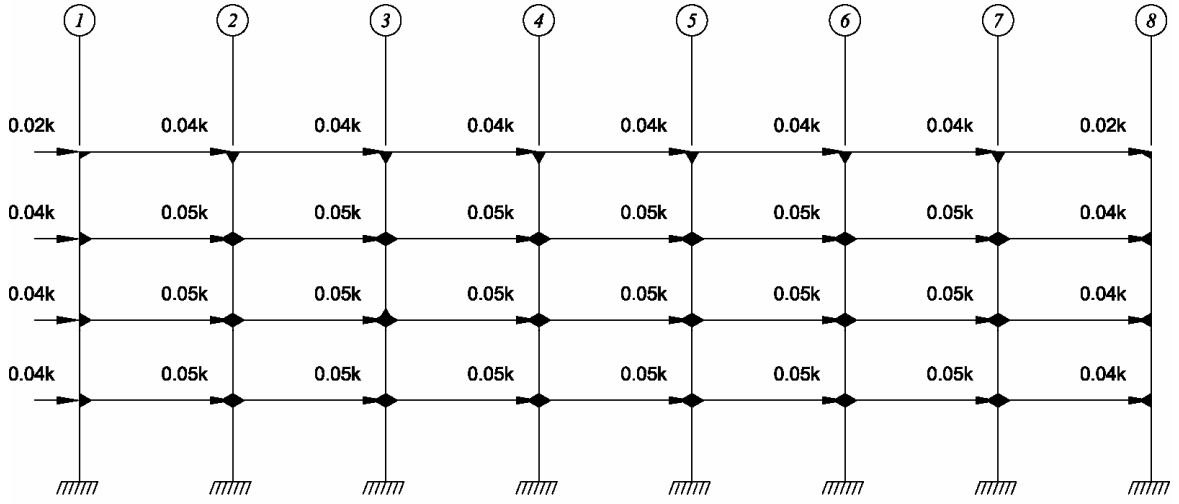
FRAME (1/2 BLDG)

SEISMIC LOADS



FRAME (1/2 BLDG)

NOTIONAL DL LOADS



FRAME (1/2 BLDG)

*NOTIONAL LL LOADS*



## CALCULATION OF REQUIRED STRENGTH - THREE METHODS

Three methods of determining the required strength including second-order effects are included below. A fourth method – second-order analysis by amplified first-order analysis – is found in Section C2.2b or the Specification. The method requires the inclusion of notional loads in the analysis, but all required strengths can be determined from a first-order analysis. For guidance on applying these methods, see the discussion in Manual Part 2 titled *Required Strength, Effective Length, and Second-Order Effects*.

### GENERAL INFORMATION FOR ALL THREE METHODS

Seismic load dominates over winds loads in the moment frame direction of this example building. Although the frame analysis that follows was run for all LRFD and ASD load combinations, the column unity design check is highly dependent on the moment portion, and therefore, the controlling equations are those with the load combinations shown below. A typical column near the middle of the frame is analyzed, but the first interior columns and the end columns were also checked. Beam analysis is covered after the three different methods are shown for the typical interior column.

Note: The second-order analysis and the unity checks are based on the moments and column loads for the particular load combination being checked, and not on the envelope of maximum values.

#### METHOD 1. EFFECTIVE LENGTH METHOD

This method accounts for second-order effects in frames by amplifying the axial forces and moments in members and connections from a first-order analysis.

Specification  
C.2.2a

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 in ASCE 7 and are summarized in Part 2 of the Manual.

A summary of the axial loads, moments and 1<sup>st</sup> floor drifts from the first-order computer analysis is shown below:

LRFD	ASD
$1.2D \pm 1.0E + 0.5L + 0.2S$ (Controls for columns and beams)	$D \pm (W \text{ or } 0.7 E)$ (Controls for columns) $D + 0.75(W \text{ or } 0.7E) + 0.75L$ $+ 0.75(L_r \text{ or } S \text{ or } R)$ (Controls for beams)
For Interior Column Design: $P_u = 335$ kips $M_{1u} = 157$ kip-ft (from first-order analysis) $M_{2u} = 229$ kip-ft (from first-order analysis)	For Interior Column Design: $P_a = 247$ kips $M_{1a} = 110$ kip-ft (from first-order analysis) $M_{2a} = 161$ kip-ft (from first-order analysis)
First-order first floor drift = 0.562 in.	First-order first floor drift = 0.394 in.

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	
$M_r = B_1 M_{nt} + B_2 M_{lt}$	$M_r = B_1 M_{nt} + B_2 M_{lt}$	Specification Eqn C2-1a
<p>Determine <math>B_1</math></p> <p><math>P_r</math> = required second-order axial strength using LRFD or ASD load combinations, kips.</p> <p>Note that for members in axial compression <math>P_r</math> may be taken as <math>P_r = P_{nt} + P_{lt}</math>. For a long frame, such as this one, the change in load to the interior columns, associated with lateral load is negligible.</p> <p>Therefore, <math>P_r = 335</math> kips (from previous calculations) and</p>	<p>Determine <math>B_1</math></p> <p><math>P_r</math> = required second-order axial strength using LRFD or ASD load combinations, kips.</p> <p>Note that for members in axial compression <math>P_r</math> may be taken as <math>P_r = P_{nt} + P_{lt}</math>. For a long frame, such as this one, the change in load to the interior columns, associated with lateral load is negligible.</p> <p>Therefore, <math>P_r = 247</math> kips (from previous calculations) and</p>	Specification Section C2.1b
$I = 1,110 \text{ in.}^4 \text{ (W14} \times 99)$	$I = 1,110 \text{ in.}^4 \text{ (W14} \times 99)$	
$P_{el} = \frac{\pi^2 EI}{(K_1 L)^2}$	$P_{el} = \frac{\pi^2 EI}{(K_1 L)^2}$	Specification Eqn C2-5
$= \frac{\pi(29,000 \text{ ksi})(1,110 \text{ in.}^4)}{(1.0)((13.5 \text{ ft})(12 \text{ in/ft}))^2}$	$= \frac{\pi(29,000 \text{ ksi})(1,110 \text{ in.}^4)}{(1.0)((13.5 \text{ ft})(12 \text{ in/ft}))^2}$	
$= 12,100 \text{ kips}$	$= 12,100 \text{ kips}$	
$C_m = 0.6 - 0.4(M_1 / M_2)$	$C_m = 0.6 - 0.4(M_1 / M_2)$	Specification Eqn C2-4
$= 0.6 - 0.4(157 \text{ kip-ft} / 229 \text{ kip-ft})$	$= 0.6 - 0.4(110 \text{ kip-ft} / 161 \text{ kip-ft})$	
$= 0.326$	$= 0.326$	
$\alpha = 1.0,$	$\alpha = 1.6,$	
$B_1 = \frac{C_m}{1 - \left( \frac{\alpha P_r}{P_{el}} \right)}$	$B_1 = \frac{C_m}{1 - \left( \frac{\alpha P_r}{P_{el}} \right)}$	Specification Eqn C2-2
$= \frac{0.326}{1 - \left( \frac{(1.0)(335 \text{ kips})}{12,100 \text{ kips}} \right)}$	$= \frac{0.326}{1 - \left( \frac{(1.6)(247 \text{ kips})}{12,100 \text{ kips}} \right)}$	
$= 0.335 \geq 1; \text{ Use } 1.0$	$= 0.337 \geq 1; \text{ Use } 1.0$	

<p><i>Calculate <math>B_2</math></i></p> $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ <p>where:</p> <p><math>\alpha = 1.0</math>,</p> <p><math>\sum P_{nt} = 5,250</math> kips (from computer output) and</p> <p><math>\sum P_{e2}</math> may be taken as <math>=R_M \frac{\sum HL}{\Delta_H}</math></p> <p>where <math>R_M</math> is taken as 0.85 for moment frames</p> <p><math>\sum H = 1.2D + 1.0E + 0.5L + 0.2S</math> = 195 kips (Horizontal) (from previous seismic force distribution calculations)</p> <p><math>\Delta_H = 0.562</math> in. (from computer output)</p> <p><math>\sum P_{e2} = 0.85 \frac{(195 \text{ kips})(13.5 \text{ ft})(12 \text{ in/ft})}{0.562 \text{ in.}}</math></p> <p>= 47,800 kips</p> $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ $= \frac{1}{1 - \left( \frac{(1.0)(5,250 \text{ kips})}{47,800 \text{ kips}} \right)}$ <p>= 1.12 <math>\geq</math> 1</p> <p><i>Calculate amplified moment</i></p> <p>= (1.0)(0.0 kip-ft) + (1.12)(229 kip-ft)</p> <p>= 256 kip-ft</p> <p><i>Calculate amplified axial load</i></p> <p><math>P_r = 335</math> kips (from computer analysis)</p>	<p><i>Calculate <math>B_2</math></i></p> $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ <p>where:</p> <p><math>\alpha = 1.6</math>,</p> <p><math>\sum P_{nt} = 3,750</math> kips (from computer output) and</p> <p><math>\sum P_{e2}</math> may be taken as <math>=R_M \frac{\sum HL}{\Delta_H}</math></p> <p>where <math>R_M</math> is taken as 0.85 for moment frames</p> <p><math>\sum H = D + 0.7E</math> = 137 kips (Horizontal) (from previous seismic force distribution calculations)</p> <p><math>\Delta_H = 0.394</math> in. (from computer output)</p> <p><math>\sum P_{e2} = 0.85 \frac{(137 \text{ kips})(13.5 \text{ ft})(12 \text{ in/ft})}{0.394 \text{ in.}}</math></p> <p>= 47,900 kips</p> $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ $= \frac{1}{1 - \left( \frac{(1.6)(3,750 \text{ kips})}{47,900 \text{ kips}} \right)}$ <p>= 1.14 <math>\geq</math> 1</p> <p><i>Calculate amplified moment</i></p> <p>= (1.0)(0.0 kip-ft) + (1.14)(161 kip-ft)</p> <p>= 184 kip-ft</p> <p><i>Calculate amplified axial load</i></p> <p><math>P_r = 247</math> kips (from computer analysis)</p>	<p>Specification Section C2.1b</p> <p>Specification Eqn C2-6b</p> <p>Specification Eqn C2-3</p>
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$P_r = P_{nt} + B_2 P_{lt}$ $= 335 \text{ kips} + (1.12)(0.0 \text{ kips})$ $= 335 \text{ kips}$ <p><i>Determine the controlling effective length</i></p> <p>For out-of-plane buckling in the braced frame</p> $K_y = 1.0$ <p>For in-plane buckling in the moment frame, use the nomograph</p> $K_x = 1.43$ <p>To account for leaning columns in the controlling load case</p> <p>For leaning columns,</p> $\sum Q = 3010 \text{ kips}$ $\sum P = 2490 \text{ kips}$ $K = K_o \sqrt{1 + \frac{\sum Q}{\sum P}}$ $= 1.43 \sqrt{1 + \frac{3010}{2490}} = 2.13$ $\frac{KL_x}{r_x / r_y} = \frac{2.13(13.5)}{1.66} = 17.3$	$P_r = P_{nt} + B_2 P_{lt}$ $= 247 \text{ kips} + (1.14)(0.0 \text{ kips})$ $= 247 \text{ kips}$ <p><i>Determine the controlling effective length</i></p> <p>For out-of-plane buckling in the braced frame</p> $K_y = 1.0$ <p>For in-plane buckling in the moment frame, use the nomograph</p> $K_x = 1.43$ <p>To account for leaning columns in the controlling load case</p> <p>For leaning columns,</p> $\sum Q = 2140 \text{ kips}$ $\sum P = 1740 \text{ kips}$ $K = K_o \sqrt{1 + \frac{\sum Q}{\sum P}}$ $= 1.43 \sqrt{1 + \frac{2140}{1740}} = 2.14$ $\frac{KL_x}{r_x / r_y} = \frac{2.14(13.5)}{1.66} = 17.4$	<p>Commentary Section C2.2b</p>
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<p><math>P_c = 1,041 \text{ kips (W14} \times 99 \text{ @ } KL = 17.3 \text{ ft)}</math></p> $\frac{P_r}{P_c} = \frac{335 \text{ kips}}{1,041 \text{ kips}} = 0.321 \geq 0.2$ <p><math>M_{cx} = 646 \text{ kip-ft (W14} \times 99)</math></p> $\frac{P_r}{P_c} + \left(\frac{8}{9}\right) \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right)$ $= 0.321 + \left(\frac{8}{9}\right) \left( \frac{256 \text{ kip-ft}}{646 \text{ kip-ft}} \right)$ $= 0.673 \leq 1.0 \quad \mathbf{o.k.}$	<p><math>P_c = 692 \text{ kips (W14} \times 99 \text{ @ } KL = 17.4 \text{ ft)}</math></p> $\frac{P_r}{P_c} = \frac{247 \text{ kips}}{692 \text{ kips}} = 0.357 \geq 0.2$ <p><math>M_{cx} = 430 \text{ kip-ft (W14} \times 99)</math></p> $\frac{P_r}{P_c} + \left(\frac{8}{9}\right) \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right)$ $= 0.357 + \left(\frac{8}{9}\right) \left( \frac{184 \text{ kip-ft}}{430 \text{ kip-ft}} \right)$ $= 0.737 \leq 1.0 \quad \mathbf{o.k.}$	<p>Specification Sect H1.1 Manual Table 4-1</p> <p>Manual Table 3-2</p> <p>Specification Eqn H1-1a</p>
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**METHOD 2. SIMPLIFIED DETERMINATION OF REQUIRED STRENGTH**

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 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.

LRFD	ASD
$1.2D \pm 1.0E + 0.5L + 0.2S$ (Controls columns and beams)	$D \pm (W \text{ or } 0.7 E)$ (Controls columns)  $D + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$ (Controls beams)
For a first-order analysis	For a first-order analysis
For Interior Column Design: $P_u = 335$ kips $M_{1u} = 157$ kip-ft (from first-order analysis) $M_{2u} = 229$ kip-ft (from first-order analysis)	For Interior Column Design: $P_a = 247$ kips $M_{1a} = 110$ kip-ft (from first-order analysis) $M_{2a} = 161$ kip-ft (from first-order analysis)
First-floor first-order drift = 0.562 in.	First-floor first-order drift = 0.394 in.

Then the following steps are executed.

LRFD	ASD
<i>Step 1:</i>  Lateral load = 195 kips  Deflection due to first-order elastic analysis $\Delta = 0.562$ in. between first and second floor  Floor height = 13.5 ft  Drift ratio = $(13.5 \text{ ft})(12 \text{ in/ft}) / 0.562 \text{ in}$ $= 288$	<i>Step 1:</i>  Lateral load = 140 kips  Deflection due to first-order elastic analysis $\Delta = 0.394$ in. between first and second floor  Floor height = 13.5 ft  Drift ratio = $(13.5 \text{ ft})(12 \text{ in/ft}) / 0.394 \text{ in}$ $= 411$
<i>Step 2:</i>  Design story drift limit = $0.025 h_{xs}$ $= h/400$  Adjusted Lateral load = $(288 / 400)(195 \text{ kips})$ $= 141$ kips	<i>Step 2:</i>  Design story drift limit = $0.025 h_{xs}$ $= h/400$  Adjusted Lateral load = $(411 / 400)(137 \text{ kips})$ $= 141$ kips

IBC Table 1617.3.1

<p><i>Step 3:</i></p> $\text{Load ratio} = (1.0) \frac{\text{total story load}}{\text{lateral load}}$ $= (1.0) \frac{5,250 \text{ kips}}{141 \text{ kips}}$ $= 37.2$ <p>Interpolating from the table: <math>B_2 = 1.1</math></p> <p>Which matches the value obtained in the first method to the 2 significant figures of the table</p>	<p><i>Step 3:</i> (for an ASD design the ratio must be factored by 1.6)</p> $\text{Load ratio} = (1.6) \frac{\text{total story load}}{\text{lateral load}}$ $= (1.6) \frac{3,750 \text{ kips}}{141 \text{ kips}}$ $= 42.6$ <p>Interpolating from the table: <math>B_2 = 1.1</math></p> <p>Which matches the value obtained in the first method to the 2 significant figures of the table</p>
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Note: Because the table is intentionally based on two significant figures, this value is taken as 1.1 rather than an interpolated value > 1.1. This convenient selection is within the accuracy of the method. Since the selection is in the shaded area of the chart,  $K=1.0$ .

Step 4. Multiply all the forces and moment from the first-order analysis by the value obtained from the table.

LRFD	ASD
$M_r = B_2(M_{nt} + M_{lt})$ $= 1.1(0 \text{ kip-ft} + 229 \text{ kip-ft}) = 252 \text{ kip-ft}$ $P_r = 1.1(P_{nt} + P_{lt})$ $= 1.1(335 \text{ kips} + 0.0 \text{ kips}) = 368 \text{ kips}$ For $\frac{P_r}{P_c} = \frac{368 \text{ kips}}{1,140 \text{ kips}} = 0.323 \geq 0.2$ where $P_c = 1,140 \text{ kips}$ (W14 × 99 @ $KL = 13.5 \text{ ft}$ ) $M_{cx} = 646 \text{ kip-ft}$ (W14 × 99) $\frac{P_r}{P_c} + \left(\frac{8}{9}\right) \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right)$ $= 0.323 + \left(\frac{8}{9}\right) \left(\frac{252 \text{ kip-ft}}{646 \text{ kip-ft}}\right)$ $= 0.670 \leq 1.0$ <b style="float: right;">o.k.</b>	$M_r = B_2(M_{nt} + M_{lt})$ $= 1.1(0 \text{ kip-ft} + 161 \text{ kip-ft}) = 177 \text{ kip-ft}$ $P_r = 1.1(P_{nt} + P_{lt})$ $= 1.1(247 \text{ kips} + 0.0 \text{ kips}) = 272 \text{ kips}$ For $\frac{P_r}{P_c} = \frac{272 \text{ kips}}{759 \text{ kips}} = 0.358 \geq 0.2$ $P_c = 759 \text{ kips}$ (W14 × 99 @ $KL = 13.5 \text{ ft}$ ) $M_{cx} = 430 \text{ kip-ft}$ (W14 × 99) $\frac{P_r}{P_c} + \left(\frac{8}{9}\right) \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right)$ $= 0.358 + \left(\frac{8}{9}\right) \left(\frac{177 \text{ kip-ft}}{430 \text{ kip-ft}}\right)$ $= 0.724 \leq 1.0$ <b style="float: right;">o.k.</b>

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Table 4-1  
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Table 3-2

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Eqn H1-1a

**METHOD 3. DIRECT ANALYSIS METHOD**

Second-order analysis by the direct analysis method is found in Appendix 7 of the Specification. This method requires that both the flexural stiffness and axial stiffness be reduced and that 0.2% notional lateral loads be applied in the analysis. The combination of these two modifications account for the second-order effects and the results for design can be taken directly from the analysis. A summary of the axial loads, moments and 1<sup>st</sup> floor drifts from first-order analysis is shown below:

LRFD	ASD
1.2D ± 1.0E + 0.5L + 0.2S (Controls columns and beams)	D ± (W or 0.7 E) (Controls columns) D + 0.75(W or 0.7E) + 0.75L + 0.75(L <sub>r</sub> or S or R) (Controls beams)
For a 1 <sup>st</sup> order analysis with notional loads and reduced stiffness:	For a 1 <sup>st</sup> order analysis with notional loads and with reduced stiffness:
For Interior Column Design: $P_u = 335$ kips $M_{1u} = 157$ kip-ft (from first-order analysis) $M_{2u} = 229$ kip-ft (from first-order analysis)	For Interior Column Design: $P_a = 247$ kips $M_{1a} = 110$ kip-ft $M_{2a} = 161$ kip-ft
First-floor drift due to reduced stiffnesses = 0.703 in.	First-floor drift due to reduced stiffnesses = 0.493 in.

Note: For ASD, this method requires multiplying the ASD load combinations by a factor of 1.6 in analyzing the drift of the structure, and then dividing the results by 1.6 to obtain the required strengths. The ASD forces shown above include the multiplier of 1.6.

LRFD	ASD
For this method, $K=1.0$ .	For this method, $K=1.0$
$P_c = 1,140$ kips (W14 × 99 @ $KL = 13.5$ ft)	$P_c = 759$ kips (W14 × 99 @ $KL = 13.5$ ft)
$\frac{P_r}{P_c} = \frac{335 \text{ kips}}{1,140 \text{ kips}} = 0.294 \geq 0.2$	$\frac{P_r}{P_c} = \frac{247 \text{ kips}}{759 \text{ kips}} = 0.325 \geq 0.2$
$M_{cx} = 646$ kip-ft (W14 × 99)	$M_{cx} = 430$ kip-ft (W14 × 99)
$M_r = B_1 M_{nt} + B_2 M_{lt}$	$M_r = B_1 M_{nt} + B_2 M_{lt}$
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.
Note that for members in axial compression $P_r$ may be taken as $P_r = P_{nt} + P_{lt}$ . For a long frame, such as this one, the change in load to the interior columns, associated with lateral load is negligible.	Note that for members in axial compression $P_r$ may be taken as $P_r = P_{nt} + P_{lt}$ . For a long frame, such as this one, the change in load to the interior columns, associated with lateral load is negligible.
Therefore, $P_r = 335$ kips (from previous calculations) and	Therefore, $P_r = 247$ kips (from previous calculations) and

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Table 4-1

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Table 3-2



<p><math>I = 1,110 \text{ in.}^4</math> (W14×99)</p> $P_{el} = \frac{\pi^2 EI}{(K_1 L)^2}$ $= \frac{\pi(29,000 \text{ ksi})(1,110 \text{ in.}^4)}{(1.0)((13.5 \text{ ft})(12 \text{ in/ft}))^2}$ $= 12,106 \text{ kips}$ $C_m = 0.6 - 0.4(M_1 / M_2)$ $= 0.6 - 0.4(157 \text{ kip-ft} / 229 \text{ kip-ft})$ $= 0.326$ <p><math>\alpha = 1.0,</math></p> $B_1 = \frac{C_m}{1 - \left( \frac{\alpha P_r}{P_{el}} \right)}$ $= \frac{0.326}{1 - \left( \frac{(1.0)(335 \text{ kips})}{12,100 \text{ kips}} \right)}$ $= 0.335 \geq 1; \text{ Use } 1.0$ <p>Calculate <math>B_2</math></p> $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ <p>where:</p> <p><math>\alpha = 1.0,</math></p> <p><math>\sum P_{nt} = 5,250 \text{ kips}</math> (from computer output)</p> <p><math>\sum P_{e2}</math> may be taken as <math>=R_M \frac{\sum HL}{\Delta_H}</math></p> <p>where <math>R_M</math> is taken as 0.85 for moment frames</p> $\sum H = 1.2D + 1.0E + 0.5L + 0.2S$ $= 195 \text{ kips (Horizontal)}$ <p>(from previous seismic force distribution calculations)</p>	<p><math>I = 1,110 \text{ in.}^4</math> (W14×99)</p> $P_{el} = \frac{\pi^2 EI}{(K_1 L)^2}$ $= \frac{\pi(29,000 \text{ ksi})(1,110 \text{ in.}^4)}{(1.0)((13.5 \text{ ft})(12 \text{ in/ft}))^2}$ $= 12,100 \text{ kips}$ $C_m = 0.6 - 0.4(M_1 / M_2)$ $= 0.6 - 0.4(110 \text{ kip-ft} / 161 \text{ kip-ft})$ $= 0.327$ <p><math>\alpha = 1.6,</math></p> $B_1 = \frac{C_m}{1 - \left( \frac{\alpha P_r}{P_{el}} \right)}$ $= \frac{0.327}{1 - \left( \frac{(1.6)(247 \text{ kips})}{12,100 \text{ kips}} \right)}$ $= 0.338 \geq 1; \text{ Use } 1.0$ <p>Calculate <math>B_2</math></p> $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ <p>where:</p> <p><math>\alpha = 1.6,</math></p> <p><math>\sum P_{nt} = 3,750 \text{ kips}</math> (from computer output)</p> <p><math>\sum P_{e2}</math> may be taken as <math>=R_M \frac{\sum HL}{\Delta_H}</math></p> <p>where <math>R_M</math> is taken as 0.85 for moment frames</p> $\sum H = D + 0.7E$ $= 137 \text{ kips (Horizontal)}$ <p>(from previous seismic force distribution calculations)</p>
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<p><math>\Delta_H = 0.703 \text{ in.}</math></p> $\sum P_{e2} = 0.85 \frac{(195 \text{ kips})(13.5 \text{ ft})(12 \text{ in/ft})}{0.703 \text{ in.}}$ $= 38,200 \text{ kips}$ $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ $= \frac{1}{1 - \left( \frac{(1.0)(5,250 \text{ kips})}{38,200 \text{ kips}} \right)}$ $= 1.16 \geq 1$ <p><i>Calculate amplified moment</i></p> $= (1.0)(0.0 \text{ kip-ft}) + (1.16)(229 \text{ kip-ft})$ $= 266 \text{ kip-ft}$ <p><i>Calculate amplified axial load</i></p> <p><math>P_r = 335 \text{ kips}</math> (from computer analysis)</p> $P_r = P_{nt} + B_2 P_{lt}$ $= 335 \text{ kips} + (1.16)(0.0 \text{ kips})$ $= 335 \text{ kips}$ $\frac{P_r}{P_c} + \left( \frac{8}{9} \right) \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right)$ $= 0.294 + \left( \frac{8}{9} \right) \left( \frac{266 \text{ kip-ft}}{646 \text{ kip-ft}} \right)$ $= 0.660 \leq 1.0$	<p><math>\Delta_H = 0.493 \text{ in. (from computer output)}</math></p> $\sum P_{e2} = 0.85 \frac{(137 \text{ kips})(13.5 \text{ ft})(12 \text{ in/ft})}{0.493 \text{ in.}}$ $= 38,300 \text{ kips}$ $B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ $= \frac{1}{1 - \left( \frac{(1.6)(3,750 \text{ kips})}{38,300 \text{ kips}} \right)}$ $= 1.19 \geq 1$ <p><i>Calculate amplified moment</i></p> $= (1.0)(0.0 \text{ kip-ft}) + (1.19)(161 \text{ kip-ft})$ $= 192 \text{ kip-ft}$ <p><i>Calculate amplified axial load</i></p> <p><math>P_r = 247 \text{ kips}</math> (from computer analysis)</p> $P_r = P_{nt} + B_2 P_{lt}$ $= 247 \text{ kips} + (1.19)(0.0 \text{ kips})$ $= 247 \text{ kips}$ $\frac{P_r}{P_c} + \left( \frac{8}{9} \right) \left( \frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right)$ $= 0.325 + \left( \frac{8}{9} \right) \left( \frac{192 \text{ kip-ft}}{430 \text{ kip-ft}} \right)$ $= 0.722 \leq 1.0$
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Specification  
Eqn H1-1b

**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 computer analysis.

LRFD	ASD
$1.2D + 1.0E + 0.5L$ =1.2(153 kip-ft) +1.0(117 kip-ft) + 0.5(80.5 kip-ft) =184 kip-ft + 117 kip-ft + 40.3 kip-ft =341 kip-ft <b>controls</b>	$D + 0.7E$ =153 kip-ft + 0.7(112 kip-ft) =153 kip-ft + 78.4 kip-ft =231 kip-ft  $D + 0.75(0.7E) + 0.75L$ =153 kip-ft + 0.75(0.7)(112 kip-ft) + 0.75(80.5 kip-ft) = 153 kip-ft + 58.8 kip-ft + 60.4 kip-ft = 272 kip-ft <b>controls</b>
The available moment is 503 kip-ft 503 kip-ft > 341 kip-ft <b>o.k.</b>	The available moment is 334 kip-ft 334 kip-ft > 272 kip-ft <b>o.k.</b>

Manual  
Table 3-2

Note: because the typical exterior beam was increased to a W24x55 to help with drift, all load combinations resulted in less demand than the available moment strength of the beam.

The ends 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 same stability design requirements from Chapter C were applied to this system.

*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.

First 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 strength,  $M_r$ , and axial strength,  $P_r$ , are as follows:

LRFD	ASD
$P_{el} = \frac{\pi^2 EI}{(K_1 L)^2}$ $= \frac{\pi(29,000 \text{ ksi})(425 \text{ in}^4)}{(1.0)((13.5 \text{ ft})(12 \text{ in/ft}))^2}$ $= 4,635 \text{ kips}$ $I = 425 \text{ in}^4 \text{ (W12} \times 58)$	$P_{el} = \frac{\pi^2 EI}{(K_1 L)^2}$ $= \frac{\pi(29,000 \text{ ksi})(425 \text{ in}^4)}{(1.0)((13.5 \text{ ft})(12 \text{ in/ft}))^2}$ $= 4,635 \text{ kips}$ $I = 425 \text{ in}^4 \text{ (W12} \times 58)$
$B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ $= \frac{1}{1 - \left( \frac{(1.0)(5,452 \text{ kips})}{150,722 \text{ kips}} \right)}$ $= 1.04 \geq 1$	$B_2 = \frac{1}{1 - \left( \frac{\alpha \sum P_{nt}}{\sum P_{e2}} \right)}$ $= \frac{1}{1 - \left( \frac{(1.6)(3,894 \text{ kips})}{150,722 \text{ kips}} \right)}$ $= 1.04 \geq 1$
$\sum P_{nt} = 5,452 \text{ kips (from computer output)}$	$\sum P_{nt} = 3,894 \text{ kips (from computer output)}$
$\sum P_{e2} = R_M \frac{\sum HL}{\Delta_H}$ $= 1.0 \frac{(195 \text{ kips})(13.5 \text{ ft})(12 \text{ in/ft})}{0.210 \text{ in.}}$ $= 150,722 \text{ kips}$	$\sum P_{e2} = R_M \frac{\sum HL}{\Delta_H}$ $= 1.0 \frac{(137 \text{ kips})(13.5 \text{ ft})(12 \text{ in/ft})}{0.147 \text{ in.}}$ $= 150,722 \text{ kips}$

Specification  
Eqn C2-5

Specification  
Eqn C2-3

Specification  
Eqn C2-6b

$\sum H = 1.2D + 1.0E + L + 0.2S$ <p>= 195 kips (from previous calculations)</p> $\Delta_H = 0.210 \text{ in. (from computer output)}$ $P_r = P_{nt} + B_2 P_{lt}$ <p>= 242 kips + (1.04)(220 kips)</p> <p>= 470 kips</p> $P_c = 553 \text{ kips (W12} \times 58)$ <p>For <math>\frac{P_r}{P_c} = \frac{470 \text{ kips}}{553 \text{ kips}} = 0.850 \leq 1.0</math></p>	$\sum H = D + 0.7E$ <p>= 137 kips (from previous calculations)</p> $\Delta_H = 0.147 \text{ in. (from computer output)}$ $P_r = P_{nt} + B_2 P_{lt}$ <p>= 173 kips + (1.04)(179 kips)</p> <p>= 360 kips</p> $P_c = 368 \text{ kips (W12} \times 58)$ <p>For <math>\frac{P_r}{P_c} = \frac{360 \text{ kips}}{368 \text{ kips}} = 0.977 \leq 1.0</math></p>	<p>Specification Eqn C2-1b</p> <p>Manual Table 4-1</p>
---	---	--

Note: Notice that the lower displacements of the braced frame produce much lower values for  $B_2$ . Similar values could be expected for the other two methods of analysis.

**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 W16x31

The loads for these edge beams with a DL of 0.75 ksf (5.5 ft) and exterior wall at 0.503 kip/ft and a LL of 0.80 ksf (5.5 ft) are  $DL_{tot} = 0.916$  kip/ft,  $LL_{tot} = 0.440$  kip/ft.

The controlling load combinations are LRFD (1.2D+1.0E+0.50L) and ASD (1.0D+0.75(0.7E)+0.75L) or (D+0.7E)

LRFD	ASD
$M_u = 1.2(58.0 \text{ kip-ft}) + 0.50(27.8 \text{ kip-ft})$ $= 83.5 \text{ kip-ft}$	$M_a = 1.0(58.0 \text{ kip-ft}) + 0.75(27.8 \text{ kip-ft})$ $= 78.9 \text{ kip-ft}$ or $M_a = 58.0 \text{ kip-ft}$
Load from the diaphragm shear	Load from the diaphragm shear
$F_p = 80.3 \text{ kips}$	$F_p = 0.75(0.70)(80.3 \text{ kips}) = 42.2 \text{ kips}$
	or $F_p = 0.70(80.3 \text{ kips}) = 56.2 \text{ kips}$
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
$V = 80.3 \text{ kips} / 2(45 \text{ ft}) = 0.892 \text{ kip/ft}$	$V = 42.2 \text{ kips} / 2(45 \text{ ft}) = 0.469 \text{ kip/ft}$ or $V = 56.2 \text{ kips} / 2(45 \text{ ft}) = 0.624 \text{ kip/ft}$
The top flange stress due to bending	The top flange stress due to bending
$= 83.5 \text{ kip-ft}(12 \text{ in/ft}) / 47.2 \text{ in}^3$	$= 78.9 \text{ kip-ft}(12 \text{ in/ft}) / 47.2 \text{ in}^3$
$= 21.2 \text{ ksi}$	$= 20.1 \text{ ksi}$
	or $= 58.0 \text{ kip-ft}(12 \text{ in/ft}) / 47.2 \text{ in}^3$
	$= 14.7 \text{ ksi}$

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.

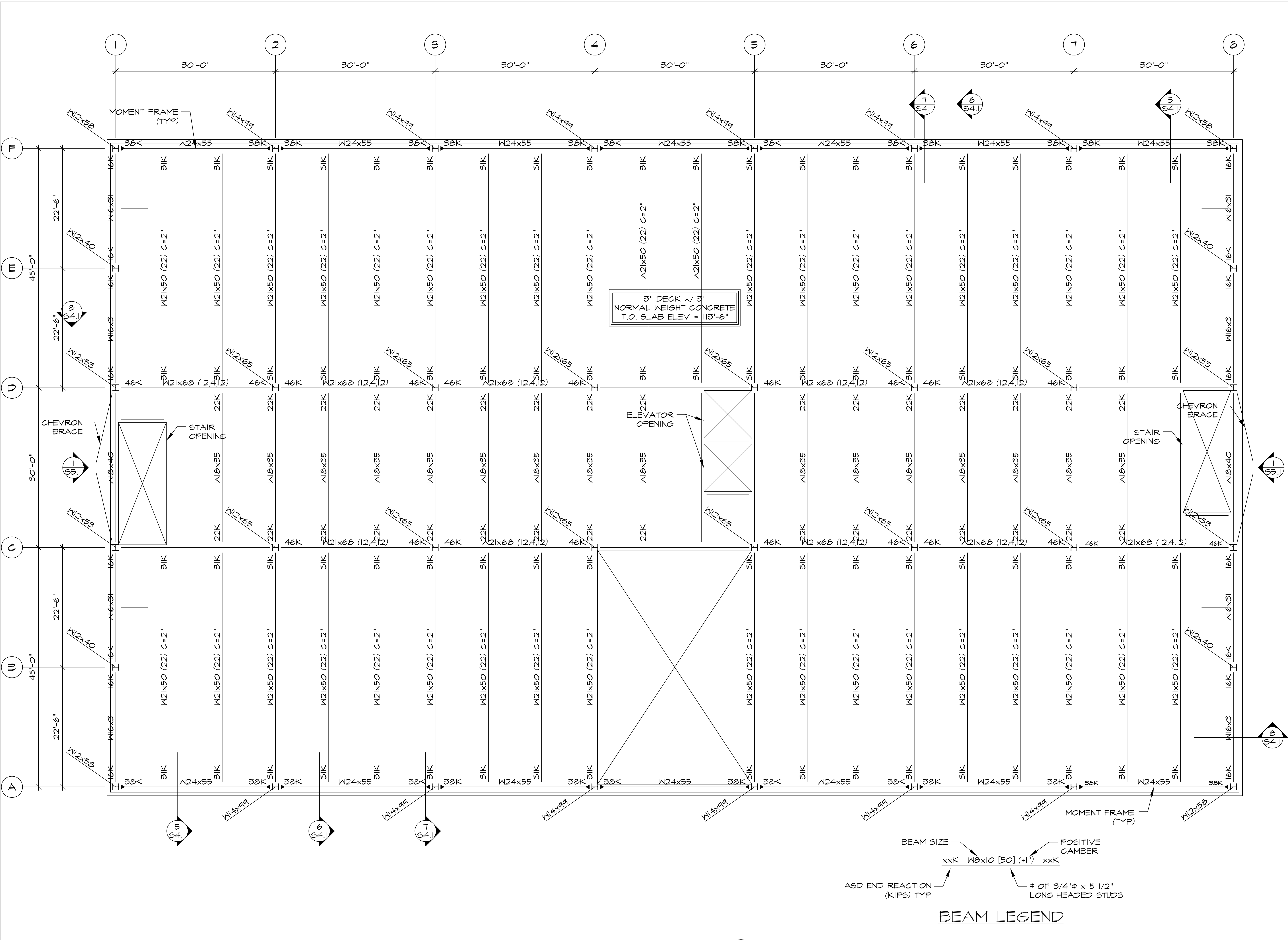
$$\text{Area} = 5.53 \text{ in.}(0.440 \text{ in.}) + 8(0.275 \text{ in.}^2) = 2.43 \text{ in.}^2 + 0.605 \text{ in.}^2 = 3.04 \text{ in.}^2$$

Ignoring the small segment of the beam between Grid C and D, the stress due to the drag strut force is:

LRFD	ASD
$= \left( \frac{80.3 \text{ ft}}{2} \right) (0.892 \text{ kip/ft}) / 3.04 \text{ in.}^2 = 13.2 \text{ ksi}$	$= \left( \frac{80.3 \text{ ft}}{2} \right) (0.469 \text{ kip/ft}) / 3.04 \text{ in.}^2 = 6.94 \text{ ksi}$
	or
	$= 45 \text{ ft}(0.624 \text{ kip/ft}) / 3.04 \text{ in.}^2 = 9.23 \text{ ksi}$
Total top flange stress	Total top flange stress
= 21.2 ksi + 13.2 ksi	= 20.1 ksi + 6.49 ksi
= 34.4 ksi.	= 26.6 ksi <b>controls</b>
	or
	= 14.7 ksi + 9.23 ksi
	= 23.9 ksi
For bending or compression $\phi = 0.90$	For bending or compression $\Omega = 1.67$
$\phi F_y = 45 \text{ ksi} > 34.4 \text{ ksi}$ <b>o.k.</b>	$F_y / \Omega = 29.9 \text{ ksi} > 26.6 \text{ ksi}$ <b>o.k.</b>

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.

**BUILDING 3  
ANYWHERE**



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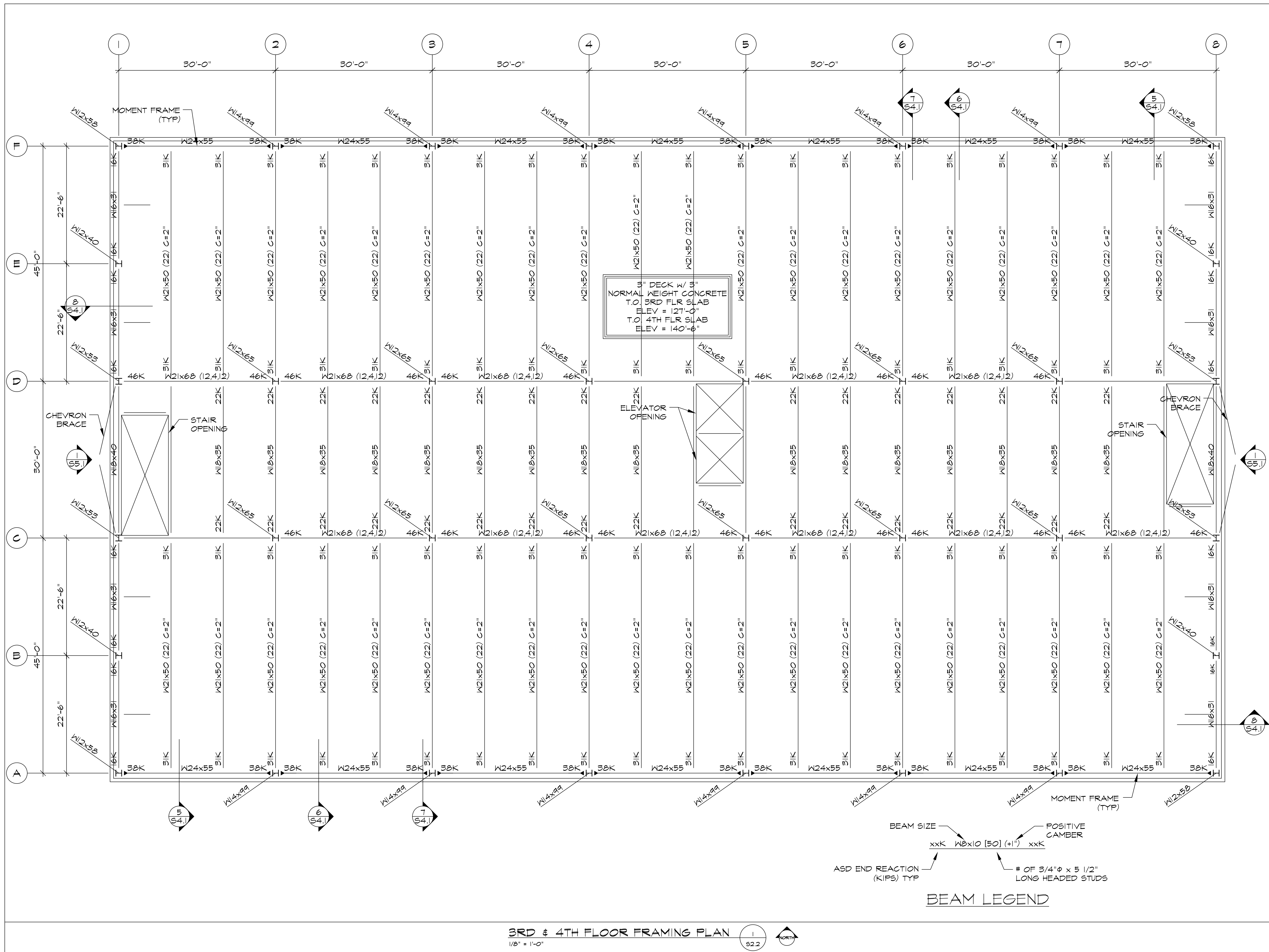
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ISSUE:	

**2ND FLOOR FRAMING PLAN**

**S2.1**



# BUILDING 3 ANYWHERE



3" DECK w/ 3"  
 NORMAL WEIGHT CONCRETE  
 T.O. 3RD FLR SLAB  
 ELEV = 127'-0"  
 T.O. 4TH FLR SLAB  
 ELEV = 140'-6"

BEAM SIZE  
 xxK W8x10 [50] (+") xxK  
 POSITIVE CAMBER  
 ASD END REACTION (KIPS) TYP  
 # OF 3/4"φ x 5 1/2" LONG HEADED STUDS

BEAM LEGEND

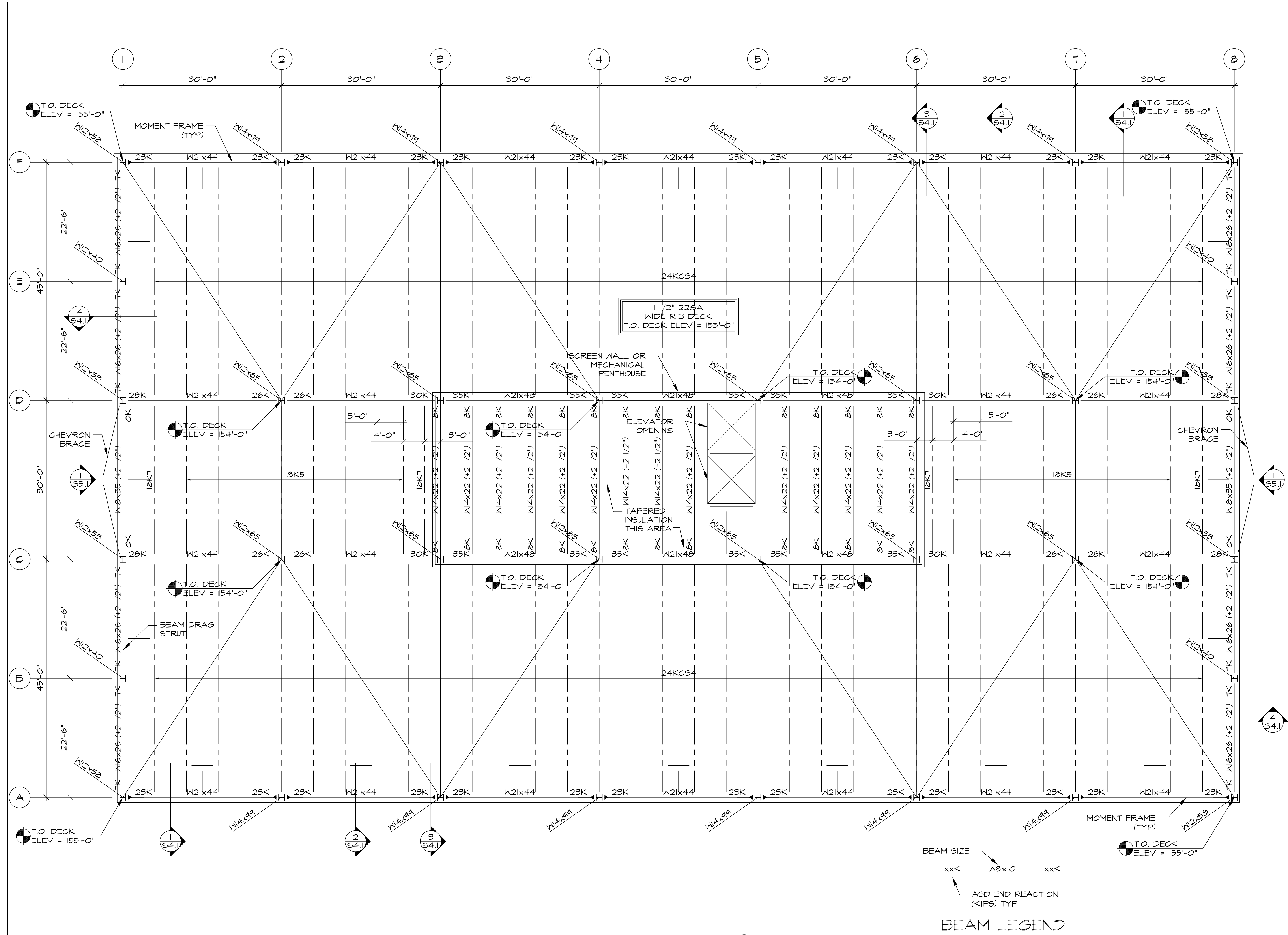
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ME	YOU
ISSUE:	

### 3RD FLOOR FRAMING PLAN

# S2.2

**BUILDING 3**  
**ANYWHERE**



**ROOF FRAMING PLAN**  
1/8" = 1'-0"

**BEAM LEGEND**  
xxK WxxK xxK  
ASD END REACTION (KIPS) TYP

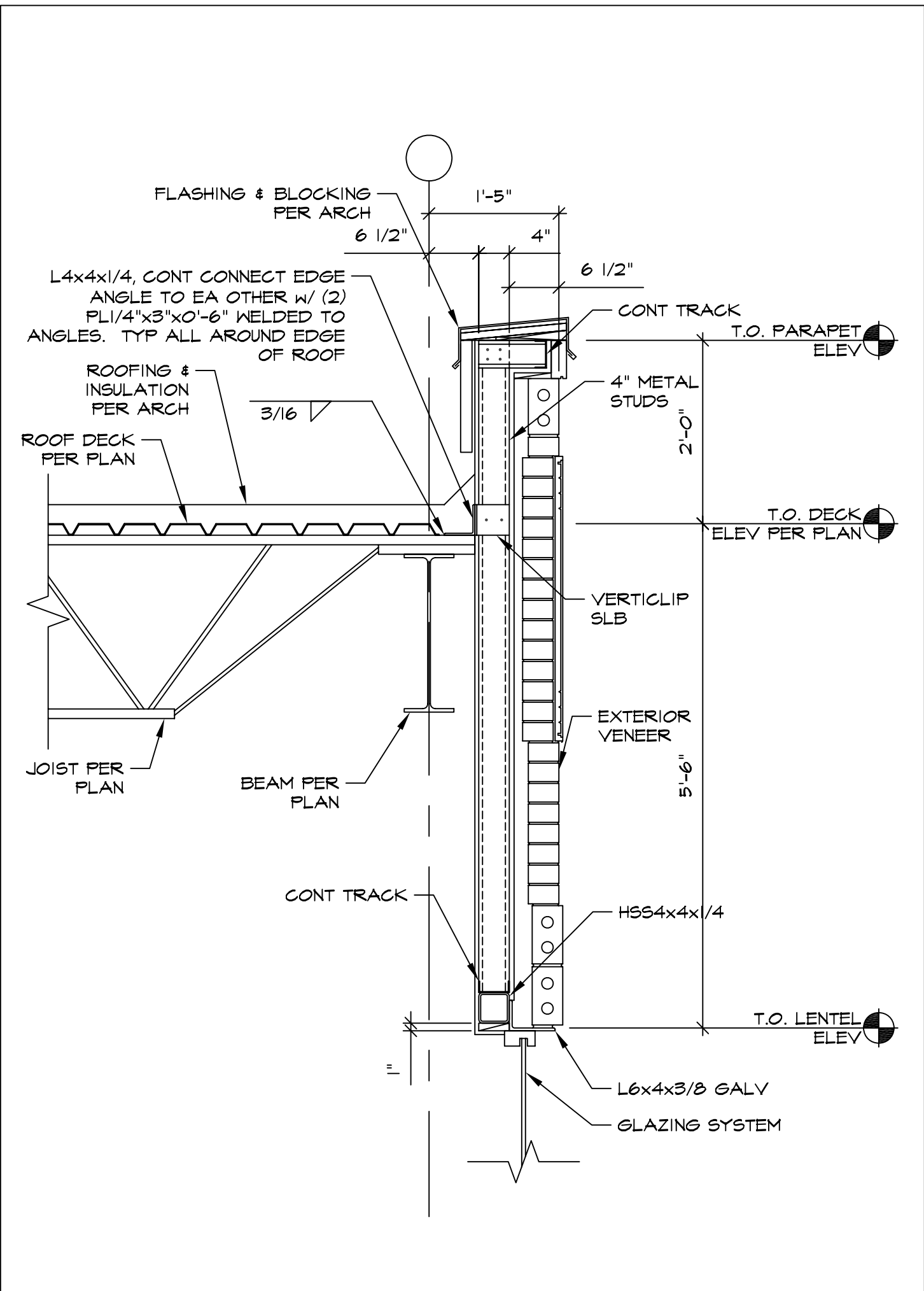
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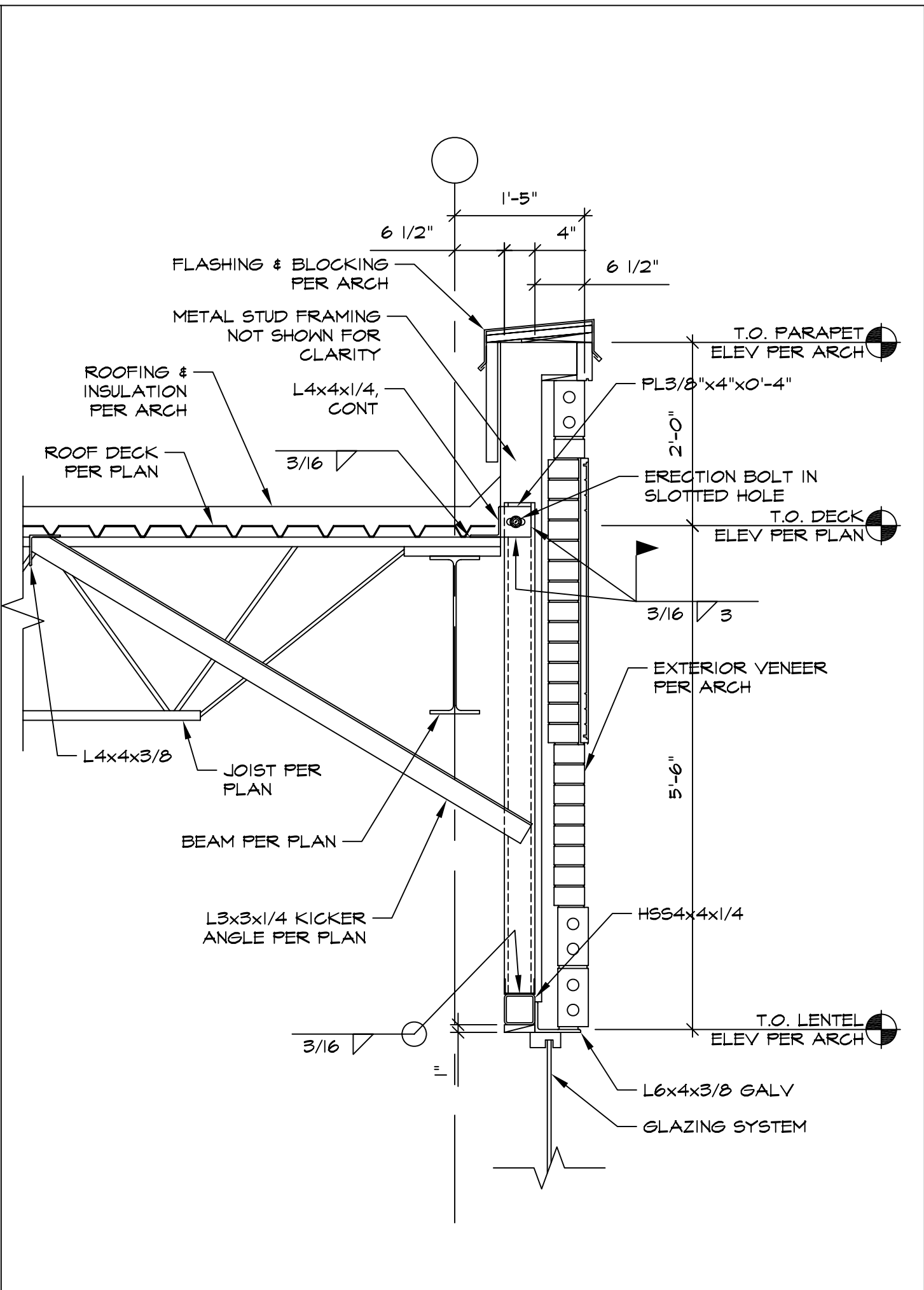
**ROOF FRAMING PLAN**

**S2.3**

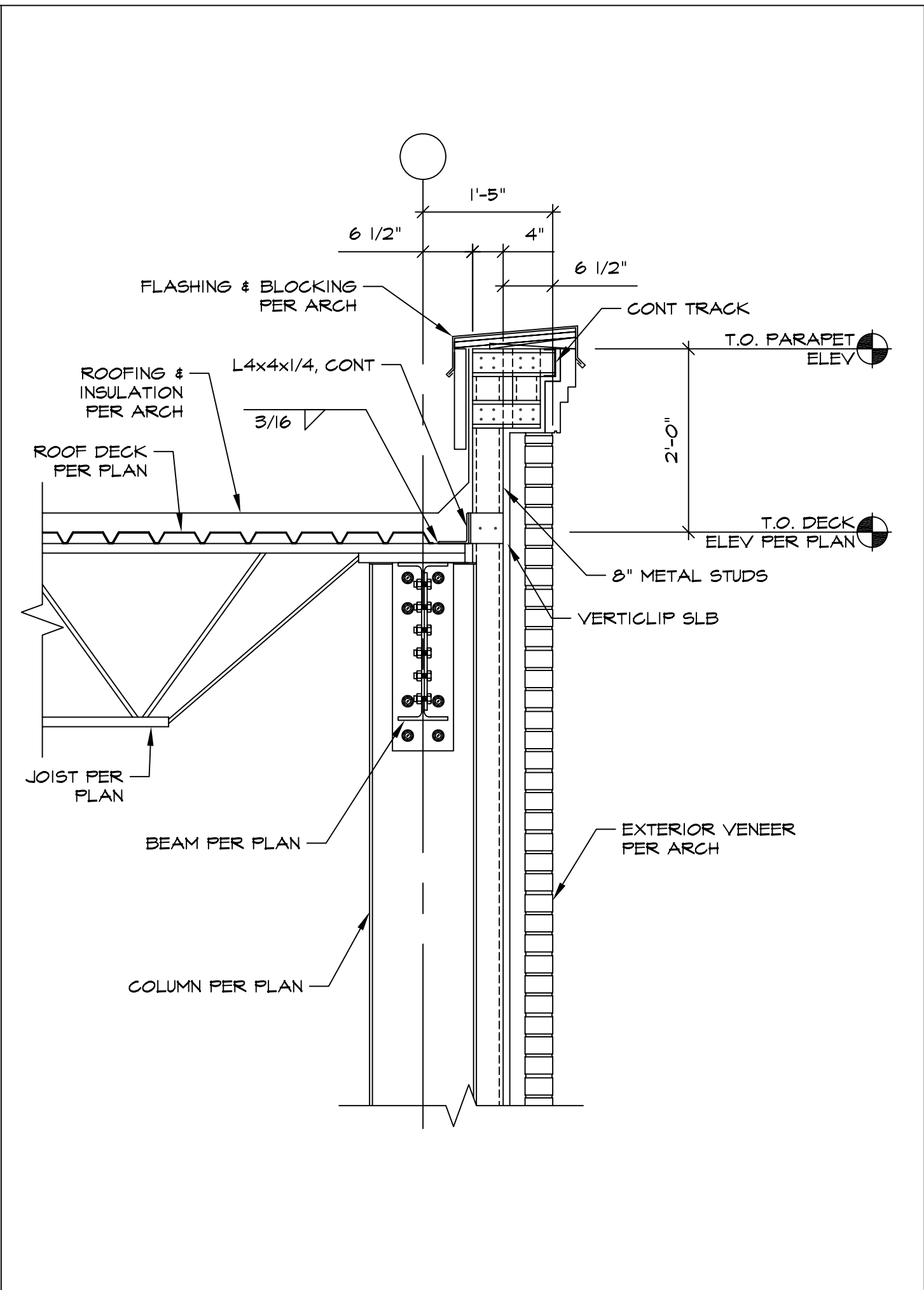
**BUILDING 3**  
**ANYWHERE**



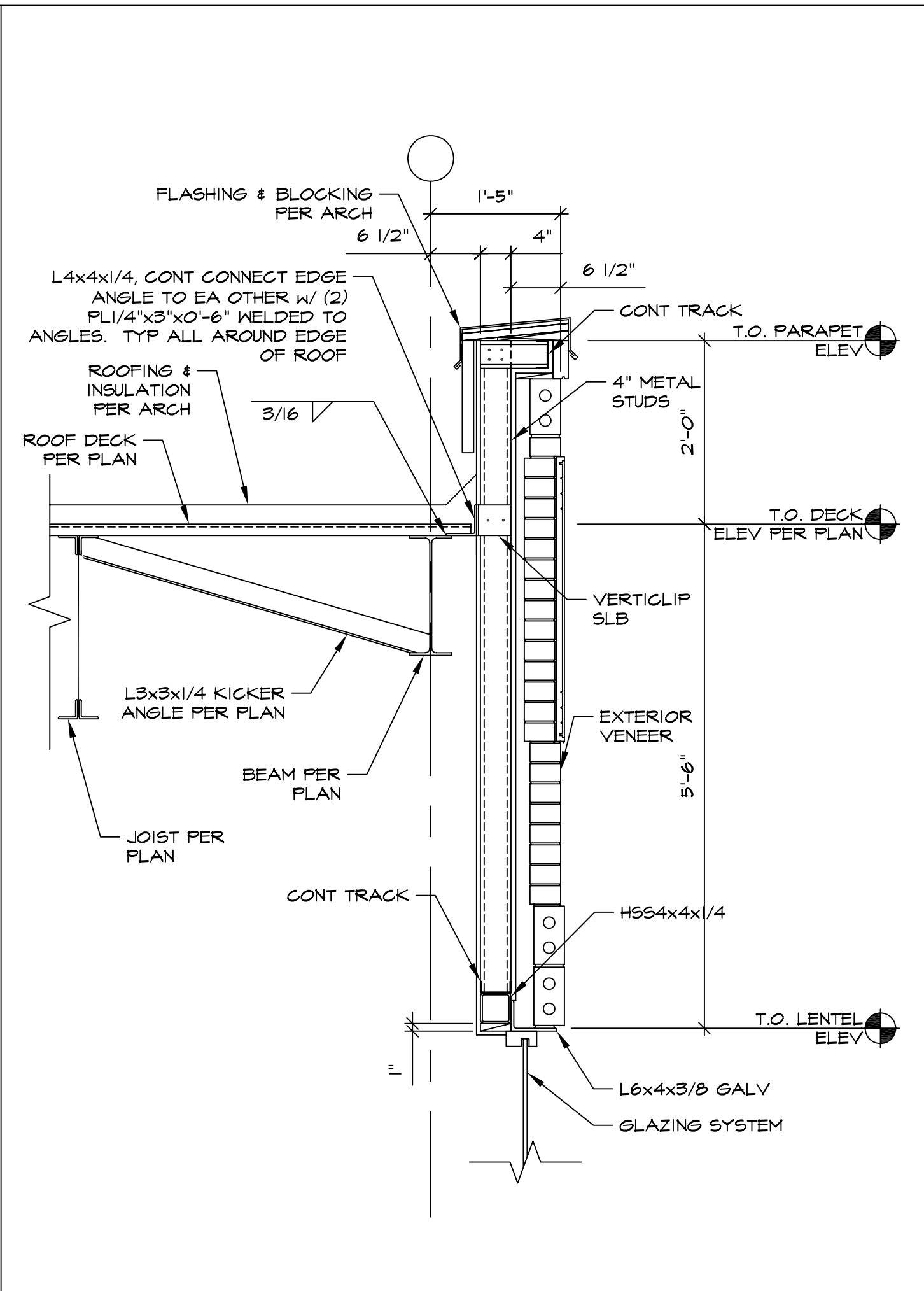
SECTION 1  
3/4" = 1'-0" S4.1



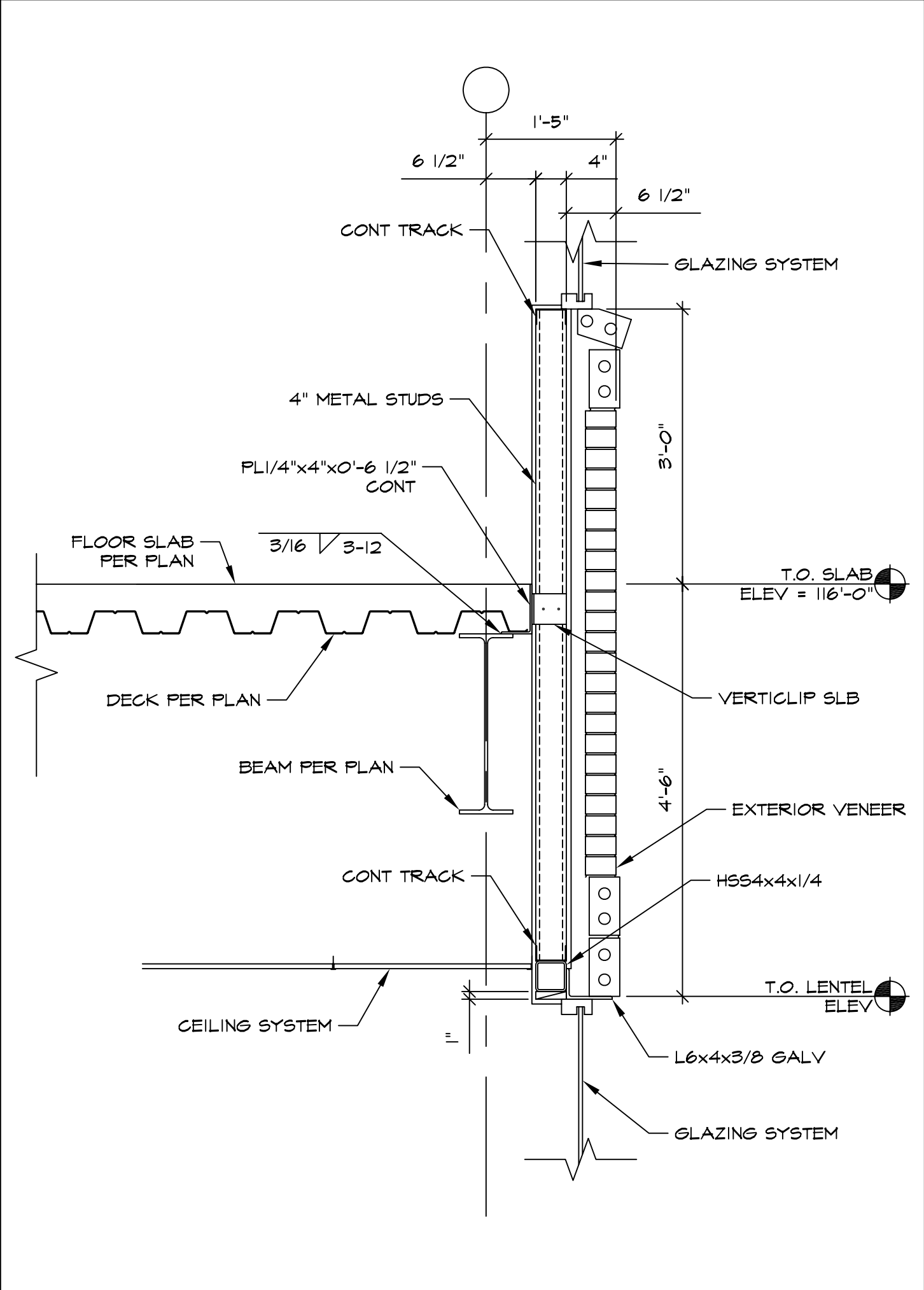
SECTION 2  
3/4" = 1'-0" S4.1



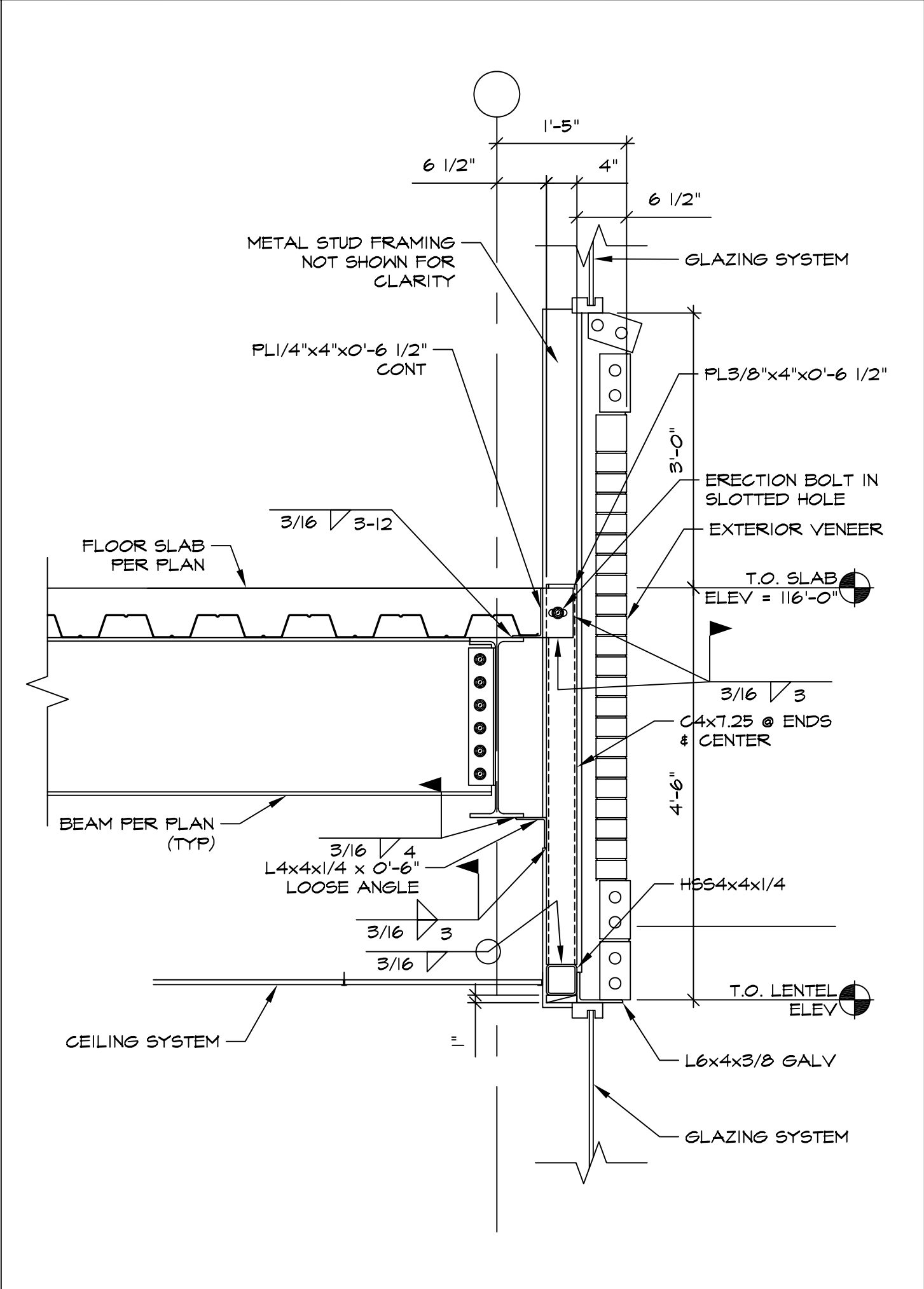
SECTION 3  
3/4" = 1'-0" S4.1



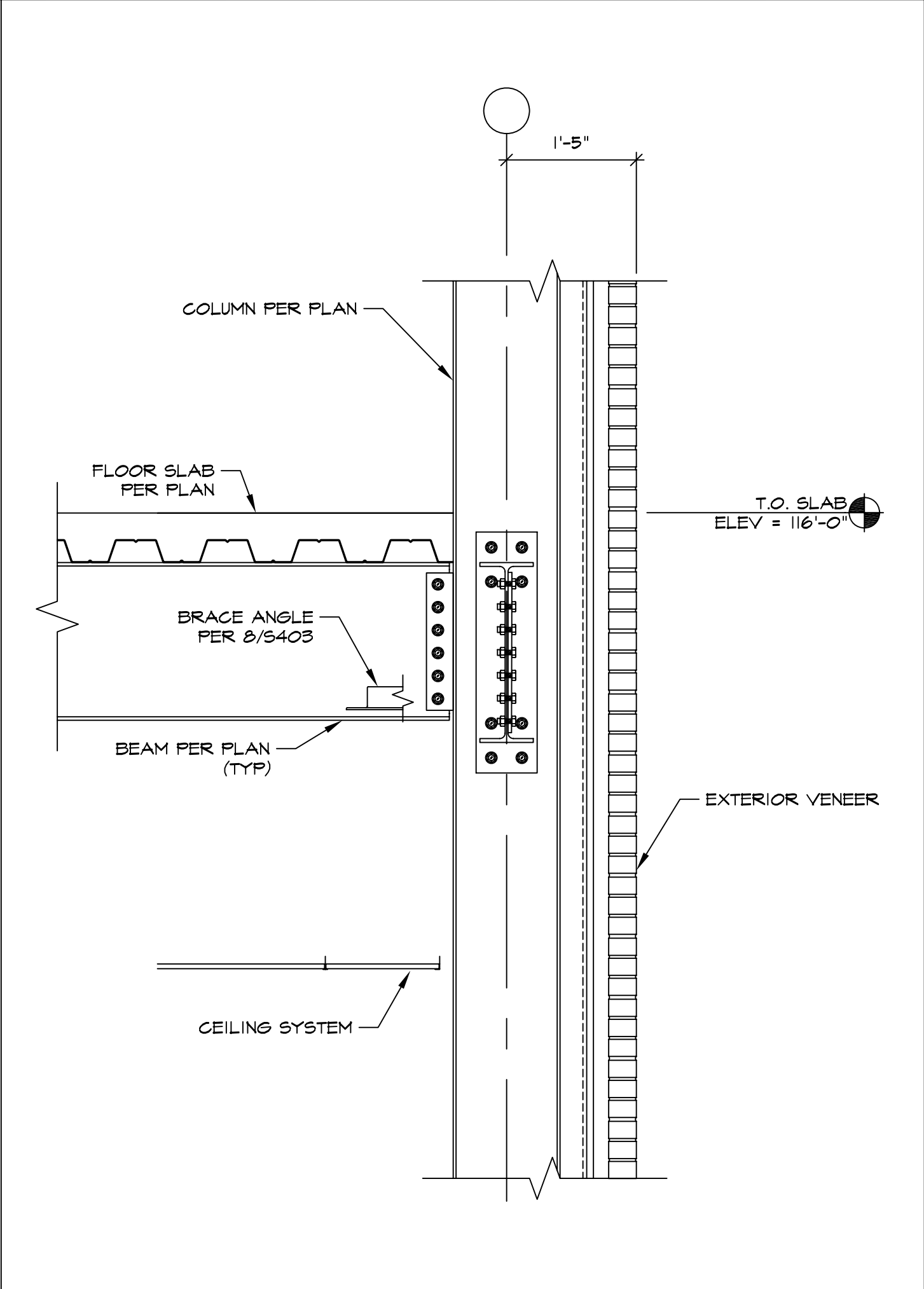
SECTION 4  
3/4" = 1'-0" S4.1



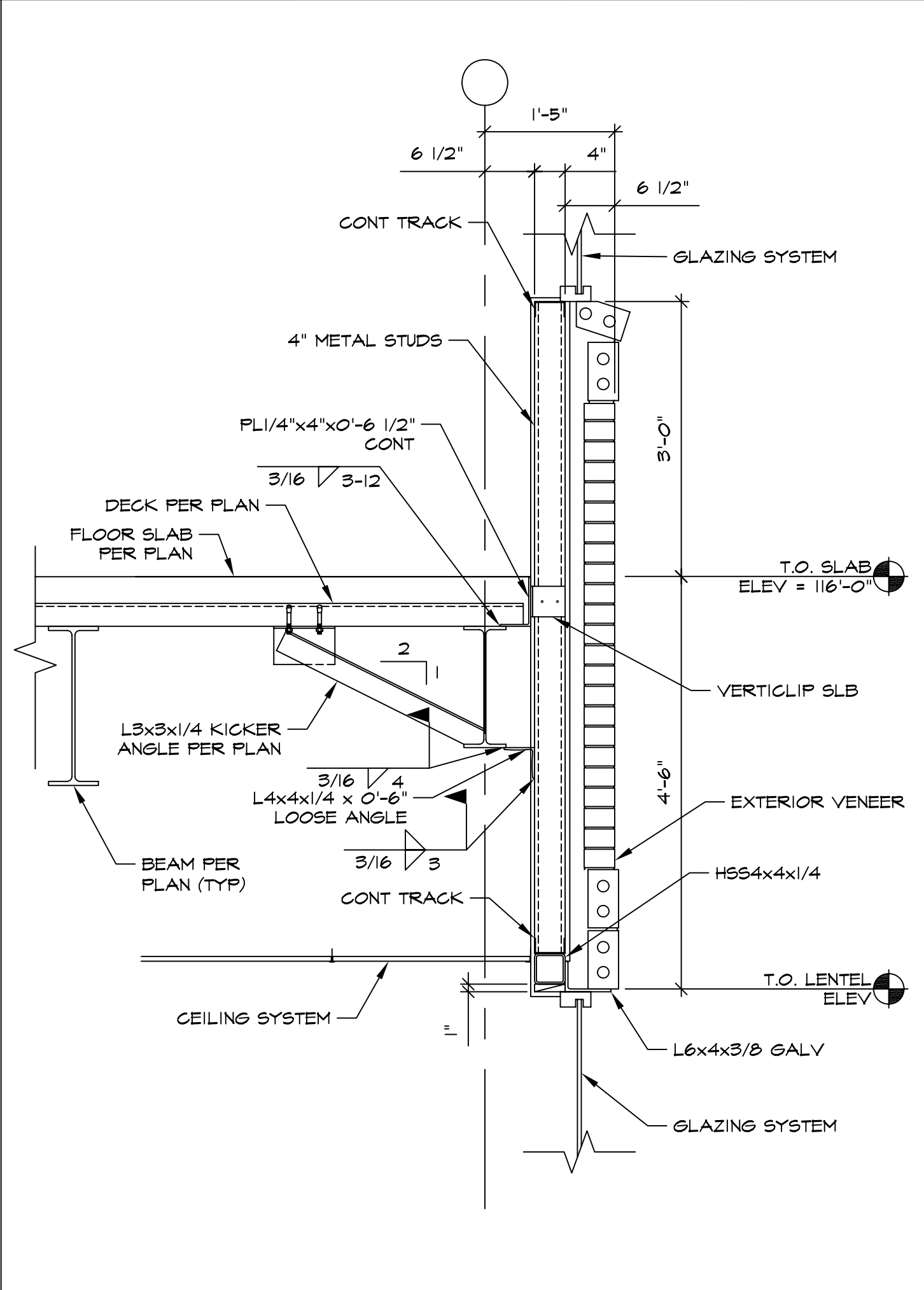
SECTION 5  
3/4" = 1'-0" S4.1



SECTION 6  
3/4" = 1'-0" S4.1



SECTION 7  
3/4" = 1'-0" S4.1



SECTION 8  
3/4" = 1'-0" S4.1

REVISIONS

DESCRIPTION	DATE

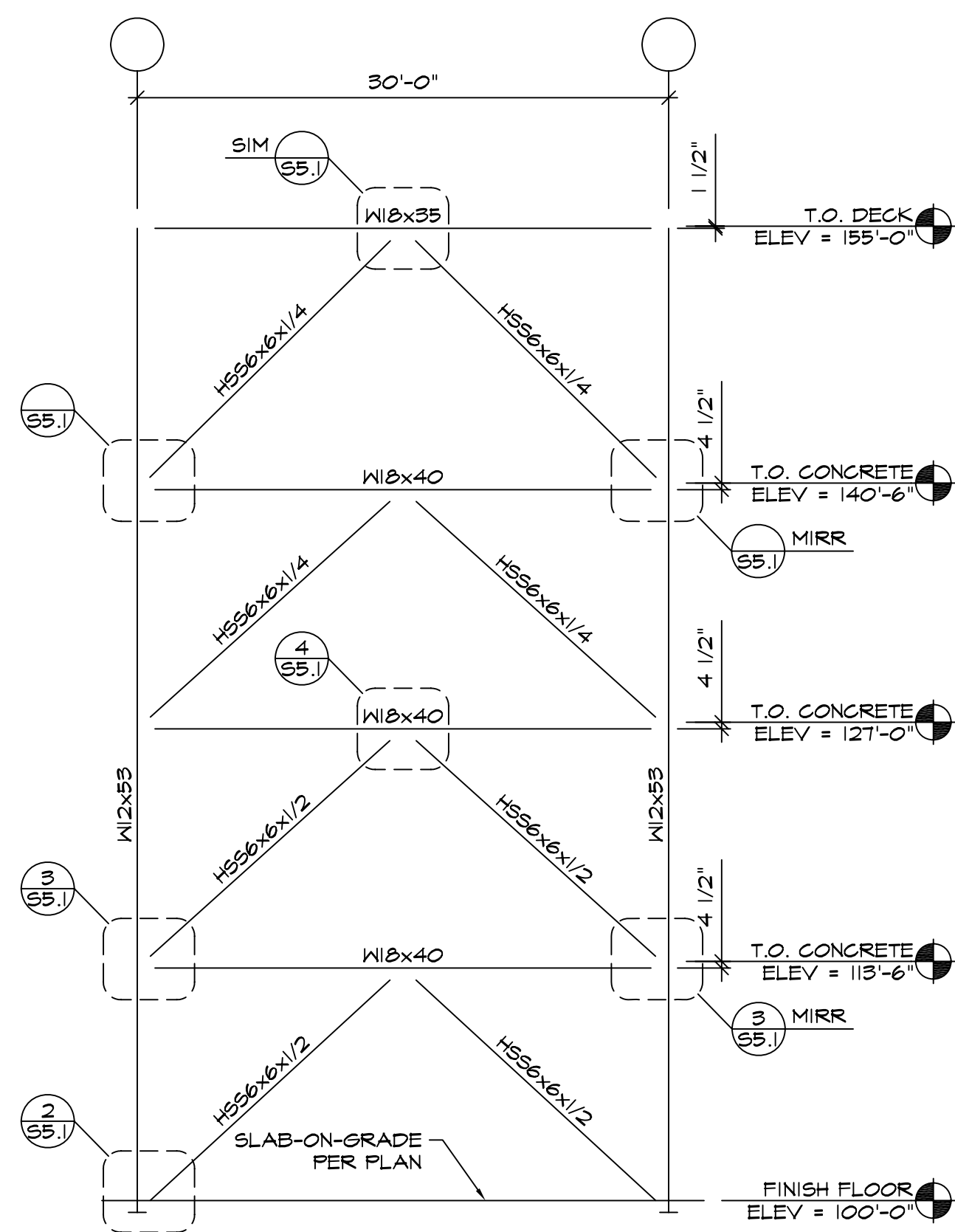
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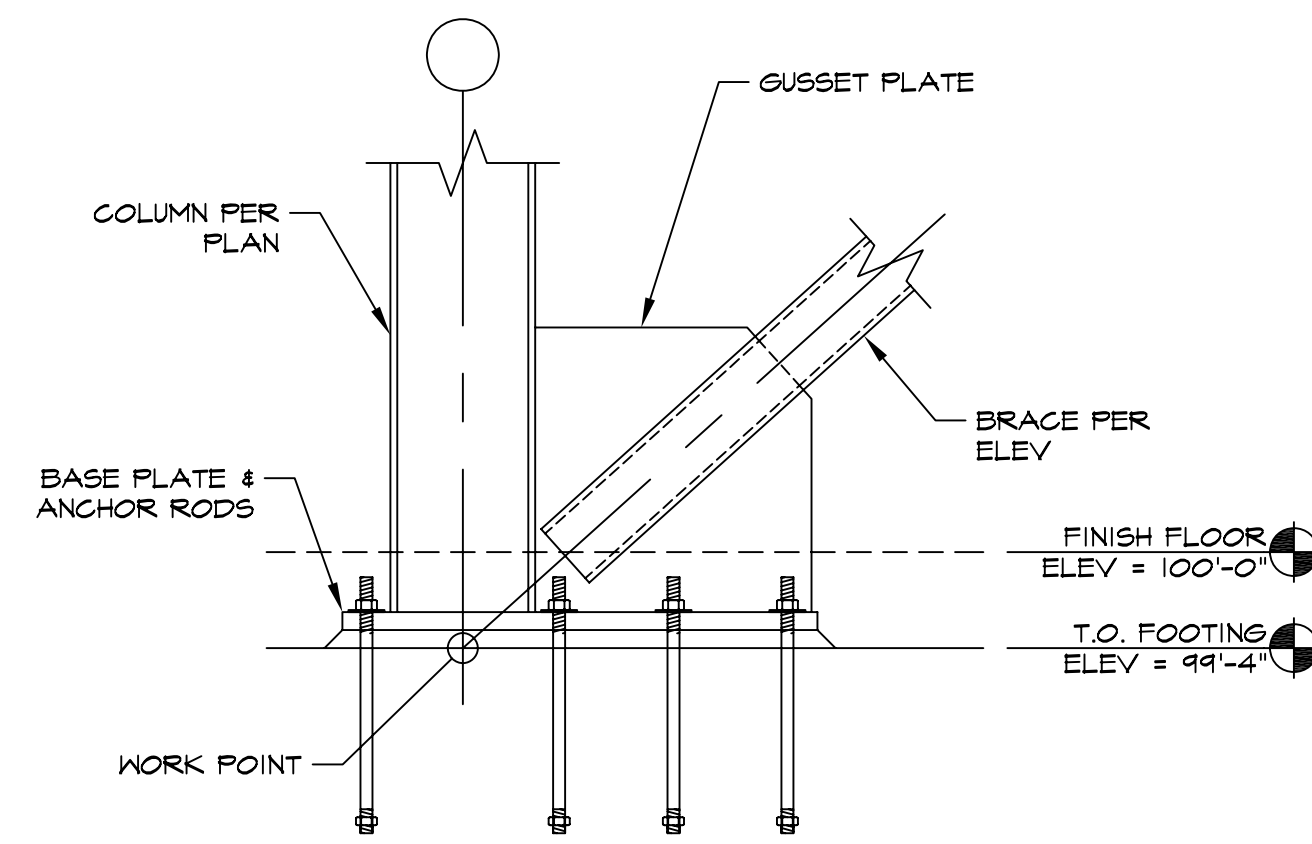
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**ROOF FRAMING SECTIONS**

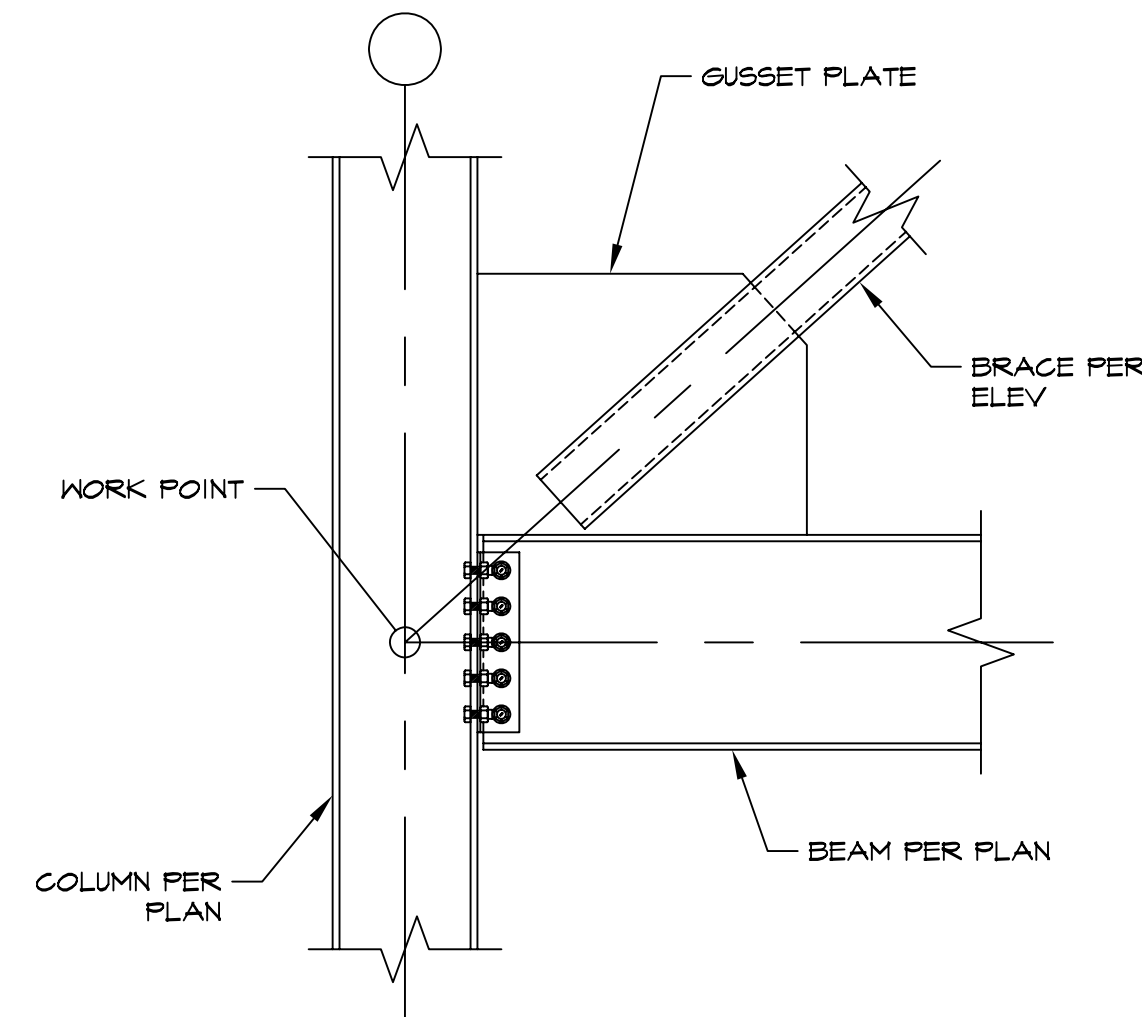
**S4.1**



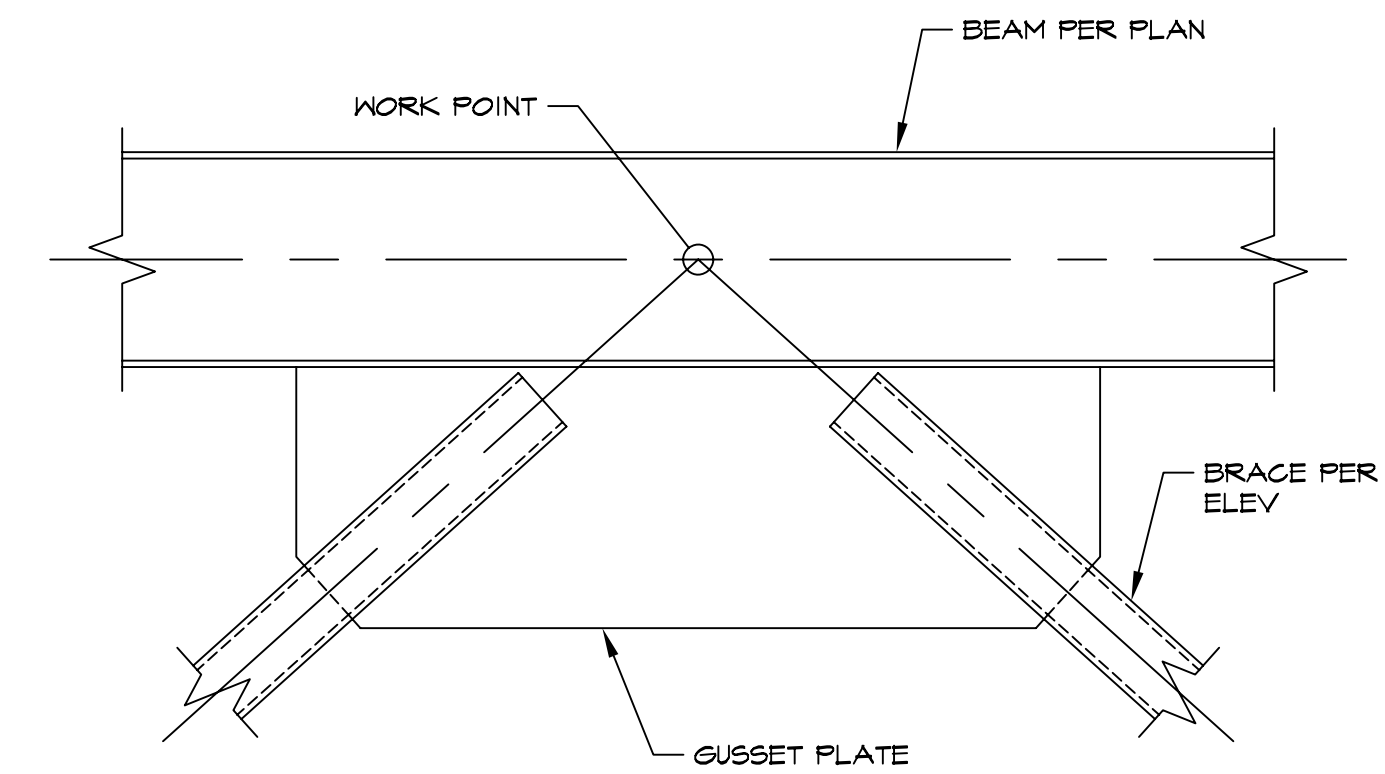
CHEVRON BRACE ELEVATION 1  
N.T.S. S5.1



DETAIL 2  
1 1/2" = 1'-0" S5.1



DETAIL 3  
1 1/2" = 1'-0" S5.1



DETAIL 4  
1 1/2" = 1'-0" S5.1

NOT USED 5  
S5.1

NOT USED 6  
S5.1

NOT USED 7  
S5.1

NOT USED 8  
S5.1

**BUILDING 3**  
**ANYWHERE**

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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	2000 LRFD
<b>A. GENERAL PROVISIONS</b>					
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>B. DESIGN REQUIREMENTS</b>					
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
<b>C. STABILITY ANALYSIS AND DESIGN</b>					
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	2000 LRFD
<b>C2. Calculation of Required Strengths</b>					
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	↓	↓	↓	↓	↓
<b>D. DESIGN OF MEMBERS FOR TENSION</b>					
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
<b>E. DESIGN OF MEMBERS FOR COMPRESSION</b>					
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.
<b>F. DESIGN OF MEMBERS FOR FLEXURE</b>					
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				
	1989 ASD	1999 LRFD	2000 LRFD HSS	SINGLE ANGLE	
				1989 ASD	2000 LRFD
F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent About Their Major Axis 1. Compression Flange Yielding 2. Lateral-Torsional Buckling 3. Compression Flange Yielding 4. Tension Flange Yielding	G2 ↓ NEW	App. F1 ↓ NEW	N.A. ↓	N.A. ↓	N.A. ↓
F6. I-Shaped Members and Channels Bent About Their Minor Axis 1. Yielding 2. Flange Local Buckling	F2 ↓	App. F1 ↓	N.A. ↓	N.A. ↓	N.A. ↓
F7. Square and Rectangular HSS and Box-Shaped Members 1. Yielding 2. Flange Local Buckling 3. Web Local Buckling	F3 ↓	App. F1 ↓	5.1 ↓	N.A. ↓	N.A. ↓
F8. Round HSS 1. Yielding 2. Local Buckling	F3 ↓	App. F1 ↓	5.1 ↓	N.A. ↓	N.A. ↓
F9. Tees and Double Angles Loaded in the Plane of Symmetry 1. Yielding 2. Lateral-Torsional Buckling 3. Flange Local Buckling of Tees	NEW ↓	F1.2c ↓ NEW	NEW ↓	N.A. ↓	N.A. ↓
F10. Single Angles 1. Yielding 2. Lateral-Torsional Buckling 3. Leg Local Buckling	NEW ↓	NEW ↓	N.A. ↓	5 5.1.2 5.1.3 5.2.2	5 5.1.2 5.1.3 5.2.2
F11. Rectangular Bars and Rounds 1. Yielding 2. Lateral-Torsional Buckling	F2 ↓	App. F1 ↓	5 5.1 NEW	N.A. ↓	N.A. ↓
F12. Unsymmetrical Shapes 1. Yielding 2. Lateral-Torsional Buckling 3. Local Buckling	NEW ↓	NEW ↓	N.A. ↓	N.A. ↓	N.A. ↓
F13. Proportions of Beams and Girders 1. Hole Reductions 2. Proportioning Limits for I-Shaped Members 3. Cover Plates 4. Built-Up Beams	B10 G1 B10 F6	B10 App. G1 B10 NEW	NEW ↓	NEW ↓	NEW ↓
<b>G. DESIGN OF MEMBERS FOR SHEAR</b>					
G1. General Provisions	F4	F2	NEW	NEW	NEW
G2. Members with Unstiffened or Stiffened Webs 1. Nominal Shear Strength 2. Transverse Stiffeners	F4 F5	F2, App. F2, App. G3 App. F2.3	5.2 N.A.	NEW NEW	NEW NEW
G3. Tension Field Action 1. Limits on the Use of Tension Field Action 2. Nominal Shear Strength with Tension Field Action 3. Transverse Stiffeners	G3 G3 G4	App. G3 App. G3 App. G4	N.A. N.A. N.A.	NEW NEW NEW	NEW 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	2000 LRFD
<b>H. DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION</b>					
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
<b>I. DESIGN OF COMPOSITE MEMBERS</b>					
II. General Provisions	I	NEW	NEW	NEW	NEW
1. Nominal Strength of Composite Sections	NEW	NEW	↓	↓	↓
1a. Plastic Stress Distribution Method	↓	↓	↓	↓	↓
1b. Strain-Compatibility 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 Columns					
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	2000 LRFD
<b>J. DESIGN OF CONNECTIONS</b>					
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. Special 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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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	↓	↓	↓
<b>K. DESIGN OF HSS AND BOX MEMBER CONNECTIONS</b>					
K1. Concentrated Forces on HSS	NEW	NEW	NEW	N.A.	N.A.
1. Definitions of Parameters	↓	↓	NEW	↓	↓
2. Limits of Applicability	↓	↓	NEW	↓	↓
3. Concentrated Force Distribution Transversely	↓	↓	8.1a	↓	↓
3a. Criterion for Round HSS	↓	↓	8.1b, 9.2	↓	↓
3b. Criteria for Rectangular HSS	↓	↓	8.2	↓	↓
4. Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis	↓	↓	8.2a	↓	↓
4a. Criterion for Round HSS	↓	↓	8.2b	↓	↓
4b. Criterion for Rectangular HSS	↓	↓	NEW	↓	↓
5. Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis	↓	↓	8.3	↓	↓
6. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate	↓	↓	9.4, 9.3.4	↓	↓
K2. HSS-to-HSS Truss Connections	NEW	NEW	9.4.1	↓	↓
1. Definitions of Parameters	↓	↓	9.4.2	↓	↓
2. Criteria for Round HSS	↓	↓	9.4.2a	↓	↓
2a. Limits of Applicability	↓	↓	9.4.2b	↓	↓
2b. Branches with Axial Loads in T-, Y- and Cross-Connections	↓	↓	9.4.2b	↓	↓
2c. Branches with Axial Loads in K-Connections	↓	↓	9.4.3	↓	↓
3. Criteria for Rectangular HSS	↓	↓	9.4.3a	↓	↓
3a. Limits of Applicability	↓	↓	9.4.3b	↓	↓
3b. Branches with Axial Loads in T-, Y- and Cross-Connections	NEW	NEW	9.4.3c	↓	↓
3c. Branches with Axial Loads in Gapped K-Connections	↓	↓	NEW	↓	↓
3d. Branches with Axial Loads in Overlapped K-Connections	↓	↓	9.2	↓	↓
3e. Welds to Branches	↓	↓	9.4	↓	↓
K3. HSS-to-HSS Moment Connections	NEW	NEW	9.4.1	↓	↓
1. Definitions of Parameters	↓	↓	9.4.2	↓	↓
2. Criteria for Round HSS	↓	↓	9.4.2a	↓	↓
2a. Limits of Applicability	↓	↓	9.4.2b	↓	↓
2b. Branches with Axial Loads in T-, Y- and Cross-Connections	↓	↓	NEW	↓	↓
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	↓	↓

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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					
<b>L. DESIGN FOR SERVICEABILITY</b>					
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
<b>M. FABRICATION, ERECTION AND QUALITY CONTROL</b>					
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			

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<b>Appendix 1. INELASTIC ANALYSIS AND DESIGN</b>					
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 Other 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
<b>Appendix 2. DESIGN FOR PONDING</b>					
2.1 Simplified Design for Ponding	K2	K2	NEW	NEW	NEW
2.2 Improved Design for Ponding	NEW	App. K2	NEW	NEW	NEW
<b>Appendix 3. DESIGN FOR FATIGUE</b>					
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
<b>Appendix 4. STRUCTURAL DESIGN FOR FIRE CONDITIONS</b>	NEW	NEW	NEW	NEW	NEW
<b>Appendix 5. EVALUATION OF EXISTING STRUCTURES</b>					
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. Serviceability Evaluation		N3.3			
5.4 Evaluation by Load Tests					
1. Determination of Load Rating by Testing	NEW	N4.1	NEW	NEW	NEW
2. Serviceability Evaluation	↓	N4.2	↓	↓	↓
5.5 Evaluation Report		N5			
<b>Appendix 6. STABILITY BRACING FOR COLUMNS AND BEAMS</b>	NEW	C3	NEW	NEW	NEW
<b>Appendix 7. DIRECT ANALYSIS METHOD</b>	NEW	NEW	NEW	NEW	NEW